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**FINAL**

**PALM DESERT GROUNDWATER  
REPLENISHMENT FEASIBILITY  
STUDY**

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**COACHELLA VALLEY WATER DISTRICT**

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**January 2017**

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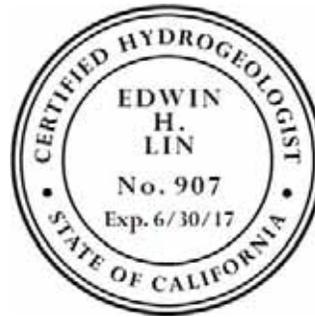
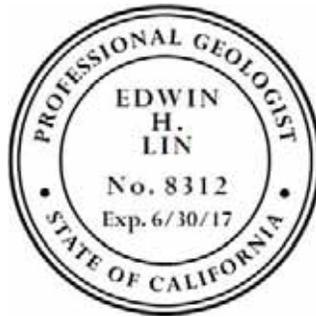
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## 1. INTRODUCTION

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The Coachella Valley Water District (CVWD) is evaluating the feasibility of recharging Colorado River water (CRW) at its Palm Desert property and in the Whitewater River Storm Channel (WWR-SC) south/southwest of the property. A groundwater replenishment facility would serve to help mitigate historical groundwater level declines and improve groundwater quality primarily within the West Whitewater River Subbasin Area of Benefit (WWR AOB) of the Coachella Valley Basin (Basin). Concepts to recharge CRW supplied via the CVWD Mid Valley Pipeline (MVP) include re-purposing existing percolation ponds associated with Water Reclamation Plant No. 10 (WRP 10) on the Palm Desert property and constructing detention structures in the WWR-SC. Preliminary evaluations by CVWD indicate that approximately 40,000 acre-feet per year (AFY) of CRW may be delivered via the MVP for possible recharge in this area.

In March 2016, CVWD contracted Todd Groundwater (TODD) to perform a groundwater replenishment feasibility study of the Palm Desert property and an adjacent two-mile reach of the WWR-SC. The scope of work for the study included (1) the review of existing hydrogeologic data to characterize local groundwater flow, water quality, and site infiltration potential, (2) concept development and cost comparison of alternative recharge facility designs, (3) an assessment of environmental permit and mitigation requirements for proposed concepts, and (4) construction of a local-scale groundwater flow and solute transport model to evaluate potential impacts of various recharge scenarios on groundwater flow, storage, and water quality. TODD subcontracted CWE Corp (CWE) to lead the development and comparative analysis of recharge facility concepts. Findings from this study will help CVWD estimate the recharge capacity of the project site and evaluate whether local benefits to groundwater storage and water quality substantially support CVWD's basin management objectives.

This report describes the local hydrogeologic conditions in the vicinity of the Palm Desert property and WWR-SC reach of interest, recharge facility concepts, environmental permit requirements, and groundwater modeling results. Key findings, knowledge gaps, and recommendations for future work are also presented.

### 1.1. LAND USE AND HYDROGEOLOGIC SETTING

#### 1.1.1. Palm Desert Property/Whitewater River Storm Channel and Land Use

The CVWD Palm Desert property is located at 75525 Hovley Ln East in Palm Desert, CA (**Figure 1**). Historically, this region of the Coachella Valley has supported irrigated agriculture and mixed urban land uses. Current land uses include residential housing, golf courses, commercial office/retail, industry, and public parks and schools.

WRP 10 is situated at the southern end of the CVWD Palm Desert property and currently consists of an activated sludge treatment plant (providing secondary treatment of wastewater), a tertiary wastewater treatment plant, recycled water delivery facilities, six

holding ponds, fifteen active and three former percolation ponds, and a lined back feed reservoir currently used to temporarily store CRW prior to delivery to CVWD irrigation customers.

The design capacity for secondary treatment is 18 million gallons per day (MGD). Design capacity for tertiary treatment used for recycled water is 15 MGD. In 2015, over 1,000 million gallons (MG) (approximately 3,400 acre-feet [AF]) of secondary treated effluent were discharged to onsite percolation ponds. CVWD is updating its Non-Potable Water Master Plan and plans to increase recycled water deliveries and eliminate the need for onsite discharge/disposal of secondary treated effluent. Future onsite storage requirements for recycled water will determine which of the percolation ponds (including possibly the back feed reservoir) may be re-purposed for CRW recharge. It is noted that one of the three former percolation ponds in the northwest portion of the Palm Desert property (maroon outlined basins on Figure 1) is paved and used for supplies and vehicles. The other two former ponds may be developed for additional storage but are also potentially available for CRW recharge. To provide a conservative estimation of recharge potential at the Palm Desert property, the three former percolation ponds were not included in the development and analysis of recharge concept alternatives and were not simulated as future recharge basins. Notwithstanding their exclusion in this study, recharge in the former percolation pond areas is possible and would provide additional recharge capacity at the CVWD Palm Desert property.

CVWD is also interested in evaluating the recharge potential of the WWR-SC south/southwest of the CVWD Palm Desert property between Portola Avenue and Fred Waring Drive (**Figure 1**). This reach is about 2 miles long and 300 feet wide. The channel banks are sloped and concrete-lined, while the channel bottom is unlined. Recharge/detention structure concepts must preserve the capacity of the channel to convey winter storms.

### **1.1.2. Hydrogeologic Setting**

Basin fill deposits in the vicinity of Palm Desert property are comprised of interbedded coarse-grained and finer-grained alluvial fan and stream wash deposits. Based on available well driller's logs, the thickness of alluvial deposits generally exceeds 1,200 feet. Groundwater occurs under unconfined to confined conditions and flows in a northwest-to-southeast direction. Rainfall on the valley floor averages about 4 inches per year. As such, aquifers in this area are fed primarily by subsurface inflows from the west-northwest, where recharge runoff along the northern flanks of the San Jacinto Mountains combines with CRW actively recharged at CVWD's Whitewater River Groundwater Replenishment Facility (WWR-GRF) northwest of Palm Springs.

Groundwater is pumped for public water supply and private (primarily landscape irrigation) uses. Recent (2015) groundwater level measurements indicate that the groundwater occurs approximately 200 feet below ground surface (feet-bgs) in the vicinity of the CVWD Palm Desert property. Municipal production wells in the area are generally screened in the lower portions of the aquifer with screen intervals ranging from 500 to 1,200 feet-bgs. Screen

intervals for private irrigation wells range from 200 to 1,200 feet-bgs. Recently, shallower screened municipal wells in this area have been taken offline as a result of nitrate detections exceeding the primary maximum contaminant level (MCL) of 45 milligrams per liter (mg/L) and replaced with wells screened in deeper aquifers. Potential nitrate sources include return flows from legacy agricultural irrigation, historical septic discharge, and more recently landscape (primarily turf) irrigation and deep percolation below WRP 10 secondary effluent ponds. A few deep production wells in the Study Area also exceed the primary MCL of 10 micrograms per liter (ug/L) for chromium-6, a naturally-occurring metal present in geologic sediments in the deeper portions of the groundwater basin.

As shown on Figure 1, there are six groundwater monitoring wells associated with WRP 10 (MW1 to MW6 on **Figure 1**). WRP 10 monitoring wells allow for assessment of groundwater level and water quality conditions in the upper 100 feet of saturated aquifer, where mixing between existing effluent discharge, proposed recharged CRW, and groundwater would occur. Shallow groundwater quality beyond the extent of WRP 10 monitoring wells in the Study Area is uncharacterized.

While WRP 10 has actively discharged secondary effluent since the mid-1970s, the percolation rate of unsaturated zone sediments at the CVWD Palm Desert property has historically not been well documented. To address this data gap, CVWD personnel conducted a series of short-duration infiltration tests (ranging from less than 1 day up to 9 days) at the existing ponds on the Palm Desert property from January to March 2016. These results provided a technical basis for estimating sustainable infiltration rates of CRW water at proposed recharge facilities.

## **1.2. SCOPE OF WORK**

In order to evaluate the feasibility of recharging CRW at the Palm Desert property and in the WWR-SC, Todd developed a scope of work that included four key tasks.

Task 1. The first task involved the review of hydrogeologic information provided by CVWD to characterize groundwater level, groundwater quality, and groundwater pumping conditions in the vicinity of the Palm Desert property. Additionally, CVWD's regional (basin-wide) MODFLOW groundwater flow model (Fogg, et al., 2000) was reviewed to assist with development of a local-scale groundwater flow and solute transport model (see Task 3).

To focus data collection and modeling efforts, a Study Area encompassing approximately 20 square miles (mi<sup>2</sup>) roughly centered on the Palm Desert property was delineated (**Figure 2**). Additionally, the list of groundwater quality constituents of concern selected for evaluation was narrowed down to five groundwater constituents of concern - TDS, chloride, sulfate, nitrate, and chromium-6. Fluoride concentrations in groundwater were also characterized, but were not evaluated in predictive model simulations, as anticipated groundwater concentration changes were determined to be insignificant. The Study Area is oriented in the general direction of groundwater flow (northwest-to-southeast) and includes 66 active and inactive CVWD production wells, 78 private production wells, and the six CVWD WRP 10

monitoring wells. The Study Area extends from the Palm Desert property approximately 2 miles northwest (upgradient) and 4 miles southeast (downgradient), crossing into the East Whitewater River Area of Benefit (EWR AOB). The central axis of the Basin forms the northeastern Study Area boundary, while shallow and outcropping bedrock forms the southern Study Area boundary.

Task 2. As-built construction drawings and topographic information provided by CVWD were reviewed to develop concept level drawings of recharge facilities at the CVWD Palm Desert property and in WWR-SC. Concepts for the Palm Desert property involve the removal of internal levees between existing percolation basins to increase the recharge area within the existing basin footprint. Concepts for the WWR-SC include use of sugar berms, flashboard dams, or inflatable rubber dams to partition the approximate 2-mile reach into a series of individual basins.

Sustainable recharge rates were estimated from short-term infiltration tests of the existing WRP 10 percolation ponds conducted by CVWD personnel and operational information for the CVWD Thomas E. Levy Groundwater Replenishment Facility (TEL-GRF) located in La Quinta. Maintenance requirements for various basin configurations were based on maintenance activities for the TEL-GRF. Construction cost estimates for proposed concepts were estimated from regional construction rates and projects recently designed by CWE and currently in construction.

Task 3. A preliminary assessment was completed of the environmental permits and potential mitigation measures required to re-purpose the existing percolation ponds associated with WRP 10 as CRW recharge basins and to construct detention/recharge structures in the WWR-SC. The assessment describes the regulatory implications for separating the proposed CRW recharge facilities on the Palm Desert property from the WRP 10 operation and environmental permit and mitigation requirements for proposed improvements in the WWR-SC.

Task 4. A local groundwater flow and solute transport model of the 20-mi<sup>2</sup> Study Area was constructed to predict groundwater flow and storage changes and water quality impacts from future recharge of CRW. Data and findings from Task 1 were used to develop model boundaries and initial groundwater level and water quality conditions and to set appropriate groundwater level and water quality model calibration targets. Recharge acreages and estimated long-term infiltration rates from concept recharge basins and storm channel facilities developed in Task 2 were used to develop and simulate five potential future recharge scenarios. Results were evaluated to assess benefits to groundwater levels and storage, and future water quality changes for five constituents of concern.

### **1.3. REPORT ORGANIZATION**

The methods and results of the technical work are described in the main body of the report.

**Section 2** documents the local groundwater level and groundwater quality conditions within the Study Area.

**Section 3** summarizes the key findings from the conceptual recharge facility design analysis conducted by CWE on the CVWD Palm Desert property and WWR-SC (the complete CWE technical memorandum is provided in **Appendix A**). **Section 4** describes the regulatory setting, permit requirements, and general approach to achieving compliance for proposed recharge concepts at the CVWD Palm Desert property and in the WWR-SC.

**Section 5** describes the modeling approach, data inputs, and predictive recharge simulations and results.

Final conclusions and recommendations are presented in **Section 6**.

## 2. LOCAL HYDROGEOLOGIC CONDITIONS

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Existing hydrogeologic data provided by CVWD were evaluated to characterize groundwater level and groundwater quality conditions and site infiltration characteristics within the 20-mi<sup>2</sup> Study Area.

### 2.1. GROUNDWATER LEVELS

Groundwater level data were available for 125 of the 150 wells in the Study Area. **Figure 3** shows groundwater elevation contour maps for four time periods covering the past 30 years (1985, 1995, 2005, and 2015). Using GIS Spatial Analyst, contours were used to create interpolated groundwater elevation surfaces and calculate groundwater level changes over time. **Figure 4** shows groundwater level change maps for the three successive 10-year periods (1985 to 1995, 1995 to 2005, and 2005 to 2015) and over the 30-year period from 1985 to 2015. **Figure 5** shows water level hydrographs for selected wells across the Study Area. Water level data for two or more adjacent wells were combined in some hydrographs to illustrate long-term water level trends and identify vertical hydraulic gradients in areas where shallow and deep monitoring data are available.

Together, the groundwater level contour and change maps and hydrographs show that groundwater generally flows across the Study Area in a northwest-to-southeast direction. South of Highway 111, storm runoff in the San Jacinto Mountains enters the Basin as subsurface inflow, resulting in localized groundwater to flow in a northeasterly direction. Water level mounding from discharge of secondary effluent at WRP 10 is indicated by local contours deflecting slightly to the east (e.g., 10-foot contour in 1995 and -20 foot contour in 2005 on Figure 3).

In 1985, groundwater levels ranged from 50 feet msl in the northwest to -20 feet msl in the southeast. However, as a result of continued overpumping, groundwater levels have declined at an average rate of between 1 and 3 feet per year over the past 30 years. In 2015, groundwater levels ranged from 0 feet msl in the northwest to -120 feet msl in the southeast. These levels represent water level declines of 40 to 100 feet across the Study Area over the past 30 years. As shown on Figure 4, declines were most pronounced from 1995 to 2005, particularly in the southeastern portion of the Study Area. Water levels have stabilized over the past 10 years, with average declines of about 1 foot per year. This trend is illustrated further on Figure 5 by the relatively stable trends since 2005 for several wells.

As indicated by the hydrograph for wells 15F1 (WRP 10 MW2) and 15H1 (upper right portion of Figure 5), a downward vertical hydraulic gradient exists downgradient of the WRP 10 percolation ponds. In contrast, based on the hydrographs for wells 16A3 (WRP 10 MW4) and 16A4 (middle left portion of Figure 5) and for wells 15M1 (WRP 10 MW1) and 16H1, no measurable vertical hydraulic gradient exists upgradient of WRP 10 percolation ponds at these locations.

As indicated by the ground surface and groundwater elevations on the Figure 5 hydrograph charts, the depth to water in the vicinity of the Palm Desert property and WWR-SC ranges from 200 to 210 feet-bgs. Depth to water increases slightly to the north and west, where it ranges from 220 to 260 feet-bgs. The relatively thick vadose zone beneath the Study Area provides a significant volume of available storage for accommodating anticipated water level mounding associated with proposed CRW recharge.

## 2.2. GROUNDWATER QUALITY

Groundwater quality data for six shallow WRP 10 monitoring wells and deeper CVWD and private production wells were used to characterize groundwater quality conditions across the Study Area for TDS, chloride, sulfate, nitrate, chromium-6, and fluoride.

**Figure 6** shows the locations of the 57 wells with water quality and well construction (screen interval) information. As noted on the figure, wells are labeled according to their abbreviated California State Well Number (e.g., 8N2 = 5N/6E-8N2) and, if available, also by their CVWD Well ID (e.g., 5623-1). To facilitate groundwater quality characterization, six water quality zones within the Study Area were delineated (Zones A through F) corresponding to six vertical cross sections (A-A' through F-F'). **Figures 7 through 12** show well concentrations for TDS, chloride, sulfate, nitrate, chromium-6, and fluoride grouped by water quality zone. From left to right, each cross section is oriented from southwest to northeast. The well screens on each cross section are color-coded according to the most recent concentration as reported from 2005 to 2015. The most recent concentration for each well was selected for solute transport model simulations to account for the generally increasing concentration trends observed for TDS, chloride, sulfate, and nitrate in many of the deeper production wells. Time-concentration plots for wells grouped by water quality zone are provided in **Appendix B**.

It is noted that constituent concentrations in the six WRP 10 monitoring wells vary considerably more than in deeper production wells, indicated that shallow groundwater quality is influenced by localized sources of recharge in addition to secondary effluent discharged to the WRP 10 percolation ponds. Concentrations for WRP 10 MW1 through MW6 shown on cross sections C-C and D-D are generally within the range of observed historical concentrations. However, time-concentration plots concentrations were examined to identify representative concentrations that also maximized model stability for future water quality simulations.

Also shown on each of the cross sections are the elevations of the four model layers selected for the local groundwater flow and solute transport model developed for this study (see Section 5.2.1 for further discussion). The model layers provide a useful reference for describing observed groundwater quality variability with depth.

In addition to the cross sections, plan-view maps were developed showing the same well concentrations (**Figures 13 through 18**). The water quality data shown on Figures 7 through 18 were used to populate initial model layer concentrations for model simulations, which is

described in further detail in Section 5.4.3. Additional observations of groundwater quality conditions in the Study Area by constituent are described below.

### **2.2.1. TDS**

Figure 13 and the upper left cross sections on Figures 7 through 12 show the groundwater TDS concentrations in available wells in the Study Area. The figures show that TDS concentrations are generally higher in the shallow aquifer (Model Layer 1) than in the deeper production wells (500 to 750 mg/L in WRP 10 monitoring wells compared to 200 to 300 mg/L in deeper Model Layer 3 and 4 production wells). TDS concentrations in Model Layer 3 gradually increase from northwest to southeast (downgradient) and towards the southern margins of the Basin. In contrast, TDS concentrations in Model Layer 4 gradually decrease from northwest to southeast (downgradient). TDS is a non-health based limit regulated by a secondary MCL that is a recommended range of 500 to 1,000 mg/L to avoid aesthetic concerns by customers, and a short-term range of 1,500 mg/L. While there are no wells screened only in Model Layer 2, some wells are screened across Model Layers 1 and 2 or Model Layers 2 and 3. Higher TDS concentrations in shallow groundwater are likely associated with legacy agricultural return flows and more recent urban return flows (e.g., outdoor irrigation return).

Several wells are screened across Model Layers 3 and 4 and have slightly higher TDS concentrations than production wells screened only in Layer 4 in the Study Area. Time-concentration plots in Appendix B reveal gradually increasing TDS trends for most wells in all six water quality zone over the past 10 years. These trends are indicative of the downward migration of higher-TDS groundwater from shallow aquifers to deeper aquifers induced by groundwater pumping.

### **2.2.2. Chloride**

Figure 14 and the upper middle cross sections on Figures 7 through 12 show the groundwater chloride concentrations for available wells in the Study Area. Similar to TDS, the figures show that chloride concentrations in the shallow aquifer (Model Layer 1) are generally much higher than in the deeper production wells (70 to 85 mg/L in WRP 10 monitoring wells compared to on average less than 10 to 25 mg/L in deeper Model Layer 3 and 4 production wells). Chloride concentrations north of Highway 11 gradually increase from northwest to southeast in Model Layer 3 but are more consistent in Model Layer 4 ranging from 11 to 15 mg/L. Chloride concentrations increase significantly towards the southern margins of the Basin. The secondary drinking water MCL for chloride is a recommended range of 250 to 500 mg/L to avoid aesthetic concerns by customers, and a short-term range of 600 mg/L. Similar to TDS, higher chloride concentrations in shallow groundwater are likely associated with legacy agricultural return flows and more recent urban return flows (e.g., landscape irrigation).

Chloride concentrations are higher and have been gradually increasing in production wells screened across Model Layers 3 and 4 as a result of the downward migration of higher-chloride groundwater from shallow aquifers to deeper aquifers. Time-concentration plots in

Appendix B reveal stable to gradually increasing chloride trends for wells in each of the water quality zones over the past 10 years.

### **2.2.3. Sulfate**

Figure 15 and the upper right cross sections on Figures 7 through 12 show the groundwater sulfate concentrations in CVWD wells in the Study Area. Similar to TDS and chloride, the figures show that sulfate concentrations in the shallow aquifer (Model Layer 1) are higher than in the deeper production wells (between 50 and 200 mg/L in WRP 10 monitoring wells compared to about 20 to 40 mg/L in deeper wells in Model Layers 3 and 4). Similar to chloride, sulfate concentrations increase gradually from northwest to southeast in Model Layer 3, are relatively consistent in Model Layer 4, and increase significantly towards the southern margins of the Basin. The secondary drinking water MCL for sulfate is a recommended range of 250 to 500 mg/L to avoid aesthetic concerns by customers, and a short-term range of 600 mg/L. Higher sulfate concentrations in shallow groundwater are likely associated with legacy agricultural return flows and more recent urban return flows.

Sulfate concentrations are higher and have been gradually increasing in production wells screened across Model Layers 3 and 4 as a result of the downward migration of higher-sulfate groundwater from shallow aquifers to deeper aquifers. Similar to chloride, time-concentration plots in Appendix B reveal stable to gradually increasing sulfate trends for wells in all water quality zones over the past 10 years.

### **2.2.4. Nitrate**

Figure 16 and the lower left cross sections on Figures 7 through 12 show the groundwater nitrate-NO<sub>3</sub> concentrations for available wells in the Study Area. Similar to TDS, chloride, and sulfate, the figures show that the nitrate-NO<sub>3</sub> concentration in the shallow aquifer ranges from 40 to 70 mg/L in WRP 10 monitoring wells, and is higher in the shallow zone than in deeper aquifer zones. For reference, the average nitrate concentration of WRP 10 secondary effluent from 2011 to 2015 was 62 mg/L. Nitrate concentrations for deeper wells in Model Layers 3 and 4 generally increase from northwest to southeast and along the southern margins of the Basins with concentrations exceeding the primary drinking water MCL for nitrate-NO<sub>3</sub> of 45 mg/L in thirteen wells. Higher nitrate concentrations in shallow groundwater are likely associated with anthropogenic return flows, including legacy agricultural return flows over areas that were once nitrogen-fixing mesquite forests.

Nitrate concentrations in almost all deeper production wells in the Study Area show an increasing trend over the past 10 years. This trend has resulted in the replacement of several existing shallower screened domestic production wells with deeper wells.

### **2.2.5. Chromium-6**

Figure 17 and the lower middle cross sections on Figures 7 through 12 show the groundwater chromium-6 concentrations in CVWD wells in the Study Area.

Chromium-6 in the Basin is naturally-occurring and has been dissolved from chromium-bearing alluvial sediments over geologic time. The primary drinking water MCL for chromium-6 is 10 ug/L. Figure 17 and the lower middle cross sections on Figures 7 through 12 show the groundwater chromium-6 concentrations for available wells in the Study Area. It is noted that chromium-6 has been analyzed only in the three newer WRP 10 monitoring wells (MW4, MW5, and MW6), with data available from 2014 to 2015. Unlike other constituents, chromium-6 concentrations in the shallow and intermediate-deep aquifers (Model Layers 1 through 3) are similar or lower in comparison with deeper production wells. The distribution of chromium-6 concentration with depth demonstrates that older groundwater (with longer residence times) has generally higher chromium-6 concentrations in the Study Area. Groundwater chromium-6 concentrations in Layers 3 and 4 increase from southwest to northeast, generally indicating that more chromium-bearing alluvial sediments are located in the northeast portion of the Study Area. Concentrations of production wells along the northeast margins of the Study Area are near or exceed the primary MCL of 10 ug/L. Time-concentration plots in Appendix B reveal stable chromium-6 trends for all wells in each of the water quality zones over the past 10 years.

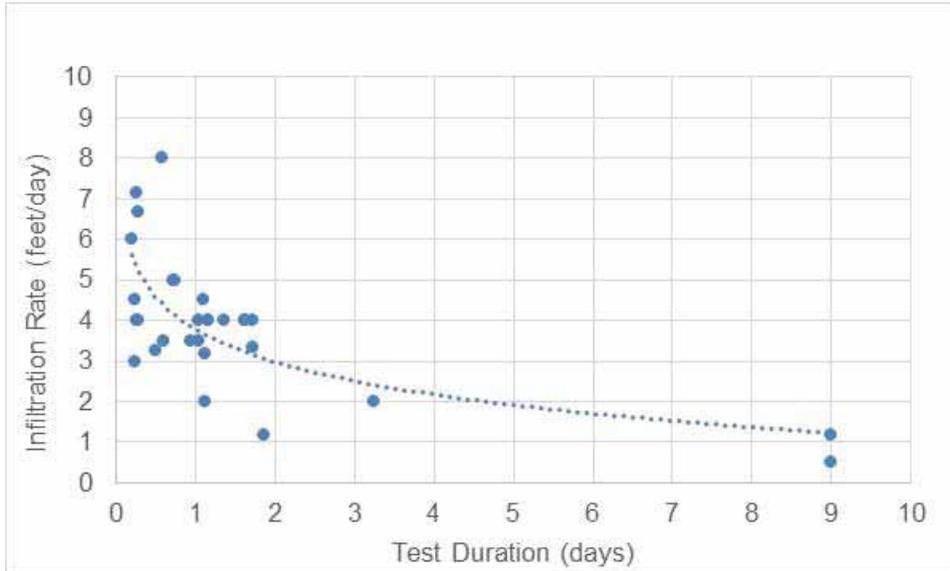
#### **2.2.6. Fluoride**

Figure 18 and the lower right cross sections on Figures 7 through 12 show the groundwater fluoride concentrations in CVWD wells in the Study Area. The figures that fluoride concentrations in the shallow WRP 10 monitoring wells and deeper production wells are low (less than 1 mg/L). Given that California's primary drinking water MCL for fluoride is 2 mg/L, and local groundwater, historical WRP 10 secondary effluent discharge, and CRW delivered via the MVP are all below 1 mg/L, proposed CRW recharge at the CVWD Palm Desert property and in WWR-SC are not anticipated to significantly impact groundwater quality. For this reason, fluoride was not simulated in solute transport modeling.

### **2.3. ESTIMATED SITE INFILTRATION RATES**

Short-term infiltration tests were conducted by CVWD personnel at the existing WRP 10 percolation ponds from January through March of 2016. Tests were conducted over a range of less than 1 day up to 9 days using secondary effluent. Final calculated infiltration tests are shown in the chart below.

### Short-Term Infiltration Test Results, WRP 10 Percolation Ponds



As shown in the chart, most of the tests were conducted for a duration of less than 2 days, with infiltration rates exceeding 6-7 feet/day over that period. Results for the two tests conducted over 9 days indicate that infiltration rates decline significantly over time (to around 1 feet/day).

Percolation basins were cleaned as part of routine annual maintenance in early January prior to infiltration testing. Activities include the clearing of vegetation and light scarification of the pond bottoms. Higher long-term infiltration rates than those observed from the two 9-day tests can likely be achieved by removing the upper approximately 5 feet of material and fine-grained sediment that has accumulated over the 40 years of discharge to the ponds. Operational performance at the TEL-GRF indicate long-term infiltration rates of 1.75 feet/day at that facility. While higher infiltration rates may be achievable, an infiltration rate of 1.75 feet/day is assumed for recharge basins at the Palm Desert property and in the WWR-SC for this study.

### **3. CONCEPTUAL RECHARGE FACILITY DESIGNS**

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This section summarizes the key findings from the conceptual recharge facility design analysis conducted by CWE for the CVWD Palm Desert property and WWR-SC. The complete CWE memorandum titled “Coachella Valley Water District Palm Desert Groundwater Replenishment Feasibility Study – Concept Alternative Analysis” is provided in **Appendix A**.

#### **3.1. APPROACH TO CONCEPT DEVELOPMENT**

For the existing basin system on the CVWD Palm Desert property, two design concepts were developed and evaluated. **Figures 19 and 20** shows the recharge pond layouts and structural features of the two concepts (Concepts 1 and 2). Both concept involve the removal of existing berms to increase the volume and infiltration surface of the recharge facility. The difference between the two proposed concepts comes from aligning the berms in either an east/west or north/south direction, which influences the total area used. Concept 2 includes the use of the existing back feed basin as a dual CRW storage and recharge basin and thus has a larger recharge area than Concept 1.

Three design concepts were developed for the WWR-SC system. Each concept differs in the selection of the partition structures, the structure height, and the basin lengths (see Appendix A for WWR-SC recharge concept drawings). Determination of the basin length is dependent on the height of the partition structure. Sugar berms (Concept 1), flashboards (Concept 2), and rubber dams (Concept 3) were evaluated with varying heights analyzed at 2 feet, 4 feet, and 6 feet. The basin lengths corresponding to the three heights are 800, 1,600, and 2,400 feet, respectively.

An optional pipe system has been included to divert MVP flows into the WWR-SC basins. To ensure flow is received by the upstream end of the river basins, an optional pump station with a pressure head ranging from 7 to 10 feet is included for the pipeline system. It may be possible to add more pumps to the existing pump station to increase head for a lower cost than provided in the estimate. In addition, four outlets with isolation valves will be stationed along the channel to provide control for system operations and maintenance. Due to the concrete lining in the channel, two alignment alternatives have been considered for the pipeline system. The first alternative will have the pipeline located in the north bank of the channel. The second alternative will have a portion of the pipeline divert into the middle of the channel at station 64+00 (see Appendix A figures) and continue to Portola Ave. This pipeline will be buried 20 feet below the channel invert.

#### **3.2. FACTORS FOR ANALYSIS OF CONCEPTS**

In order to improve the existing recharge basins on the Palm Desert property and propose new basins in the WWR-SC, several key factors will have to be satisfied to influence the amount of water that can be recharged. Improvement in the existing basin system requires reconfiguration and expansion of the infiltration basins to increase the infiltration capacity for imported water. Addition of a new WWR-SC basin system will increase the amount of

CRW water that can be infiltrated and some of the design alternatives may also increase the system capacity to capture stormwater. Factors that influence this goal include percolation rate, existing groundwater levels and available vadose zone storage capacity, future groundwater levels, site geology, and the size and volume of the infiltration basins. Percolation rates are based on native materials and geology. Local groundwater levels are influenced by rainfall and local recharge, and geology is determined by the location of the project. While all the factors stated above play a significant role in groundwater recharge, maximizing the amount of infiltration in the recharge basin systems relies heavily on the surface area of the basins and maintenance of the infiltration basins once the basins are constructed.

### 3.3. ESTIMATED RECHARGE CAPACITIES AND COST ANALYSIS

**Table 1** provides a cost summary for the each recharge facility concept. Included are the initial construction cost and annual operation and maintenance (O&M) costs. The table also shows the estimated recharge capacities for the each concept based on an assumed infiltration rate of 1.75 feet/day and operation of 304 days per year (10 months), based on two maintenance cycles each year. The annual volume of water replenished for the basins on the Palm Desert property is approximately 16,000 AFY for Concept 1 and 20,000 AFY for Concept 2. The volume of water replenished per year in the Whitewater River system is estimated to range between 44,000 and 54,000 AFY. The WWR-SC recharge capacities would accommodate the approximately 40,000 AFY of CRW that could be delivered by the MVP.

**Table 1  
Recharge Capacity and Cost Estimate Summary**

CVWD Palm Desert Property Recharge Facility			
Item	Concept 1		Concept 2
<b>Initial Construction</b>	\$1,597,170		\$1,929,130
<b>Annual O&amp;M</b>	\$175,481 – \$588,181		\$211,982 – \$724,265
<b>Recharge Capacity</b>	16,000 AFY		20,000 AFY
Whitewater River Storm Channel Recharge Facility			
Item	Sugar Berm	Flashboard Dam	Rubber Dam
<b>Initial Construction</b>	\$11,480 – \$17,870	\$2,812,040 – \$4,884,180	\$4,632,870 – \$7,746,910
<b>Annual O&amp;M<sup>1</sup></b>	\$758,019 - \$783,624	\$682,412 - \$711,569	\$660,754
<b>Recharge Capacity<sup>2</sup></b>	44,383 AFY	41,944 AFY	54,300 AFY

Notes:

1 – includes annual channel maintenance and vegetation and sediment removal; for sugar berms, berm replacement is also included at 50% initial berm construction cost four times per year.

2 – MVP CRW delivery capacity may be limited to approximately 40,000 AFY

For the two basin concepts on the CVWD Palm Desert property, three O&M strategies were evaluated. The first O&M strategy assumes semi-annual basin cleanings with silt pushed onto the basin sides (as is currently performed for the WRP 10 percolation basins). The second maintenance strategy similarly involves pushing silt onto the basin sides but assumes that basin cleanings occur on a quarterly basis. The last maintenance strategy involves semi-annual basin cleaning with offsite disposal of silts. These parameters were used to generate the range of operation and maintenance costs for the CVWD Palm Desert property recharge basins in Table 1.

The recharge system in the WWR-SC will operate differently than the recharge basins on the CVWD Palm Desert property. Stormwater runoff events must be conveyed through the system, and each concept reacts differently to the flows. Sugar berms will be washed out by large flows. Depending on the height of flashboards and the channel capacity, they may need to be removed to maintain channel capacity. The time to remove and reinstall the flashboards varies by height of the system. Rubber dams are able to operate during storms and have the greatest ability to capture stormwater runoff. Stormwater will replace MVP water if it is captured, maintaining the capture, but reducing costs for water used to recharge the system. River basins will be out of commission during reconstruction of sugar berms or removal and replacement of the flashboards. The system will flush itself during the storm events, reducing the river system maintenance to one cycle per year.

As shown in Table 1, initial construction costs are much lower for the sugar berm concepts (\$11,480 – \$17,870) compared to the flashboard and rubber dam concepts (\$2.8 million [M] to \$7.7 M). However, annual operations for the sugar dams are slightly higher because berms will need to be re-constructed following measurable storm events. O&M costs for the sugar berms assume 50% berm re-construction four times a year.

In order to compare project costs, the total project costs with construction and O&M were evaluated. **Tables 2a and 2b** summarize the costs for the recharge facility concepts on the CVWD Palm Desert property and in the WWR-SC, respectively. Total project costs are calculated at present value for equal comparison based on a 3 percent discount rate and a 50-year lifecycle. Total project costs are divided by the total volume of CRW recharged over 50 years for each concept to calculate a unit project cost per AF of CRW.

As shown in Table 2a, Concept 2 (the larger recharge area that utilizes the back feed reservoir as a dual-recharge detention basin) is slightly more cost-effective than Concept 1. Unit costs per acre foot of CRW recharged were slightly lower for Concept 2 for each maintenance strategy.

As shown in Table 2b, total project costs for the WWR-SC concepts include the construction cost for extending the MVP to Portola Ave using two alignment alternatives are included (along the northern bank of the WWR-SC [bank alignment] and buried beneath the WWR-SC between Cook St and Portola Ave. [channel alignment]). As shown in Table 2b, the sugar berm concept is the most cost effective option (when potential stormwater capture benefits are not considered). Taller berm heights are also more cost-effective for all three concepts.

**Table 2a**  
**CVWD Palm Desert Property Recharge Facility Cost Estimate Summary**

	Concept 1	Concept 2
Basin Construction Costs	\$1,597,170	\$1,929,130
<b>O&amp;M Strategy</b>	<b>O&amp;M Costs (50-yr life cycle / 3% discount)</b>	
Semi-Annual / Silt Onsite	\$4,515,085	\$5,454,247
Quarterly / Silt Onsite	\$9,030,169	\$10,803,465
Semi-Annual / Silt Offsite	\$15,133,758	\$18,635,168
<b>O&amp;M Strategy</b>	<b>Cost / AF of CRW Recharge<sup>1</sup></b>	
Semi-Annual / Silt Onsite	\$7.64	\$7.38
Quarterly / Silt Onsite	\$13.28	\$12.73
Semi-Annual / Silt Offsite	\$20.91	\$20.56

Notes:

1 - Based on total recharge of 800,000 AF for Concept 1 and 1,000,000 AF for Concept 2

Concept 1 does not include backfeed basin conversion

Silt onsite = accumulated silt pushed onto basin sides during basin cleaning

Silt offsite = accumulated silt removed and disposed offsite during basin cleaning

**Table 2b**  
**Whitewater River Storm Channel Recharge Facility Cost Estimate Summary**

	Sugar Berm		Flashboard Dam		Rubber Dam	
MVP Pipeline Extension Bank Alignment	\$8,030,486		\$8,030,486		\$8,030,486	
MVP Pipeline Extension Channel Alignment		\$9,496,248		\$9,496,248		\$9,496,248
	<b>Initial Basin Construction Costs</b>					
Berm Height 2'	\$11,480		\$4,884,180		\$7,746,910	
Berm Height 4'	\$14,560		\$3,034,230		\$5,541,820	
Berm Height 6'	\$17,870		\$2,812,040		\$4,632,870	
	<b>O&amp;M Costs<sup>1</sup> (50-yr life cycle / 3% discount)</b>					
Berm Height 2'	\$20,162,461		\$18,308,502		\$17,001,044	
Berm Height 4'	\$19,683,630		\$17,820,615		\$17,001,044	
Berm Height 6'	\$18,111,624		\$16,754,873		\$15,558,364	
	<b>Total Project Costs per AF of Colorado River Water Recharged<sup>2</sup></b>					
Berm Height 2'	\$14.10	\$14.84	\$15.61	\$16.34	\$16.39	\$17.12
Berm Height 4'	\$13.86	\$14.60	\$14.44	\$15.18	\$15.29	\$16.02
Berm Height 6'	\$13.08	\$13.81	\$13.80	\$14.53	\$14.11	\$14.84

Notes:

1 - Includes channel maintenance and vegetation removal costs; for sugar berms, also includes berm re-construction following storm events assumed 4 times per year at 50 percent of initial berm construction

2 - Based on total recharge of 2,000,000 AF; includes MVP pipeline extension construction, basin initial year construction, and O&M costs. Costs to purchase and deliver CRW to the proposed recharge and conveyance facilities are not included.

Initial construction costs for flashboards and rubber dams decrease with increasing berm height, due to the reduction in the number of concrete pads for taller berms. This relationship does not exist for the sugar berms, since the height governs the overall dimensional width of each berm. It is recognized that the ability to capture stormwater for the WWR-SC concepts represents a benefit that adds potential economic value to the project. However, because of the flashy nature of runoff in the WWR-SC, prediction of storm events and potential stormflow capture is difficult. Therefore, for the purposes of the concept cost comparison, storm capture benefits are not included in Table 2b. While construction costs for flashboard and rubber dam concepts are higher, it is noted that potential stormwater capture benefits afforded by the flashboard and rubber dam concepts are not possible with the sugar berm concept.

The CWE Technical Memorandum provided in Appendix A contains a full analysis of the cost estimates, including potential stormwater capture benefits.

In summary, based on an analysis of construction and O&M costs, Concept 2 for the recharge facility at the Palm Desert property (larger recharge area including the back feed reservoir) and the sugar berm concept for the WWR-SC basins are recommended. In addition to construction and O&M costs, potential mitigation measures and the acceptable impacts in the channel have become a primary factor in the selection process for the WWR-SC recharge system. Based on preliminary discussions between CVWD and the U.S. Army Corps of Engineers (USACE), it is understood that the sugar berms may require little to no mitigation and may be quicker to implement. Accordingly, the sugar berms may be the preferred choice for a WWR-SC recharge facility from a project mitigation perspective.

## **4. ENVIRONMENTAL PERMIT AND MITIGATION REQUIREMENTS FOR CRW RECHARGE**

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Proposed recharge facilities may require coordination with federal and state regulatory agencies to ensure that appropriate environmental permits are obtained where required, and to ensure Endangered Species Act and Clean Water Act environmental compliance is satisfied for project implementation. A preliminary assessment was completed and analyzed the environmental notifications and permits required to re-purpose the existing WRP 10 percolation ponds as CRW recharge basins and construct proposed detention/recharge structures in the WWR-SC. This assessment describes the general approach to creating CRW recharge facilities on the Palm Desert property distinct from the existing WRP 10 facility. Additionally, the assessment identifies the regulatory agencies that have approval authority required for the proposed improvements.

### **4.1. Regulatory Approach for Re-Purposing WRP 10 Ponds as Recharge Basins**

In order to re-purpose the existing WRP 10 effluent ponds as CRW recharge basins, CVWD will need to submit a Notice of Intent (NOI) to the Regional Water Quality Control Board - Colorado River Basin Region (Regional Board). The NOI shall describe the existing and future facilities used to provide recycled water, and the reduced demand for existing ponds at WRP 10 used for land disposal. Accordingly, CVWD will need to coordinate with the Regional Board to update Waste Discharge Requirements (WDRs) for WRP 10 to remove from the facility description percolation ponds no longer needed for land disposal of secondary treated effluent.

The proposed recharge project requires an environmental review and analysis process to evaluate and determine potential environmental effects. While CRW recharge is expected to improve notable constituents of concern, including nitrate and chromium-6, groundwater salinity increases are also anticipated. The CVWD Board Resolution to adopt the Supplemental Programmatic Environmental Impact Report (EIR) for the 2010 Water Management Plan Update (CVWD, 2012), acknowledged in the Statement of Overriding Considerations that: 1) CRW is a significant contributor of salt in the Basin, 2) the rate at which groundwater salinity increases will be higher with CRW recharge and cannot be fully mitigated to a less than significant level; and 3) groundwater salinity increases are most significant near CRW recharge facilities. These acknowledgements, along with the mitigation measures identified in the EIR, are applicable to all of the proposed recharge concepts presented herein and address the main impact of CRW recharge on groundwater quality. The extent and timing of salinity increases from CRW recharge in proposed recharge facilities have been evaluated with groundwater modeling as documented in this report.

## **4.2. Approach to Regulatory Compliance for Project within WWR-SC**

A summary of applicable environmental laws and required regulatory agency coordination for constructing a recharge facility within the WWR-SC is presented below. Regulatory agencies that have authority over proposed improvements in the WWR-SC include the U.S. Fish and Wildlife Service (USFWS), California Department of Fish and Wildlife (CDFW), USACE, and the Regional Board.

### **4.2.1. Endangered Species Act (USFWS and CDFW)**

Potential impacts to sensitive habitat and/or sensitive species along the proposed project site may require mitigation. The proposed project is not located in one of the 26 Conservation Areas associated with the Coachella Valley Multiple Species Habitat Conservation Plan (MSHCP) that protects 27 sensitive and/or listed species; therefore, formal coordination with the Coachella Valley Conservation Commission (CVCC) is not required. The CVCC is a joint powers authority that developed and administers the MSHCP that was implemented in 2008. The MSHCP provides comprehensive compliance with federal and state endangered species laws. The intent of the MSHCP is to transfer the authority of state and federal wildlife agencies to local government, consolidate Endangered Species Acts for land under one permit, and, in turn, provide for more efficient environmental processing of infrastructure projects, including flood control and water projects, in the Coachella Valley. The MSHCP is recognized by the USFWS and CDFW who officially granted federal and state permits in 2008.

As part of the MSHCP, CVWD has already committed resources to establish permanent habitat for natural communities and endangered species to replace habitat that is periodically altered by maintenance activities in flood control facilities, including the WWR-SC. The MSHCP acknowledges the responsibility of CVWD to ensure the proper function of the WWR-SC as a flood control facility. Permitted activities include routine maintenance by CVWD of facilities (i.e. drop structures, low –flow channel) and controlling vegetation within the channel. Review of the MSHCP indicates there is no sensitive habitat or species located within the 2-mile reach of the WWR-SC between Portola Avenue and Fred Waring Drive. Given the lack of sensitive species or habitat within this portion of the WWR-SC, a proposed recharge project in the WWR-SC is not expected to require mitigation.

#### **4.2.2. Clean Water Act Section 404 (USACE) and Section 401 (Regional Board)**

##### Section 404 Permit for Dredge and Fill Discharges (USACE)

CVWD will be required to file for a CWA Section 404 permit for any dredge or fill discharges into waters of the U.S. associated with the proposed project. Given there is no sensitive habitat delineated in this reach of the channel, mitigation would likely not be required by USACE for that purpose.

Based on preliminary communications with USACE staff and given that anticipated impacts from proposed recharge concepts in the WWR-SC will be limited to low-flow conditions with no sensitive habitat in the reach of interest, it is anticipated that any required mitigation by USACE will not significantly increase the cost or delay the schedule of constructing and operating a CRW recharge facility within this portion of the WWR-SC.

##### Section 401 State Water Quality Certification (RWQCB)

Section 401(a) of the Clean Water Act requires that any applicant for a federal permit to construct or operate facilities that result in discharge to a water body must provide the licensing or permitting agency a state certification that any such discharge will comply with state water quality standards (i.e., Basin Plan Objectives). This state certification is referred to as the Section 401 State Water Quality Certification and is typically issued in connection with USACE CWA Section 404 permits for dredge and fill discharges.

Proposed recharge facilities in the WWR-SC fall under the category of projects involving discharges of dredged or fill materials to federal waters.

#### **4.2.3. Streambed Alteration Agreement (CDFW)**

California Fish and Game Code Section 1602 requires an entity to notify CDFW prior to commencing any activity that may do one or more of the following:

- Substantially divert or obstruct the natural flow of any river, stream or lake;
- Substantially change or use any material from the bed, channel or bank of any river, stream, or lake; or
- Deposit debris, waste or other materials that could pass into any river, stream or lake.

Water bodies include those that are episodic ( dry for periods of time), which is characteristic of the WWR-SC. Potential impacts from channel re-contouring for the preferred recharge facility concept (rubber dams or sugar berms) would be limited to low-flow conditions. Given that there is no sensitive habitat within the WWR-SC reach of interest, it is anticipated that coordination with CDFW to procure a streambed alteration agreement for the proposed project will not significantly affect the feasibility of recharging CRW within this portion of the WWR-SC.

### **4.3. SUMMARY OF ENVIRONMENTAL PERMITTING AND MITIGATION REQUIREMENTS**

A preliminary assessment of applicable environmental laws, regulatory agencies, permit requirements, and potential measures to mitigate loss of sensitive habitat and channel functionality for WWR-SC recharge concepts was completed. Findings indicate that requirements should not significantly increase the cost or delay the schedule of constructing and operating recharge facilities on the Palm Desert WRP 10 property or within a portion of the WWR-SC. Additional coordination with respective regulatory agencies along with submittal of CWA permit applications are needed to better understand any required project-specific mitigation measures and associated costs and/or schedule implications.

## 5. PALM DESERT GROUNDWATER FLOW AND SOLUTE TRANSPORT MODEL

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### 5.1. INTRODUCTION

A groundwater flow and solute transport model was used to evaluate the feasibility and predict the effectiveness of recharging CRW via recharge ponds at the Palm Desert property and via the WWR-SC. The numerical model was constructed on the basis of available aquifer property, groundwater flow, and groundwater quality data. The model was calibrated to recent groundwater elevations and water quality trends, then used to predict groundwater flow conditions and groundwater quality changes resulting from current operations (irrigation return flow and recharge of secondary effluent at WRP 10) and five potential CRW recharge alternatives.

#### 5.1.1. Model Objectives

The objectives of the groundwater model are to 1) aid in characterization and evaluation of local groundwater flow conditions, including flow rates and directions, 2) predict hydraulic impacts of various recharge rates and scenarios, including groundwater flowpaths and travel times from recharge basins and the adjacent WWR-SC to downgradient areas, 3) predict the dynamic water table mounding response and time required to reach equilibrium, and 4) evaluate potential future groundwater quality changes that could result from mixing of recharged water with native groundwater. Water quality simulations were conducted for five parameters: TDS, chloride, sulfate, nitrate, and chromium-6. Fluoride was not simulated, as concentrations for WRP 10 secondary effluent, shallow and deep groundwater, and CRW are similar and significantly below the State of California fluoride primary MCL of 2 mg/L.

#### 5.1.2. Regional Model Review

The existing regional MODFLOW model of the entire Coachella Valley (Fogg, et al., 1998) was reviewed to aid in development of the local model. The regional model simulates flow throughout the Coachella Valley, from the San Geronio Pass area to the Salton Sea. The historical simulation period is 1936 and 2015, using annual stress periods. The model includes four MODFLOW layers representing the shallow and deep aquifers and a regional aquitard within Model Layer 3. Variable model layer thickness and heterogeneous hydraulic conductivities are simulated. Variable annual production well pumping rates along with non-uniform and time-varying annual recharge (both natural recharge and return flows and managed recharge at the Whitewater Groundwater Replenishment Facility) are simulated.

TODD reviewed the model documentation and input parameters, obtained the regional model MODFLOW input files, imported the regional model to the Groundwater Modeling System™ (GMS), ran the historical simulation, and evaluated the results.

One characteristic of the regional model historical simulation is declining water levels over time in the Mid-Valley area. Beginning in the early 1940s, the southern portion of the regional model experiences declining water levels to below the defined bottom of Model Layer 1, and model cells become “dry” (inactive). By the mid-1950s, the majority of the mid-Valley area of the local model area is dry in Model Layer 1, and dry cells extend into Model Layers 2 and 3. In Model Layer 4 (where most municipal pumping occurs), simulated groundwater levels decline continuously throughout the 1940s, 1950s, and 1960s, then stabilize in recent decades.

Selected regional model aquifer parameters, including the distribution of horizontal hydraulic conductivity, were extracted and used as input to the local flow and water quality model. However, changes were made to the local model layering relative to the regional model. The local model retained four model layers, but used lower layer bottom elevations in Model Layers 1 and 2 in order to provide a fully-saturated simulation of the upper portions of the aquifer. Other characteristics of the regional model, including hydraulic conductivity distributions, appeared to be reasonable based on hydrogeologic information and available data, and were applied to the local model.

### **5.1.3. Local Model Approach and Scope**

The local model was constructed using the United States Geologic Survey (USGS) MODFLOW code, along with the particle-tracking flowpath model MODPATH, and the multi-species solute transport code MT3D-MS. These numerical models were selected for their usability, accuracy, efficiency and transportability. In particular the transportability of the public domain MODFLOW/MODPATH/MT3D-MS programs and site model input files are advantageous for future site modeling. Model construction and calibration was performed using GMS version 10.1, which pre- and post-processes MODFLOW/MODPATH/MT3D-MS files.

The local groundwater model area and boundaries are shown on **Figure 2**. The active model area encompasses around 20 mi<sup>2</sup>, and extends from near Gerald Ford Drive and Highway 10 on the north and east to near Route 111 and Jefferson Street on the southeast, and from the groundwater basin boundaries along outcropping bedrock south and west of the CVWD Palm Desert property. The model boundaries are located approximately 2.5 miles northwest, 2.5 miles east, 4 miles southeast, and 4 miles southwest of the Palm Desert property.

Input parameters for the MODFLOW model include aquifer layer geometry, layer thickness, horizontal and vertical hydraulic conductivity, aquifer storage coefficients, and areal recharge. General head boundary conditions were assigned based on observed groundwater elevations. Pumping wells were simulated in model layers corresponding to each well’s depth, and average flow rates were assigned, as discussed below.

In the local model area, groundwater inflow occurs via northwest to southeast flow from the upgradient areas into the model, and from irrigation return flow and discharge of treated

secondary effluent at WRP 10. For the recharge basin simulations, additional inflow occurs as simulated at the Palm Desert property and within the WWR-SC. Groundwater outflow occurs via wells and primarily through the southern model boundary.

The local model simulates steady-state and transient groundwater flow and transient water quality transport and mixing. Steady-state models were used for most of the predictive analyses. For each of the six predictive scenarios (described below) future well pumping and CRW recharge rates were assumed to be constant. Additional transient flow model simulations were performed to assess mounding response over time. The flow model was used to predict water table mounding beneath proposed recharge facilities, drawdown around nearby water supply wells, and flowpaths from the recharge facilities to downgradient areas and nearby production wells.

The MT3D-MS model utilizes the flow solution from the MODFLOW model to simulate horizontal and vertical advection and dispersion of the chemical constituents. Solute dispersion was simulated using longitudinal, transverse, and vertical dispersivities. Chemical reactions were not simulated. Initial concentration conditions and inflow concentrations at sources (i.e., model boundaries, irrigation return flow, WRP 10 effluent, and CRW recharge) are specified.

A qualitative calibration of the water quality model was performed to replicate observed recent concentration trends in deep aquifer production wells. Once calibrated, predictive water quality simulations were performed for each of the four water quality parameters.

## **5.2. MODFLOW MODEL INPUT**

### **5.2.1. Model Domain Grid and Layering**

The model area and boundaries are shown on **Figure 2**. The model area covers approximately 20 square miles, roughly centered on the CVWD Palm Desert property. The model grid comprises 326 rows by 245 columns with a variable cell size of 25 feet in the WRP 10 area to 250 feet at the model boundaries. Model grid columns were oriented north to south.

Four model layers were used to simulate the shallow and deep aquifers. **Figure 21** shows the elevations of the bottoms of the Model Layers 1 through 4. Model Layer 1 is unconfined with the sloping base of the layer defined as approximately 50 feet below the 2015 water table (between 250 and 320 feet-bgs). Model Layer 2 is the model layer with variable thickness at it includes the low-permeability regional Pleistocene clay aquitard, which was simulated in the regional MODFLOW model. Model Layer 2 ranges from less than 10 feet in the southern portion of the Study Area up to 160 feet thick in the eastern portion of the Study Area. Model Layer 2 is approximately 100 feet thick beneath the project site. Model Layers 3 and 4 are parallel with the Model Layer 2 bottom elevation surface, and were defined as 300 and 400 feet thick, respectively. While permeable alluvial sediments exist beneath Model Layer 4, a 400-foot thickness for Model Layer 4 was selected to effectively

simulate water quality changes in deeper aquifers as a result of CRW recharge. Four model layers were also used in the regional MODFLOW model; however, in the local model the bottoms of model layers were lowered to ensure that layers stayed saturated during the flow simulations.

### 5.2.2. Aquifer Hydraulic Properties

Aquifer hydraulic properties include horizontal hydraulic conductivity ( $K_H$ ), vertical hydraulic conductivity ( $K_V$ ) and storage coefficients ( $S$ ), are based on the respective values assigned in the regional model. Adjustments to  $K_H$  values were made to represent the aquitard zone in local Model Layer 2, as compared with its simulation in Model Layer 3 of the regional model. Final  $K_H$  values assigned to each model layer are shown on **Figure 22**. As shown on the figure, final  $K_H$  values for Model Layers 1, 3, and 4 are identical and range from 30 feet per day (feet/day) in the northeastern margins of the Study Area up to 225 feet/day in the western portion of the Study Area. In Model Layer 2, low  $K_H$  values (blue colors on the upper right map on Figure 22) in the central portion of the Study Area exist and correspond to the low permeability regional clay aquitard.  $K_H$  values in these areas of Model Layer 2 are  $1/10^{\text{th}}$  of the  $K_H$  values assigned to Model Layers 1, 3, and 4.

In contrast to  $K_H$  values,  $K_V$  values were modified during calibration. Initial simulations used a horizontal to vertical hydraulic conductivity ( $K_H/K_V$ ) ratio of 10:1 for each model layer. However, the apparent hydraulic isolation of the lower aquifer zone from the overlying aquifers (based on water quality distributions) warranted larger  $K_H/K_V$  ratios. The final calibrated model used a  $K_H/K_V$  ratio of 100:1, similar to the regional flow model.

Limited transient flow simulations also were run to assess aquifer response to CRW recharge, and evaluate the effect of aquifer storage properties (specific yield and specific storage) on aquifer response. A specific yield value of 0.15 was assigned to Model Layer 1, and specific storage values of 0.001 to 0.00005 were assigned to Model Layers 2 through 4.

### 5.2.3. Boundary Conditions

Because the local model comprises a much smaller area than the regional MODFLOW model, different model boundary conditions were required. Specified (general) heads were defined along each of the model boundaries (**Figure 2**). The assigned general head values are based on 2015 groundwater elevation measurements across the local model area (**Figure 3**) and range from approximately 30 feet msl at the northwestern boundary to -95 feet msl at the southeastern corner of the model. These boundary conditions were selected to allow accurate calibration of ambient groundwater elevations. The same groundwater elevation values were used for each model layer.

### 5.2.4. Production Well Pumping

There are 76 production wells in the local model that have been pumped between 2011 and 2015. These include 36 active CVWD production wells and 40 private irrigation wells. **Figure 23** shows the well locations and average annual pumping rates. As shown in the chart on the

figure, groundwater production has declined steadily over the past 15 years. Average annual production over the most recent five-year period (2011 to 2015) was 58,764 AFY, much lower than the 70,000 to 80,000 AFY production rates reported from 2000 to 2004.

The average annual pumping rates from 2011 to 2015 were assigned to each of the 76 well in the local model. As shown on water quality cross sections (Figures 7 through 12), production wells are completed (screened) at various depths within the deeper aquifer zones, with most wells completed across Model Layers 3 and 4. For these wells, percentages of the average annual pumping rate from 2011 to 2015 were allocated to respective model layers based on the total screen length within each model layer. For example, for a well completed with 300 feet of screen in Model Layer 3 and 200 feet of screen in Model Layer 4, 60 percent of the pumping was allocated to Layer 3, and 40 percent was allocated to Model Layer 4.

### 5.2.5. Baseline Outdoor Irrigation Return Flow

A large portion of the Study Area is represented by land uses that require nearly year-round irrigation. Not all of the irrigation water applied to golf course turf/lakes, municipal parks, and commercial/residential landscaping is consumptively used. The volume of water that infiltrates past the root zone and returns to groundwater is herein referred to as outdoor irrigation return flow and represents a significant component of groundwater recharge in the Study Area. A relatively simple approach was used to estimate the annual quantity of applied irrigation water and outdoor irrigation return flow. Calculations are summarized below in **Table 3** and described in further detail below.

**Table 3**  
**Summary of Applied Outdoor Irrigation and Return Flow**

Palm Desert Study Area	Rate (AFY)
A. Average Annual Study Area Groundwater Production (2011-15)	58,846
B. Imported Water Delivery for Irrigation (WRP 10) (2011-15)	1,618
C. Total Study Area Water Use (A+B)	60,464
Non Consumptive Use (34% of Total Water Use)	20,558
50% x Average Annual WRP 10 Effluent Produced (2011-15)	5,536
Outdoor Return Flow (= 20,558 AFY - 5,536 AFY)	15,022

The total water use in the Study Area is estimated to be 60,464 AFY, equivalent to the average annual (2011 to 2015) groundwater production (58,846 AFY) plus the average annual (2011 to 2015) imported CRW delivered via WRP 10 to local irrigation customers (1,618 AFY).

Non-consumptive use (including indoor and outdoor water use) is currently estimated at 34 percent of total water use, according to the 2016-17 Engineer’s Report on Water Supply and Assessment (CVWD, 2016). While non-consumptive use is expected to decline to about 30 percent by 2035 as a result of conservation measures and more efficient irrigation practices,

the 34 percent rate was applied in the model and equates to combined annual indoor and outdoor return flows of 20,558 AFY.

The portion of non-consumptive use represented by indoor return flows can be estimated from effluent flow data at WRP 10. The average annual effluent produced at WRP 10 from 2011 to 2015 was 11,072 AFY. However, only approximately 50 percent of the effluent at WRP 10 (5,536 AFY) is produced within the Study Area. By subtracting Study Area indoor effluent return flow (5,526 AFY) from total return flow (20,558 AFY), the Study Area outdoor irrigation return flow is estimated at approximately 15,000 AFY, or about 25 percent of the estimated outdoor applied water in the Study Area.

Average annual outdoor irrigation return flows were apportioned across mapped irrigated areas within the Study Area. To facilitate import of recharge in the form of outdoor irrigation return flow into the model, recharge was aggregated into 38 polygons as shown on **Figure 24**. It is noted that irrigated areas identified in the figure do not include irrigated turf on the numerous golf courses in the Study Area. However, outdoor irrigation return flows were also apportioned to irrigated golf course areas.

#### **5.2.6. Baseline WRP 10 Secondary Treated Effluent Discharge**

Discharge of secondary treated effluent at WRP 10 was also simulated. As shown on **Figure 25**, annual secondary treated effluent discharge rates at WRP 10 have steadily declined since the 1990s and have been relatively stable since 2009-2010. Accordingly, for the baseline flow and water quality simulation (see below), the average annual discharge rate from 2011 to 2015 (3,686 AFY) was applied as inflow over the existing WRP 10 percolation pond area. It is acknowledged that CVWD has recently accelerated plans to expand deliveries of non-potable water, including recycled water, to golf courses in the mid-valley area, which is expected to further reduce land disposal of secondary treated effluent at WRP 10.

#### **3.2.6. CRW Recharge Scenarios**

For the proposed CRW spreading basin simulations, uniform steady-state recharge was simulated at the proposed recharge site. Although actual recharge operations are predicted to be seasonal, for the purposes of the modeling task, recharge was simulated at a constant rate based on average annual recharge volumes applied to the simulated recharge area(s).

### **5.3. MODFLOW MODEL RESULTS**

#### **5.3.1. Calibration and Scenario 1 (Baseline) Flow Model Results**

The MODFLOW and MT3D-MS models were qualitatively calibrated to observed groundwater elevations and water quality trends. For MODFLOW model calibration, average annual pumping from existing production wells was simulated along with average WRP 10 secondary treated effluent land disposal from 2011 to 2015. To achieve model calibration, selected input parameters including vertical hydraulic conductivities and boundary

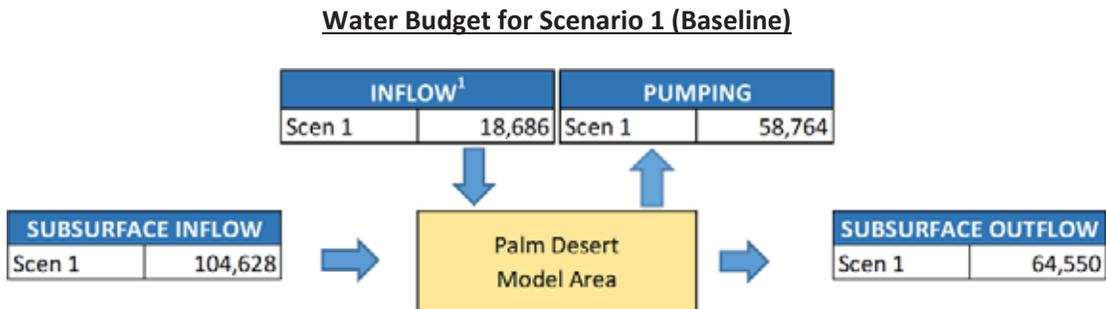
conditions were adjusted, while horizontal hydraulic conductivities, recharge, and production well pumping rates were not adjusted. The flow model calibration also accounted for simulated water quality trends. Specifically, TDS and nitrate up-trends in deep aquifer wells (Model Layers 3 and 4) predicted using the MT3D-MS model were compared with measured historical trends. An ad-hoc, trial-and-error flow and water-quality model calibration approach was used, in which aquifer hydraulic properties along with longitudinal and vertical solute dispersivities were varied (further discussed below).

Scenario 1 (Baseline) Groundwater Levels

The steady-state MODFLOW model results were compared with measured 2015 groundwater elevations (Figure 3). Figure 26 shows the simulated groundwater elevations in Model Layers 1, 3, and 4. As shown in the figure, the direction of groundwater flow in each layer is northwest to southeast across the Study Area. Simulated groundwater elevations in all layers range from around 0 feet msl at the northwest (upgradient) model boundary to about -90 feet msl at the southeastern boundary. In Model Layer 1, a water table mound of around 15-20 feet is simulated beneath the WRP 10 effluent discharge pond area (see closed -15 elevation contour). In Model Layers 3 and 4, small cones of depression are simulated around the existing production wells. Overall, the simulated water levels reasonably match the observed 2015 groundwater levels.

Scenario 1 (Baseline) Water Budget

The model water budget results for Scenario 1 (Baseline) is summarized in the schematic below:



1 - Scenario 1 Inflow = secondary treated effluent (3,686 AFY) + irrigation return (15,000 AFY)

Under current (baseline) conditions, inflows to the local model domain are represented by subsurface inflow, primarily through the northwestern (upgradient) boundary. Minor subsurface inflow also occurs through the western and eastern model boundaries. Total subsurface inflow is 104,628 AFY. Inflow from secondary treated effluent at WRP 10 (3,686 AFY) and irrigation return flow (15,000 AFY) totals 18,686 AFY. In comparison, total groundwater pumping (CVWD and private wells) within the local model area is 58,764 AFY. Total subsurface outflow through primarily the southern boundary, is 64,550 AFY, with total inflows equal to total outflows in the steady-state model.

It is noted that simulated baseline subsurface inflows and outflows tabulated above include flows into and out of all model boundaries and are higher than simulated in the regional flow model. The differences are primarily a result of using specified (general) heads along the south/southwestern model boundary in the local model versus specified fluxes in the regional model. Additionally, the steady-state groundwater flow simulation of the local model does not account for the effect of annual groundwater storage losses and their effects on subsurface inflows and outflows. Notwithstanding these differences, using general head model boundaries for the south/southwest model boundaries does not significantly affect the simulated flowpaths and travel times of recharged water or estimated groundwater storage benefits within the model area and adjacent upgradient and downgradient areas in the Basin.

Forward flowpaths from the WRP site were simulated with MODPATH. MODPATH uses the three-dimensional flow solution from MODFLOW and tracks particles (flowpaths) from the ponds to downgradient areas. Flow velocities and travel times are also calculated. Note that estimated flow velocities and travel times from the ponds to the production wells are dependent on aquifer hydraulic properties including hydraulic conductivity and effective porosity. For this analysis, an effective porosity of 15% was assumed for all model layers.

The upper left map on **Figure 27** shows the simulated forward flowpaths from the Palm Desert property in map view under baseline conditions. The flowpath lines are 2-dimensional projections of the simulated 3-dimensional curvilinear paths calculated within the three-layer model. The figures reveal that secondary treated effluent discharged at the existing WRP percolation ponds spreads out over an area approximately 2 miles at its maximum width. A portion of the percolated water flows vertically into the deeper aquifers. **Figure 28** provides a cross-sectional view of the predicted forward flowpaths, and illustrates the vertical spreading of percolated water between the shallow and deep aquifer zones over the course of several decades as indicated by the red arrowheads reflecting 10-year travel times. The figure reveals that this percolated water primarily travels to Model Layers 1, 2, and 3. A relatively small fraction of this percolated water travels into the upper portions of Model Layer 4.

Estimated travel times of individual water particles are correlated with expected water quality impacts over space and time. The travel time from the existing ponds to the nearest CVWD production well (22B2 as shown on Figure 26) is approximately 8 years. Subsurface flow velocities of secondary treated effluent decrease with increasing distance away from the percolation ponds. For example, travel time of percolated water to CVWD production well 14G1 is 21 years, while travel time of percolated water to CVWD production well 13R1 is 90 years.

It is noted that the Palm Desert model was also calibrated to recent stable to increasing water quality trends in wells. This calibration process and results are further discussed in Sections 5.4 and 5.5. Based on the qualitative groundwater flow (and water quality) calibration results, model calibration was judged to be acceptable, and the model was used to predict recharge mounding and flowpaths for five CRW recharge scenarios.

### 5.3.2. Colorado River Water Recharge Scenarios

In addition to Scenario 1 (Baseline), which simulated current conditions with discharge of secondary treated effluent, five alternative CRW recharge scenarios were simulated as summarized below in **Table 4**.

**Table 4**  
**Summary of Baseline and Future CRW Recharge Simulations**

Future CRW Recharge Scenarios	CRW Recharge Rates (AFY)		Comment
	Palm Desert Property Basins	WWR-SC Basins	
Scenario 1	0	0	Existing Baseline
Scenario 2	20,000	0	Concept 2 (north-south basins; with reservoir conversion)
Scenario 3	16,000	0	Concept 1 (east-west basins; no reservoir conversion)
Scenario 4	0	40,000	WWR-SC Only - Portola Ave to Fred Waring Dr.
Scenario 5	0	15,000	WWR-SC Only - Cook St. to Fred Waring Dr.
Scenario 6	16,000	20,000	Concept 1 Basins / WWR-SC - Portola Ave. to Fred Waring Dr.

As summarized in the table, Scenarios 2 and 3 include CRW recharge at the CVWD Palm Desert property only, while Scenarios 4 and 5 include recharge in the WWR-SC only. Scenario 6 includes recharge in both recharge basins on the Palm Desert property and in the WWR-SC. Annual recharge rates for the re-configured spreading basins on the CVWD Palm Desert property are based on an assumed sustainable recharge rate of 1.75 feet/day and active recharge period of 10 months (or 304 days) per year. These assumptions are deemed reasonable, yet conservative based on recent infiltration test results at the existing percolation ponds at the Palm Desert property and current operations and maintenance programs at the TEL-GRF.

For Scenario 2, the larger 38-acre basin area concept (Concept 2 that includes re-purposing the existing lined back feed reservoir as a dual purpose, storage-recharge basin) was simulated. For Scenarios 3 and 6, the smaller 31-acre concept (Concept 1; which does not include the back feed reservoir) was simulated. As stated earlier, the three former percolation ponds in the northwest portion of the Palm Desert property (maroon outlined basins shown on Figure 1) may someday be developed for storage but are also potentially available for CRW recharge. To provide a conservative estimation of recharge potential at the Palm Desert property, the three former percolation ponds were not included in the development and analysis of recharge concept alternatives and were thus not simulated as future recharge basins. Notwithstanding their exclusion in this study, recharge in the former percolation pond areas is certainly feasible and would increase the recharge capacity of the Palm Desert property.

A maximum annual recharge capacity of about 50,000 AFY was estimated for the 2-mile, approximately 93-acre reach of the WWR-SC based on a sustainable recharge rate of 1.75 feet/day and active recharge period of 10 months (or 304 days). However, due to anticipated limitations of CRW delivery via the MVP pipeline, a more realistic annual

replenishment program of 40,000 AFY in the WWR-SC was simulated in Scenario 4. During initial project discussions, CVWD staff indicated that a recharge project in the eastern portion of the WWR-SC from Cook St. to Fred Waring Ave. may be preferred over the entire 2-mile reach. The eastern reach has approximately 29 acres of spreading area and a recharge capacity of 15,000 AFY. This smaller WWR-SC recharge program was simulated in Scenario 5. A 36,000 AFY replenishment project combining recharge at both the CVWD Palm Desert property and in the WWR-SC recharge facility was simulated in Scenario 6. Concept 1 (16,000 AFY) was simulated for the Palm Desert spreading basins along with a moderate recharge program of 20,000 AFY for the entire 2-mile reach of the WWR-SC from Portola Ave. to Fred Waring St.

For each scenario, uniform steady-state recharge was simulated over the defined Palm Desert property and WWR-SC areas. Future groundwater production from Study Area wells was simulated at the same average annual pumping rates as in Scenario 1 (Baseline). For each scenario, groundwater elevations and water table changes relative to Scenario 1 (Baseline) were simulated, and forward flowpaths from the recharge areas were simulated using MODPATH.

### 5.3.3. Groundwater Flow Model Results

Simulated forward flowpaths from recharge areas for all six scenarios are depicted in plan-view and cross-sectional view on **Figures 27 and 28**, respectively. Results are also captured in future groundwater elevation contour maps and water level change maps on **Figures 29 through 38**. The water level change maps show the simulated change (increases) in groundwater levels relative to Scenario 1 (Baseline) in Model Layers 1, 3, and 4.

The steady-state model predicts an equilibrium groundwater flow solution for ongoing continuous recharge. Thus, the predicted water level change (or mounding) represents a maximum water table condition. To facilitate comparison of scenarios, the maximum groundwater mound height above baseline conditions beneath simulated recharge areas is summarized below in **Table 5**.

Because the historical depth to water beneath the site (as measured in the shallow site monitoring wells) is currently in the range of approximately 200 feet-bgs, and the predicted maximum mounding for any scenario is less than 100 feet, the aquifer has sufficient capacity to accept proposed CRW recharge without hydraulic rejection and/or daylighting of water.

A translation of steady-state water level mound heights in terms of volumetric groundwater storage change within the Study Area and quantification of regional groundwater storage benefits outside of the Study Area from CRW recharge is presented in the following section (Section 5.3.4).

Additional description of groundwater flow modeling results within the Study Area by scenario is presented below.

**Table 5**  
**Simulated Maximum Water Level Mounding beneath Recharge Areas**

Future Scenario with CRW Recharge	CRW Recharge Rates (AFY)		Maximum Water Level Mounding beneath Recharge Areas <sup>1</sup> (feet)		
	Palm Desert Property Basins	WWR-SC Basins	Model Layer 1	Model Layer 3	Model Layer 4
Scenario 2	20,000	0	100	8	4
Scenario 3	16,000	0	80	6	3
Scenario 4	0	40,000	95	14	8
Scenario 5	0	15,000	60	5	2
Scenario 6	16,000	20,000	100	12	8

Notes:

1 – represents water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions

*Scenario 2 – 20,000 AFY Recharge at Palm Desert Property*

**Figure 29** shows the simulated groundwater elevations with 20,000 AFY recharge, and **Figure 30** shows the simulated change (increases) in groundwater levels relative to Scenario 1 (Baseline) in Model Layers 1, 3, and 4. Simulated groundwater elevations in Model Layer 1 exceed 90 feet msl beneath the WRP 10 pond area. The effects of recharge is also observed in Model Layer 3, where the -25 feet msl contour is deflected.

As shown on Figure 30, the simulated recharge mound shape is roughly rectangular, reflecting the rectangular recharge area at the site. The maximum water table mounding simulated beneath the center of the recharge area is approximately 100 feet. The predicted maximum mound height in Model Layers 3 and 4 is approximately 8 and 4 feet, respectively.

**Figures 27** and **28** shows the simulated forward flowpaths for Scenario 2 in plan-view and cross-sectional view. The figures reveal that the Scenario 2 recharged water spreads out over a relatively broad area (approximately 2.5 miles at its maximum width), and a larger portion of the recharged water flows vertically into Model Layer 4 as compared to water percolated under Scenario 1 (Baseline) conditions. The flow velocities for Scenario 2 are increased relative to Scenario 1 (Baseline). For example, the travel time from the proposed recharge basins on the Palm Desert property to the nearest CVWD production well (22B2 as shown on Figure 26) is approximately 3 years (versus 8 years for percolated water in Scenario 1). Subsurface flow velocities of CRW decrease with increasing distance away from the ponds but are still much faster in comparison to percolated water in Scenario 1. For example, travel time of recharge water to CVWD production well 14G1 is 8 years (versus 21 years in Scenario 1), while travel time of recharge water to CVWD production well 13R1 is 60 years (versus 90 years in Scenario 1).

A number of downgradient CVWD wells capture some of the recharged water. The MT3D-MS modeling also indicates recharge water is captured by the wells, and water quality changes occur in these downgradient wells, as described in Section 5.4. A portion of the recharge water is not captured by the downgradient wells and continues to flow to the southeast beyond the production wells and exits the local model boundary.

Scenario 3 – 16,000 AFY Recharge at Palm Desert Property

Scenario 3 is similar to Scenario 2, except only 16,000 AFY are recharged over a slightly smaller area than for Scenario 2. **Figure 31** shows the simulated groundwater elevations with 16,000 AFY recharge, and **Figure 32** shows the simulated change in water levels relative to no recharge in Model Layers 1, 3, and 4. Simulated groundwater elevations in Model Layer 1 exceed 70 feet msl beneath the WRP 10 pond area.

As shown on Figure 32, the maximum water table mounding simulated beneath the center of the recharge area is over 80 feet. The predicted maximum mound height in Model Layers 3 and 4 is approximately 6 and 3 feet, respectively.

**Figures 27** and **28** shows the simulated forward flowpaths for Scenario 3 in plan-view and cross-sectional view. Comparison of the flowpath results for Scenarios 2 and 3 reveals that the Scenario 3 recharged water spreads out over a slightly smaller area, and a smaller portion of the recharged water flows vertically into the deeper aquifer, although the differences are relatively small. Subsurface travel times of CRW water are similar but slightly slower under Scenario 3 than under Scenario 2.

Scenario 4 – 40,000 AFY Recharge via WWR-SC (Portola Avenue to Fred Waring Drive)

**Figure 33** shows the simulated groundwater elevations with 40,000 AFY recharged via the River Channel, and **Figure 34** shows the simulated change in water levels relative to the Baseline Scenario. This scenario represents the maximum rate of recharge for all scenarios simulated. For Scenario 4, groundwater elevations in Model Layer 1 exceed 65 feet msl beneath the river channel area. The effects of recharge is also observed in Model Layer 3, where the -15 feet msl contour is deflected.

As shown on Figure 34, the maximum water table mounding simulated beneath the eastern portion of the recharge area in the WWR-SC is over 95 feet. Even though twice as much water is recharged than compared to Scenario 2, the maximum mound heights are similar under both scenarios, because recharge applied within the WWR-SC is over a larger area. The recharge mound for this scenario is elongated based on the linear recharge area in the WWR-SC. The predicted maximum mound height in Model Layers 3 and 4 is approximately 14 and 8 feet, respectively. Mound heights in Model Layers 3 and 4 are the highest of any scenario, which is expected given that CRW recharge in Scenario 4 is the highest of any scenario.

**Figures 27** and **28** shows the simulated forward flowpaths for Scenario 4 in plan-view and cross-sectional view. Comparison of the flowpath results for Scenarios 1 through 4 reveals that Scenario 4 recharged water spreads out over a much greater area, and a larger portion of the recharged water flows vertically into the deeper aquifer. As shown on Figure 27, recharge flowpaths extend approximately 0.5 miles west of Portola Ave in Scenario 4. The MT3D-MS modeling indicates recharge water is captured by local production wells, and water quality changes occur in these downgradient wells, as described in Section 5.4. Similar to the other scenarios, a portion of the recharge water is not captured by the downgradient wells and continues to flow to the southeast beyond the production wells and exits the local model boundary.

Given that annual recharge rates in Scenario 4 are the highest simulated, subsurface flow velocities of CRW are generally the fastest. The travel time of CRW to the nearest CVWD production well (22B2) is 0.5 years, due to the close proximity of the WWR-SC recharge area to the well. Subsurface flow velocities of CRW decrease with increasing distance away from the ponds but are still faster in comparison to secondary effluent in Scenario 1. For example, travel time of recharge water to CVWD production well 14G1 is approximately 7 years (versus 21 years in Scenario 1 and 8 years in Scenarios 2 and 3), while travel time of recharge water to CVWD production well 13R1 is approximately 50 years (versus 90 years in Scenario 1 and 60 years in Scenarios 2 and 3).

*Scenario 5 – 15,000 AFY Recharge via WWR-SC (Cook Street to Fred Waring Drive)*

**Figure 35** shows the simulated groundwater elevations with 15,000 AFY recharged via the eastern (Cook to Fred Waring) portion of the River Channel. **Figure 36** shows the simulated change in water levels relative to the Baseline Scenario. For Scenario 5, groundwater elevations in Model Layer 1 exceed 30 feet msl beneath the eastern Channel area. The effects of recharge is also observed in Model Layer 3, where the -25 feet msl contour is deflected.

As shown on Figure 36, the maximum water table mounding simulated beneath the eastern portion of the River Channel recharge area is over 60 feet. It is noted that in the northern portion of the Palm Desert property, groundwater levels decrease by about 2 feet relative to Scenario 1 conditions. This occurs because this scenario assumes that land disposal of secondary treated effluent is halted in the existing WRP 10 percolation ponds, and the 15,000 AFY of CRW recharge occurs only in the WWR-SC. The predicted maximum mound height in Model Layers 3 and 4 is approximately 5 and 2 feet, respectively.

**Figures 27** and **28** shows the simulated forward flowpaths for Scenario 5 in plan-view and cross-sectional view. Comparison of the flowpath results for Scenarios 4 and 5 reveals that the Scenario 5 recharged water spreads out over a much smaller area and does not migrate as deeply into Model Layer 4 as in Scenario 4.

Subsurface travel times of CRW water for Scenario 5 are similar to velocities estimated under Scenarios 2 and 3 with travel time to the nearest CVWD production well (22B2) being

slightly faster due to the closer proximity of the WWR-SC recharge area compared to recharge basins on the CVWD Palm Desert property.

Scenario 6 – 36,000 AFY Recharge (16,000 AFY at Palm Desert Property and 20,000 AFY via WWR-SC (Portola Avenue to Fred Waring Drive))

Scenario 6 simulates 36,000 AFY of CRW recharged at the Palm Desert site (16,000 AFY) and via the WWR-SC from Portola Avenue to Fred Waring Drive (20,000 AFY). **Figure 37** shows the simulated groundwater elevations, and **Figure 38** shows the simulated change in water levels relative to the Baseline Scenario. For Scenario 6, groundwater elevations in Model Layer 1 exceed 90 feet msl beneath the WRP recharge area. The effects of recharge is also observed in Model Layer 3, where the -20 feet msl contour is deflected.

As shown on Figure 38, the maximum water table mounding simulated beneath the WRP 10 pond area is just over 100 feet. Again, the aquifer appears to have the capacity to accept this recharge volume and distribution, but greater volumes of recharge may exceed the aquifer's capacity. The predicted maximum mound height in Model Layers 3 and 4 is approximately 12 and 8 feet, respectively.

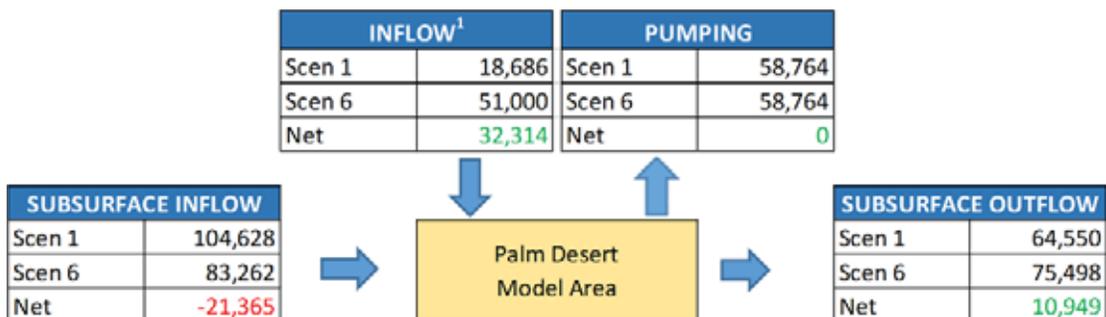
**Figures 27** and **28** shows the simulated forward flowpaths for Scenario 6 in plan-view and cross-sectional view. Comparison of the flowpath results for Scenarios 1 through 6 reveals that the Scenario 6 recharged water spreads out over the second largest area (less than only Scenario 4), and a large portion of the recharged water flows vertically into the deep portions of Model Layer 4.

Subsurface travel times of CRW water are similar to those under Scenario 4 with some variability given that recharge was simulated over a broader area in two recharge facilities. The travel time of CRW to the nearest CVWD production well (22B2) is about 0.5 years, similar to Scenario 4. Subsurface flow velocities of CRW decrease with increasing distance away from the ponds. Travel time of recharge water to CVWD production well 14G1 is approximately 8 years, while travel time of recharge water to CVWD production well 13R1 is approximately 55 years. These travel times are only slightly greater than in Scenario 4.

#### **5.3.4. Local and Regional Groundwater Storage Benefits from CRW Recharge**

To illustrate the local and regional groundwater storage benefits from CRW recharge, model water budgets for Scenario 1 (baseline) and Scenario 6 (36,000 AFY Recharge) are summarized in the schematic below:

### Water Budgets for Scenario 1 (Baseline) and Scenario 6 (36,000 AFY Recharge)



1 - Scenario 1 Inflow = secondary treated effluent (3,686 AFY) + irrigation return (15,000 AFY)  
 Scenario 6 Inflow = CRW recharge (36,000 AFY) + irrigation return (15,000 AFY)

The schematic shows that total recharge under Scenario 6 increases by 32,314 AFY compared to Scenario 1 (from 18,686 AFY to 51,000 AFY). Transient flow results indicate that near steady-state water level mounding is generally achieved within approximately 5 years after the start of recharge operations. The final steady-state water level mound correlates to an increase in groundwater storage of 28,750 AF within the Study Area. During the 5-year non-equilibrium period, subsurface inflows and outflows through model head boundaries change in response to the development of the mound and changing hydraulic gradients along local model boundaries. Once equilibrium conditions are achieved, there is no further groundwater storage change in the Study Area, and the benefit of increased recharge over baseline conditions is directed to adjacent portions of the Basin. Specifically, subsurface inflow into the Study Area, primarily through the northwestern (upgradient) boundary, decreases by 21,365 AFY (from 104,628 AFY to 83,262 AFY). This decrease represents the annual groundwater storage benefit to portions of the Basin upgradient of the local model. In contrast, subsurface outflow from the Study Area, primarily through the southeastern (downgradient) boundary, increases by 10,949 AFY (from 64,550 AFY to 75,498 AFY). This increase effectively represents the annual benefit to the EWR AOB.

Similar local and regional groundwater storage benefits for Scenario 6 would be expected for Scenario 4 (40,000 AFY of CRW Recharge). Local and regional storage benefits under Scenarios 2, 3, and 5 (15,000 AFY to 20,000 AFY of CRW Recharge) are roughly one-half of those under Scenarios 4 and 6.

#### **5.4. MT3D-MS SOLUTE TRANSPORT MODEL**

Potential water quality changes resulting from these six recharge scenarios were simulated using the MT3D-MS model. Assumptions, input parameters, and water quality simulation results are discussed below.

#### **5.4.1. MT3D-MS Model Input**

The MT3D-MS model was used to predict future changes in shallow and deep aquifer groundwater quality in response to recharge. This section summarizes the input parameters and assumptions used to conduct the water quality modeling.

Solute transport model input parameters are those variables that control migration and mixing of the water quality parameters of concern. In addition to the advective flow system (horizontal and vertical flow directions and flow rates), other solute transport model input include solute initial conditions, concentration or mass flux sources (recharge and boundary inflows), solute dispersivity, and parameters for chemical reactions.

#### **5.4.2. Solute Species Simulated**

Based on communications with CVWD staff, five water quality constituents of concern were simulated: TDS, chloride, sulfate, nitrate, and chromium-6. These chemicals were modeled, because they are present at variable concentrations across the Study Area and Basin, and each constituent has been detected at concentrations approaching or exceeding primary MCLs or secondary MCL ranges for drinking water.

The spatial distributions of the five water quality constituents in groundwater are highly variable. For example, the highest concentrations of TDS, chloride, sulfate, and nitrate are measured and inferred in shallow upper aquifer zone, whereas for chromium-6, the highest concentrations are observed in the deep aquifer zone. Historical concentration increases are noted for TDS, chloride, sulfate, and nitrate in wells screened in Model Layers 3 and 4 (see time-concentration plots in Appendix B), indicating anthropogenic sources and pumping influences. In addition, the water quality characteristics of CRW vary by species. CRW has very low to non-detectable nitrate and chromium-6 concentrations but higher concentrations of TDS, chloride, and sulfate than are currently present in the deeper production zone aquifers (Model Layer 3 and 4). The water quality responses in the shallow and deep aquifer zones are anticipated to vary significantly based on these different ambient and recharge water quality characteristics.

#### **5.4.3. Initial Groundwater Conditions and Sources**

Simulation of future chemical concentrations requires development of representative chemical initial conditions and source terms, both with respect to the concentrations used as starting conditions for each chemical and aquifer zone/model layer and potential future chemical mass input to the aquifer.

Initial concentrations for each constituent by model layer were developed by manually interpreting and contouring the water quality data shown on cross sections A-A through F-F (Figures 7 through 12). As discussed in Section 2.2, the only data available to characterize groundwater quality in the shallow aquifer (Model Layer 1) were from the six WRP 10 monitoring wells; almost all of the local CVWD production wells are completed in the deeper aquifer zones (Model Layers 3 and 4). Accordingly, uncertainty exists regarding the

distribution of chemicals in Model Layers 1 and 2 outside of the monitoring well network. For Model Layer 1, representative concentrations for MW1 through MW6 were extrapolated from respective datasets and used as initial concentrations. For Model Layer 2, intermediate concentrations were defined based on the trends observed between the shallow monitoring wells and deeper production wells primarily screened in Model Layers 3 and 4.

**Figures 39 through 43** show the initial concentrations used for each chemical by model layer. Non-uniform initial concentration distributions were used for each constituent and model layer. For TDS, chloride, sulfate, and nitrate, higher initial concentrations are simulated in the shallow model layers than in the deeper layers. For chromium-6, generally lower concentrations are simulated in the shallow aquifer. Based on extrapolation of monitoring and production well water quality, some variability in initial concentrations is simulated within each layer.

During the solute transport simulations, the initial concentration variability within and between model layers, along with the variable concentrations of recharge water, results in mixing and changes in concentration within the model area. For Scenario 1 (baseline), these changes were carefully evaluated to ensure predicted baseline trends reasonably matched observed trends in deeper production wells (see time-concentration plots in Appendix B), prior to simulating the CRW recharge scenarios.

#### 5.4.4. Concentrations for Irrigation Return Flow, WRP 10 Secondary Treated Effluent, CRW via MVP, and Model Boundaries

Each identified source of recharge in the local model (outdoor irrigation return flow, WRP 10 secondary effluent, CRW recharge, and subsurface inflow via model boundaries) was assigned a set of representative constituent concentrations. **Table 6** summarizes the concentrations assigned for each recharge source.

**Table 6**  
**Concentrations of Simulated Recharge Sources for Local Model**

Water Source	Concentrations (mg/L)				
	TDS	Chloride	Sulfate	Nitrate	Cr6 <sup>4</sup>
Irrigation Return Flow <sup>1</sup>	1,000	100	160	80	8
WRP 10 Secondary Effluent <sup>2</sup>	453	71	69	62	8
CRW via Mid Valley Pipeline <sup>3</sup>	742	112	271	0.5	0
Inflow through Model Boundaries	Initial, variable concentrations along model boundaries by model layer				

Notes:

- 1 – Based on estimated weighted-average concentration of pumped groundwater and evapoconcentration factor of 4 for TDS, chloride, and sulfate and analysis of nitrate leaching (US Davis, 2012)
- 2 – Average concentration from 2011 to 2015
- 3 – Average concentration of monthly samples collected at Avenue 52 East from 2012 to 2015
- 4 – Chromium-6 (Cr6) concentration based on 2015 WRP 10 influent concentration sample; no evapoconcentration factor applied to irrigation return flow

### Outdoor Irrigation Return Flow

For irrigation return flow, source water concentrations were estimated based on the pumping weighted-average concentrations across the Study Area in Layers 3 and 4 and increased to account for evapoconcentration effects (TDS, chloride, and sulfate) and leaching of nitrogen from applied fertilizers (nitrate).

As discussed in Section 5.2.5, return flows across the Study Area are estimated to be approximately 15,000 AFY. This return flow rate equates to about 25 percent of applied outdoor irrigation and a conservative evapoconcentration factor of 4. The percentage of groundwater pumping in the Study Area is approximately 33 percent from Layer 3 and 67 percent from Layer 4. Therefore, applied irrigation water is weighted towards the generally higher quality water in Layer 4 (with exception of slightly poorer quality water for chromium-6).

The pumping-weighted average groundwater concentrations applied to outdoor irrigation water are 250 mg/L for TDS, 25 mg/L for chloride, 40 mg/L for sulfate, 20 mg/L for nitrate as NO<sub>3</sub>, and 8 ug/L for chromium-6. For chromium-6, the average historical influent concentrations of 8 ug/L for WRP 10 was assumed to reasonably represent the average concentration of locally pumped groundwater for irrigation.

Based on a conservative evapoconcentration factor of 4, estimated outdoor irrigation return flow concentrations are 1,000 mg/L for TDS, 100 mg/L for chloride, and 160 mg/L for sulfate. These concentrations are relatively similar to estimated shallow TDS, chloride, and sulfate groundwater concentrations in the Study Area beyond the influence of WRP 10 effluent discharge.

For nitrate, key findings from the recent study of nitrate loading by UC Davis (2012) indicate that up to approximately 8.9 pounds per acre of nitrate on average can leach below irrigated turf with fertilizer application accounting for nitrogen evapoconcentration and uptake. This equates to 13 mg/L of additional nitrate (as NO<sub>3</sub>) added to the source water. While previous studies found that nitrate leaching potential beneath irrigated turfgrass is generally low (Gibeault et. al., 1999 and Wu et. al., 2007), the UC Davis values were applied for this evaluation to provide a conservative estimate of nitrate loading. Based on a 20 mg/L source water nitrate concentration, a return flow nitrate-NO<sub>3</sub> concentration of 33 mg/L was estimated and applied in the local model.

With respect to chromium-6, monitoring data for shallow groundwater do not exist in the Study Area. However, chromium concentrations in shallow groundwater in the Rancho Mirage area (TODD, 2014) indicated that any evapoconcentration of chromium-6 in applied water appears to be counter-balanced by precipitation in the near-surface. Vadose zone soil leaching results also indicate that chromium-6 dissolution in the vadose zone by recharge water is minimal. Therefore, the estimated chromium-6 irrigation source water concentration of 8 ug/L was assumed for irrigation return flow in the local model.

#### WRP 10 Secondary Treated Effluent

For the WRP effluent discharge simulated in Scenario 1 (Baseline), the respective average concentrations for WRP 10 secondary treated effluent over the five-year period from 2011 to 2015 were applied. As shown in Table 1, the TDS concentration for secondary effluent (453 mg/L) is lower than shallow groundwater across the Study Area, which ranges from about 500 to 750 mg/L.

#### CRW via Mid Valley Pipeline

For the CRW recharge simulations (Scenarios 2 through 6), the average concentrations for CRW water sampled at Avenue 52 East over the four-year period from 2012 to 2015 were applied. As shown in Table 1, the TDS, chloride, and sulfate concentrations of CRW water are similar to concentrations in shallow groundwater in the Study Area and elevated in comparison to groundwater in deeper production zone aquifers (Model Layers 3 and 4). In contrast, CRW water has very low concentrations of nitrate (0.5 mg/L) and chromium-6 (non-detect).

#### Subsurface Inflow through Model Boundaries

Ongoing chemical sources from upgradient areas were simulated using constant concentration cells defined along the upgradient boundaries in each layer. The concentration values assigned to the boundaries were identical to the initial condition concentrations along each boundary for each model layer.

#### **5.4.5. Dispersivity**

The transport model parameter dispersivity reflects several physical and chemical processes that cause dissolved constituents to spread along and perpendicular to groundwater flowpaths. These dispersive processes include molecular chemical diffusion, small scale physical dispersion that occurs as the groundwater flows through torturous pathways in pore spaces, and larger scale physical dispersion as the solutes flow through macroscopic paleo-channel deposits.

One characteristic of the solute transport model is dispersive mixing of the non-uniform initial concentrations in each model layer, and mixing between model layers, even in the absence of recharge. For example high-TDS groundwater in the upper model layers will disperse vertically into the lower model layers, resulting in concentration increases in lower model layers over time.

Simulated longitudinal, transverse, and vertical dispersivities were varied during model calibration to achieve an appropriate level of vertical plume dispersion from shallow to deep model layers. The solute transport model results using different dispersivities were compared with measured TDS trends in deep production wells to evaluate the calibration quality of the MT3D-MS model. The results are discussed below. Ultimately, a homogeneous

dispersivity distribution with a longitudinal dispersivity of 25 feet, transverse dispersivity of 2.5 feet, and vertical dispersivity of 0.25 feet was used in the predictive simulations.

#### **5.4.6. Chemical Reactions**

No chemical reactions (decay/degradation, adsorption/retardation, or other reactions) were simulated. Although it is likely that some denitrification occurs in the deeper aquifer zones and attenuates nitrate, this process was not simulated. The absence of nitrate degradation is considered a conservative assumption in predicting future nitrate concentrations in the lower model layers.

### **5.5. MT3D-MS MODEL RESULTS**

The MT3D-MS model was constructed to predict potential future TDS, chloride, sulfate, nitrate, and chromium-6 concentrations for a 30-year period. To assess the validity of the predictive models, qualitative calibration simulations were first performed using baseline irrigation return flow and average WRP 10 effluent discharge, and the results were compared with observed historical concentration trends.

#### **5.5.1. MT3D Model Calibration**

The initial model runs simulated future water quality changes with current pumping and recharge, but without additional CRW recharge. Adjustments to model layer horizontal hydraulic conductivity, vertical anisotropy, and solute transport model dispersivity values were made until predicted future concentration trends in the deep aquifer (Model Layers 3 and 4) matched recent observed trends.

##### *Scenario 1 (Baseline) Predicted Future Concentrations*

**Figures 44 through 48** shows the simulated time-concentration charts for three Layer 1 wells, five Layer 3 wells, and five Layer 4 wells for all six scenarios. Simulated future concentrations for Scenario 1 (Baseline) are shown as black-colored curves on each time-concentration chart. In the Layer 1 wells, simulated concentrations for all water quality parameters fluctuate during the first few years of the simulation. Concentrations then either trend toward the concentration values of the WRP 10 secondary treated effluent or asymptote toward a constant value reflective of mixing of WRP 10 secondary treated effluent and the background Layer 1 concentration. In Layers 3 and 4, simulated TDS, chloride, sulfate, nitrate concentrations start at the initial concentration defined for the respective model layer, and generally increase over time. After 30 years, TDS concentrations in the wells have increased by around 20 to 40 percent relative to initial concentrations. The simulated concentrations increase at rates similar to the recent historical concentration trends observed (and shown on time-concentration plots provided in appendix B). This indicates that the local model reasonably represents current and potential future water quality conditions. Chromium-6 concentrations in Layers 3 and 4 wells gradually decrease,

as lower chromium-6 concentrations in shallow groundwater and recharge water migrates into and mixes with higher concentrations of groundwater in deeper aquifers.

Additional sensitivity simulations using higher values of dispersivity or lower flow model vertical anisotropy resulted in larger baseline concentration increases.

### **5.5.2. Predicted Water Quality Changes for CRW Recharge Scenarios**

The calibrated MT3D-MS model was used to predict future TDS, chloride, sulfate, nitrate, and chromium-6 concentrations for the baseline and five CRW recharge scenarios. Results are summarized in a series of time-concentration plots for selected well locations by constituent as shown on **Figure 44 through 48**. The iso-concentration contours on each map are the same initial concentrations used in the simulations (Figures 39 through 43) and are shown to aid in the interpretation of future water quality trends. Discussion of the simulated future concentration trends and comparison by scenario is presented below.

#### *Scenario 2 – 20,000 AFY Recharge at Palm Desert Property*

Scenario 2 results are represented by the maroon-colored concentration curves on Figures 44 through 48. As shown on Figure 44, predicted TDS concentrations in the Layer 1 wells initially fluctuate, then approach a constant value (748 mg/L) close to the CRW recharge concentration for TDS (742 mg/L). In Layer 3 wells, TDS concentrations increase over time from around 200 to 300 mg/L up to between 550 and 750 mg/L. In downgradient well 13R1, only small concentration changes are simulated. Travel time calculations based on particle tracking indicates that the simulated CRW recharge does not migrate as far as 13R1 within the 30-year simulation period; however, the mound development from CRW recharge influences the rate of migration of existing groundwater between the Palm Desert property and well 13R1. In Layer 4 wells, TDS concentrations increase over time from around 150 to 300 mg/L to between 300 and 450 mg/L. Similar to Layer 3, relatively small concentration changes are simulated within the 30-year simulation period.

For chloride (Figure 45), predicted concentrations in the Layer 1 wells initially fluctuate, then approach a constant value close to the CRW recharge concentration for chloride. In Layer 3 and 4 wells, chloride concentration trends are similar to simulated TDS concentrations, and increase over time. Sulfate trends in Layer 1 wells decrease then stabilize at the CRW recharge water concentration (Figure 46). Sulfate in Layer 3 and 4 wells generally increase, similar to the predicted TDS and chloride trends.

Figure 47 shows the predicted nitrate concentrations. The predicted nitrate trends differ from the TDS, chloride, and sulfate trends, due to the lower concentration of nitrate in CRW compared to local groundwater. For Layer 1 WRP 10 monitoring wells, nitrate concentrations decrease rapidly upon arrival of the higher-quality CRW. In downgradient wells in Layers 3 and 4, nitrate concentrations generally decrease as a result of downward migration of CRW recharge. In Layer 3, future ambient nitrate concentrations under baseline conditions are predicted to be near or above the primary MCL of 45 mg/L at selected well locations. Future nitrate concentrations in the three wells closest to the Palm Desert

property (9Q1, 22B2, and 14G1) decrease to below the primary MCL in Scenario 2 (and under most other scenarios; the rate of decline is commensurate with the rate of CRW recharge). In the most downgradient well (13R1) nitrate concentrations are relatively similar for all scenarios. Slightly higher nitrate concentrations are predicted over the 30-year period at 13R1 for larger CRW recharge scenarios. This trend is a result of shallow groundwater with elevated migrating downward at a slightly faster rate prior to the arrival of CRW at that location.

In the most upgradient well (16E1), nitrate concentrations in Layer 3 and 4 increase from around 25 and 10 mg/L, respectively, to around 50 and 40 mg/L after 30 years for the three scenarios with lower CRW recharge. The increasing trend is a result of 1) shallow/upgradient groundwater with elevated nitrate concentrations migrating downward/downgradient due to natural hydraulic gradients and localized pumping and 2) water quality benefits from CRW recharge not reaching Model Layer 3 at the location of 16E1. Nitrate concentrations for well 16E1 (in Layers 3 and 4) are predicted to decrease for Scenarios 4 and 6, illustrating the benefits of the larger CRW recharge rates in the WWR-SC on water quality in deeper aquifers.

Figure 48 shows the predicted chromium-6 concentrations in Layer 1, 3, and 4 wells. The predicted chromium-6 trends are different than the simulated TDS, chloride, and sulfate trends, due to the improved quality of chromium-6 in the recharged water as compared with native groundwater in deeper aquifers. For the Layer 1 wells, chromium-6 concentrations quickly drop to between 0 and 2 micrograms per liter (ug/L). In downgradient Layer 3 and 4 wells, chromium-6 concentrations steadily decrease over time except in distant downgradient well 13R1, indicating the recharged water at WRP 10 does not reach this well within 30 years.

In combination with positive local and regional groundwater storage changes predicted, the improvements in groundwater quality for nitrate and chromium-6 are a significant benefit of CRW recharge at the proposed recharge facilities.

#### Scenario 3 – 16,000 AFY Recharge at Palm Desert Property

Scenario 3 results are represented by the peach-colored concentration curves on Figures 44 through 48. As expected, for all constituents of concern and wells, predicted water quality trends for Scenario 3 closely resemble those for Scenario 2. Where concentration increases are predicted (TDS, chloride, and sulfate), the magnitude of increase for Scenario 3 is slightly less than for Scenario 2. Similarly, where concentration decreases are predicted (nitrate and chromium-6), the magnitude of decrease for Scenario 3 is slightly less than for Scenario 2.

#### Scenario 4 – 40,000 AFY Recharge via WWR-SC (Portola Avenue to Fred Waring Drive)

Scenario 4 results are represented by the dark blue-colored concentration curves on Figures 44 through 48. Scenario 4 simulated 40,000 AFY via the WWR-SC, representing the largest recharge rate scenario.

As shown on Figure 44, predicted TDS concentrations in the Layer 1 wells initially fluctuate, then approach a constant value close to the CRW recharge concentration for TDS (742 mg/L). In Layer 3 wells except distant downgradient well 13R1, TDS concentrations increase over time from around 200-300 to close to the recharge concentration indicating that those wells eventually receive almost 100% recharge water. In Layer 4 wells, TDS concentrations increase over time from around 150-300 to between approximately 300-650 mg/L.

As shown on Figures 45 and 46, chloride and sulfate trends in Layer 1, 3, and 4 wells are similar to predicted TDS trends.

The predicted nitrate trends for Scenario 4 (Figure 47) are similar to the Scenario 2 and 3 trends, except the magnitude of change/departure from Scenario 1 trends are larger, due to the larger recharge rate. For the Layer 1 wells, nitrate rapidly decreases to nearly 0 mg/L as the high-quality recharge water arrives at these wells. In the upgradient well 16E1 in Layers 3 and 4, nitrate concentrations increase from around 25 and 10 mg/L, respectively, to around 50 and 35 mg/L after 30 years. In downgradient wells in Layers 3 and 4, nitrate concentrations decrease significantly, indicating the arrival of large percentages of recharged CRW.

As shown on Figure 48, chromium-6 concentrations in Layer 1 wells quickly drop to less than 2 ug/L. In downgradient Layer 3 and 4 wells, chromium-6 concentrations steadily decrease over time except in distant downgradient well 13R1, indicating the recharged CRW does not reach this well within 30 years.

#### Scenario 5 – 15,000 AFY Recharge via WWR-SC (Cook Street to Fred Waring Drive)

Scenario 5 results are represented by the light blue-colored concentration curves on Figures 44 through 48. Scenario 5 simulates 15,000 AFY in the eastern portion of the WWR-SC. For all constituents of concern, the predicted concentration trends for Scenario 5 are similar to Scenarios 2 through 4. The magnitudes and patterns of water quality trends predicted for Scenario 5 are more similar to those for Scenarios 2 and 3 than Scenario 4. This relationship indicates that the influence of the annual CRW recharge rate on local groundwater quality is greater than the recharge location (i.e., Palm Desert property vs. WWR-SC).

#### Scenario 6 – 36,000 AFY Recharge (16,000 AFY at Palm Desert Property and 20,000 AFY via WWR-SC (Portola Avenue to Fred Waring Drive))

Scenario 4 results are represented by the purple-colored concentration curves on Figures 44 through 48. Scenario 6 simulates 36,000 AFY of CRW recharge apportioned to the Palm Desert property and the WWR-SC. For all constituents of concern, the predicted concentration trends for Scenario 6 are most similar to Scenario 4 (40,000 AFY via River), indicating that the total recharge rate is a significant factor in water quality responses in the deeper aquifers. The location of recharge and the rate-to-area ratio is also an important factor in water quality response. The difference in water quality trends between Scenarios 6 and 4 is most notable at wells 9Q1 and 16E1 in Layer 3. At these upgradient well locations, the departure from Scenario 1 concentration trends is greater for Scenario 4, because CRW

recharge is concentrated in the WWR-SC, resulting in larger water quality changes to the west. While smaller, concentration differences between Scenario 6 and 4 at 9Q1 and 16E1 in Model Layer 4 are also observed.

## **5.6. ASSUMPTIONS AND LIMITATIONS**

The groundwater flow and water quality models incorporate several simplifying assumptions. Flow model assumptions include quasi-steady-state groundwater flow, although limited transient flow simulations were performed to assess the time required to reach flow equilibrium. The models assumed constant future rates of pumping and recharge. Additionally, the water quality models assume uniform values of dispersivity and aquifer porosity, contain assumptions regarding chemical concentration initial and boundary conditions, do not account for nitrate transformations, and do not account for mineral dissolution within the model area.

These input parameter assumptions yield some uncertainty in model predictions. Accordingly, simulated groundwater elevations, flowrates and directions, and predicted future chemical concentrations should be considered relative estimates of potential conditions.

Finally, the conceptual scenarios on recharge rates and locations are intended to bracket potential future operations. If appropriate, different recharge (and well pumping rates) can be simulated during future phases of work.

## 6. CONCLUSIONS

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### 6.1. CONCLUSIONS

Based on the hydrogeologic characterization and modeling results, engineering design and cost analysis, and environmental permit assessment, the following conclusions can be made:

#### 6.1.1. Site Recharge Capacity

- Results of short-term infiltration testing of existing WRP10 percolation basins and operational performance at the TEL-GRF indicate that long-term infiltration rates of 1.75 feet/day can be reasonably achieved at the Palm Desert property and in the WWR-SC south/southwest of the property. This infiltration rate would accommodate estimated maximum CRW rates delivered via the MVP (up to 40,000 AFY) based on recharge facility concepts developed for the Palm Desert property and WWR-SC between Portola Avenue and Fred Waring Drive.
- The aquifer has the physical capacity to accept proposed recharge rates without excessive water table mounding. Maximum predicted water table mounding for the largest recharge rates (20,000 AFY at the Palm Desert property, 40,000 AFY via the WWR-SC, and 36,000 AFY combined at Palm Desert property and via the WWR-SC) is on the order of 100 feet, or approximately 100-110 feet-bgs. This indicates the aquifer has the physical capacity to accept the preliminarily estimated maximum CRW delivery capacity of 40,000 AFY via the MVP.

#### 6.1.2. Recharge Concept Design and Cost Analysis

- Preliminary concepts for reconfiguring the existing percolation ponds at the CVWD Palm Desert property involve removing internal levees to increase the recharge area within the same basin footprint. The primary differences between the two concepts evaluated are the basin orientation and whether the existing back feed reservoir is re-purposed as a dual CRW storage and recharge basin.
- The preliminary concepts for the WWR-SC include use of sugar berms, flashboard dams, or inflatable rubber dams to partition the reach into basins.
- Recharge capacities of recharge basin system concepts on the Palm Desert property range from 16,000 to 20,000 AFY. Recharge capacities of recharge basin system concepts in the WWR-SC range from approximately 42,000 to 54,000 AFY.
- Based on an analysis of the costs, operations, and maintenance of proposed concepts, Concept 2 for the basin system at the Palm Desert property (larger recharge area including the back feed reservoir) is the preferred concept.

- Significantly lower initial construction costs make the use of sugar berms in the WWR-SC the preferred choice for a WWR-SC recharge facility.

### **6.1.3. Environmental Permit and Mitigation Requirements**

- A preliminary assessment of applicable environmental laws, regulatory authorities, permit requirements, and potential measures to mitigate loss of sensitive habitat and channel functionality for WWR-SC recharge concepts was completed.
- Preliminary findings indicate that requirements should not significantly increase the cost or delay the schedule of constructing and operating recharge facilities on the Palm Desert property or in the WWR-SC.
- Additional discussions with applicable regulatory agencies and submittal of permit applications are needed to better understand the required project-specific mitigation measures and associated implications to costs and schedule.

### **6.1.4. Groundwater Storage Benefits and Recharge Flow and Recovery**

- Implementation of a CRW recharge project in the Palm Desert area will provide local and regional water supply benefits in the WWR AOB and EWR AOB.
  - Total groundwater basin storage and yield will be increased by the amount of water recharged. Assuming consistent recharge operations, hydraulic gradient and groundwater flow responses are expected to equilibrate within approximately 5 years.
  - Predicted groundwater level increases as compared to baseline scenarios within the Study Area are commensurate with the rate of future CRW recharge. With 36,000 AFY of CRW recharge (Scenario 6), groundwater storage gains are estimated to be about 29,000 AF.
  - Regional benefits will be directed to both upgradient Basin areas in the form of reduced subsurface inflow to the model area and also downgradient Basin areas in the form of increased subsurface outflow from the model area. Upon establishment of new equilibrium conditions following 36,000 AFY of CRW recharge (Scenario 6), subsurface flows from upgradient areas into the Study Area are expected to decline by about 21,000 AFY, while subsurface flows from the Study Area to downgradient areas of the Basin (i.e., EWR AOB) would increase by about 11,000 AFY. The combined 32,000 AFY benefit to areas outside of the Study Area is equal to the net increase in Study Area recharge from current baseline conditions.
- Simulated groundwater flowpaths for all recharge scenarios indicate that recharge water is captured by some of the upgradient and downgradient production wells in both shallow and deep aquifers. A portion of the recharged water flows vertically to

deeper aquifers and deep production wells, and some of the recharged water flows beyond the local model area to downgradient portions of Coachella Valley.

- Travel times from proposed recharge sites to downgradient production wells within the Study Area range from less than 1 year up to several decades, depending on the distance between the recharge areas and wells.

#### **6.1.5. Groundwater Quality**

- Groundwater quality in the Study area varies considerably with depth. TDS, chloride, sulfate, and nitrate concentrations decrease considerably with depth, while chromium-6 concentrations increase slightly with depth.
- Groundwater quality changes in both the shallow and deep aquifer zones will occur as a result of mixing of recharged CRW and ambient groundwater.
- For all five recharge scenarios evaluated, CRW recharge will provide water quality benefits with respect to nitrate and chromium-6.
- Increases in TDS, chloride, and sulfate concentrations are expected in the deeper aquifer zones as overlying shallow groundwater and CRW migrates downward.
- The depth of groundwater quality changes from CRW recharge are directly correlated with the location, absolute rates, and rate-to-area ratio of CRW recharge. Higher recharge rates and higher rate-to-area ratios result in deeper migration of CRW and mixing with groundwater in deeper aquifer zones.

## **6.2. RECOMMENDATIONS**

Key findings from this study are based on evaluation of available hydrogeologic and engineering information for the project site and the Study Area. Should CVWD decide to move forward with implementation of a groundwater replenishment project at either the CVWD Palm Desert property or WWR-SC, the following additional data collection and evaluation tasks are recommended:

- Long-term infiltration tests (up to 30 days) should be conducted in the existing WRP 10 percolation ponds and in the WWR-SC to confirm surface infiltration rates. Infiltration tests would involve the recharge of CWR or secondary effluent in existing WRP 10 percolation ponds and potable water or CRW in the WWR-SC. Prior to testing, the upper 3 feet of sediments in the WRP 10 ponds should be excavated to remove accumulated fine-grained material beneath the bottom of the ponds. Both reaches of the WWR-SC (between Portola Ave. and Cook St. and between Cook St. and Fred Waring Dr.) should be excavated to create a temporary test basin with sufficient area to minimize the effect of lateral spreading of recharge water (e.g., at least 25 feet x 25 feet). During the test, the discharge rate and ponded water level should be measured and converted to a vertical infiltration rate. For testing at the

existing WRP 10 percolation ponds, water levels in WRP 10 MW2 should be monitored to confirm vadose zone travel times and adjust calculated vertical infiltration rates to account for lateral spreading inferred from test results.

- An evaluation of historical storm discharge in the WWR-SC reach between Portola Ave. and Fred Waring Dr. is recommended to further evaluate the long-term maintenance requirements of the sugar berm concepts (i.e., the frequency and associated cost of re-establishing the sugar berms following washouts by large storm events).
- The MVP is buried 20 feet below the bottom of WWR-SC south of the CVWD Palm Desert property for scour protection. The MVP is constructed of cathodic-protected concrete steel pipe. If a recharge facility is constructed in the WWR-SC, the MVP will be subject to near-continuous saturated soil conditions and more corrosion potential. Accordingly, a cost evaluation is recommended to determine whether maintaining cathodic protection into the future factors significantly into long-term project operational costs.
- An evaluation of the existing MVP pump station is recommended to better determine the feasibility of expanding the pump station to deliver CRW for recharge in the WWR-SC.

## 7. REFERENCES

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Coachella Valley Conservation Commission (2007) Final Recirculated Coachella Valley Multiple Species Habitat Conservation Plan. September 2007.

CVWD (2012) Resolution No. 2012-16, A Resolution of the Board of Directors of the Coachella Valley Water District, Certifying the Subsequent Program Environmental Impacts Report for the 2010 Coachella Valley Water Management Plan Update; Adopting Environmental Findings Pursuant to the California Environmental Quality Act; and Adopting a Statement of Overriding Considerations and a Mitigation Monitoring and Reporting Program.

Fogg, G.E., G.T. O'Neill, E.M. LaBolle, and D.J. Ringel (2000) Groundwater Flow Model of Coachella Valley, California: An Overview.

Gibeault, V. A., M. Yates, J. Meyer, and M. Leonard (1999) Movement of Nitrogen Fertilizer in a Turfgrass System. California Turfgrass Culture. Volume 48, Nos. 1 & 2, 1998. University of California Cooperative Extension.

U.C. Davis (2012) Addressing Nitrate in California's Drinking Water. Prepared for the State Water Resources Control Board Report to the Legislature. Principal Investigators Thomas Harter and Jay R. Lund - Centre for Watershed sciences.

Wu, L. , R. Green, M. V. Yates, P. Pacheco, and G. Klein (2007) Nitrate Leaching in Overseeded Bermudagrass Fairways. Crop Science Vol. 47, November-December 2007.

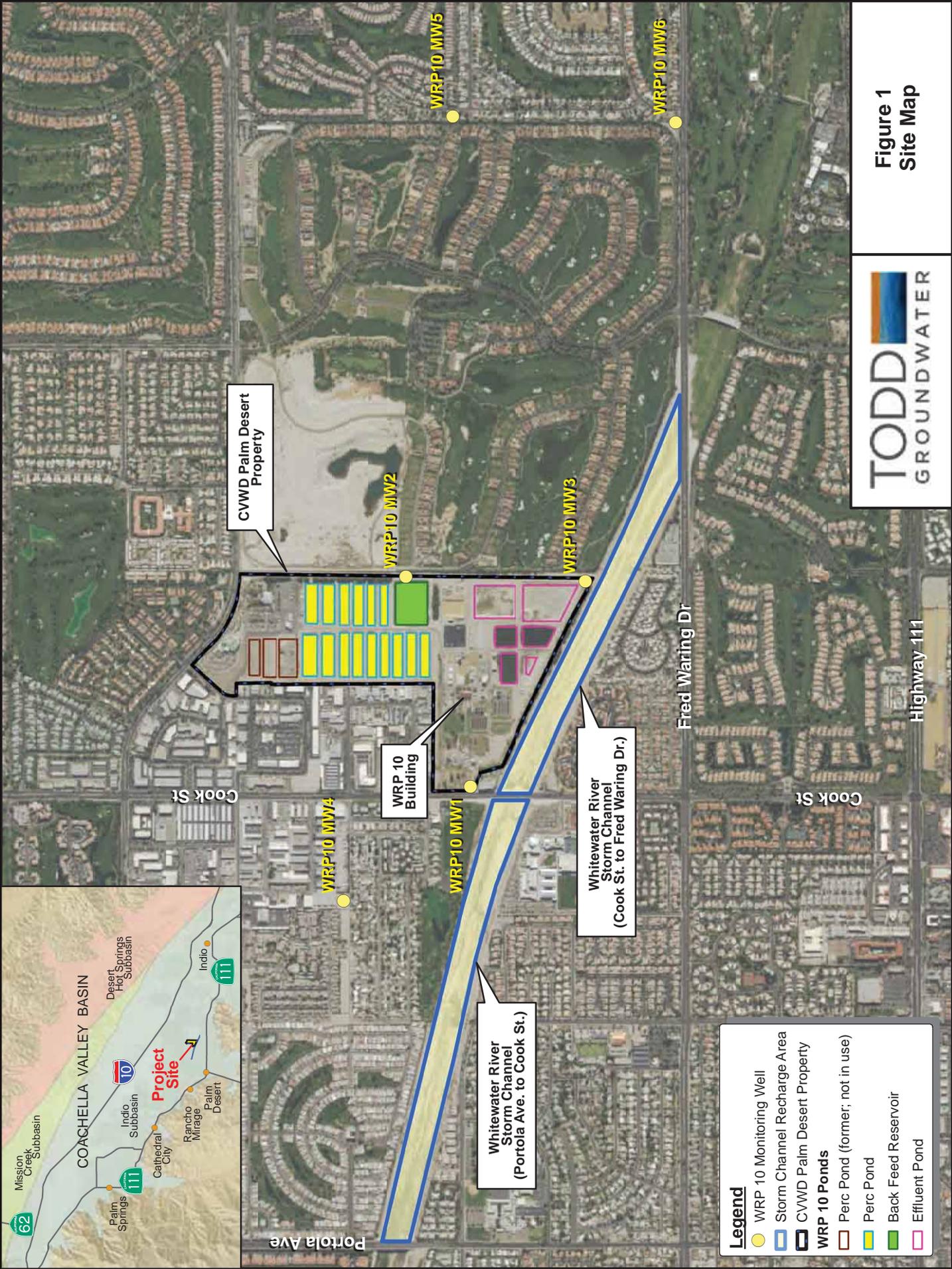
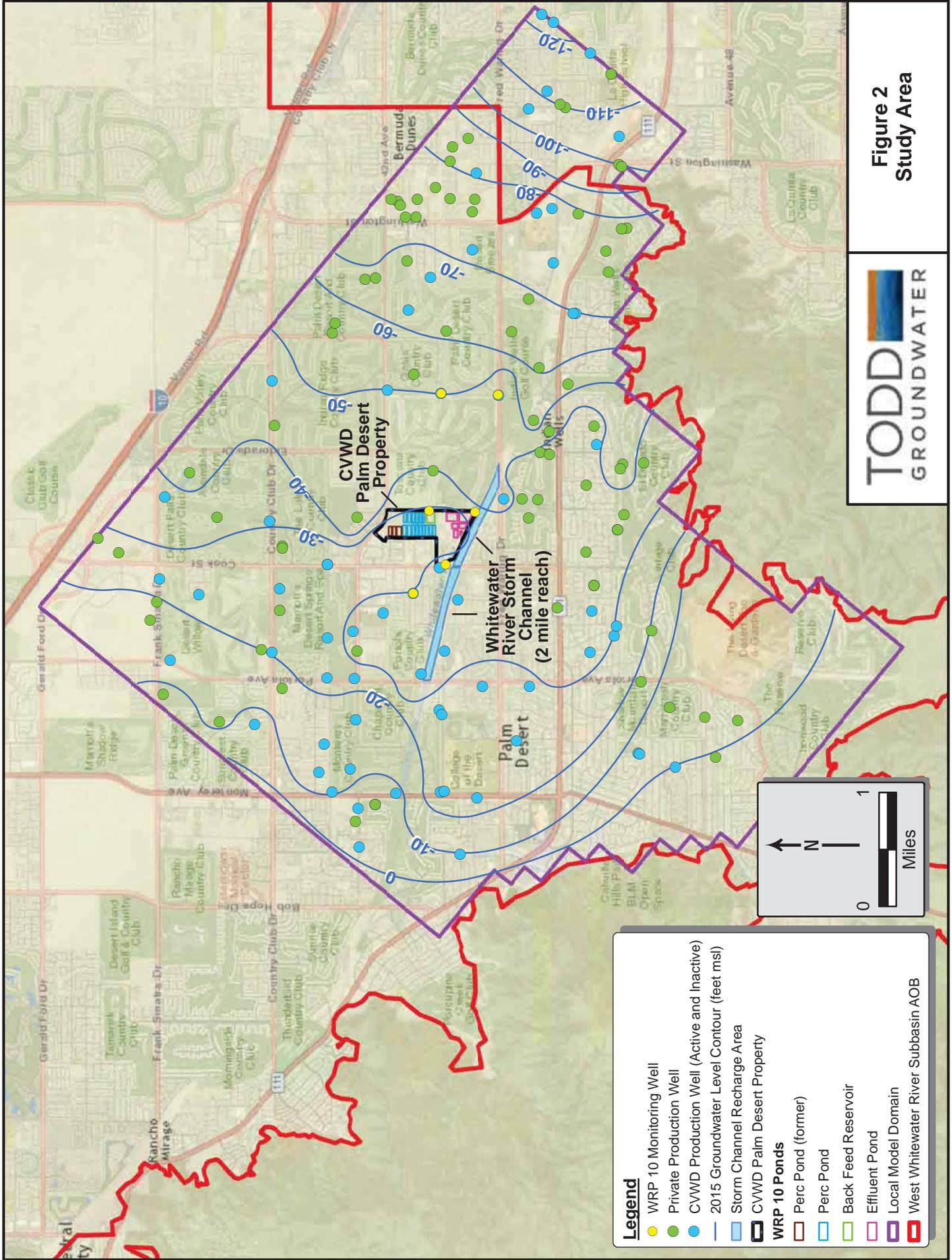
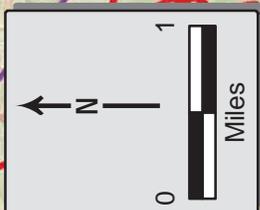


Figure 1  
Site Map

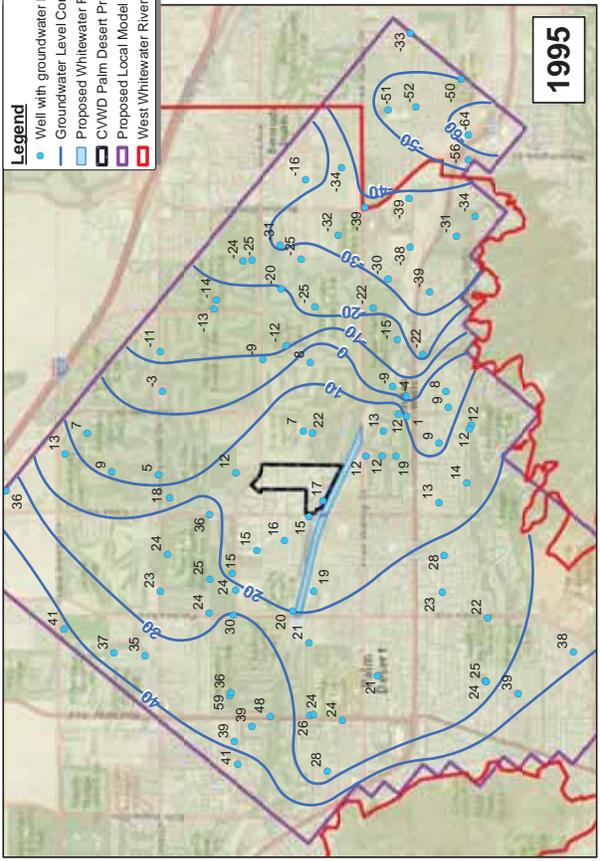
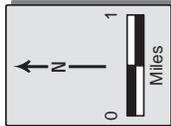


- Legend**
- WRP 10 Monitoring Well
  - Private Production Well
  - CVWD Production Well (Active and Inactive)
  - 2015 Groundwater Level Contour (feet msl)
  - Storm Channel Recharge Area
  - CVWD Palm Desert Property
- WRP 10 Ponds**
- Perc Pond (former)
  - Perc Pond
  - Back Feed Reservoir
  - Effluent Pond
  - Local Model Domain
  - West Whitewater River Subbasin AOB

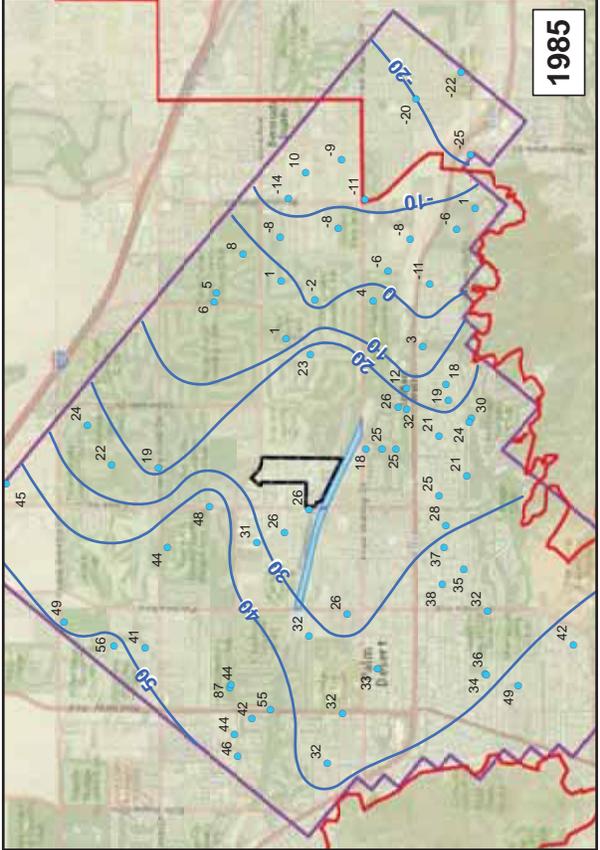


**Figure 2**  
**Study Area**

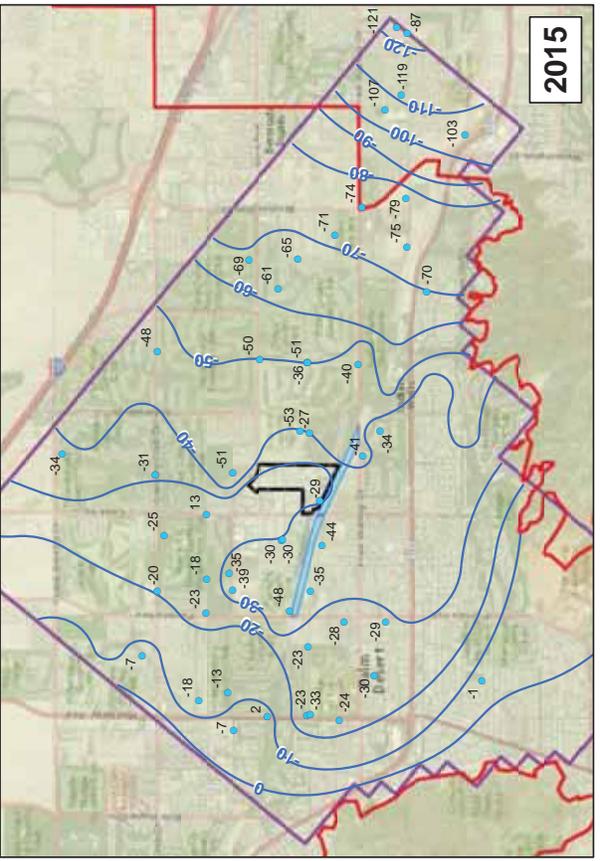
- Legend**
- Well with groundwater level in feet msl
  - Groundwater Level Contour (feet msl)
  - Proposed Whitewater River-Storm Channel Recharge Area
  - CWWD Palm Desert Property
  - Proposed Local Model Domain
  - West Whitewater River Subbasin Area of Benefit



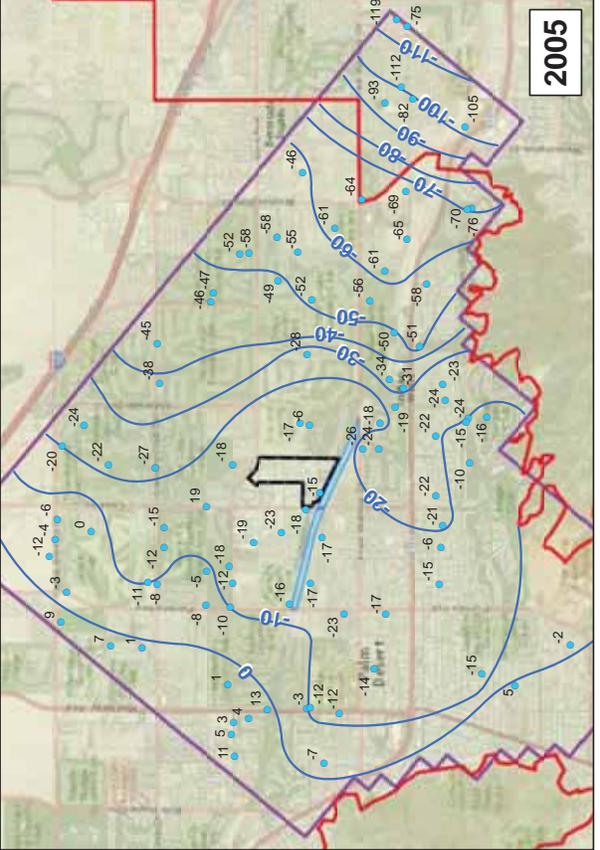
1995



1985



2015



2005



Figure 3  
Groundwater Levels  
1985, 1995, 2005, 2015

- Legend**
- WWRF Storm Channel Recharge Area
  - West Whitewater River Subbasin AOB
  - Local Model Domain
  - CWWD Palm Desert Property
- Groundwater Level Change (feet)**
- 0 to 10
  - 10 to 0
  - 20 to -10
  - 30 to -20
  - 40 to -30
  - 50 to -40
  - 60 to -50
  - 70 to -60
  - 80 to -70
  - 90 to -80
  - 100 to -90

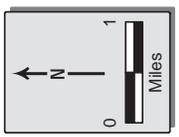
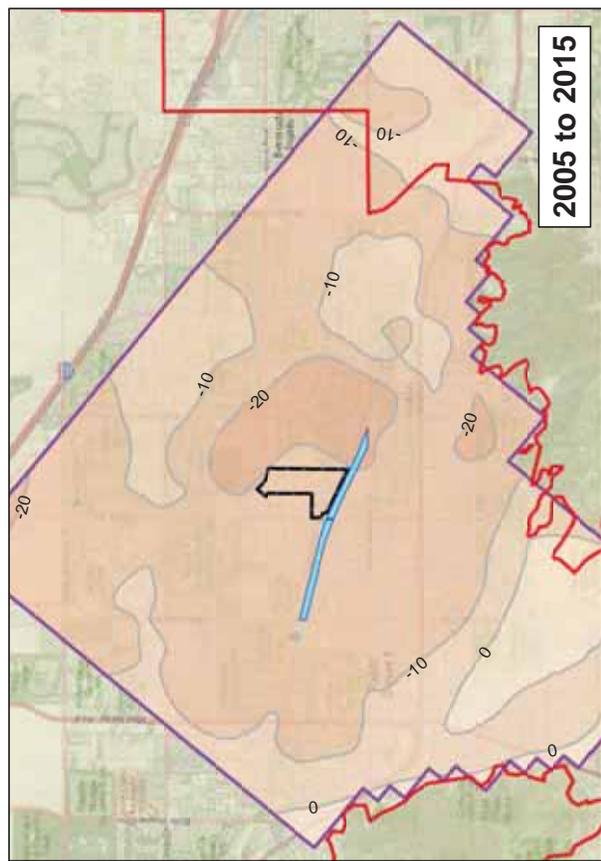
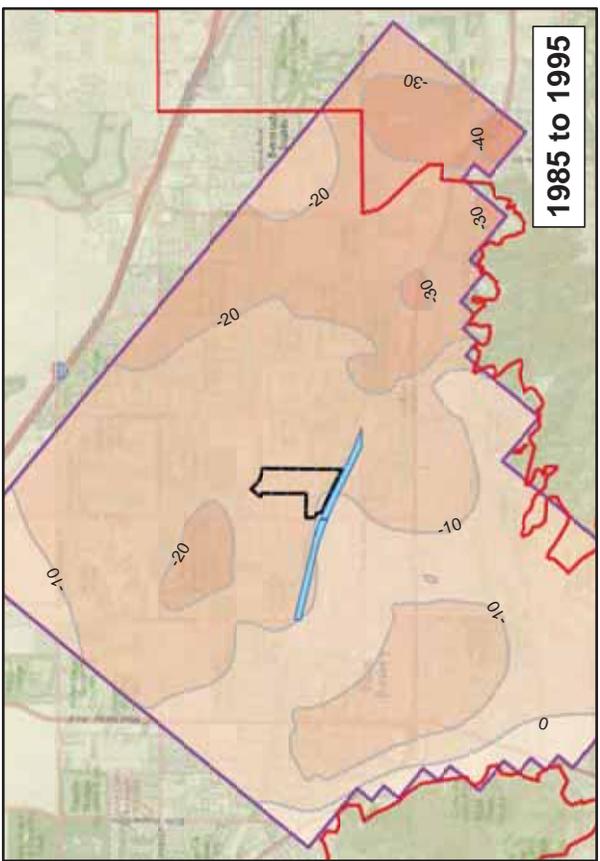
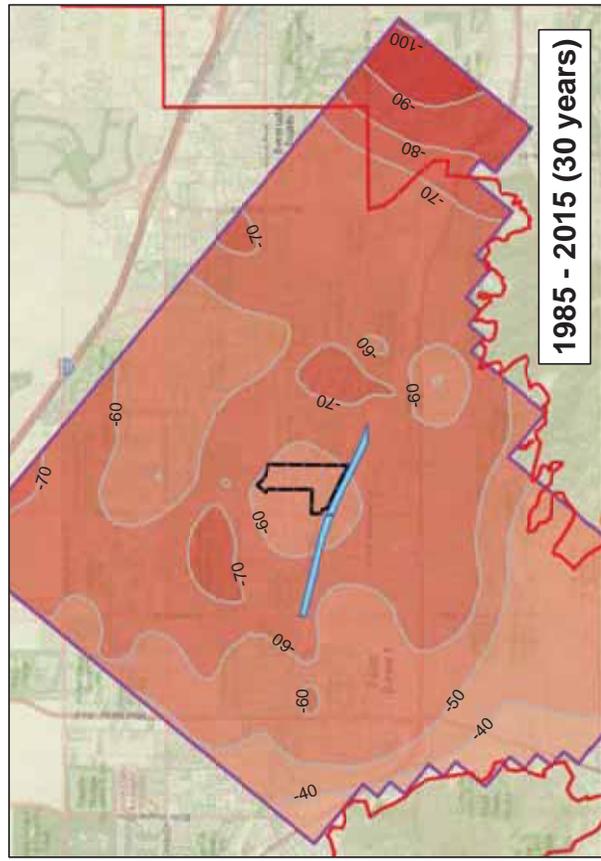
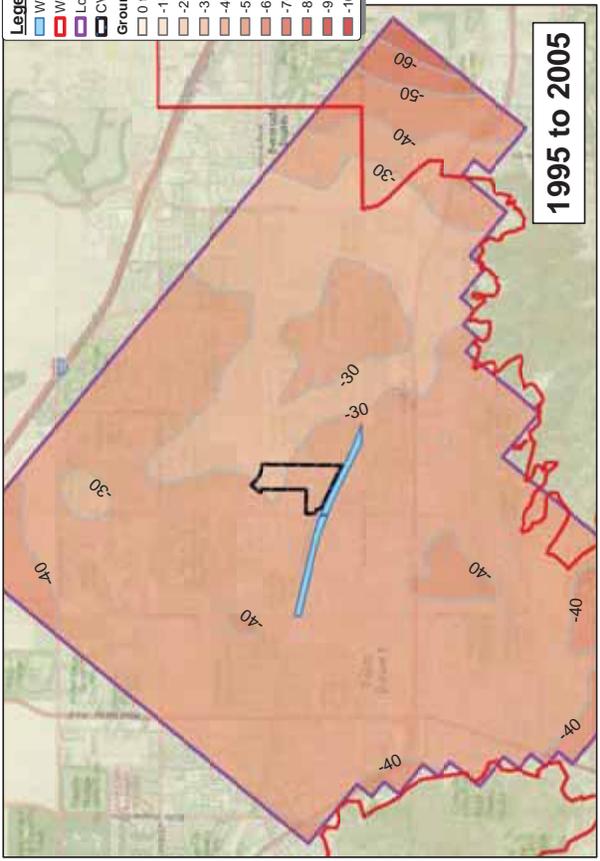
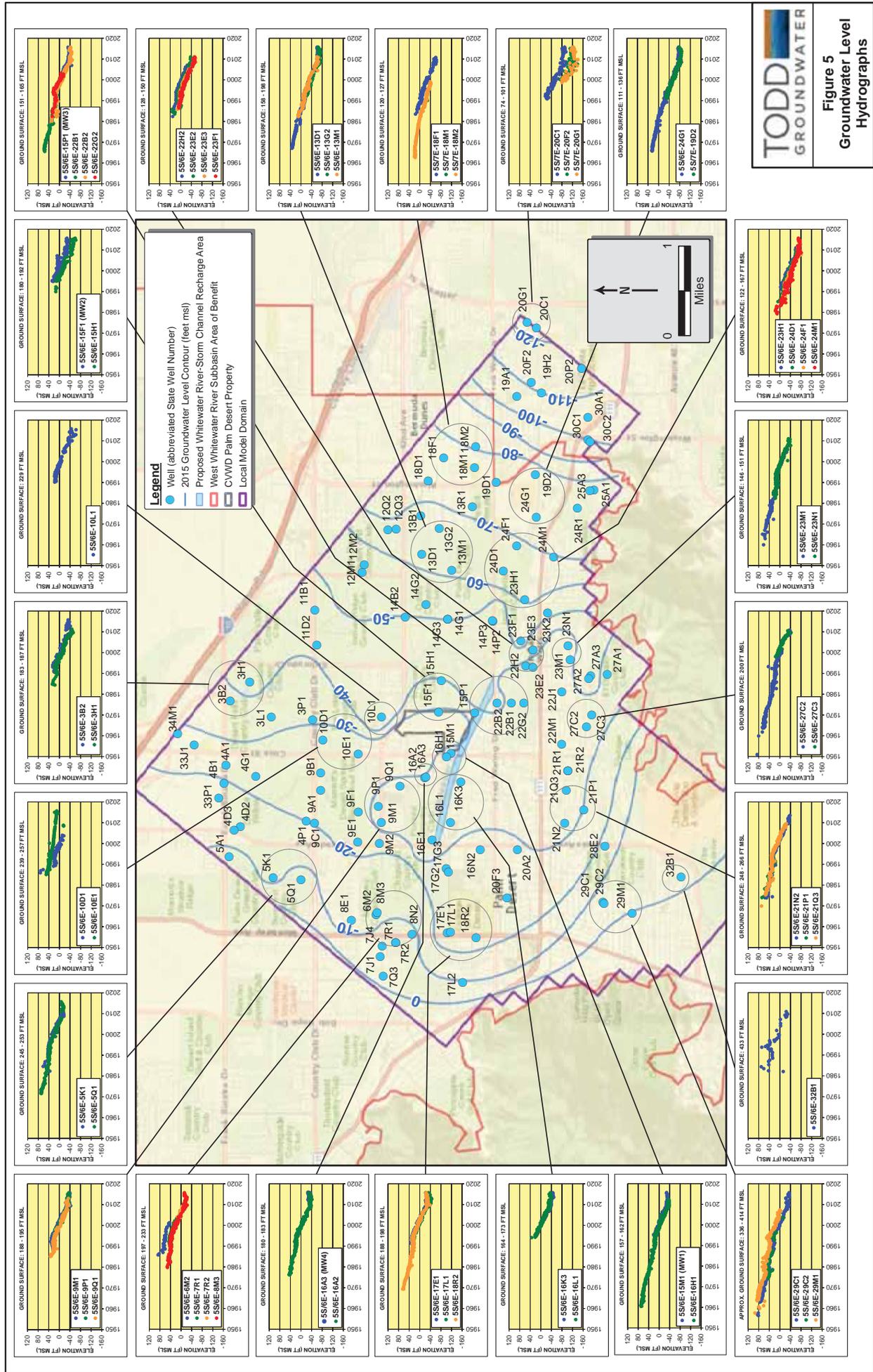
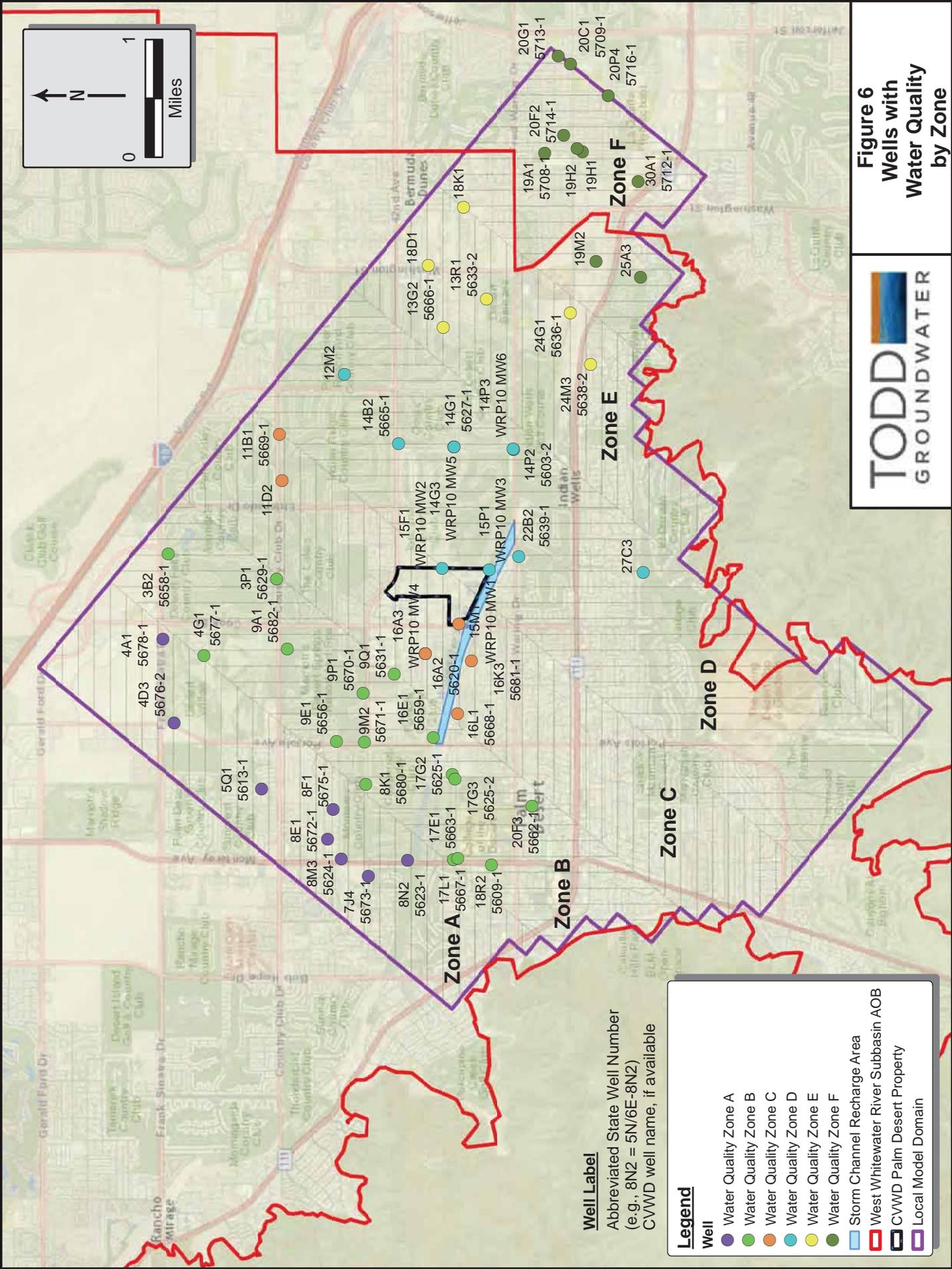


Figure 4  
Groundwater Level  
Change Maps







**Figure 6**  
**Wells with**  
**Water Quality**  
**by Zone**



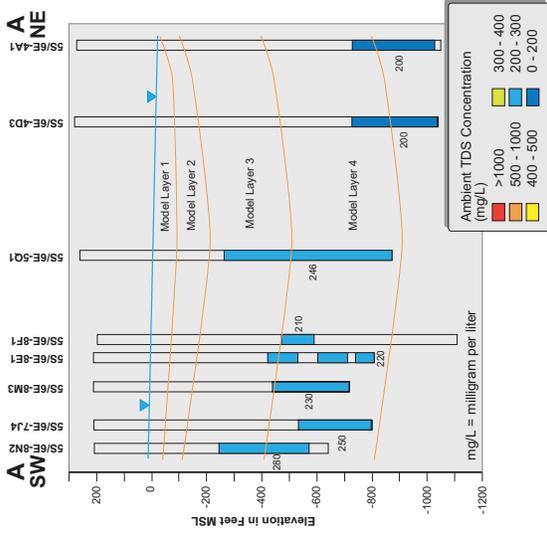
**Well Label**

Abbreviated State Well Number  
 (e.g., 8N2 = 5N/6E-8N2)  
 CVWD well name, if available

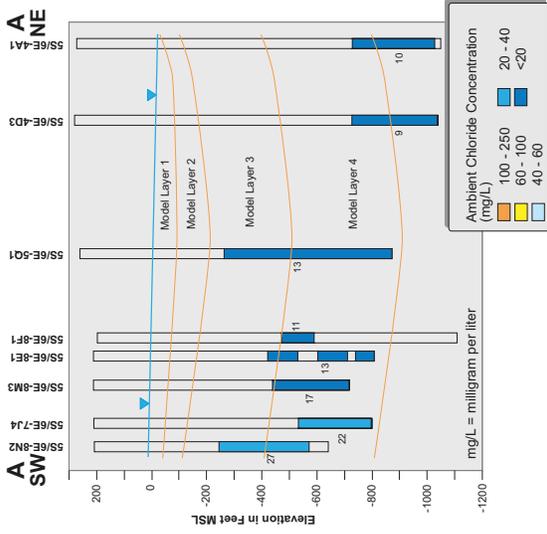
**Legend**

- Water Quality Zone A
- Water Quality Zone B
- Water Quality Zone C
- Water Quality Zone D
- Water Quality Zone E
- Water Quality Zone F
- Storm Channel Recharge Area
- West Whitewater River Subbasin AOB
- CVWD Palm Desert Property
- Local Model Domain

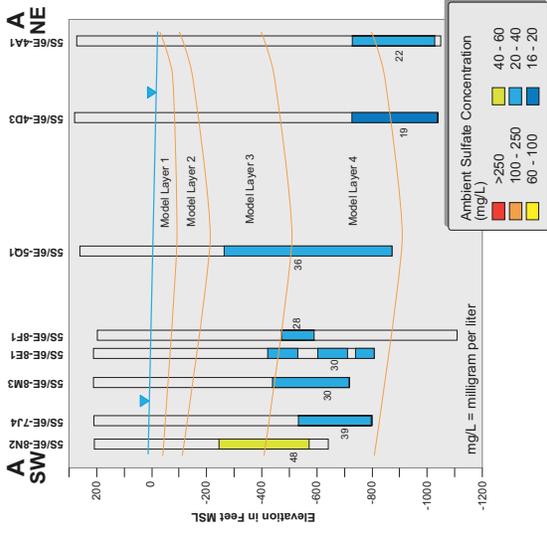
### TDS



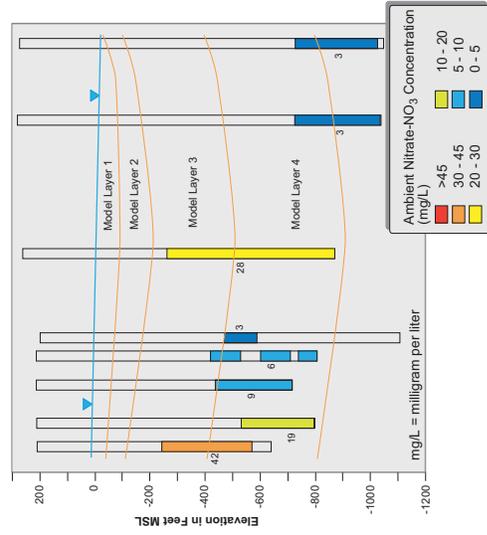
### Chloride



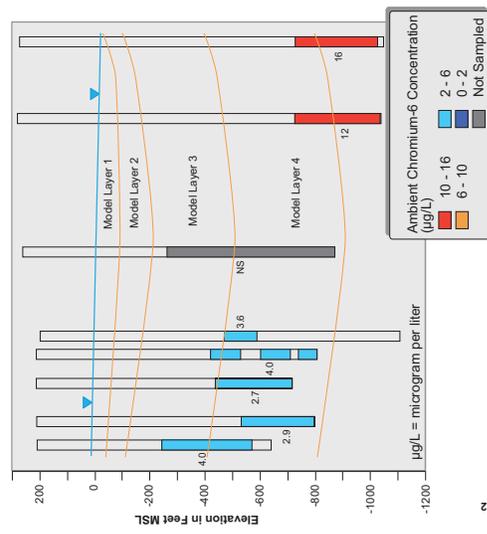
### Sulfate



### Nitrate-NO<sub>3</sub>



### Chromium-6



### Fluoride

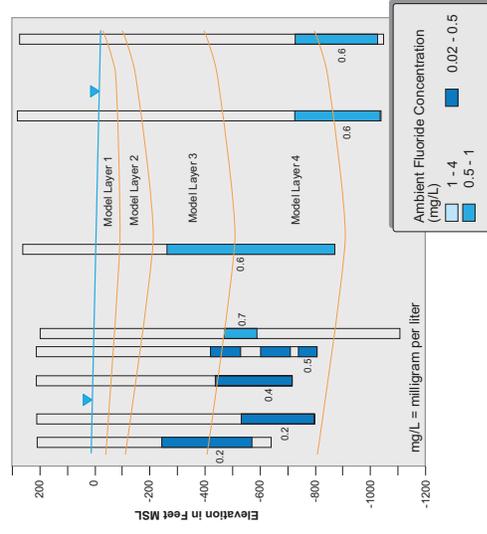
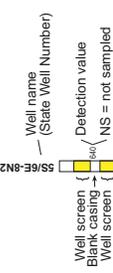
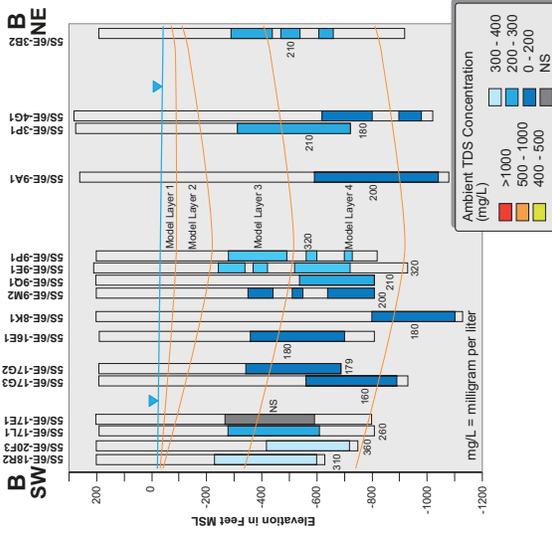


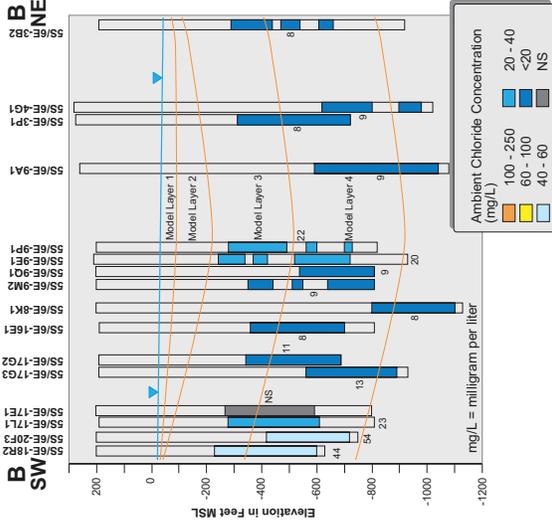
Figure 7  
Cross Section A - A



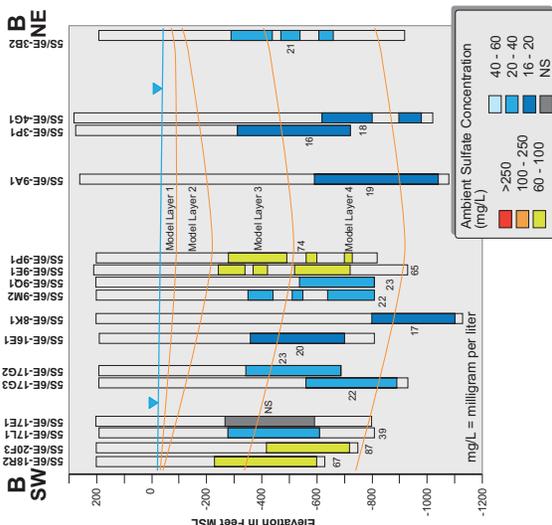
### TDS



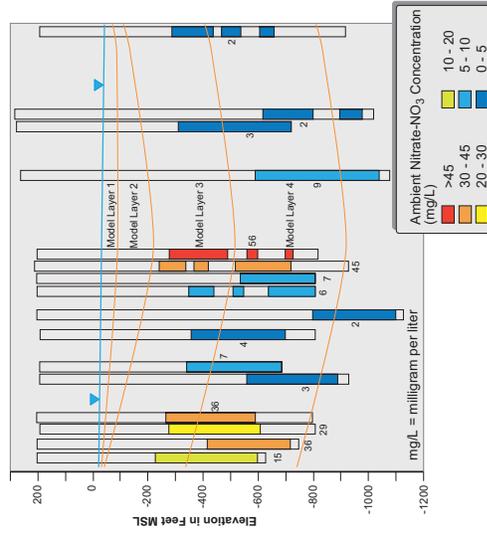
### Chloride



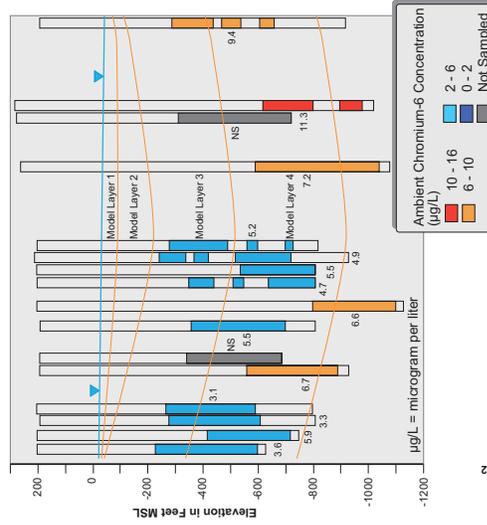
### Sulfate



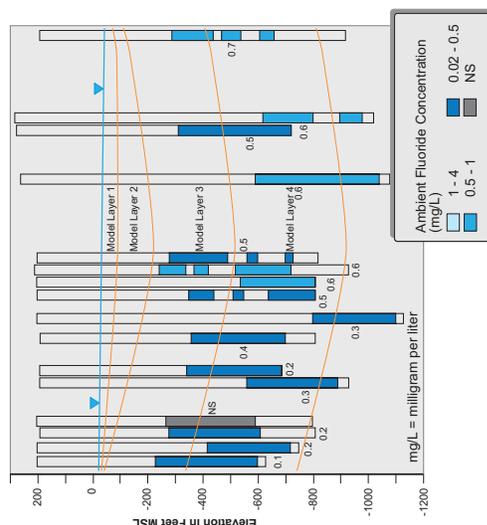
### Nitrate-NO<sub>3</sub>



### Chromium-6



### Fluoride



Well name (State Well Number)  
 Well screen {  
 Blank casing {  
 Well screen {

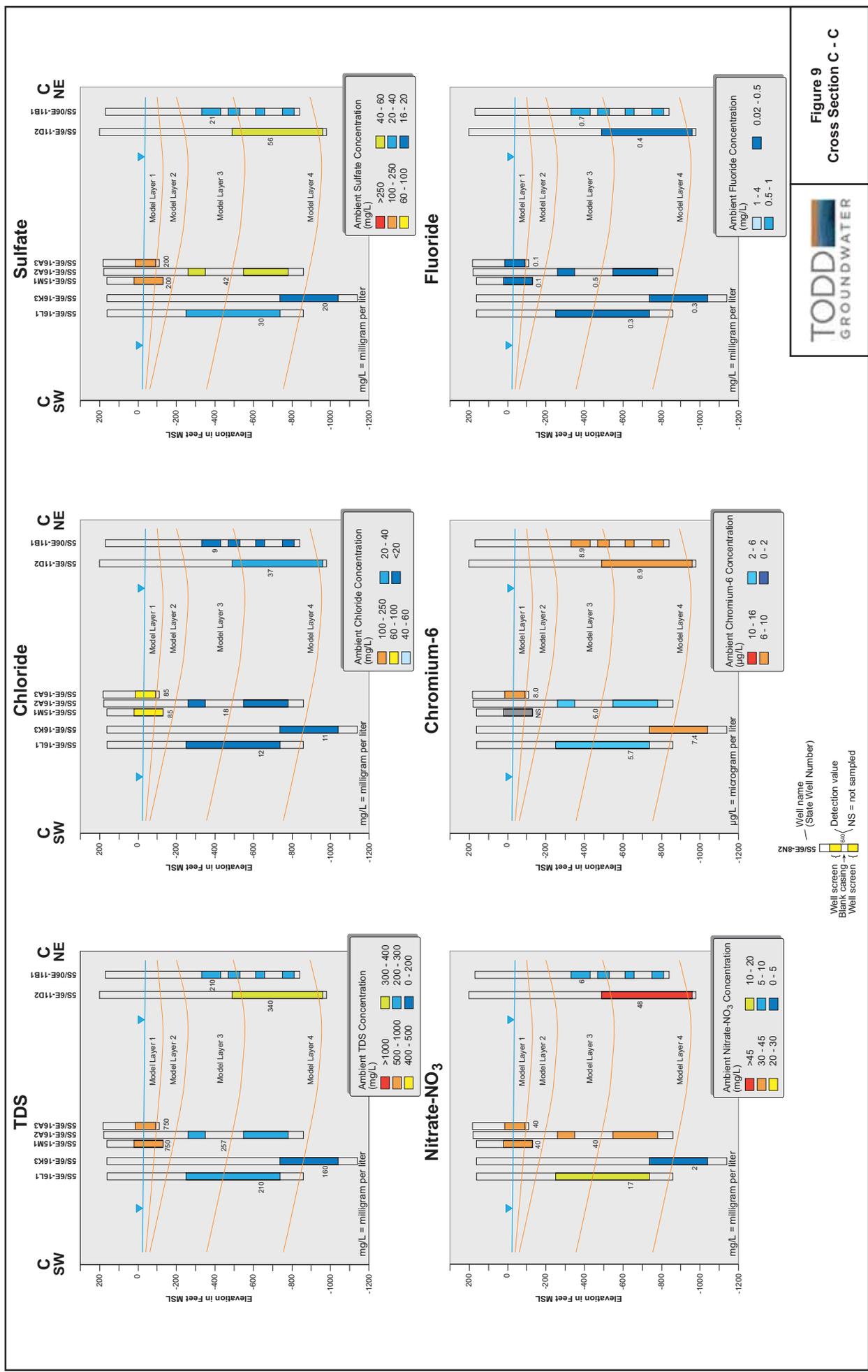


Figure 9  
Cross Section C - C



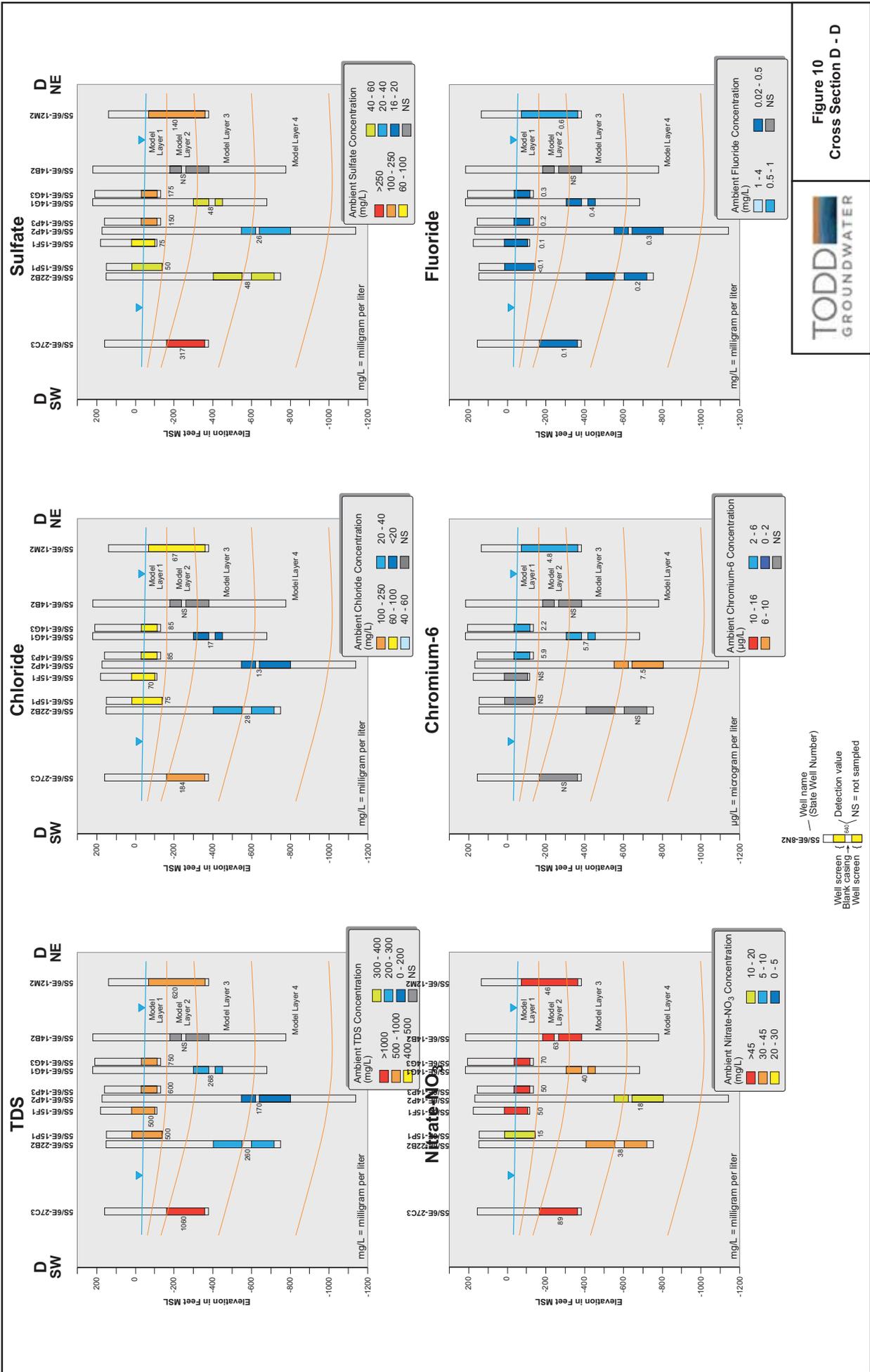
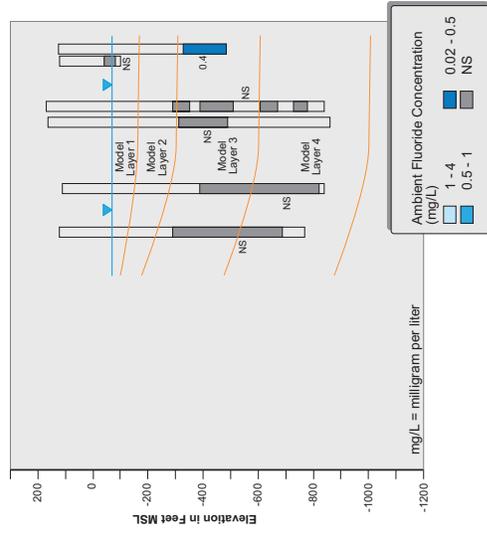
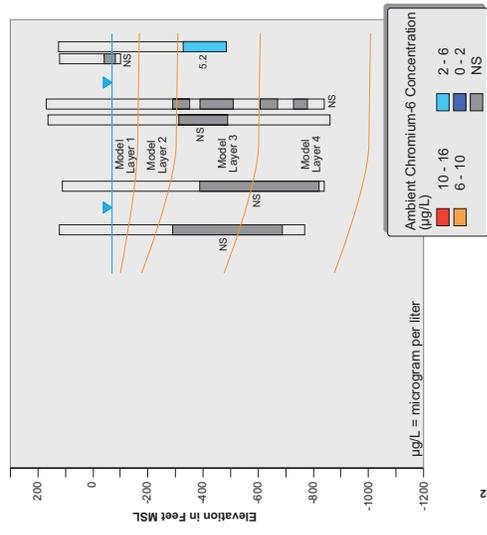
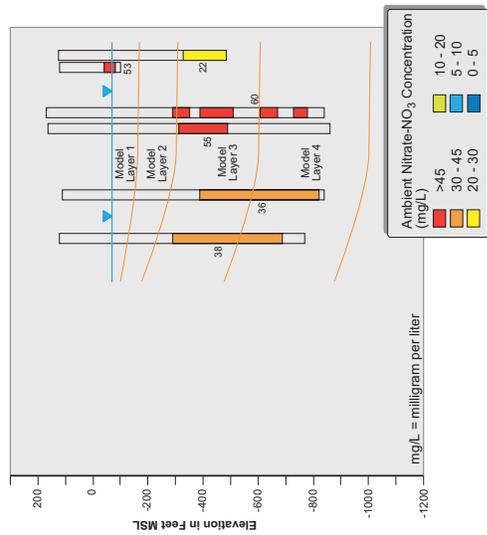
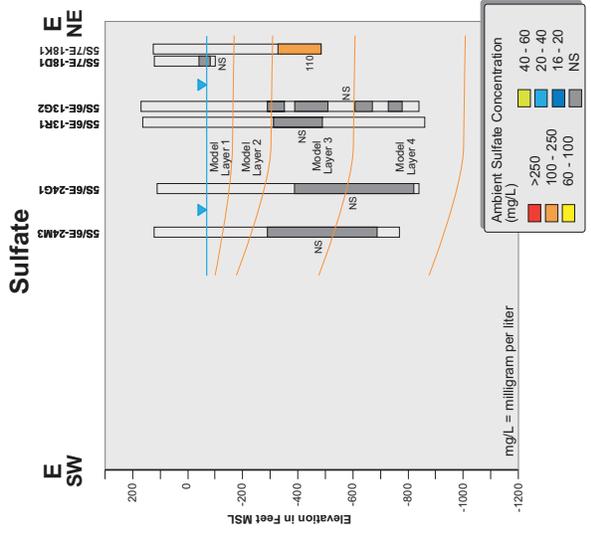
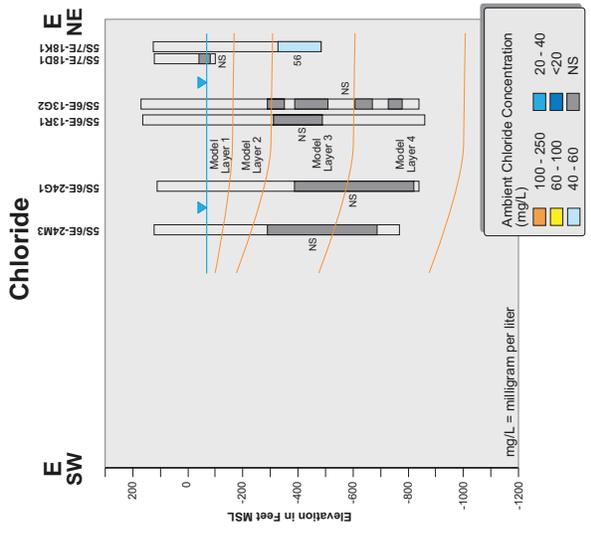
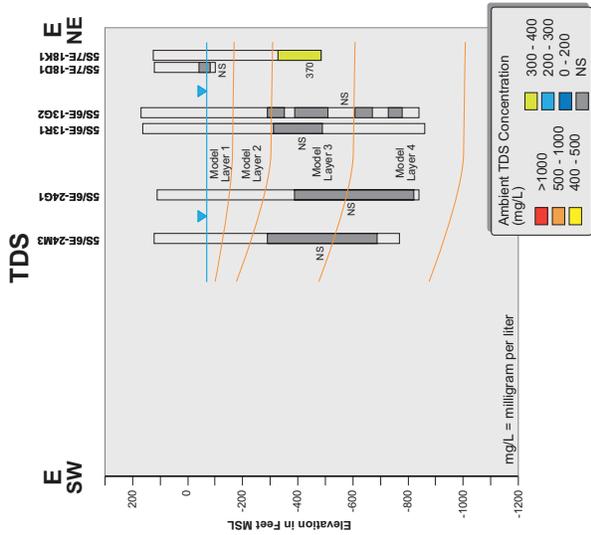


Figure 10  
Cross Section D - D





Well name (State Well Number)  
 Well screen { }  
 Blank casing { }  
 Well screen { }  
 NS = not sampled

Figure 11  
 Cross Section E - E



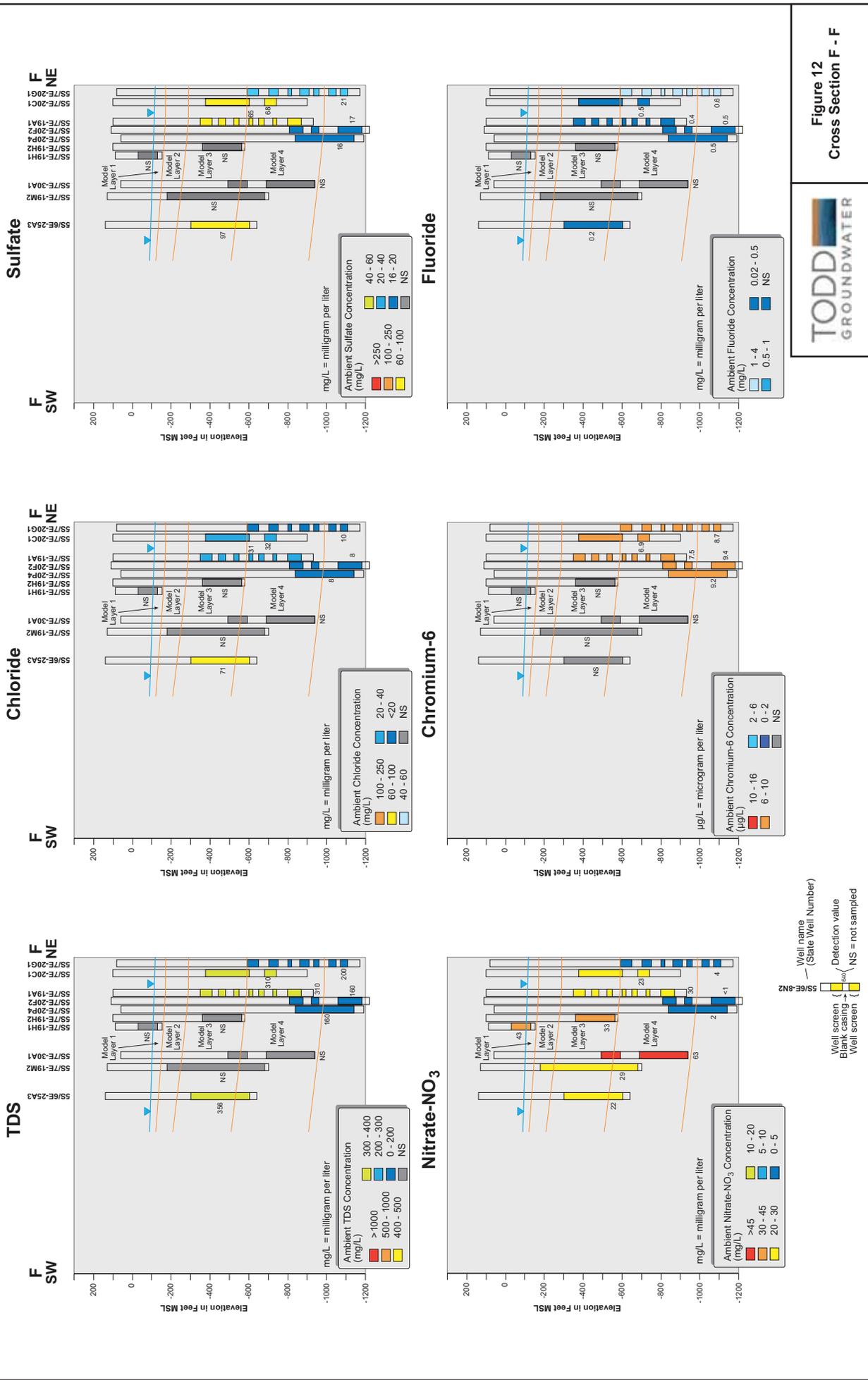
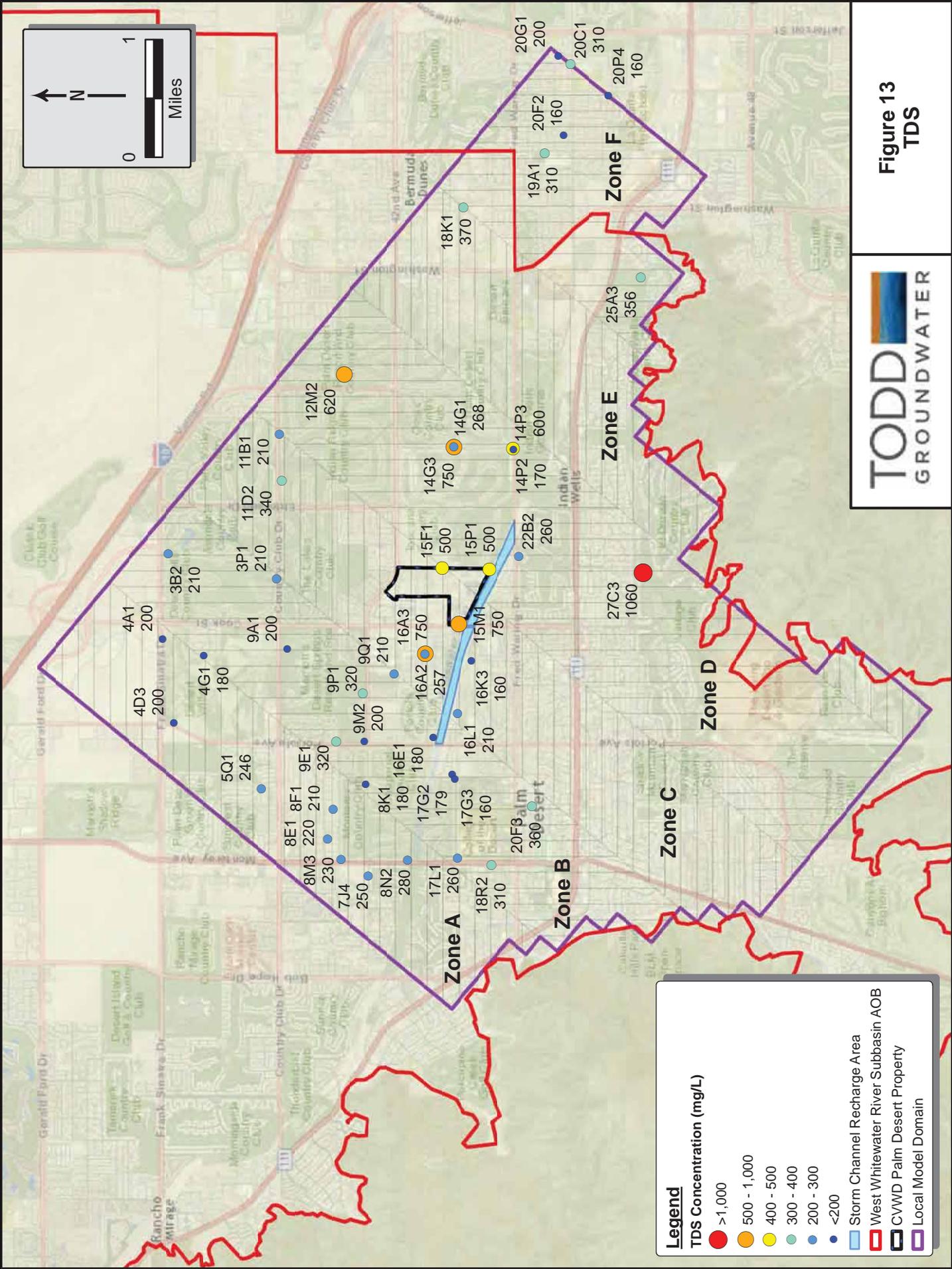


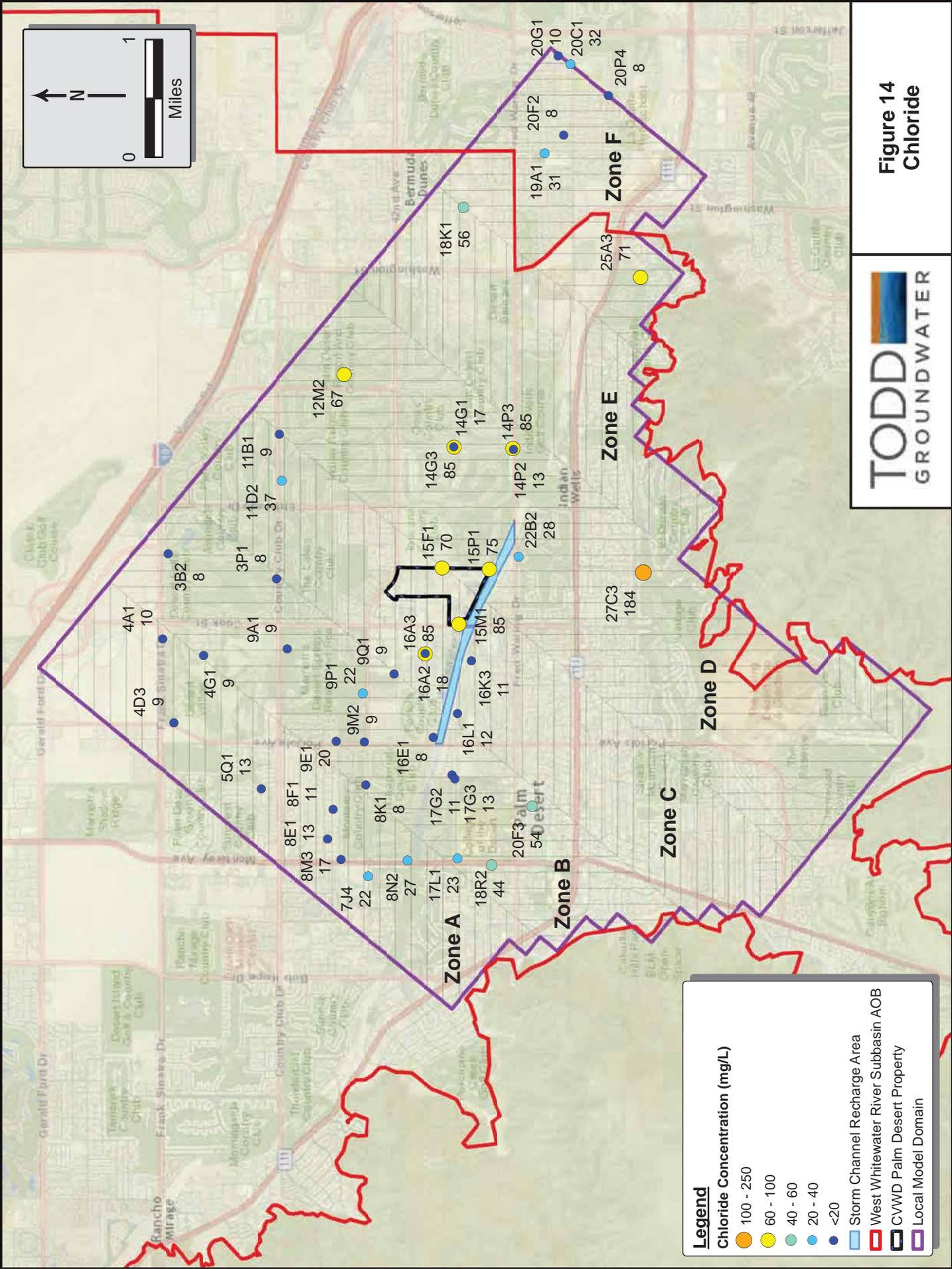
Figure 12  
Cross Section F - F





**Figure 13**  
TDS





**Figure 14**  
**Chloride**

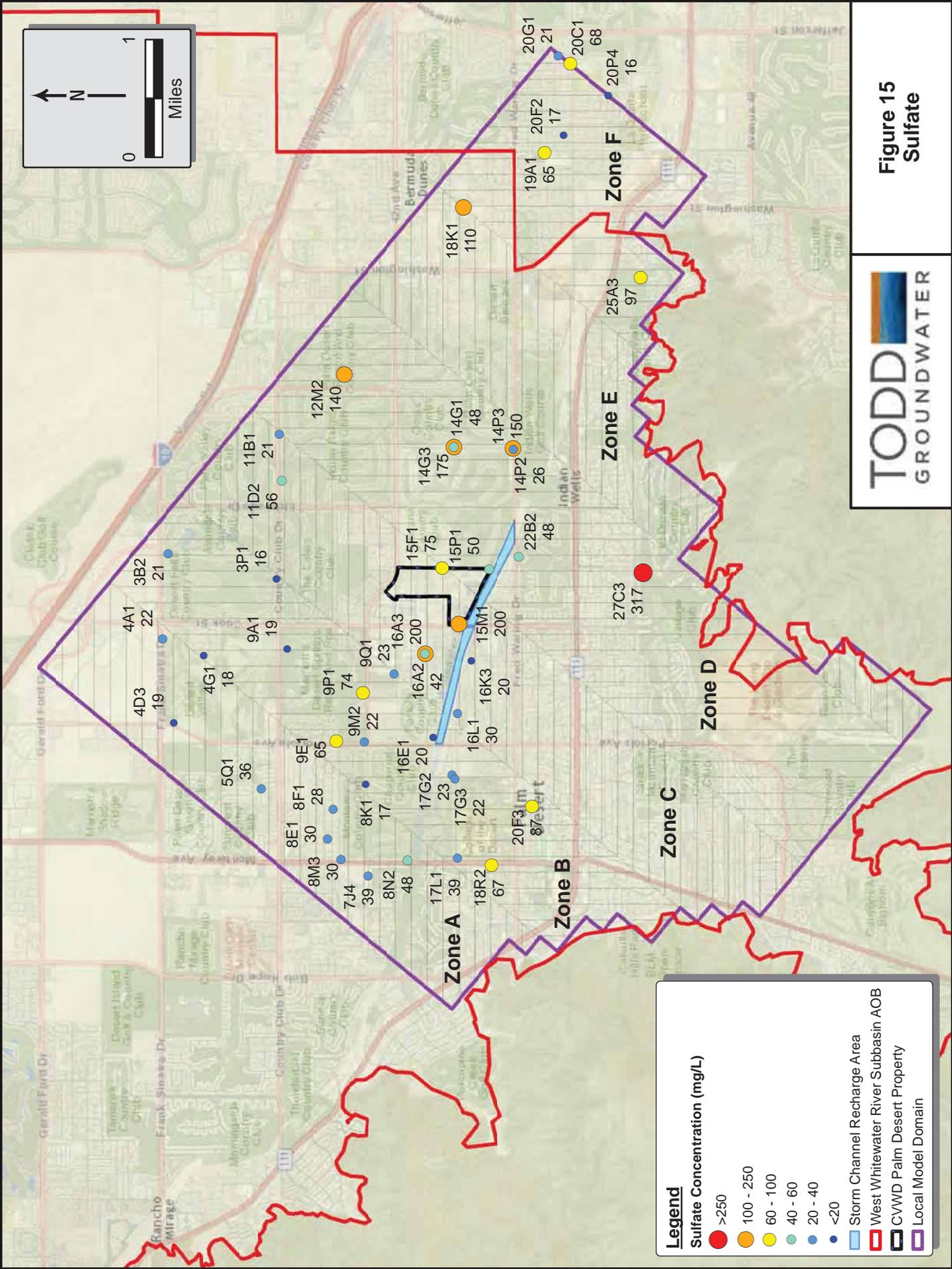


Figure 15  
Sulfate

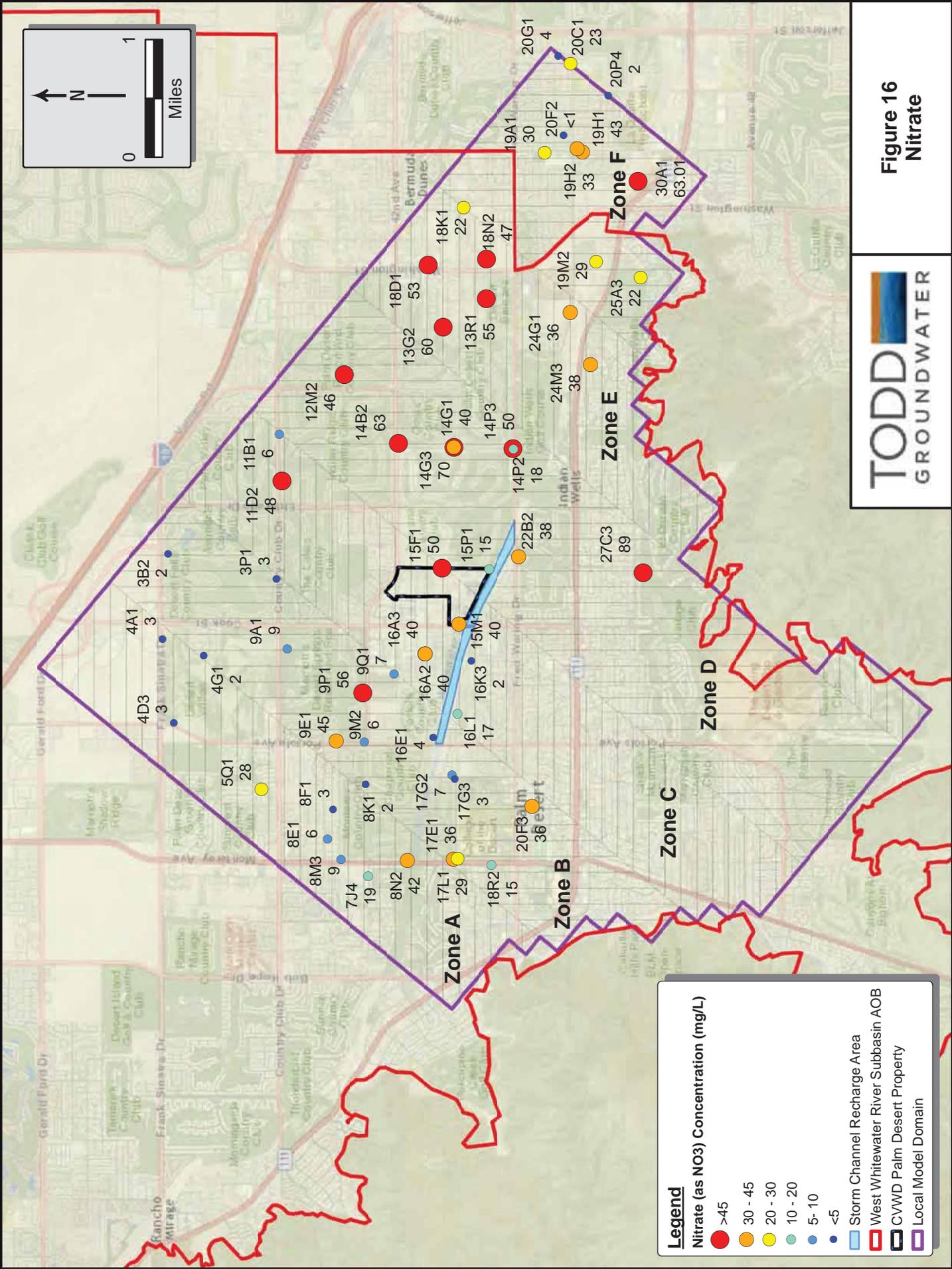


**Legend**

**Sulfate Concentration (mg/L)**

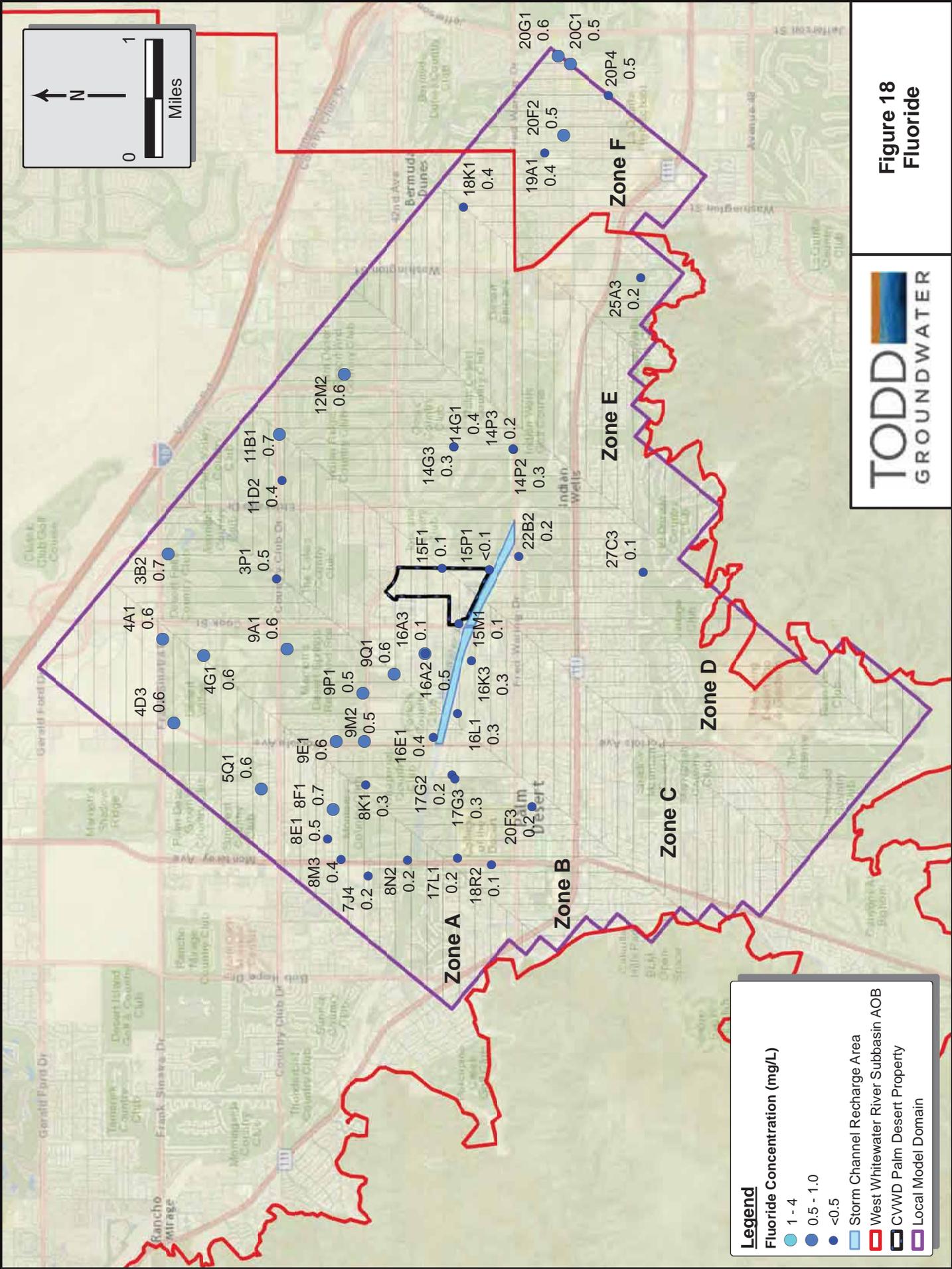
- >250
- 100 - 250
- 60 - 100
- 40 - 60
- 20 - 40
- <20

- Storm Channel Recharge Area
- West Whitewater River Subbasin AOB
- CVWD Palm Desert Property
- Local Model Domain



**Figure 16**  
Nitrate

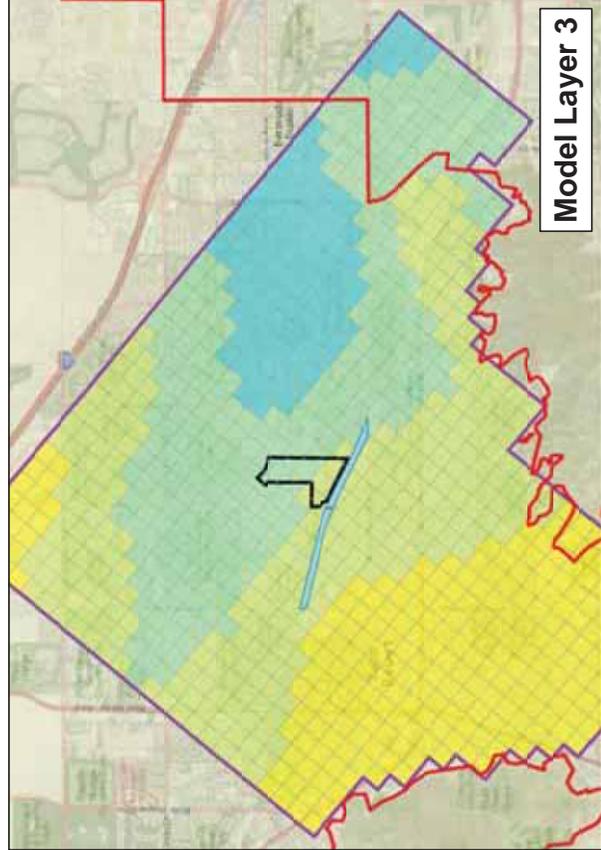
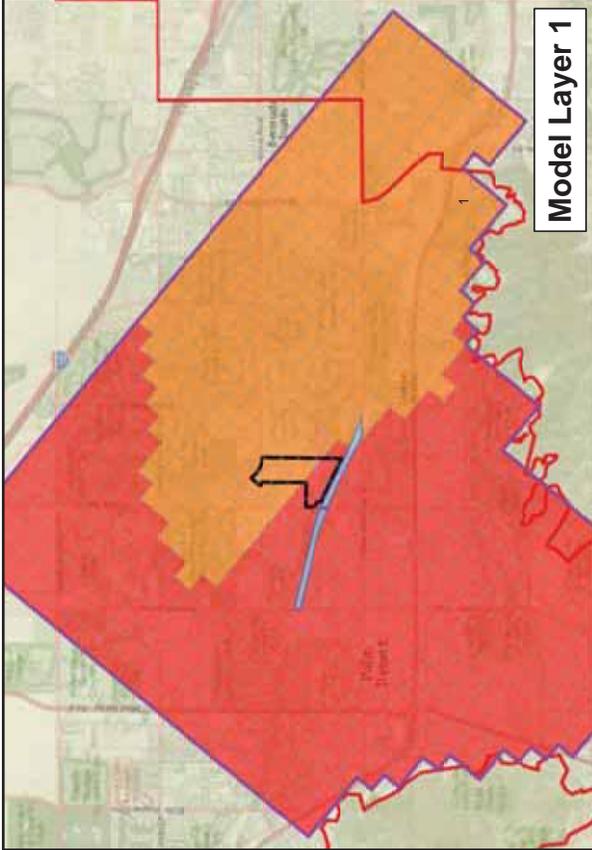
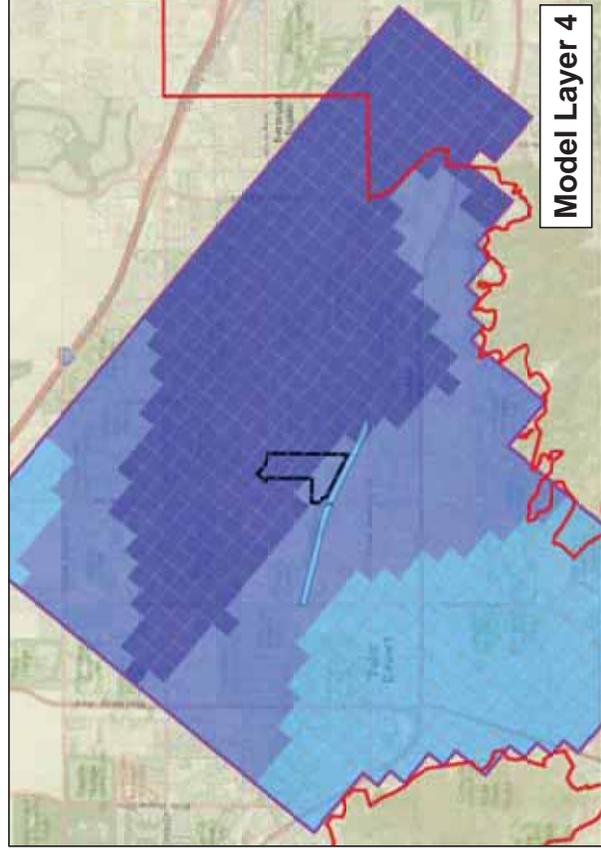
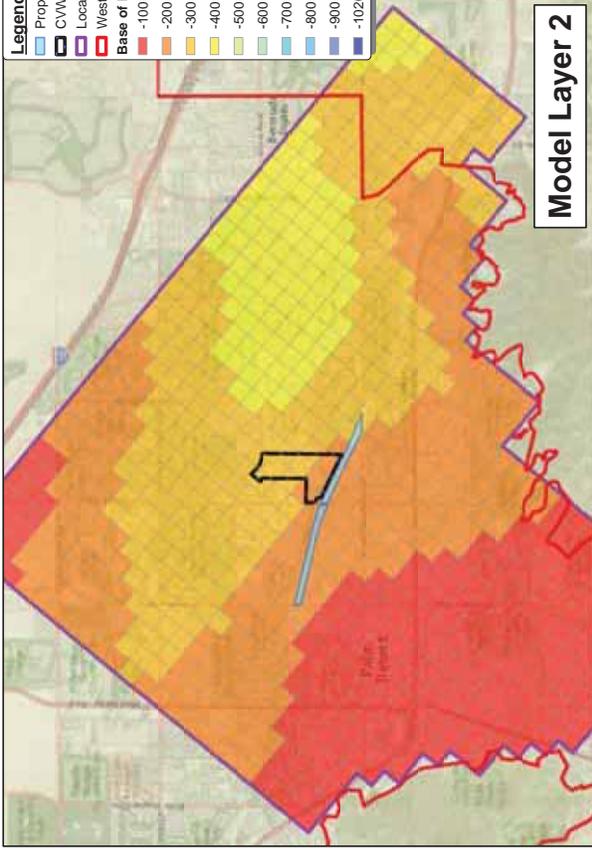
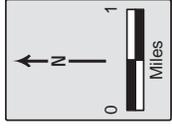




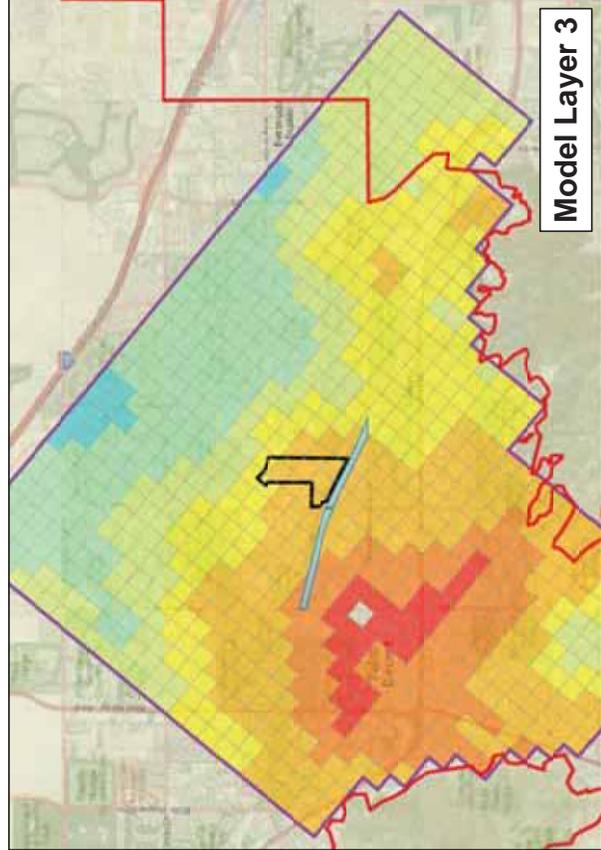
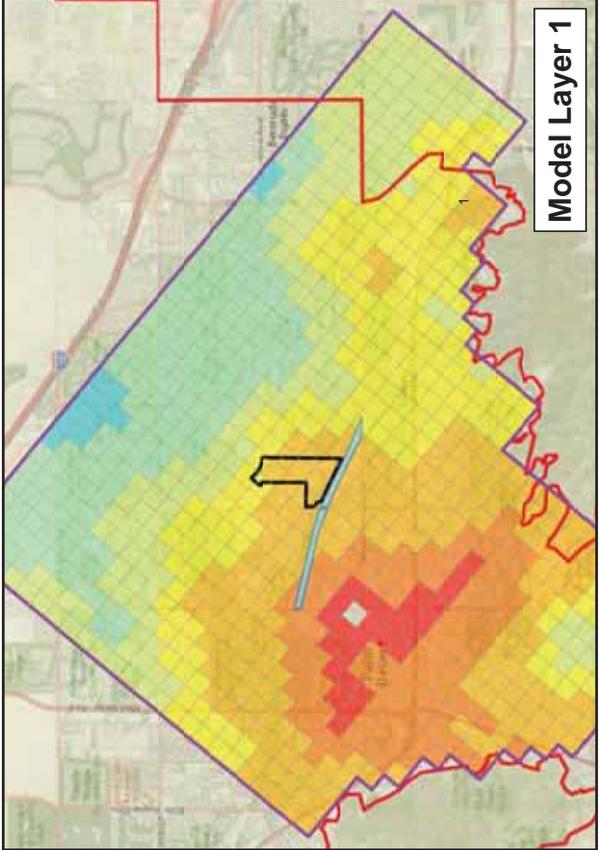
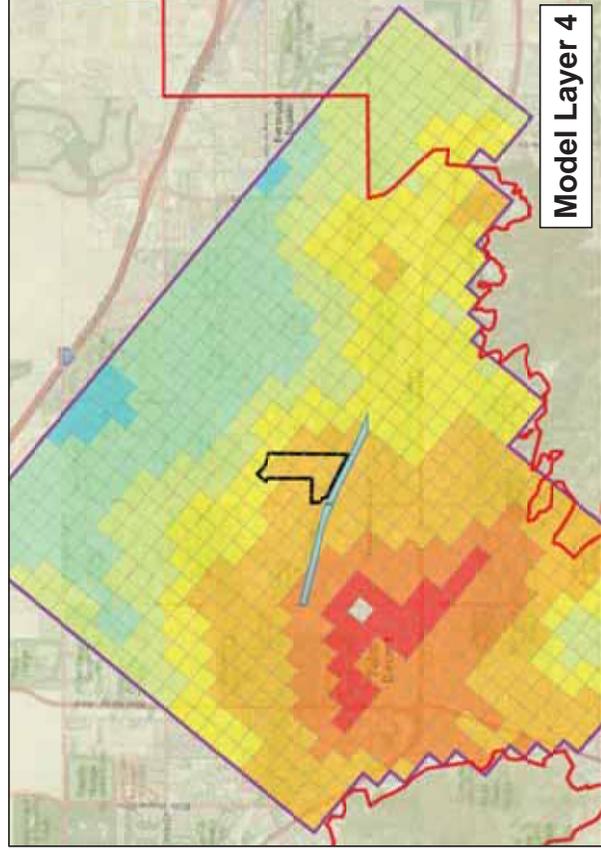
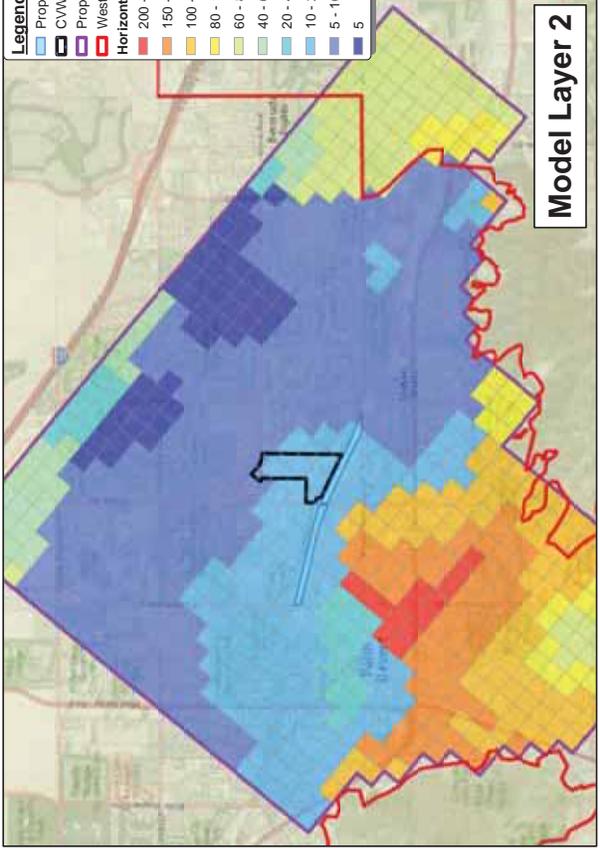
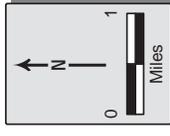


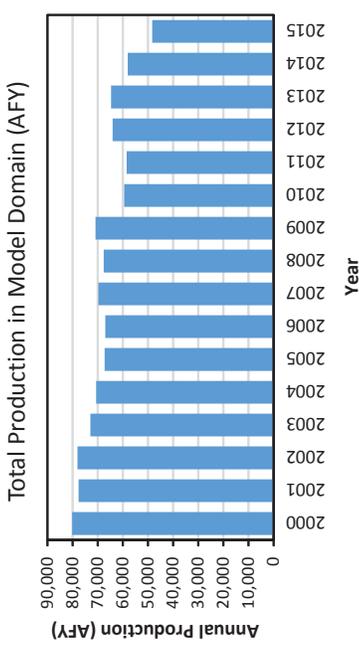
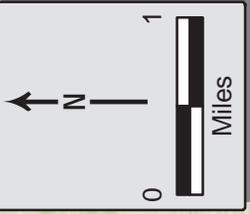


- Legend**
- Proposed Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB
  - Base of Model Layer (feet msl)
    - 100 - 0
    - 200 - -100
    - 300 to -200
    - 400 to -300
    - 500 to -400
    - 600 to -500
    - 700 to -600
    - 800 to -700
    - 900 to -800
    - 1020 to -900

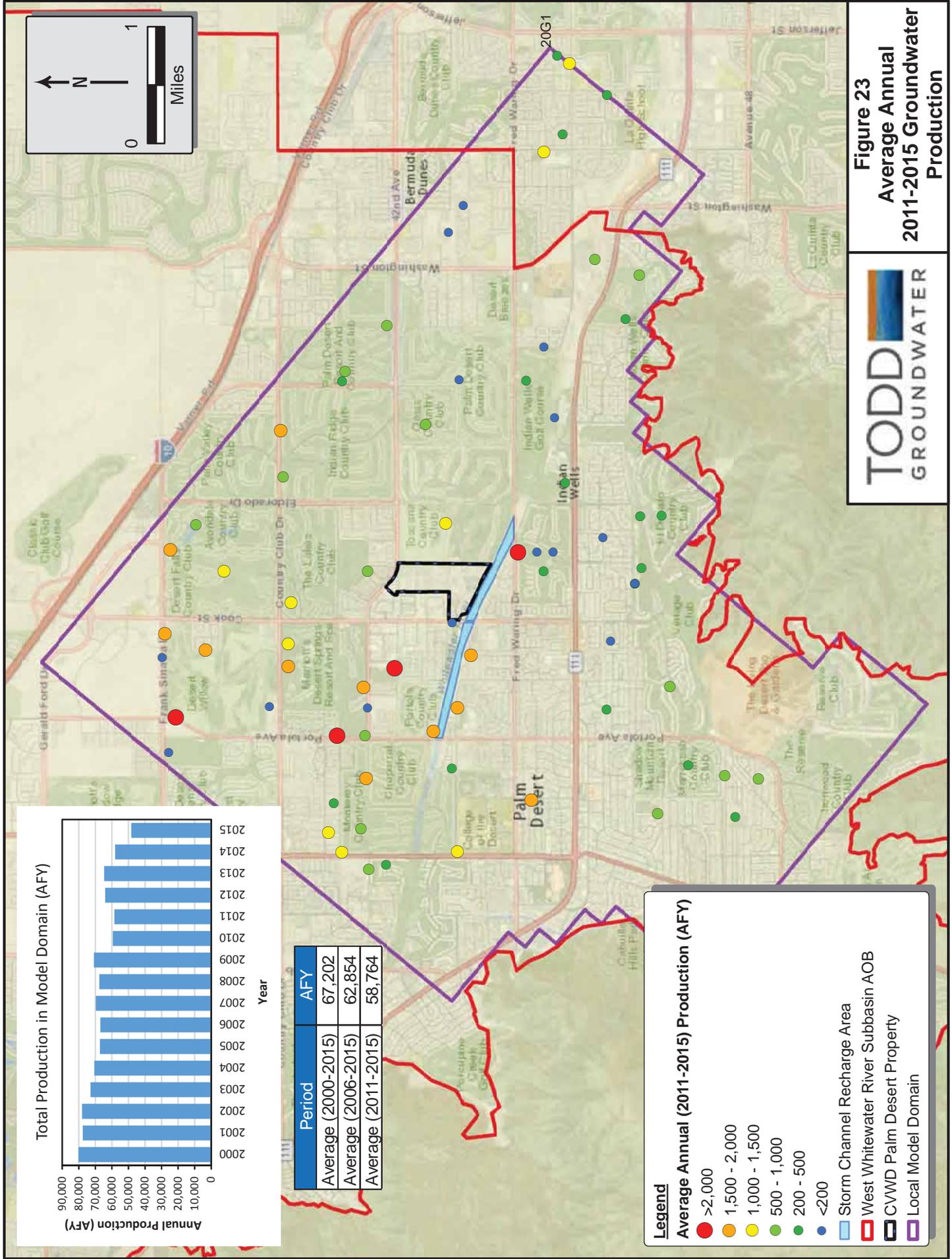
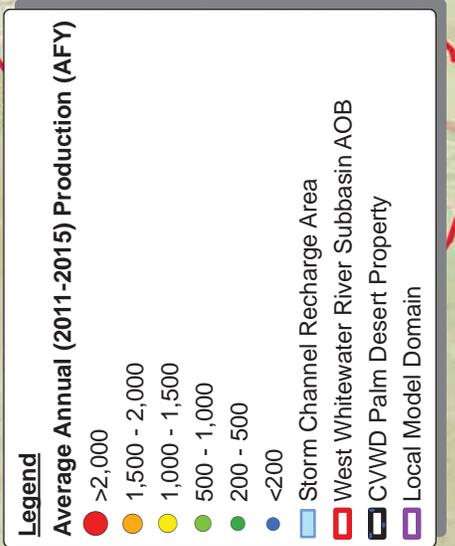


- Legend**
- Proposed Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Proposed Local Model Domain
  - West Whitewater River Subbasin AOB
  - Horizontal Hydraulic Conductivity (feet/day)
- |           |
|-----------|
| 200 - 225 |
| 150 - 200 |
| 100 - 150 |
| 80 - 100  |
| 60 - 80   |
| 40 - 60   |
| 20 - 40   |
| 10 - 20   |
| 5 - 10    |
| 5         |

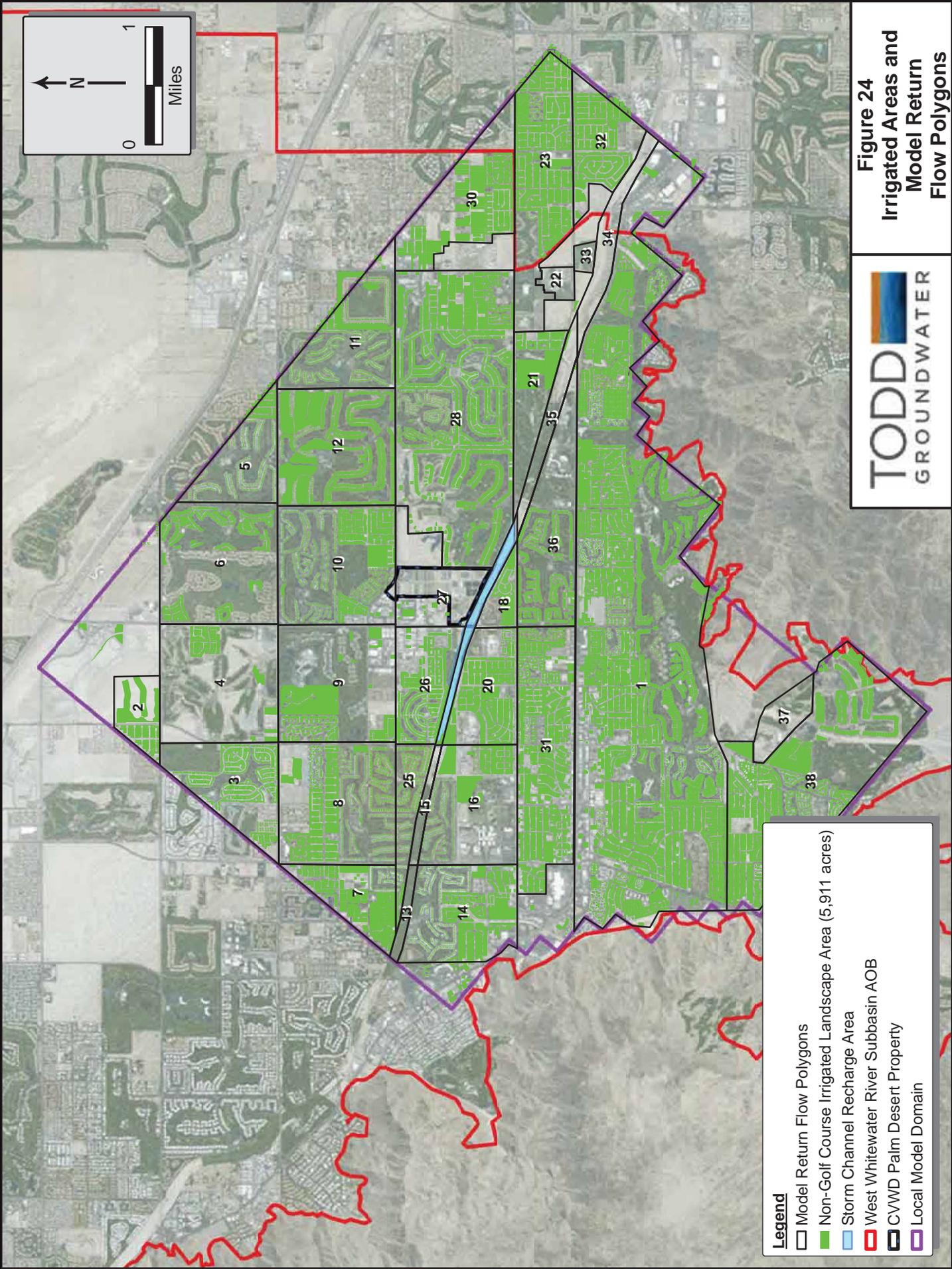




Period	AFY
Average (2000-2015)	67,202
Average (2006-2015)	62,854
Average (2011-2015)	58,764



**Figure 23**  
**Average Annual**  
**2011-2015 Groundwater**  
**Production**



**Figure 24**  
**Irrigated Areas and**  
**Model Return**  
**Flow Polygons**



- Legend**
- Model Return Flow Polygons
  - Non-Golf Course Irrigated Landscape Area (5,911 acres)
  - Storm Channel Recharge Area
  - West Whitewater River Subbasin AOB
  - CVWD Palm Desert Property
  - Local Model Domain

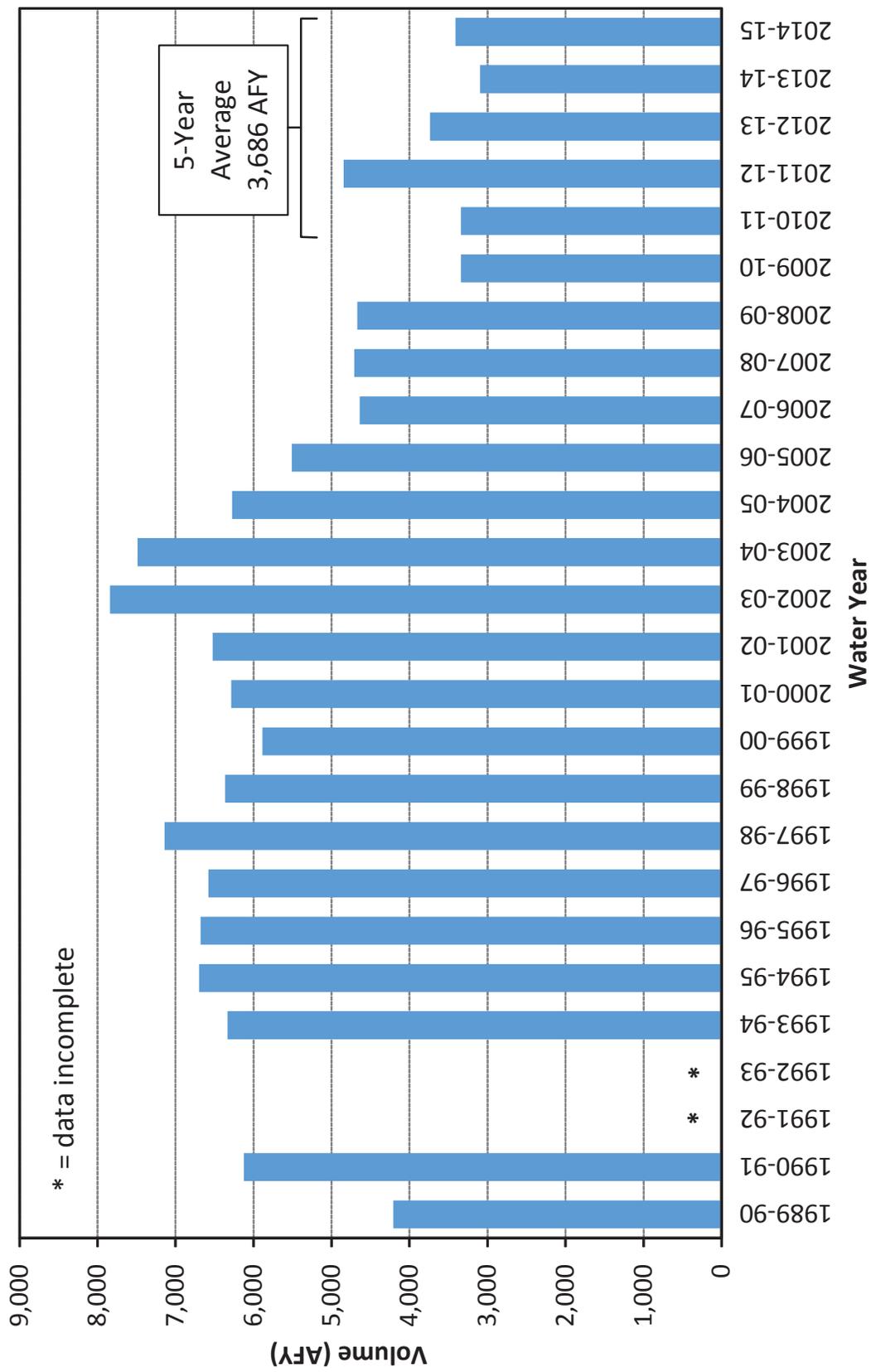
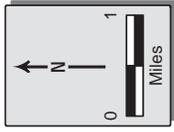
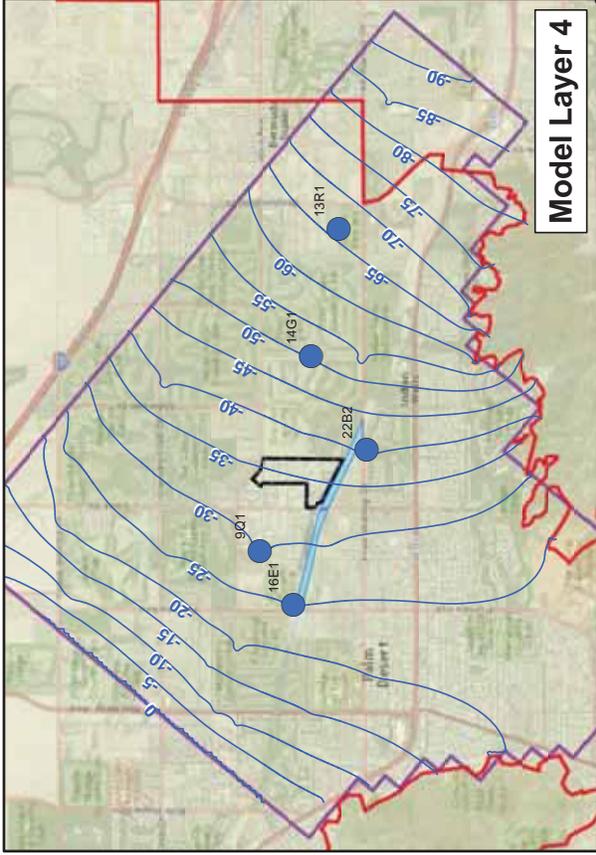
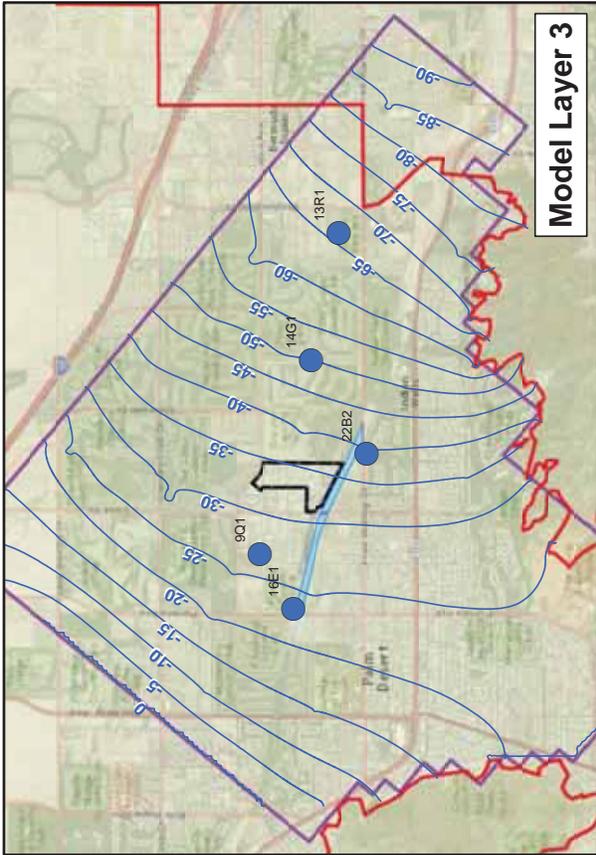
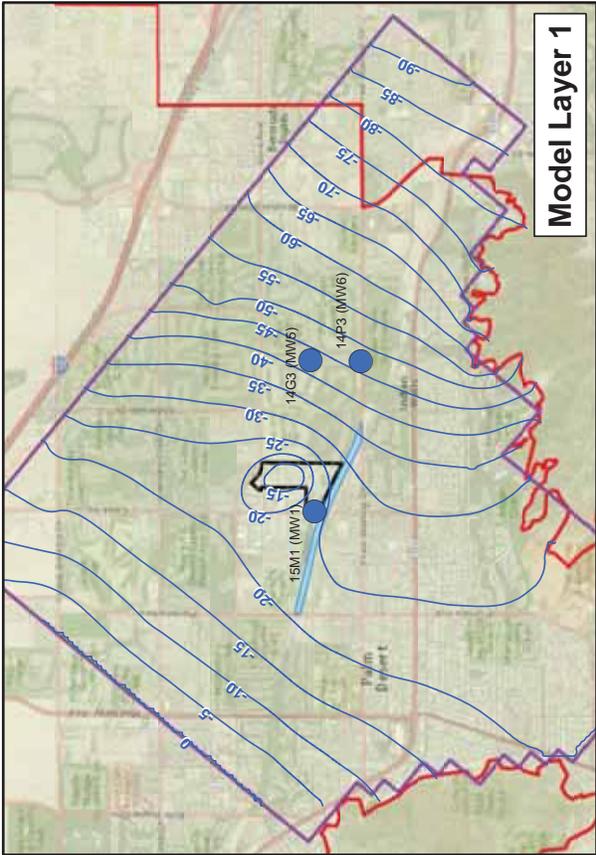


Figure 25  
WRP 10 Annual  
Secondary Effluent  
Discharge

- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB

<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



- Legend**
- Simulated Flowpaths from Recharge Area
  - Groundwater Elevation Contour (feet msl)
  - Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB

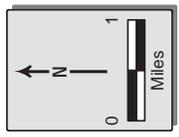
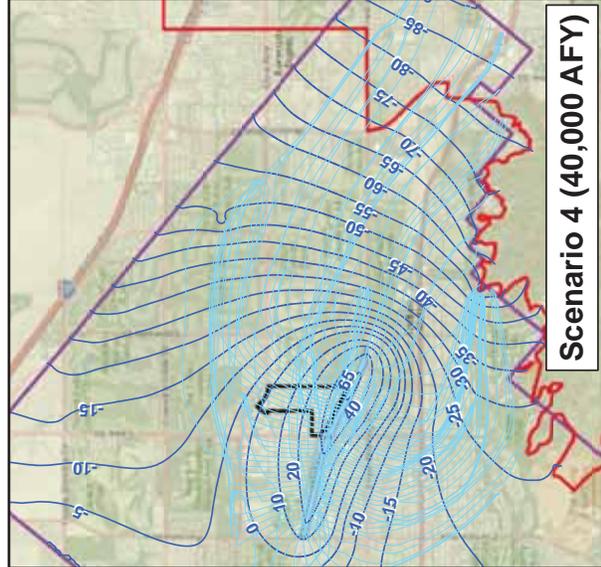
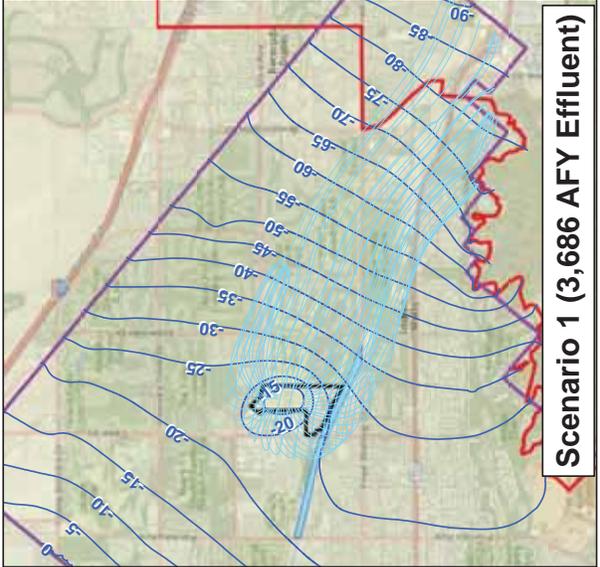
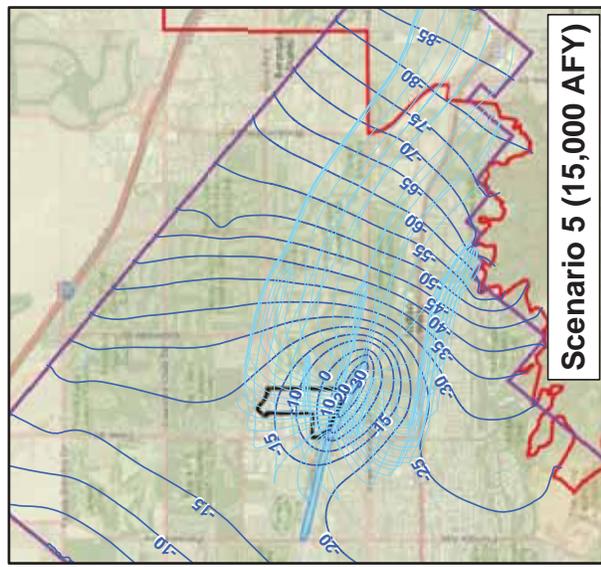
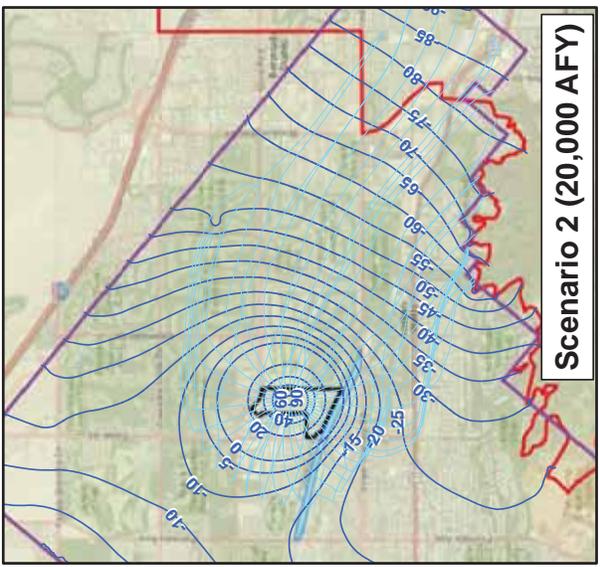
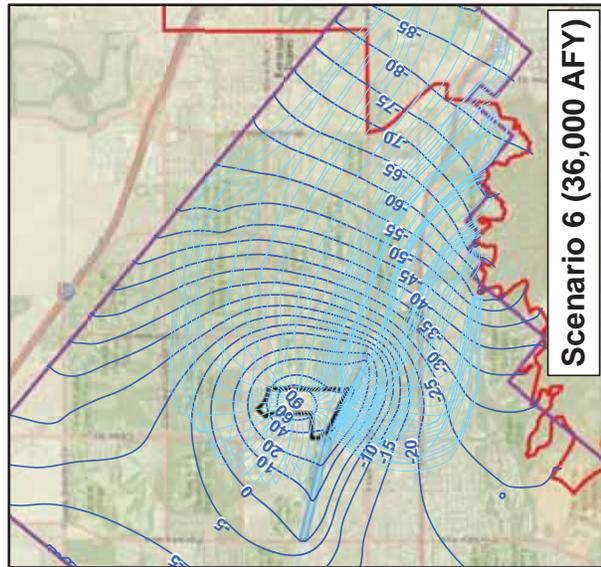
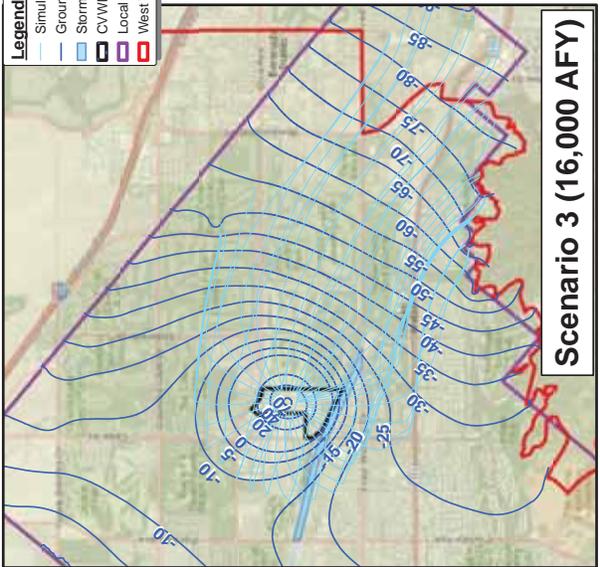
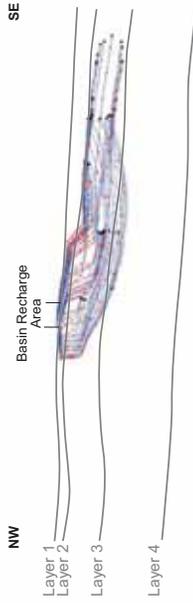


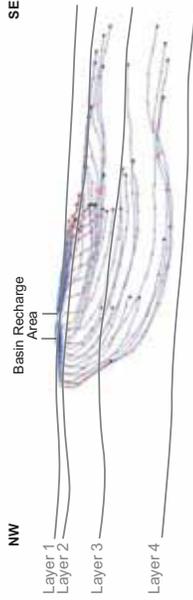
Figure 27  
Plan View of  
Simulated Flowpaths  
Scenarios 1 through 6



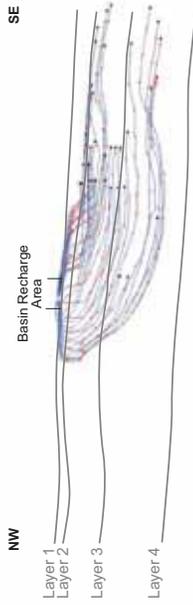
Scenario 1 Baseline



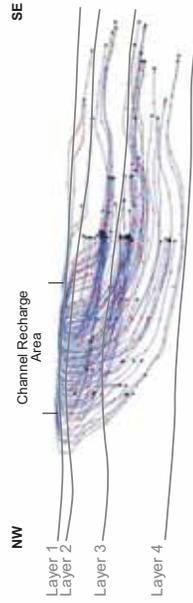
Scenario 2 20,000 AFY Ponds



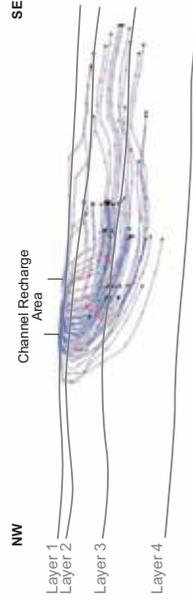
Scenario 3 16,000 AFY Ponds



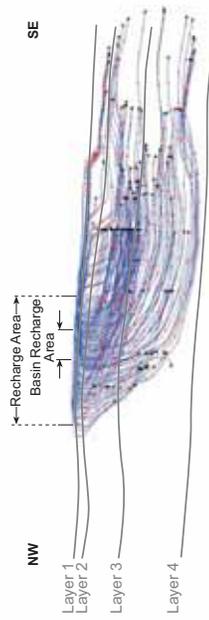
Scenario 4 40,000 AFY Channel



Scenario 5 15,000 AFY Channel



Scenario 6 36,000 AFY Ponds and Channel



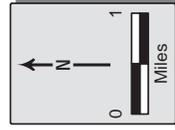
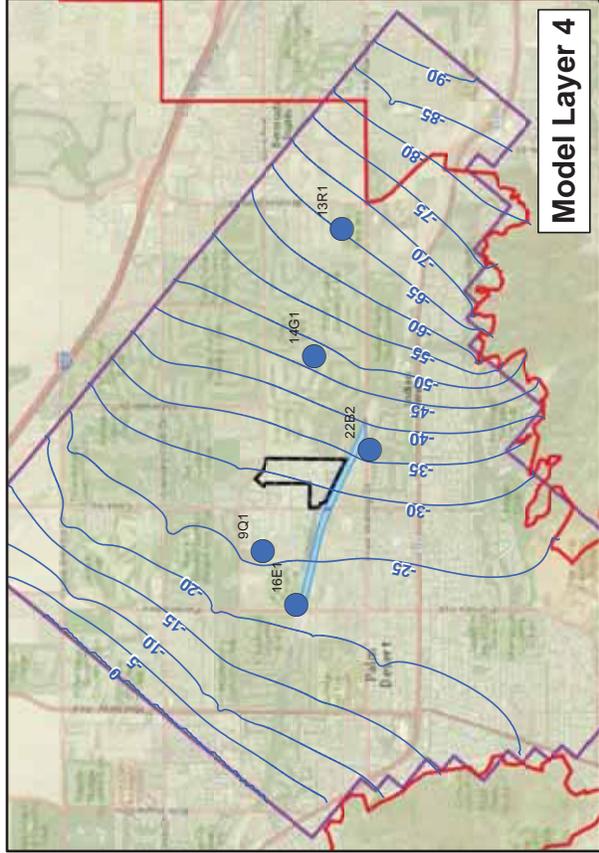
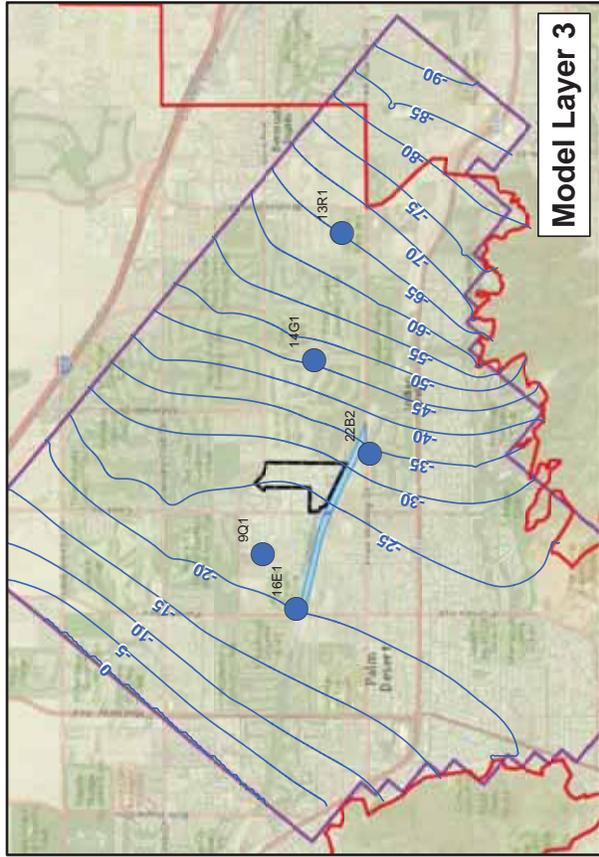
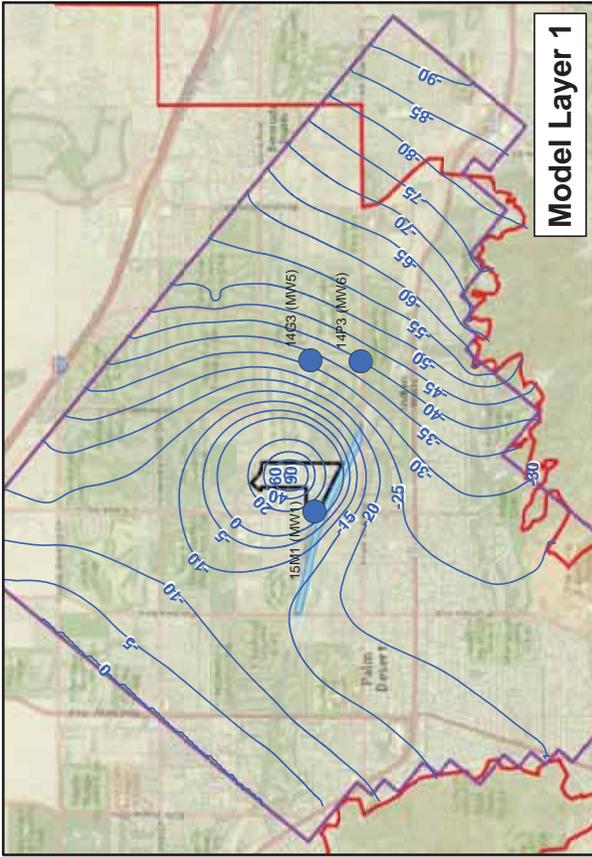
- Notes:
- Red arrowheads reflect 10-year travel times.
  - • Pathline endpoint (pumping well or boundary).
  - Grey lines are model layer bottoms.
  - Vertical exaggeration = 10x



Figure 28  
 Cross-Sectional View  
 of Simulated Flowpaths  
 of Scenarios 1 through 6

- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

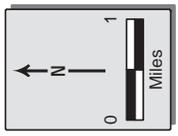
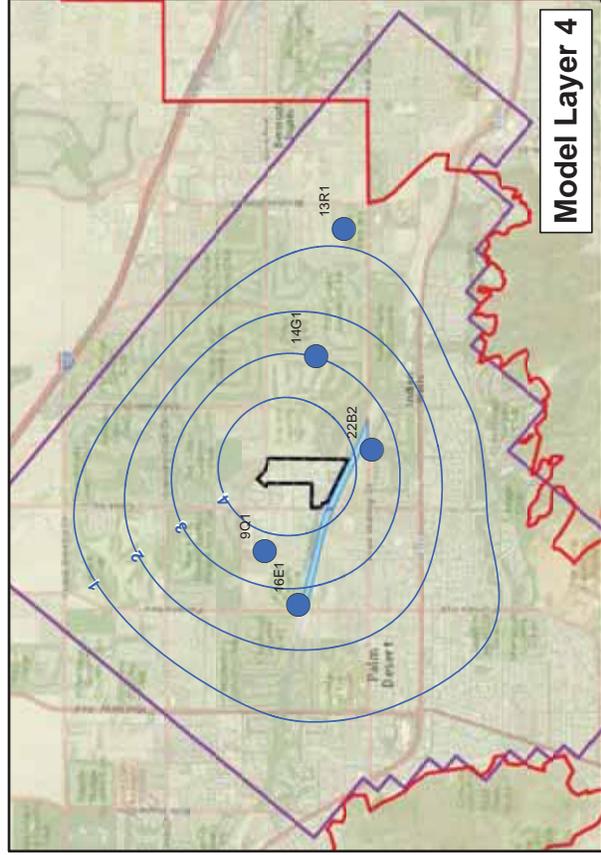
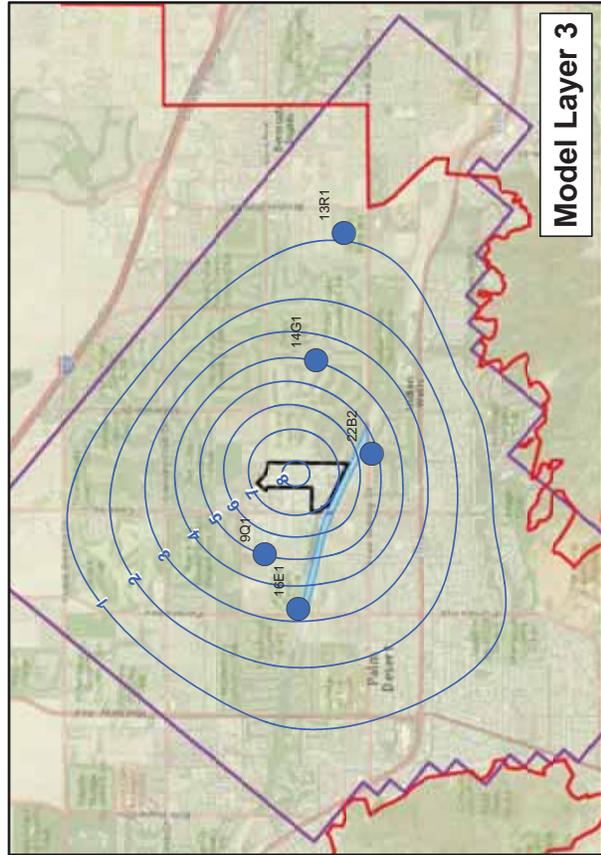
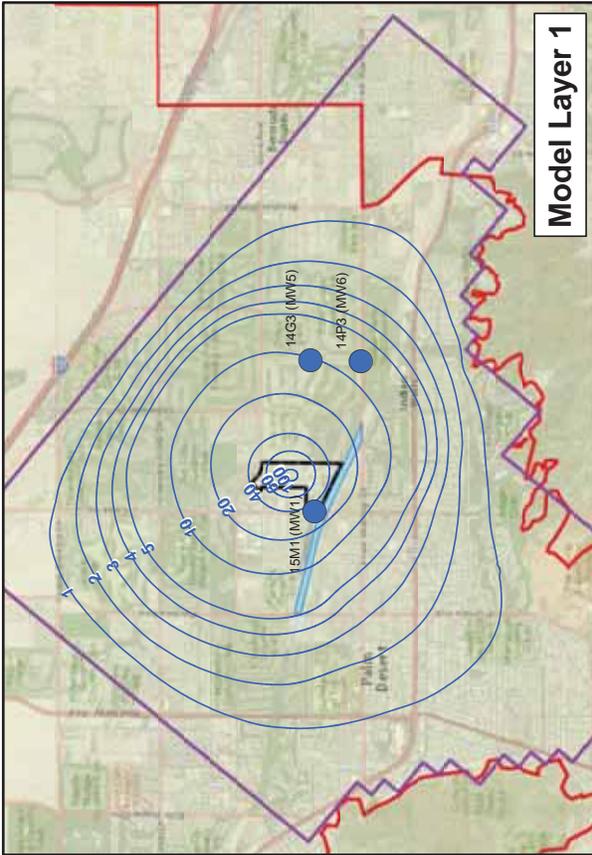
<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



- Legend**
- Well
  - Groundwater Level Change with 20,000 AFY (feet)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

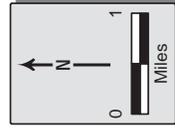
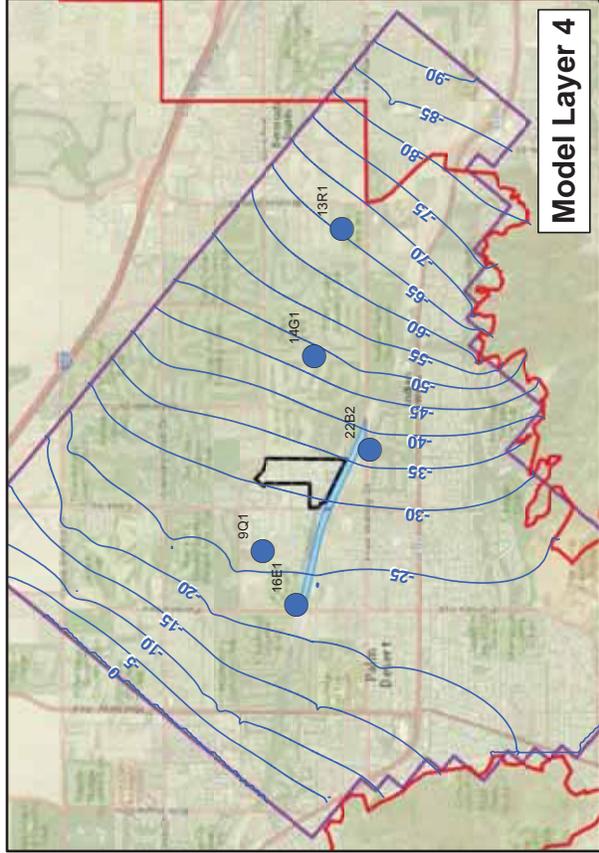
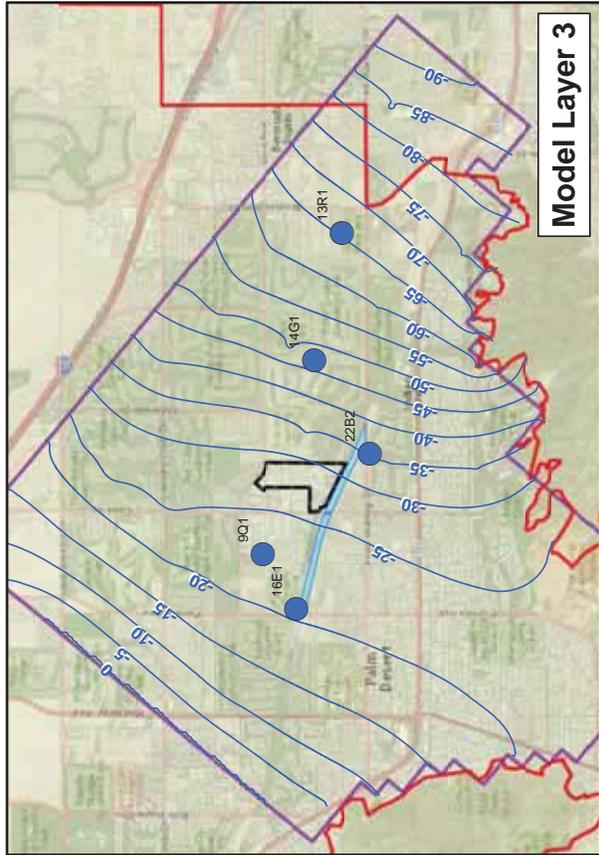
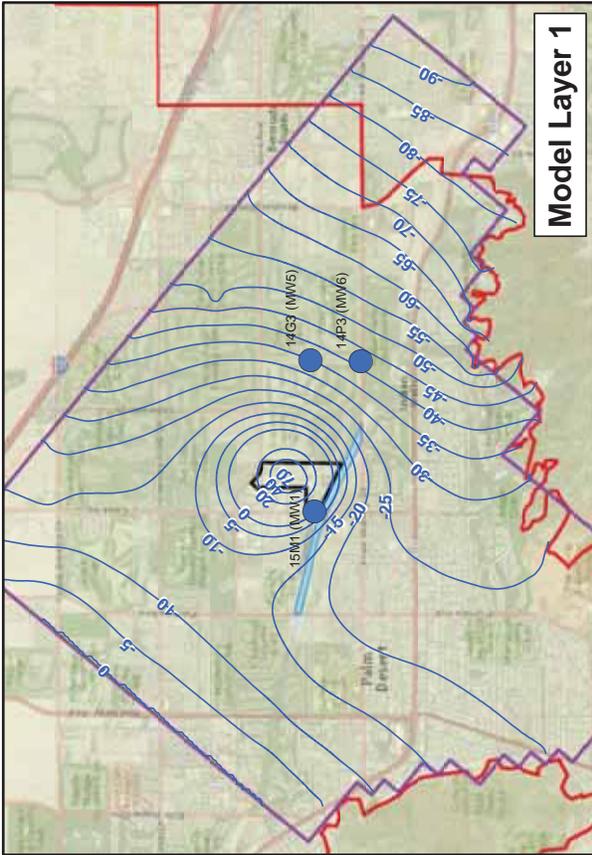
Note: Contours represent water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions.

Model Layer 1	270 ft (range 250 to 320 ft)
Model Layer 2	80 ft (range 0 to 160 ft)
Model Layer 3	300 ft
Model Layer 4	400 ft



- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

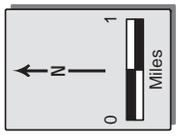
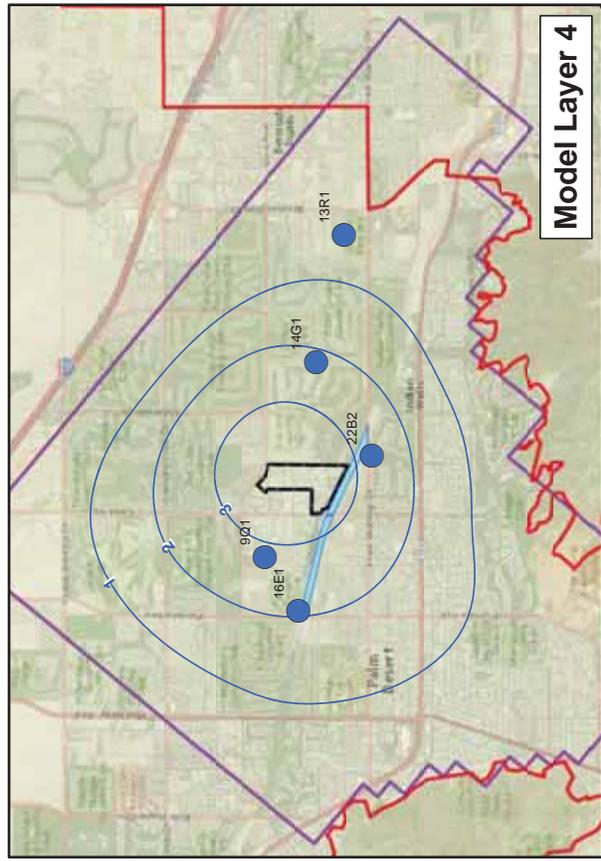
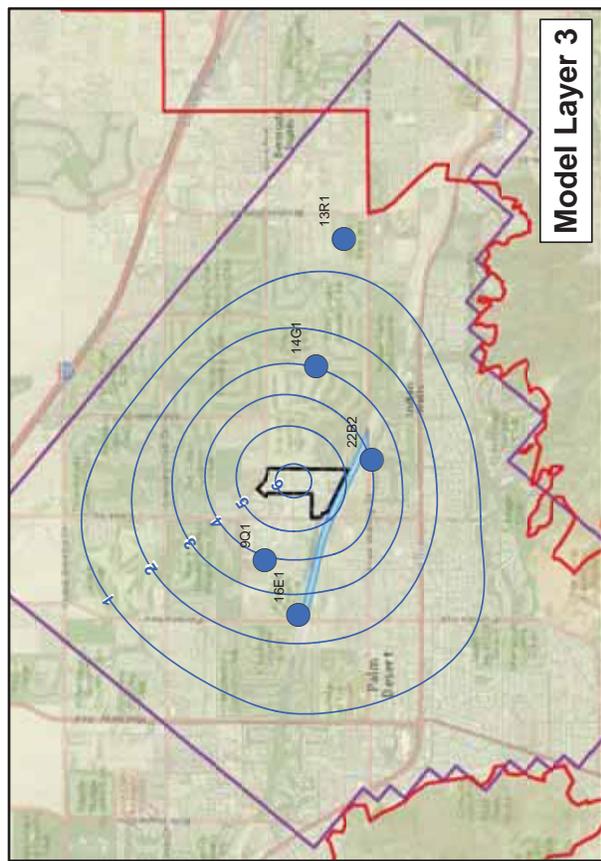
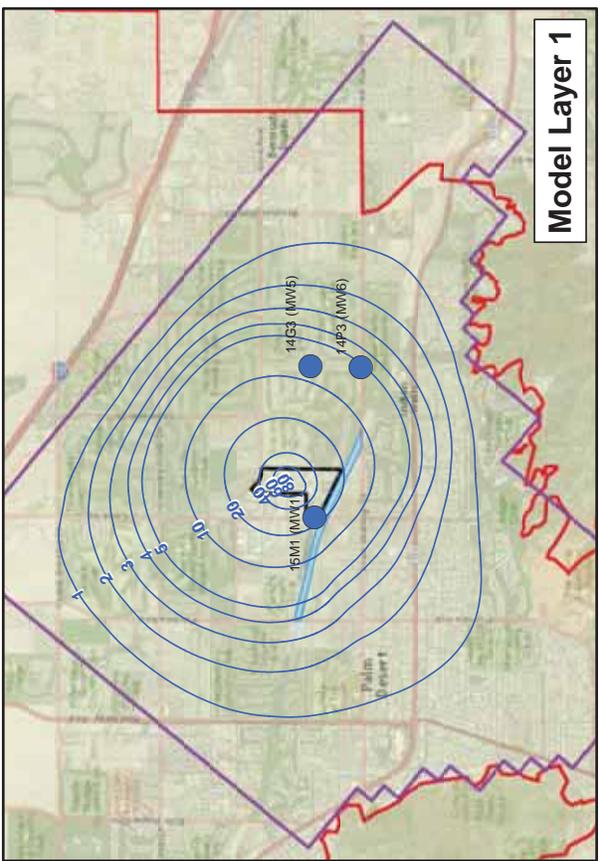
<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



- Legend**
- Well
  - Groundwater Level Change with 16,000 AFY (feet)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

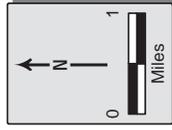
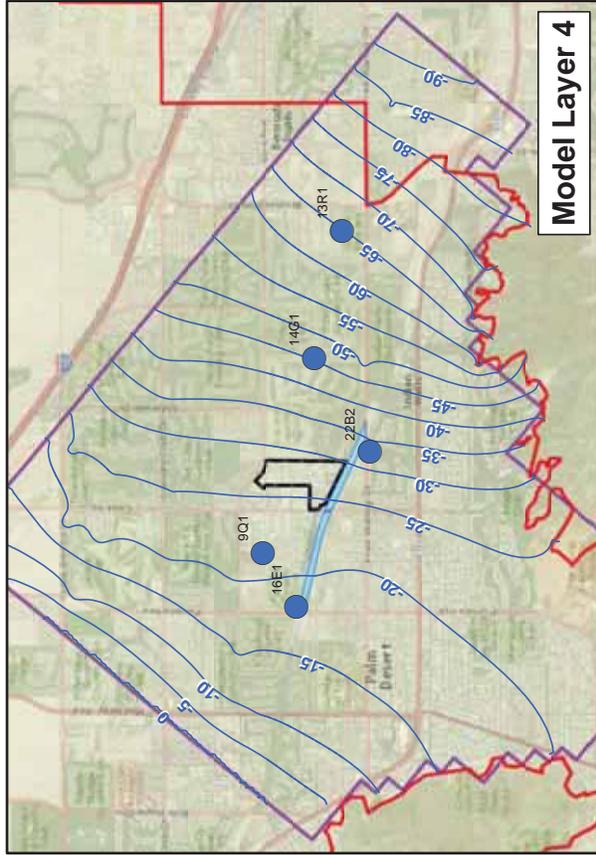
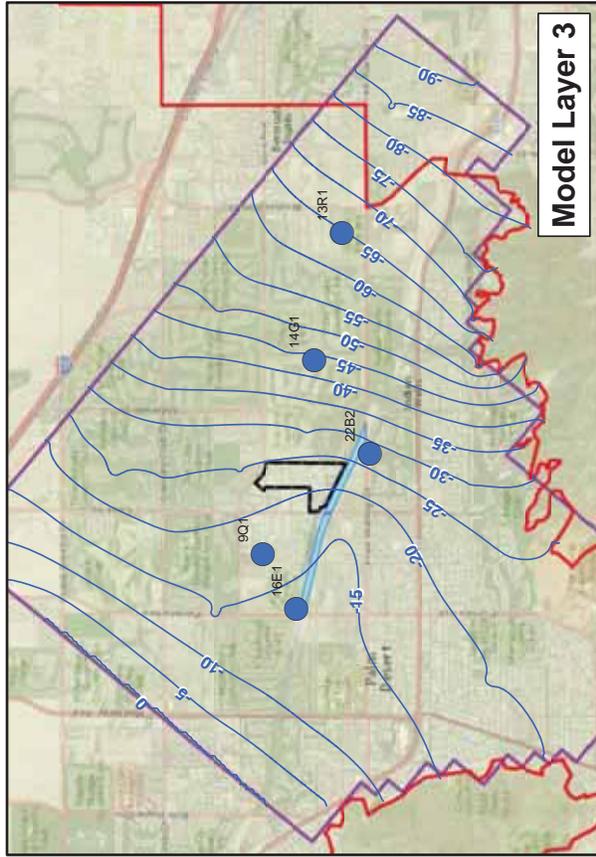
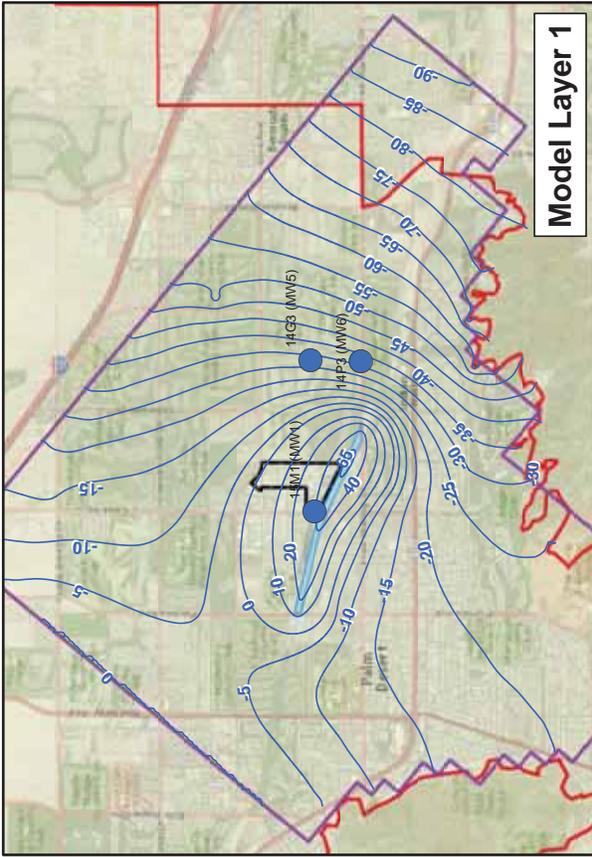
**Note:** Contours represent water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions.

<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

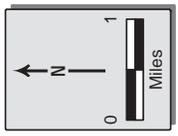
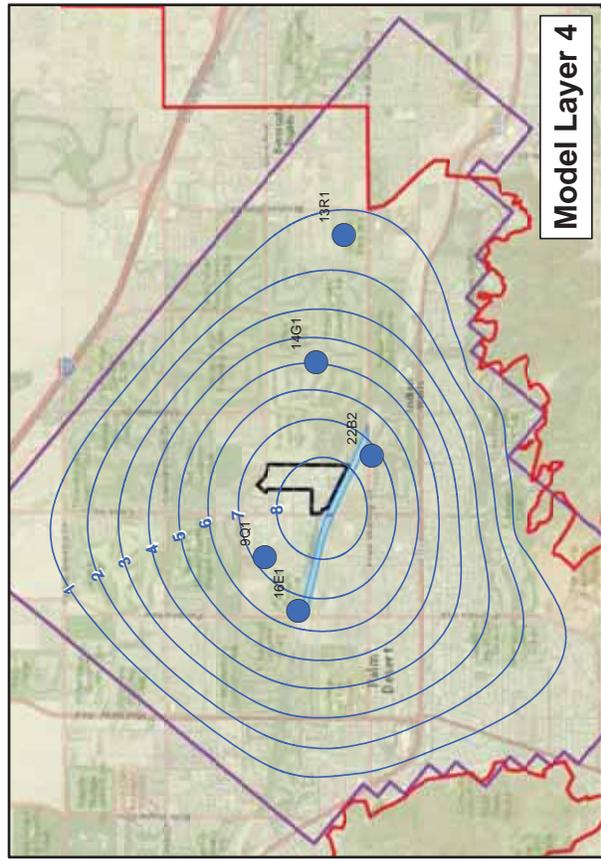
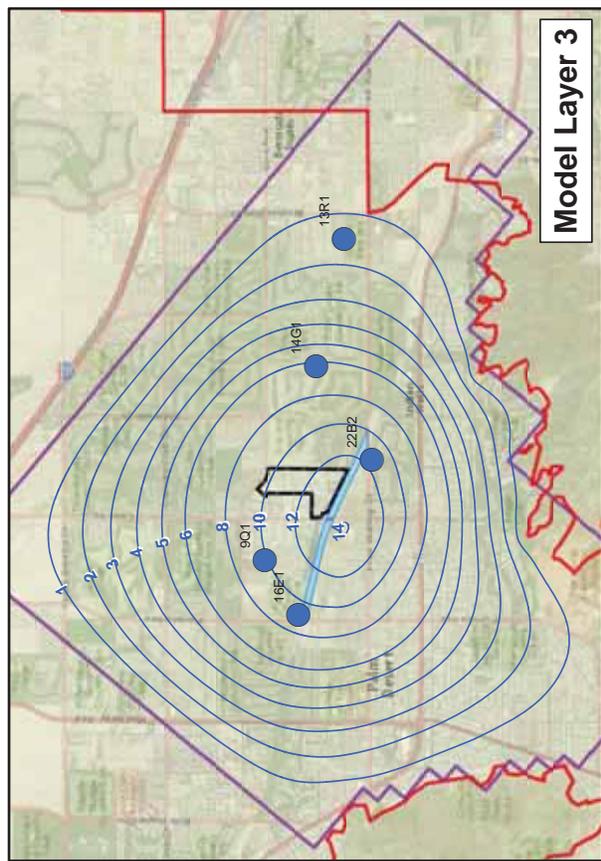
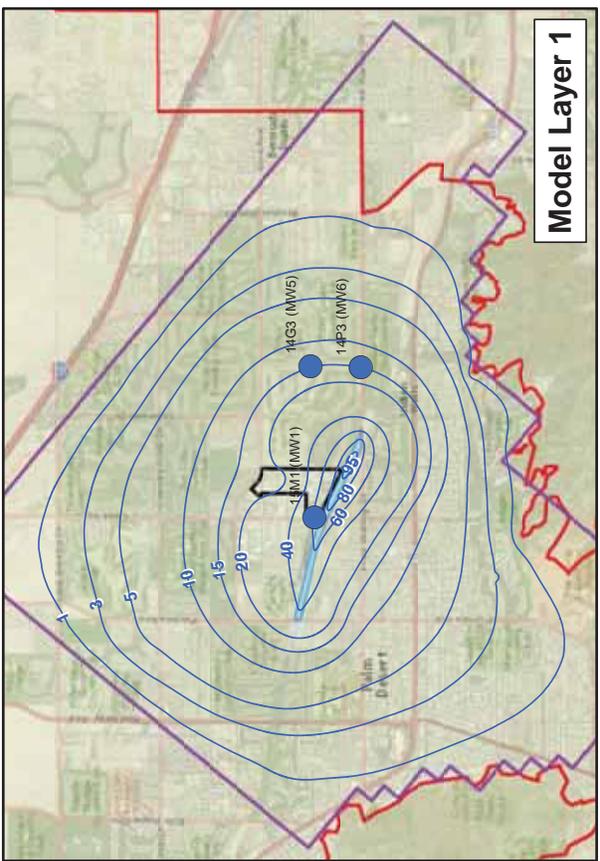
<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



- Legend**
- Well
  - Groundwater Level Change with 40,000 AFY (feet)
  - Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB

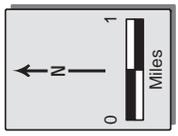
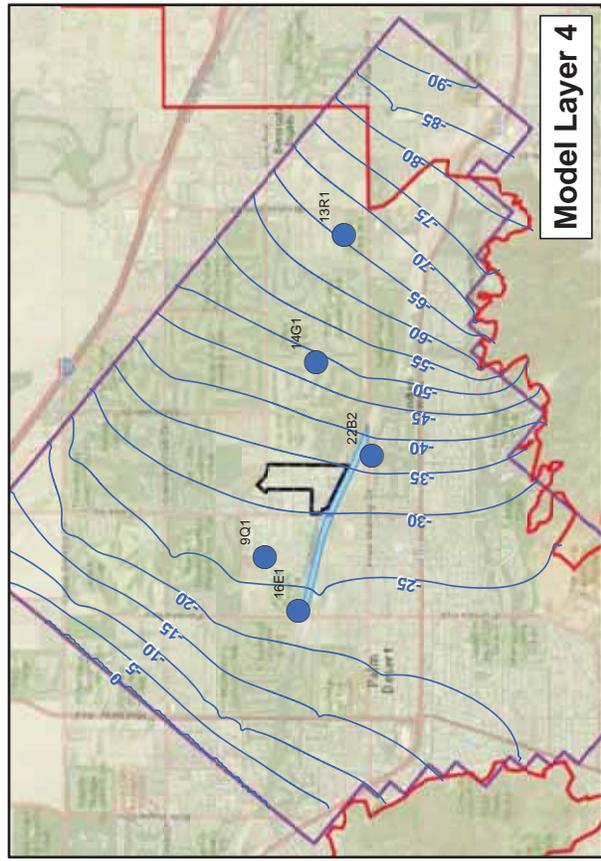
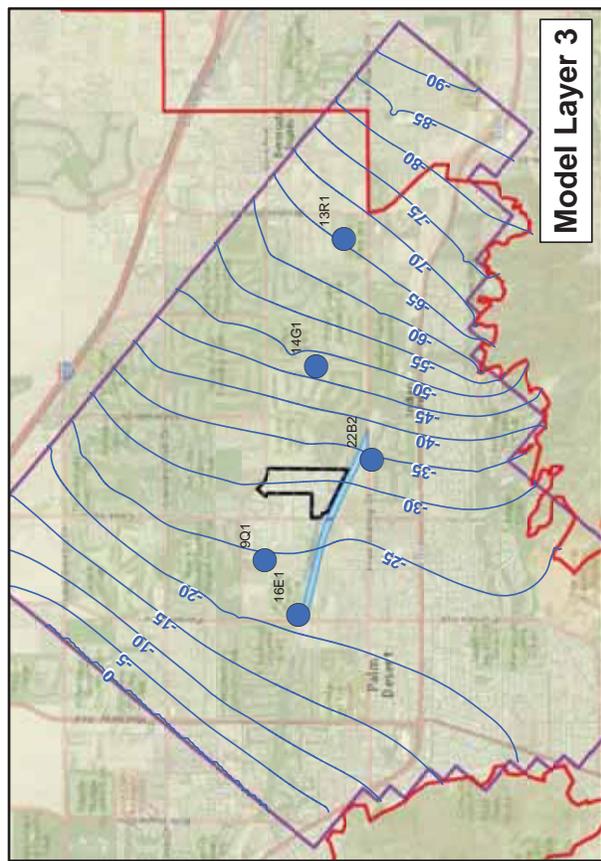
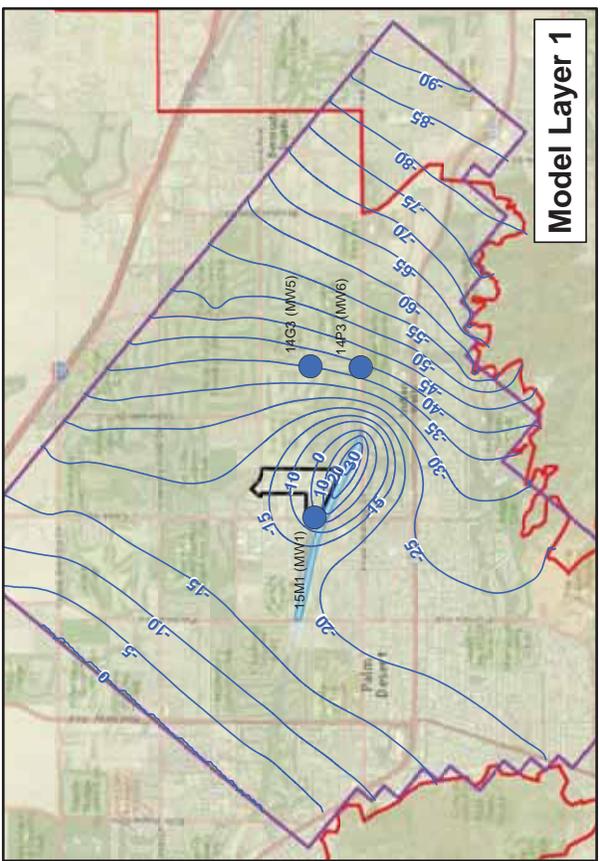
Note: Contours represent water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions.

Model Layer 1	270 ft (range 250 to 320 ft)
Model Layer 2	80 ft (range 0 to 160 ft)
Model Layer 3	300 ft
Model Layer 4	400 ft



- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

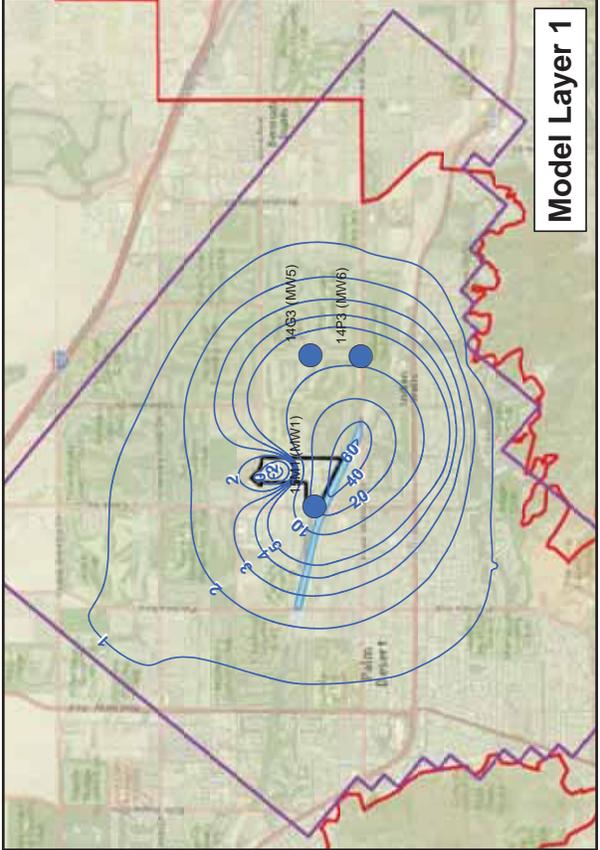
<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft



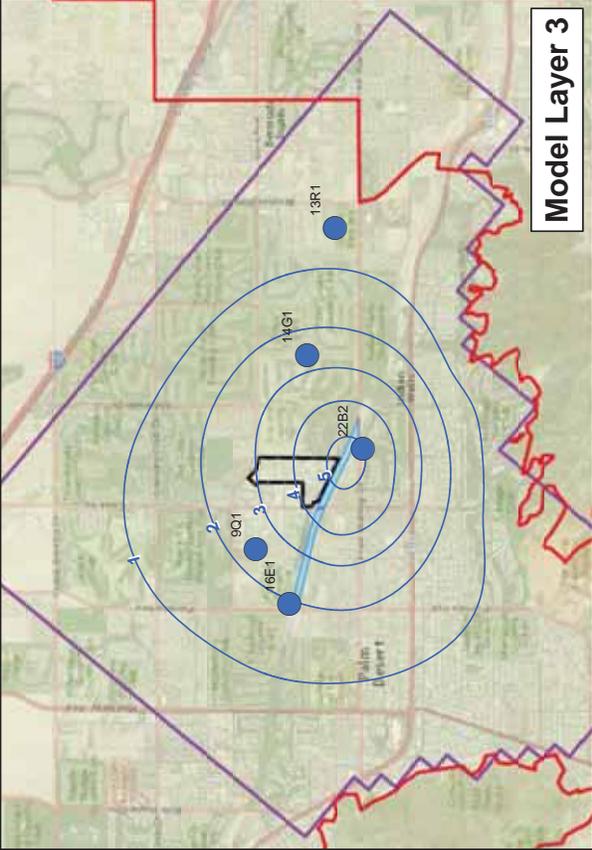
- Legend**
- Well
  - Groundwater Level Change with 15,000 AFY (feet)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

Note: Contours represent water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions.

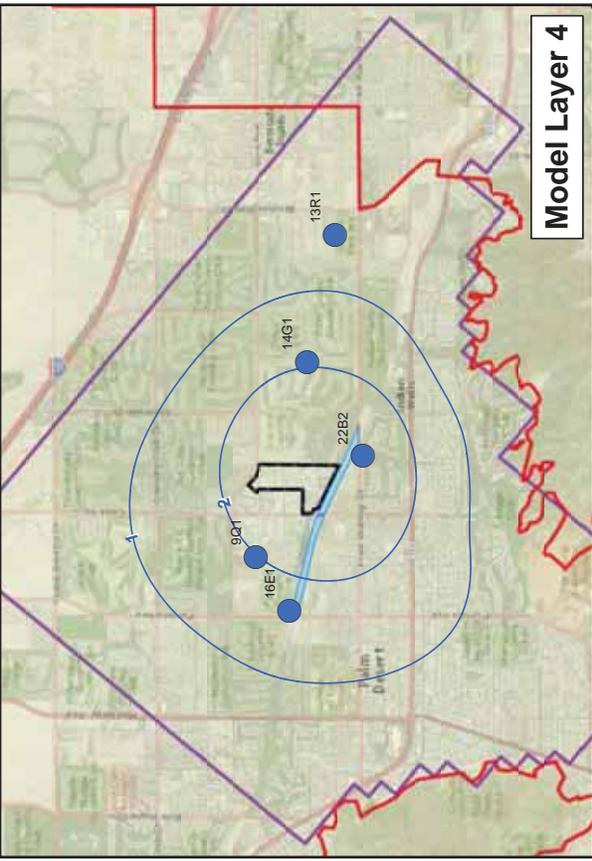
Model Layer 1	270 ft (range 250 to 320 ft)
Model Layer 2	80 ft (range 0 to 160 ft)
Model Layer 3	300 ft
Model Layer 4	400 ft



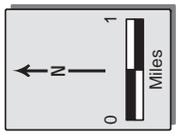
Model Layer 1



Model Layer 3

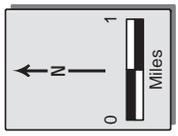
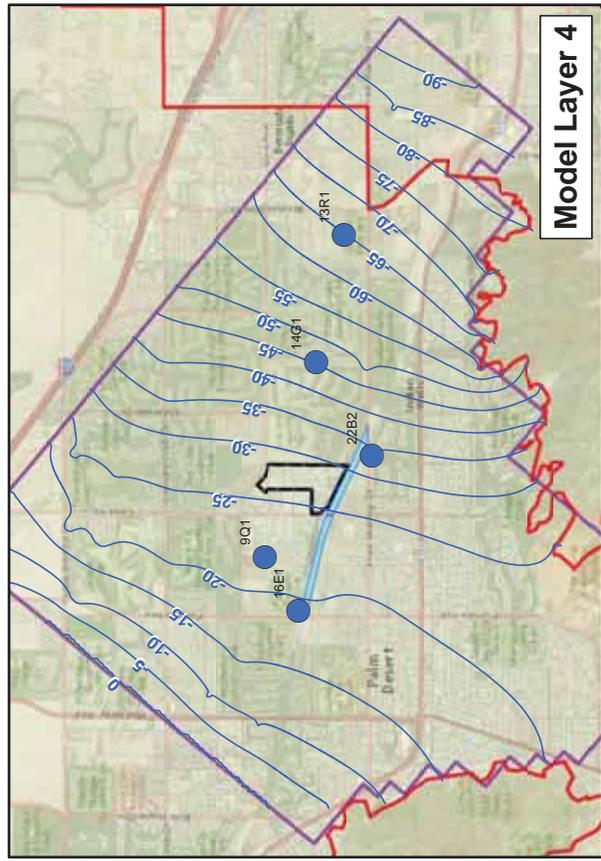
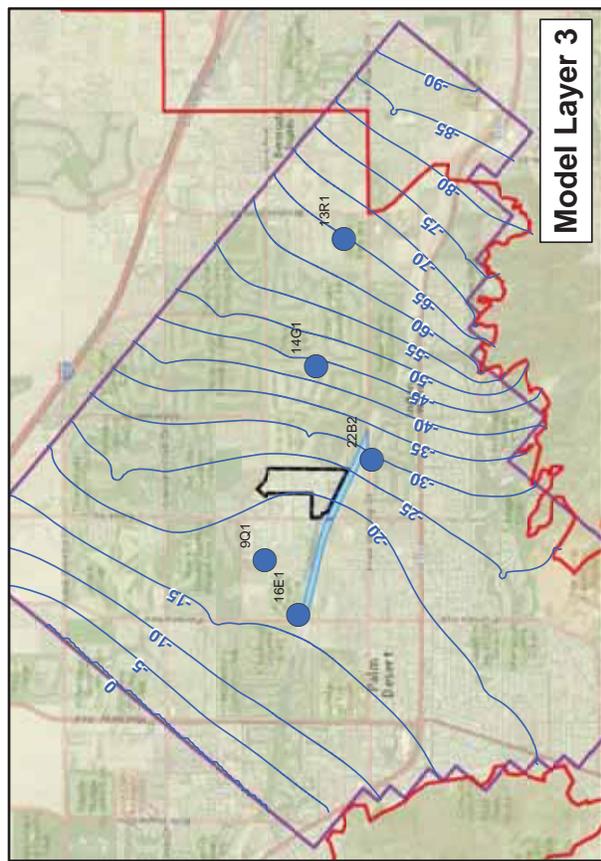
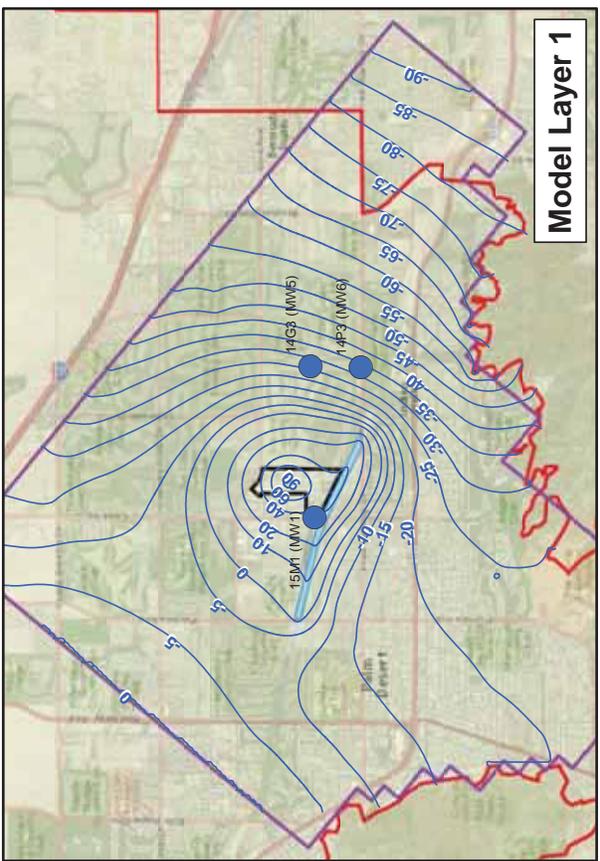


Model Layer 4



- Legend**
- Well
  - Groundwater Elevation Contour (feet msl)
  - Storm Channel Recharge Area
  - CVWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB

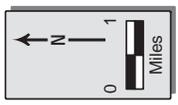
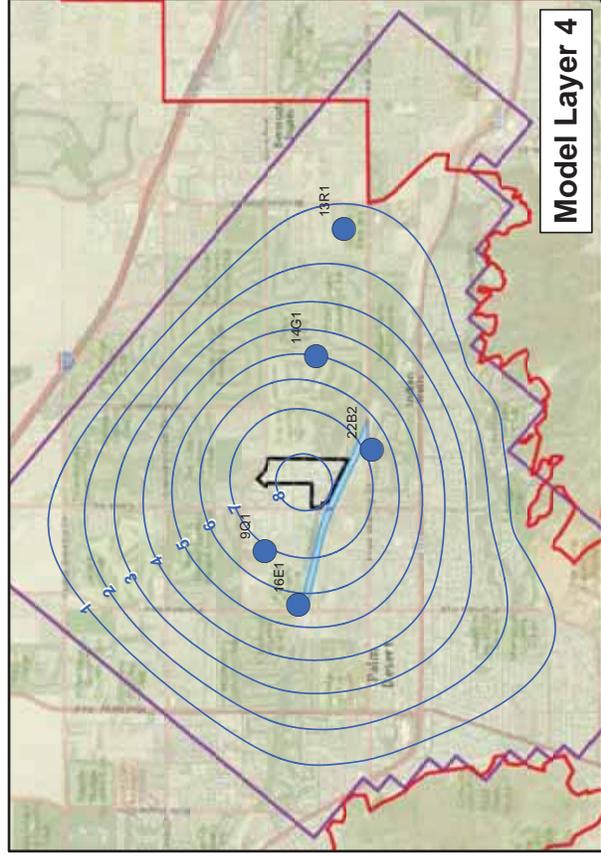
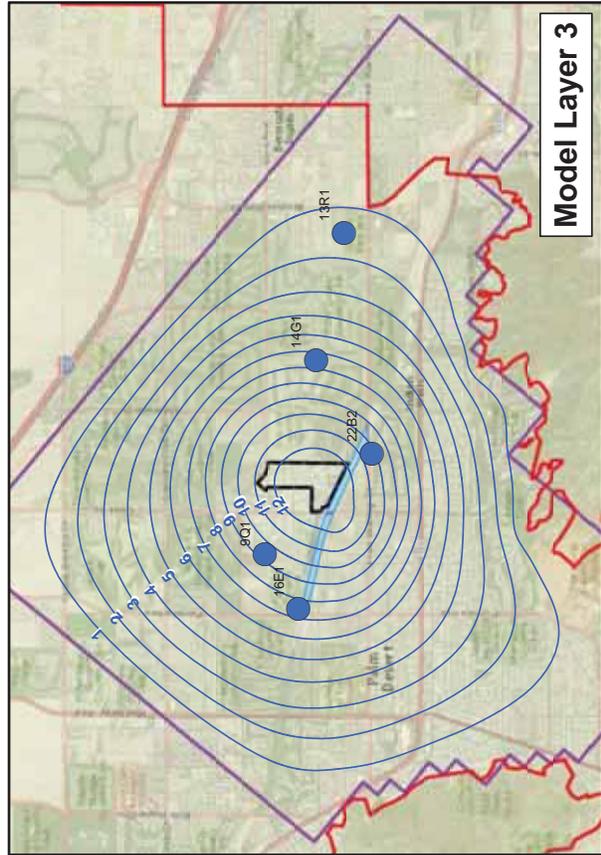
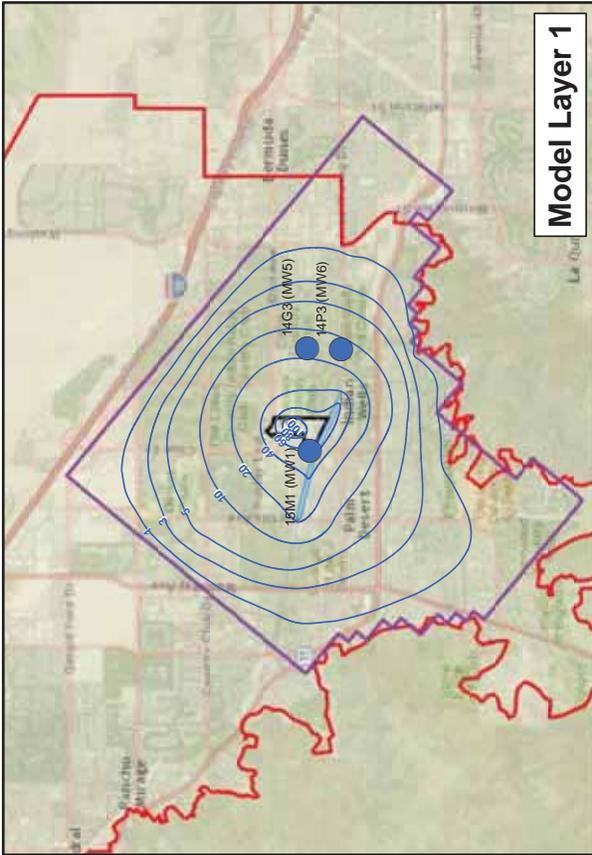
<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft

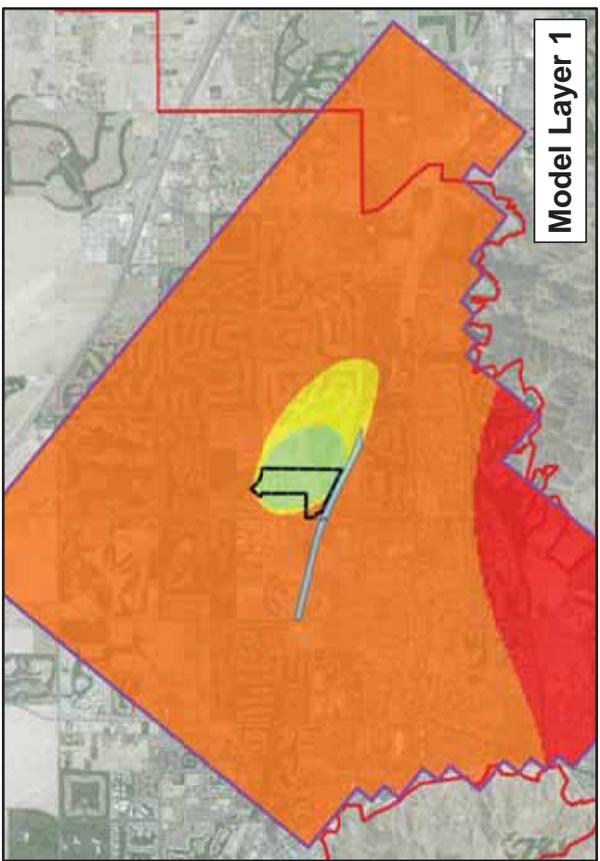


- Legend**
- Well
  - Groundwater Level Change with 36,000 AFY (feet)
  - ▭ Storm Channel Recharge Area
  - ▭ CVWD Palm Desert Property
  - ▭ Local Model Domain
  - ▭ West Whitewater River Subbasin AOB

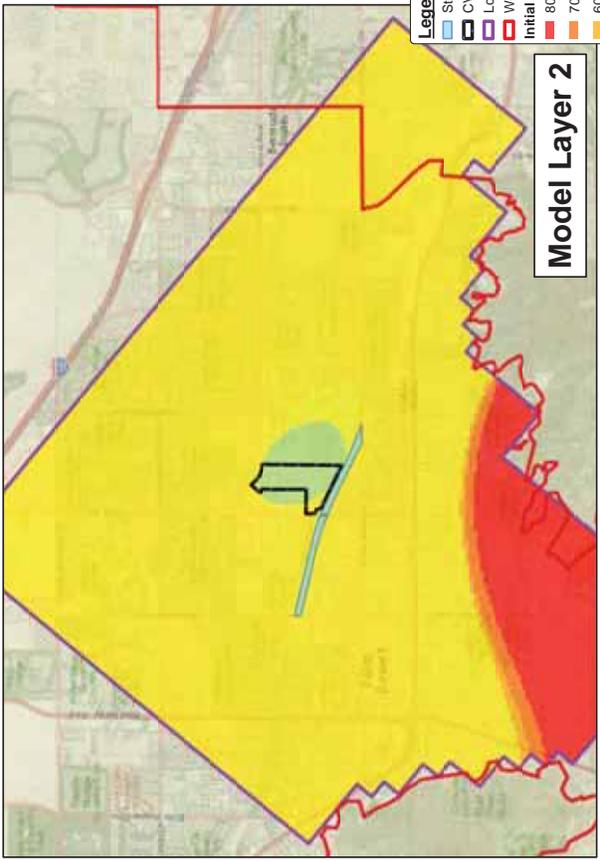
**Note:** Contours represent water levels in feet above (positive values) simulated groundwater elevations under Scenario 1 (baseline) conditions.

<b>Model Layer 1</b>	270 ft (range 250 to 320 ft)
<b>Model Layer 2</b>	80 ft (range 0 to 160 ft)
<b>Model Layer 3</b>	300 ft
<b>Model Layer 4</b>	400 ft

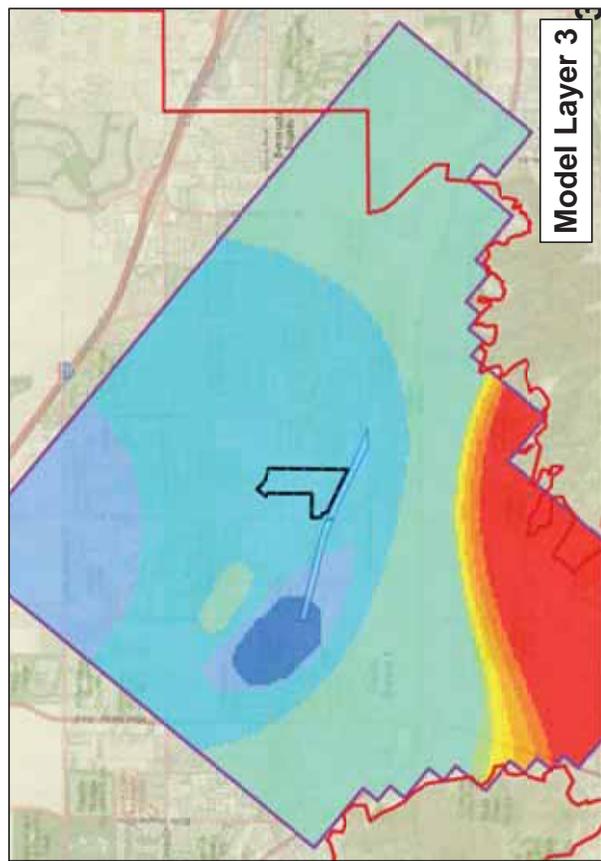




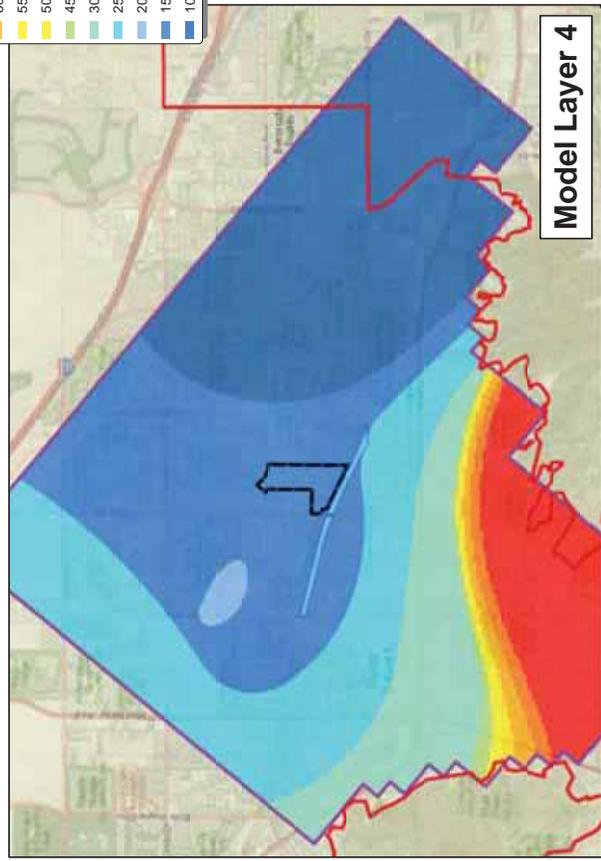
**Model Layer 1**



**Model Layer 2**



**Model Layer 3**



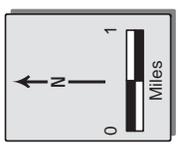
**Model Layer 4**

**Legend**

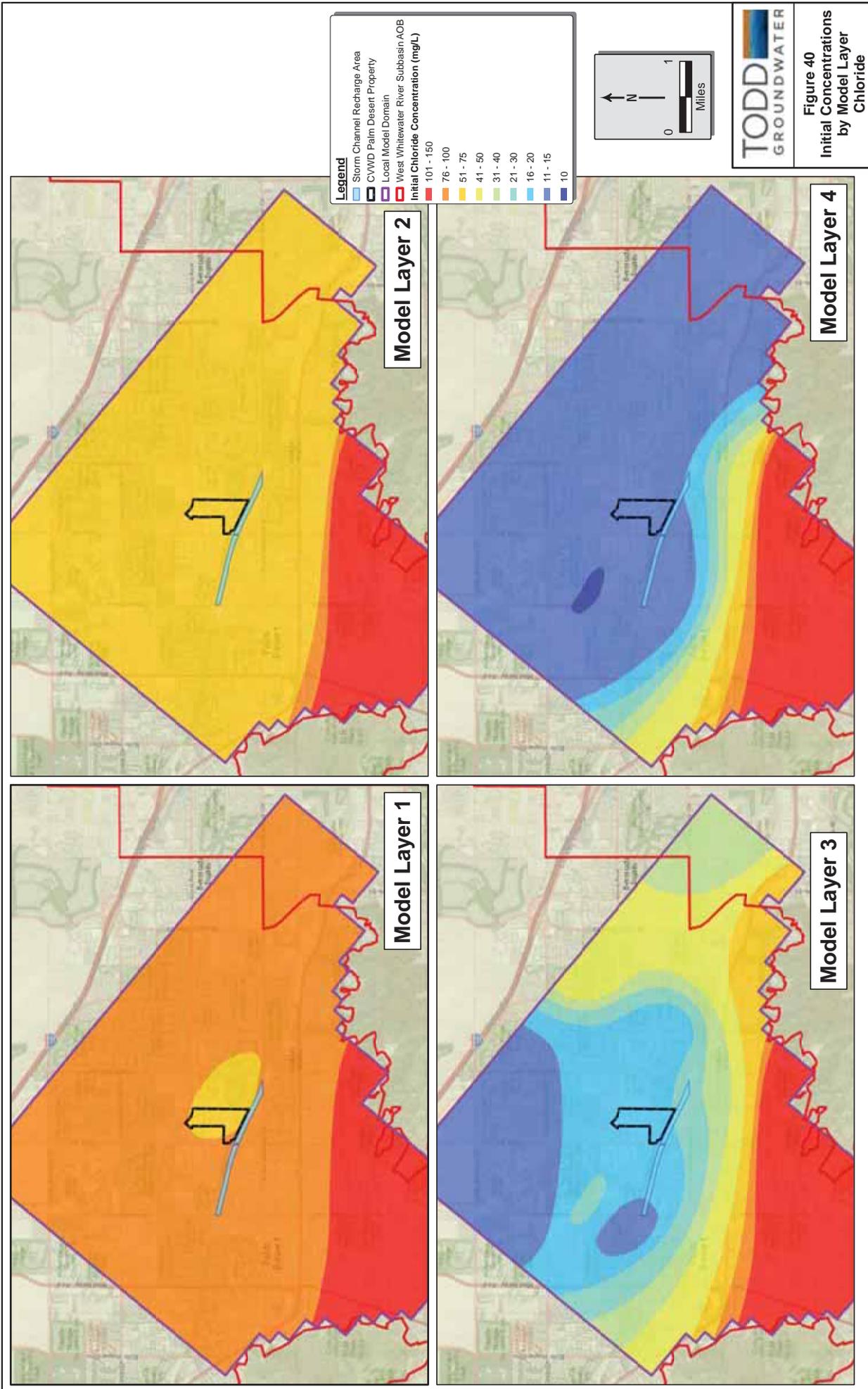
- Storm Channel Recharge Area
- CWWD Palm Desert Property
- Local Model Domain
- West Whitewater River Subbasin AOB

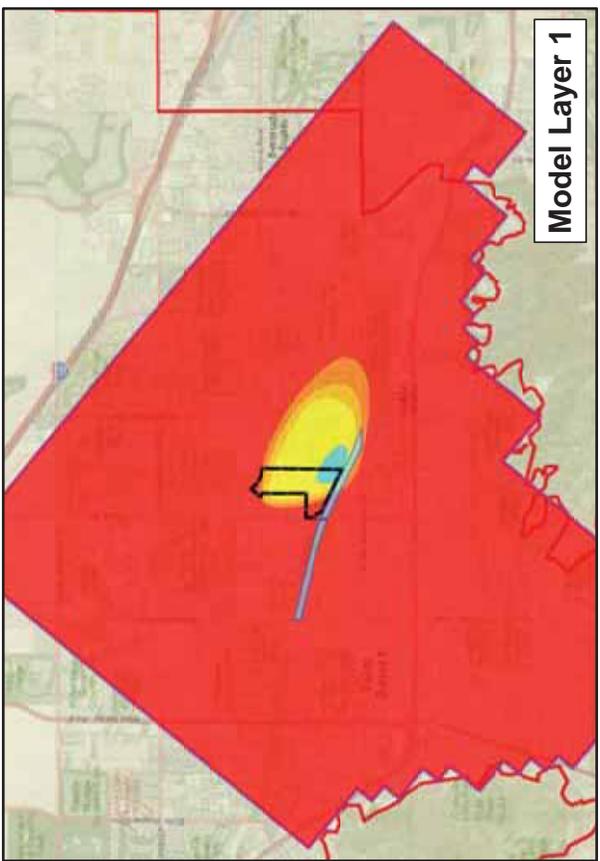
**Initial TDS Concentration (mg/L)**

- 801 - 1000
- 701 - 800
- 601 - 700
- 551 - 600
- 501 - 550
- 451 - 500
- 301 - 450
- 251 - 300
- 201 - 250
- 151 - 200
- 100 - 150

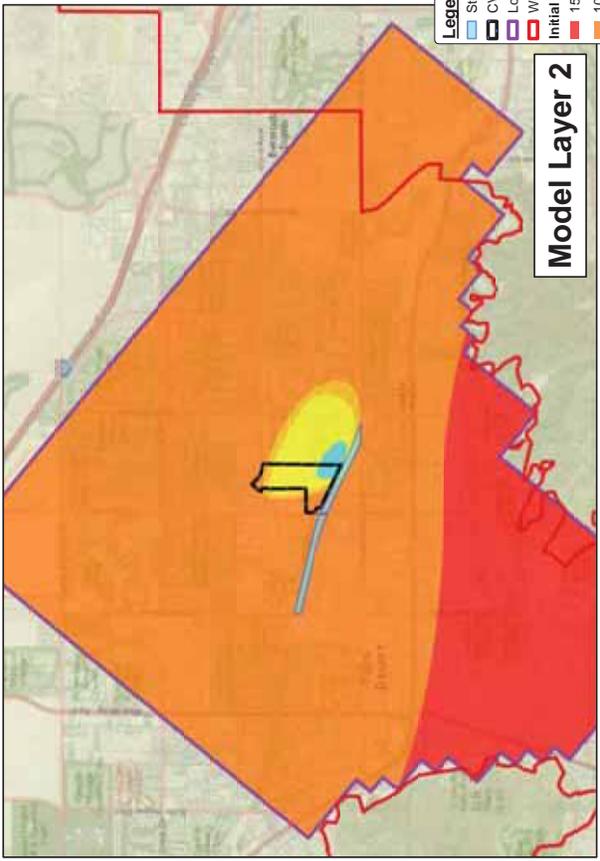


**Figure 39**  
Initial Concentrations  
by Model Layer  
TDS

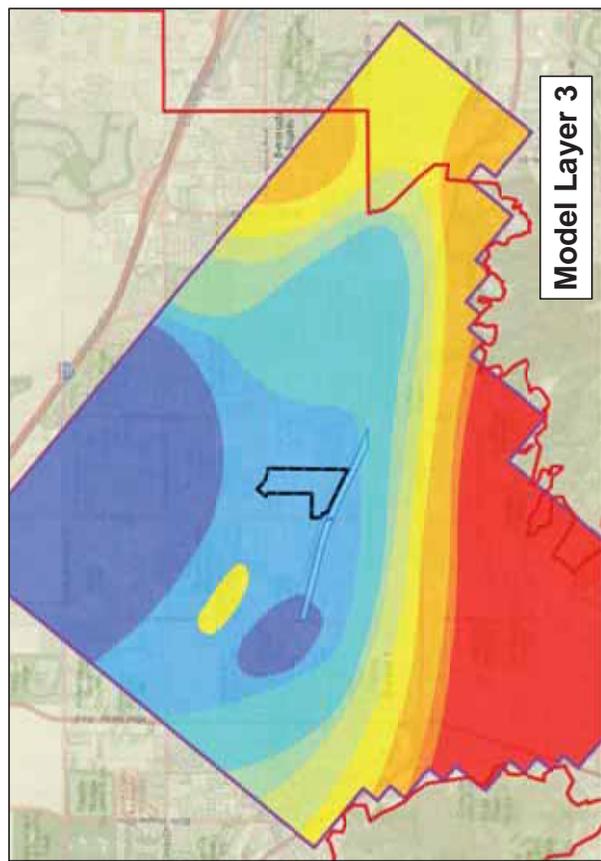




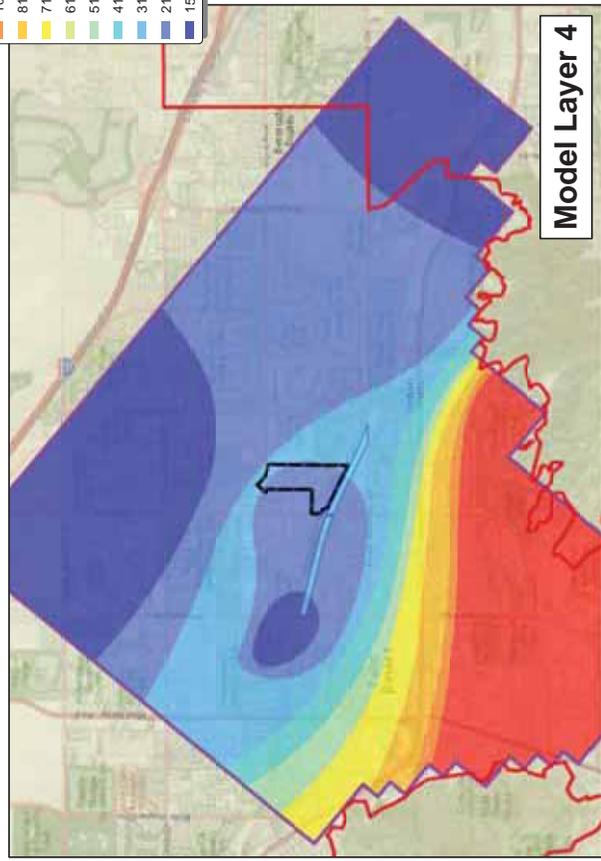
**Model Layer 1**



**Model Layer 2**



**Model Layer 3**



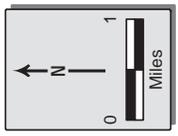
**Model Layer 4**

**Legend**

- Storm Channel Recharge Area
- CWWD Palm Desert Property
- Local Model Domain
- West Whitewater River Subbasin AOB

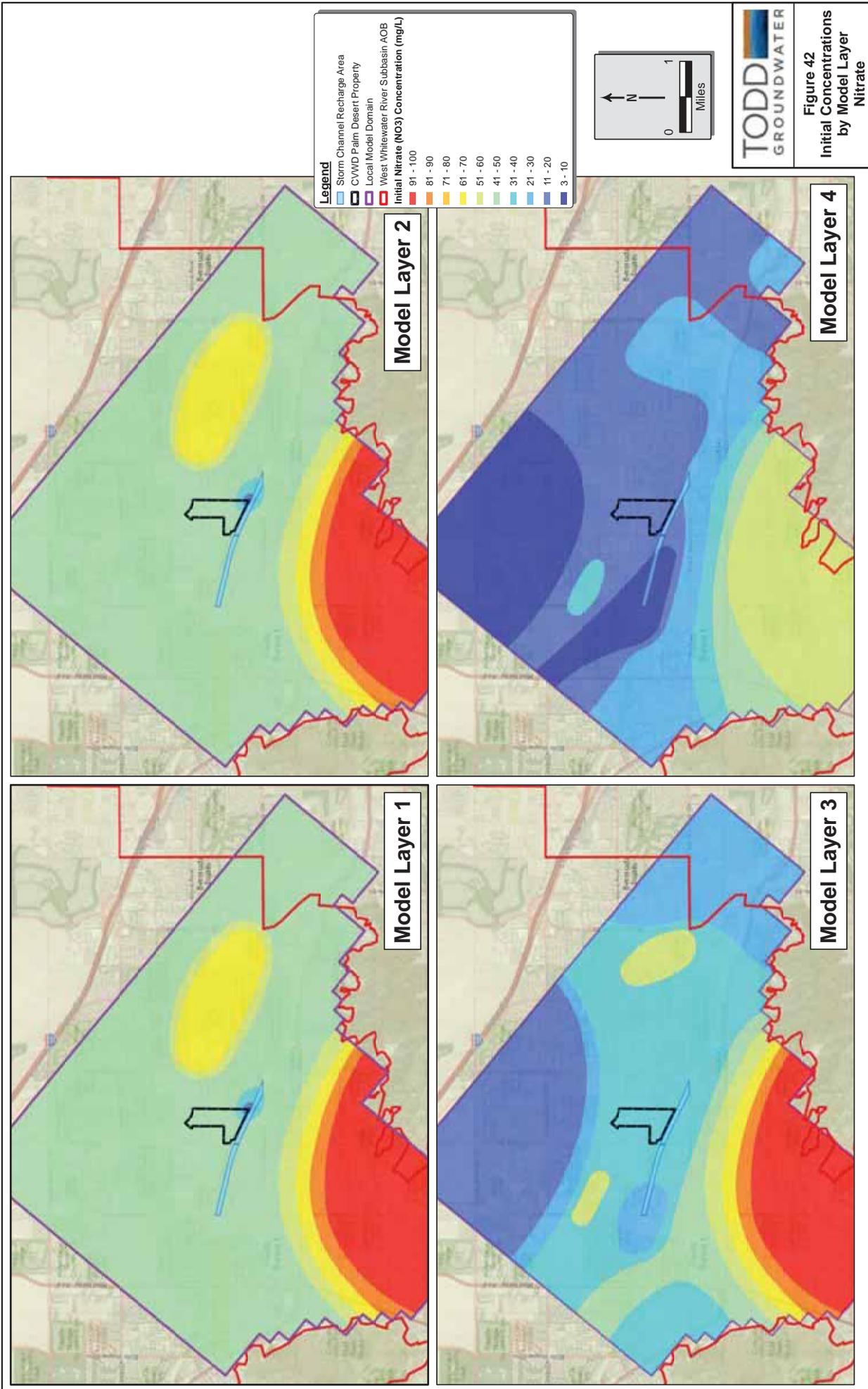
**Initial Sulfate Concentration (mg/L)**

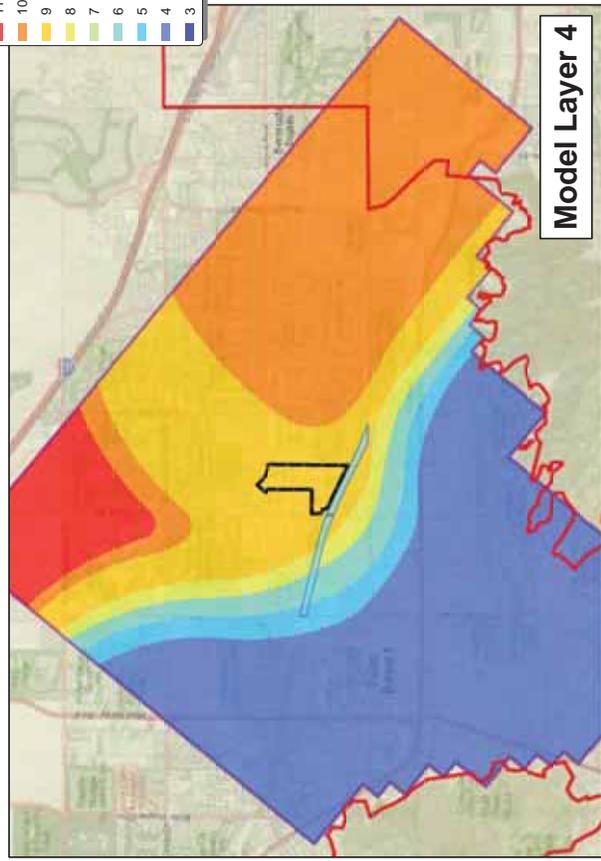
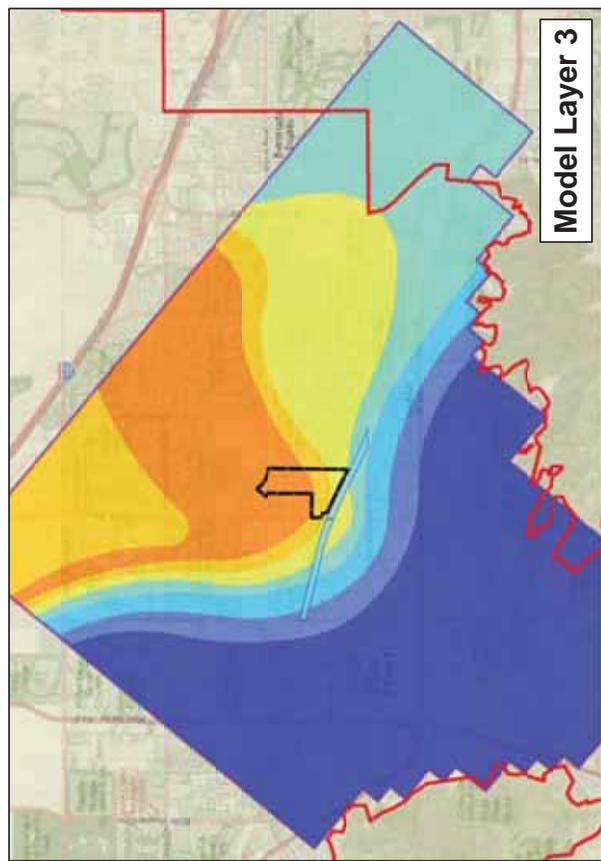
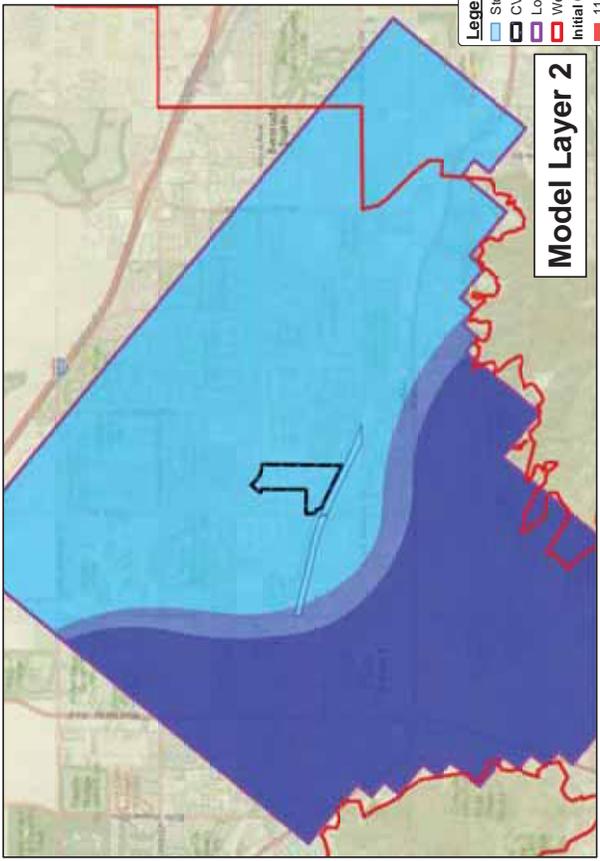
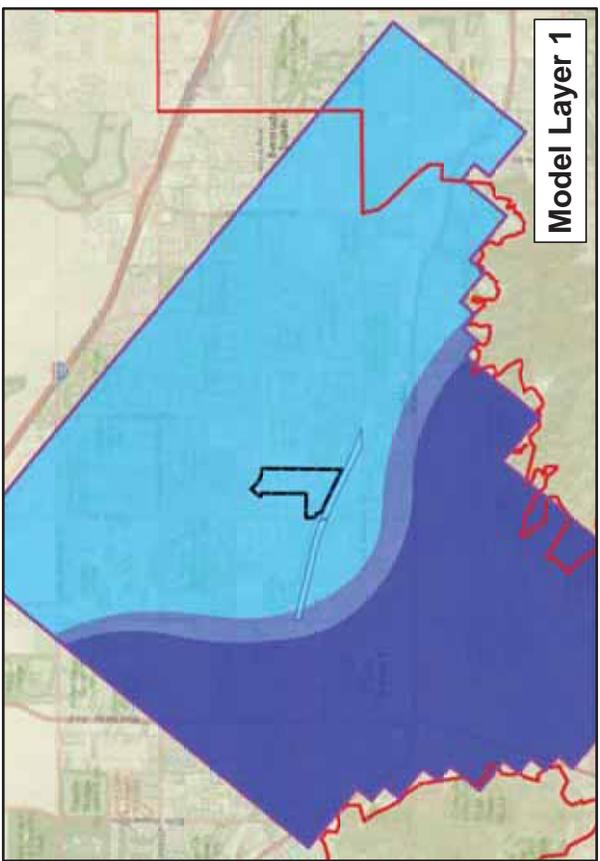
- 151 - 300
- 101 - 150
- 81 - 100
- 71 - 80
- 61 - 70
- 51 - 60
- 41 - 50
- 31 - 40
- 21 - 30
- 15 - 20



**TODD**  
GROUNDWATER

**Figure 41**  
Initial Concentrations  
by Model Layer  
Sulfate



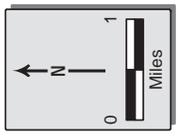


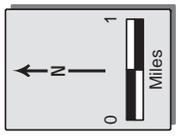
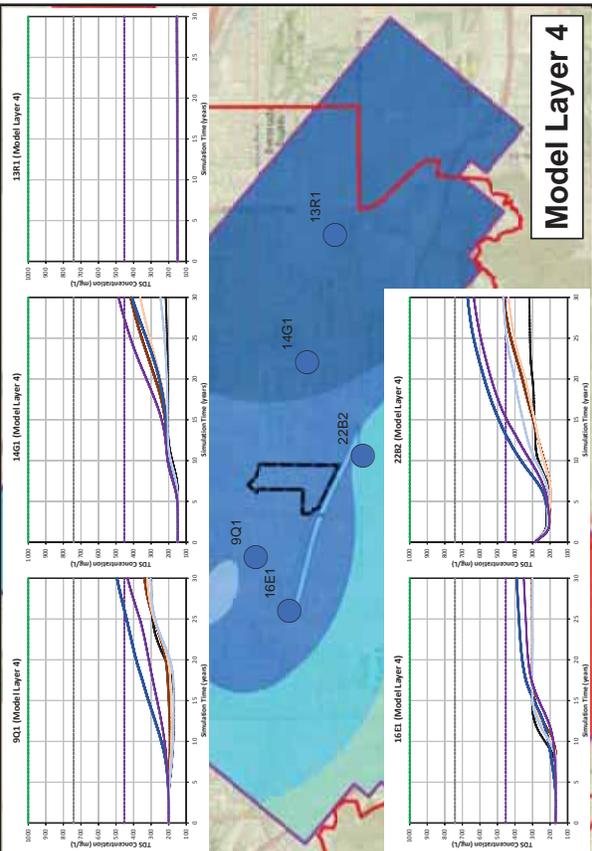
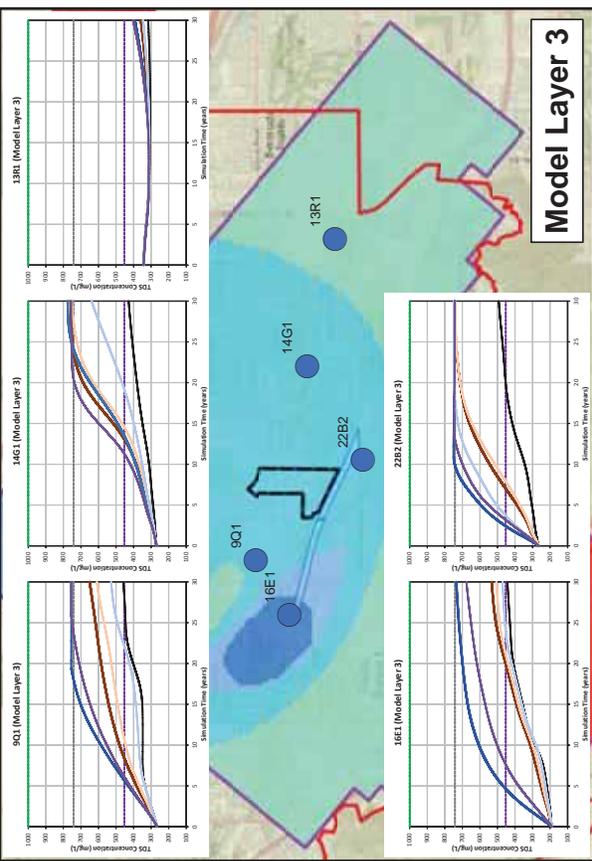
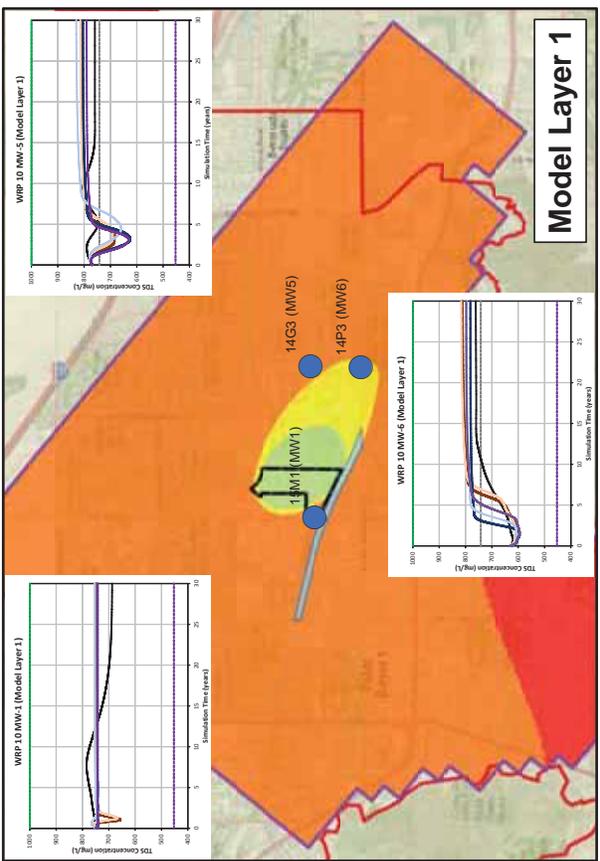
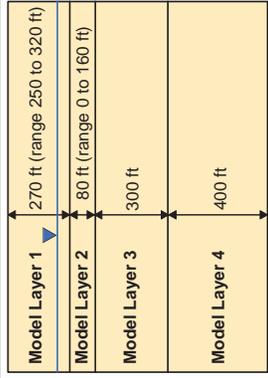
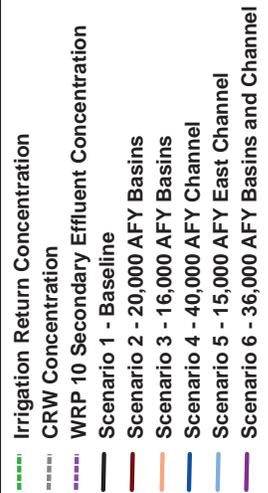
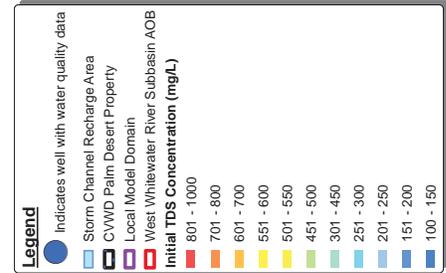
**Legend**

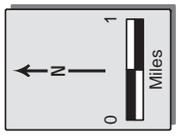
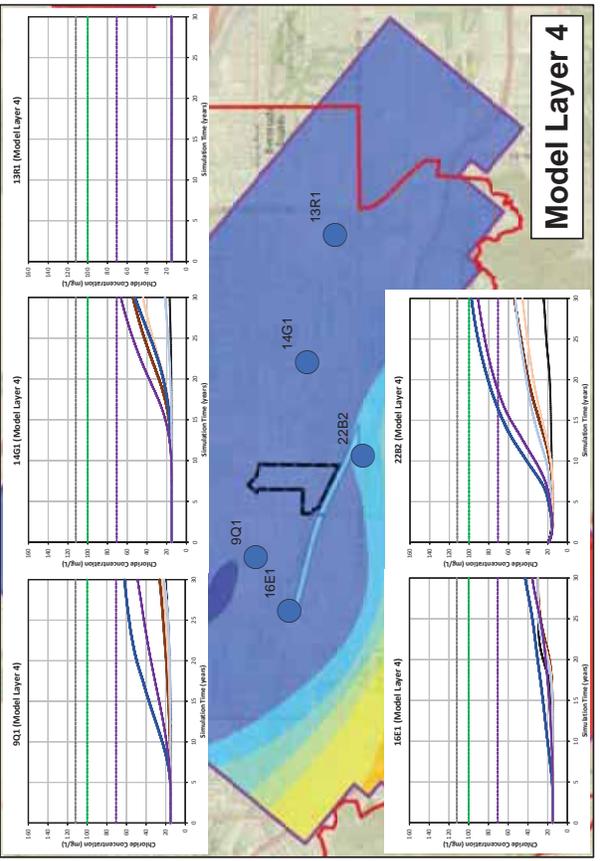
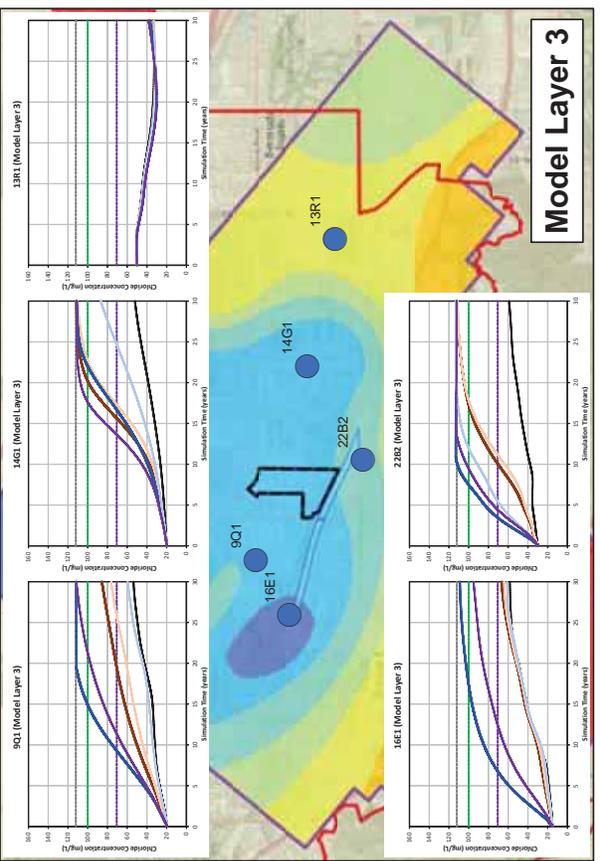
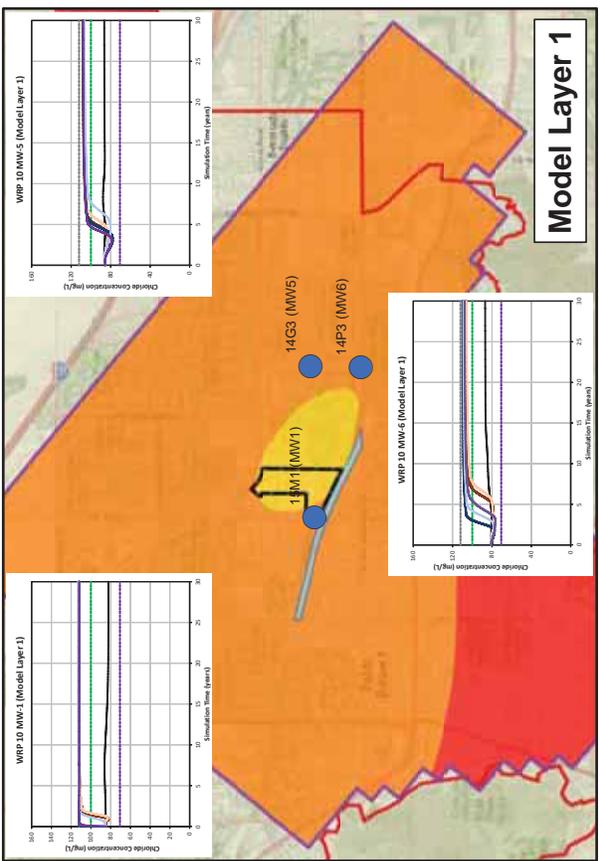
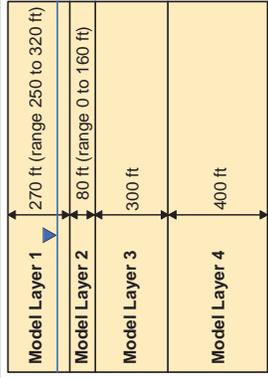
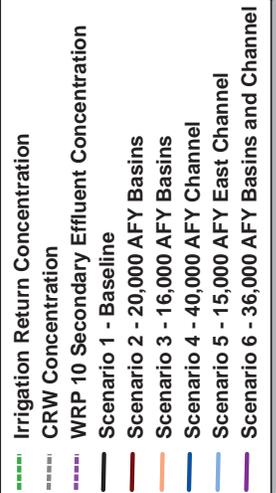
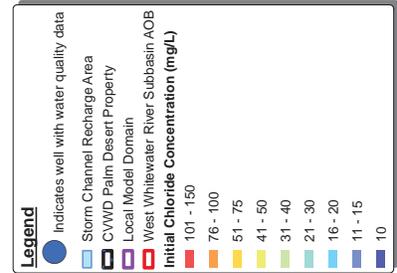
- Storm Channel Recharge Area
- CWD Palm Desert Property
- Local Model Domain
- West Whitewater River Subbasin AOB

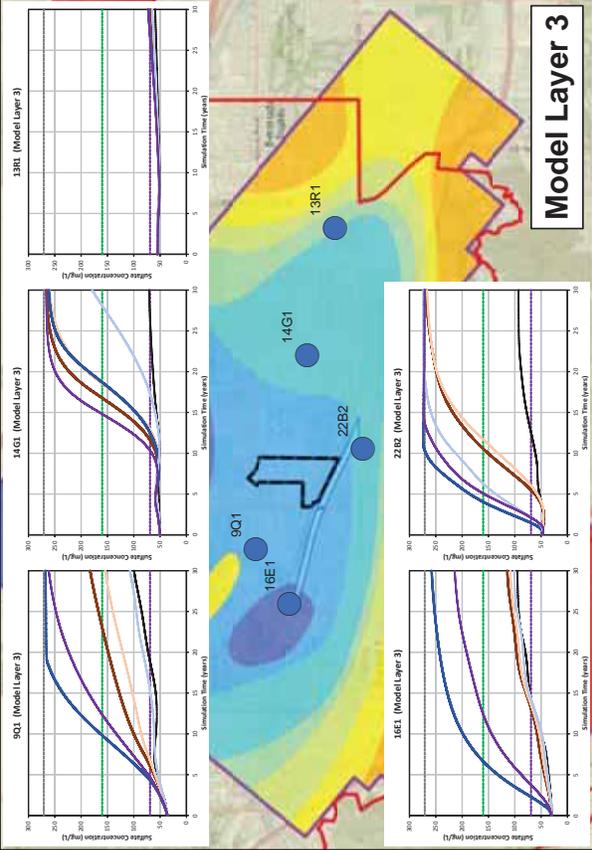
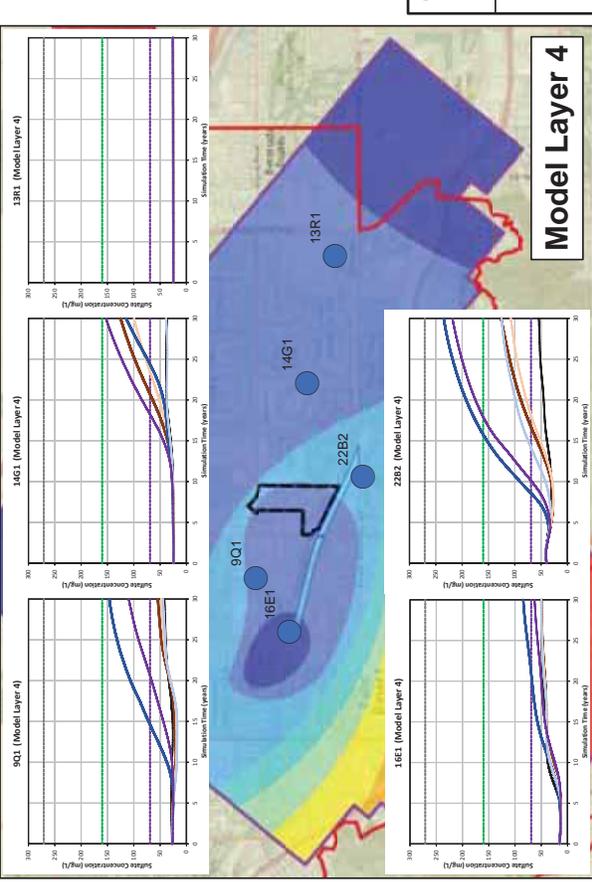
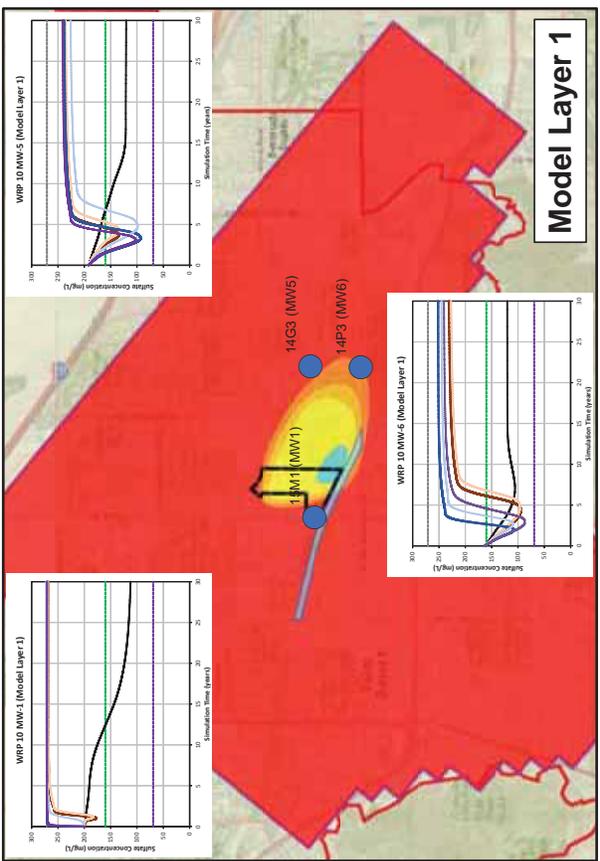
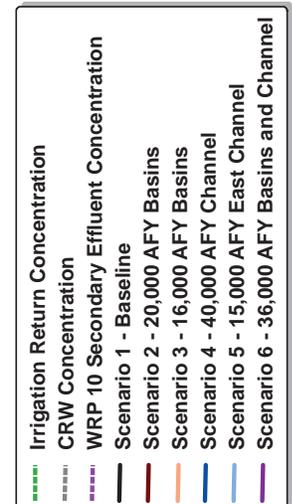
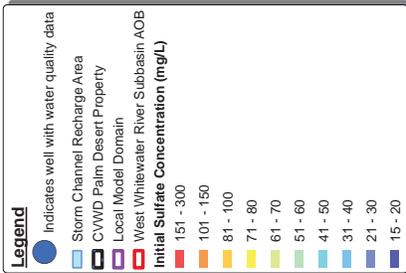
**Initial Chromium-6 Concentration (ug/L)**

11	10	9	8	7	6	5	4	3









**Legend**

- Indicates well with water quality data
- Storm Channel Recharge Area
- CWWD Palm Desert Property
- Local Model Domain
- West Whitewater River Subbasin AOB
- Initial Sulfate Concentration (mg/L)
- 151 - 300
- 101 - 150
- 81 - 100
- 71 - 80
- 61 - 70
- 51 - 60
- 41 - 50
- 31 - 40
- 21 - 30
- 15 - 20

**Irrigation Return Concentration**

**CRW Concentration**

**WRP 10 Secondary Effluent Concentration**

**Scenario 1 - Baseline**

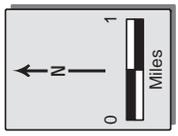
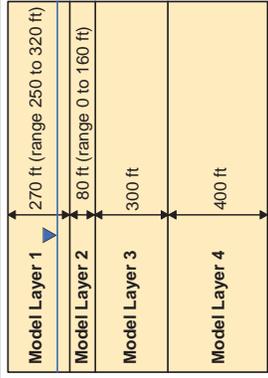
**Scenario 2 - 20,000 AFY Basins**

**Scenario 3 - 16,000 AFY Basins**

**Scenario 4 - 40,000 AFY Channel**

**Scenario 5 - 15,000 AFY East Channel**

**Scenario 6 - 36,000 AFY Basins and Channel**



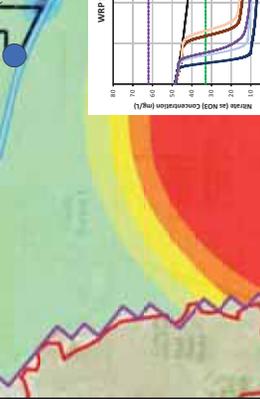
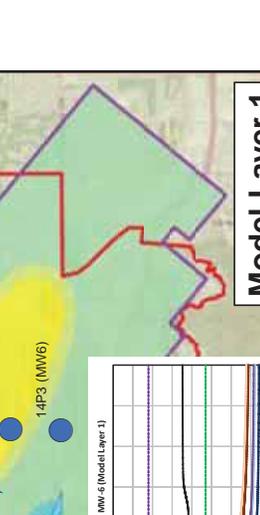
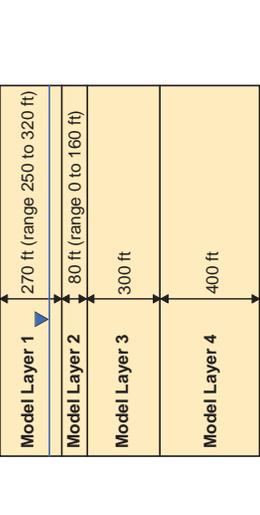
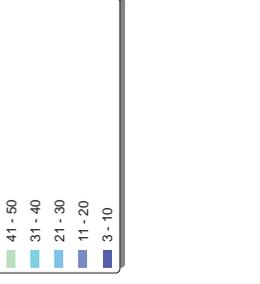
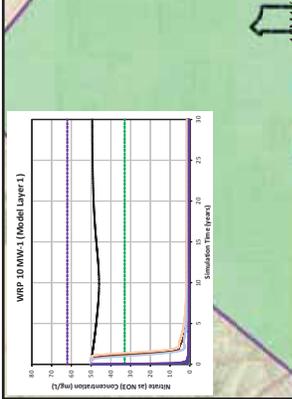
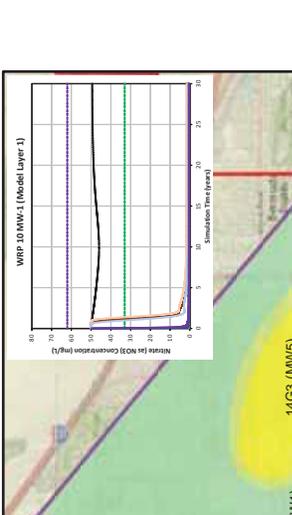
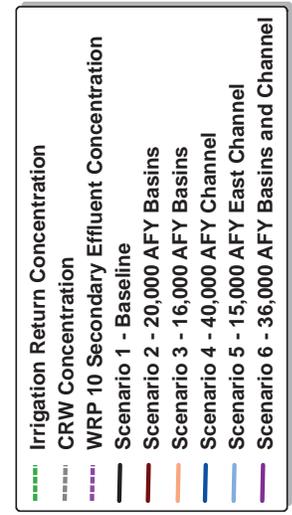
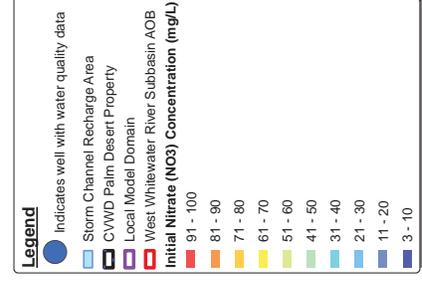
**TODD**  
GROUNDWATER

**Figure 46**  
Simulated  
Time-Concentration  
by Model Layer  
Sulfate

**Model Layer 1**

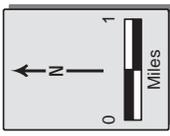
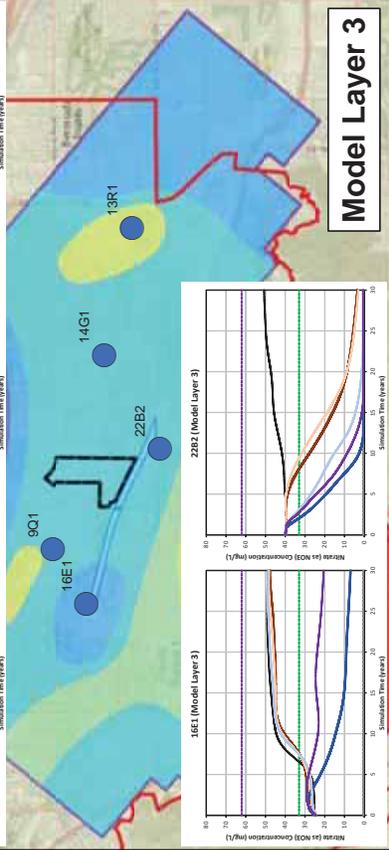
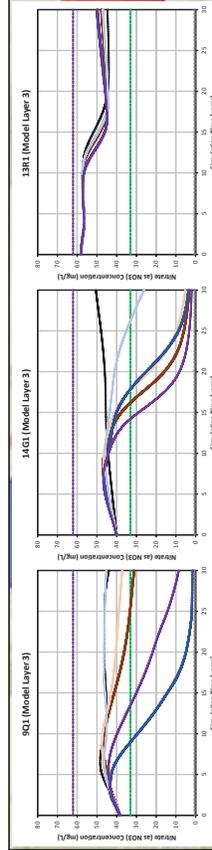
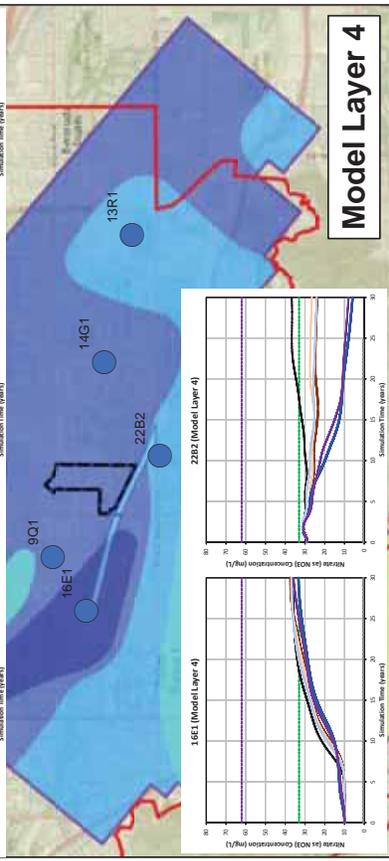
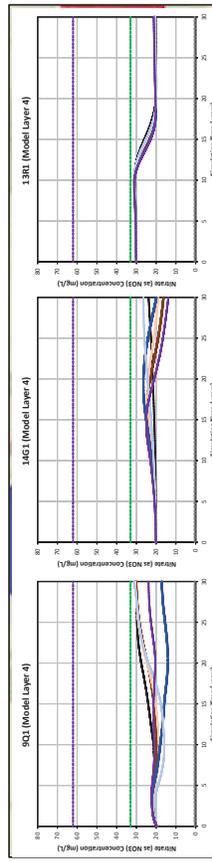
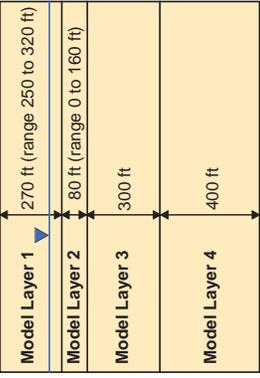
**Model Layer 3**

**Model Layer 4**



- Irrigation Return Concentration**
- Scenario 1 - Baseline
  - Scenario 2 - 20,000 AFY Basins
  - Scenario 3 - 16,000 AFY Basins
  - Scenario 4 - 40,000 AFY Channel
  - Scenario 5 - 15,000 AFY East Channel
  - Scenario 6 - 36,000 AFY Basins and Channel
- CRW Concentration**
- WRP 10 Secondary Effluent Concentration**
- WRP 10 MW-1 (Model Layer 1)**
- WRP 10 MW-4 (Model Layer 3)**

- Legend**
- Indicates well with water quality data
  - Storm Channel Recharge Area
  - CWWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB
  - Initial Nitrate (NO3) Concentration (mg/L)
  - 81 - 100
  - 81 - 90
  - 71 - 80
  - 61 - 70
  - 51 - 60
  - 41 - 50
  - 31 - 40
  - 21 - 30
  - 11 - 20
  - 3 - 10



**TODD**  
GROUNDWATER

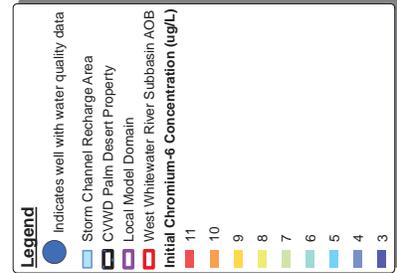
**Figure 47**  
Simulated  
Time-Concentration  
by Model Layer  
Nitrate

**Model Layer 4**

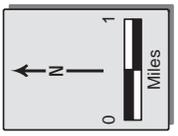
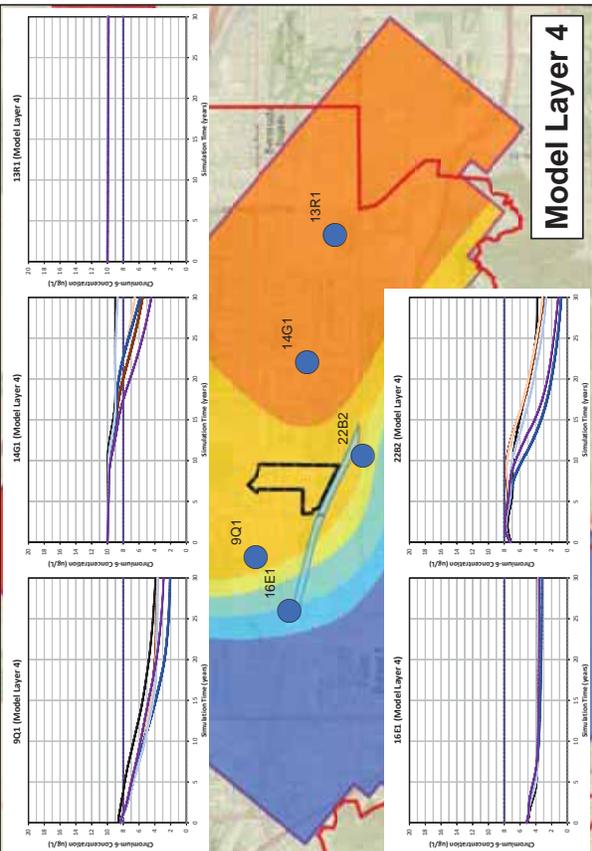
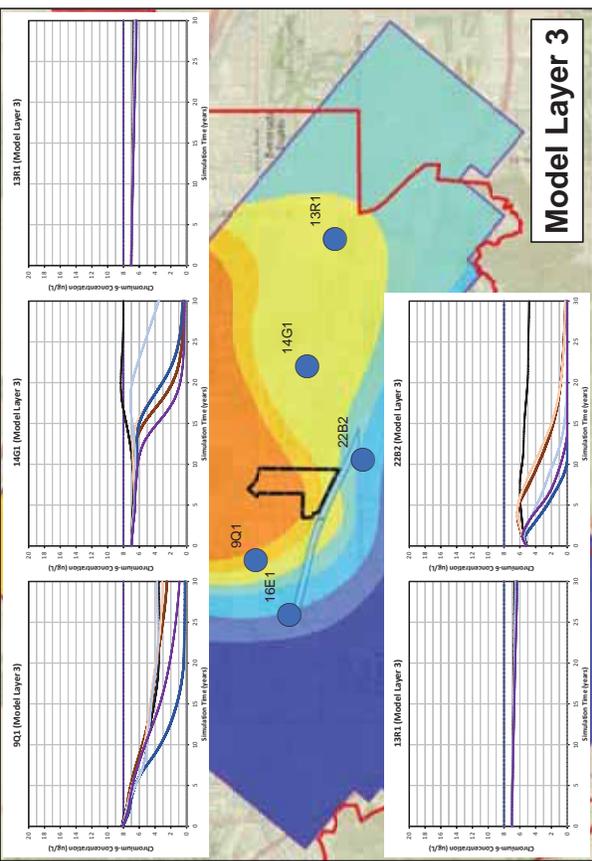
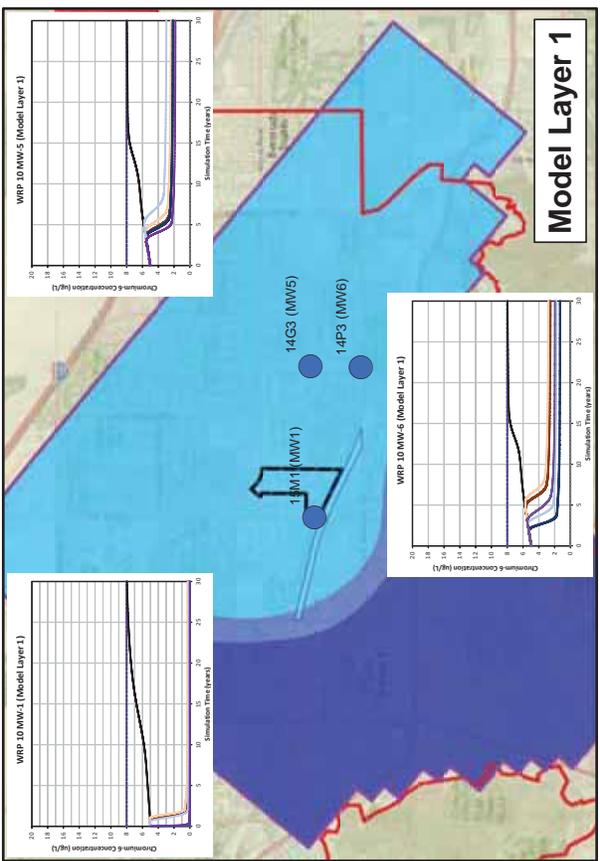
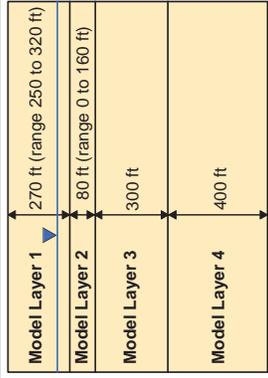
**Model Layer 3**

**Model Layer 1**

**Model Layer 4**



- Irrigation Return Concentration**
- Scenario 1 - Baseline
  - Scenario 2 - 20,000 AFY Basins
  - Scenario 3 - 16,000 AFY Basins
  - Scenario 4 - 40,000 AFY Channel
  - Scenario 5 - 15,000 AFY East Channel
  - Scenario 6 - 36,000 AFY Basins and Channel
- CRW Concentration**
- WRP 10 Secondary Effluent Concentration
  - Scenario 1 - Baseline
  - Scenario 2 - 20,000 AFY Basins
  - Scenario 3 - 16,000 AFY Basins
  - Scenario 4 - 40,000 AFY Channel
  - Scenario 5 - 15,000 AFY East Channel
  - Scenario 6 - 36,000 AFY Basins and Channel
- Legend**
- Indicates well with water quality data
  - Storm Channel Recharge Area
  - CWWD Palm Desert Property
  - Local Model Domain
  - West Whitewater River Subbasin AOB
  - Initial Chromium-6 Concentration (ug/L)
- 11, 10, 9, 8, 7, 6, 5, 4, 3



## **Appendix A**

### **Memorandum - Coachella Valley Water District Palm Desert Recharge Feasibility Study – Concept Alternative Analysis (CWE, September 28, 2016)**



## MEMORANDUM

To: Edwin Lin, P.G., C.Hg.

From: Ben Willardson, Ph.D., P.E., D.WRE  
CWE

Date: September 28, 2016

Subject: **Coachella Valley Water District Palm Desert Groundwater Replenishment Feasibility Study – Concept Alternative Analysis**

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### 1. Background

The main goal of the **Coachella Valley Water District Palm Desert Groundwater Replenishment Feasibility Study** is to provide performance evaluations for recharging Colorado River water (CRW) in the Palm Desert area for the Coachella Valley Water District (CVWD). CWE has worked with Todd Groundwater to propose and evaluate alternatives that involve the recharge of CRW at the CVWD Palm Desert property located at 75525 Hovley Ln East in Palm Desert, CA and in the Whitewater River Storm Channel (WWR-SC) south of the property. Located on the southern end of the CVWD Palm Desert property is CVWD's Water Reclamation Plant No. 10 (WRP 10), which currently consists of an activated sludge treatment plant (providing secondary treatment of wastewater), a tertiary wastewater treatment plant, six effluent ponds, fifteen active and three former percolation ponds (or basins), and a lined back feed reservoir currently used for temporary CRW storage prior to delivery to CVWD irrigation customers.

**Figure 1** shows the fifteen active basins and back feed reservoir that were evaluated for this feasibility study. It is noted that the three former percolation ponds in the northwest portion of the Palm Desert property may be developed for storage warehouses but are also potentially available for CRW recharge. To provide a conservative estimation of recharge potential at the Palm Desert property, the three former percolation ponds were not included in the development and analysis of concept alternatives. Also shown on Figure 1 is the reach of the WWR-SC proposed for CRW recharge, which is approximately 2 miles long and 300 feet wide. The banks are concrete-lined with a soil cover that provides a more natural look.



**Figure 1 Project Site and Existing Features**

This memorandum discusses the existing conditions of the project area and several proposed concepts for expanding and improving the recharge system. Two concepts for reconfiguring the existing basins involve removing internal levees to increase the recharge area within the same basin footprint. The preliminary concepts for the Whitewater River include use of sugar berms, flashboard dams, or inflatable rubber dams to partition the reach into basins. The memorandum provides preliminary cost estimates for construction, operations, and maintenance of the proposed facilities. The advantages and disadvantages regarding hydraulic properties, engineering feasibility and project benefits are summarized for each concept. The comparison summary table provides guidance for use in determining the optimal recharge design concept.

## **2. Existing Conditions**

The existing percolation ponds at the CVWD Palm Desert property have been in operation since the 1970s and are currently used for discharge/disposal of secondary effluent from the Water Reclamation Plant No. 10 (WRP 10). CVWD is updating its Non-Potable Water Master Plan and plans to expand the tertiary treatment facility at WRP 10 to increase recycled water deliveries and eliminate onsite discharge/disposal of secondary effluent. Future onsite storage requirements for recycled water will determine which of the percolation ponds (including possibly the back feed reservoir) may be repurposed for CRW recharge. The WWR-SC is not currently being used for recharge in the area. **Table 1** shows the existing project site with the existing recharge basin system that includes 15 percolation ponds and 1 back feed reservoir. The project site also includes a reach of the WWR-SC between Portola Avenue and Fred Waring Drive that may be used for groundwater recharge. **Table 1** shows the existing recharge surface areas of the basins and main channel bed between the concrete-lined banks.

Table 1 Existing Basin ID & Area			
Basin ID	Area (acre)	Basin ID	Area (acre)
9W	2.37	5W	1.16
9E	1.68	5E	1.13
8W	1.82	4W	1.12
8E	1.74	4E	1.33
7W	1.33	3W	1.17
7E	1.37	2W	1.35
6W	1.37	1W	1.28
6E	1.32	WWR-SC	92.90

The water is expected to percolate at varying rates between 1 to 3 feet per day, depending on deposited fine sediment, soil saturation, biofouling, and the local groundwater level. Basin recharge effectiveness is also influenced by the water supply availability and maintenance work. The major water source of the existing basin system is the WRP10 secondary effluent discharge. **Table 2** shows the WRP 10 monthly discharge record from 2004 to 2015. As shown, water supply during summer months is expected to be much less than during winter months because the reclaimed water is used for watering golf courses. **Table 3** shows the estimated time to fill the basins with potential percolation rates at 1 to 3.5 ft/day, based on the existing basin depths and a constant Mid Valley Pipeline (MVP) CRW inflow of 100 cfs. The percolation rate over the basin surface area is much lower than the water supply flow rate. The time to fill the basins is about 1.1 days under the optimal operational situation with full delivery and the highest expected infiltration rates. The filling time will increase with lower inflow rates and decrease as sediments or algae accumulate in the basins.

Table 2 WRP 10 Secondary Effluent Discharge to Basins (acre-ft/day)												
Year/Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2004	N/A	28.9	26.4	23.9	14.1	11.2	8.4	12.0	18.2	20.4	26.8	26.2
2005	33.7	33.7	27.0	4.6	4.4	1.2	5.0	8.2	16.0	19.3	24.5	29.4
2006	22.8	21.0	16.0	15.4	6.2	3.4	1.5	1.9	14.0	15.6	16.6	23.0
2007	21.5	19.1	14.7	13.6	10.9	7.5	0.3	1.2	8.8	13.1	16.5	23.8
2008	26.1	24.3	20.2	10.6	4.1	1.3	2.5	2.5	9.8	9.3	14.3	22.0
2009	21.3	22.9	14.8	8.4	2.3	0.0	9.5	18.1	11.0	10.2	11.2	19.7
2010	21.5	20.4	13.7	6.1	1.7	0.0	0.0	0.5	7.5	17.3	12.2	22.5
2011	18.8	21.1	12.4	2.3	0.5	0.0	0.8	1.8	10.0	10.1	18.1	22.2
2012	20.2	21.0	22.6	19.5	10.3	0.7	2.1	7.0	5.4	7.4	13.5	21.5
2013	21.3	19.7	13.2	6.6	2.4	2.5	0.4	4.3	11.8	6.4	17.2	18.3
2014	17.4	16.1	13.6	8.2	0.4	0.0	0.0	0.0	4.7	7.3	14.0	20.7
2015	18.3	16.7	10.6	4.0	3.4	1.3	4.0	2.3	9.9	10.9	10.3	13.8
Monthly Average	22.1	22.1	17.1	10.3	5.1	2.4	2.9	5.0	10.6	12.3	16.3	21.9
Historical High	33.7	33.7	27.0	23.9	14.1	11.2	9.5	18.1	18.2	20.4	26.8	29.4
Historical Low	17.4	16.1	10.6	2.3	0.4	0.0	0.0	0.0	4.7	6.4	10.3	13.8

The existing reach of the WWR-SC between Portola Avenue and Fred Waring Drive is relatively flat with a slope of 0.0025 ft/ft. The channel is nearly dry all year. There are no transverse structures within the section to hold water and allow percolation. As a result, the existing recharge effectiveness of the channel system is limited compared to capacity. The existing system for recharging groundwater within the river is highly dependent on the number of storms and the storm duration, which control the amount of time that flows are in the channel to percolate.

The maintenance cost estimates for this memorandum were estimated based on CVWD's maintenance activities for Thomas E. Levy (TEL) Groundwater Replenishment Facility (TEL-GRF). The TEL-GRF has 41 basins separated into eight cells. They are operated using CRW. The basins start out with a percolation flow rate of approximately 100 cfs within the 85 acre system, which is roughly 1.5 ft/day. The basins are cleaned twice a year. The first four months of the cycle have limited reduction in percolation rates. After four months the infiltration rates start to decline and by the end of the sixth month the flow rates are half of the initial flow rates. When this happens, the basins are dried out and cleaned. The basins are operated at a depth of 2 to 4 feet and take two weeks to dry if the temperature is higher than 90 degrees Fahrenheit. The basins take three weeks to dry if the temperature is less than 90 degrees. Each cell of approximately 15 acres takes ten days to two weeks to clean depending on staff availability. The basins are cleaned by pushing the accumulated silt up onto the banks of the basin berms. This has been done to reduce infiltration between basins. This procedure also prevents fines from migrating into the infiltration media and reduces maintenance costs for hauling away materials.

The time and cost for maintenance activities at various basins can be quite different. The majority of maintenance work includes removal of silt on the basin bottom and treatment/delivery of the waste. The current maintenance routines include removal of silts from the basin bottoms and placement on the berms with some other maintenance related to weather impacts. The costs for maintenance are broken into three components: labor, equipment, and weather related cleanup and repair.

The estimated average semiannual cost for the existing maintenance routines for the TEL basins is about \$2,153 per acre based on laborers and equipment such as loaders, dozers, trucks and trailers. According to TEL maintenance record, the maintenance labor cost accounts for \$511 per acre, roughly 24 percent of the maintenance cost. Equipment and fuel cost is roughly 71% of the cost at \$1,534 per acre. The routine maintenance work is commonly performed every six months or twice a year. The total annual maintenance cost per acre for the TEL basins is \$4,300. Operation of the basins is estimated to cost another \$1,200 per acre. The total operations and maintenance costs is estimated to be \$5,500 per acre.

Another minor cost is related to storm event clean up and repairs. The cost is estimated 5% of the total maintenance cost at \$108 per acre for the TEL data. The yearly maintenance cost estimate for the existing basin conditions is shown in **Table 4**.

Table 3 Required Time to Fill Existing Basins (days)							
Existing System		Percolation Rate (ft/day)					
Basin ID	Depth (ft)	1.0	1.5	2.0	2.5	3.0	3.5
1W	7	0.045	0.046	0.046	0.046	0.046	0.046
2W	14	0.096	0.096	0.097	0.097	0.097	0.098
3W	8	0.047	0.048	0.048	0.048	0.048	0.048
4E	11	0.074	0.075	0.075	0.075	0.075	0.076
4W	8	0.045	0.046	0.046	0.046	0.046	0.046
5E	10	0.057	0.057	0.058	0.058	0.058	0.058
5W	10	0.059	0.059	0.059	0.059	0.060	0.060
6E	11	0.074	0.074	0.074	0.074	0.075	0.075
6W	13	0.090	0.091	0.091	0.091	0.092	0.092
7E	13	0.090	0.091	0.091	0.091	0.092	0.092
7W	10	0.068	0.068	0.068	0.068	0.068	0.069
8E	9	0.080	0.080	0.080	0.081	0.081	0.081
8W	10	0.093	0.093	0.093	0.094	0.094	0.095
9E	7	0.060	0.060	0.060	0.061	0.061	0.061
9W	10	0.121	0.122	0.122	0.123	0.124	0.125
<b>Total</b>		1.100	1.104	1.108	1.112	1.117	1.121

Table 4 Cost Estimate for Existing Basin System Maintenance		
Item Description	Quantity	Cost
<b>Annual Maintenance</b>		
2" Silt Removal – Labor (\$511/acre)	21.5 acre	\$11,007
2" Silt Removal – Equipment & Fuel (\$1534/acre)	21.5 acre	\$33,042
Weather Event Clean Up & Repair (\$108/acre)	21.5 acre	\$2,326
<b>Total:</b>		<b>\$46,376</b>

### 3. Basin System Concepts

Improvements to the existing basin system at the Palm Desert property and increased basin capacity through use of the WWR-SC have been investigated to evaluate the most cost effective system improvements. This section covers the proposed concepts for improving the existing recharge basins. Section 4 details concepts for developing basins within the WWR-SC. Several key factors influence the amount of water that can be recharged through surface basins. These factors include percolation rate, the size and volume of infiltration basins, groundwater levels, and site geology. Infiltration rates are based on native materials and geology, but can also be influenced by maintenance of basins. The size and volume of basins affect the recharge surface and how much water can be captured and stored. Local groundwater levels are influenced by rainfall and local recharge. Geology is determined by the location of the project. The most easily influence variables to modify in surface water recharge basins is the size of the basins and maintaining native recharge rates through proper maintenance of the system.

Two design concepts for the system were developed and evaluated. Both concepts involve removal of existing berms to increase the area of the basin. We are also proposing removal of 1 to 2 feet of material from the basin bottoms to create more storage volume and remove any sediment that may have been compromised through infiltration of silt over the last 40 years of operation. Berms will divide the basins and serve as access and maintenance roads. The following subsections illustrate the layouts of the two concepts. The engineering, construction, and maintenance costs required to enhance the basins and to sustain recharge operations for each concept are provided.

#### 3.1 Concept 1

**Figure 1** shows the first concept that converts the existing basin system with 15 basins into a new system with 4 combined basins. Proposed Basins #1 #2 and #3 have the same length of 1,138 feet and run parallel to each other from north to south. Basin #4 is located at the southwest corner of the basin system and is slightly smaller than the other three basins because it does not modify the Backfeed Basin. The new areas of the four proposed basins are shown in **Table 5**.

Detailed basin layouts and facilities are provided in **Attachment A**. The levees designed to separate the basins are 9 feet in height and 12 feet in top width with 3:1 side slopes. **Table 6** shows the time of 1.4 to 1.6 days required to fill the basins (9 feet depth) using MVP inflow at the rate of 100 cfs. The table represents a range of infiltration rates, beginning from 1.0 feet per day to 3.5 feet per day. Soil materials existing in the basins allow an average percolation rate of 1.75 feet per day. Estimating recharge to be 304 days per year, the volume of water replenished per year is approximately 16,000 acre-feet.

**Table 7** shows the engineering cost for Concept 1 including excavation, grading, new structure installation, and maintenance. Maintenance requirements and routine establishment are heavily influenced by the quality of the water, the length of time the water remains in the ponds, and other operational constraints. Tertiary treated recycled water often has nutrients and suspended solids that result in algal growth and biofouling. Imported water has some suspended solids, while stormwater has significant suspended solids. The quality of water used significantly impacts both infiltration rates and maintenance routines to maintain optimal percolation. Silt deposition of the top 2 inches of the bed is expected to be removed to enhance percolation efficiency in each maintenance plan.

The cost per acre for operations and maintenance described above was used to evaluate costs for three potential alternatives since it will influence the lifecycle costs of the project. The first alternative is to maintain the basin using the current maintenance routine with two cleanouts each year and leaving the silt on the basin sides.

The second alternative is to clean the basins once a quarter and leave the silts on the berms. This would prevent any degradation of infiltration capacity, but would require more maintenance funding.

The third alternative is clean the basins twice a year and remove the silts for offsite disposal. This removes the silts, to prevent them from migrating back to the basin bottom during operation and potentially into the soil matrix. The removal to offsite disposal will increase maintenance costs over the current routing and the silts will not be available for controlling inter-berm seepage. These options cover the range of expected maintenance routines.

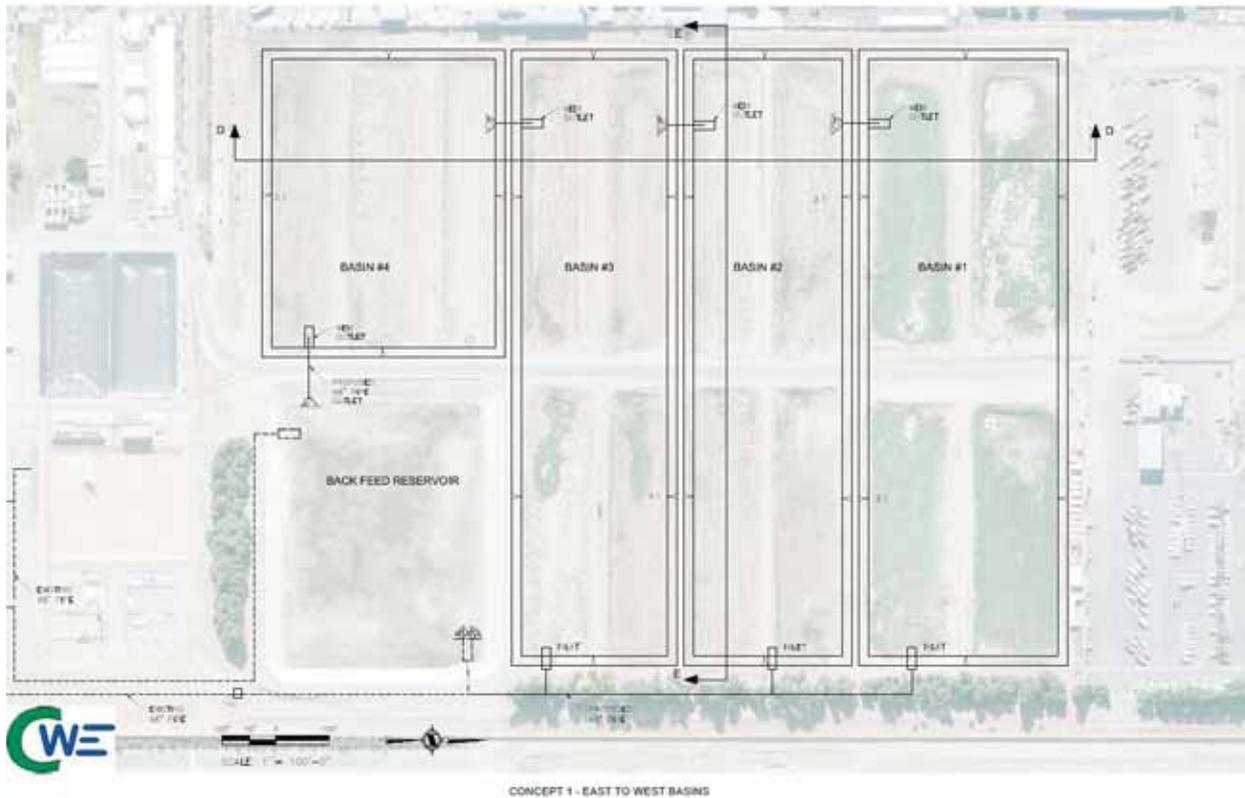


Figure 1 Concept 1 Layout and Structural Features

Table 5 Concept 1 Basin IDs & Areas			
Basin ID	Area (acre)	Basin ID	Area (acre)
Design Basin #1	9.7	Design Basin #3	7.4
Design Basin #2	8.1	Design Basin #4	5.6

<b>Table 6 Required Time to Fill Concept 1 Basins (days)</b>						
<b>Existing System</b>	<b>Percolation Rate (ft/day)</b>					
<b>Basin ID</b>	<b>1.0</b>	<b>1.5</b>	<b>2.0</b>	<b>2.5</b>	<b>3.0</b>	<b>3.5</b>
Basin #1	0.46	0.47	0.49	0.50	0.51	0.53
Basin #2	0.38	0.39	0.40	0.41	0.42	0.43
Basin #3	0.35	0.35	0.36	0.37	0.38	0.38
Basin #4	0.26	0.26	0.27	0.27	0.28	0.28
<b>Total</b>	1.45	1.48	1.51	1.55	1.58	1.62

<b>Table 7 Basin System Concept 1 Cost Estimate</b>		
<b>Item Description</b>	<b>Quantity</b>	<b>Cost</b>
<b>Construction</b>		
Mobilization (10%)	1 LS	\$117,000
Install Weir Box (\$15,000 each)	4 EA	\$60,000
3" AC Access Road (\$80/ton)	649 TONS	\$51,890
4" CMB Base (\$20/cubic yard)	427 CY	\$8,550
Rip-Rap (Light Class) Energy Dissipator (\$100/cubic yard)	112 CY	\$11,200
48" RCP (\$285/linear foot)	1284 LF	\$365,940
Excavation (\$12/cubic yard)	54,490 CY	\$653,890
Labor of Construction	1 LS	\$8,700
Contingency (25%)	1 LS	\$320,000
<b>Subtotal:</b>		<b>\$1,597,170</b>
<b>Alternative 1 Annual Maintenance</b>		
Basin Operation and Maintenance (\$5,500/acre)	30.7 acre	\$168,850
Weather Event Clean Up & Repair (\$108/acre)	30.7 acre	\$6,631
<b>Subtotal:</b>		<b>\$175,481</b>
<b>Alternative 2 Annual Maintenance</b>		
Basin Operation and Maintenance (\$11,000/acre)	30.7 acre	\$337,700
Weather Event Clean Up & Repair (\$108/acre)	30.7 acre	\$13,262
<b>Subtotal:</b>		<b>\$350,962</b>
<b>Alternative 3 Annual Maintenance</b>		
Basin Operation and Maintenance (\$5,500/acre)	30.7 acre	\$168,850
2" Silt Offsite Removal (\$25/cubic yard)	8254 cy	\$412,700,
Weather Event Clean Up & Repair (\$108/acre)	30.7 acre	\$6,631
<b>Subtotal:</b>		<b>\$588,181</b>

### 3.2 Concept 2

As shown in **Figure 2**, Concept 2 also combined 15 existing basins into four larger basins (Basin #1, #2, #3 and #4). This concept differs from Concept 1 in terms of the allocation and array of new basins. The existing Backfeed Basin was integrated with two existing northern basins into Basin #4. As a result, four proposed basins are relatively similar in size. The total area for this concept that includes the Backfeed Basin is larger than the area for Concept 1. The infiltration areas of the four proposed basins are shown in **Table 8**.

Detailed layouts and facilities are provided in **Attachment A**. The levees designed to separate the basins are 9 feet in height and 12 feet in top width with 3:1 side slopes. **Table 9** shows the time of 1.8 to 2.1 days required to fill the basins with a 9-foot depth and assuming the MVP inflow rate of 100 cfs. It is worth noting that this alternative makes Basin #4 containing the existing Backfeed Basin deeper to preserve its storage function. For this reason, the annual volume of water rises to an estimated 20,000 acre-feet per year. Assuming an average percolation rate of 1.75 feet per day, with a run time of 304 days a year, Concept 2 surpasses Concept 1 with the inclusion of the Backfeed Basin. As shown in **Table 10**, a similar approach was used to evaluate the cost estimates of Concept 2. The same three

alternatives account for potential maintenance plans. The maintenance costs are expected to be slightly higher than those in Concept 1 due to larger basin surface area.

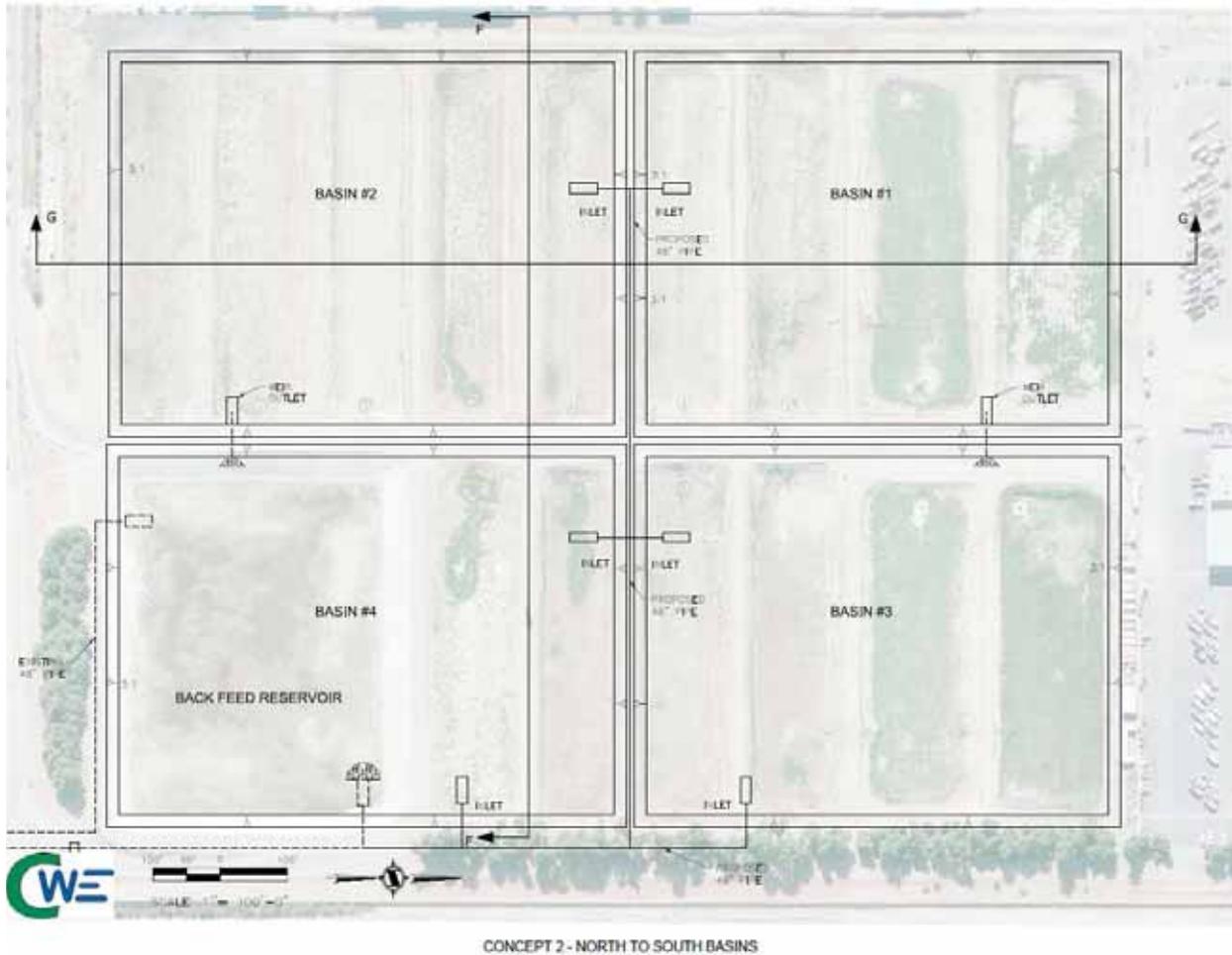


Figure 2 Concept 2 Layout and Structure Features

Table 8 Concept 2 Basin ID & Area			
Basin ID	Area (acre)	Basin ID	Area (acre)
Design Basin #1	9.10	Design Basin #3	10.11
Design Basin #2	8.42	Design Basin #4	10.21

<b>Table 9 Required Time to Fill Concept 2 Basin (day)</b>						
<b>Existing System</b>	<b>Percolation Rate (ft/day)</b>					
<b>Basin ID</b>	<b>1.0</b>	<b>1.5</b>	<b>2.0</b>	<b>2.5</b>	<b>3.0</b>	<b>3.5</b>
Basin #1	0.43	0.44	0.45	0.47	0.48	0.49
Basin #2	0.40	0.41	0.42	0.43	0.44	0.45
Basin #3	0.48	0.50	0.51	0.53	0.54	0.56
Basin #4	0.49	0.50	0.52	0.53	0.55	0.57
<b>Total</b>	1.80	1.85	1.90	1.95	2.01	2.06

<b>Table 10 Basin System Concept 2 Cost Estimate</b>		
<b>Item Description</b>	<b>Quantity</b>	<b>Cost</b>
<b>Construction</b>		
Mobilization (10%)	1 LS	\$141,000
Install Weir Box (\$15,000 each)	2 EA	\$30,000
3" AC Access Road (\$80/ton)	600 TONS	\$47,990
4" CMB Base (\$20/cubic yard)	395 CY	\$7,900
Rip-Rap (Light Class) Energy Dissipater (\$100/cubic yard)	56 CY	\$5,600
48" RCP (\$240/linear foot)	1059 LF	\$254,160
Excavation (\$12/cubic yard)	87,165 CY	\$1,045,980
Labor of Construction	1 LS	\$10,500
Contingency (25%)	1 LS	\$386,000
<b>Subtotal:</b>		<b>\$1,929,130</b>
<b>Alternative 1 Annual Maintenance</b>		
Basin Operation and Maintenance (5,500/acre)	37.8 acre	\$207,900
Weather Event Clean Up & Repair (\$108/acre)	37.8 acre	\$4,082
<b>Subtotal:</b>		<b>\$211,982</b>
<b>Alternative 2 Annual Maintenance</b>		
Basin Operation and Maintenance (11,000/acre)	37.8 acre	\$415,800
Weather Event Clean Up & Repair (\$108/acre)	37.8 acre	\$4,082
<b>Subtotal:</b>		<b>\$419,882</b>
<b>Alternative 3 Annual Maintenance</b>		
Basin Operation and Maintenance (5,500/acre)	37.8 acre	\$207,900
2" Silt Offsite Removal (\$25/cubic yard)	10164 cy	\$508,200
Weather Event Clean Up & Repair (\$108/acre)	37.8 acre	\$8,165
<b>Subtotal:</b>		<b>\$724,265</b>

## 4. Whitewater River Reach Concepts

The WWR-SC provides an excellent option for groundwater recharge throughout most of the year for imported and reclaimed water. CVWD has expressed interest in utilizing the reach between Portola Avenue and Fred Waring Drive for recharge. The WWR-SC may also be used to capture stormwater if the river system is designed for these events. Three design concepts were developed for the channel system. The main difference between the concepts is the selection of the structures to partition the channel reach into basins. Sugar berms, flashboards, and rubber dams were considered as design options. Besides the structure types, three alternatives accounting for different structure heights were developed for each concept. The three heights analyzed were 2 feet (Alternative 1), 4 feet (Alternative 2) and 6 feet (Alternative 3). The alternatives give options that result in various depths at the structures and distances between them. The basin lengths corresponding to the three heights are 800, 1600, and 2400 feet, respectively. The length is determined by the existing channel slope of 0.0025 ft/ft. Detailed descriptions for the layouts and cost estimates for each concept and alternative are provided in the following subsection. Detailed channel system and structure locations for the concepts and alternatives are shown in the drawings found in **Attachment B**.

The Whitewater River basins will be fed by the MVP at four outlets along the river between Portola Avenue and Fred Waring Drive. Based on the information provided by CVWD on the MVP, it is expected that there will be 7 to 10 feet of pressure head at the upstream end of the river basin systems. However, due to the preliminary nature of the data provided and the conceptual analysis of these systems without specific data on the MVP flows and pressures, we have included a pump station as an optional component in the cost estimates for the river basin system pipeline concept. The existing pump station for the MVP has additional bays available to add pumps to the system and this may reduce the costs of adding pumps for water delivery to the Whitewater River basins. The major cost for Whitewater River System is the optional pipe system. The system would divert storm water from the Basin Systems to the channel between Portola Avenue and Fred Waring Drive. Included in the pipe system will be four turnouts, along with isolation valves to provide control for system operations and maintenance.

Two alternatives have been devised for the placement of pipeline between Portola Avenue and Cook Street. The first alternative will be as detailed in **Attachment B**, with the pipeline located in the north bank of the channel. The second alternative, however, will divert the pipe into the channel riverbed at station 44+00 and continue until Portola Avenue with the pipe buried at a depth of 20 feet below the channel invert. This alternative will preserve the concrete lining in the channel and may result in a faster installation. The cost estimate for the bank and river alternative are detailed below in **Table 11** and **Table 12**. The MVP utilizes cathodic protection for the existing system and additional sections may also require cathodic protection. The cost estimate below does not include a cost estimate for cathodic protection due to the specific nature of cathodic protection rates. CVWD can determine the costs for the MVP system additions based on the costs for the existing MVP line.

Preventative measures will be taken to reduce the accessibility of the river reach to the public. In order to prevent public access to the channel system, 6' high fencing will be constructed to prohibit any foot traffic into the river reach. Fencing will only be required along Fred Waring Drive, the north side, the west side, and some sections on the south side of the channel. The existing fence along the south east corner of the channel will remain in place, while an additional line of fencing will be constructed to continue to Portola Avenue. Sloped protection barriers will need to be installed on the culverts along the upstream and downstream end at Cook Street and Fred Waring Drive, to keep unauthorized people out of the basins and reduce liability.

<b>Table 11 Pipe System Engineering Cost Estimate – Bank Alternative 1</b>		
<b>Item Description</b>	<b>Unit</b>	<b>Cost</b>
<b>Construction – Access Road, Pipe, and Pump Station</b>		
Clearing and Grubbing	1 LS	\$24,000
Trench Excavation (\$12/cubic yard)	24,287 CY	\$291,436
Shoring for 54" Pipe	1 LS	\$30,486
54" Steel Pipe (\$180/linear foot)*	8,359 LF	\$1,504,620
Knife Valves	11 LS	\$902,000
Pump Station	1 LS	\$2,500,000
Chain Link Fence (\$8/linear foot)	19,108 LF	\$152,864
Sloped Protection Barrier (\$12/pound)	204,000 LBS	\$2,448,000
3" AC Access Road (\$80/ton)	1,799 TONS	\$143,930
4" CMB Base (\$20/cubic yard)	1,185 CY	\$23,700
Rip-Rap (Light Class) by Pipe Outlet (\$100/cubic yard)	94 CY	\$9,450
<b>Subtotal:</b>		<b>\$8,030,486</b>

\*Does not include cathodic protection

<b>Table 12 Pipe System Engineering Cost Estimate – River Alternative 2</b>		
<b>Item Description</b>	<b>Unit</b>	<b>Cost</b>
<b>Construction – Access Road, Pipe, and Pump Station</b>		
Clearing and Grubbing	1 LS	\$24,000
Trench Excavation (\$12/cubic yard)	12,594 CY	\$151,125
Shoring for 54" Pipe	1 LS	\$15,918
Open Trench (\$8/cubic yard)	202,580 CY	\$1,620,641
54" Steel Pipe (\$180/linear foot)*	8,359 LF	\$1,504,620
Knife Valves	11 LS	\$902,000
Pump Station	1 LS	\$2,500,000
Chain Link Fence (\$8/linear foot)	19,108 LF	\$152,864
Sloped Protection Barrier (\$12/pound)	204,000 LBS	\$2,448,000
3" AC Access Road (\$80/ton)	1,799 TONS	\$143,930
4" CMB Base (\$20/cubic yard)	1,185 CY	\$23,700
Rip-Rap (Light Class) by Pipe Outlet (\$100/cubic yard)	94 CY	\$9,450
<b>Subtotal:</b>		<b>\$9,496,248</b>

\*Does not include cathodic protection

## 4.1 River Reach Concept 1

The first concept adopts sugar berms as the partition structure. One significant advantage of this concept is the availability of the structural material needed to create the berms. Channel bed sediments can be pushed into a berm. The local sediment availability reduces the construction cost. This concept using natural material is also considered the best among the proposed concepts from the environmental and aesthetics points of view. However, a major drawback of this concept is the tendency of flow erosion to cause failure of the berms, increasing the maintenance costs.

The Whitewater River reach length is approximately 10,900 feet. In order to use the entire reach for infiltration, the number of berms needed for each concept based on the heights of 2 feet, 4 feet, and 6 feet is 11, 5 and 3 berms, respectively. The 4-foot and 6-foot dams just downstream of Cook Street will use a 2-foot dam to pond water while preventing upstream flooding due to the shortened reach length.

**Table 13** shows the engineering cost estimates for the three Concept 1 alternatives.

While lower berms will create more basins, the number of structures to maintain also increases, leading to higher construction costs. The maintenance cost is estimated by using a similar approach adopted for the Basin system. The main channel bed area is 10,900 feet long and 260 feet wide. The system will require regular maintenance including silt removal and repair after storm events. With the economy of scale related to a larger area between berms and silt flushing related to large storm events, the cost per acre for routine sediment removal decreases to approximately \$2,153/acre. Determining the cost associated with the removal of vegetation assumes a removal of 17,494 cubic yards per year. Removal will be at a cost of \$25/CY. The berms may need periodic repair during the spreading operations due to erosion of berms by flowing water. The cost estimate for weather related repairs and clean up for the sugar berms are based on four storm events per year. The amount of the berm system expected to need repairs was assumed to be 50% of the initial berm formation for each major storm event.

The estimated cost is based on the total area and unit cost of each berm for construction. One benefit of the sugar berms is that they can be repaired and put back into operation after storm events. However, this requires additional labor and equipment expenses. The number of storms and timing of runoff events will impact the ability to spread water for this system.

One drawback is that the sugar berms do not have the ability to capture water from larger storm events that will wash out the sugar berms. Infiltration of MVP flows and stormwater flows will temporarily be suspended. Approximately 1 month, will be used for reconstructing the sugar berms that have failed. Capture of stormwater has the potential to offset construction, operations, and maintenance costs due to the increased recharge of free water. With an average cost per acre-foot priced at current prices of approximately \$150/ac-ft, this is a loss of significant potential benefits over the 50-year lifecycle of the project. **Table 13** details the cost of construction and the annual maintenance using the assumptions mentioned above.

<b>Table 13 River Basin Concept 1 - Engineering and Maintenance Cost Estimate</b>		
<b>Item Description</b>	<b>Unit</b>	<b>Cost</b>
<b>Construction – Alternative 1 (2' High Sugar Berm)</b>		
Grading 2' High (4' Top Width) (\$5/cubic yard)	1,695 CY	\$8,480
Contingency (25%)	1 LS	\$3,000
<b>Construction Subtotal:</b>		<b>\$11,480</b>
<b>Annual Berm O&amp;M (4 storm events):</b>	<b>1 LS</b>	<b>\$146,070</b>
<b>50 Year Life Cycle:</b>		<b>\$3,758,346</b>
<b>Construction – Alternative 2 (4' High Sugar Berm)</b>		
Grading 4' High (4' Top Width) (\$5/cubic yard)	2,311 CY	\$11,560
Contingency (25%)	1 LS	\$3,000
<b>Construction Subtotal:</b>		<b>\$14,560</b>
<b>Annual Berm O&amp;M (4 storm events):</b>	<b>1 LS</b>	<b>\$127,460</b>
<b>50 Year Life Cycle:</b>		<b>\$3,279,516</b>
<b>Construction – Alternative 3 (6' High Sugar Berm)</b>		
Grading 6' High (4' Top Width) (\$5/cubic yard)	2,773 CY	\$13,870
Contingency (25%)	1 LS	\$4,000
<b>Construction Subtotal:</b>		<b>\$17,870</b>
<b>Annual Berm O&amp;M (4 storm events):</b>	<b>1 LS</b>	<b>\$120,465</b>
<b>50 Year Life Cycle:</b>		<b>\$3,099,536</b>
<b>Annual River Basin Maintenance</b>		
River Basin Operation and Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Disposal (\$25/cubic yards)	17,494 CY	\$437,325
<b>Annual Maintenance Subtotal:</b>		<b>\$637,554</b>

## 4.2 River Reach Concept 2

The second concept for the river basins adopts flashboard dams. Flashboards are a type of bulkhead used to raise water level and or adjust the flow rate. The advantages of using this concept include economic materials and removable structures. Flashboard systems require a concrete footing to spread forces from the boards and frames to the channel bed. The structure also acts as energy dissipation when flows overtop the flashboards. Each of the flashboards has to be removed manually, which increases the time and effort required to adjust system settings, which is critical in the time before expected storm events. For minor events, the flashboards can be left in place to capture flood flows for percolation. However, for larger events, the boards require removal to prevent overtopping of the channel banks. This requires decisions prior to storm events and involves risk and liability. These factors must be weighed during the selection process.

The same three height alternatives used in Concept 1 were evaluated for the flashboard system. The number of dams needed for lower dam heights add cost to engineering and construction. **Table 14** shows the engineering cost estimates for the different alternatives. The estimated maintenance cost is based on the time needed to insert and remove the flashboards. We have assumed that the flashboards would need to be removed and replaced for 3 storms each year based on predicted rainfall intensities.

Maintenance of the silts assumes the cost estimate from the TEL facility with no off-site disposal of sediments. We only expect to have one complete maintenance cycle for the year, rather than two, due to the flushing nature of larger storm events. We have assumed a cost of \$2,153/acre of basin. There will be a cost associated with removal of vegetation. We have assumed 17,494 cubic yards of vegetative material per acre per year at a cost of \$25/cubic yard.

Operation for the flashboard dams entails insertion and removal of stop logs by 2 workers, the equipment needed for the insertion and removal from the river bed, and the fuel for the equipment. The removal and insertion time per stop log is expected to be approximately 15 minutes per log location at each barrier for the 2' high system. We have added five minutes at the location to pull each additional board for the 4' high and 6' high flashboard, giving a removal time of 20 minutes and 25 minutes, respectively. For this reason, the operation and maintenance cost for the flashboard dam structures is considered lower than that required for sugar berms, but is higher than rubber dams. Preparing for storms will take approximately 60 hrs for the 2' high system, 33 hours for the 4' high system, and 22 hours for the 6' high system. Replacing the boards after a storm will take the same amount of time. Approximately 1.5 months out of the year will be used for these operations, thus suspending the time for potential infiltration. In this duration, the loss of captured water includes MVP flows and stormwater flows. **Table 14** shows the construction and O&M cost items for the flashboard dam alternative.

The flashboard system will capture flows, but it is less desirable due to liability related to causing overbank flooding during large events. An assumption of 1.75 feet of capture per storm is estimated for all three alternatives. The other assumptions related to this analysis include a 50-year lifecycle with 3 storms producing runoff for capture each year. This will have a potential benefit equal to approximately \$6,291,600 for an additional 41,944 ac-ft of captured stormwater. The true potential of the flashboard system would need to be evaluated hydraulically with the design flow rates for the channel during the design phase with various heights of dams to see what the capacity is. The capture would have to be weighed against the risk of flooding. **Table 16** shows an estimated cost related to capturing 1.75 feet of stormwater in each basin during the three events.

<b>Table 14 River Basin Concept 2 - Engineering and Maintenance Cost Estimate</b>		
<b>Item Description</b>	<b>Quantity</b>	<b>Cost</b>
<b>Construction – Alternative 1 (2' High Flashboard Dam)</b>		
Mobilization (10%)	1 LS	\$356,000
Install 2' High Flashboard System	1 LS	\$2,102,100
Concrete Footing (\$320/cubic yard)	556 CY	\$177,960
Rip-Rap Energy Dissipator (\$100/cubic yard)	12,711 CY	\$1,271,120
Contingency (25%)	1 LS	\$977,000
<b>Construction Subtotal:</b>		<b>\$4,884,180</b>
<b>Annual Operation and Maintenance – Alt. 1</b>		
Annual Operation	1 LS	\$74,015
Annual Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$711,569</b>
<b>Construction – Alternative 2 (4' High Flashboard Dam)</b>		
Mobilization (10%)	1 LS	\$222,000
Install 4' High Flashboard System	1 LS	\$1,428,000
Concrete Footing (\$320/cubic yard)	253 CY	\$80,890
Rip-Rap Energy Dissipator - (\$100/cubic yard)	6,933	\$693,340
Contingency (25%)	1 LS	\$611,000
<b>Construction Subtotal:</b>		<b>\$3,034,230</b>
<b>Annual Operation and Maintenance – Alt. 2</b>		
Annual O&M	1 LS	\$55,053
Annual Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$692,607</b>
<b>Construction – Alternative 3 (6' High Flashboard Dam)</b>		
Mobilization (10%)	1 LS	\$205,000
Install 6' High Flashboard System	1 LS	\$1,371,500
Concrete Footing (\$320/cubic yard)	152 CY	\$48,540
Rip-Rap Energy Dissipator (\$100/cubic yard)	6,240 CY	\$624,000
Contingency (25%)	1 LS	\$563,000
<b>Construction Subtotal:</b>		<b>\$2,812,040</b>
<b>Annual Operation and Maintenance – Alt. 3</b>		
Annual Operation	1 LS	\$44,858
Annual Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$682,412</b>

### 4.3 River Reach Concept 3

The third concept evaluates rubber dams as the basin partition structure. Rubber dams offer more engineering and operational flexibility for groundwater recharge using stormwater, recycled water, and imported water. The height of rubber dams can be easily, and even automatically, adjusted for water conservation operations. This is particularly helpful for operations during high stormwater flow situations where the system can be kept at a set water level height to encourage water conservation during small events, while the rubber dams would lower themselves to pass larger runoff events. During runoff recession flows, the dams could be kept at safe heights to capture as much water as possible. Rubber dams are also more reliable than sugar berms during storm flows and more flexible than flashboards.

The same three alternatives for structure height are proposed as alternatives for Concept 3. As mentioned, smaller dam heights require more dams to pond water within the same river reach footprint, increasing the number of structures required and impacting construction costs. **Table 15** shows the engineering cost estimates for the rubber dam alternatives.

Maintenance of the silts assumes the cost estimate from the TEL facility with no off-site disposal of sediments. We only expect to need one complete maintenance cycle for the year, rather than two, due to the flushing nature of larger storm events. We have assumed a cost of \$2,153/acre of basin. There will be a cost associated with removal of vegetation. We have assumed 17,494 cubic yards of vegetative material per year at a cost of \$25/CY.

Maintenance of the rubber dams is expected to include inspection of the control houses and rubber bladders, maintenance of mechanical features, and minor repairs. This cost is estimated to be 2% of the construction cost.

Operation of the system is expected to require limited effort due to the option to set sensors and level controls on the bladders to meet the recharge and stormwater runoff requirements of the river system. **Table 15** lists the cost items mentioned above.

Over the life of the project, the rubber dam system is conservatively expected to capture an additional ac-ft of stormwater for the 2', 4' and 6' high dams. This estimate assumes a 50-year lifecycle, 3 storms producing runoff, and capture of 2 to 6 feet of water in each basin during each event. With an average cost per acre-foot priced at current prices of approximately \$150/ac-ft, this is a potential benefit equal to approximately \$4,185,000 to \$12,555,000 over the 50-year lifecycle of the project. A summary of the benefit of the captured stormwater is provided in **Table 16**. The ability of the rubber dams to operate during storm flows with limited liability for CVWD is a key benefit.

<b>Table 15 Concept 3 Engineering and Maintenance Cost Estimate</b>		
<b>Item Description</b>	<b>Quantity</b>	<b>Cost</b>
<b>Construction – Alternative 1 (2' High Rubber Dam)</b>		
Mobilization (10%)	1 LS	\$564,000
Install 2' High Rubber Dam	11 EA	\$3,960,000
Control Building	11 EA	\$495,000
Concrete Footing (\$320/cubic yard)	1,695 CY	\$542,350
Rip-Rap Energy Dissipator (\$100/cubic yard)	6,356 CY	\$635,560
Contingency (25%)	1 LS	\$1,550,000
<b>Construction Subtotal:</b>		<b>\$7,746,910</b>
<b>Annual Operation and Maintenance – Alt. 1</b>		
Annual O&M	1 LS	\$23,200
Annual River Basin Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$660,754</b>
<b>Construction – Alternative 2 (4' High Rubber Dam)</b>		
Mobilization (10%)	1 LS	\$403,000
Install 4' High Rubber Dam	5 EA	\$3,150,000
Control Building	5 EA	\$225,000
Concrete Footing (\$320/cubic yard)	963 CY	\$308,150
Rip-Rap Energy Dissipator (\$100/cubic yard)	3,467 CY	\$346,670
Contingency (25%)	1 LS	\$1,109,000
<b>Construction Subtotal:</b>		<b>\$5,541,820</b>
<b>Annual Operation and Maintenance – Alt. 2</b>		
Annual O&M	1 LS	\$23,200
Annual River Basin Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$660,754</b>
<b>Construction – Alternative 3 (6' High Rubber Dam)</b>		
Mobilization (10%)	1 LS	\$337,000
Install 6' High Rubber Dam	3 EA	\$2,700,000
Control Building	3 EA	\$135,000
Concrete Footing (\$320/cubic yard)	693 CY	\$221,870
Rip-Rap Energy Dissipator (\$100/cubic yard)	3,120 CY	\$312,000
Contingency (25%)	1 LS	\$927,000
<b>Construction Subtotal:</b>		<b>\$4,632,870</b>
<b>Annual Operation and Maintenance – Alt. 3</b>		
Annual O&M	1 LS	\$23,200
Annual River Basin Maintenance (\$2,153/acre)	93 acre	\$200,229
Vegetation and Sediment Removal (\$25/cubic yard)	17,494 CY	\$437,325
<b>Annual O&amp;M Subtotal:</b>		<b>\$660,754</b>

## 5. Cost Estimate Summary and Comparison

In order to compare project costs, the lifecycle cost with construction and O&M needs to be evaluated. Sugar berms have a lower upfront cost, but have significant costs over time to repair and rebuild on an annual basis. Rubber dams have a higher upfront cost, but lower maintenance costs. The project costs were brought back to a present value for equal comparison using an interest rate of 3% for a 50-year life cycle. A summary of the basin systems and the river reach concepts is provided below in **Table 16**.

The total cost includes the initial construction and 50-year life cycle of the annual operation and maintenance. For the basin systems, Concept 2 has a higher cost due to its larger acreage. In comparison to the sugar berms and the flashboard dams, the rubber dam is the most cost effective alternative. Unlike the sugar berms, the flashboards and rubber dams show a decline in price as the number of dams decrease. This is not seen in the sugar berms because each height determines the overall dimensional width of the each berm. Therefore, more cubic yards of soil are needed for the 6-foot high dam than are needed for the 4-foot or 2-foot dam.

As discussed in the sections for the Whitewater River Reach alternatives, the ability to capture free stormwater adds a benefit that can be assigned an economic value. These values have been used in the table below to show an offset of project cost by increased benefit. The amount of stormwater captured for sugar berms will not be retained during large storm events, therefore, **Table 16** shows a benefit value of \$0 for all three alternatives. The flashboard dam is assumed to only capture stormwater to a height of 1 foot for each alternative. This is primarily due to its inability to contain water levels to its designed height. An occurrence of overtopping is more likely in a large storm event. The rubber dam, however, can capture stormwater to its entire height, making it the most cost effective concept for each alternative.

<b>Table 16 Cost Estimate Summary and Comparison</b>			
<b>Basin Comparison</b>			
<b>Alternative</b>	<b>Concept 1</b>		<b>Concept 2</b>
<b>Alternative 1</b>	\$6,112,255		\$738,377
<b>Alternative 2</b>	\$10,627,339		\$12,732,595
<b>Alternative 3</b>	\$16,730,928		\$20,564,298
<b>River Reach Comparison &amp; Stormwater Capture</b>			
<b>Alternative</b>	<b>Sugar Berm</b>	<b>Flashboard Dam</b>	<b>Rubber Dam</b>
<b>Alternative 1</b>	-\$20,173,941 +\$0	-\$23,192,682 +\$2,092,500	-\$24,747,954 +\$4,185,000
<b>Total</b>	<b>\$20,173,941</b>	<b>\$21,100,182</b>	<b>\$20,562,954</b>
<b>Alternative 2</b>	-\$19,698,190 +\$0	-\$20,854,845 +\$2,092,500	-\$22,542,864 +\$8,370,000
<b>Total</b>	<b>\$19,698,190</b>	<b>\$18,762,345</b>	<b>\$14,172,864</b>
<b>Alternative 3</b>	\$19,521,520 +\$0	-\$20,370,340 +\$2,092,500	-\$21,633,914 +\$12,555,000
<b>Total</b>	<b>\$19,521,520</b>	<b>\$18,277,840</b>	<b>\$9,078,914</b>

## 6. Concept Comparison

A matrix was developed to comprehensively compare the existing conditions with the proposed concepts for the basin and channel systems discussed in this memorandum. The matrix summarizes the pros and cons associated with the cost, operation, and maintenance for the existing condition and each concept. A score was assigned to each concept within the categories based on engineering judgement and related experience. Each concept is ranked from 1 to 5. A score of 5 represents the highest ranking based on the cost and/or effectiveness, while a score of 1 represents the lowest ranking based on the cost and/or effectiveness.

When water demands and the current drought are taken into account, Concept 2 should be considered as the best alternative. Although the total cost for Concept 2 exceeds Concept 1, the increased recharge capability is expected to be 200,000 ac-ft over the next 50-years. The value of the difference between the two systems is \$600,000, based on an average cost per acre of \$150.

The rubber dam system has an initial installation cost that exceeds the costs of the sugar berms and the flashboard dams. However, over time, the operation and maintenance costs of the rubber dam system are considerably lower. The capabilities of the rubber dam system are enhanced by its automatic control system. The height of the dam can be automatically maintained at a desired pressure. In the event of high flood, the dam has the ability to deflate based on a pre-determined water level setting. Although the height can be adjusted to intermediate levels, it is recommended that Alternative 3 be used. The six foot rubber dam is the most cost effective and provides a larger subbasin area throughout the Whitewater River reach.

The concepts for the Whitewater River system have significant potential for increasing recharge. Assuming the maximum MVP delivery, while maintaining the infiltration rate assumed for the Basin Systems (1.75 ft/day), the volume of water replenished for the Channel System is estimated to be approximately 49,000 acre feet per year. Levels of infiltrated water will vary depending on the selected dam structure.

Over the life of the project, the rubber dam system is conservatively expected to capture an additional 27,900 ac-ft to 83,700 ac-ft of stormwater over the project lifecycle. This estimate assumes a 50-year lifecycle, 3 storms producing runoff, and capture of 2 to 6 feet of water in each basin during each event. With an average cost per acre-foot priced at current prices of approximately \$150/ac-ft, this is a potential benefit equal to approximately \$4,185,000 to \$12,555,000 over the 50-year lifecycle of the project. This additional capture is not possible for the sugar berms due to sugar berm failure during flooding. The sugar berms do not have the capacity to capture significant stormwater due to overtopping and erosion.

The flashboard system will capture the flows, but is less desirable due to liability related to causing flooding during large events. Similar to the rubber dam, an assumption of 1 foot per storm is estimated for a 50-year lifecycle, with 3 storms producing runoff. This will have a potential benefit equal to approximately \$2,092,500 for an additional 13,950 ac-ft of captured stormwater. The ability of the rubber dams to operate during storm flows with limited liability for CVWD is a key benefit.

The purpose of this feasibility study is to provide concepts that will be economical and practical. Concept 2, Alternative 3, falls under this category. Our recommendation is to implement Concept 2 for the WRP10 Basin System and Concept 3 for the Whitewater River Basins.

Memorandum

Diversion Coachella Valley Water District Palm Desert Recharge Feasibility Study

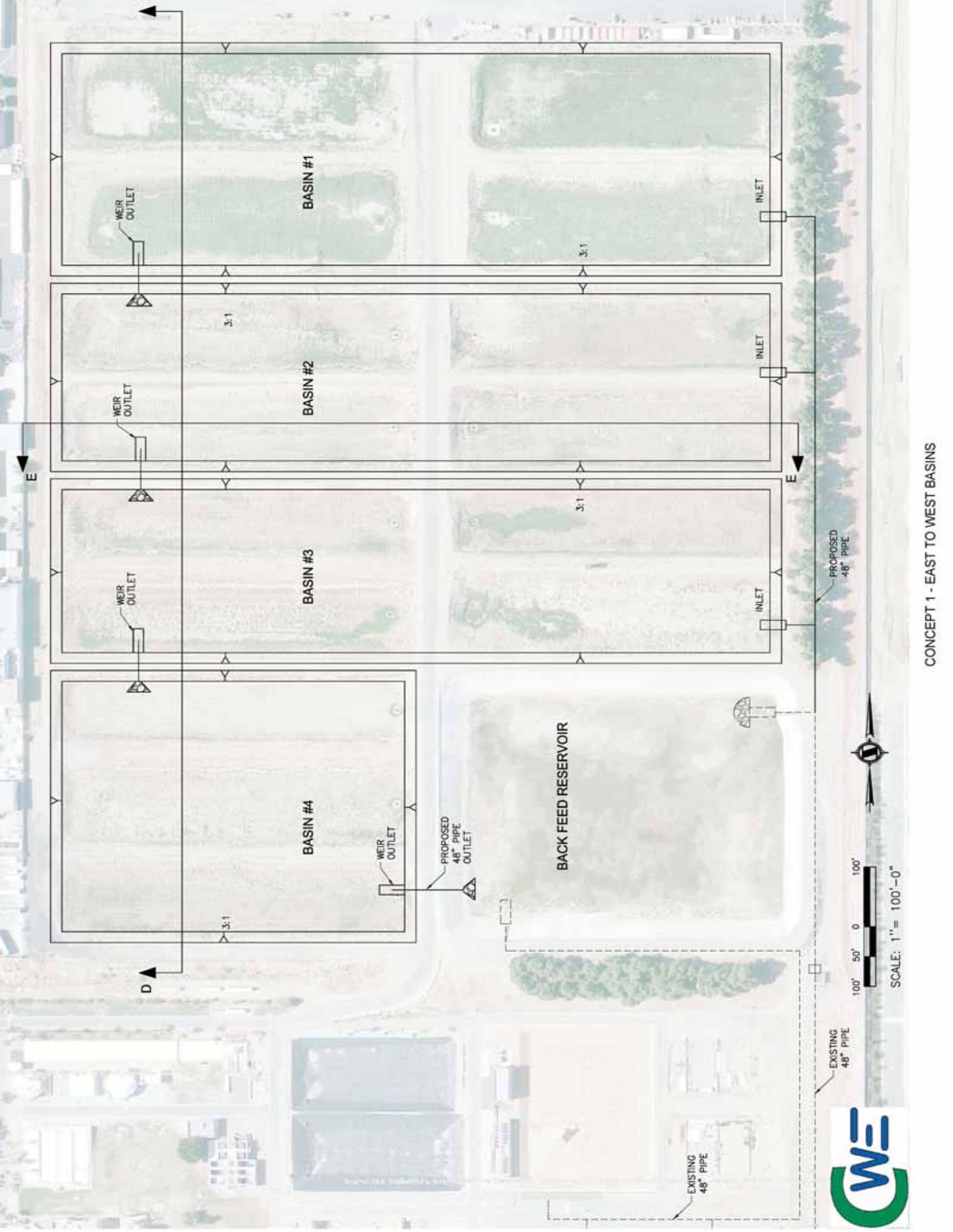
Page 22

However, CVWD has begun discussions with the U.S. Army Corps of Engineers (Corps) permitting staff to determine which impacts are more acceptable and what potential mitigation measures may be required for working within the WWR-SC. Preliminary discussions indicate that the sugar berms may require no mitigation and may be quicker to implement, which may lead to a decision to use the sugar berms as the preferred alternative for the WRR-SC projects.

**Table 17 Basin and Channel System Concept Comparison Matrix**

Comparison Category	Basin System Existing Condition		Basin System Concept 1		Basin System Concept 2		Channel System Existing Condition		Channel System Concept 1 – Sugar Berms		Channel System Concept 2 – Flashboard Dams		Channel System Concept 3 – Rubber Dams	
	Discussion	Score	Discussion	Score	Discussion	Score	Discussion	Score	Discussion	Score	Discussion	Score	Discussion	Score
Cost	Pros	No expenses related to installation of new facilities.	Relatively cheaper than Concept 2 due to less storage area improvements and excavation.	4	None	2	Using the most basin area requires more engineering and maintenance cost	No additional engineering, operation and maintenance cost.	4	Little to no engineering cost. Cost can be reduced by using local channel bed materials.	2	The construction materials are durable, less expensive than rubber dams, and have a life cycle of approximately 50 years.	3	Rubber dams are durable and have a life cycle of approximately 25 years. Maintenance cost is very low.
	Cons	None	None	3	Using the most basin area requires more engineering and maintenance cost	Lost opportunity cost for recharge of stormwater	4	Maintenance costs are related to berm formation, repair, and replacement after storms.	More maintenance expenses for moving water between basins and removing and replacing boards manually before and after storm events.	2	Construction cost is the highest of the three concepts.	3	Construction cost is the highest of the three concepts.	
Recharge Opportunity	Pros	Recharge amount can be easily predicted.	Larger basin area has higher percolation potential.	3	Largest basin area has highest percolation potential.	5	Some recharge of stormwater occurs naturally.	Increased recharge potential for imported water	1	Will retain water to a certain height before overtopping occurs. Can withstand overtopping to certain depths.	3	Will retain water to a certain height before overtopping occurs. Can withstand overtopping to certain depths.	5	Can be operated at various heights and can even be overtopped without falling. Will retain water to its full height during flood events, thus increasing stormwater capture and recharge rates.
	Cons	Smaller basin area for recharge would lower percolation efficiency and amount of water recharged.	Backfeed Basin is not used in this concept for recharge.	2	Backfeed Basin is used for recharge. This increases the basin area for recharge.	3	Upstream flows have very limited recharge capacity without channel barrier structures	Erosion to the sugar berm may result in overtopping failures, especially during storm events.	1	Will not retain water to its full height. Overtopping and erosion may damage facilities.	3	Will not retain water to its full height. Overtopping and erosion may damage facilities.	5	None
Operation and Maintenance	Pros	The operation and maintenance work is known.	Larger basins increase area that will simplify moving across berms, increasing efficiency. Overall cost is higher based on increased area.	2	Larger basins increase area will simplify moving across berms, increasing efficiency.	3	There is no need to perform operations and maintenance work for the channel reach.	Weather cleanup and repairs are needed semi-annually. Increased labor is included in this cost.	3	Lifespan is at least 25 years. Few annual repairs will be needed.	1	Lifespan is at least 25 years. Few annual repairs will be needed.	4	Operation of the rubber dam height is automated. Lifespan is 25 to 50 years. Few annual repairs will be needed.
	Cons	Less storage volume. Decreases the amount of water for recharge.	Does not include the Backfeed Basin in configuration. Recharge is not as significant as Concept 2. Cost per acre is expected to go down	3	Overall cost is higher due to basins increased size. Cost per acre will go down due to the larger area of the basins.	None	Highest labor and equipment costs per river basin due to maintenance of berms. Increased liability if not maintained or removed during storm events	Highest labor and equipment costs per river basin due to maintenance and repair of berms.	3	Highest labor and equipment costs per river basin due to maintenance of berms. Increased liability if not maintained or removed during storm events	1	Highest labor and equipment costs per river basin due to maintenance of berms. Increased liability if not maintained or removed during storm events	4	More specialized equipment within the system.
Cost and Total Score:	Existing System	9	Basin System Concept 1	8	Basin System Concept 2	10	Existing River	7	Sugar Berms	8	Flashboard Dams	6	Rubber Dams	12
Alternative 1			\$6,112,255		\$7,383,377				\$20,173,941		\$21,100,182		\$20,562,954	
Alternative 2			\$10,627,399		\$12,732,595				\$19,698,190		\$18,762,345		\$14,172,864	
Alternative 3			\$16,730,928		\$20,564,298				\$19,521,520		\$18,277,840		\$9,078,914	

**Attachment A**  
**Full Size Figures**



BASIN #1

BASIN #2

BASIN #3

BASIN #4

BACK FEED RESERVOIR

WEIR  
OUTLET

WEIR  
OUTLET

WEIR  
OUTLET

WEIR  
OUTLET

PROPOSED  
48" PIPE  
OUTLET

INLET

INLET

INLET

PROPOSED  
48" PIPE

EXISTING  
48" PIPE

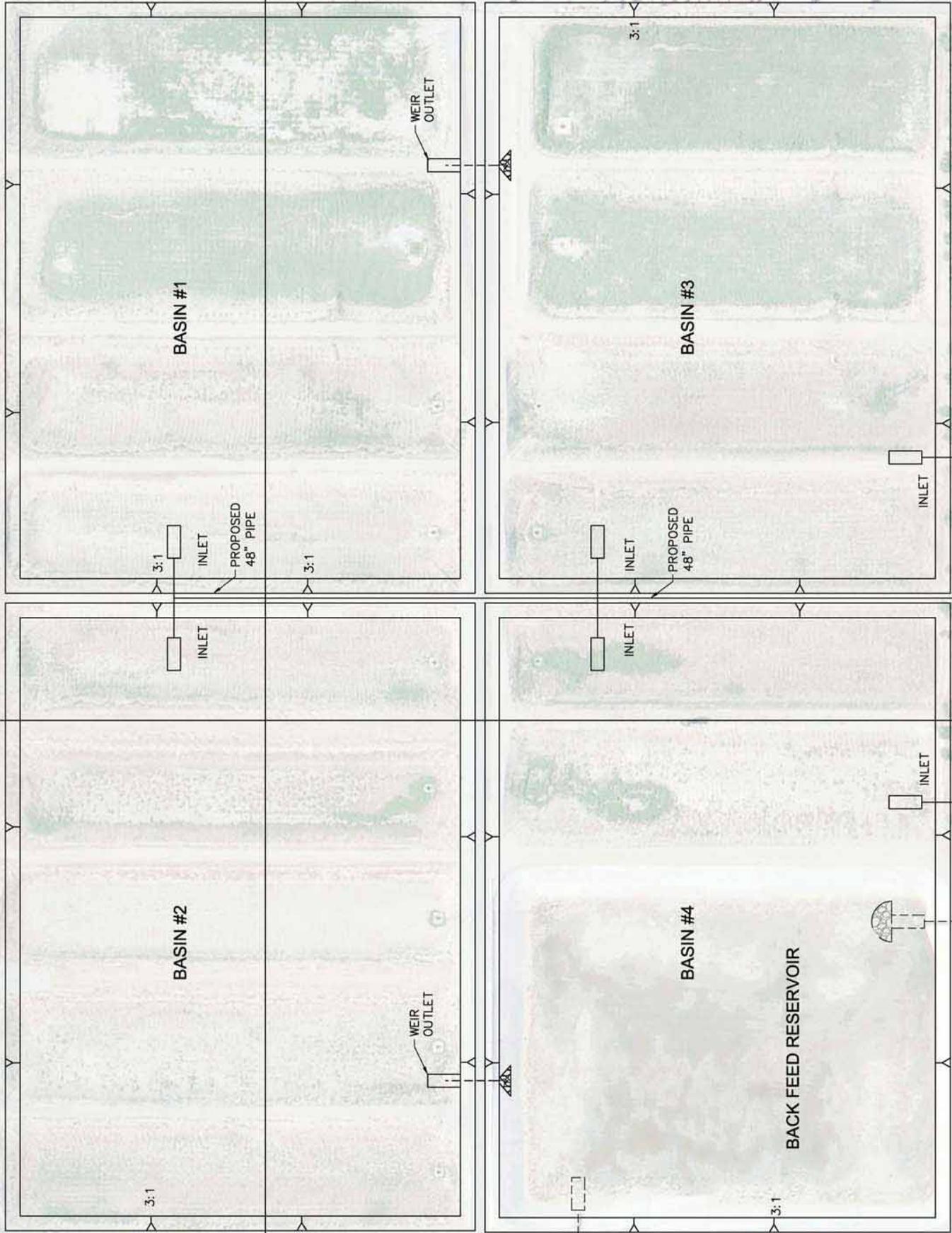
EXISTING  
48" PIPE



SCALE: 1" = 100'-0"



CONCEPT 1 - EAST TO WEST BASINS



CONCEPT 2 - NORTH TO SOUTH BASINS

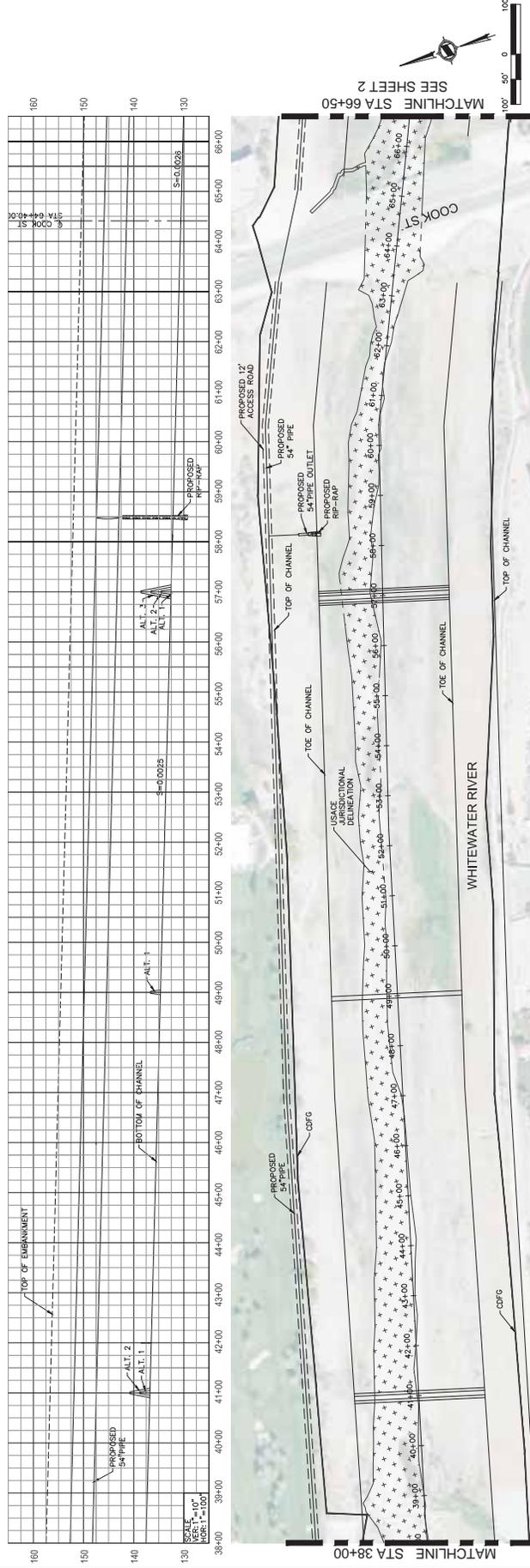
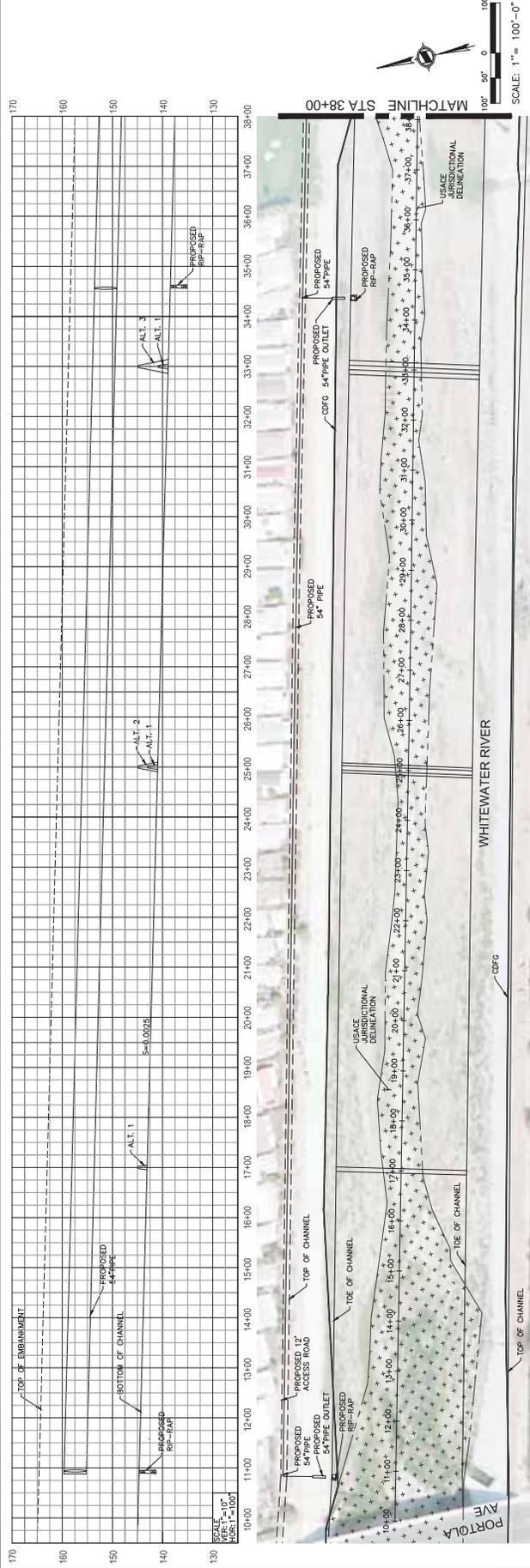
**Attachment B**  
**Concept Drawings**

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JUN 2016	T.T.	B.J.W.
JUN 2016	E.A.	
JUN 2016		

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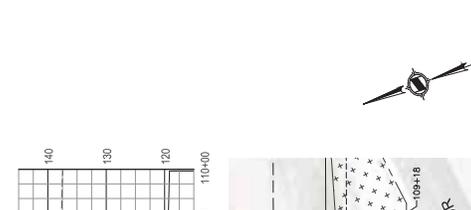
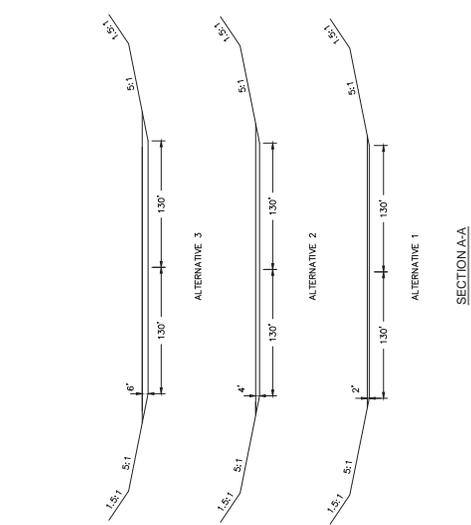
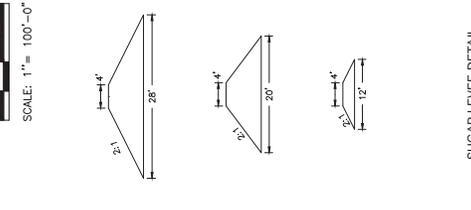
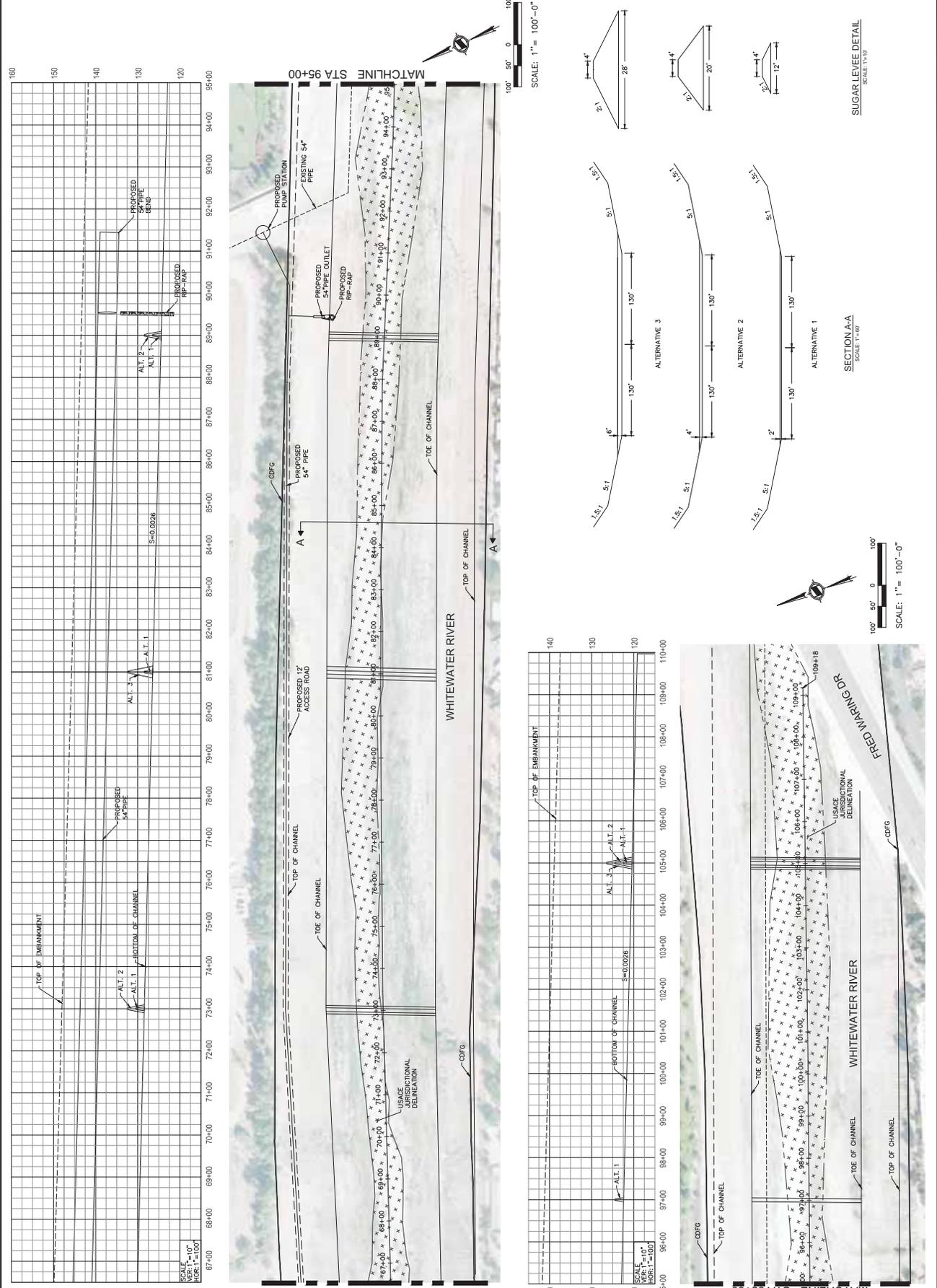


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 MATCHLINE STA 94+00  
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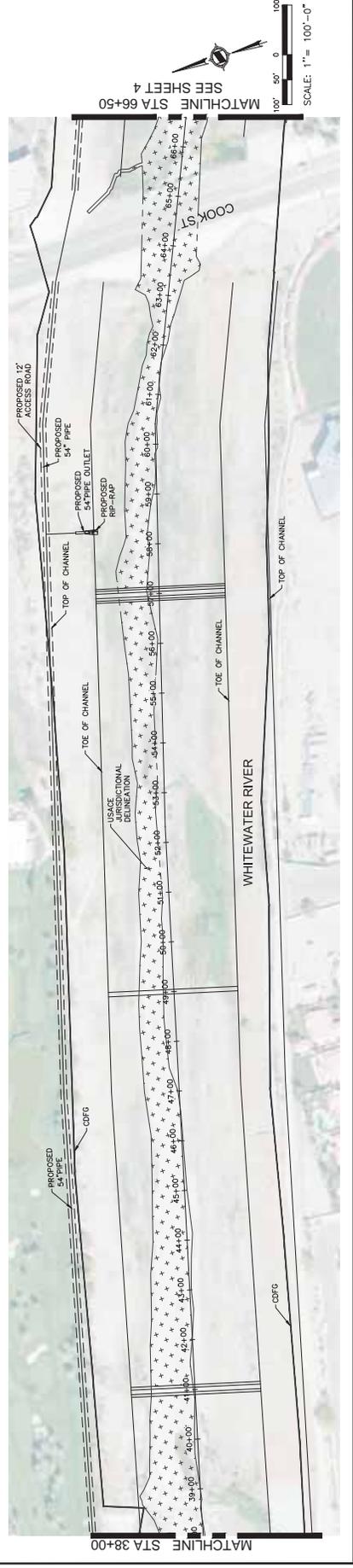
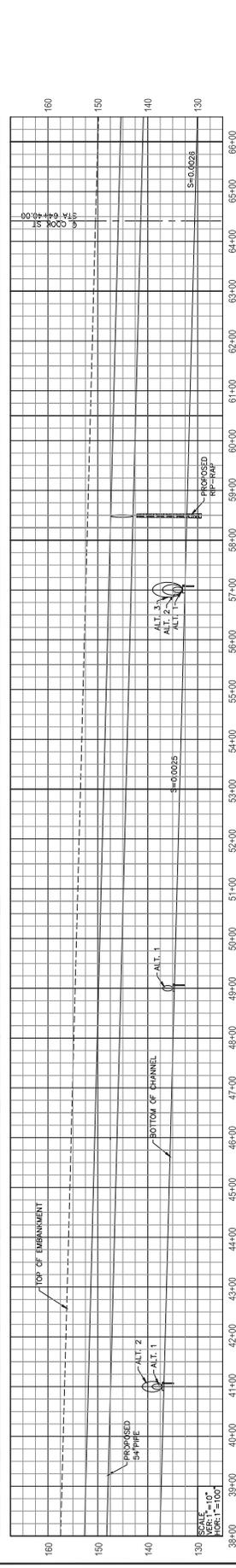
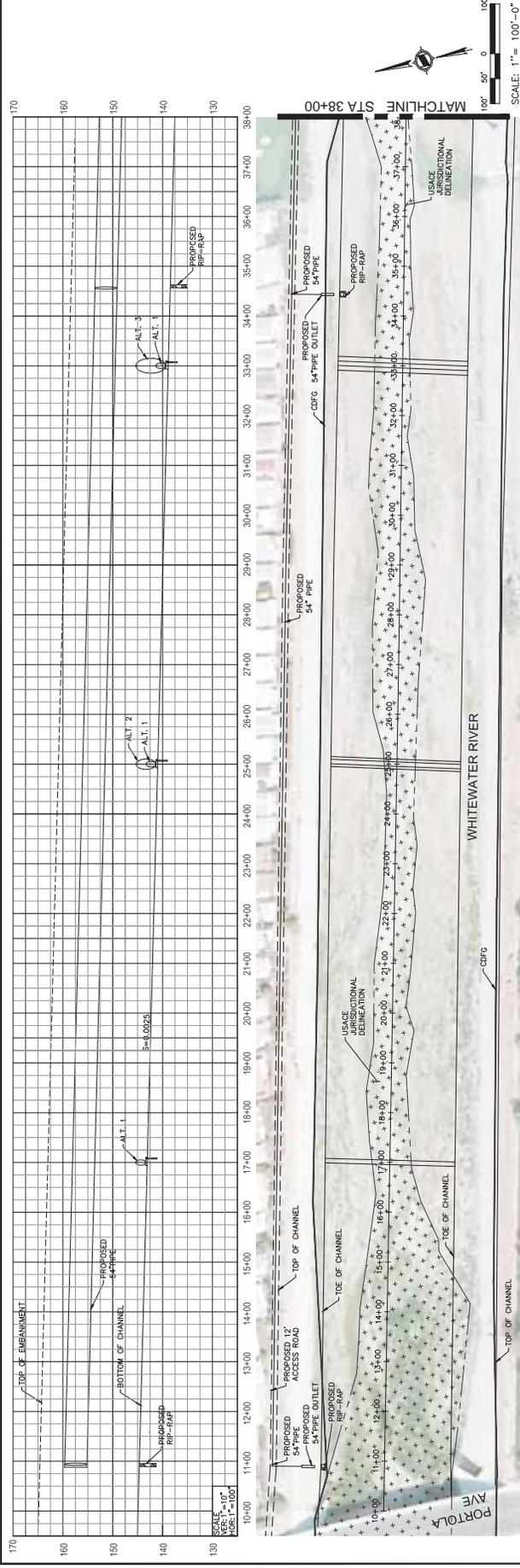


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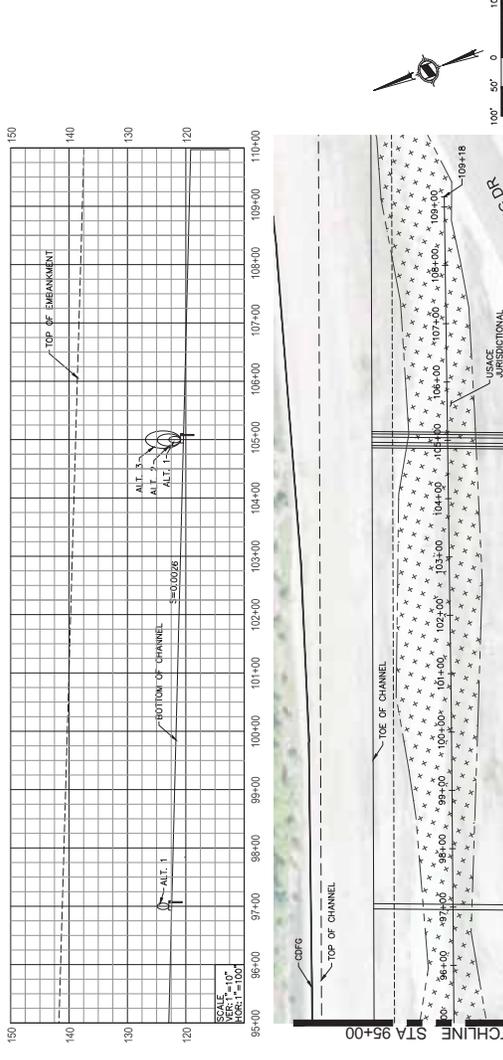
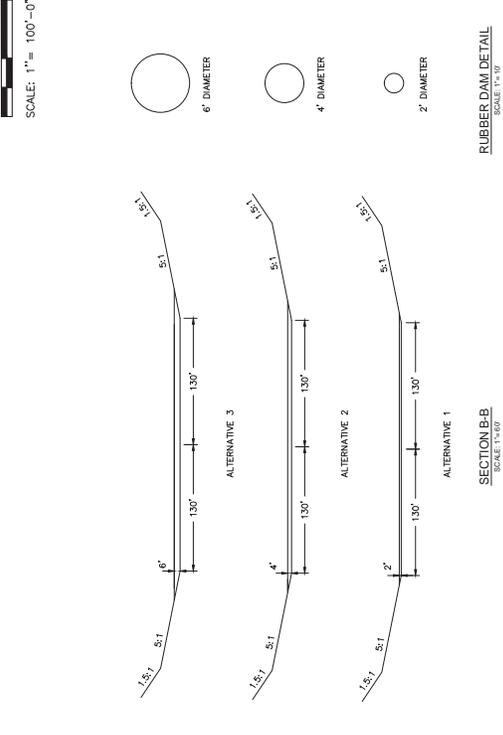
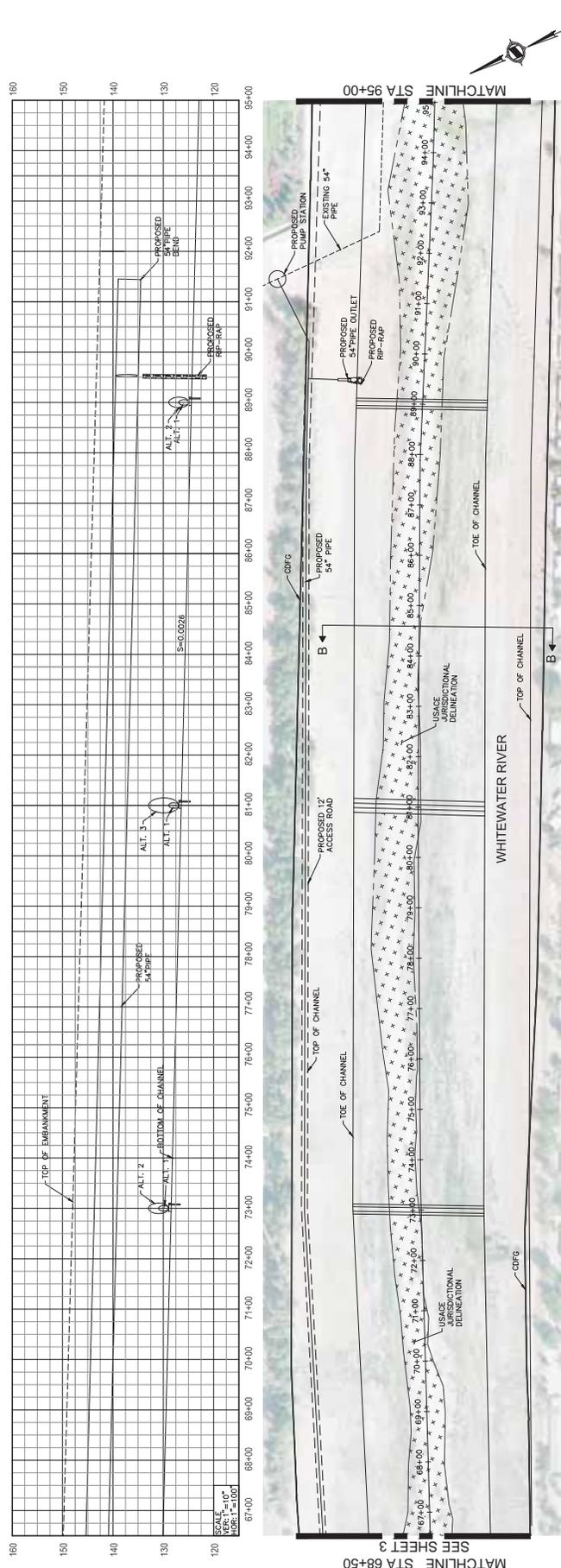
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PROJECT NO.	16172
PROJECT TITLE	COACHELLA VALLEY WATER DISTRICT
SHEET NO.	5
SHEET TITLE	PRELIMINARY PLAN AND PROFILE
OF	8
CONCEPT 3 - FLASHBOARDS	
PALM DESERT RECHARGE FEASIBILITY STUDY	

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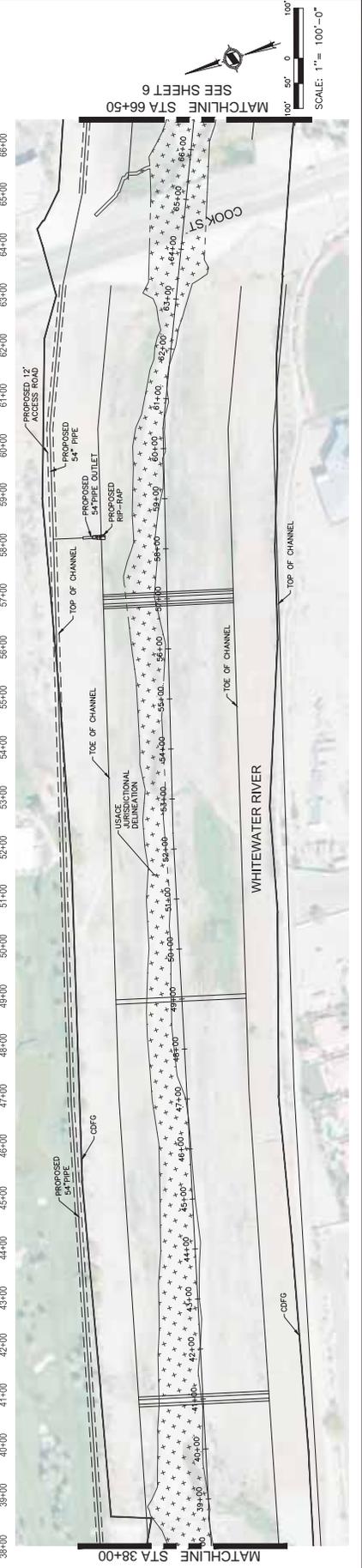
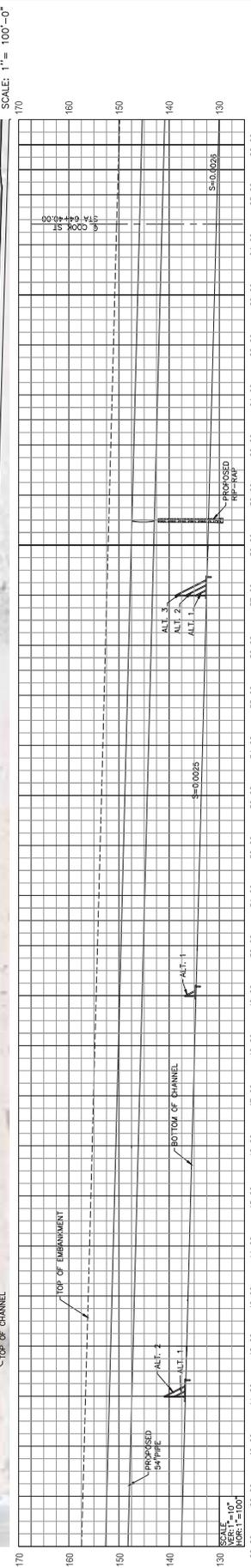
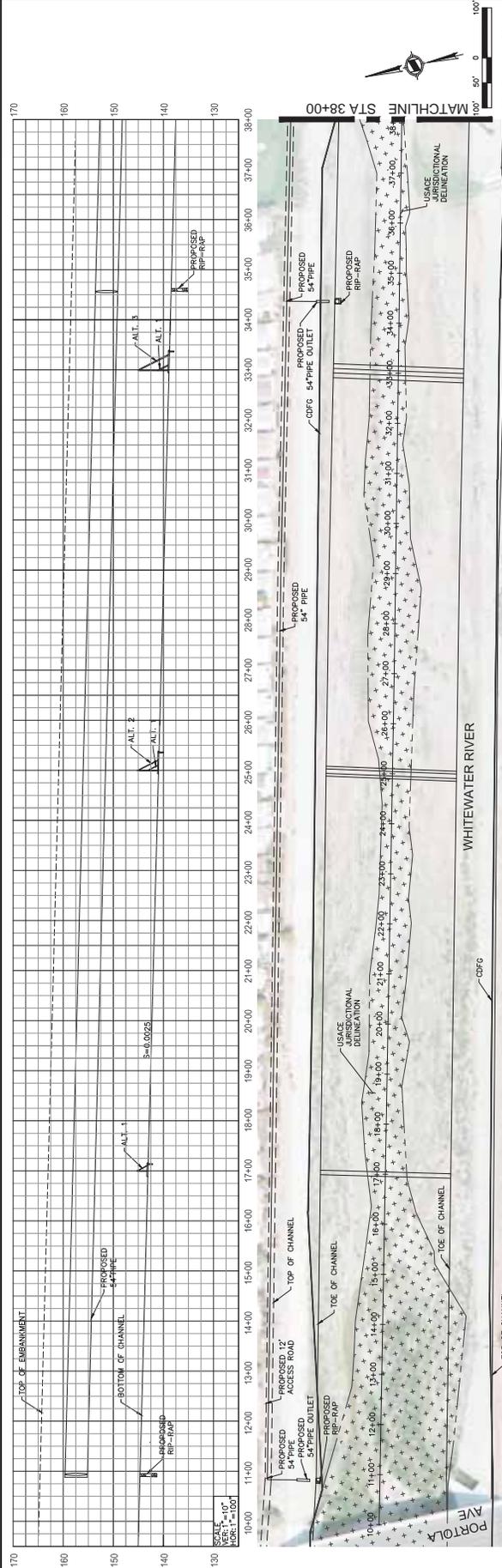
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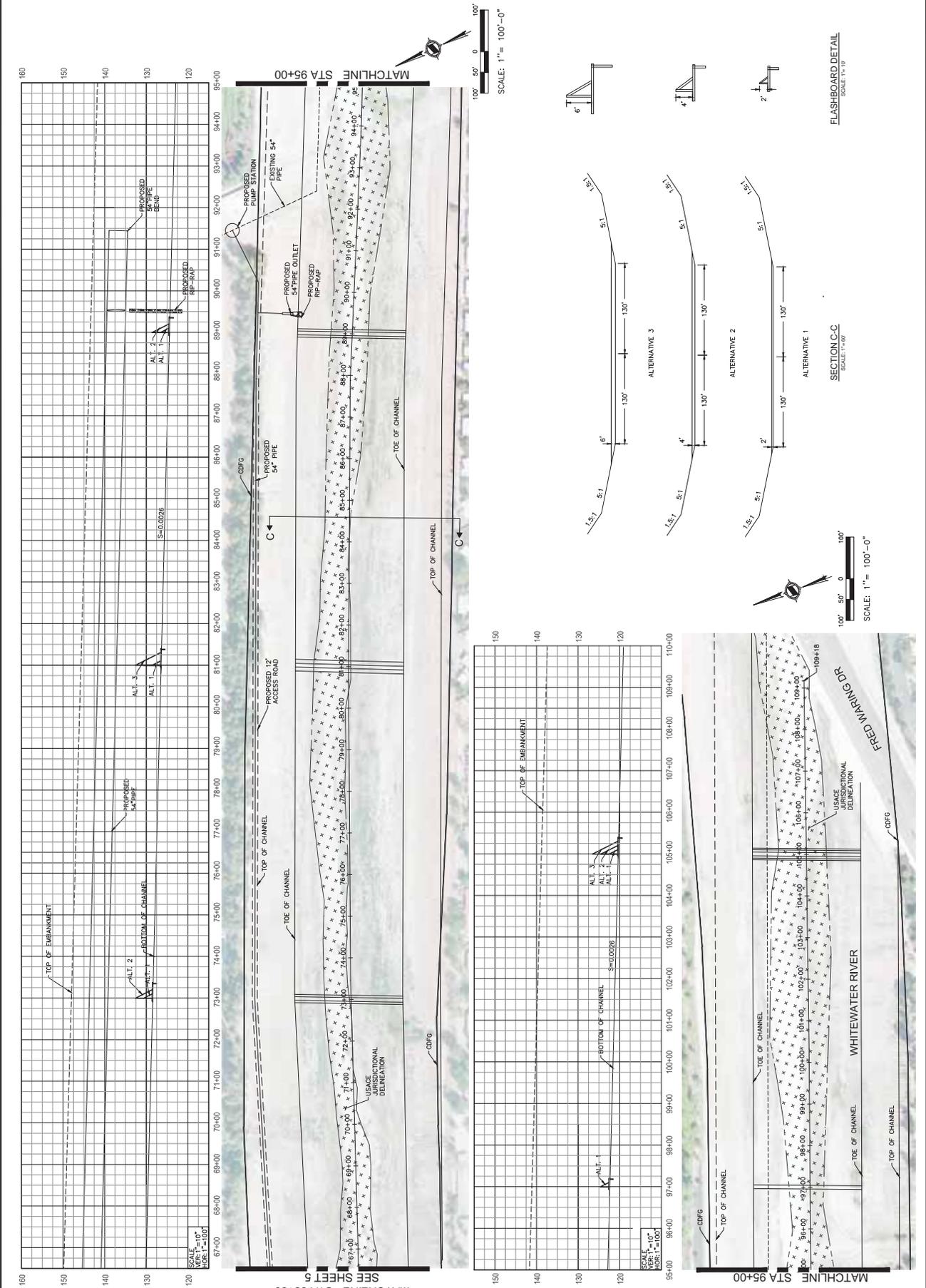


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SHEET NO. 5  
SHEET TITLE: PRELIMINARY PLAN AND PROFILE  
OF 8  
CONCEPT 3 - FLASHBOARDS  
PALM DESERT RECHARGE FEASIBILITY STUDY

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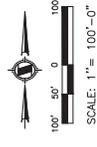
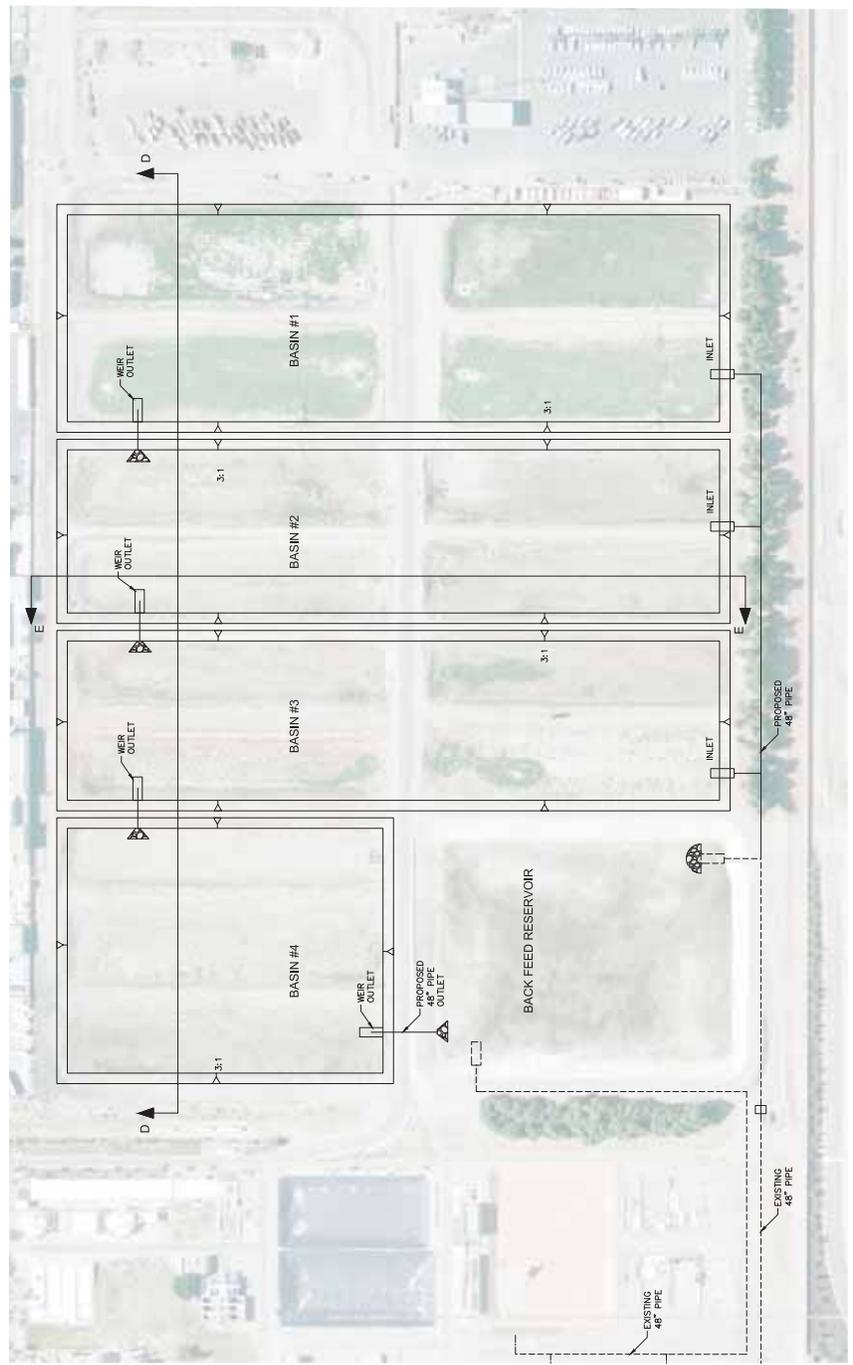
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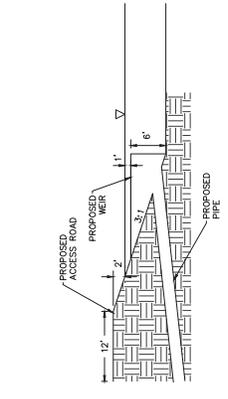
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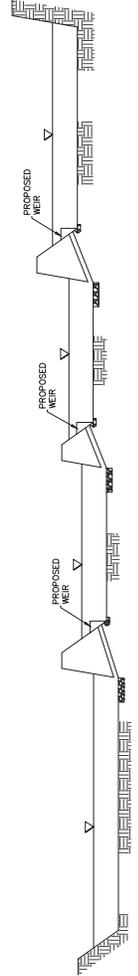
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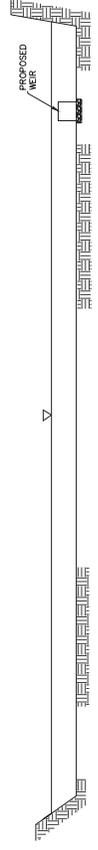
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SECTION E-E  
 N.T.S.

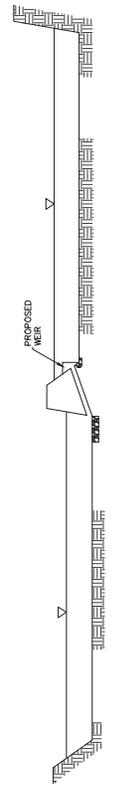
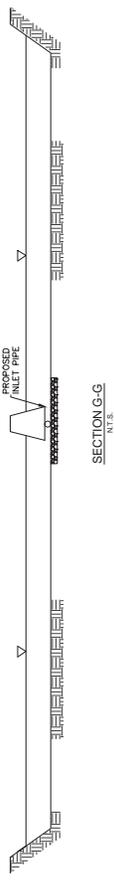
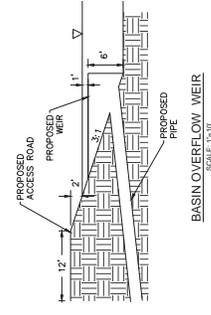
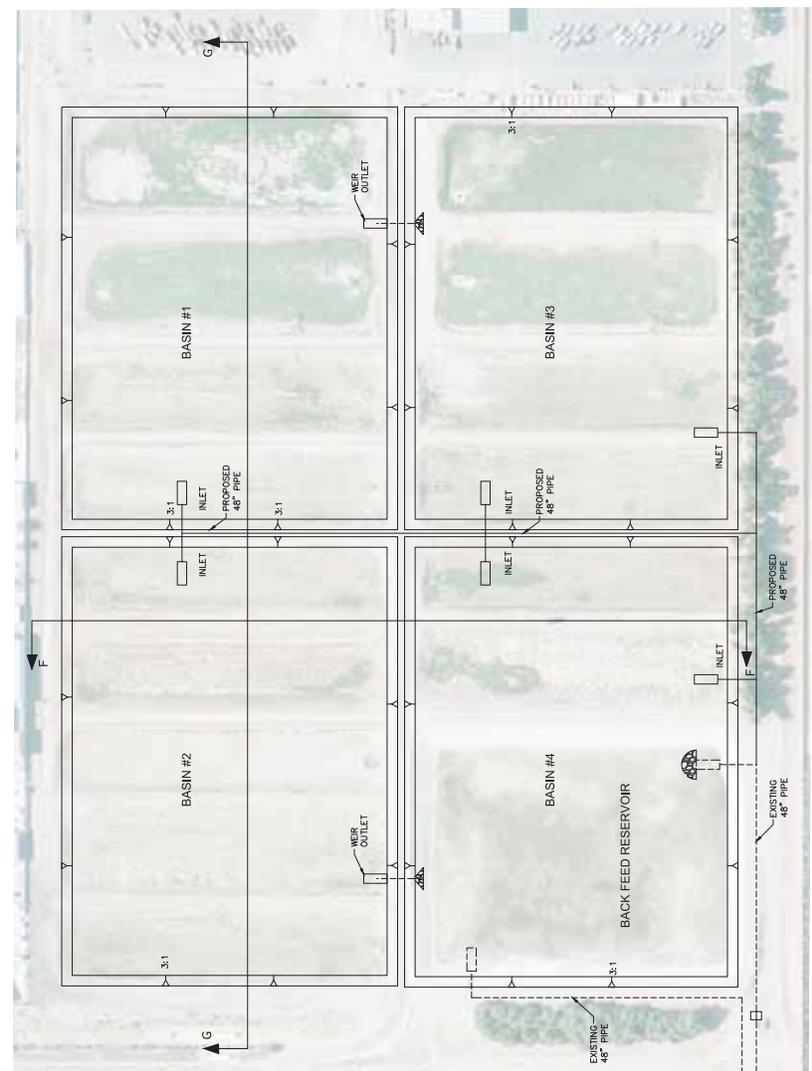


CONCEPT 1 - EAST TO WEST BASINS

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**Appendix B**

**Time-Concentration Plots for Study Area Wells  
by Water Quality Zone**

Water Quality Zone A



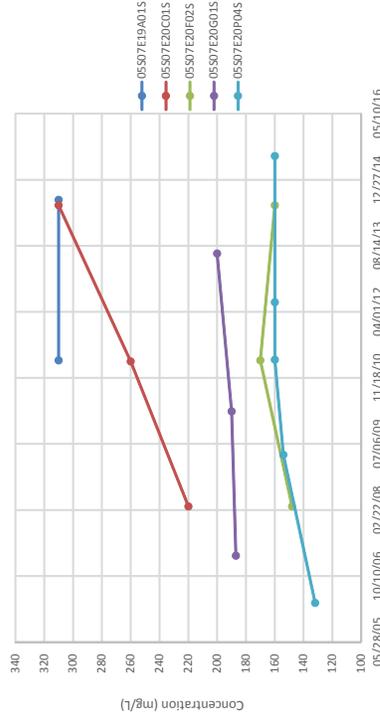
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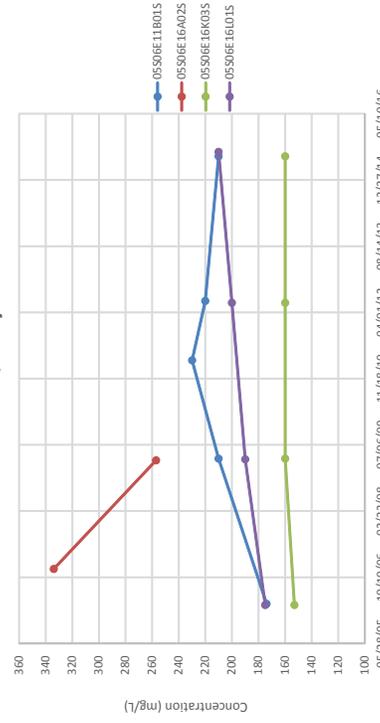
Water Quality Zone B



Water Quality Zone E and F



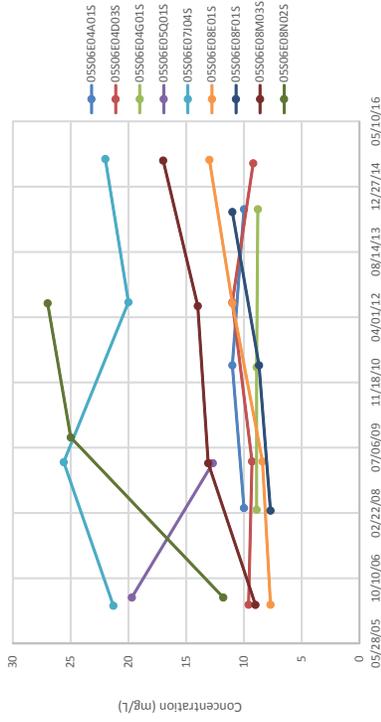
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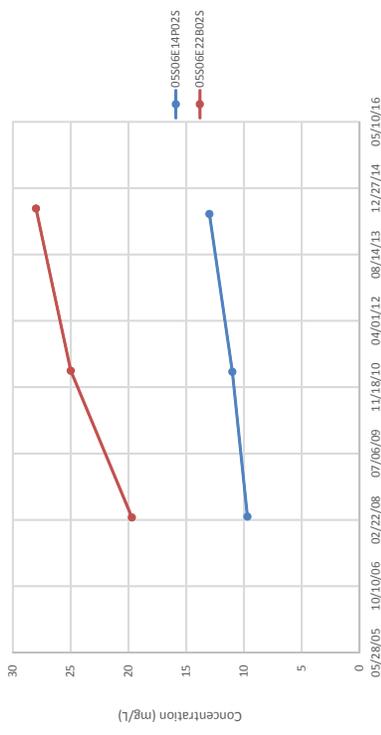
Note: charts do not include WRP 10 monitoring wells



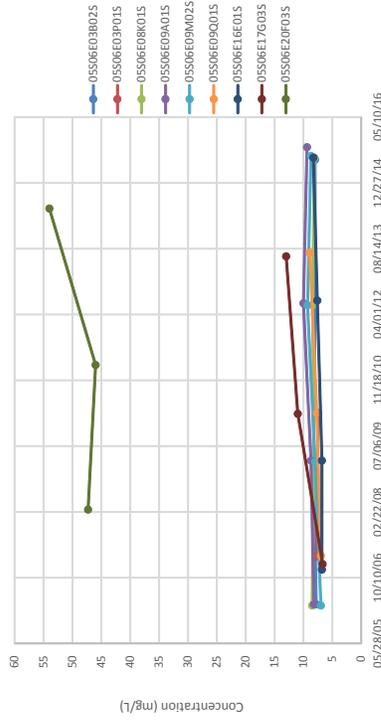
Water Quality Zone A



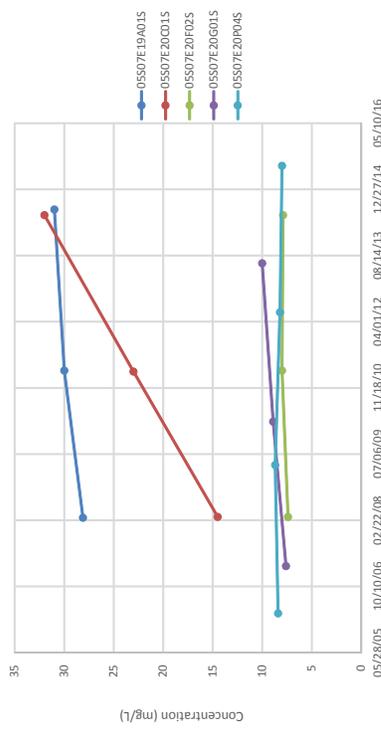
Water Quality Zone D



Water Quality Zone B



Water Quality Zone E and F



Water Quality Zone C

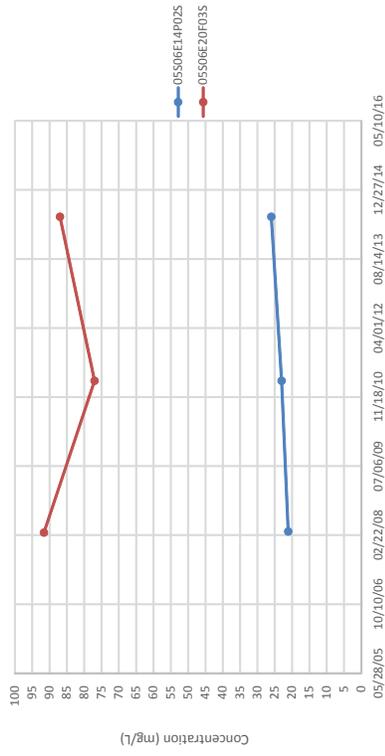


Note: charts do not include WRP 10 monitoring wells



Appendix B2  
Time-Concentration  
Plots - Chloride

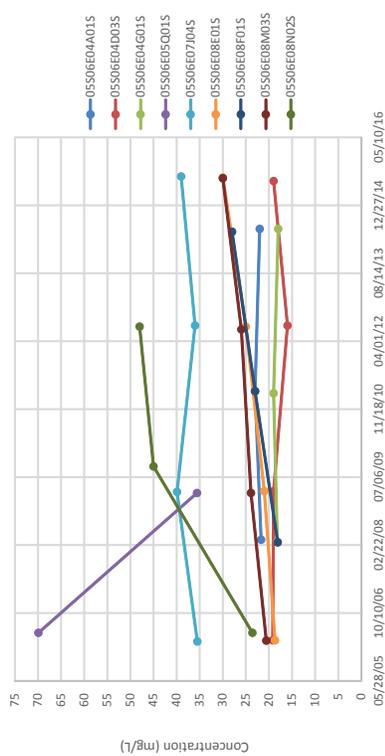
Water Quality Zone D



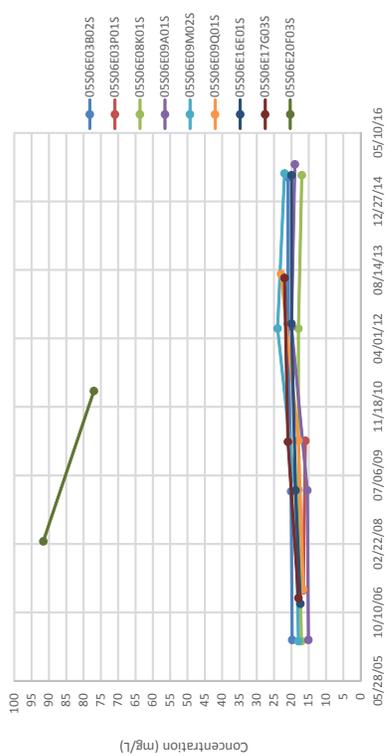
Water Quality Zone E and F



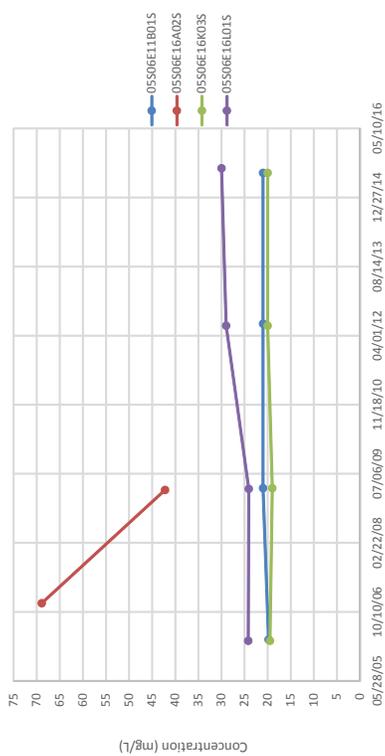
Water Quality Zone A



Water Quality Zone B



Water Quality Zone C

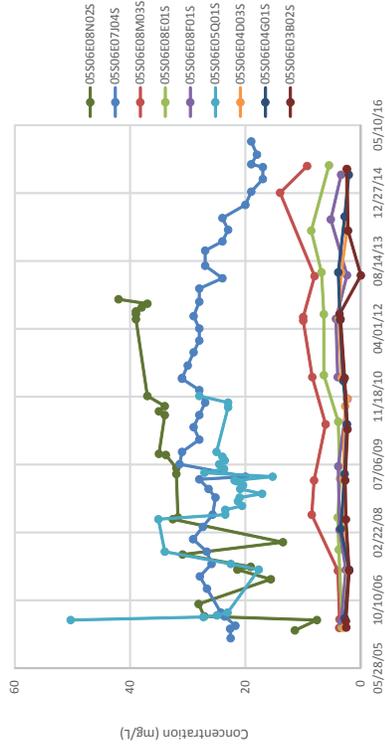


Note: charts do not include WRP 10 monitoring wells

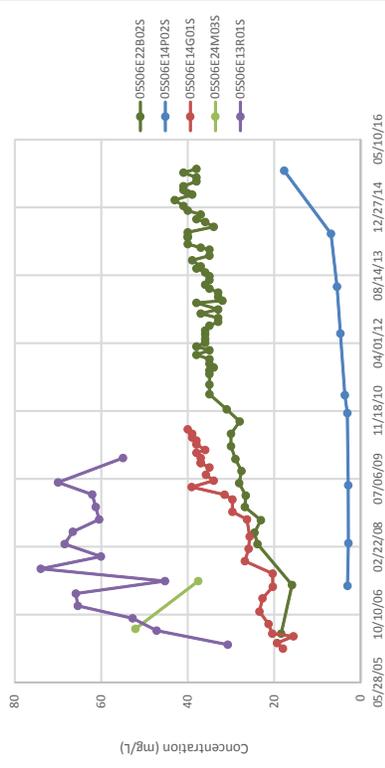


Appendix B3  
Time-Concentration  
Plots - Sulfate

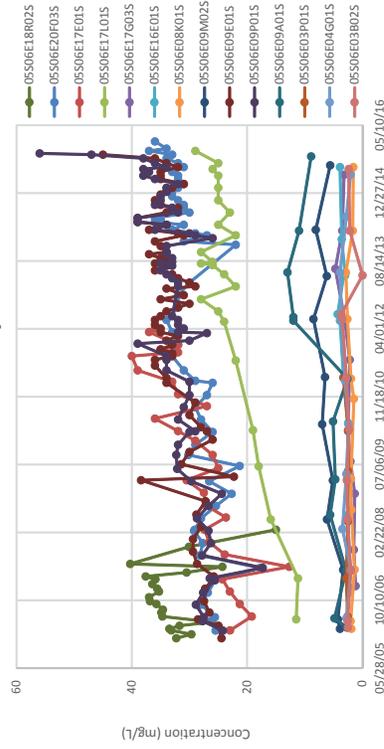
Water Quality Zone A



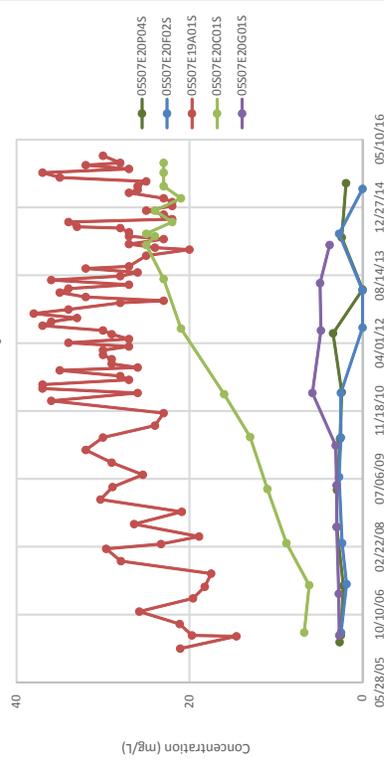
Water Quality Zone D



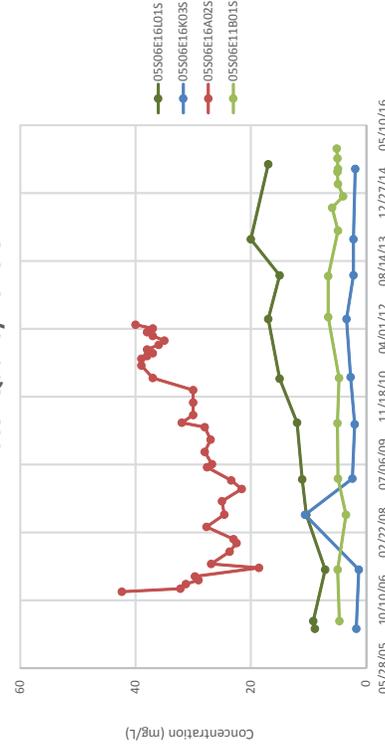
Water Quality Zone B



Water Quality Zone F



Water Quality Zone C

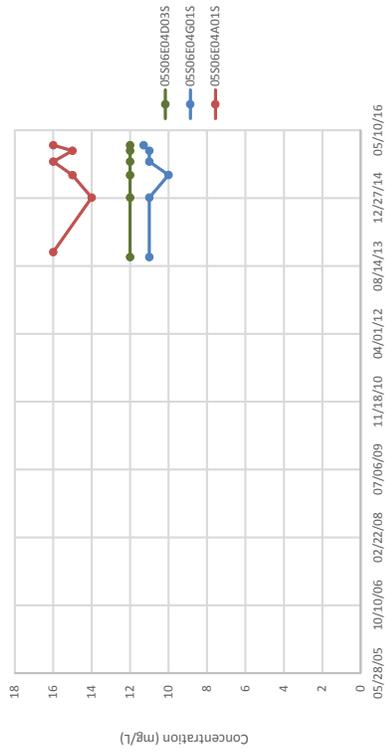


Note: charts do not include WRP 10 monitoring wells



Appendix B4  
Time-Concentration  
Plots - Nitrate

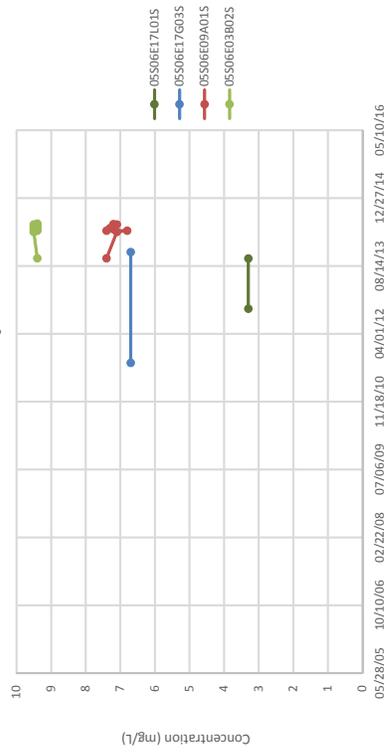
### Water Quality Zone A



### Water Quality Zone D



### Water Quality Zone B



### Water Quality Zone E and F



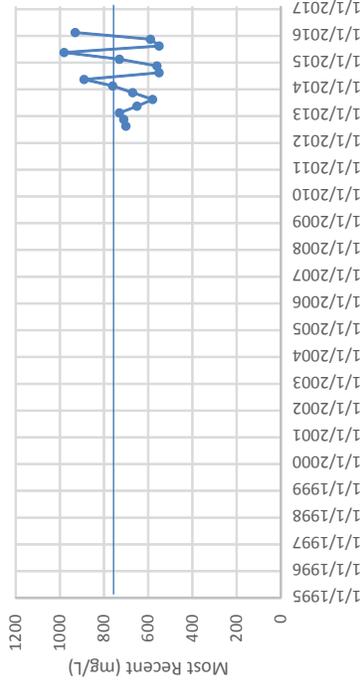
### Water Quality Zone C



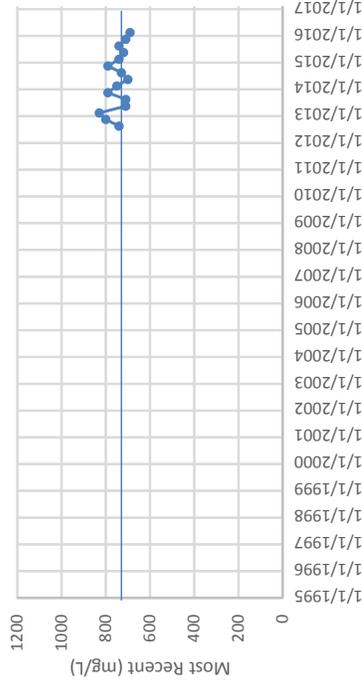
Note: charts do not include WRP 10 monitoring wells



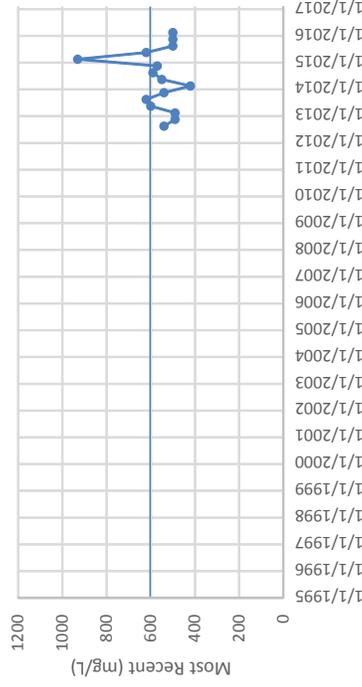
MW4 TDS



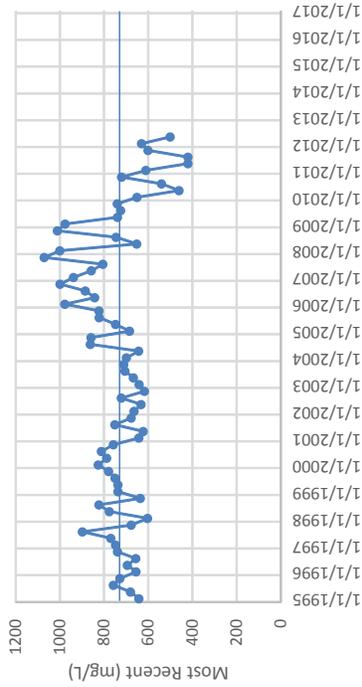
MW5 TDS



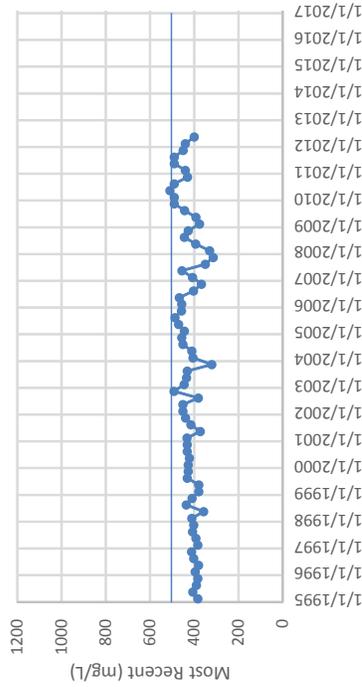
MW6 TDS



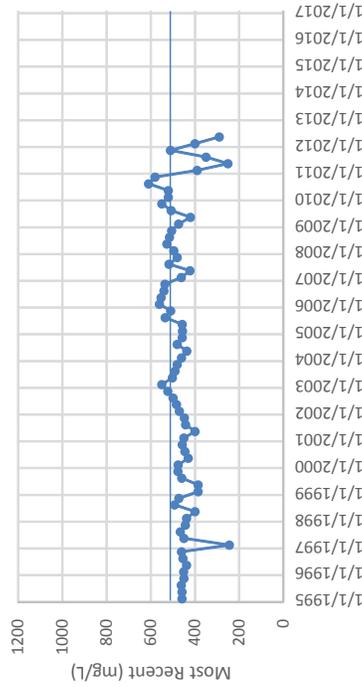
MW1 TDS



MW2 TDS



MW3 TDS

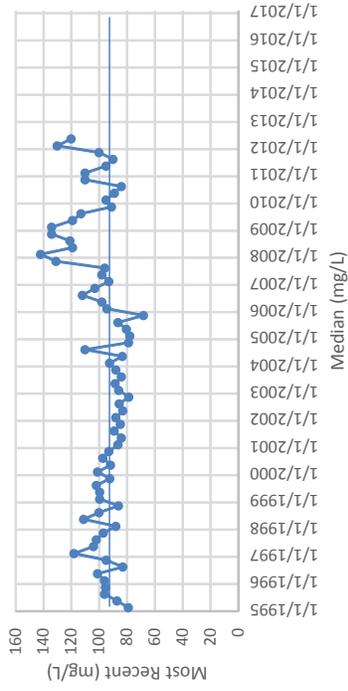


blue line = representative concentration shown on cross sections C-C and D-D and selected for initial concentration in Model Layer 1

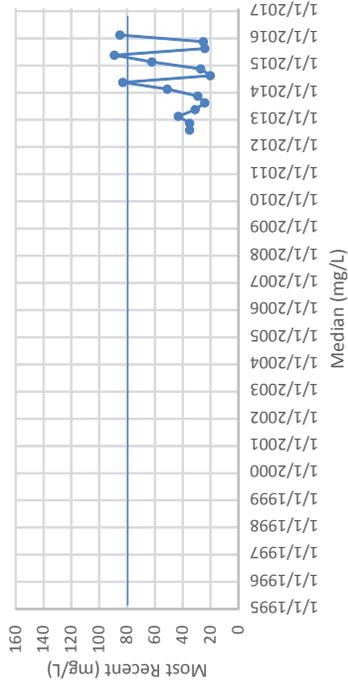


**Appendix B6**  
**Time-Concentration**  
**Plots - WRP 10 MWS**  
**TDS**

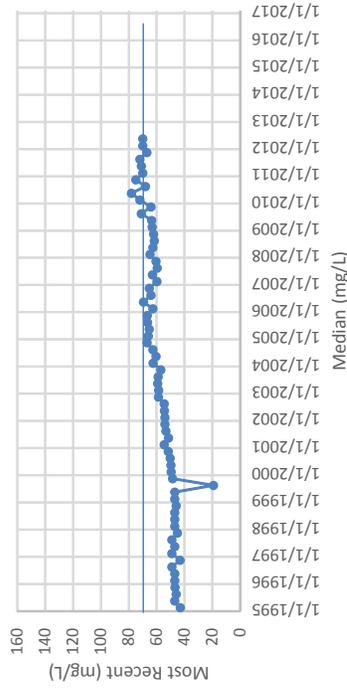
MW1 Chloride



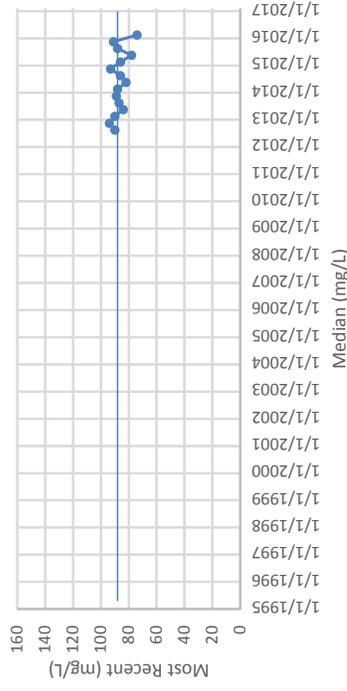
MW4 Chloride



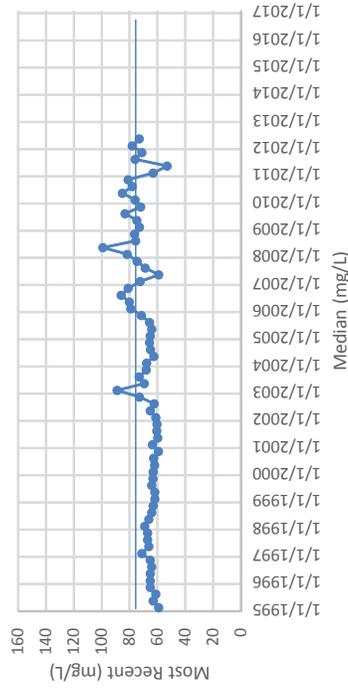
MW2 Chloride



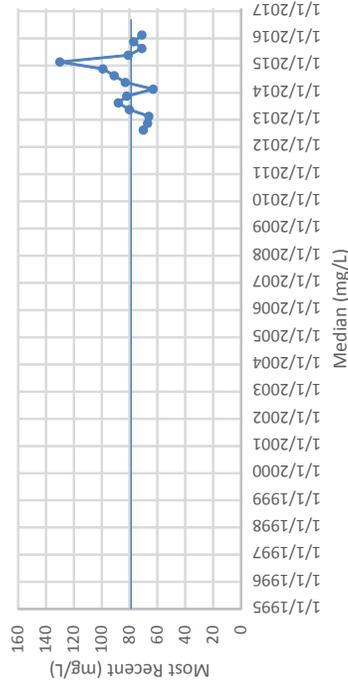
MW5 Chloride



MW3 Chloride



MW6 Chloride

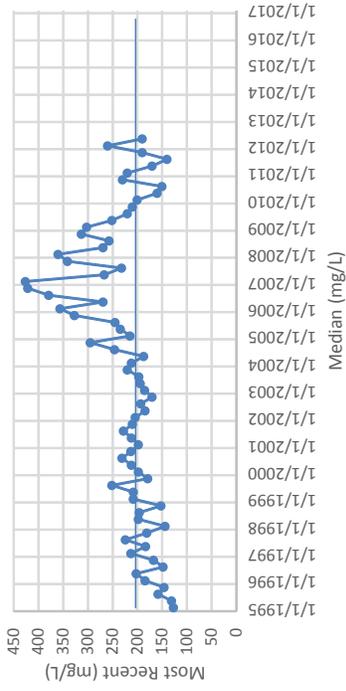


blue line = representative concentration shown on cross sections C-C and D-D and selected for initial concentration in Model Layer 1

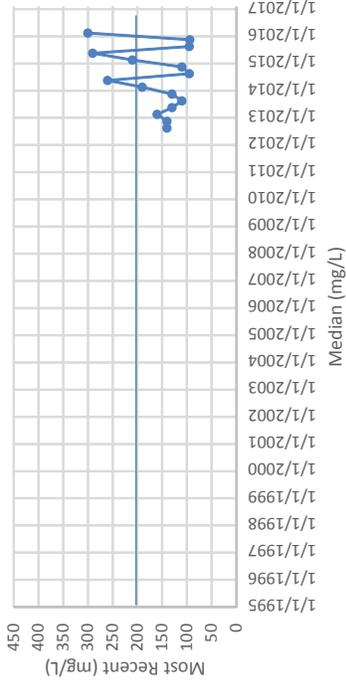


Appendix B7  
Time-Concentration  
Plots - WRP 10 MWs  
Chloride

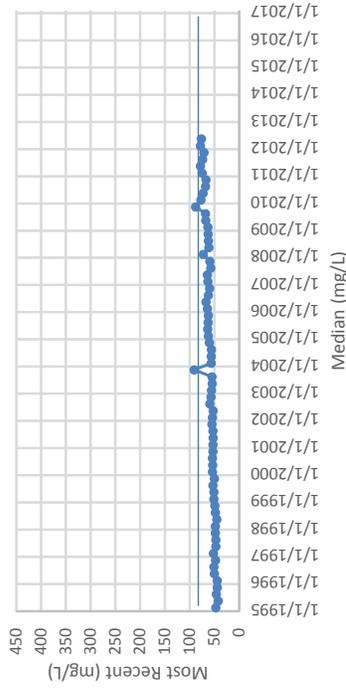
MW1 Sulfate



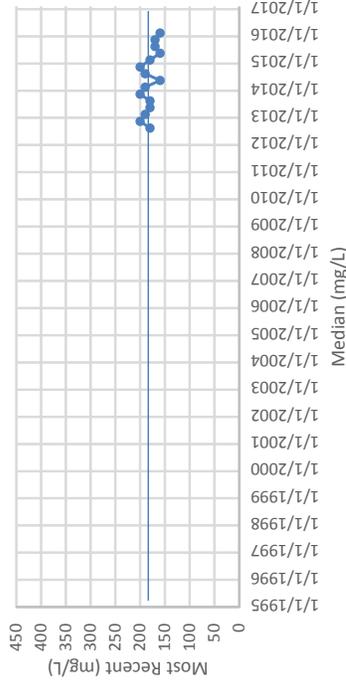
MW4 Sulfate



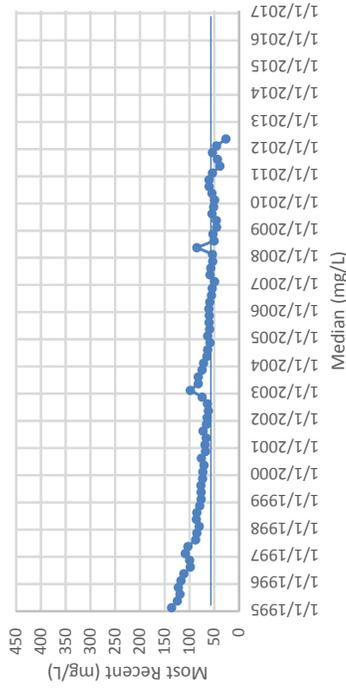
MW2 Sulfate



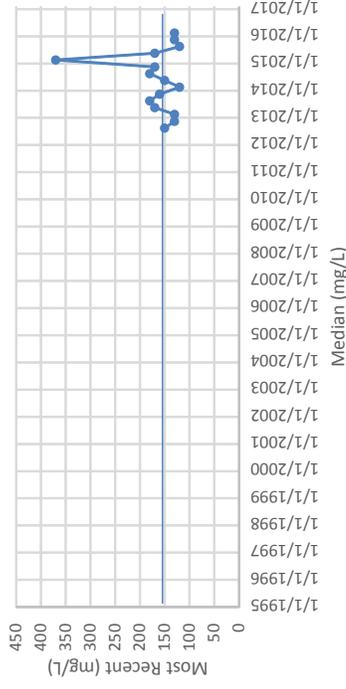
MW5 Sulfate



MW3 Sulfate



MW6 Sulfate

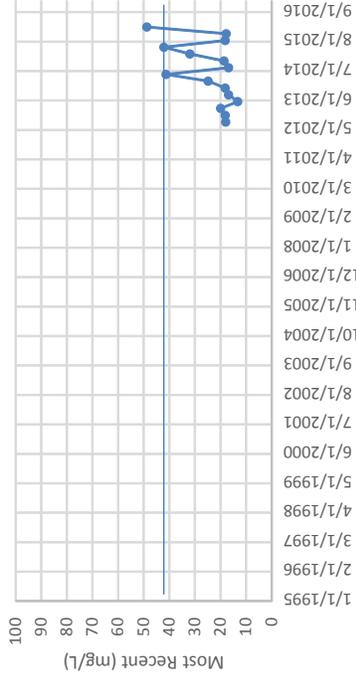


blue line = representative concentration shown on cross sections C-C and D-D and selected for initial concentration in Model Layer 1

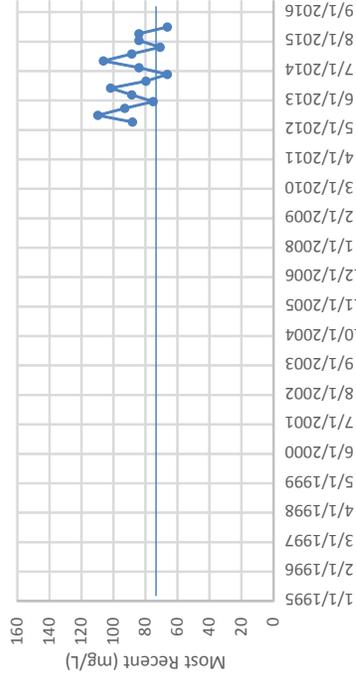


Appendix B8  
Time-Concentration  
Plots - WRP 10 MWS  
Sulfate

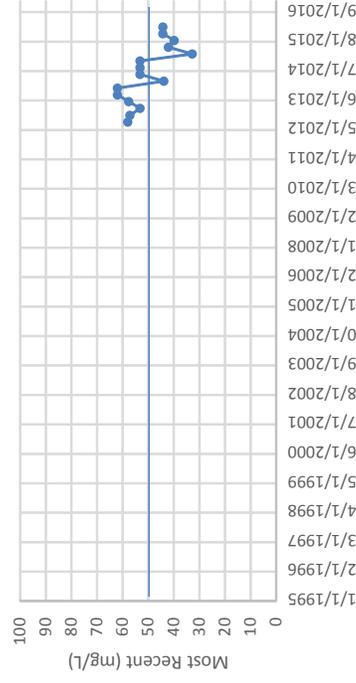
MW4 NO3 as NO3



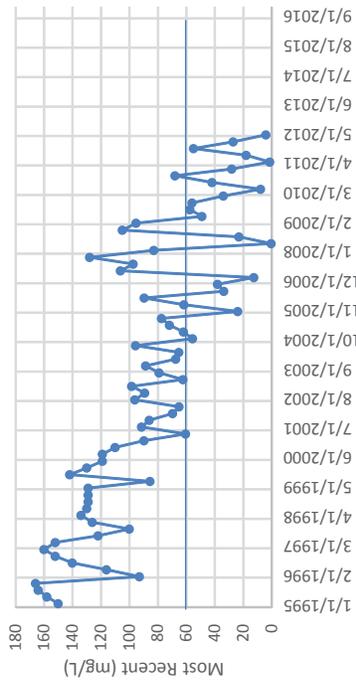
MW5 NO3 as NO3



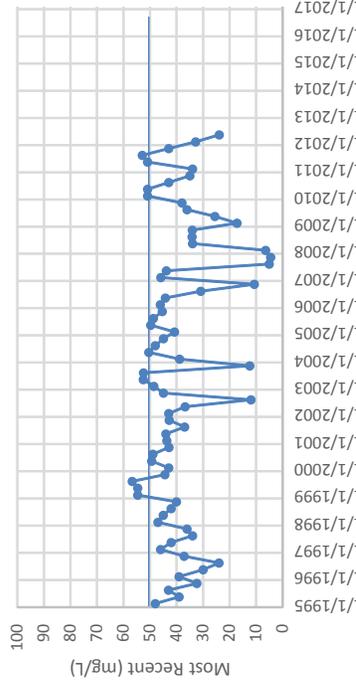
MW6 NO3 as NO3



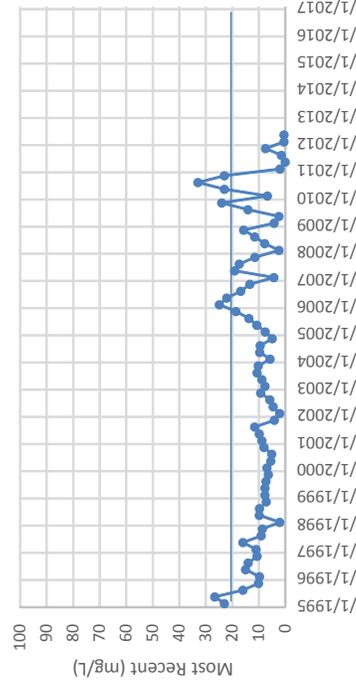
MW1 NO3 as NO3



MW2 NO3 as NO3



MW3 NO3 as NO3



blue line = representative concentration shown on cross sections C-C and D-D and selected for initial concentration in Model Layer 1



Appendix B9  
Time-Concentration  
Plots - WRP 10 MWS  
Nitrate