Groundwater Supply Study

Prepared for

Rainbow Municipal Water District

January 2016

WEST YOST ASSOCIATES
Consulting Engineers

577-00-13-01

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WATER WASTEWATER STORMWATER

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List of Terms and Abbreviations

°F	Degrees Fahrenheit
µS/cm	microSiemens Per Centimeter
afm	Acre-Feet Per Month
afy	Acre-Feet Per Year
AWMP	Agricultural Water Management Program
AWW	Air/Water Wash
CCPP	Calcium Carbonate Precipitation Potential
CDC	California Department of Conservation
CIMIS	California Irrigation Management Information System
CRW	Colorado River Water
CT	Contact Time

CU	Color Units
CVP	Central Valley Project
D/DBPR	Disinfectants and Disinfection Byproduct Rules
DDW	Division of Drinking Water
DWR	California Department of Water Resources
ED	Electrodialysis
EPA	Environmental Protection Agency
EQ	Equalization
$Fe(OH)_3$	Insoluble Iron Hydroxide Complex
Fe^{2+}	Ferrous Iron
Feasibility Study	Alternative Water Source Feasibility Study
FMMP	Farmland Mapping and Monitoring Program
FPUD	Fallbrook Public Utility District
GDF	Groundwater Desalination Facility
GDP	Groundwater Desalination Plant
GMF	Granular Media Filters
GPF	Groundwater Purification Facility
gpm	Gallon Per Minute
GWUDI	Groundwater Under the Direct Influence
H&T	Hungerford and Terry
HAAs	Haloacetic Acids
HDD	Horizontal Directional Drill
IM	Iron and Manganese
LSI	Langelier Saturation Index
mg/L	Milligrams Per Liter
MGD	Million Gallons Per Day
Mn^{2+}	Soluble Manganese
Mn^{4+}	Insoluble Manganese
MnO_2	Insoluble Manganese Dioxide
MRCD	Mission Resource Conservation District
MRDLs	Maximum Residual Disinfect Levels
msl	Mean Sea Level
MWD	Metropolitan Water District of Southern California
NADP	National Atmospheric Deposition Program
NaOCl	Sodium Hypochlorite
NaOH	Sodium Hydroxide
NDMA	N-nitrosodimethylamine
NED	National Elevation Dataset
NF	Nanofiltration
NL	Notification Levels
NOM	Natural Organic Matter
NRCS	Natural Resources Conservation Service

NTU	Nephelometric Turbidity Units
O&M	Operations and Maintenance
OEM	Original Equipment Manufacturer
OPCC	Opinion of Probable Construction Cost
OSU	Oregon State University
PFD	Process Flow Diagram
pMCLs	Primary Maximum Contaminant Levels
PRISM	Parameter-elevation Relationships on Independent Slope
PVC	Polyvinylchloride
RMWD	Rainbow Municipal Water District
RO	Reverse Osmosis
SDCWA	San Diego County Water Authority
SDS	Simulated Distribution System
SLR	San Luis Rey
sMCLs	Secondary Maximum Contaminant Level
SSURGO	Soil Survey Geographic
SWP	State Project Water
SWRCB	State Water Resources Control Board
SWTR	Surface Water Treatment Rule
TDS	Total Dissolved Solids
THMs	Trihalomethanes
TOC	Total Organic Concentration
TTHM	Total Trihalomethanes
USGS	U.S. Geological Survey
VCMWD	Valley Center Municipal Water District
WDL	Water Data Library
WRCC	Western Regional Climate Center
WRF	Water Reclamation Facility
WWTP	Wastewater Treatment Plant
WWW	Waste Wash Water

Errata Sheet

At the request of Rainbow Municipal Water District, West Yost Associates revised *Chapter 8 Conclusions and Recommendations* to include a summary of the anticipated regulatory requirements, key permitting milestones and estimated permitting costs for the groundwater supply project. The revision was completed June 2016.

This report documents the identification and evaluation of a Proposed Groundwater Supply Project that could be implemented by Rainbow Municipal Water District (RMWD) and potential partners to capture and utilize the imported water return flows in the portion of the San Luis Rey Valley Groundwater Basin overlain by RMWD's service area.

ES.1 PURPOSE

RMWD is located in northwestern San Diego County, inland of the City of Oceanside (City) (Figure ES-1). The study area discussed in this report includes the majority of the service areas of RMWD and neighboring districts, Fallbrook Public Utility District (FPUD) and Valley Center Municipal Water District (VCMWD) that drain into the San Luis Rey Valley Groundwater Basin (California Department of Water Resources Bulletin 118 Basin Designation 9-7).

RMWD, FPUD and VCMWD are participating members of the San Diego County Water Authority (SDCWA) and purchase their entire supplies as treated water from SDCWA. These supplies are imported from the State Water Project and Colorado River watersheds and are increasingly expensive and over-allocated. RMWD is seeking alternative supplies to provide some autonomy over water supply decisions and to develop reliable, lower-cost local supplies.

RMWD delivers imported water to its customers for municipal and agricultural use. A portion of the imported water delivered to the San Luis Rey River watershed is not consumed through these uses and becomes a source of recharge to the San Luis Rey Valley Groundwater Basin. As an importer, RMWD has the right to recapture its imported water, provided that this can be done without injury to existing users. No new water right is required for such recapture, but the importer cannot recapture more than it has input into the system without complying with the rules of appropriation.

The purpose of this feasibility level report is to document:

1. Hydrologic analysis performed to quantify the imported water return flows in the San Luis Rey watershed within the defined study area,
2. Water entitlements analysis performed to identify the legal framework and approach for recovering the imported water return flows, and
3. Engineering analysis performed to identify recapture, treatment, and delivery options.

This report addresses the hydrologic and engineering feasibility of the Proposed Groundwater Supply Project at a conceptual level. Environmental review of the Proposed Groundwater Supply Project is a separate topic to be addressed under future efforts.

ES.2 BACKGROUND

Figure ES-2 shows the location of the water districts within the area of interest. RMWD was formed in 1953 and covers approximately 49,800 acres of land, most of which is agricultural. In recent years, there has been significant residential growth, and this trend is expected to continue.

FPUD was first incorporated in 1922, when it served water from local area wells to just 500 acres of land. Today, FPUD has grown to an area of 28,000 acres, providing imported water to approximately 35,000 residents. Roughly half of the water is used for agriculture (FPUD, 2015).

VCMWD was established in 1954 and now serves over 64,253 acres. Water use is divided into about 70 percent agriculture, 22 percent residential, and 8 percent commercial (VCMWD, 2015).

RMWD, FPUD and VCMWD purchase their entire treated water supplies from SDCWA, which acts as a wholesale importer for its member agencies. SDCWA's principal supplies consist of water purchased from Metropolitan Water District of Southern California (MWD), water transfers from Imperial Irrigation District that are wheeled through MWD's conveyance facilities, and short-term water transfers needed to offset dry-year reductions in supplies from MWD. The principle source areas for these supplies are the State Water Project and Colorado River watersheds.

The general pattern of water use in the study area pre-World War II was to rely on groundwater as the primary source, with diversions from the San Luis Rey River as a secondary supply source. As development in the study area continued, groundwater levels in the San Luis Rey Valley Groundwater Basin declined and overdraft ensued. As this occurred, base flow in the San Luis Rey River was greatly reduced.

Today, SDCWA imports Colorado River water and Sacramento-San Joaquin Delta water into San Diego County through a series of aqueducts, including the California Aqueduct and the Colorado River Aqueduct, to the San Diego Aqueduct. Figure ES-2 shows the water delivery facilities. Figure ES-3 shows the historical annual imported water deliveries for each agency. A portion of these imported water deliveries become return flows, which recharge the study area portion of the San Luis Rey Valley Groundwater Basin. Imported water purchases have generally increased over time, until the economic downturn in 2008 and recent drought conditions (2011-2015), resulting in increases in imported water return flows.

ES.3 STUDY AREA

Figure ES-4 shows the study area boundary. The study area boundary was delineated based on hydrologic considerations supporting the water budget and numerical model developed for the assessment. An iterative watershed delineation process was used to define the reach of the San Luis Rey River which captured the imported water return flows from RMWD, and that had adequate historical gauging records along the San Luis Rey River. As shown on Figure ES-4, the study area watershed is the portion of the San Luis Rey River hydrologic basin that contains virtually all of RMWD's service area that is within the basin. The consequence of selecting a watershed boundary that captures essentially all of the imported water return flows to the San Luis Rey River (including groundwater underflow) from RMWD is that the study area watershed also includes those portions of FPUD and VCMWD that drain into the San Luis Rey Valley Groundwater Basin. Each of these agencies also purchase and deliver imported water to their customers and are included within the study area. Their imported water return flows co-mingle with RMWD return flows in this portion of the San Luis Rey Valley Groundwater Basin.

ES.4 HYDROLOGIC MODELING

A numerical model was developed to estimate the recoverable imported water within the study area. The groundwater model was developed using the computer program FEMFLOW3D, Version 3.1, which is a program for simulating three dimensional groundwater systems and stream flow using the finite element method. Figure ES-5 shows the model mesh and boundaries. The model boundaries coincide with the study area boundary. The model covers the entire 123,000-acre or 192-square-mile study area with a network of 16,414 elements and 14,921 nodes. Most of the model area is underlain by fractured bedrock terrain. The model is configured to simulate flow in this bedrock terrain and in the alluvial aquifer of the San Luis Rey Valley.

The San Luis Rey River is represented by 212 stream nodes. The 17 simulated tributaries are represented by 493 nodes for a total of 705 stream nodes.

The model was calibrated to hydrologic conditions during water years 1947-1977. This period was selected because streamflow measurements were collected for the San Luis Rey River at both the western model extent and just westward from the eastern model boundary. The model calibration involved scaling the streamflow and precipitation recharge inputs to best match the San Luis Rey River streamflow measurements near the western boundary of the model. The adjusted parameters were: (1) the scaling factor for specifying the San Luis River inflow at the eastern model boundary, (2) the scaling factor for the tributary runoff within the model boundary, and (3) the scaling factor for the groundwater recharge from precipitation. Adjustment were made to these to best match the average annual streamflow at the U.S. Geological Survey (USGS) stream gauge for San Luis Rey River near Bonsall (1104100) for water years 1947-1977.

Based on the calibration results, the model is suitably calibrated and appropriately represents the San Luis Rey Valley stream-aquifer system in the study area for the purposes of conducting the planning-level comparisons of baseline and Proposed Groundwater Supply Project scenarios as presented in this report.

The model simulations included the calibration version of the model for historical water years 1947 through 1977, three baseline simulations, and a projected 30-year groundwater pumping simulation based on the historical hydrology (water years 1947 through 1977), and projected from 2016 through 2046 based on imported water quantities.

The baseline simulations were developed to establish baseline conditions without the Proposed Groundwater Supply Project under three sets of assumptions regarding the future volume of imported water return flows to the study area. The project pumping simulation was developed to assess the effect of Proposed Groundwater Supply Project pumping in comparison to the three baseline simulations.

The hydrologic analysis conducted to support development of the hydrologic model determined that projected imported water return flows recharging the study area portion of the San Luis Rey River Valley Groundwater Basin for water years 2016 through 2046 ranged from approximately 7,200 to 7,600 acre-feet per year (afy).

The results of modeling of the study area portion of the San Luis Rey River Valley Groundwater Basin indicated that median annual pumping rates up to 5,700 afy, or 460 acre-feet per month (afm), could be supported without significant impacts to the hydrologic conditions that would exist in the absence of imported water. Groundwater pumping of this scale under the Proposed Groundwater Supply Project would have only limited effects on the projected future hydrology of the study area portion of the groundwater basin. These effects were limited to localized drawdowns near active pumping wells. These effects can be limited by siting wells at locations away from neighboring wells and by adjusting pumping rates during project operations to limit potentially adverse drawdown in neighboring wells.

A capacity of 4,000 afy or 333 afm was selected for preliminary design purposes. These pumping rates are less than evaluated in the model, because the modeled rates include some very wet years when simulated pumping rates were higher than would be implemented in the Proposed Groundwater Supply Project. Groundwater levels in the study area change quickly in response to stream flow events and groundwater pumping, because of the relatively large size of the study area watershed in relation to the relatively limited extent and thickness of the alluvial aquifer. Project infrastructure would not be designed to capture relatively infrequent and unpredicted able events.

ES.5 WATER RIGHTS OVERVIEW AND DISCUSSION

Review of the water rights associated with the Proposed Groundwater Supply Project was undertaken and is contained in Chapter 6. Part 1 of Chapter 6 provides an overview of the water rights rules pursuant to California law. Those rules were applied, as described in Chapter 6, based upon review of the conclusions from the hydrologic analysis[1].

The primary legal principles that govern an imported water return flow recapture project are summarized as: (1) the importing agency has a right of recapture the imported water return flow it brought into the basin by its efforts; and (2) the recapture is not allowed to adversely affect native water and uses thereof.

Based on the hydrologic analysis, this report concludes, among other things, that native water and uses thereof will not be affected except in the vicinity of the proposed project wells, and that those impacts can be mitigated. This is based upon a project yield greater than what is ultimately recommended, which reduction may reduce or eliminate these localized impacts. Continued pumping by others is assumed in the hydrologic analysis. Mitigation of any such impacts should be accomplished.

[1] As stated above, this report addresses the hydrologic and engineering feasibility of the Proposed Groundwater Supply Project at a conceptual level. Environmental review of the Proposed Groundwater Supply Project is a separate topic to be addressed under future efforts.

The recommended project yield is greater than the volume of water RMWD imports. Contractual or other arrangements to ensure that RMWD is entitled to recapture the increment of imported water return flow contributed by other agencies is recommended. Other observations and recommendations are described in Chapter 6.

The recapture project itself increases beneficial use of imported water, which is endorsed by the California Constitution Article X section 2, and the California Water Code. It would also reduce the need to export water from the California Bay-Delta, with attendant statewide benefits.

ES.6 TREATMENT AND DISTRIBUTION ALTERNATIVES ANALYSIS

Drinking water standards were found to be the applicable water quality standards, and treatment to reduce iron, manganese, and total dissolved solids (TDS) concentrations was found to be necessary to comply with these standards, based on the results of the treatment analysis, which included review of available water quality data, consideration of alternative uses for the groundwater supply, and applicable water quality goals.

Figure ES-6 shows the groundwater conveyance and desalination alternatives. The alternatives considered were treatment of 3.6 mgd (approximately 4,000 afy) of raw water at:

> Alternative 1: a Proposed Bonsall Basin Groundwater Desalination Facility using preoxidation with free chlorine followed by greensand filtration and RO; and
>
> Alternative 2: conveyance of 3.6 mgd of raw water to the City of Oceanside Mission Basin Groundwater Purification Facility for treatment. Under both alternatives, brine would be disposed via the City ocean outfall.

Estimated probable costs in present day dollars per acre-feet of water treated over the 30-year period from 2016 through 2046 were relatively similar at $920 per acre-foot for the Proposed Bonsall Basin Groundwater Desalination Facility, and $881 per acre-foot for treatment at the City of Oceanside Mission Basin Groundwater Purification Facility. The estimated probable costs include a 20 percent estimating contingency, a 10 percent construction contingency, and a 30 percent allocation for other project costs such as administration, construction management, and engineering services during construction. Land acquisition costs were not included in the costs estimates.

ES.7 CONCLUSIONS

The findings of this preliminary analysis of the Proposed Groundwater Supply Project are favorable in terms of hydrology, water entitlements, water quality and treatment, and disposal of reverse osmosis (RO) concentrate.

ES.8 RECOMMENDATIONS

The following are recommended next steps for further evaluation of the Proposed Groundwater Supply Project.

ES.8.1 Information Gathering

Stakeholder agencies should be contacted to share information on Proposed Groundwater Supply Project and to assess opportunities for cooperation. The following specific steps are recommended.

- The City of Oceanside should be contacted to assess available capacity at the Mission Basin Groundwater Purification Facility and brine line, permitting requirements, and connection fees and ongoing treatment costs. This information should be used to refine the estimated cost of Alternative 2, Treatment at the Mission Basin Groundwater Purification Facility.
- FPUD and VCMWD should be engaged to assess interest and opportunities for conducting the Proposed Groundwater Supply Project as a multi-agency project.
- Caltrans should be contacted to further assess the feasibility of pipeline construction within Caltrans rights of way.

ES.8.2 Water Rights

The source of imported water to RMWD is based upon RMWD's purchase of water from the SDCWA, and the water rights holders for the Colorado River and SWP water which is imported. Water right terms and contractual arrangements can be the basis for limitations on imported water recapture. Typically, the wholesale agencies support imported water recapture projects due to the constraints on imported water availability. RMWD should verify that there are no water right terms and contractual arrangements limiting imported water recapture.

If RMWD determines to proceed with a project for the recapture of imported water return flow, it should make its intent to cease abandonment, and to recapture, known by public notice to potentially affected parties, including RMWD customers, the City of Oceanside, FPUD, VCMWD, and potentially the Santa Margarita River Watermaster, to update them regarding the Proposed Groundwater Supply Project and to identify any concerns.

RMWD should consider protesting water rights filings at the SWRCB that might involve diversion of its imported water return flow, to ensure inclusion of protective terms such as standard permit term 25, which specifically addresses the right to imported water return flow.

ES.8.3 Siting Studies

Siting studies are recommended to identify and prioritize potential well locations, the site of the Proposed Bonsall Basin Groundwater Desalination Facility, and pipeline alignments.

ES.8.4 Treatment Studies

The following next steps are recommended for the development of the treatment process:

- Conceptual level water quality and treatment evaluation should be expanded to the level of a 10 percent design.
- Additional sampling should be conducted at representative locations with the proposed wellfield. Samples should be collected quarterly for all constituents with primary MCLs, secondary MCLs, unregulated Notification Levels, as well as general mineral and physical parameters needed to define the treatment train. The design basis water quality should be re-evaluated after completing the four rounds of sampling.
- Sizing of the iron and manganese treatment should be reevaluated based on the results of the additional iron and manganese sampling.
- Sizing of the RO treatment system should be reevaluated based on the TDS and chloride concentrations observed in the additional sampling.
- Additional suppliers of pressure filters for iron and manganese treatment should be considered in the development of the 10 percent design to assure the opportunity for a competitive bid.
- Additional suppliers of RO membranes (e.g., Dow in addition to already included Toray and Hydranautics) should be considered in the 10 percent design to assure the opportunity for a competitive bid.
- A life cycle cost evaluation should be performed as part of the 10 percent design.

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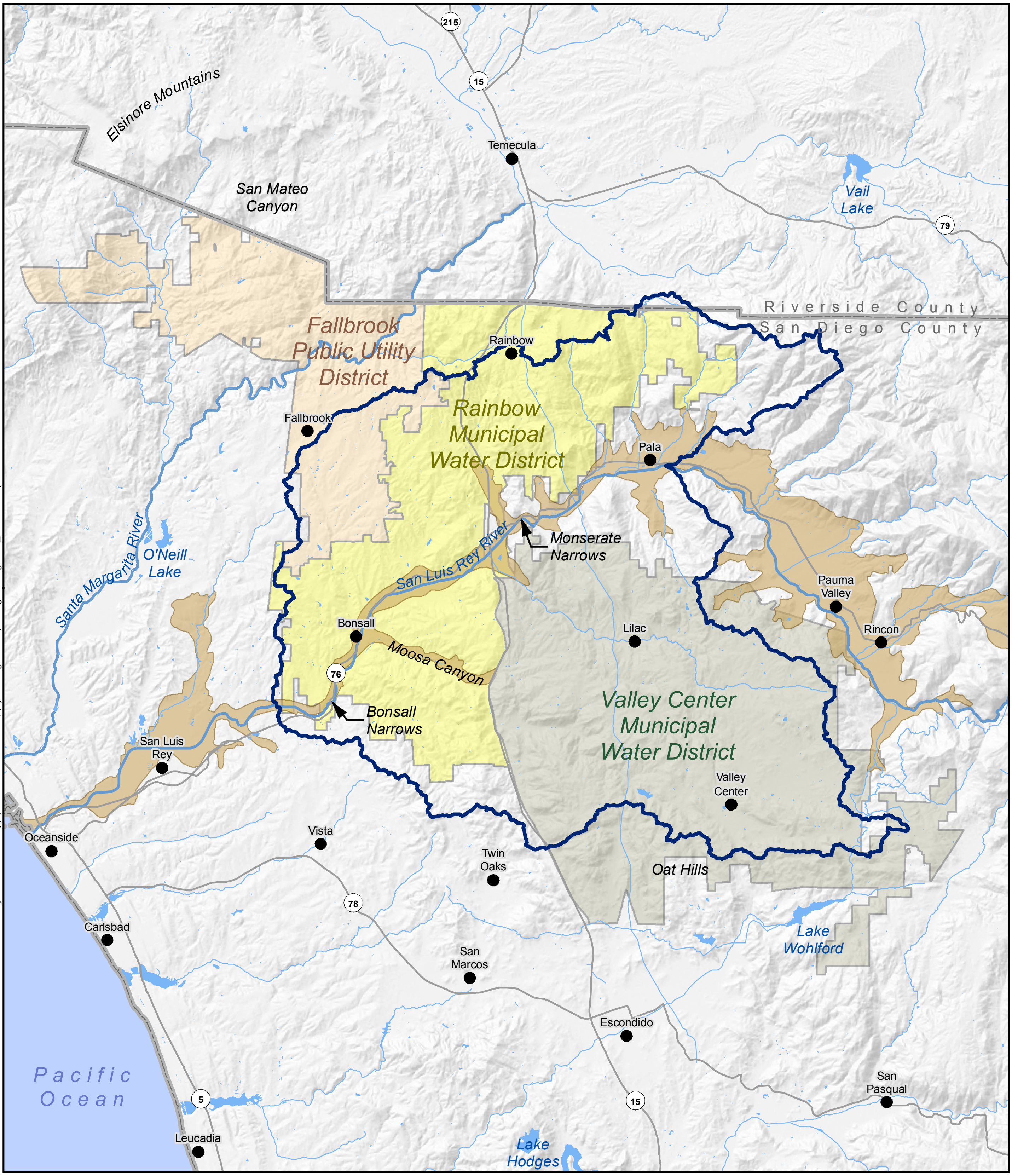

Symbology

- Study Area
- DWR Bulletin 118 San Luis Rey Valley Groundwater Basin (Designation 9-7)

N

0 2 4

Scale in Miles

RAINBOW MUNICIPAL WATER DISTRICT
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Figure ES-1

Location Map

Rainbow Municipal Water District
Groundwater Supply Study

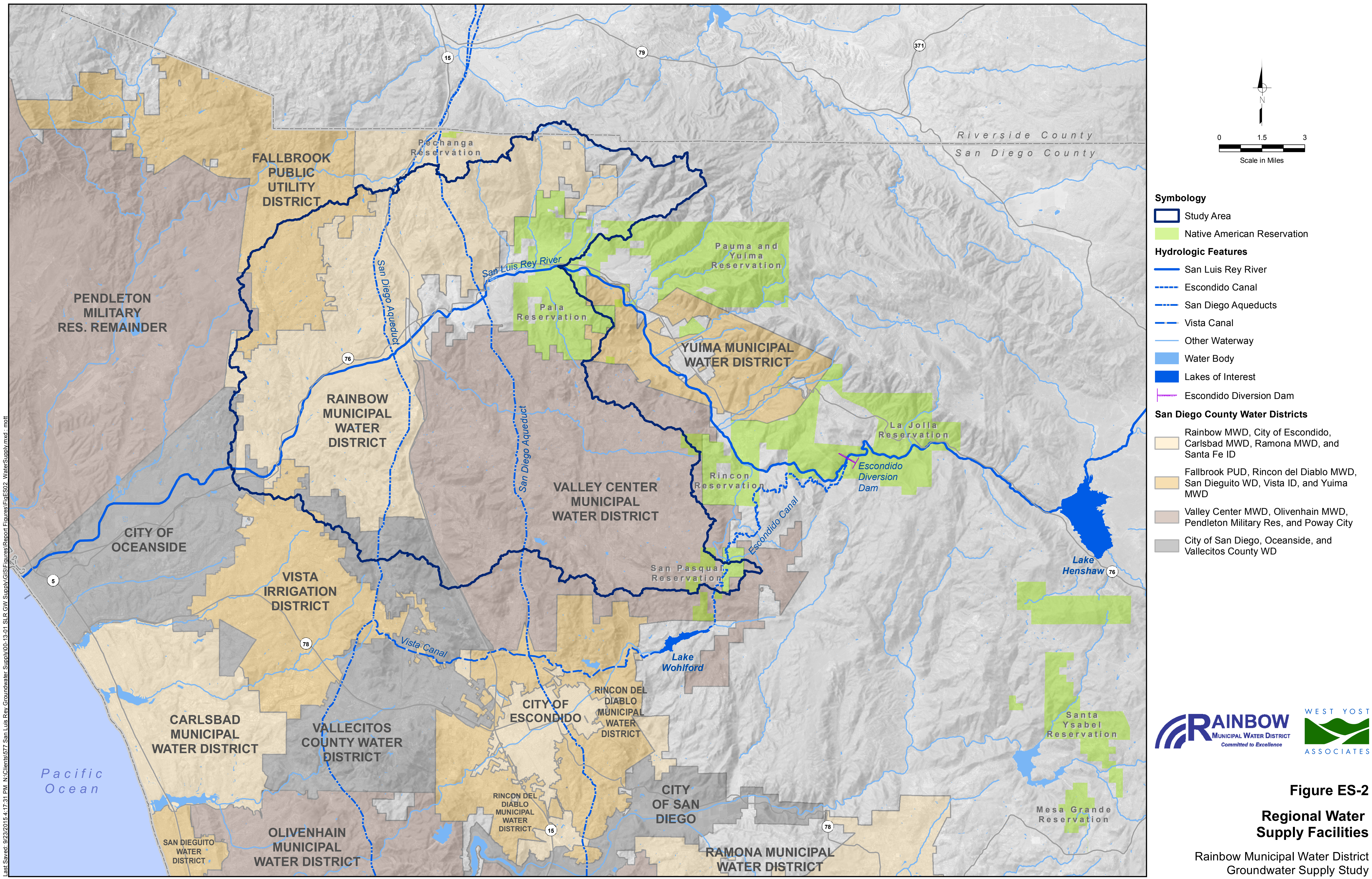
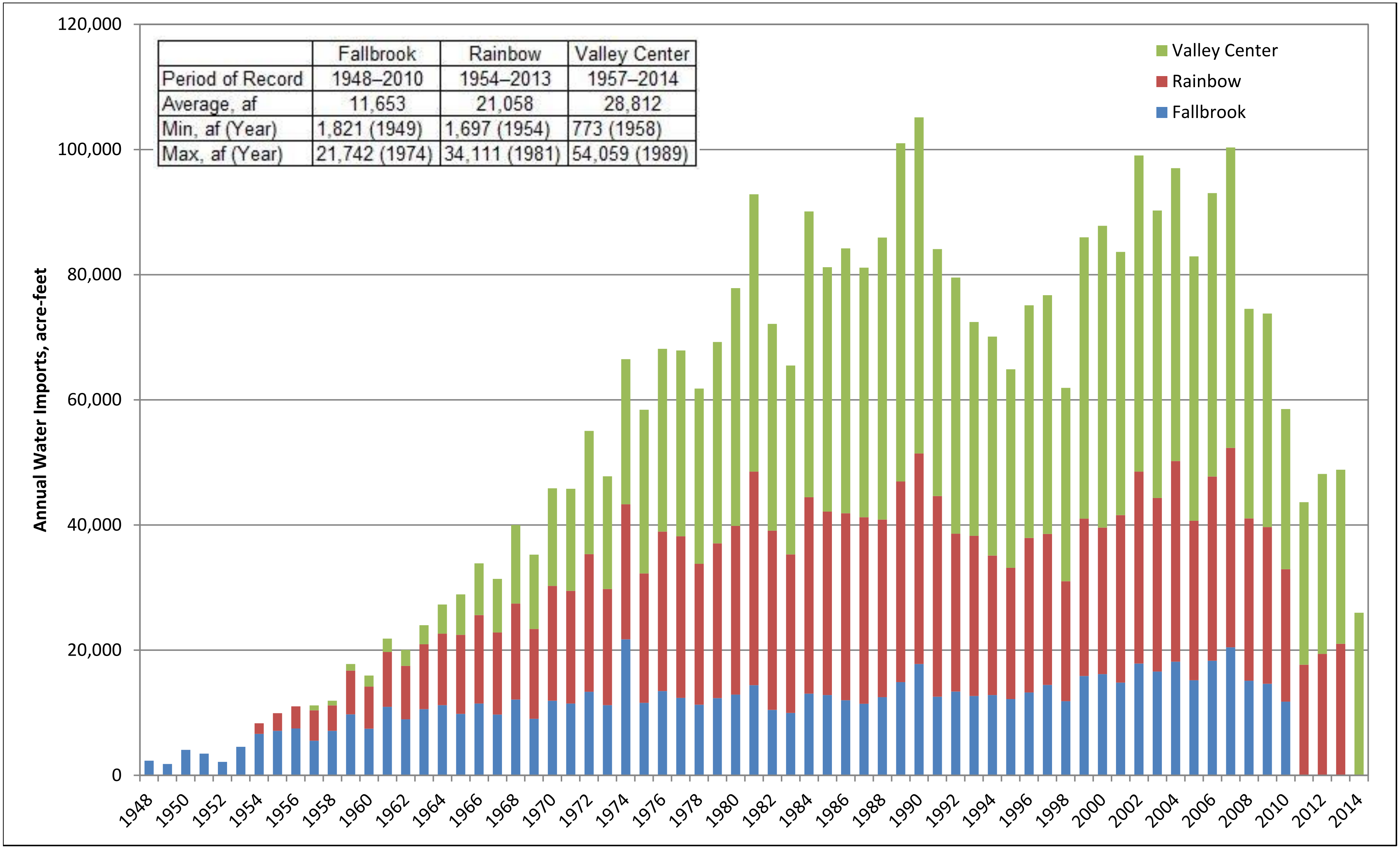


Figure ES-2

Regional Water Supply Facilities

Rainbow Municipal Water District
Groundwater Supply Study



	Fallbrook	Rainbow	Valley Center
Period of Record	1948–2010	1954–2013	1957–2014
Average, af	11,653	21,058	28,812
Min, af (Year)	1,821 (1949)	1,697 (1954)	773 (1958)
Max, af (Year)	21,742 (1974)	34,111 (1981)	54,059 (1989)

RAINBOW
Municipal Water District
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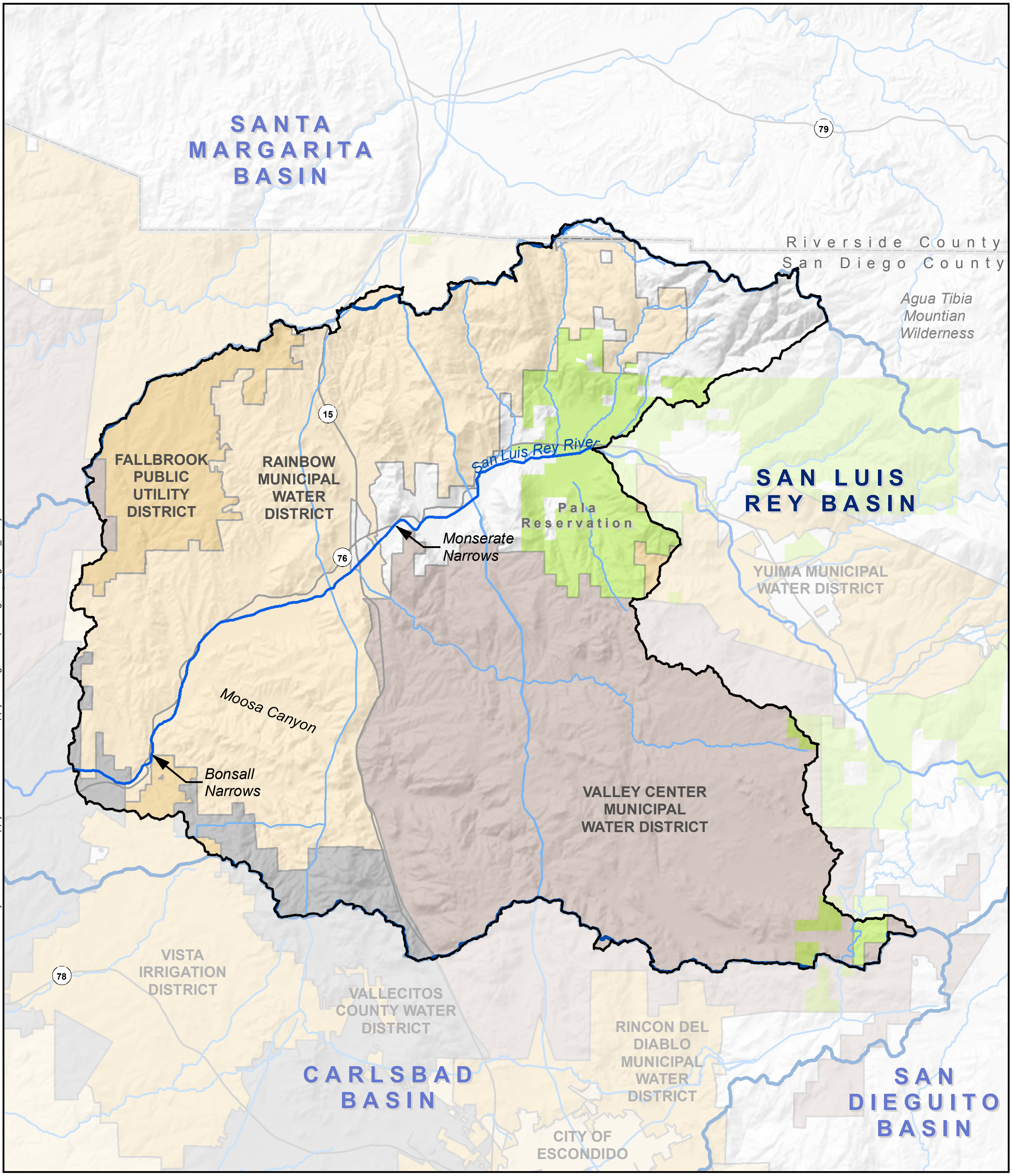
Figure ES-3

Imported Water Deliveries to Study Area

Rainbow Municipal Water District
Groundwater Supply Study



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Symbology

Study Area

Hydrologic Basins

Hydrologic Features

San Luis Rey River

Other Waterway

N

0 1.5 3

Scale in Miles

RAINBOW MUNICIPAL WATER DISTRICT
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Figure ES-4

Study Area

Rainbow Municipal Water District
Groundwater Supply Study

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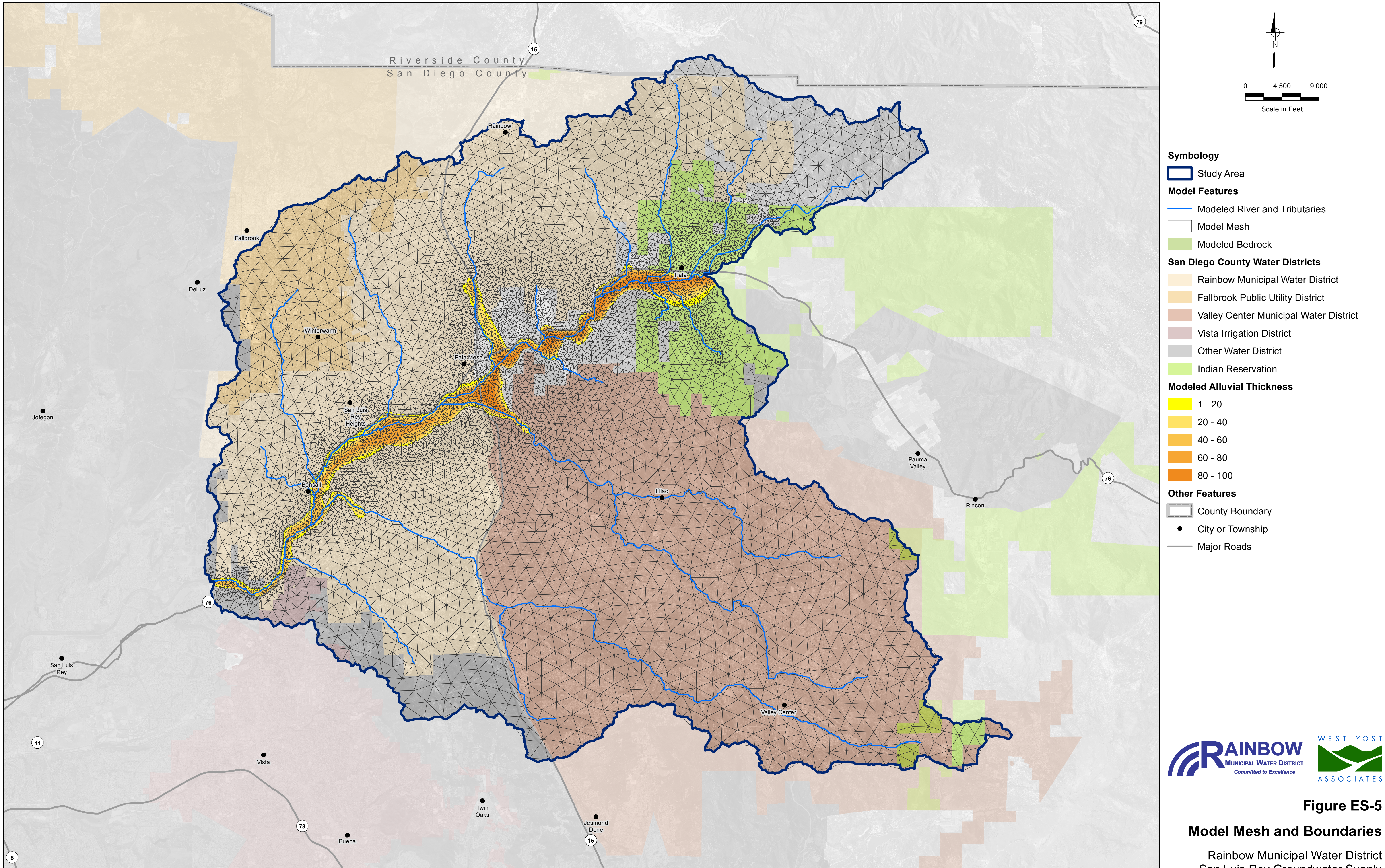

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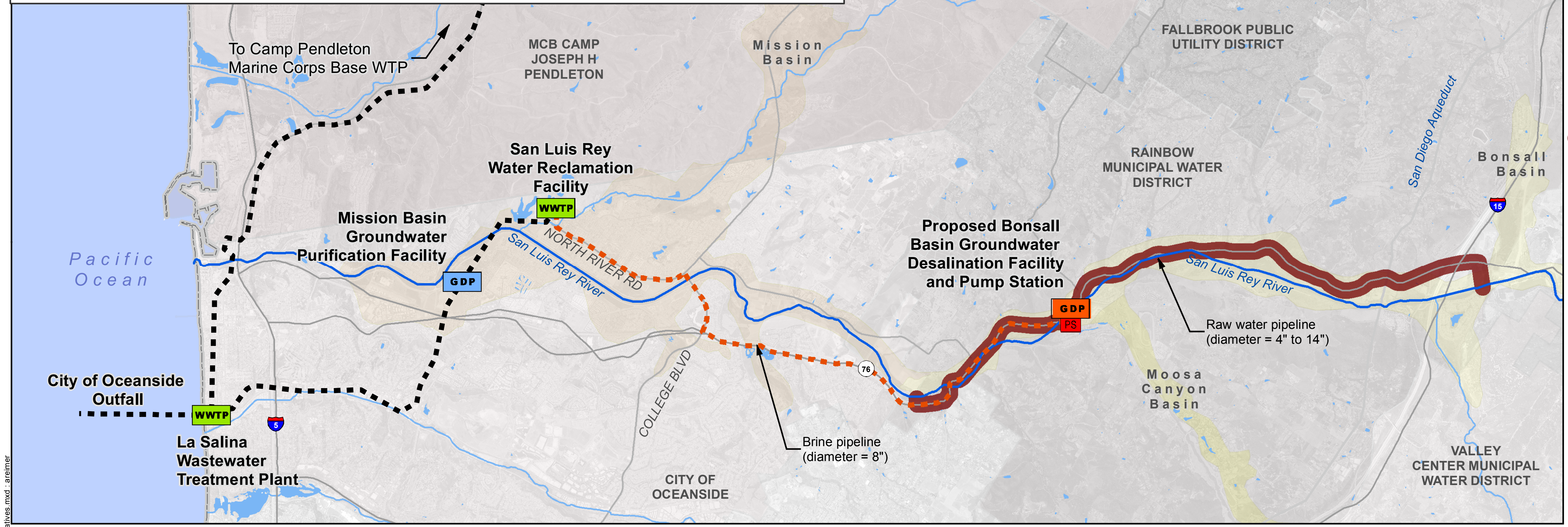
Figure ES-5

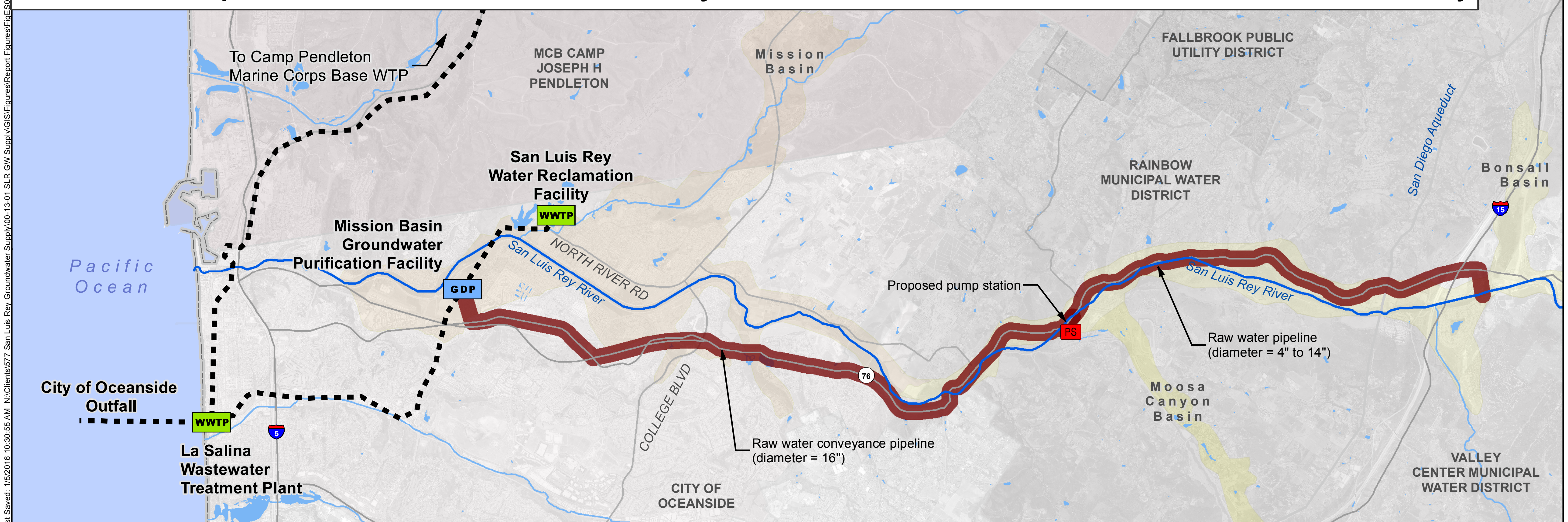
Model Mesh and Boundaries

Rainbow Municipal Water District
San Luis Rey Groundwater Supply

Alternative 1: Proposed Bonsall Basin Groundwater Desalination Plant



Alternative 2: Proposed Bonsall Basin Groundwater Conveyance Facilities to Mission Basin Groundwater Purification Facility



0 0.75 1.5
Scale in Miles

Alternative Alignments and

- Proposed Raw Water Conveyance Pipeline
- Proposed Brine Line
- PS Proposed Pump Station
- GDP Proposed Bonsall Basin Groundwater Desalination Plant

Water Treatment and Conveyance

- WWTP Wastewater Treatment Plant
- GDP Existing Groundwater Desalination Plant
- Existing Brine Line and Outfall

Groundwater Basins

- Bonsall Basin
- Mission Basin
- Moosa Canyon Basin
- Pala Basin

Hydrologic Features

- San Luis Rey River
- Escondido Canal
- San Diego Aqueducts
- Vista Canal
- Natural Waterway

Notes:

1. Under Alternative 1 brackish groundwater would be treated at a proposed new groundwater desalination plant located in the Bonsall Basin. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin and conveyed to the proposed Bonsall Basin Groundwater Desalination Facility for treatment. Brine disposal would be via a proposed new brine line extending from the proposed Bonsall Basin Groundwater Desalination Facility to the City of Oceanside San Luis Rey Water Reclamation Facility, then through the existing brine line to the City of Oceanside Ocean Outfall.
2. Under Alternative 2 brackish groundwater would be treated at the existing City of Oceanside Mission Basin Groundwater Purification Facility. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin, and conveyed to the Mission Basin Groundwater Purification Facility for treatment. Brine disposal would be via the existing brine line to the City of Oceanside Ocean Outfall.

Figure ES-6

Groundwater Conveyance and Desalination Alternatives

Rainbow Municipal Water District
Groundwater Supply Study

This report documents the identification and evaluation of a Proposed Groundwater Supply Project that could be implemented by Rainbow Municipal Water District (RMWD) and potential partners to capture and utilize the imported water return flows in the portion of the San Luis Rey Valley Groundwater Basin overlain by RMWD's service area.

This report addresses the hydrologic and engineering feasibility of the Proposed Groundwater Supply Project at a conceptual level. Environmental review of the Proposed Groundwater Supply Project is a separate topic to be addressed under future efforts.

1.1 PURPOSE

RMWD is located in northwestern San Diego County, inland of the City of Oceanside (Figure 1-1). The study area discussed in this report includes the majority of the service areas of RMWD and neighboring districts, Fallbrook Public Utility District (FPUD) and Valley Center Municipal Water District (VCMWD) that drain into the San Luis Rey Valley Groundwater Basin.

RMWD, FPUD and VCMWD are participating members of the San Diego County Water Authority (SDCWA) and purchase their entire supplies as treated water from SDCWA. These supplies are imported from the State Water Project and Colorado River watersheds and are increasingly expensive and over-allocated. RMWD is seeking alternative supplies to provide some autonomy over water supply decisions and to develop reliable, lower-cost local supplies.

RMWD delivers imported water to its customers for municipal and agricultural use. A portion of the imported water delivered to the San Luis Rey River watershed is not consumed through these uses and becomes a source of recharge to the San Luis Rey River Groundwater Basin. As an importer, RMWD has the right to recapture its imported water, provided that this can be done without injury to existing users. No new water right is required for such recapture, but the importer cannot recapture more than it has input into the system without complying with the rules of appropriation.

The purpose of this report is to document:

1. Hydrologic analysis performed to quantify the imported water return flows in the San Luis Rey watershed within the defined study area,
2. Water entitlements analysis performed to identify the legal framework and approach for recovering the imported water return flows, and
3. Engineering analysis performed to identify recapture, treatment, and delivery options.

1.2 OBJECTIVES

The objectives of this effort were to identify and demonstrate the feasibility of a proposed project that can capture and utilize the RMWD's imported water return flows, without potentially impacting natural flows. Supporting goals are to:

- Evaluate the hydrology of the San Luis Rey basin within the defined study area to quantify the amount of the RMWD's imported water return flows that could be recovered.
- Identify required notifications or filings regarding water rights determinations used for quantifying the imported water return flows available.
- Assist with water quality analysis and feasibility analysis of groundwater treatment plant requirements for these recovered return flows. Develop a footprint and preliminary capital and operations and maintenance (O&M) costs for the associated treatment facilities, wells and conveyance facilities.
- Identify anticipated regulatory requirements and approvals necessary to construct and operate the project.

1.3 SCOPE

The scope of work for this project included the following:

- Information Collection
- Hydrological Assessment
- Water Rights Assessment
- Water Quality, Treatment, and Distribution Assessment
- Regulatory Assessment
- Report Preparation

Information collection activities consisted of research and review of existing studies, and information regarding water supply development, water rights and water quality in the RMWD service area, including the portions overlying the San Luis Rey Valley Groundwater Basin.

The information collected was used to develop an initial water balance and a numerical model of the hydrologic system, including potential return flow contributions from FPUD, RMWD and VCMWD. The hydrological model was used to quantify baseline streamflow and groundwater discharge conditions without proposed project pumping. The model was also used to implement a groundwater pumping scenario and estimate the recoverable imported water return flows. The groundwater pumping scenario was also compared to the baseline conditions simulations to assess changes in streamflow and groundwater discharge caused by groundwater pumping.

Water quality from existing wells was evaluated for selected constituents to assess the need for groundwater treatment. Recommended treated water quality goals and constituent removal requirements were developed. Potential treatment process trains, including waste stream disposal options, were then developed based on constituent removal requirements. Conceptual design of facilities, including a schematic of the treatment process, process system sizing/performance parameters, an overall site plan and conceptual capital and O&M costs, was also developed. A conceptual plan based on expected production and existing distribution system piping was developed to deliver water to the intended users.

The expected permitting and regulatory approvals necessary to support project development were identified and key regulatory permitting milestones were summarized.

The scope for the water rights assessment included providing:

- An overview of the water right issues relevant to the recovery of imported water return flows by the Proposed Groundwater Supply Project
- A focused discussion of the water rights aspects of the Proposed Groundwater Supply Project based on the conclusions of the hydrologic analysis
- Recommendations for further steps regarding rights to the recovery of imported water return flows.

The water right analysis does not address potential environmental impacts of the proposed groundwater supply project.

1.4 ORGANIZATION

This report is organized as follows:

- Executive Summary
- Chapter 1. Introduction
- Chapter 2. Background and Previous Studies
- Chapter 3. Physical and Hydrologic Setting
- Chapter 4. Groundwater-Stream Flow Model Development
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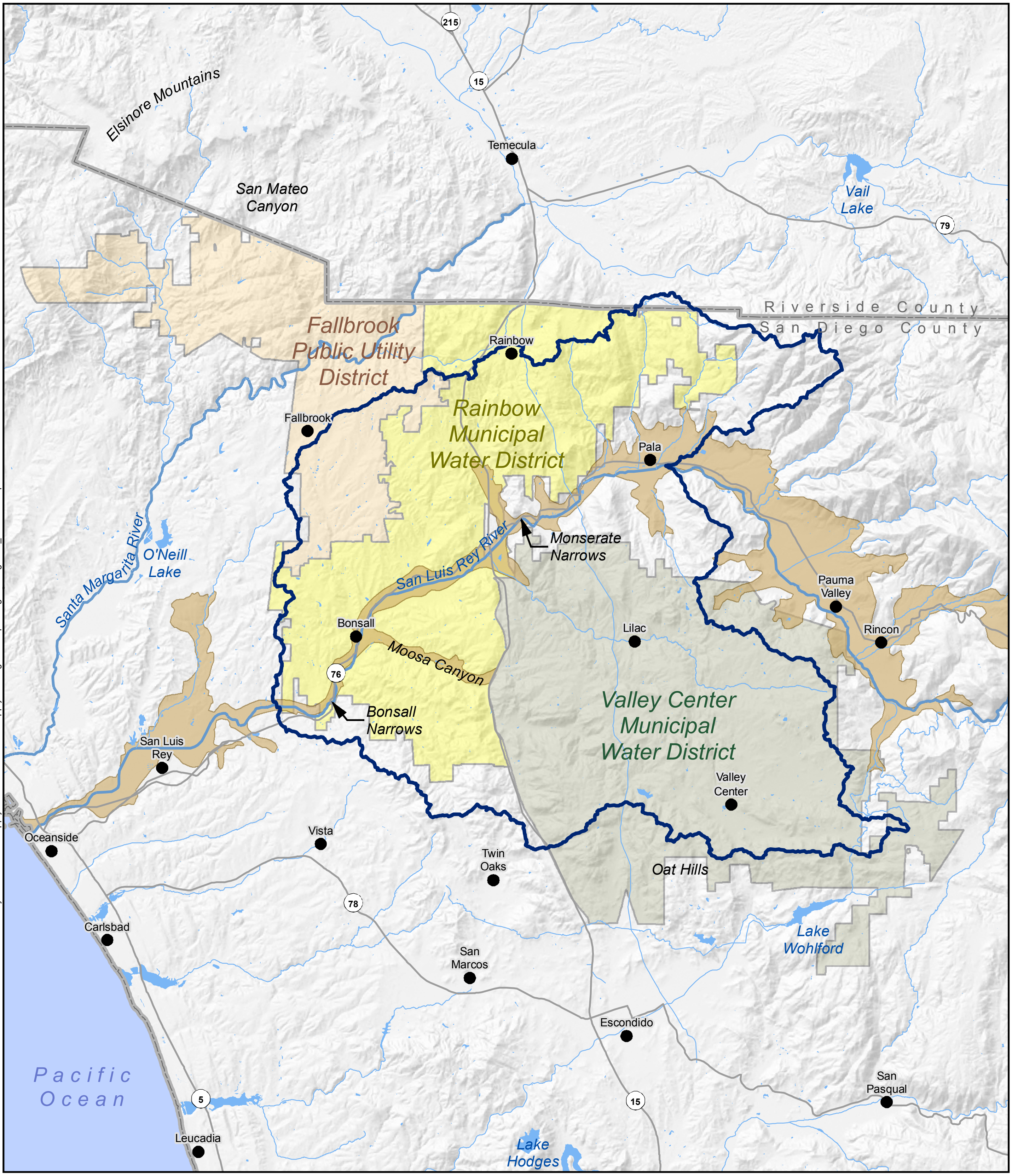
- Chapter 9. References
- Appendices



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Symbology

- Study Area
- DWR Bulletin 118 San Luis Rey Valley Groundwater Basin (Designation 9-7)

N

0 2 4

Scale in Miles

RAINBOW MUNICIPAL WATER DISTRICT
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Figure 1-1

Location Map

Rainbow Municipal Water District
Groundwater Supply Study

2.1 DISTRICT BACKGROUND

Figure 2-1 shows the location of the water districts within the area of interest. RMWD was formed in 1953 and covers approximately 49,800 acres of land, most of which is agricultural. In recent years, there has been significant residential growth, and this trend is expected to continue. That land which remains in its natural state is mostly chaparral, oak, and coastal sage vegetation (RMWD, 2010).

FPUD was first incorporated in 1922, when it served water from local area wells to just 500 acres of land. Today, FPUD has grown to an area of 28,000 acres, providing imported water to approximately 35,000 residents. Roughly half of the water is used for agriculture (FPUD, 2015).

VCMWD was established in 1954 and now serves over 64,253 acres. Water use is divided into about 70 percent agriculture, 22 percent residential, and eight percent commercial (VCMWD, 2015).

RMWD, FPUD and VCMWD purchase their entire treated water supplies from SDCWA, which acts as a wholesale importer for its 24 member agencies. SDCWA's principal supplies consist of water purchased from Metropolitan Water District of Southern California (MWD), water transfers from Imperial Irrigation District that are wheeled through MWD's conveyance facilities, and short-term water transfers needed to offset dry-year reductions in supplies from MWD. The principle source areas for these supplies are the State Water Project and Colorado River watersheds.

2.2 WATER SUPPLY DEVELOPMENT

The general pattern of water use in the study area pre-World War II was to rely on groundwater as the primary source, with diversions from the San Luis Rey River as a secondary supply source. As development in the study area continued, groundwater levels in the San Luis Rey Valley Groundwater Basin declined and overdraft ensued. As this occurred, base flow in the San Luis Rey River was greatly reduced.

SDCWA was formed on June 9, 1944, by the California State Legislature and operates under the County Water Authority Act for the purpose of importing water into San Diego County. Water imports to the study area began in 1948 for FPUD, 1954 for RMWD, and 1956 for VCMWD. As reliance on imported water increased, reliance on groundwater decreased and groundwater levels in the San Luis Rey Valley Groundwater Basin began to recover. Base flows in the river and the extent of riparian vegetation increased.

Today, SDCWA imports Colorado River water and Sacramento-San Joaquin Delta water into San Diego County through a series of aqueducts, including the California Aqueduct and the Colorado River Aqueduct, to the San Diego Aqueduct. Figure 2-1 shows the water delivery facilities in the region, including the San Diego Aqueduct, Lake Henshaw, Escondido Dam, Escondido Canal, Lake Wohlford, and the Vista Canal.

RMWD is a SDCWA member agency. Member agency status entitles RMWD to directly purchase water for its needs from SDCWA on a wholesale basis. RMWD currently does not utilize groundwater as a source of water supply nor does it distribute recycled water to meet any of its water demands (RMWD, 2010).

FPUD and VCMWD also rely solely on imported water purchased from MWD through SDCWA (FPUD, 2010; VCMWD, 2010). Neither of these agencies rely on other sources of supply at this time.

Figure 2-1 shows the location of Lake Henshaw on the San Luis Rey River east of the study area. Note that releases from Lake Henshaw by Vista Irrigation District are typically diverted from the San Luis Rey River at the Escondido Diversion Dam and flow through the Escondido Canal to Lake Wohlford (Figure 2-1). These diverted flows do not affect the water budget for the study area.

2.3 PREVIOUS STUDIES

Previous studies were fundamental in gaining an understanding of the hydrology of the study area. The following summarizes studies considered during this evaluation.

2.3.1 Hydrologic and Salt Balance Investigations Utilizing Digital Models, Lower San Luis Rey River Area, San Diego County, California, USGS WRI 24-74

A comprehensive water quality study was completed to compute the hydrologic and salt balances in the lower San Luis Rey River Groundwater Basin. Digital hydrologic models were created and used to supplement the development of the hydrologic balances due to the sparse availability of data at the time of investigation. Salt balances were computed for near steady-state inflows and outflows for 1972, when inflows and outflows were not steady-state. The 1972 analysis revealed that salt inflow exceeded salt outflow. Hydraulic conductivities within the model were estimated from specific-capacity tests for older and younger alluvium (Moreland, 1974). These estimates of hydraulic conductivity were used in the development of the numerical model discussed in this report.

2.3.2 Predicted Effects of a Proposed Water-Resource Management Plan in the Lower San Luis Rey River Valley, California, Using Digital Ground-Water Flow Models, USGS OFR 76-754

A water resource management plan was proposed using the U.S. Geological Survey (USGS) WRI 24-74 digital hydrologic model. Modifications were made to the model and two forecasts of water-level changes and salt balance were projected. While recognizing model limitations, the predicted water-level changes from 1972-1977 were compared with the same scenario but including the new water management plan. Under the management plan, groundwater-levels increased in the Bonsall basin and the salt balance improved (Skrivan, 1976).

2.3.3 Maps of the Bonsall Area of the San Luis Rey River Valley, San Diego County, California Showing Geology, Hydrology, and Ground-Water Quality, USGS WRIR 85-4112

USGS WRIR 85-4112 consists of four detailed maps of the Bonsall groundwater basin. The maps all include the same extents and classification of alluvium, colluvium, or crystalline rocks within the area. The map details include: thickness of the alluvial fill, the groundwater-level contours from November 1983 with hydrographs of selected wells, groundwater quality from the spring of 1960 with graphs showing changes in the concentration of dissolved-solids for selected wells, and groundwater quality in the spring of 1984 (Izbicki, 1985).

The Bonsall area maps were used in the current groundwater supply study for delineating the extent and thickness of the alluvial aquifer, illustrating the depth to groundwater for selected wells, and comparing temporal changes in groundwater quality between 1960 and 1984.

2.3.4 Demineralization of Groundwater within the Rainbow Municipal Water District – Phase I: Quantity and Quality Analysis (1/26/1996)

Phase I of the Demineralization of Groundwater within RMWD provides an evaluation of the groundwater quantity and quality of the Bonsall basin, to potentially develop a local groundwater source for RMWD. Different pumping scenarios and potential impacts on the groundwater flow system were modeled, in conjunction with groundwater quality conditions, to evaluate the implementation of desalination. It was found that the eastern Bonsall groundwater basin had better water quality (for total dissolved solids [TDS]) than in the western part, and hence would have a reduced cost of demineralization. Using a groundwater flow model to determine a sustainable amount of water to pump and realizing the need for a well field instead of a single point of extraction, desalinized groundwater was identified as an additional source of water to RMWD, with the added benefit of shifting the basin's salt balance and improving overall groundwater quality (Camp Dresser and McKee, 1996a).

2.3.5 Demineralization of Groundwater within the Rainbow Municipal Water District – Phase II: Quantity and Quality Analysis (4/9/1996)

Phase II of the Demineralization of Groundwater within RMWD was an extension of Phase I in which the feasibility of developing a local groundwater source for RMWD was studied. Phase II sought to define the demineralization options and costs. The two treatment options studied were electrodialysis (ED) and reverse osmosis, of which reverse osmosis was found to be the most efficient candidate both in terms of practicality and cost for the eastern Bonsall basin (Camp Dresser and McKee, 1996b).

2.3.6 Alternative Water Source Feasibility Study (01/28/2013)

The purpose of the Alternative Water Source Feasibility Study (Feasibility Study) was to identify ways of diversifying RMWD's water source beyond purchasing treated water from SDCWA and MWD. Water demand was projected based on anticipated future residential development, population growth, and land use. Water diversifications that were addressed included: using raw or treated water instead of potable water for irrigation, possible use of treated effluent from a wastewater treatment plant; exploring the riparian and appropriative rights of water users and the possibility of purchasing, leasing, or exchanging water rights; recapturing imported water within the basin for reuse; and pumping groundwater (a desalting treatment process could be required). It was concluded that the largest efficiencies, both financially and with regard to water availability, would be to pursue irrigating with raw or treated water and recapturing imported water (J.C. Heden, 2013).

The Feasibility Study identified a first priority recommendation of recapturing imported water to the San Luis Rey River Groundwater Basin, as addressed in this report.

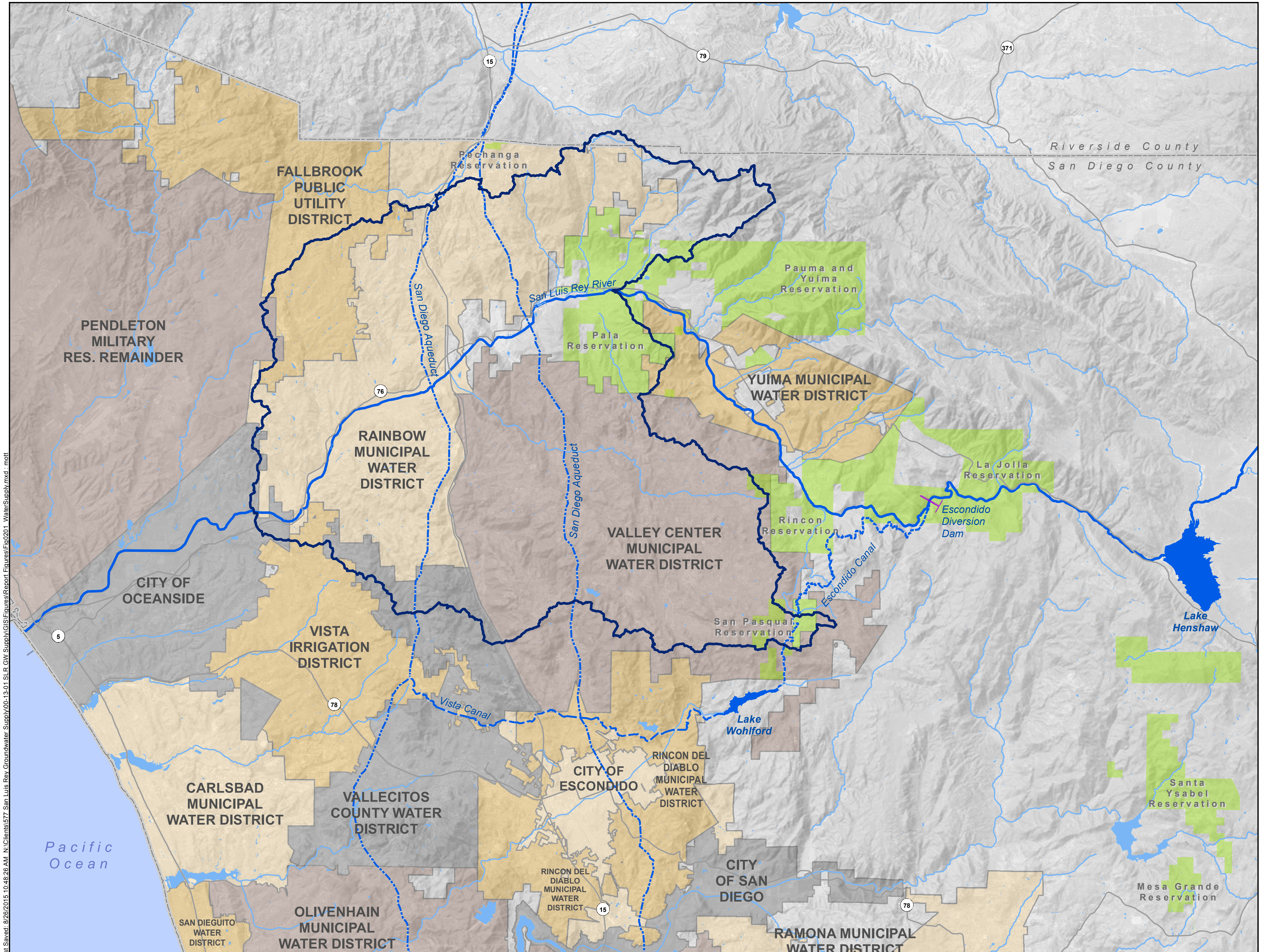


Figure 2-1

Regional Water Supply Facilities

Rainbow Municipal Water District
Groundwater Supply Study

This chapter describes the physical and hydrologic setting of the study area. Information described in this chapter was used to develop the hydrologic model discussed in Chapter 4.

3.1 STUDY AREA

Figure 3-1 shows the study area boundary. The study area boundary was delineated based on hydrologic considerations supporting the water budget and numerical model developed for the assessment, as discussed in Chapter 4. An iterative watershed delineation process was used to define the reach of the San Luis Rey River which captured the imported water return flows from RMWD, and that had adequate historical gauging records along the San Luis Rey River. As shown on Figure 3-1, the study area watershed is the portion of the San Luis Rey River hydrologic basin that contains virtually all of RMWD's service area that is within the basin. The consequence of selecting a watershed boundary that captures essentially all of the imported water return flows to the San Luis Rey River (including groundwater underflow) from RMWD is that the study area watershed also includes those portions of FPUD and VCMWD that drain into the San Luis Rey River Groundwater Basin. Each of these agencies also purchase and deliver imported water to their customers and are included within the study area. Their imported water return flows co-mingle with RMWD return flows in this portion of the San Luis Rey River Groundwater Basin.

3.2 GEOGRAPHY

The study area is located within the San Luis Rey River Valley, in northwestern San Diego County, near its border with Riverside County. Camp Pendleton and the City of Oceanside lie between the study area and the Pacific Ocean to the west (Figure 2-1). VCMWD and Native American Reservations are located to the east of RMWD.

The San Luis Rey River flows westward from Lake Henshaw, through RMWD's service area, and through the City of Oceanside where it then discharges into the Pacific Ocean. The majority of RMWD is in the San Luis Rey River watershed, and approximately 10 percent of the service area is in the Santa Margarita River watershed, to the north. No parts of the Santa Margarita River watershed are included in the study area.

3.3 TOPOGRAPHY

Figure 3-2 shows the USGS National Elevation Dataset (NED) digital elevation model for the study area (USGS, 2011). Elevations within the study area range from approximately 4,600 feet above mean sea level (msl) near Agua Tibia Mountain Wilderness in the northeast to approximately 120 feet msl downstream of Bonsall Narrows to the southwest. Generally, the terrain slopes both down towards San Luis Rey River valley and westward towards the City of Oceanside, where it levels off before meeting the coastline.

3.4 CLIMATE AND PRECIPITATION

Precipitation and temperature data were downloaded from the Oregon State University (OSU) Parameter Elevation Regressions on Independent Slopes Model (PRISM) Climate Group website. The 1971 through 2000 datasets represents the 30-year normals as calculated by the PRISM Climate Group (OSU, 2015).

Figures 3-3 and 3-4 show the average annual minimum and maximum temperature, respectively, for 1971 through 2000 according to the Oregon State University PRISM Climate Group (OSU, 2015). The San Luis Rey River basin within the study area has a relatively temperate climate. Within the study area, average annual minimum temperatures are around 46 to 52 degrees Fahrenheit (ºF) while average annual maximum temperatures are around 74ºF to 78ºF. According to the Western Regional Climate Center (WRCC) station 049378 "Vista 1 NE" experienced average winter highs of 68ºF, average winter lows of 44ºF, average summer highs of 83ºF, and average summer lows of 62ºF between 1962 and 2014 (WRCC, 2014).

Figure 3-5 shows the average annual precipitation over the study area based on the 1971 through 2000 30-year normal from PRISM. Average annual rainfall within the study area is approximately 16 inches (OSU, 2015). As is common with most of California, the majority of the rainfall occurs between October and May. The WRCC Vista 1 NE weather station shows average annual precipitation of 12 inches between 1962 and 2014 (WRCC, 2014). PRISM 30-year normal annual precipitation at WRCC Vista 1 NE is 14 inches.

3.5 LAND AND WATER USE

Figure 3-6 shows the California Department of Water Resources (DWR) Land Use Survey data from 1998 for San Diego County (DWR, 2014). Historically, the primary land use in much of the study area was agricultural. While urban land use has increased with population growth, the majority of the study area is still dedicated to agricultural land use. As can be seen on Figure 3-6, agriculture within the study area consists primarily of citrus/subtropical fruit (predominantly oranges and lemons) and avocado.

3.6 GEOLOGY

Figures 3-7 and 3-8 show the geologic mapping and cross sections within the San Luis Rey River valley. The study area is located within the Peninsular Ranges Province of Southern California. The Peninsular Ranges extend from Baja California through Southern California and are underlain by an extensive plutonic complex known as the Southern California batholith (San Diego County, 2010). The exposed geology within the region is predominately plutonic with marine deposits along the ridgelines, Pleistocene non-marine deposits lining the valley floor, and alluvium and younger stream channel deposits within the river basin (Rogers, 1965). Giessner (1971) classified the Pleistocene non-marine deposits as Pleistocene and Holocene aged alluvium. Izbicki (1985) classified the valley deposits slightly differently, differentiating colluvium from alluvium, and mapping the colluvium with weathered bedrock. All geologic sources of information were taken into consideration when defining the alluvial extents for the numerical model used in this study.

3.6.1 Bedrock and Colluvium

The bedrock formations bounding the San Luis Rey River valley are composed primarily of Cretaceous and upper Jurassic-aged granodiorite, diorite, and tonalite (Rogers, 1965) that have low specific capacities but yield small quantities of water from fractures. Metamorphosed Jurassic marine sedimentary rocks line the peaks and ridges surrounding San Luis Rey River valley.

The bedrock surface is fractured, weathered and locally overlain by colluvium. This zone of colluvium and fractured rock separates the alluvial basins from the underlying crystalline bedrock. The specific capacity of the colluvium, weathered bedrock, and crystalline bedrock decreases with depth; higher capacities are measured in the colluvium and the lowest capacities measured within the upper portions of the crystalline bedrock. The bedrock is essentially impermeable below the fractured zone. Figure 3-8 shows cross sections depicting the bedrock and alluvial thickness used for the model.

3.6.2 Alluvium and Stream Deposits

The upstream portion of the study area contains exposed older Pleistocene aged alluvial sediments, according to Rogers (1965). This older alluvium consists of non-marine sedimentary deposits and terrace deposits composed of fanglomerates and breccias. The majority of the river basin is comprised of Quaternary-aged alluvial and unconsolidated stream deposits.

The DWR Water Well Completion Reports (DWR, 2014b), alluvial thickness contours from Izbicki (1985), and geologic mapping by Giessner (1975) and Rogers (1965) were all used to determine the alluvial extents and thickness for the groundwater flow model discussed in Chapter 5. Figure 3-7 shows the alluvial thickness throughout the study area. Figure 3-8 shows longitudinal and transverse cross sections through the alluvium. The thickness of the alluvial fill varies throughout the study area, ranging from 50 feet near Bonsall Narrows to more than 80 feet near Monserate Narrows (Figures 3-7 and 3-8).

3.7 SOILS

Figure 3-9 shows the Natural Resources Conservation Service (NRCS) Soil Survey Geographic (SSURGO) soil designations (NRCS, 2013). The majority of the study area is covered in sandy loam soils. Loam has a relatively high permeability, therefore the majority of RMWD's service area experiences little runoff, with most excess precipitation and irrigation waters returning to the San Luis Rey River Valley Groundwater Basin via underflow as opposed to surface flow runoff.

3.8 HYDROLOGY

The San Luis Rey River flows westward from Lake Henshaw in the east, past the Escondido Diversion, through RMWD, through the City of Oceanside, and into the Pacific Ocean. Numerous small tributaries drain into the river (Figure 3-10).

3.8.1 Stream Flow

USGS stream gauge data were obtained and analyzed for the nine gauge locations shown on Figure 3-10. Downstream USGS stream gauge 11041000 has a period of record starting October 1929 and ending September 1979. Upstream gauges USGS 1039800 and 11040000 are located close to one another and have a collective period of record spanning from October 1937 through January 1993. Other stream gauges along the San Luis Rey River were evaluated, but the data does not provide sufficient temporal coverage to determine net gains and losses along the San Luis Rey River within the study area.

Figure 3-11 is a graph of net gains and losses to the river for the 1947-1977 period. The San Luis Rey River was a losing stream in the early years when little or no imported water delivery was available (1947 to 1959), and groundwater was still a significant part of the regional water supply. During the period from 1960 to 1964 the San Luis Rey River sustained almost no base flow within the study area. However, by 1965 and extending through the period of record, the river gained flow, likely in response to infiltrating imported water return flows.

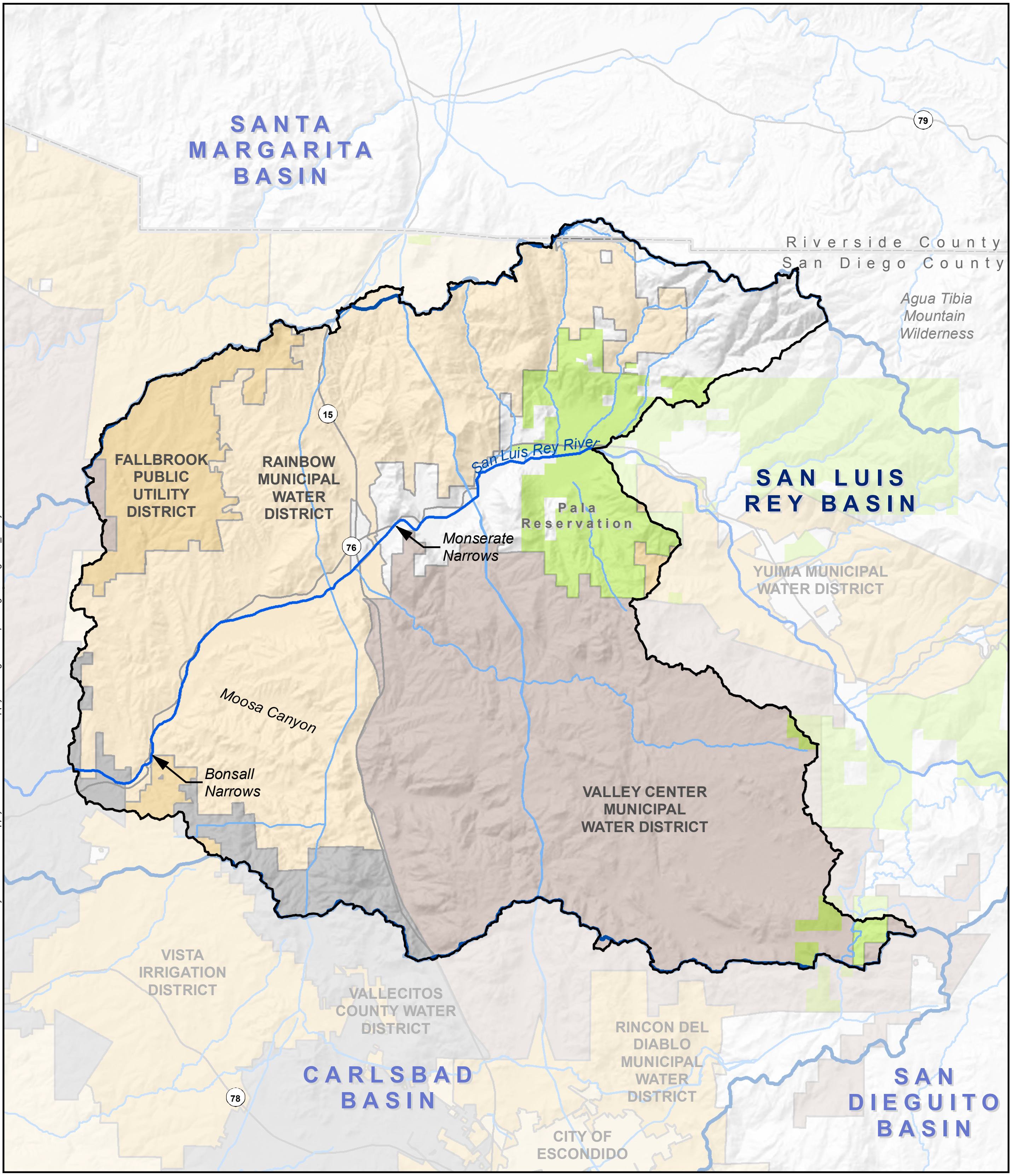
3.8.2 Groundwater

Groundwater in San Luis Rey Valley Groundwater Basin occurs in the alluvial deposits and the underlying colluvium and fractured bedrock. Figure 3-10 shows the extent of the alluvial San Luis Rey Valley Groundwater Basin as defined in California DWR, Bulletin 118, San Luis Rey Valley Groundwater Basin (Basin Designation 9-7). Within the study area, the San Luis Rey Valley Groundwater Basin is further divided into groundwater subbasins. These subbasins are the Moosa Canyon, Bonsall, and the western portion of the Pala subbasins (Figure 3-10). The boundaries of each of these subbasins are based on the following sources of information:

- Metropolitan Water District of Southern California (MWD) San Diego County Basins Report (MWD, 2007)
- United States Geological Survey Hydrologic and Salt Balance Investigations, Water Resources Investigations Report 24-74 (Moreland, 1974)

Groundwater levels within the basin were much lower during the 1950s and 1960s than they are today. This is due to the historic over-pumping of the basin to meet agricultural and urban demands in the 1950s and 1960s, prior to the introduction of imported water. Since the addition of imported water, groundwater levels have mostly returned to pre-development conditions (MWD, 2007). Hydrographs for DWR Water Data Library wells (DWR, 2015) within the modeled alluvial extents are included in Appendix A. Depths to groundwater near the downstream end of the study area range from 0 to 30 feet, or groundwater elevations of 130 to 160 feet, msl. Depths to groundwater near the upstream end of the study area range from 5 to 110 feet below land surface, or approximately 350 to 400 feet elevation, msl, depending on how close to the river the well is located (assuming the well is located within the alluvial basin).

Groundwater quality within the San Luis Rey River Groundwater Basin has been variable over the historical period. Importation of water has resulted in an increase in TDS within the aquifer. In 1960, TDS concentrations measured in wells located within the Bonsall basin were 425 to 1,800 milligrams per liter (mg/L), with the lower TDS concentrations being in the eastern area of the basin. In spring of 1984, TDS concentrations ranged from 574 to 2,370 mg/L (Izbicki, 1985). Again, the wells with TDS below 1,000 mg/L were located in the eastern reaches of the basin.



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Symbology

Study

Hydrologic Basins

Hydrologic Features

San Luis Rey River

Other Waterway

N

0 1.5 3

Scale in Miles

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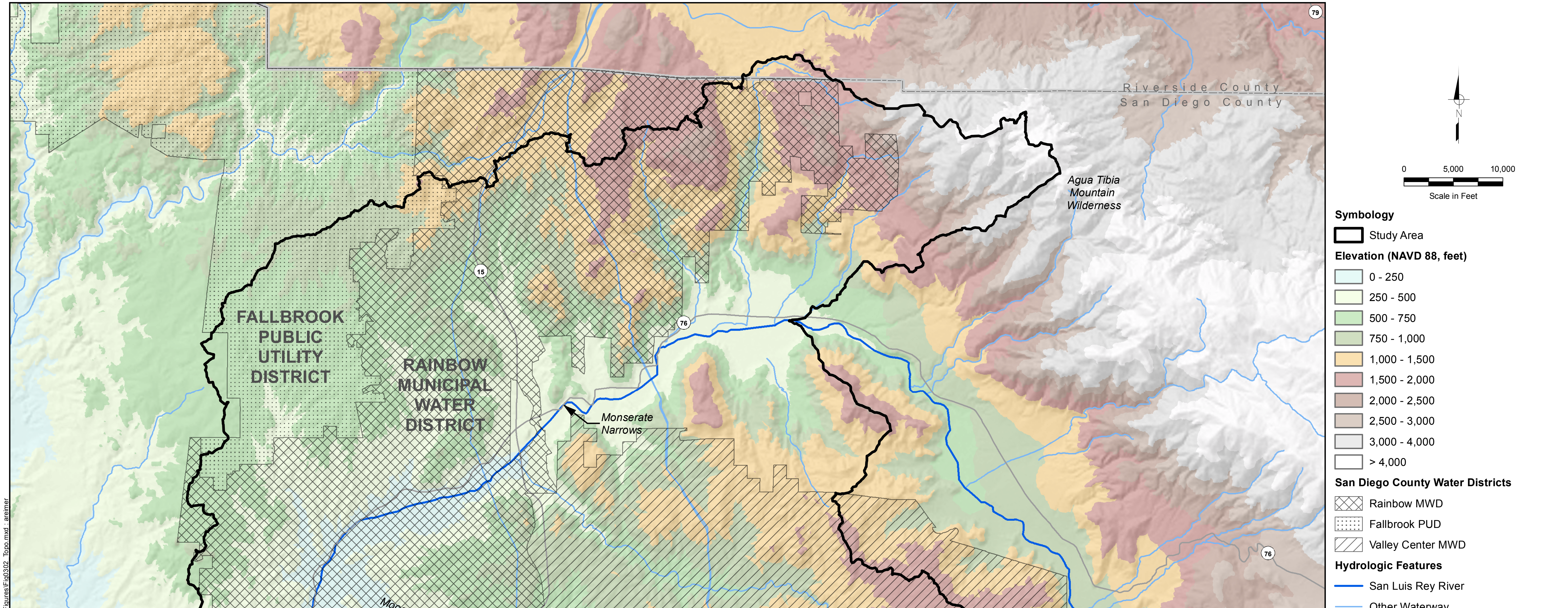
Figure 3-1

Study Area

Rainbow Municipal Water District
Groundwater Supply Study



Notes:
1. Topography is the U.S. Geological Survey National Elevation Dataset 2011 digital elevation models for tiles n34w117 and n34w118. Vertical datum is NAVD 1988 (feet). Data was downloaded from National Map Viewer website at http://viewer.nationalmap.gov/viewer/

Figure 3-2

Topography

Rainbow Municipal Water District
Groundwater Supply Study

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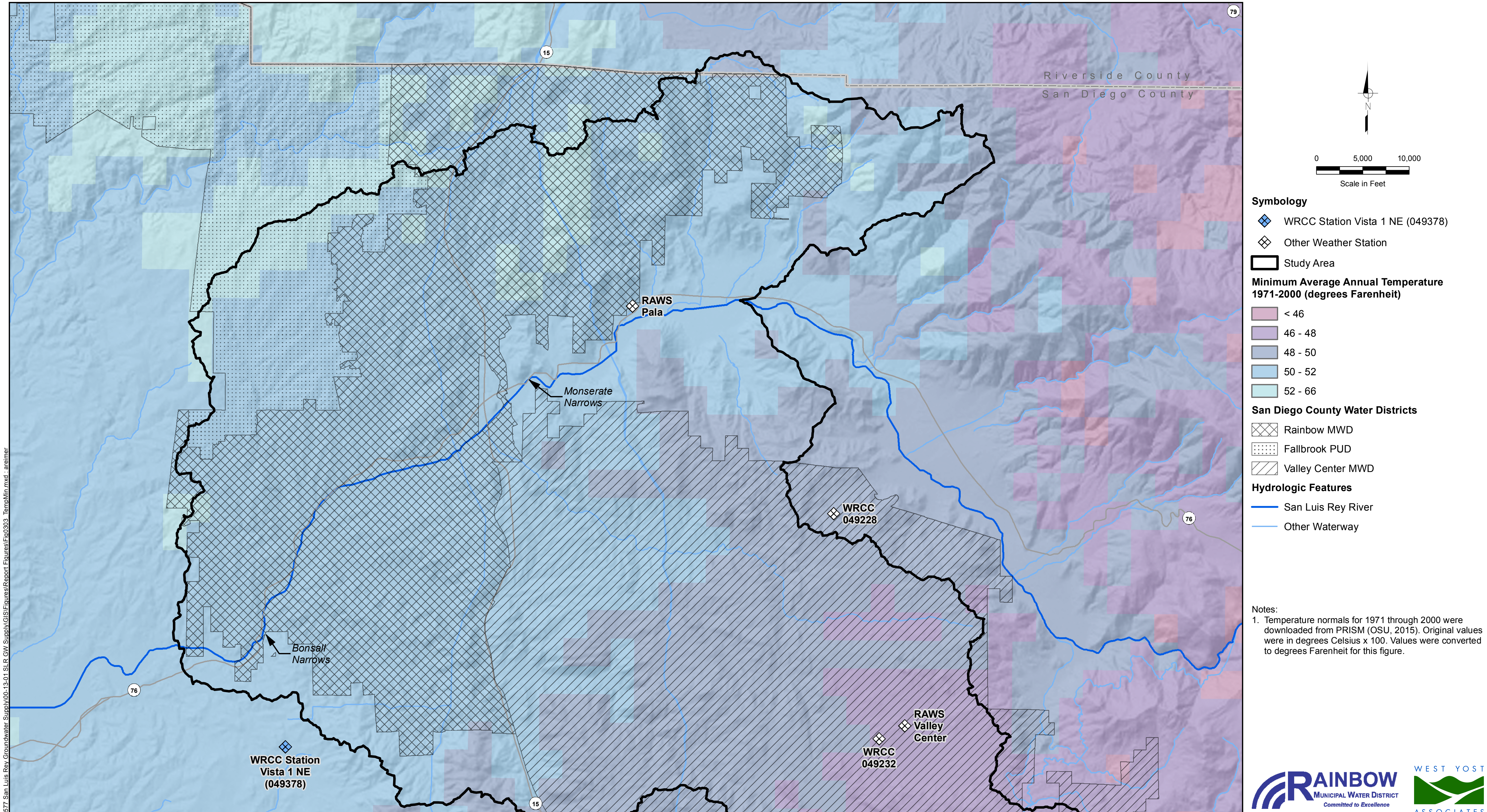

Figure 3-3

Average Annual Minimum Temperature 1971 through 2000

Rainbow Municipal Water District
Groundwater Supply Study

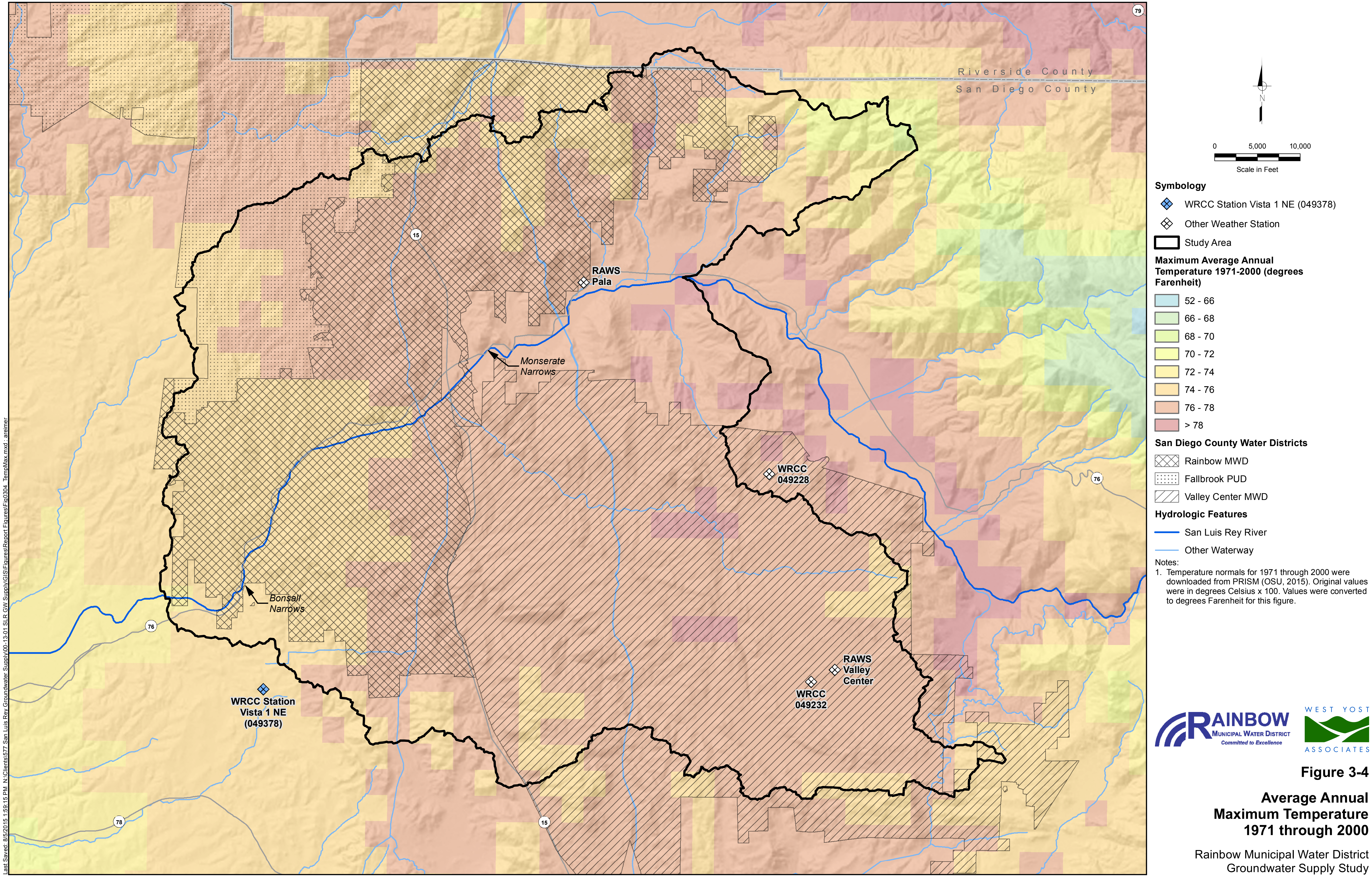

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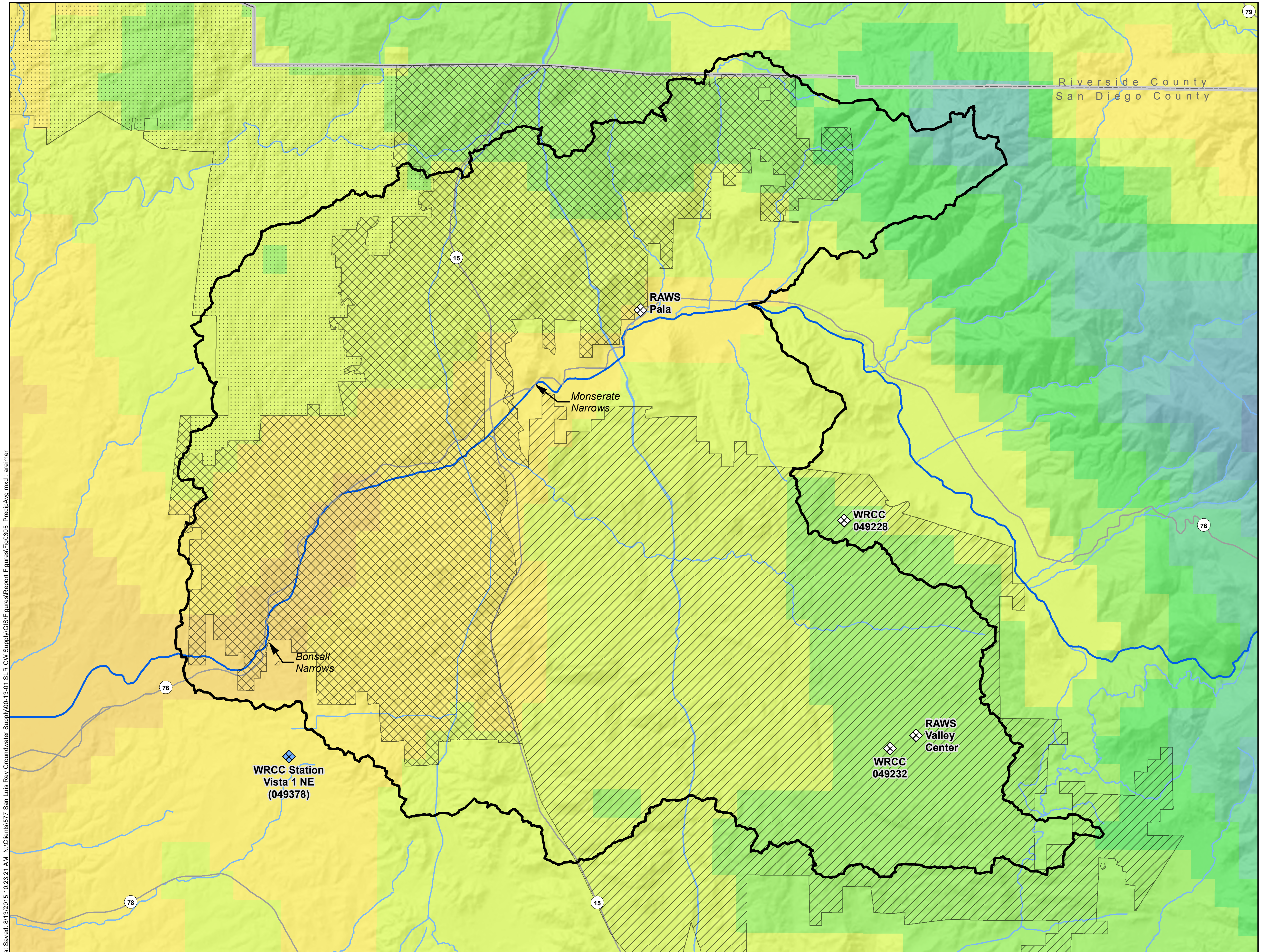
Figure 3-4

Average Annual Maximum Temperature 1971 through 2000

Rainbow Municipal Water District
Groundwater Supply Study



Symbology

- WRCC Station Vista 1 NE (049378)
- Other Weather Station
- Study Area

Average Annual Precipitation 1971-2000 (inches)

- 12 - 14
- 14 - 16
- 16 - 18
- 18 - 20
- 20 - 22
- 22 - 24
- 24 - 26
- 26 - 28
- 28 - 30

San Diego County Water Districts

- Rainbow MWD
- Fallbrook PUD
- Valley Center MWD

Hydrologic Features

- San Luis Rey River
- Other Waterway

Notes:
1. Precipitation normals for 1971 through 2000 were downloaded from PRISM (OSU, 2015). Original values were in millimeters x 100. Values were converted to inches for this figure.

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Figure 3-5

Average Annual Precipitation 1971 through 2000

Rainbow Municipal Water District
Groundwater Supply Study

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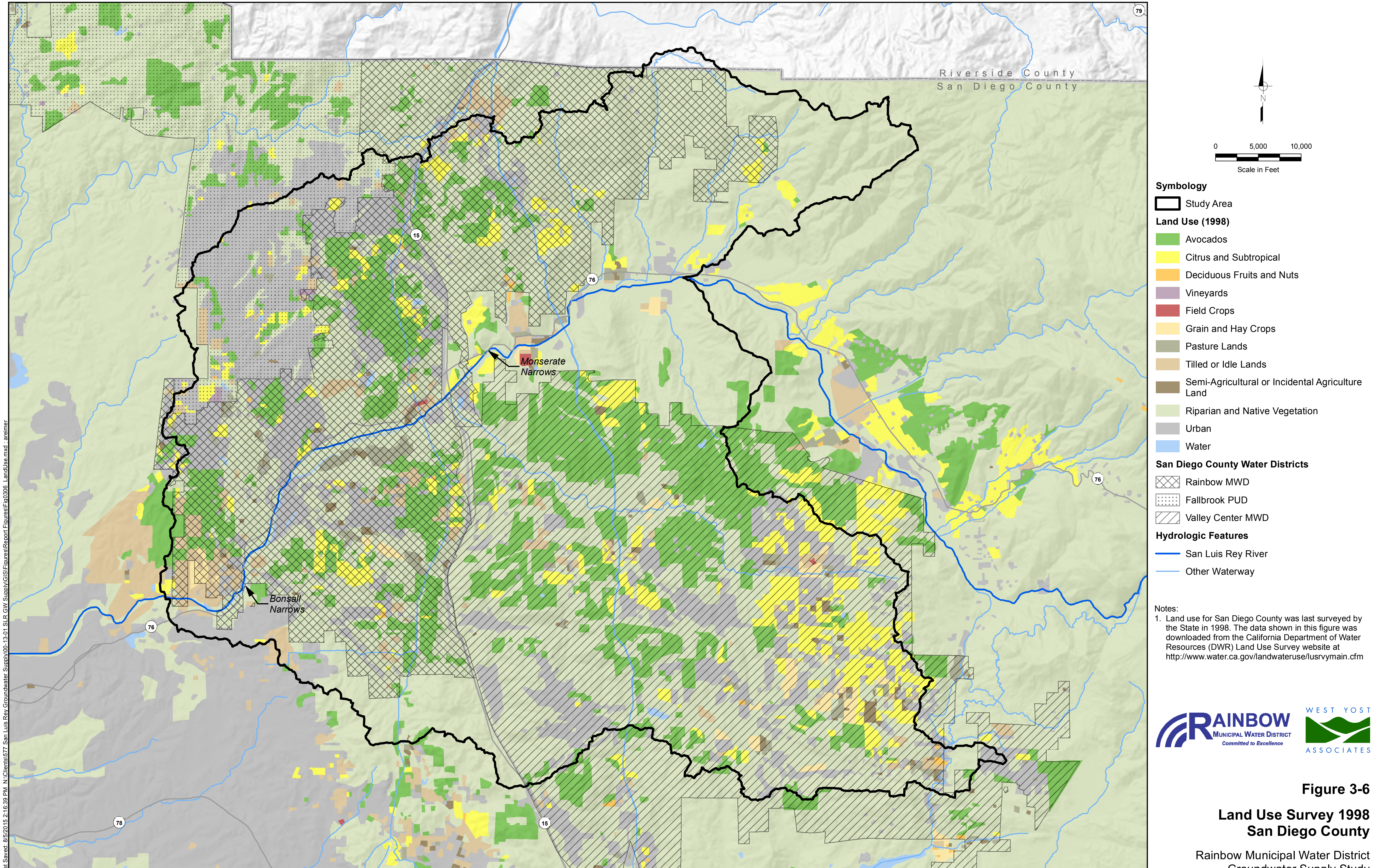

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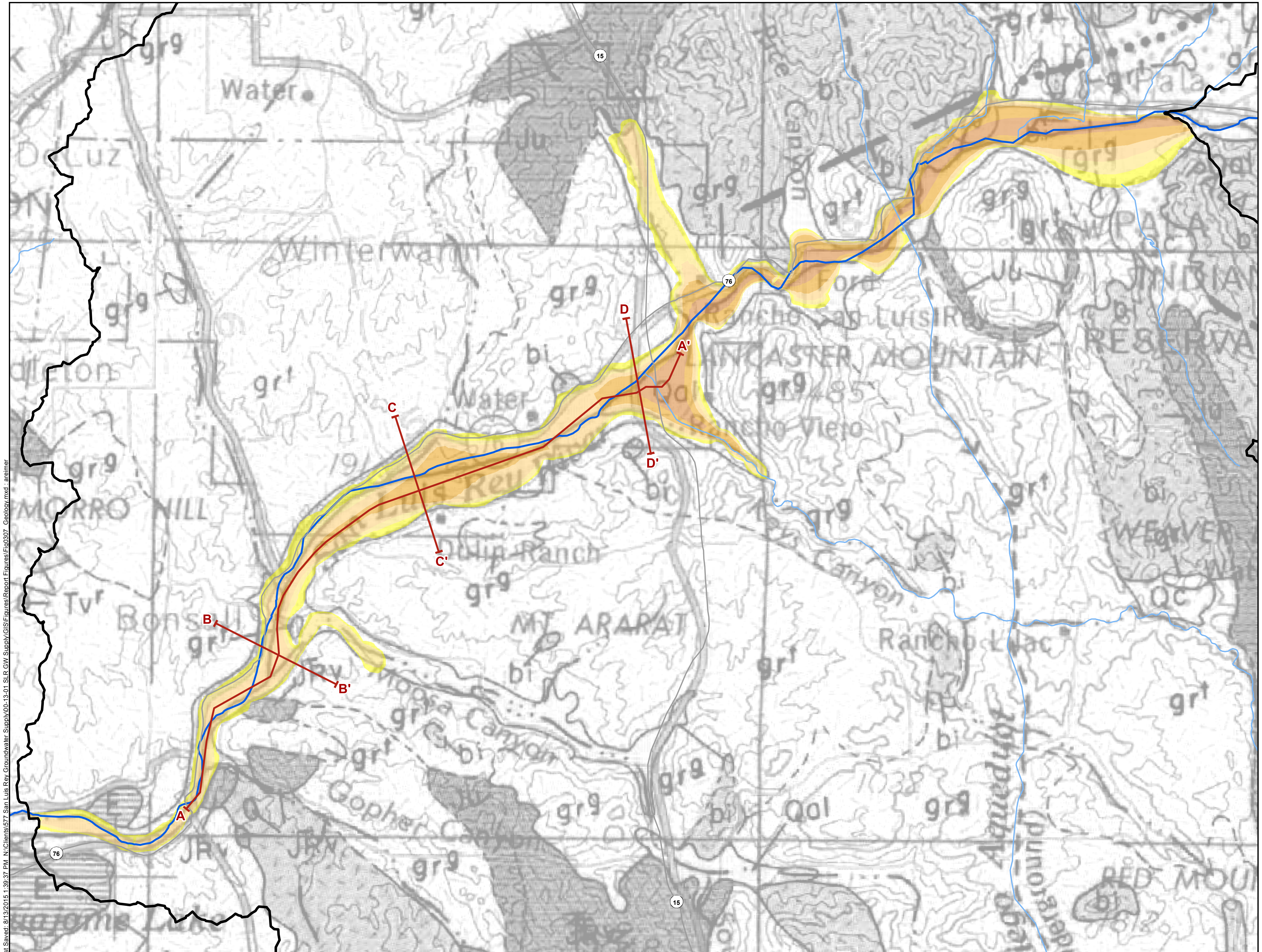
Figure 3-6

Land Use Survey 1998
San Diego County

Rainbow Municipal Water District
Groundwater Supply Study



0 2,500 5,000
Scale in Feet

Symbology

Cross Section

Study Area

Alluvial Thickness (feet)

- 1 - 20
- 20 - 40
- 40 - 60
- 60 - 80
- 80 - 100

Geologic Units

- Qal Holocene Alluvium
- Qc Pleistocene Non-Marine Sedimentary Deposits
- E Eocene Marine Sedimentary Rocks
- Tv^r Tertiary Volcanics, Rhyolytic
- gr^g Mesozoic Granitic Rocks, Granodiorite
- gr^t Mesozoic Granitic Rocks, Tonalite and Diorite
- bi Mesozoic Basic Intrusive Rocks
- JTRv Jurassic-Triassic Metavolcanic Rocks
- Ju Upper Jurassic Marine Sedimentary and Metasedimentary Rocks

Hydrologic Features

San Luis Rey River

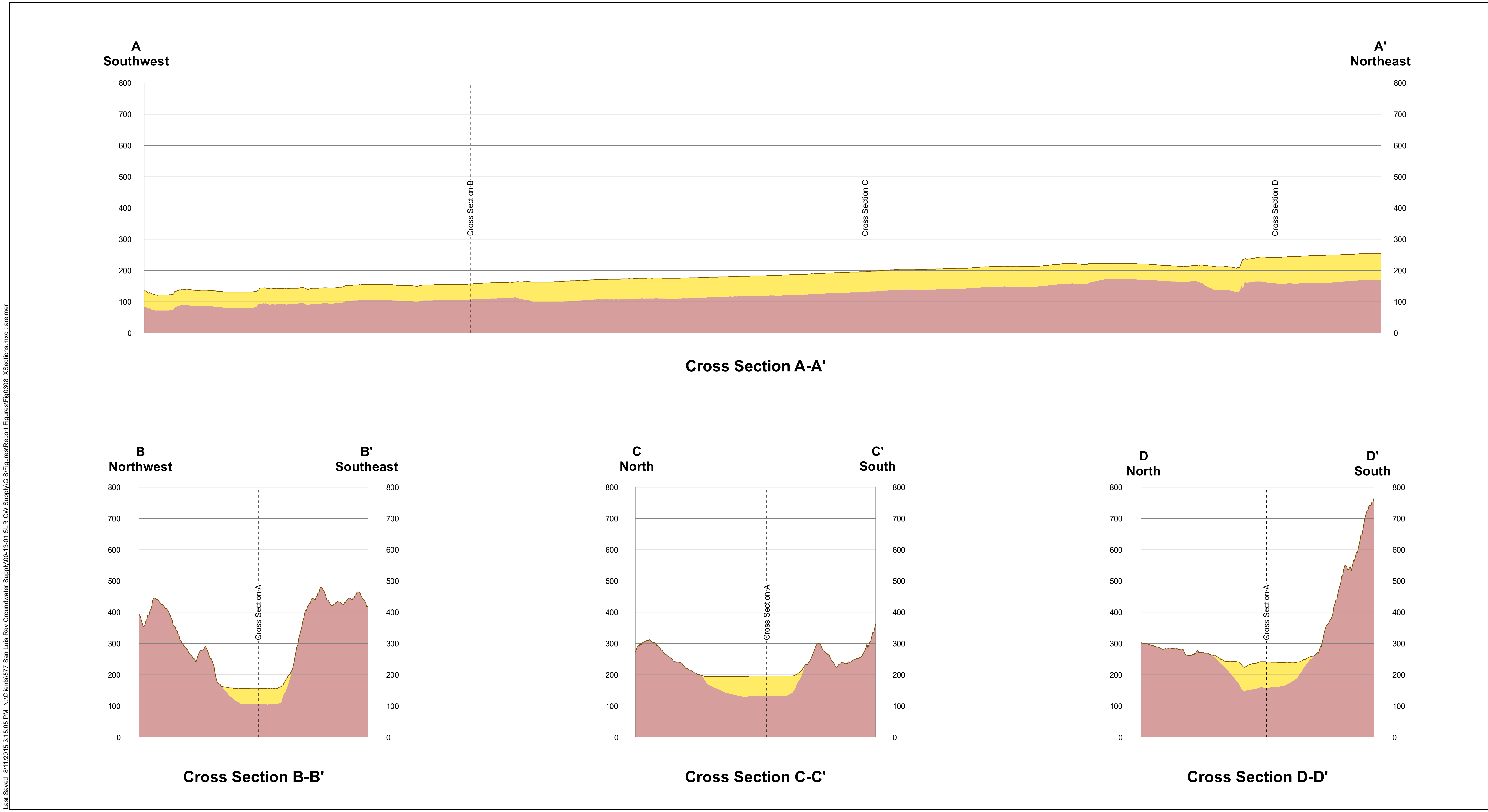
Other Waterway

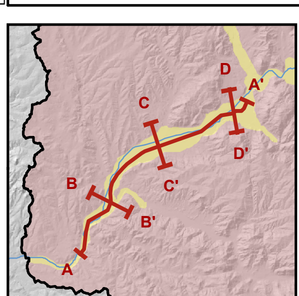
Notes:
1. Rogers, T.H., 1965, Geologic Map of California: Santa Ana Sheet, California Division of Mines and Geology GAM 019, second printing 1973, scale 1:250,000.

Figure 3-7

Geologic Mapping

Rainbow Municipal Water District
Groundwater Supply Study





Symbology

Land Surface Elevation

Intersection

Modeled Geologic Units

Alluvium

Bedrock

0 1,500 3,000

Scale in Feet

Vertical Exaggeration = 10x

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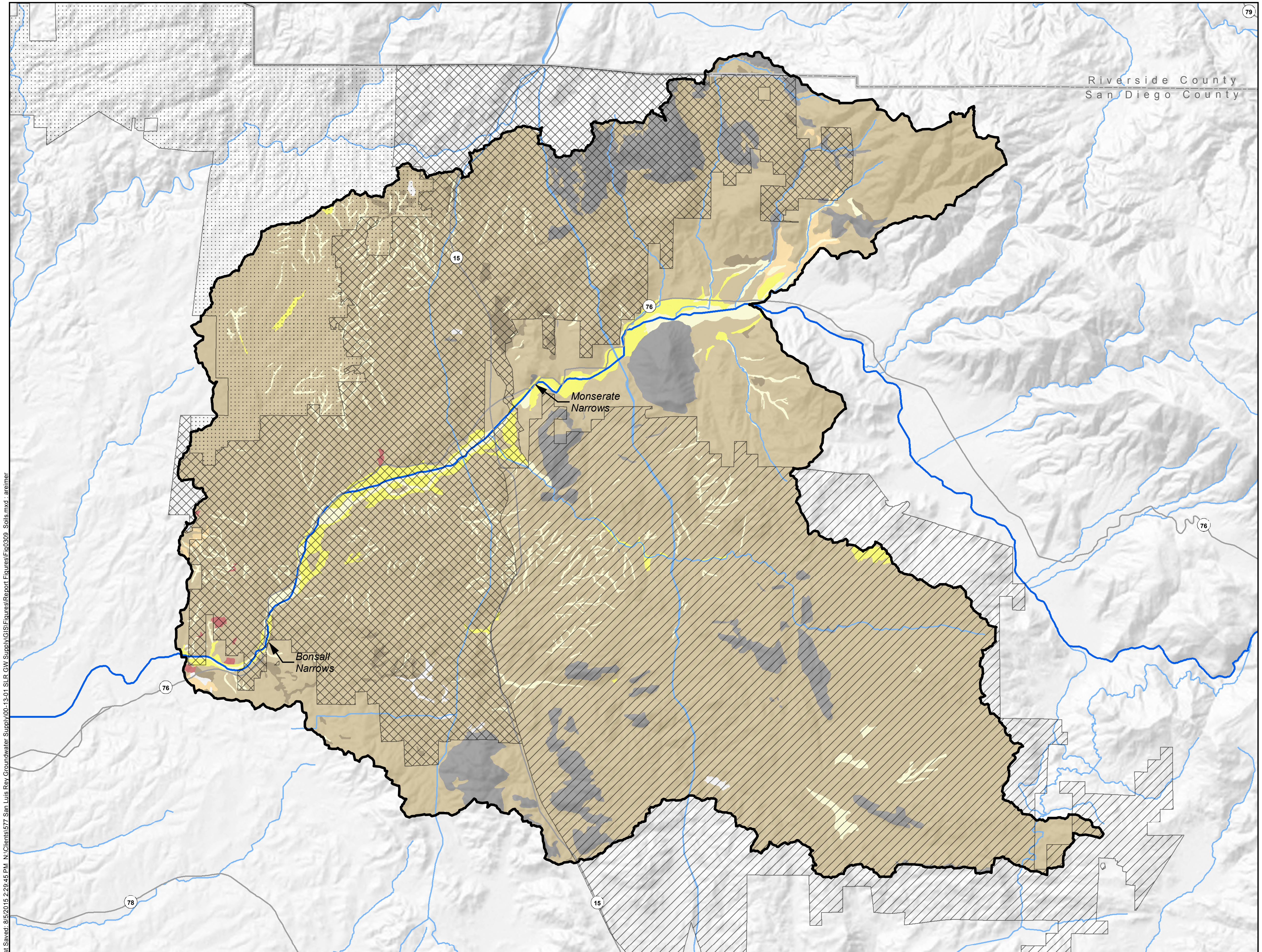
Figure 3-8

Geologic Cross Sections

Rainbow Municipal Water District
Groundwater Supply Study



N

0 5,000 10,000

Scale in Feet

Symbology

Study Area

Soils

Riverwash

Sand

Loamy Sand

Sandy Loam

Loam

Clay

Rock or Terrace

San Diego County Water Districts

Rainbow MWD

Fallbrook PUD

Valley Center MWD

Hydrologic Features

San Luis Rey River

Other Waterway

Notes:

1. Soils are from the National Resources Conservation Service (NRCS) Web Soil Survey (WSS) Soil Survey Geographic (SSURGO) database for San Diego County from http://websoilsurvey.sc.egov.usda.gov

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Figure 3-9

Soils

Rainbow Municipal Water District
Groundwater Supply Study

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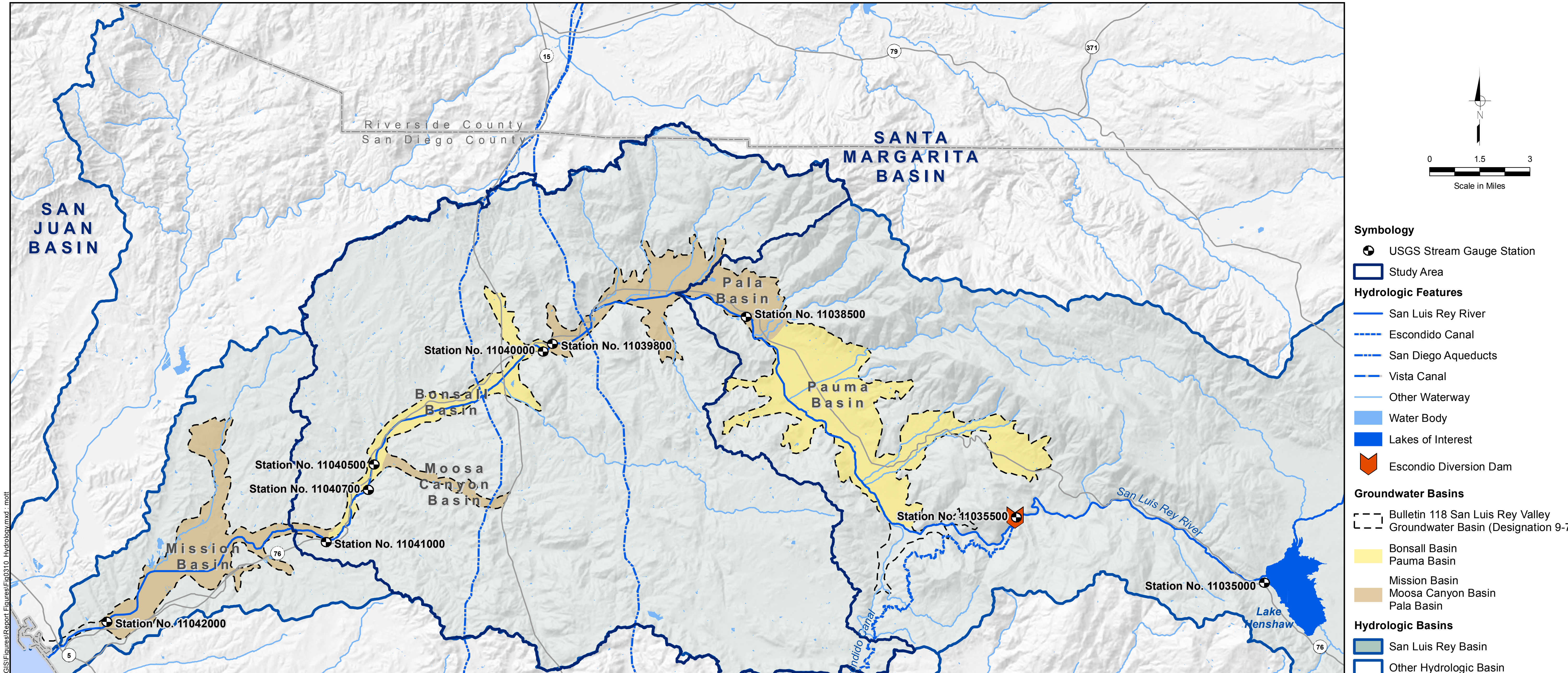


Figure 3-10

Surface and Groundwater Hydrology

Rainbow Municipal Water District
Groundwater Supply Study

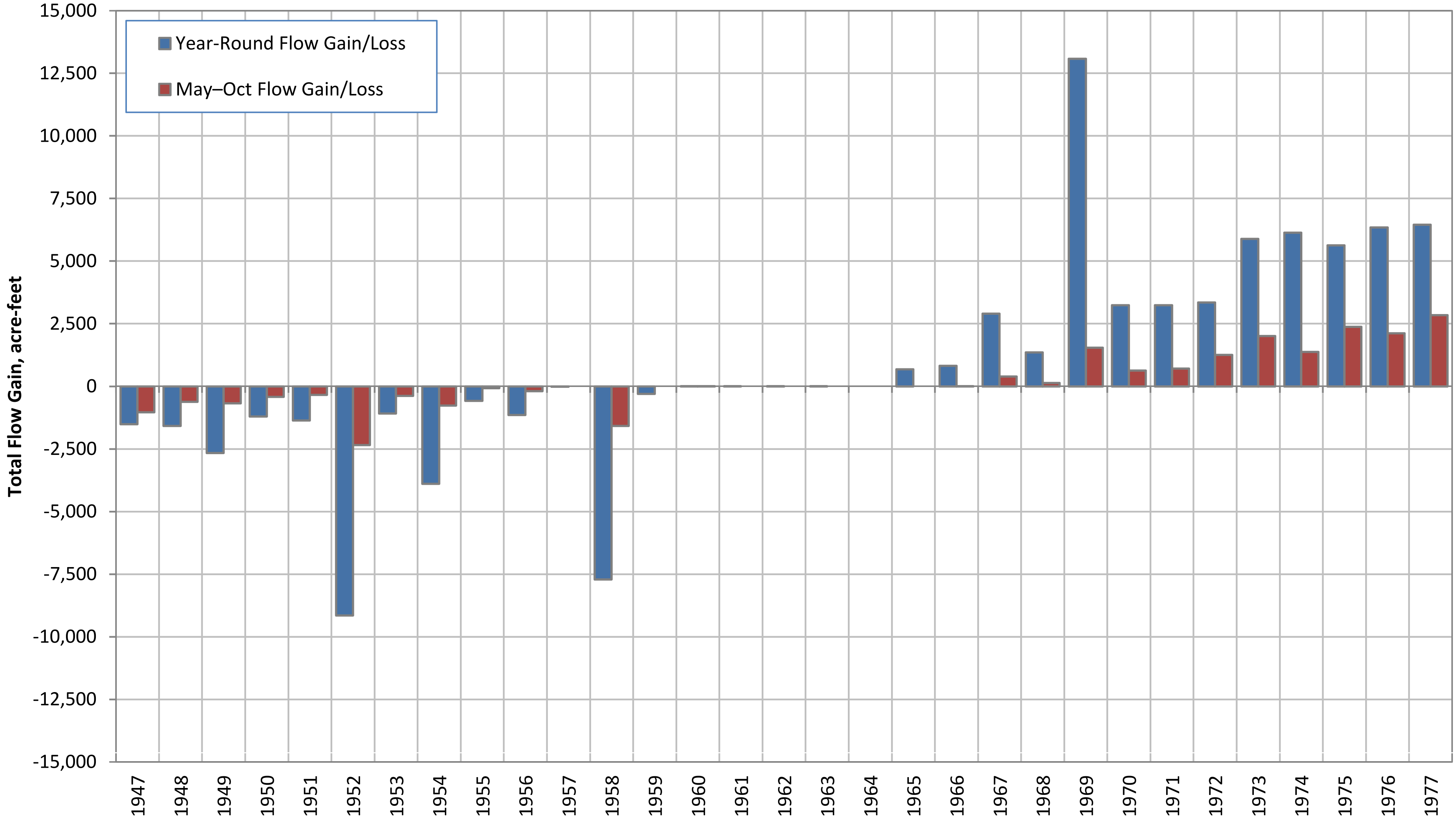


Figure 3-11

San Luis Rey River Gains/Losses Upstream to Downstream, 1947-1977

Rainbow Municipal Water District
Groundwater Supply Study

CHAPTER 4
Groundwater-Stream Flow Model Development

4.1 OVERVIEW OF NUMERICAL MODEL

A numerical model was developed to estimate the recoverable imported water within the study area. The groundwater model was developed using the computer program FEMFLOW3D, Version 3.1, which is a program for simulating three-dimensional groundwater systems and stream flow using the finite element method (Durbin, 2010). FEMFLOW3D, Version 3.1 is an updated version of the USGS computer program FEMFLOW3D, Version 1.0 (Durbin and Bond, 1998). FEMFLOW3D, Version 3.1 includes a module for vertically expanding or contracting the model grid to follow a fluctuating groundwater table (Durbin and Berenbrock, 1985). FEMFLOW3D includes equation solvers and a time step iteration scheme that are efficient when modeling complex large-scale hydrologic settings. These mathematical details are documented in the FEMFLOW3D user's manual (Durbin, 2010).

Aquifers and other three-dimensional features of a groundwater system are represented in FEMFLOW3D with wedge, pyramid, or tetrahedral elements. The basic building block is a six-node wedge-shaped element, although the internal model calculations are performed using tetrahedral elements. Internally within FEMFLOW3D, wedge and pyramid elements are separated into corresponding tetrahedral elements. A wedge-shaped element is a five-sided prismatic solid that has triangular ends connected by three rectangular sides. The wedge-shaped elements are oriented spatially, wherein the triangular faces are horizontal and the rectangular faces are vertical. Wedge elements with one or two zero-height sub-vertical edges can be used to represent particular hydrogeologic units that pinchout, or to make geographic changes in the vertical discretization of the grid. A wedge-shaped element with one zero-height edge reduces to a five-node pyramid, and a wedge-shaped element with two zero-height edges reduces to a tetrahedron.

4.2 MODEL EXTENT

Figure 4-1 shows the model mesh and boundaries. The model boundaries coincide with the study area boundary. The model covers the entire 123,000-acre or 192-square-mile study area with a network of 16,414 elements and 14,921 nodes. Most of the model area is underlain by fractured bedrock terrain. The model is configured to simulate flow in this bedrock terrain and in the alluvial aquifer of the San Luis Rey Valley. Correspondingly, the model mesh has two layers. One layer represents the fractured bedrock, and the other layer represents the alluvial aquifer. The fractured bedrock layer occurs everywhere within the model boundaries. The alluvial aquifer layer occurs along the San Luis Rey River, where it overlies the fractured bedrock layer.

The extent and thickness of the alluvial aquifer were estimated from published reports (USGS, 1971 and 1983), and the DWR Water Well Completion Reports obtained as part of the project. The extent and thickness of the alluvial aquifer is shown on Figure 3-7 and 3-8. The maximum thickness of the alluvial aquifer ranges from 80 to 100 feet in the study area.

4.3 REPRESENTATION OF STREAMS

To simulate river-aquifer interactions, river flow is routed through a main channel and its tributaries. The exchange of water between the local river-channel reach and the groundwater system depends on the wetted width of the reach, flow depth, river-bed elevation, river-bed thickness, and river-bed hydraulic conductivity. Additionally, when water seeps from the river reach into the groundwater system, the simulation allows for a break in the hydraulic connection between the river and the groundwater system. At the break, the rate of groundwater recharge from the river is essentially independent of the depth to the groundwater table. This condition is assumed in FEMFLOW3D to occur when the hydraulic gradient from the river to the water table becomes vertical and the rate of flow between the river and groundwater system is constant and independent of the depth to groundwater. FEMFLOW3D simulates the break based on the groundwater head beneath the river channel.

The San Luis Rey River is represented by 212 stream nodes. The 17 simulated tributaries are represented by 493 nodes for a total of 705 stream nodes. The modeled streams are shown on Figure 4-1.

4.4 BOUNDARY CONDITIONS

The model domain is bounded by no-flow boundaries in bedrock areas; a constant-head boundary on the western alluvial boundary; and a specified-flux boundary on the eastern alluvial boundary. The constant-head boundary at the western modeled extent of the alluvial aquifer represents the westward subsurface outflow from that aquifer. This boundary condition was selected to allow the model to simulate changes in outflow due to pumping and recharge. The specified-flux boundary at the eastern extent of the alluvial aquifer represents the westward subsurface inflow to that aquifer.

4.5 INITIAL HYDRAULIC PROPERTIES

The hydraulic properties of the bedrock and alluvial aquifers were also estimated from published reports (USGS, 1974, 1994, and 1983), and Water Well Completion Reports with specific capacity data. The Theis equation (Theis, 1935) was used to estimate hydraulic conductivity from measured specific capacity. The hydraulic properties assigned to the alluvial aquifer are the same as described by the USGS (Moreland, 1974).

4.6 HYDROLOGIC INPUTS

Figure 4-2 illustrates the conceptual water budget, which was developed around a control volume consisting of the San Luis Rey River and the alluvial aquifer. The water budget was used to conceptualize the inflows and outflows from the study area, which were then quantified as input to the numerical model or generated as output from the numerical model. Inflows consist of stream inflow, subsurface groundwater underflow in the alluvial aquifer, and recharge from imported water return flow, and precipitation. Water leaves the system as stream outflow, subsurface groundwater underflow in the alluvial aquifer, and consumptive use by groundwater pumping and riparian vegetation (evapotranspiration).

The following discussion describes the water budget components that were developed as inputs to the numerical model or produced as model output. These hydrologic inputs were developed as specified fluxes or boundary conditions.

4.6.1 Precipitation Recharge

Groundwater recharge from precipitation was evaluated using the chloride mass balance method and adjusted during model calibration. Groundwater recharge originating from precipitation can be estimated by comparing the chloride concentration in precipitation and in groundwater. Since evapotranspiration and evaporation do not remove chloride mass from soil water, the chloride mass loading to the watershed surface due to precipitation equals the chloride mass loading to the groundwater system. Based on this enrichment, the recharge can be calculated from the relationship (Dettinger, 1989):

$$R = P*(C_p/C_{gw})$$

where:

- R is the average annual recharge
- P is the average annual precipitation
- C_p is the precipitation chloride concentration
- C_{gw} is the groundwater chloride concentration

Figure 4-3 shows the location of precipitation chloride data stations. Seasonal precipitation-weighted mean chloride concentrations were obtained for stations included in the National Atmospheric Deposition Program (NADP) (NADP, 2014). Additional results were obtained from a study by Rebecca Winans and David Huntley entitled, "Using Chloride Mass Balance to Assess Recharge in the Fractured Rock Aquifers of East San Diego County (Winans and Huntley, 2004)."

Figure 4-4 shows the bedrock wells used for analysis of groundwater chloride and the alluvial wells with chloride data within the study area. The groundwater chloride concentration was characterized by the average groundwater chloride concentrations developed from groundwater quality data obtained from the DWR Water Data Library (WDL) (DWR, 2015). Chloride concentration in wells located in bedrock terrain within the study area were considered representative of the effects of recharge, because bedrock wells are more likely to be located in areas receiving recharge from precipitation.

4.6.1.1 Analysis

Average annual precipitation was calculated over the period 1971-2000. This analysis was completed by using the average PRISM precipitation grid across the entire study area for each year, and then averaged. The normal mean from 1971-2000 was approximately 17.5 inches per year.

Figure 4-5 shows the precipitation chloride concentration results for the eight stations shown on Figure 4-3. In several cases, the available chloride data for a given precipitation station were very limited. Some of the intrastation variation may be attributable to redeposition of chloride during the dry season. Interstation differences are a function of station location and elevation.

The range of average annual precipitation recharge was estimated based on the NADP site nearest the study area (NADP CA68) and the separate Winans and Huntley data (Figure 4-3). The minimum of the range was estimated based on the median value of 0.61 mg/L for site NADP CA68. The maximum of the range was estimated based on the median value of 2.45 mg/L from the Winans and Huntley data.

Figure 4-6 shows the groundwater chloride concentration results for the bedrock wells. The median value of 119.5 mg/L for the period 1952 through 1961 was selected because this period predates extensive use of imported water in the study area. Chloride concentrations in some bedrock wells increased significantly after 1961. Presumably chloride concentrations in bedrock wells increased with increasing use of imported water after about 1961 because the imported water has higher chloride concentrations than precipitation.

4.6.1.2 Chloride Mass Balance Results

Table 4-1 provides estimates of the precipitation recharge in the study area using the chloride mass balance method and the parameters listed above.

Table 4-1. Estimated Range of Average Annual Precipitation Recharge based on the Chloride Mass Balance Method

PRISM Normal Precipitation 1971-2000, inches per year	17.5	
	Minimum[a]	Maximum[b]
Median C_p, mg/L	0.61	2.45
C_{gw}, mg/L	119.5	119.5
Estimated Average Annual Precipitation Recharge, inches per year	0.09	0.36
Estimated Average Annual Precipitation Recharge, afy	923	3,690

(a) Minimum C_p from NADP CA68.
(b) Maximum C_p Winans and Huntley (2004).
afy = acre-feet per year

Based on these results, and using Equation 1, average annual precipitation recharge was estimated to range between 0.09 and 0.36 inches per year. This is equivalent to average annual recharge ranging from approximately 923 afy to 3,690 afy over the entire 123,000-acre study area.

The precipitation recharge estimates were further developed using a power function relating the average annual precipitation recharge to average annual precipitation, where the average annual precipitation is derived from PRISM GIS coverage of the study area for 1971-2000. The power function was developed as part of the Winans and Huntley (2004) study (Timothy Durbin, 2014, written communication). The function has the form:

$$R_P = a*(P_0 - P)^b$$

where:

R_P	is the average annual recharge at a point
P	is the average annual precipitation at the point
P_0	is a threshold precipitation
a	is a coefficient
b	is the exponent

With precipitation and recharge in the unit of inches per year, the coefficient equals 0.019, the threshold precipitation equals 10 inches, and the exponent equals 1.12. The coefficient was derived from the model calibration. Based on the calibration, the average annual precipitation recharge was estimated to be approximately 0.19 inches per year or approximately 2,000 afy over the entire study area. These values fall within the range estimated using the chloride mass balance method.

4.6.2 Imported Water Return Flows

RMWD, FPUD, and VCMWD purchase and deliver imported water to their customers from MWD through SDCWA. Figure 4-7 shows the historical annual imported water deliveries for each agency. These imported water return flows co-mingle in the study area portion of the San Luis Rey Valley Groundwater Basin. Imported water purchases have generally increased over time, until the economic downturn in 2008 and on-going recent drought conditions (2011-2015), resulting in increases in imported water return flows.

The recharge of imported water supplied to agricultural uses was estimated from the delivery of imported water to a purveyor's service area scaled as expressed in the relation:

$$R_I = D_{SA}*f_{AP}*_{PM}*f_R$$

where:

R_I	is the annual recharge of imported water
D_{SA}	is the annual imported water delivered to the purveyor's service area
f_{AP}	is the fraction of the imported water used for agricultural production within the purveyor's service area
f_{PM}	is the fraction of the purveyor's service area within the model extent
f_R	is the fraction of applied water that becomes groundwater recharge

Based on analyses by the Mission Resource Conservation District (MRCD), the fraction of agricultural water use that becomes groundwater recharge is 24 percent. The equation above was applied separately to the three water purveyors within the model extent: RMWD, FPUD, and VCMWD.

The recharge of imported water supplied to residential and commercial uses was assumed to be essentially zero. This follows from assumptions that a substantial part of the water use is exported from the model extent as wastewater, landscape irrigation is limited, and the landscape irrigation is efficient.

The basis for the estimated imported water return flows is discussed in more detail in the following sections.

4.6.2.1 Apportionment of Imported Water to the Study Area

The study area includes only parts of RMWD, VCMWD, and FPUD, and only a portion of each district's imported water is delivered to the study area. GIS mapping of the imported water distribution piping constructed in each historical year was available for RMWD and FPUD. This mapping was used to estimate the portion of each district's imported water delivered to the study area by year. Allocations of water delivery to the study area for these two districts were completed for each year by multiplying the proportion of constructed water pipeline length in the study area to the total constructed water pipeline length within the district by the reported total water delivery for the year. The effective service area to which this water was applied was estimated for each historical year based on the RMWD and FPUD GIS pipe development mapping. GIS-based pipeline mapping was not available for VCMWD. Therefore, VCMWD's historical deliveries of water to the study area were estimated by multiplying VCMWD's total delivery for each year by the proportion of the district lying within the study area. Using this approach, the percentage of each district's service area increased over time for the model calibration period (water years 1947 through 1977) before reaching the maximum percentages used in the pumping scenario. These maximum percentages were:

- RMWD: 92 percent of service area in study area
- FPUD: 49 percent of service area in study area
- VCMWD: 71 percent of service area in study area

The areas to which imported water was applied for agricultural purpose were further delineated using data from the Farmland Mapping and Monitoring Program (FMMP), which was available on a two-year interval for years beginning in 1984. FMMP produces maps and statistical data of California's agricultural resources based on soil quality and irrigation status (California Department of Conservation, 2013). FMMP maps were used from 1984 to 2010 to estimate the irrigated areas within the study area for each year.

4.6.2.2 Apportionment of Imported Water to Agricultural and Urban Uses

Imported water deliveries to the study were further apportioned based on the percentage of agricultural and urban use within each district's service area as documented in each district's 2010

UWMP. Table 4-2 lists the reported percentages of agricultural and urban water use for each district.

Table 4-2. Percentage Agricultural and Urban Water Use by District

Year	Water District[a]					
	FPUD		RMWD		VCMWD	
	Ag (%)	Urban, %	Ag, %	Urban, %	Ag, %	Urban, %
1959[b]	80	20	80	20	80	20
2000	NR[c]	NR	NR	NR	83	17
2005	46	54	76	24	NR	NR
2010	42	58	63	37	73	27
2015	32	68	NR	NR	NR	NR
2046[d]	32	68	60	40	70	30

(a) Percentages reported in each district's 2010 UWMP.
(b) Values for 1959 were assumed. Percentages in years prior to 1959 were held fixed at assumed 1959 values. Annual percentages between 1959 and 2000 or 2005 were interpolated from UWMP values.
(c) NR = Not reported
(d) Values for 2046 were assumed.

The UWMP values listed in Table 4-2 for each district were interpolated and extrapolated to estimate annual percentages for the model calibration period (water years 1947 through 1977), and the groundwater pumping scenario (water year 2016 through 2046). For the calibration period, the extrapolation of past agricultural and urban percentages was assumed fixed for 1959 and earlier years at the values listed in Table 4-2.

4.6.2.3 Imported Water Return Flows from Agricultural Uses

Imported water return flows from agricultural uses were estimated based on irrigation efficiency testing conducted by the MRCD. MRCD is a special purpose district established in 1944 to support implementation of technically sound conservation practices by local property owners and growers. MRCD began implementing an irrigation water management program in 1983 and is currently implementing the Agricultural Water Management Program (AWMP) with funding provided by SDCWA and its member agencies.

MRCD conducts on-farm irrigation efficiency testing and documents the results in annual AWMP reports. Irrigation efficiency is termed emission uniformity in the AWMP reports. The emission uniformity indicates water that is available for plants. The remaining percentage is lost to infiltration. During fiscal year 2013-2014, MRCD conducted 101 on-farm emission uniformity tests, which yielded an average irrigation efficiency of 76 percent, indicating that 76 percent of the water used for agriculture is consumed by evapotranspiration (MRCD, 2015). MRCD staff reported that past results were similar and that future results are also expected to be similar (MRCD personal communication, 2015).

Based on these results, imported water return flows from agricultural uses were estimated to be 24 percent of the imported water used for irrigation.

MRCD staff also confirmed that the soil properties are such that there is essentially no runoff from applied irrigation water. Applied water that is not consumed by the crops infiltrates and either moves along the bedrock/soil interface to discharge areas along local tributaries to the San Luis Rey River or enters the bedrock fracture system, and ultimately flows to the San Luis Rey River Groundwater Basin (MRCD personal communication, 2015). The numerical model developed for this project is consistent with this conceptual model of flow. The model simulates infiltration of imported water, groundwater flow through fractured bedrock and the alluvial aquifer, and stream flow in the San Luis Rey River and its tributaries.

4.6.2.4 Imported Water Return Flows from Urban Uses

For the purposes of this study, water imported for outdoor urban uses was assumed to be consumed, and water imported for indoor urban uses was assumed to be exported from the study area after use via the sanitary sewer system.

Sewage from RMWD is conveyed to the City of Oceanside's San Luis Rey Wastewater Treatment Plant and discharged at the City of Oceanside's Oceanside Ocean Outfall (RMWD, 2010).

Most of the FPUD service area within the study area is served by a public sewer system. Wastewater is treated at FPUD Wastewater Treatment Plant No. 1 with most of its treated effluent being disposed to the ocean using the City of Oceanside Ocean Outfall. A minor amount of tertiary effluent is sold as recycled water, which is then used for agricultural purposes and landscape irrigation (FPUD, 2010).

Within VCMWD, wastewater collection, treatment, and effluent disposal or recycling are provided through the Lower Moosa Canyon Water Reclamation Facility and the Woods Valley Ranch Water Reclamation Facility (VCMWD, 2010).

4.6.2.5 Annual Imported Water Return Flows to the Study Area

Table 4-3 lists the estimated imported water return flows to the study area for the model calibration period (water years 1947 through 1977). Table 4-4 lists the estimated imported water return flows to the study area for the future scenarios (water years 2016 through 2046).

4.6.3 Stream Flow

The study area tributary watershed acreages and the spatial and temporal distribution of precipitation from PRISM were used to simulate runoff and tributary streams flows to the San Luis Rey River during model calibration and simulation of project groundwater pumping.

Table 4-3. Imported Water Return Flows for Model Calibration

Year	Flows by District, acre feet		
	VCMWD	FPUD	RMWD
1947	-	-	-
1948	-	136	-
1949	-	106	-
1950	-	237	-
1951	-	202	-
1952	-	125	-
1953	-	266	-
1954	-	408	324
1955	0.1	445	579
1956	81	487	726
1957	119	368	994
1958	119	472	835
1959	178	715	1,449
1960	287	526	1,404
1961	330	742	1,829
1962	408	601	1,715
1963	445	728	2,087
1964	501	786	2,179
1965	728	730	2,431
1966	1,024	914	2,731
1967	1,159	802	2,545
1968	1,454	1,037	2,981
1969	1,698	675	2,881
1970	2,032	917	3,686
1971	2,229	923	3,617
1972	2,619	1,089	4,410
1973	2,469	897	3,739
1974	3,209	1,753	4,325
1975	3,650	921	4,144
1976	4,154	1,088	5,092
1977	4,319	1,001	5,120
Average	1,071	648	1,994

Table 4-4. Imported Water Return Flows for Future Scenarios			
	Flows by District, acre feet		
Year	VCMWD	FPUD	RMWD
2016	4,058	575	2,562
2017	4,058	575	2,562
2018	4,058	575	2,562
2019	4,058	575	2,562
2020	4,045	612	2,514
2021	4,045	612	2,514
2022	4,045	612	2,514
2023	4,045	612	2,514
2024	4,045	612	2,514
2025	4,303	665	2,669
2026	4,303	665	2,669
2027	4,303	665	2,669
2028	4,303	665	2,669
2029	4,303	665	2,669
2030	4,303	665	2,669
2031	4,303	665	2,669
2032	4,303	665	2,669
2033	4,303	665	2,669
2034	4,303	665	2,669
2035	4,303	665	2,669
2036	4,303	665	2,669
2037	4,303	665	2,669
2038	4,303	665	2,669
2039	4,303	665	2,669
2040	4,303	665	2,669
2041	4,303	665	2,669
2042	4,303	665	2,669
2043	4,303	665	2,669
2044	4,303	665	2,669
2045	4,303	665	2,669
2046	4,303	665	2,669
Average	4,263	656	2,645

USGS stream gauge data were obtained and analyzed for the gauge locations shown on Figure 3-10. Figure 3-11 is a graph of net gains and losses to the river as calculated using stream flow data from gauges 11040000 and 1104100 for water years 1947-1977. The San Luis Rey River was a losing stream in the early years of imported water delivery (1947 to 1959), when groundwater was still a significant part of the water supply. During the period from 1960 to 1964 the San Luis Rey River sustained almost no base flow within the study area, because of groundwater depletion by pumping. However, by 1965 and extending through the period of record, the river gained flow, likely in response to groundwater recharge from imported water return flows, which resulted in higher groundwater levels capable of supporting baseflow in the river.

The streamflow inputs to the model include streamflows for the San Luis Rey River at the eastern boundary of the model extent. They additionally include runoff into tributaries of the San Luis Rey River within the model extent. The boundary streamflow is based on the measured streamflow for the San Luis Rey River at Monserate Narrows (USGS station 11040000), which is located approximately four miles downstream from the eastern boundary of the model. To account for the tributary inflows between Monserate Narrows and the eastern boundary, the boundary streamflows were adjusted during model calibration to be specified as 80 percent of the measured streamflows.

The runoff to tributaries was based on the measured streamflow for Santa Maria Creek near Ramona, as measured at USGS gauge 11028500 (Figure 3-10). Gauge 11028500 was selected because it had a period of record overlapping that of the calibration period, and the gauged tributary receives runoff from terrain with similar elevations as the watersheds of tributary streams in the study area. For a particular tributary watershed within the model extent, the tributary runoff is the measured streamflow scaled for differences in watershed area, average annual precipitation, and land-surface factors. The scaling for differences in land-surface factors was derived from the model calibration. The scaling for differences in precipitation was derived from a PRISM map of average annual precipitation for 1971-2000.

4.6.4 Groundwater Underflow

Groundwater underflow through the San Luis Rey Valley Groundwater Basin was estimated at the upstream and downstream boundaries of the study area using Darcy's Law (Figure 4-1). The cross-sectional areas of the alluvial aquifer at the upstream and downstream boundaries were estimated based on the thickness of aquifer material documented in Water Well Completion Reports obtained from the DWR Southern Regional Office and basin geometry studies completed by the USGS (Izibicki, 1985). Depth to groundwater information needed to calculate aquifer saturated thickness and flow gradient was obtained from DWR and USGS groundwater elevation measurements at nearby wells. Aquifer hydraulic conductivity was based on reports (USGS, 1974, 1994, and 1983), and specific capacity measurements recorded in the DWR Water Well Completion Reports. Depending on the variation in historical groundwater elevations, underflows into the study area ranged from roughly 400-500 afy. For the historical period, the magnitude of underflow was similar at the upstream and downstream boundaries of the study area.

Based on this analysis, the upstream boundary, representing underflow into the study area through the alluvial aquifer, was defined as a specified flux boundary with a flux of 490 afy. The downstream boundary, representing underflow out of the study area through the alluvial aquifer, was established as a constant head boundary, and the outflows were simulated by the model.

4.6.5 Groundwater Pumping

Historical groundwater pumping was derived from the estimates of the USGS (Moreland, 1974). The USGS estimated annual pumping for 1958 and 1967 and the corresponding groundwater recharge from irrigation. The difference between the pumping and recharge is the consumptive use, which represent the net pumping. The USGS (1974) additionally describes the locations of production wells, but not the pumping from individual wells. The pumping data were used to estimate the annual net pumping for the calibration period (water years 1947-1977). The well-location information was used to geographically distribute the annual pumping based on the assumption of equal pumping from the individual wells. Figure 4-8 shows the estimated net pumping used for model calibration for the water years 1947-1977.

The future gross pumping in the study area was assumed to be approximately 2,500 afy based on the annual pumping estimated in MWD (2007). Applying a 76 percent consumptive use factor derived from Moreland (1974), the net pumping used for future simulations was approximately 1,900 afy.

4.6.6 Consumptive Use by Riparian Vegetation

Groundwater-dependent riparian vegetation is present in the study area. The extent of the riparian vegetation over time was determined through review and analysis of aerial photography mosaic images obtained from the University of California at Santa Barbara Aerial Imagery Research Service, Map & Imagery Laboratory. The WDL (DWR, 2015) was also used to identify wells within the study area that had a reported depth-to-water of 15 feet or less (indicating the possibility of groundwater-dependent riparian vegetation) for years corresponding to the historical aerial photography. Using the mosaic aerial images in conjunction with the groundwater level data, the areas with riparian vegetation were identified for each of the years with available photography.

Figure 4-9 shows the estimated extents of riparian vegetation for the years 1953, 1963, 1976, and 1980. The extent of riparian vegetation was interpolated from the years with available aerial photography for the water year 1947 through 1977 calibration period. For simulation of future conditions, the extent of riparian vegetation was assumed to be the extent mapped in the 1980 aerial photography.

Consumptive use by riparian vegetation was simulated by the model based on modeled groundwater levels and a maximum evapotranspiration rate according to the following relationship:

$$ET = ET_{max} (d - d_0)/d_0$$

where:

ET	is simulated evapotranspiration from riparian vegetation
ET_{max}	is the estimated maximum evapotranspiration from riparian vegetation
d	is the water table depth
d_0	is the extinction depth

This equation assumes that, with the groundwater table very near the land surface, the evapotranspiration from riparian vegetation is maximum. As the depth to the water table increases, the evapotranspiration decreases until it is zero at the extinction depth.

The maximum evapotranspiration was derived using the reference evapotranspiration zone map from the California Irrigation Management Information System (CIMIS). This map provides monthly reference evapotranspiration records based on different zones that make up California. The study area is in Zones 4 and 6 (Figure 4-10). Table 4-5 shows the CIMIS monthly average reference evapotranspiration for Zones 4 and 6 (CIMIS, 1999). Consumptive use increased with time after water imports began, as a result of increased groundwater levels allowing riparian vegetation to expand and consume more water.

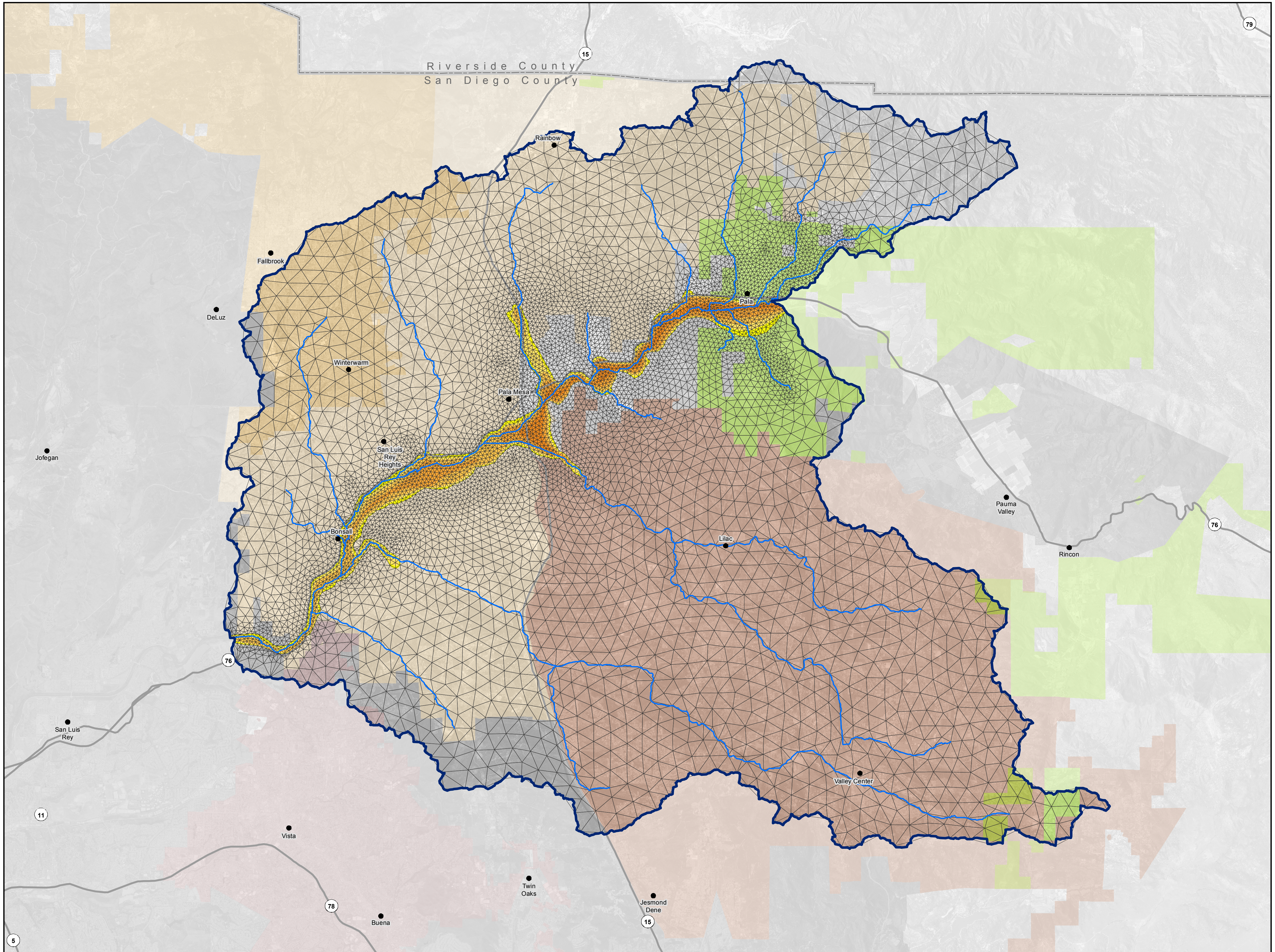
Table 4-5. CIMIS Monthly Average Reference Evapotranspiration for Et_o Zones 4 and 6 (inches/month)

	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
Zone 4	1.86	2.24	3.41	4.5	5.27	5.7	5.89	5.58	4.5	3.41	2.4	1.86	46.62
Zone 6	1.86	2.24	3.41	4.8	5.58	6.3	6.54	6.2	4.8	3.72	2.4	1.86	49.71
Average	1.86	2.24	3.41	4.65	5.43	6	6.22	5.89	4.65	3.57	2.4	1.86	48.17

4.7 MODEL CALIBRATION

The model was calibrated to hydrologic conditions during water years 1947-1977. This period was selected because streamflow measurements were collected for the San Luis Ray River at both the western model extent and just westward from the eastern model boundary. At the western model boundary, streamflow data are available for the San Luis Ray River near Bonsall (USGS station 11041000). These data allow the calibration of the water budget for the hydrologic system. The model calibration involved scaling the streamflow and precipitation recharge inputs to best match the San Luis Rey River streamflow near the western boundary of the model. The adjusted parameters were: (1) the scaling factor for specifying the San Luis River inflow at the eastern model boundary, (2) the scaling factor for the tributary runoff within the model boundary, and (3) the scaling factor for the groundwater recharge from precipitation. Adjustment were made to these to best match the average annual streamflow at the USGS stream gauge for San Luis Rey River near Bonsall (1104100) for water years 1947-1977. The measured average annual streamflow at the gauge is 3,200 afy. The modeled average annual streamflow is 2,900 afy, which is about nine percent lower than the measured streamflow. While aquifer or stream-channel parameters were not adjusted during the calibration, a comparison of the measured groundwater levels and the model storage change indicates a suitable match. The USGS (1974) model has a better fit between measured groundwater levels and the modeled storage change. The difference is that, while the USGS model had available pumping data for each well, such data were not available for the current model.

Based on the calibration results, the model is suitably calibrated and appropriately represents the San Luis Rey Valley stream-aquifer system in the study area for the purposes of conducting the planning-level comparisons of baseline and Proposed Groundwater Project scenarios in the next chapter.



0 4,500 9,000

Scale in Feet

Symbology

Study Area

Model Features

Modeled River and Tributaries

Model Mesh

Modeled Bedrock

San Diego County Water Districts

Rainbow Municipal Water District

Fallbrook Public Utility District

Valley Center Municipal Water District

Vista Irrigation District

Other Water District

Indian Reservation

Modeled Alluvial Thickness (feet)

1 - 20

20 - 40

40 - 60

60 - 80

80 - 100

Other Features

County Boundary

City or Township

Major Roads

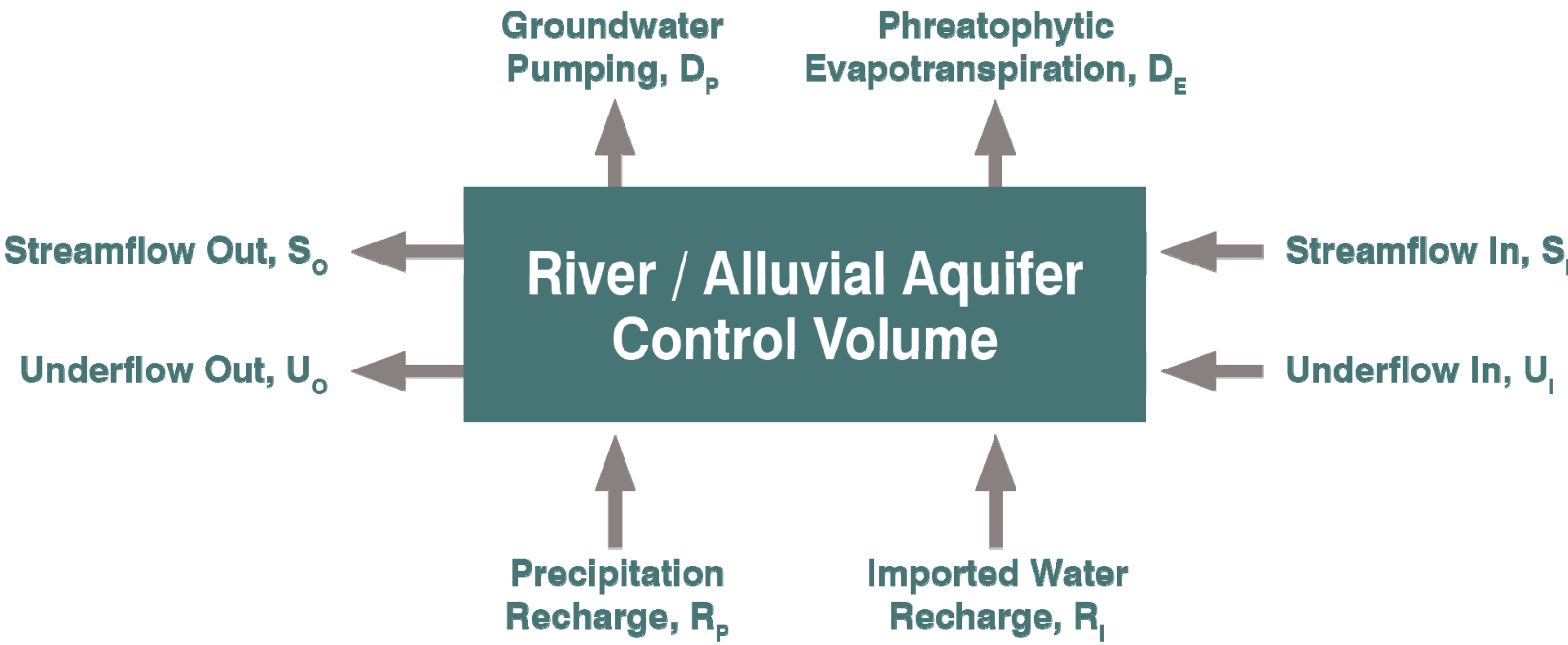
RAINBOW MUNICIPAL WATER DISTRICT Committed to Excellence

WEST YOST ASSOCIATES

Figure 4-1

Model Mesh and Boundaries

Rainbow Municipal Water District
San Luis Rey Groundwater Supply



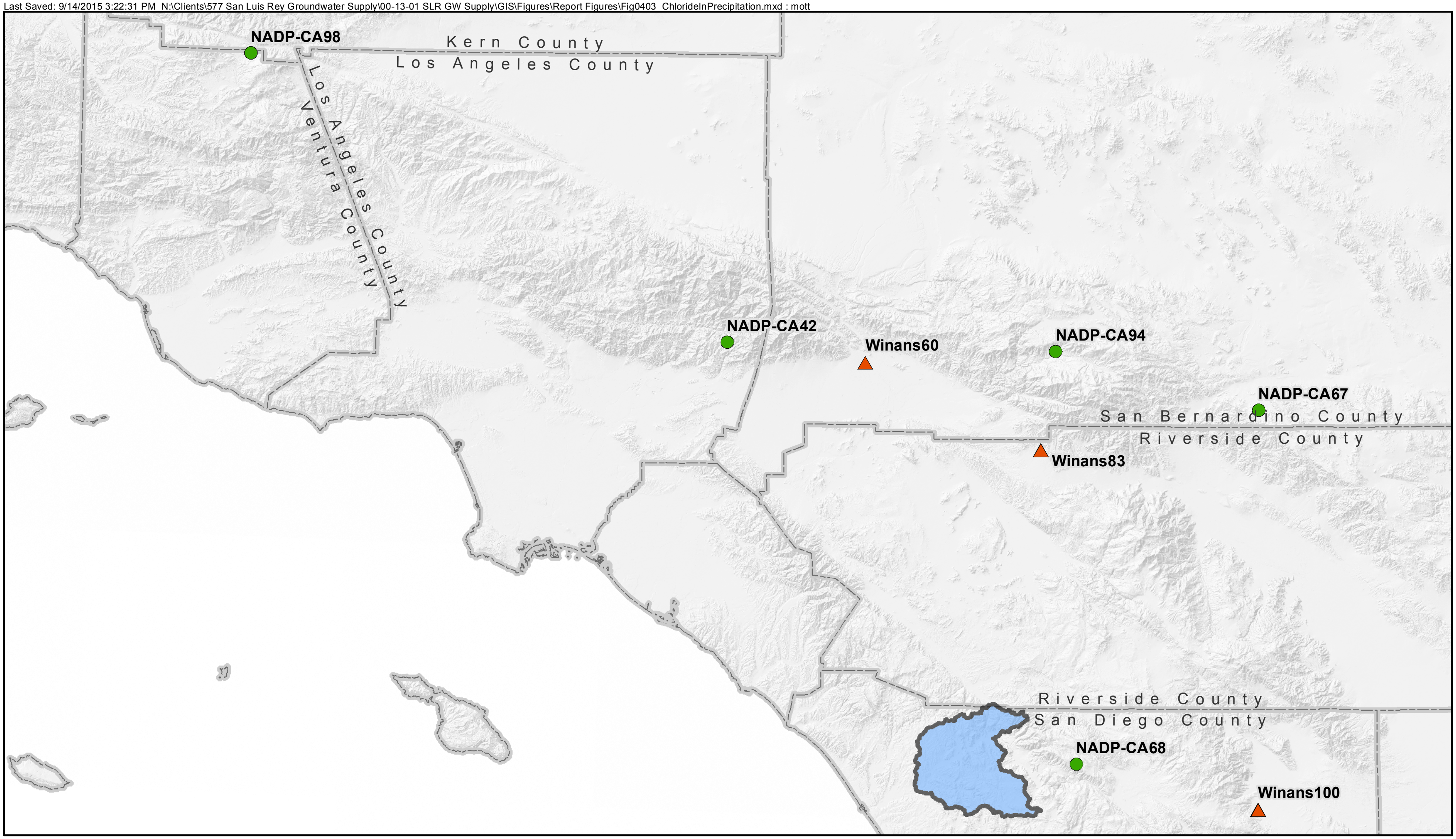
Water Budget Equation

$$R_P + R_I = S_O - S_I + U_O - U_I + D_P + D_E$$

Figure 4-2

Water Budget Diagram

Rainbow Municipal Water District
Groundwater Supply Study



Symbology

- Approximate location of stations used in Winans/Huntley Study
- National Atmospheric Deposition Program Monitoring Station
- Study Area
- County Boundary

0 10.5 21
Miles

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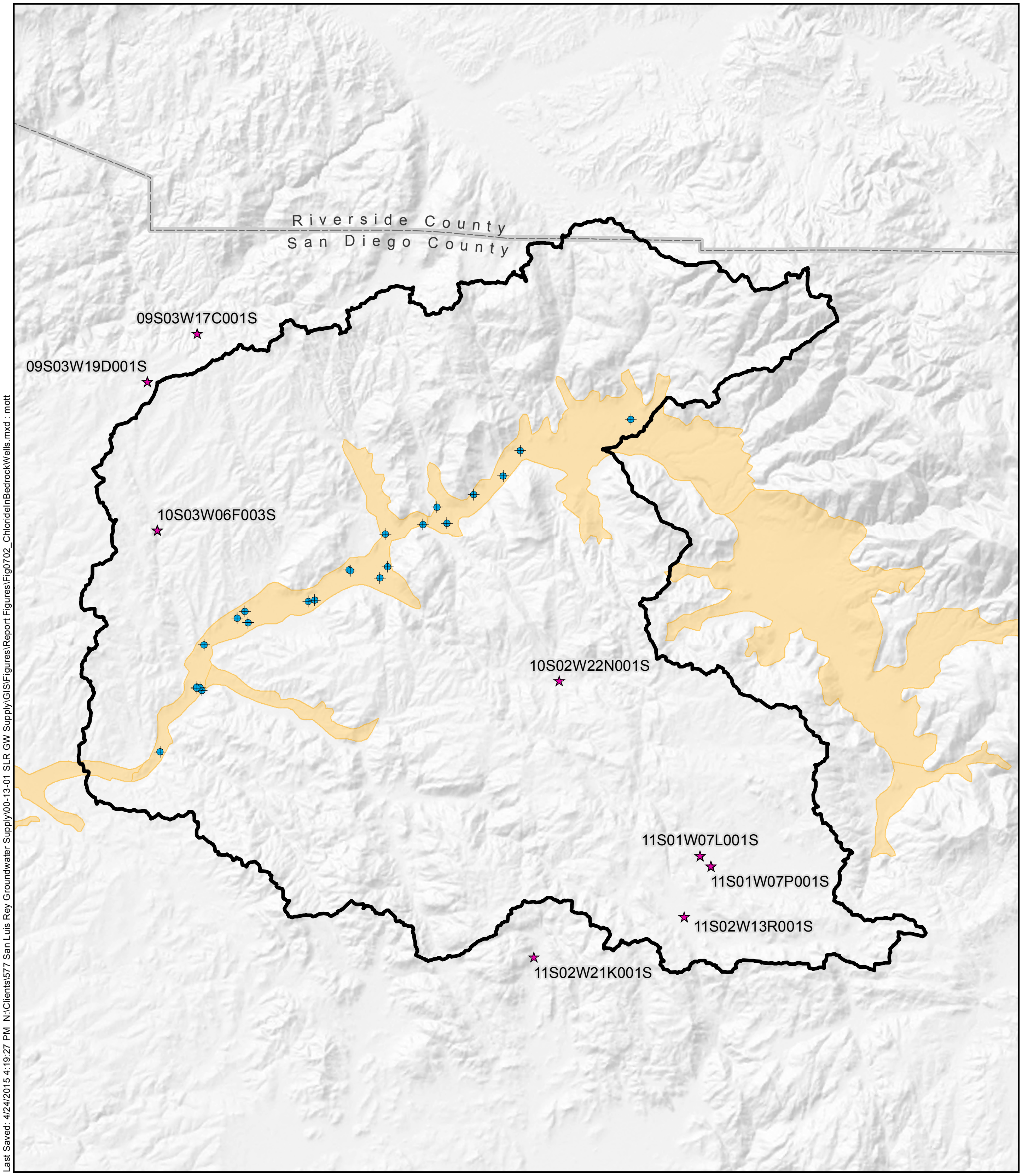
Figure 4-3

Precipitation Chloride Monitoring Sites

Rainbow Municipal Water District
Groundwater Supply Study



Symbology

- Study Area
- Wells in Alluvium
- Wells in Bedrock
- Alluvium

N

0 1.5 3

Scale in Miles

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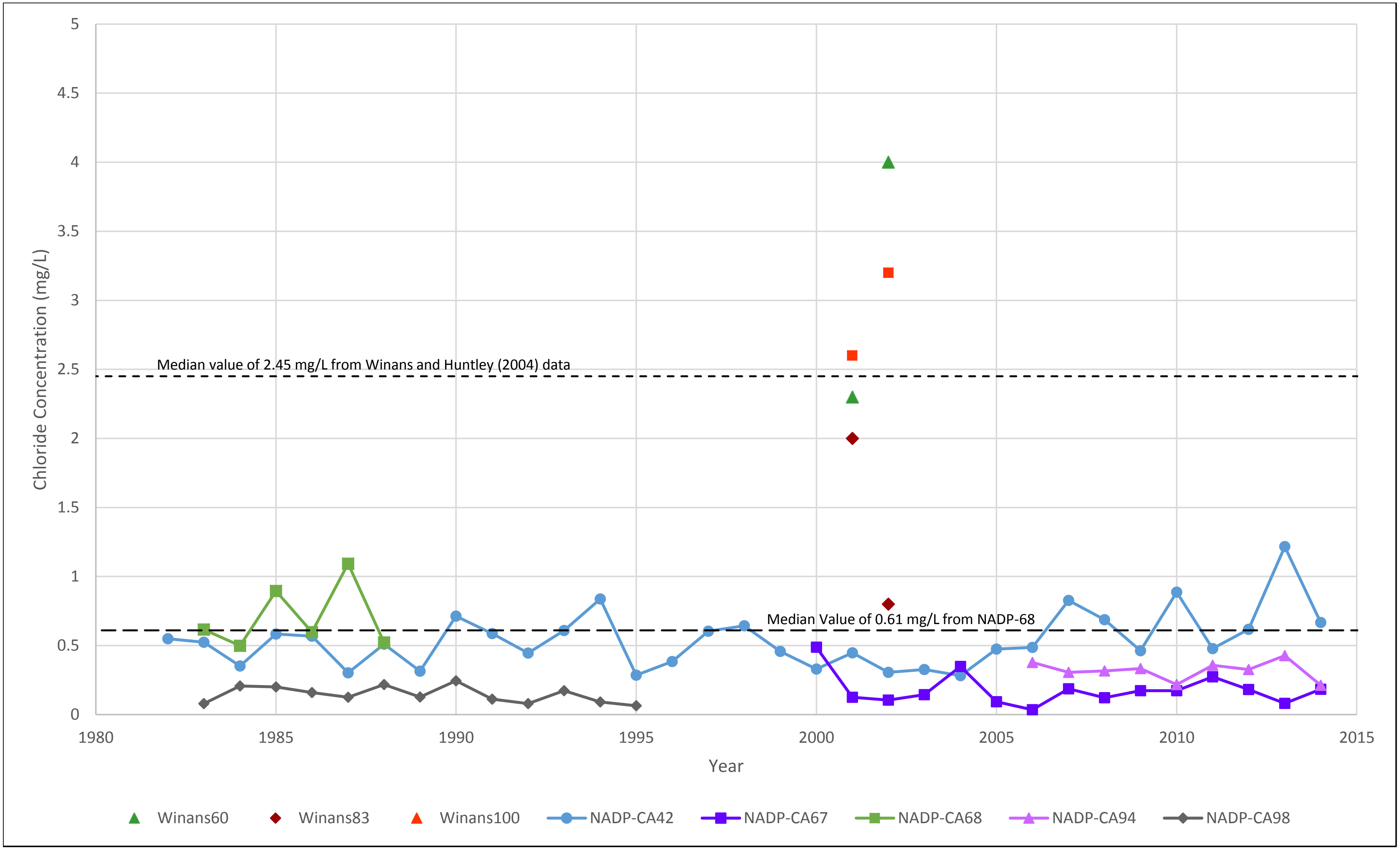
Figure 4-4

Groundwater Chloride Monitoring Wells

Rainbow Municipal Water District
Groundwater Supply Study



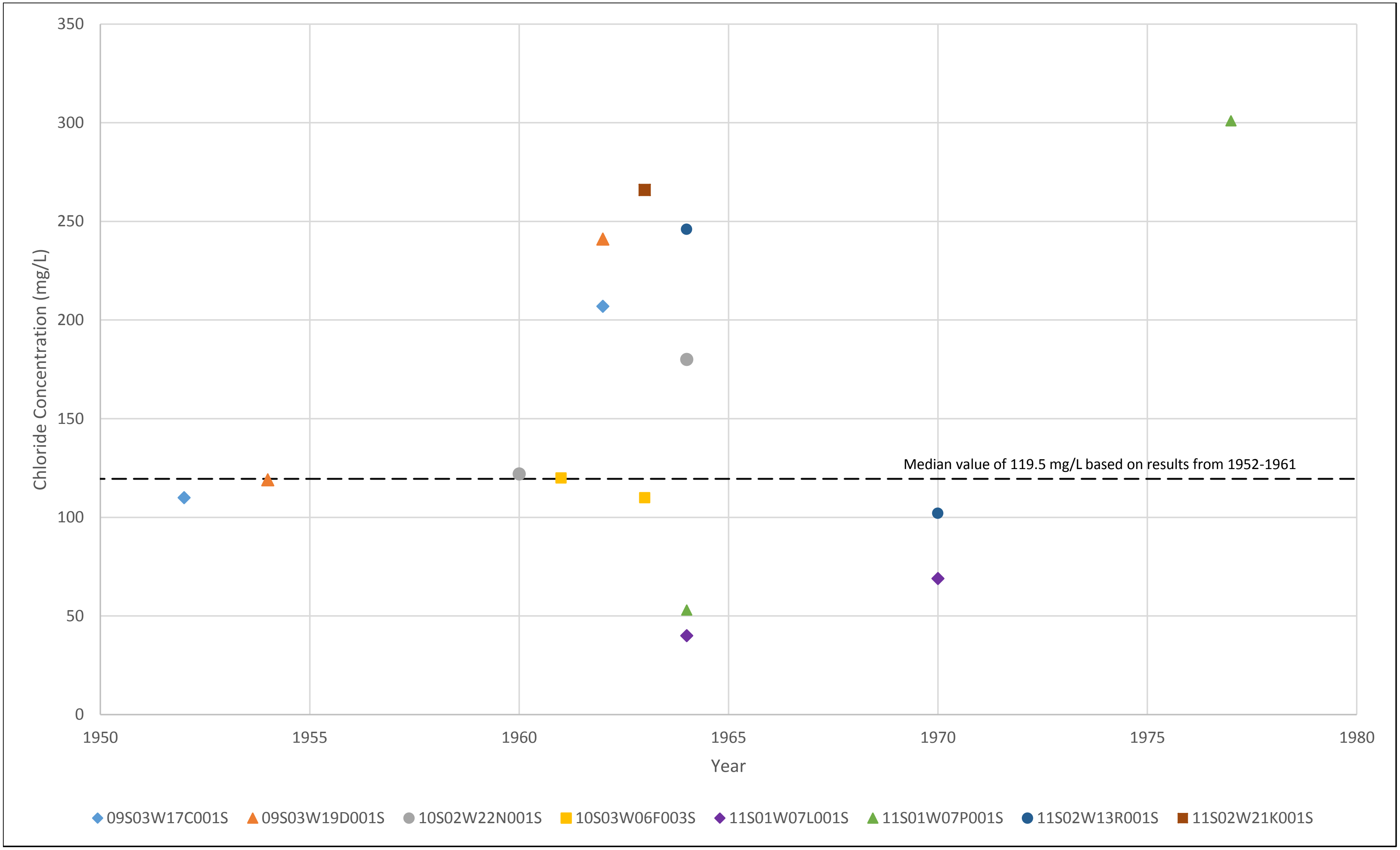
Notes:

1. Results listed as Winans are from "Using Chloride Mass Balance to Assess Recharge in Fractured Rock Aquifers of East San Diego County" (Winans and Huntley, 2004).
2. Results listed as NADP were downloaded from the National Atmospheric Depostion Program website as http://napd.sws.uiuc.edu on April 22, 2015. NADP-68 is the site closest to the study area.

Figure 4-5

Chloride Concentration in Precipitation

Rainbow Municipal Water District
Groundwater Supply Study



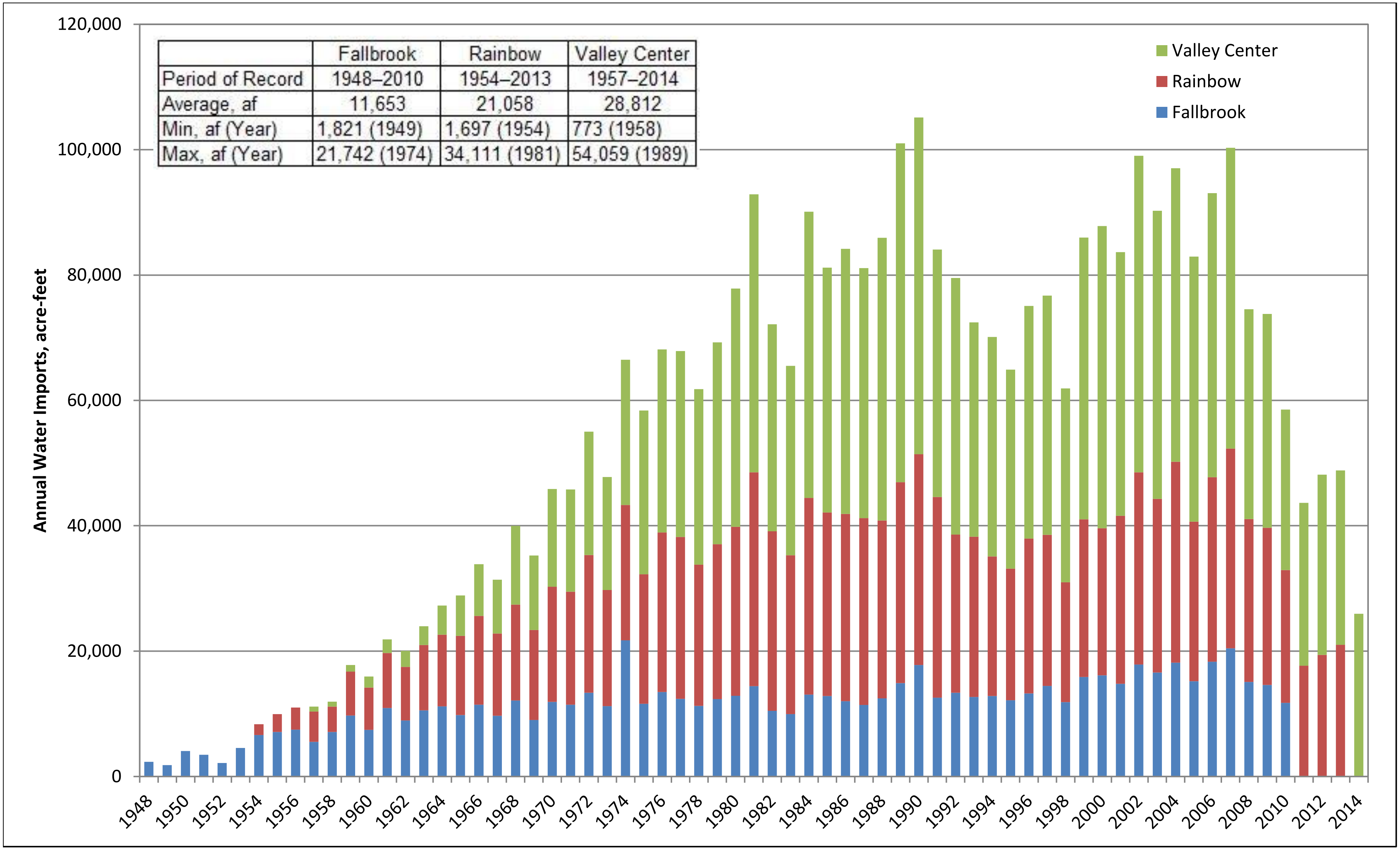
Notes:

1. The median chloride concentrations in bedrock wells was based on data from 1952-1961, which predates the use of significant quantities of imported water. For this early period, the available data suggest concentrations were relatively uniform. After 1961, concentrations were higher and more variable. This is probably attributable to the onset of recharge from imported water, which had higher chloride concentrations than recharge from precipitation.

Figure 4-6

Chloride Concentrations in Bedrock Wells

Rainbow Municipal Water District
Groundwater Supply Study

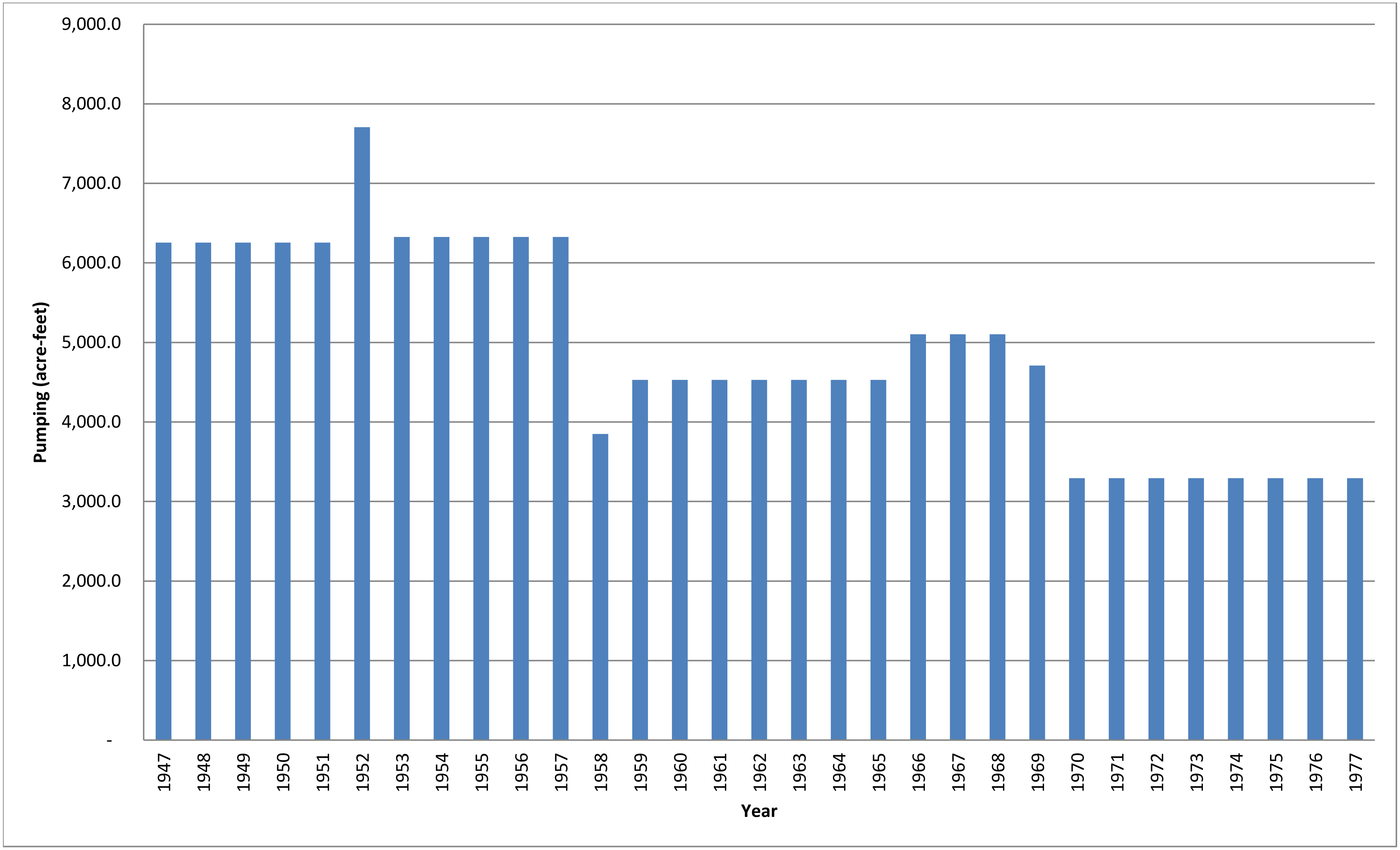


	Fallbrook	Rainbow	Valley Center
Period of Record	1948–2010	1954–2013	1957–2014
Average, af	11,653	21,058	28,812
Min, af (Year)	1,821 (1949)	1,697 (1954)	773 (1958)
Max, af (Year)	21,742 (1974)	34,111 (1981)	54,059 (1989)

Figure 4-7

Imported Water Deliveries to Study Area

Rainbow Municipal Water District
Groundwater Supply Study

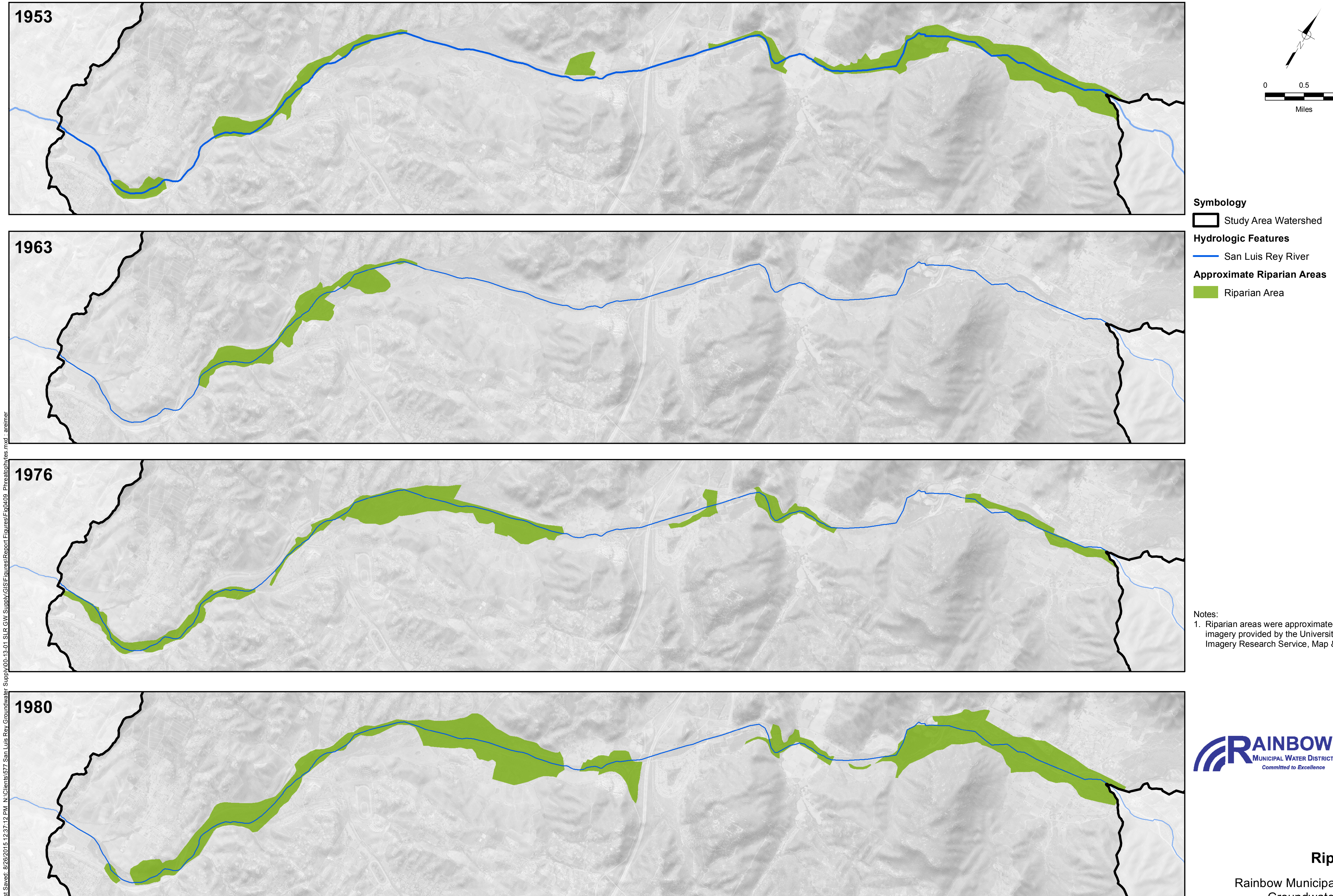


Historical net pumping is from estimates documented in Moreland, J.A., 1974, Hydrologic and Salt Balance Investigations Utilizing Digital Models, Lower San Luis Rey River Area, San Diego County, California: U.S. Geological Survey Water Resources Investigations Report 24-27.

Figure 4-8

Historical Net Groundwater Pumping used for Model Calibration

Rainbow Municipal Water District
Groundwater Supply Study



Symbology

Study Area Watershed

Hydrologic Features

San Luis Rey River

Approximate Riparian Areas

Riparian Area

Notes:
1. Riparian areas were approximated based on the aerial imagery provided by the University of Santa Barbara Aerial Imagery Research Service, Map & Imagery Laboratory.

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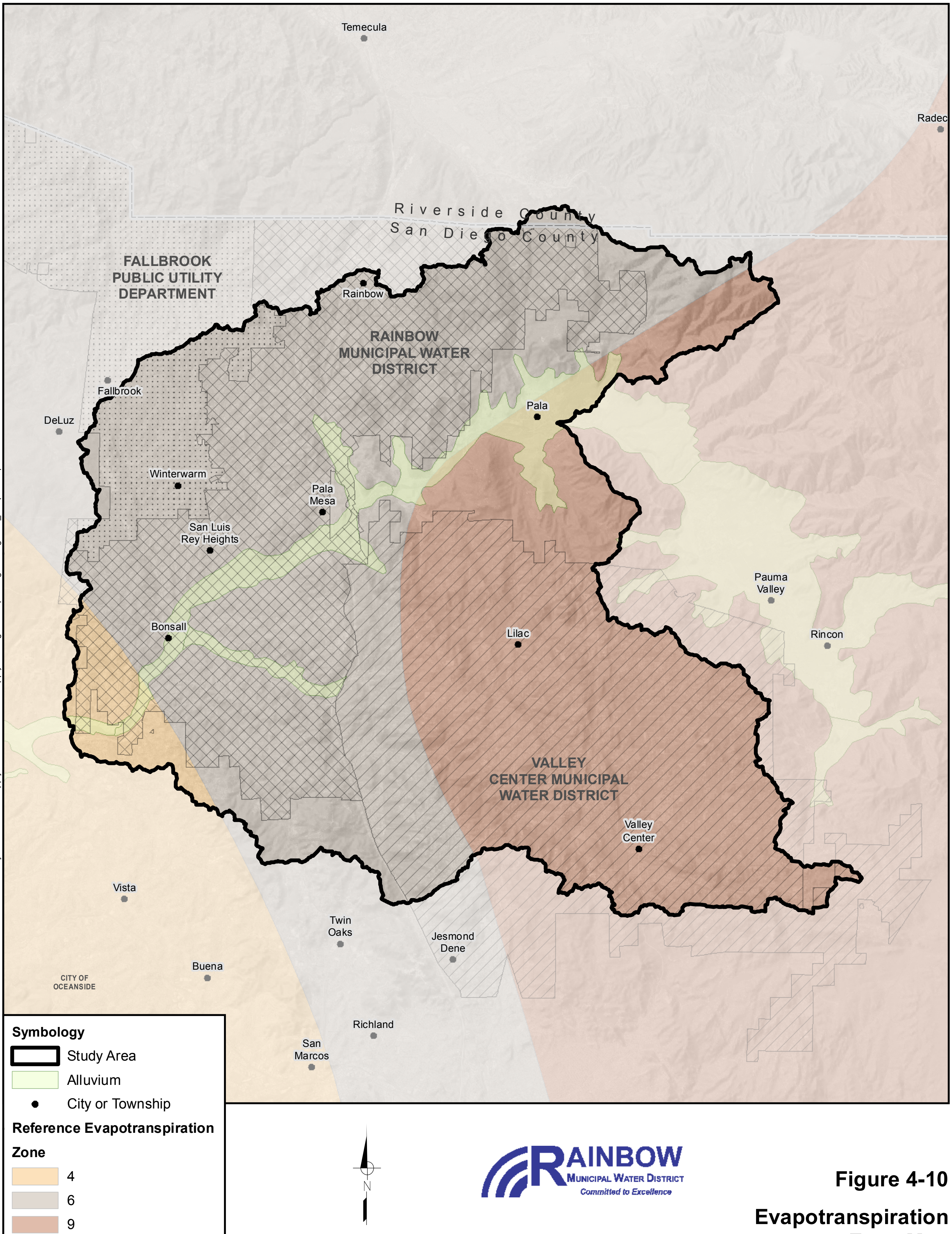
Figure 4-9

Historical Riparian Areas

Rainbow Municipal Water District
Groundwater Supply Study



Last Saved: 1/7/2016 4:50:47 PM N:\Clients\577 San Luis Rey Groundwater Supply\00-13-01 SLR GW Supply\GIS\Figures\Report Figures\Fig0410_Evapotranspiration.mxd : areimer

0 1.5 3
Scale in Miles

Notes:
1. The Reference Evapotranspiration Zone Map was downloaded from the CA Irrigation Management Information System (CIMIS, 1999)

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Figure 4-10

Evapotranspiration Zone Map

Rainbow Municipal Water District
Groundwater Supply Study

CHAPTER 5
Modeling Results

This chapter describes the simulations used to calibrate the model and assess the hydrologic effects of imported water return flows in the study area. The model simulations include the calibration version of the model for historical water years 1947 through 1977, three baseline simulations, and a projected 30-year groundwater pumping simulation based on the historical hydrology (water years 1947 through 1977), and projected 2016 through 2046 imported water quantities.

The baseline simulations were developed to establish baseline conditions without the Proposed Groundwater Supply Project under three sets of assumptions regarding the future volume of imported water return flows to the study area. The project pumping simulation was developed to assess the effect of Proposed Groundwater Supply Project pumping in comparison to the three baseline simulations.

5.1 DESCRIPTION OF MODELED SCENARIOS

The calibration version of the model was based solely on historical information from water years 1947 through 1977, as discussed in Chapter 4. The baseline and Proposed Groundwater Supply Project pumping simulations were based on the water year 1947 through 1977 hydrology used for model calibration, but incorporated projected future conditions in three ways:

1. Imported water return flows were projected for 2016 through 2046 using the methodology described in Chapter 4. Figure 5-1 shows the projected imported water return flows for the baseline and Proposed Groundwater Supply Project pumping simulations.
2. The extent of riparian vegetation along the San Luis Rey River was assumed constant at levels mapped using 1980 aerial photography, as described in Chapter 4. Note that the model simulated riparian consumptive use over this area based on the simulated depth to groundwater for the period 2016 through 2046.
3. Groundwater pumping by other parties from the portion of the San Luis Rey Valley Groundwater Basin within the study area was projected to be constant at a net annual average value of approximately 1,900 afy as discussed in Chapter 4.

5.1.1 Calibration

The calibration version of the model spans water years 1947 through 1977, which is the only time period with concurrent stream gauging records across the study area. Chapter 4 describes the model inputs and calibration.

5.1.2 Baseline 1: Baseline Conditions prior to Imported Water

This scenario simulates stream flow in the San Luis Rey River and groundwater flow in the alluvial aquifer using the historical hydrology projected for water years 2016 through 2046 assuming no imported water and no groundwater pumping project to recover imported water return flows.

The Proposed Groundwater Supply Project scenario was compared to this baseline to evaluate potential impacts to diverters of natural flows in the San Luis Rey River.

5.1.3 Baseline 2: Baseline Conditions with Imported Water Return Flows from RMWD, FPUD and VCMWD

This scenario simulates stream flow in the San Luis Rey River and groundwater flow in the alluvial aquifer, assuming projected 2016 through 2046 imported water quantities for RMWD, FPUD and VCMWD, and no groundwater pumping project to recover imported water return flows. Figure 5-1 shows the projected imported water return flows.

The Proposed Groundwater Supply Project scenario was compared to this baseline to evaluate potential impacts to appropriators of imported water return flows not recovered by RMWD, FPUD and VCMWD.

5.1.4 Baseline 3: Baseline Conditions with Imported Water Return Flows from FPUD and VCMWD Only

This scenario simulates stream flow in the San Luis Rey River and groundwater flow in the alluvial aquifer, assuming projected 2016 through 2046 imported water quantities for FPUD and VCMWD only, and no groundwater pumping project to recover imported water return flows.

The Proposed Groundwater Supply Project scenario was compared to this baseline to evaluate potential impacts to appropriators of imported water return flows abandoned by FPUD and VCMWD. Stream flow under Baseline 2 is greater than under Baseline 3, because Baseline 2 includes RMWD imported water return flows and Baseline 3 does not. The differences between the two baselines were compared to the differences between Baseline 2 and the groundwater pumping scenarios to evaluate whether or not pumping to recover RMWD's imported water return flows effected stream flow relative to the stream flow that would have occurred if FPUD and VCMWD imported water but RMWD did not import water.

5.1.5 Groundwater Project Scenario: Pumping of Imported Water Return Flows at Distributed Sites in the Lower San Luis Rey Valley Groundwater Basin

This scenario simulates stream flow in the San Luis Rey River and groundwater flow in the alluvial aquifer, assuming projected 2016 through 2046 imported water quantities for RMWD, FPUD and VCMWD, and groundwater pumping from 18 wells distributed across the RMWD service area overlying the Lower San Luis Rey Valley Groundwater Basin to recover imported water return flows.

The locations of the simulated wells are shown on Figure 5-2.

5.2 SIMULATION RESULTS

The following discussion provides analysis of the results of the baseline and Proposed Groundwater Supply Project simulations. The discussion first describes the pumping simulated for the Proposed Groundwater Supply Project and then provides an assessment of the simulated potential effects of the pumping on the hydrology of the Lower San Luis Rey Valley Groundwater Basin and flow in the San Luis Rey River relative to baseline conditions.

Tables 5-1 through 5-4 provide the simulated water budgets for the baseline and Proposed Groundwater Supply Project scenarios.

5.2.1 Simulated Pumping for the Proposed Groundwater Supply Project

Figures 5-3 and 5-4 show the simulated annual and monthly Proposed Groundwater Supply Project pumping. The time scale on the figures shows the period from 2016 through 2046. This period is represented in the model by the historical hydrology for water years 1947 through 1977 and the projected 2016 through 2046 return flow quantities accruing from RMWD, FPUD and VCMWD water imports to the study area.

The pumping quantities shown on the figures were developed in multiple steps using the numerical model. Initially, the proposed project pumping was specified as model input. This input pumping was estimated based on the water budget discussed in Chapter 4, which shows that proposed project pumping equals imported water return flows minus groundwater consumptive uses supported by imported water return flows, and changes in groundwater storage. However, it became apparent that the pumping volumes expected to be reasonable based on the water budget could not be sustained in the simulations because of the dynamic nature of the San Luis Rey Valley stream-aquifer system. Groundwater levels change quickly in response to stream flow events and groundwater pumping, because of the relatively large size of the study area watershed in relation to the relatively limited extent and thickness of the alluvial aquifer. In essence, pumping rates needed to be matched to the dynamics of basin inflows because of the limited storage volume in the alluvial aquifer. This need was addressed in subsequent modeling steps.

The pumping locations shown on Figure 5-2 were reconfigured as drains, which removed water from the alluvial aquifer whenever groundwater elevations were at or above an elevation between the base of the San Luis Rey River bed and the base of the alluvial aquifer. The fluxes from the drains are a representation of the pumping that could be accomplished without drawing the aquifer down past the drain elevation. Initially, the drain elevations at each of the well locations shown on Figure 5-2 were set to an elevation midway between San Luis Rey River bed and the base of the alluvial aquifer. Multiple iterations of the proposed groundwater pumping scenario were then run with varying drain elevations. Drain elevations set 23 feet above the midpoint between San Luis Rey River bed and the base of the alluvial aquifer were found to produce favorable results for the San Luis Rey streamflow-aquifer system relative to baseline conditions.

Figures 5-3 and 5-4 show the resulting annual and monthly pumping volumes respectively. Annual volumes ranged from approximately 4,800 to 9,300 afy and had a median value of approximately 5,700 afy.

Table 5-1. Water Budget for Hydrologic System for Baseline 1 Scenario, acre-ft

Water Year	Inflows					Outflows				
	San Luis Rey River Inflow	San Luis Rey Tributary Inflow	Precipitation Recharge	Eastern Boundary Underflow	Imported Water	San Luis Rey River Outflow	Consumptive Use by Riparian Vegetation	Groundwater Pumping by Others	Groundwater Project Pumping	Western Boundary Underflow
2016	4,756	7	2,000	490	-	4,617	4,752	1,871	-	399
2017	1,948	5	2,000	490	-	1,443	4,626	1,868	-	396
2018	2,714	48	2,000	490	-	1,742	4,464	1,871	-	374
2019	1,272	1	2,000	490	-	387	4,208	1,876	-	357
2020	881	-	2,000	490	-	300	3,785	1,867	-	339
2021	14,361	3,392	2,000	490	-	23,962	4,498	2,027	-	342
2022	1,251	16	2,000	490	-	273	4,249	1,871	-	336
2023	3,630	742	2,000	490	-	3,981	4,124	1,871	-	324
2024	637	17	2,000	490	-	200	3,549	1,871	-	311
2025	1,098	0	2,000	490	-	295	3,259	1,889	-	292
2026	-	1	2,000	490	-	121	2,668	1,868	-	273
2027	12,240	2,899	2,000	490	-	20,990	3,610	2,040	-	289
2028	287	3	2,000	490	-	144	3,193	1,871	-	285
2029	-	9	2,000	490	-	97	2,578	1,868	-	261
2030	-	-	2,000	490	-	71	2,321	1,871	-	255
2031	-	37	2,000	490	-	52	2,200	1,875	-	239
2032	-	-	2,000	490	-	38	2,137	2,048	-	238
2033	-	-	2,000	490	-	27	2,081	1,868	-	226
2034	-	15	2,000	490	-	16	2,043	1,871	-	219
2035	1,495	693	2,000	490	-	991	2,377	1,871	-	250
2036	7,252	1,005	2,000	490	-	7,939	2,744	1,871	-	270
2037	224	18	2,000	490	-	18	2,455	1,853	-	222
2038	13,134	3,070	2,000	490	-	19,075	3,570	2,050	-	249
2039	454	277	2,000	490	-	233	3,152	1,871	-	253
2040	-	36	2,000	490	-	41	2,536	1,871	-	239
2041	-	0	2,000	490	-	26	2,254	1,868	-	225
2042	591	951	2,000	490	-	613	2,607	1,886	-	226
2043	225	153	2,000	490	-	24	2,478	1,868	-	232
2044	127	134	2,000	490	-	9	2,302	2,041	-	212
2045	500	275	2,000	490	-	12	2,444	1,868	-	205
2046	729	3	2,000	490	-	5	2,627	2,027	-	196
Average	2,252	445	2,000	490	-	2,830	3,093	1,903	-	275

Table 5-2. Water Budget for Hydrologic System for Baseline 2 Scenario, acre-ft										
	Inflows					Outflows				
Water Year	San Luis Rey River Inflow	San Luis Rey Tributary Inflow	Precipitation Recharge	Eastern Boundary Underflow	Imported Water	San Luis Rey River Outflow	Consumptive Use by Riparian Vegetation	Groundwater Pumping by Others	Groundwater Project Pumping	Western Boundary Underflow
2016	4,756	7	2,000	490	7,194	5,349	4,890	1,871	-	399
2017	1,948	5	2,000	490	7,194	3,230	4,960	1,868	-	397
2018	2,714	48	2,000	490	7,194	4,020	4,944	1,871	-	398
2019	1,272	1	2,000	490	7,194	2,756	4,881	1,876	-	397
2020	881	-	2,000	490	7,173	2,452	4,826	1,867	-	397
2021	14,361	3,392	2,000	490	7,170	29,019	5,059	2,027	-	406
2022	1,251	16	2,000	490	7,170	3,185	4,950	1,871	-	398
2023	3,630	742	2,000	490	7,170	7,989	4,930	1,871	-	404
2024	637	17	2,000	490	7,170	2,864	4,792	1,871	-	398
2025	1,098	0	2,000	490	7,582	3,280	4,719	1,889	-	398
2026	-	1	2,000	490	7,637	2,951	4,352	1,868	-	398
2027	12,240	2,899	2,000	490	7,637	27,097	4,803	2,040	-	402
2028	287	3	2,000	490	7,637	3,219	4,633	1,871	-	398
2029	-	9	2,000	490	7,637	3,209	4,304	1,868	-	398
2030	-	-	2,000	490	7,637	3,224	4,206	1,871	-	398
2031	-	37	2,000	490	7,637	3,288	4,179	1,875	-	398
2032	-	-	2,000	490	7,637	3,308	4,183	2,048	-	398
2033	-	-	2,000	490	7,637	3,336	4,161	1,868	-	398
2034	-	15	2,000	490	7,637	3,368	4,166	1,871	-	398
2035	1,495	693	2,000	490	7,637	6,365	4,458	1,871	-	399
2036	7,252	1,005	2,000	490	7,637	13,755	4,745	1,871	-	401
2037	224	18	2,000	490	7,637	3,601	4,514	1,853	-	398
2038	13,134	3,070	2,000	490	7,637	28,594	4,872	2,050	-	410
2039	454	277	2,000	490	7,637	4,622	4,683	1,871	-	399
2040	-	36	2,000	490	7,637	3,768	4,391	1,871	-	398
2041	-	0	2,000	490	7,637	3,705	4,259	1,868	-	398
2042	591	951	2,000	490	7,637	6,570	4,620	1,886	-	401
2043	225	153	2,000	490	7,637	4,115	4,555	1,868	-	399
2044	127	134	2,000	490	7,637	4,055	4,447	2,041	-	398
2045	500	275	2,000	490	7,637	4,439	4,616	1,868	-	398
2046	729	3	2,000	490	7,637	3,852	4,816	2,027	-	398
Average	2,252	445	2,000	490	7,503	6,600	4,610	1,903	-	399

Table 5-3. Water Budget for Hydrologic System for Baseline 3 Scenario, acre-ft

Water Year	Inflows					Outflows				
	San Luis Rey River Inflow	San Luis Rey Tributary Inflow	Precipitation Recharge	Eastern Boundary Underflow	Imported Water	San Luis Rey River Outflow	Consumptive Use by Riparian Vegetation	Groundwater Pumping by Others	Groundwater Project Pumping	Western Boundary Underflow
2016	4,749	7	2,000	490	4,632	4,816	4,788	1,871	-	399
2017	1,947	5	2,000	490	4,632	2,069	4,751	1,868	-	397
2018	2,711	48	2,000	490	4,632	2,618	4,692	1,871	-	398
2019	1,271	1	2,000	490	4,632	1,248	4,596	1,876	-	397
2020	879	-	2,000	490	4,653	865	4,510	1,867	-	397
2021	14,341	3,392	2,000	490	4,656	26,959	4,835	2,027	-	404
2022	1,249	16	2,000	490	4,656	1,516	4,700	1,871	-	397
2023	3,625	742	2,000	490	4,656	6,075	4,661	1,871	-	403
2024	637	17	2,000	490	4,656	1,069	4,498	1,871	-	397
2025	1,096	0	2,000	490	4,931	1,382	4,404	1,889	-	397
2026	-	1	2,000	490	4,968	995	4,024	1,868	-	397
2027	12,223	2,899	2,000	490	4,968	24,696	4,541	2,040	-	401
2028	287	3	2,000	490	4,968	1,234	4,364	1,871	-	397
2029	-	9	2,000	490	4,968	1,132	4,006	1,868	-	397
2030	-	-	2,000	490	4,968	1,105	3,898	1,871	-	397
2031	-	37	2,000	490	4,968	1,132	3,873	1,875	-	396
2032	-	-	2,000	490	4,968	1,129	3,879	2,048	-	396
2033	-	-	2,000	490	4,968	1,143	3,865	1,868	-	396
2034	-	15	2,000	490	4,968	1,155	3,873	1,871	-	396
2035	1,493	693	2,000	490	4,968	4,005	4,193	1,871	-	396
2036	7,242	1,005	2,000	490	4,968	11,335	4,500	1,871	-	399
2037	224	18	2,000	490	4,968	1,366	4,262	1,853	-	396
2038	13,116	3,070	2,000	490	4,968	25,947	4,660	2,050	-	407
2039	453	277	2,000	490	4,968	2,376	4,461	1,871	-	396
2040	-	36	2,000	490	4,968	1,485	4,147	1,871	-	396
2041	-	0	2,000	490	4,968	1,409	4,011	1,868	-	396
2042	590	951	2,000	490	4,968	4,045	4,395	1,886	-	399
2043	224	153	2,000	490	4,968	1,814	4,326	1,868	-	396
2044	127	134	2,000	490	4,968	1,715	4,207	2,041	-	396
2045	506	275	2,000	490	4,968	2,061	4,382	1,868	-	396
2046	722	3	2,000	490	4,968	1,482	4,588	2,027	-	396
Average	2,249	445	2,000	490	4,873	4,561	4,351	1,903	-	398

Table 5-4. Water Budget for Hydrologic System for Pumping Scenario, acre-ft

Water Year	Inflows					Outflows				
	San Luis Rey River Inflow	San Luis Rey Tributary Inflow	Precipitation Recharge	Eastern Boundary Underflow	Imported Water	San Luis Rey River Outflow	Consumptive Use by Riparian Vegetation	Groundwater Pumping by Others	Groundwater Project Pumping	Western Boundary Underflow
2016	4,756	7	2,000	490	7,194	44	3,504	1,871	10,534	264
2017	1,948	5	2,000	490	7,194	3	3,193	1,868	5,839	151
2018	2,714	48	2,000	490	7,194	3	3,120	1,871	6,095	144
2019	1,272	1	2,000	490	7,194	4	2,992	1,876	5,262	146
2020	881	-	2,000	490	7,173	5	2,879	1,867	4,857	151
2021	14,361	3,392	2,000	490	7,170	20,676	3,578	2,027	8,823	247
2022	1,251	16	2,000	490	7,170	3	3,172	1,871	5,835	204
2023	3,630	742	2,000	490	7,170	3,136	3,202	1,871	6,459	246
2024	637	17	2,000	490	7,170	3	2,941	1,871	5,291	261
2025	1,098	0	2,000	490	7,582	7	2,842	1,889	5,284	281
2026	-	1	2,000	490	7,637	21	2,414	1,868	5,160	315
2027	12,240	2,899	2,000	490	7,637	20,643	3,162	2,040	7,131	361
2028	287	3	2,000	490	7,637	54	2,827	1,871	5,597	357
2029	-	9	2,000	490	7,637	63	2,406	1,868	5,351	363
2030	-	-	2,000	490	7,637	70	2,252	1,871	5,296	368
2031	-	37	2,000	490	7,637	77	2,196	1,875	5,312	373
2032	-	-	2,000	490	7,637	84	2,174	2,048	5,307	376
2033	-	-	2,000	490	7,637	88	2,151	1,868	5,316	378
2034	-	15	2,000	490	7,637	91	2,147	1,871	5,334	381
2035	1,495	693	2,000	490	7,637	2,004	2,538	1,871	5,948	384
2036	7,252	1,005	2,000	490	7,637	8,915	2,909	1,871	6,400	392
2037	224	18	2,000	490	7,637	123	2,607	1,853	5,629	395
2038	13,134	3,070	2,000	490	7,637	20,821	3,309	2,050	8,468	405
2039	454	277	2,000	490	7,637	581	2,939	1,871	6,186	398
2040	-	36	2,000	490	7,637	163	2,525	1,871	5,756	398
2041	-	0	2,000	490	7,637	150	2,338	1,868	5,656	398
2042	591	951	2,000	490	7,637	1,517	2,823	1,886	6,375	400
2043	225	153	2,000	490	7,637	323	2,704	1,868	5,886	398
2044	127	134	2,000	490	7,637	288	2,536	2,041	5,806	398
2045	500	275	2,000	490	7,637	486	2,702	1,868	5,856	398
2046	729	3	2,000	490	7,637	174	2,873	2,027	5,785	398
Average	2,252	445	2,000	490	7,503	2,601	2,773	1,903	6,059	327

Monthly volumes ranged from approximately 370 to 1,350 acre-feet per month (afm) and had a median value of approximately 460 afm.

In some wet years significant flow was simulated in the San Luis Rey River. This flow sustained groundwater elevations, which resulted in drain fluxes that were larger than would be anticipated for a Proposed Groundwater Supply Project to recover imported water return flows. For example, a drain flux of over 9,000 acre-feet was simulated in water year 2016, but only a portion of this water would be pumped as imported water return flows, because project infrastructure would not be designed to capture relatively infrequent and unpredictable events. For this reason, the drain fluxes were capped at 4,000 afy for the purpose of sizing Proposed Groundwater Supply Project infrastructure. This is equivalent to a monthly pumping of approximately 333 afm, which is slightly less than the simulated minimum monthly value of 370 afm.

5.2.2 Simulated Groundwater Conditions

Figure 5-2 shows the modeled portion of the San Luis Rey Valley Groundwater Basin. The following sections discuss the simulated changes in groundwater underflow and levels resulting from the Proposed Groundwater Supply Project.

5.2.2.1 Simulated Groundwater Underflow

Figure 5-5 shows the annual volumes of groundwater underflow leaving the study area through the alluvial aquifer. Tables 5-1 through 5-4 show that groundwater underflow is typically the smallest component of the water budget. Stream flow, recharge, groundwater pumping and consumptive use by riparian vegetation are all measured in thousands of acre-feet per year, while under flow is measured in hundreds of acre-feet per year. Even though outgoing underflow is a small component of the water budget, it is an important indicator of potential effects of the Proposed Groundwater Supply Project on the San Luis Rey Valley stream-aquifer system, because reductions in underflow could be associated with capture of stream flow by groundwater pumping.

Under Baseline 1, groundwater underflow gradually decreased over the time frame of the simulation from an initial value of approximately 400 afy to a final value of approximately 200 afy. This is attributable to ongoing, existing pumping that gradually removes water from storage in the aquifer in the absence of any imported water return flows. This simulated result is similar to the actual history of the groundwater basin. The groundwater basin was essentially dewatered by groundwater pumping prior to the importation of water (Izbicki, 1985). Removal of groundwater from storage would have reduced groundwater outflows, consistent with the results for Baseline 1.

Under Baseline 2, which includes imported water return flows from RMWD, FPUD and VCMWD, outgoing groundwater underflows were stable at approximately 400 afy for the duration of the simulation. This volume of outflow is the approximate maximum amount of water that can flow through the alluvial aquifer at the downstream end of the study area. The aquifer is essentially full and any additional flows out of the study area would be manifested as stream flow. This simulated stream flow is confirmed by comparison of the water budgets for Baseline 1 and 2, in Tables 5-1 and 5-2, respectively, which show a 3,770-afy average annual increase in stream flow resulting from imported water return flows to RMWD, FPUD and VCMWD.

Baseline 3 simulates the effects of return flows from water imported by FPUD and VCMWD, but not RMWD. Outgoing groundwater underflows were stable at approximately 400 afy for the duration of the simulation, again, indicating that the aquifer is essentially full and any additional flows out of the study area would be manifested as stream flow. Comparison of the water budgets for Baseline 1 and 3, in Tables 5-1 and 5-3, respectively, which show an average 1,731-afy annual increase in stream flow resulting from imported water return flows to FPUD and VCMWD.

Figure 5-5 shows that outgoing groundwater underflows were initially reduced under the proposed project groundwater scenario, but increased after the first few years of the simulation and stabilized at 400 afy.

The initial decrease in outgoing groundwater underflows is in response to the initial conditions of the simulation and is not reflective of the actual response of the alluvial aquifer to groundwater pumping under the proposed project. In the Baseline 1 simulation, outflows start at 400 afy and decrease each year due to preexisting pumping by others in the absence of any imported water return flows. At the beginning of the Proposed Groundwater Supply Project simulation, the pumping under the proposed project is added to the preexisting pumping, but imported water return flows have not infiltrated through the bedrock system to recharge the alluvial aquifer sufficiently to offset the pumping, which captures more of the outflow. After the first few years of the Proposed Groundwater Supply Project simulation, increasing amounts of recharge reach the alluvial aquifer causing an increase in outgoing groundwater outflows and their ultimate stabilization at levels indicative of a full aquifer.

In actuality, imported water return flows would not be initiated simultaneously with the proposed groundwater pumping project but instead would have been flowing to the San Luis Rey streamflow-aquifer system for many years. Because the Proposed Groundwater Supply Project simulation shows ultimate stabilization of groundwater outflows at approximately 400 afy, it would also show outflows of approximately 400 afy at the beginning of the simulation if imported water return flows were simulated to recharge the alluvial aquifer initially and throughout the simulation.

The simulation results show that the Proposed Groundwater Supply Project will not reduce groundwater outflows relative to baseline conditions, and, in fact, underflows with existing imported water return flows and the Proposed Groundwater Supply Project are larger than would exist under conditions without imported water return flows (Baseline 1) and no Proposed Groundwater Supply Project.

5.2.2.2 Simulated Groundwater Levels

Figure 5-6 shows the model-simulated observation wells used to compare simulated groundwater levels generated by the baseline and Proposed Groundwater Supply Project. The observation wells are located at the same locations used in the prior USGS modeling study of the San Luis Rey River Valley Groundwater Basin (Moreland, 1974).

Figures 5-7 through 5-13 are simulated hydrographs of the groundwater levels under the baseline and Proposed Groundwater Supply Project scenarios at each of the observation wells shown on Figure 5-6. The hydrographs are presented from downstream (Figure 5-7) to upstream (Figure 5-13).

Figure 5-7 shows hydrographs for an observation well located approximately one mile downstream of the Proposed Groundwater Supply Project. All of the modeled scenarios produced virtually identical groundwater elevations, which exhibited very little change seasonally and over the length of the various simulations.

Figure 5-8 through 5-10 show hydrographs for observation wells located in the Proposed Groundwater Supply Project area. Proposed project wells are shown on Figure 5-2. Each of the hydrographs show a similar pattern, in which Baselines 2 and 3 have similar, stable groundwater elevations that show minimal seasonal fluctuation and long-term stability. Groundwater levels are approximately five to 15 feet lower in Baseline 1 and the Proposed Groundwater Supply Project hydrographs, but groundwater levels are generally higher in the Proposed Groundwater Supply Project than in Baseline 1. Several of the observation wells (Figure 5-9 and Figure 5-10) have simulated hydrographs that show lower groundwater levels in the Proposed Groundwater Supply Project than in Baseline 1 during the initial years of the simulation. As described in the preceding section on groundwater underflow, this is a consequence of the assumed initial conditions of the simulation. The combined effects of existing pumping and proposed project pumping cause groundwater levels to decrease until imported water return flows move through the bedrock fracture system and begin recharging the alluvial aquifer. In the real system, these flows have been recharging the alluvial aquifer for decades and would offset the initial groundwater withdrawals caused by pumping for the Proposed Groundwater Supply Project.

Figure 5-11 shows hydrographs for an observation well located in the Proposed Groundwater Supply Project area. The hydrographs are similar to those shown on Figures 5-8 through 5-10, except that the groundwater levels generated by the Proposed Groundwater Supply Project simulation are consistently approximately ten to twenty feet lower than the baseline results. There are two reasons for this. First, comparison of Figures 5-2 and 5-6 shows that the observation well (10S03W15E1) is close to one of the pumping wells include in the Proposed Groundwater Supply Project (Well 14). Drawdown in the local area near Well 14 causes a lowering of the groundwater elevation in observation well 10S03W15E1 because of its proximity. Second, observation well 10S03W15E1 is the most upstream of the observations wells within the area of the proposed project. Because it is upstream, this part of the proposed project area receives the least amount of cumulative recharge from imported water return flows. This means that any impacts that might occur during project operations could be mitigated by optimizing flow rates in the project wells to balance pumping with the available recharge on a well by well basis.

Figure 5-12 and 5-13 show hydrographs for observation wells located approximately 1.5 and 4.5 miles upstream of the Proposed Groundwater Supply Project. All of the modeled scenarios produced virtually identical groundwater elevations, which exhibited very little change seasonally and over the length of the simulation at both locations.

These results demonstrates that the Proposed Groundwater Supply Project does not reduce groundwater levels below the level that would exist in the absence of imported water (Baseline 1), except at locations near proposed project wells. These effects can be mitigated by optimizing flow rates across the well network to balance pumping with the geographic availability of recharge.

The simulated groundwater elevation results show that the Proposed Groundwater Supply Project lowers groundwater elevations relative to the baseline scenarios that include imported water return flows (Baselines 2 and 3). However these effects are limited to the area of the Proposed Groundwater Supply Project and result in no more than approximately 20 feet of reduction in groundwater levels.

5.2.3 Simulated Streamflow Conditions

Figures 5-14 and 5-15 show the simulated annual discharge in the San Luis Rey River at downstream and upstream locations respectively for the baseline and Proposed Groundwater Supply Project scenarios. Figure 5-14 represent USGS gauge 1104100, and Figure 5-15 represents USGS gauge 1104000. The USGS gauge locations are shown on Figure 5-2.

Figure 5-14 shows that annual river discharge out of the study area decreases over the simulation period under Baseline 1. Except for occasional wet years (e.g., 2021, 2027, 2036 and 2038) when stream flows exceed 5,000 afy, annual river discharge decreases from slightly less than 5,000 afy in 2016 to values typically less than 100 afy by 2029. This decrease is consistent with the historical river flow prior to significant importation of water and occurs because Baseline 1 does not include imported water return flows but does include preexisting groundwater pumping. The groundwater pumping causes decrease in groundwater elevations, as discussed in Section 5.2.2, and captures stream flow by reducing or preventing groundwater discharges to the stream that otherwise would contribute to base flow and by increasing riverbed infiltration rates to their maximum, as discussed in Chapter 4.

The simulated annual San Luis Rey River discharge out of the study area increases over time under the Proposed Groundwater Supply Project scenario as a consequence of the initial conditions in the model (Figure 5-14). Except in the wettest years, flows near the beginning of the simulation are less than 100 afy. The discharges increases over time as imported water return flows are simulated to move through the fractured bedrock aquifer to recharge the alluvial aquifer and restore base flows. Consistent with the trends in outgoing groundwater underflow and groundwater levels in the alluvial aquifer, San Luis Rey River discharge begins to increase in approximately in 2026 and reaches levels in the low 100s of acre-feet per year by 2035. As with groundwater underflow and levels, this simulated trend indicates that annual river discharge will be sustained in the low 100s of acre-feet (and significantly higher in the occasional wet year), because imported water has been recharging the San Luis Rey Valley stream-aquifer system for decades. In contrast the model assumes that imported water recharge in the Proposed Groundwater Supply Project scenario is initiated at the same time as the Proposed Groundwater Supply Project.

Except for the occasional wet year, Baselines 2 and 3 simulate relatively stable levels of annual discharge in the San Luis Rey River. Under Baseline 2 the typical discharge is approximately 3,500 afy, and under Baseline 3 the typical discharge is approximately 1,500 afy. The differences in discharge are because Baseline 2 includes imported water return flows from RMWD, FPUD and VCMWD, while Baseline 3 includes imported water return flows from only FPUD and VCMWD. Comparison of Baselines 1, 2 and 3 shows that imported water return flows are the source of the increased river discharges. These discharges are captured by groundwater pumping under the Proposed Groundwater Supply Project, but not to the extent that river flow is reduced to conditions existing prior to importation of water by RMWD, FPUD and VCMWD. In other words, river discharge is enhanced by imported water return flows even with the Proposed Groundwater Supply Project.

In summary, the typical simulated annual San Luis Rey River discharges (exclusive of occasional wet years) are:

- Baseline 1: <100 afy
- Baseline 2: 3,500 afy
- Baseline 3: 1,500 afy
- Proposed Groundwater Supply Project: 150 afy

Figure 5-15 shows simulated annual discharge at the upstream gauge location. Discharges are nearly identical under the three baselines and the Proposed Project scenario. This demonstrates that the Proposed Groundwater Supply Project has no effect on upstream flows.

5.2.4 Simulated Groundwater Pumping by Others

As discussed in Chapter 4 and above, preexisting groundwater pumping from the portion of the San Luis Rey River Groundwater Basin in the study area was assumed to continue at a gross value of 2,500 afy and a net value of 1,900 afy in the baseline and Proposed Groundwater Supply Project scenarios. These gross and net values are estimated to be representative of actual groundwater pumping. The discussion in the preceding sections demonstrates that this preexisting pumping can continue along with the Proposed Groundwater Supply Project, because groundwater levels are not significantly affected, except in the proximity of active wells, and stream discharge is maintained at levels higher than prior to importation of water by RMWD, FPUD and VCMWD.

5.2.5 Simulated Consumptive Use by Riparian Vegetation

Tables 5-1 through 5-3 shows the water budgets for the baseline and Proposed Groundwater Supply Project scenarios. The water budgets include the simulated annual consumptive use by riparian vegetation in the portion of the San Luis Rey River Groundwater Basin in the study area. The average annual consumptive use was as follows:

- Baseline 1: 3,093 afy
- Baseline 2: 4,610 afy
- Baseline 3: 4,351 afy
- Proposed Groundwater Supply Project: 2,773 afy

These results show that imported water return flows have resulted in expansion of riparian consumptive use, and the Proposed Groundwater Supply Project reduces the consumptive use of imported water return flows by riparian vegetation.

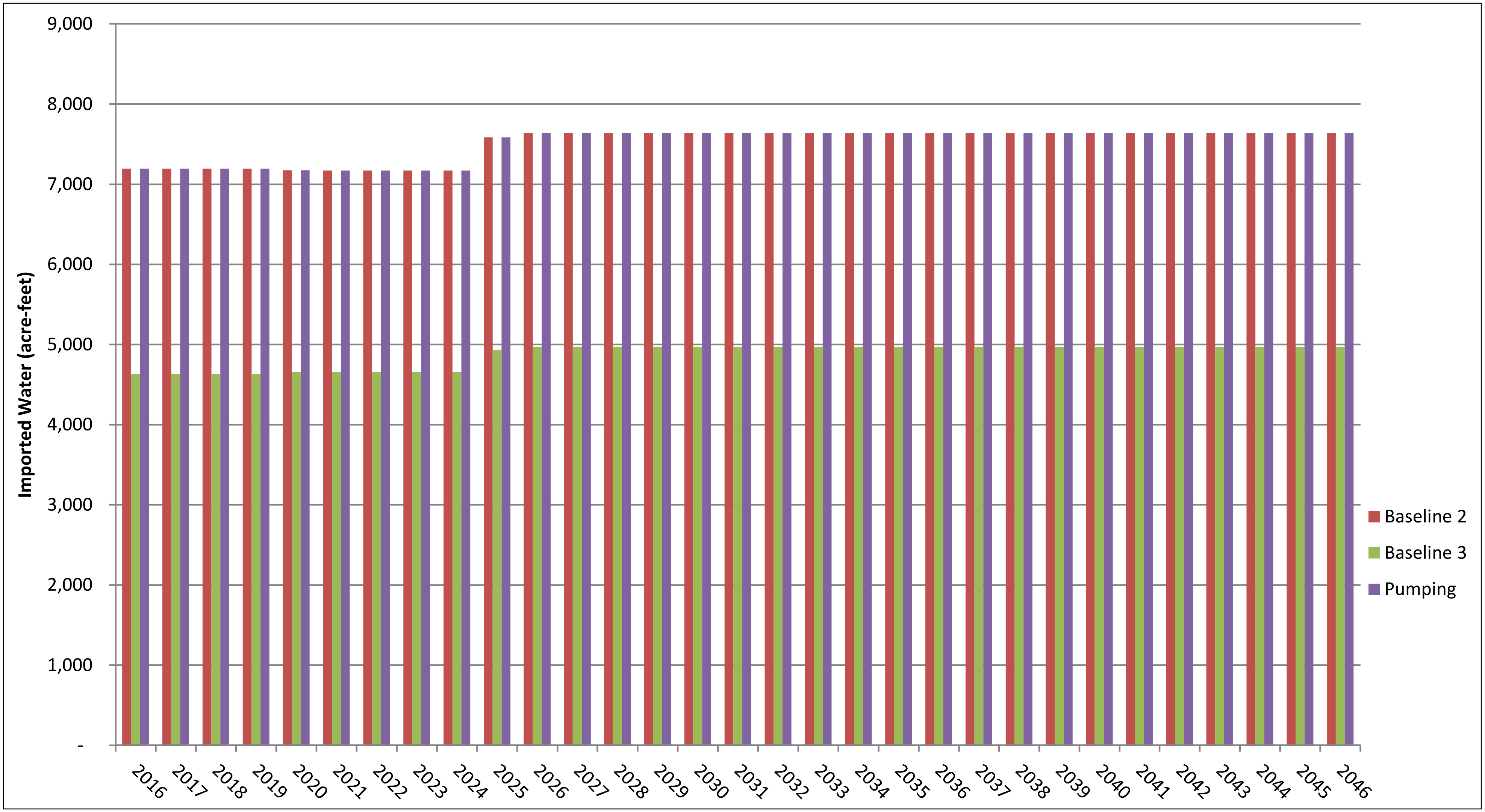
Based on the typical values listed above, the Proposed Groundwater Supply Project appears to result in a slight decrease in riparian consumptive use relative to conditions prior to the importation of water. However, comparison of the annual riparian consumptive use values under Baseline 1 (Table 5-1) and the Proposed Groundwater Supply Project (Table 5-4) scenarios show that the initial conditions of the model again need to be considered. In the Proposed Groundwater Supply Project scenario, imported water return flows are not expressed as recharge in the alluvial aquifer during the initial years of the simulation, because these flows need to travel through the fractured bedrock aquifer system. During this initial period, riparian consumptive use is reduced relative to Baseline 1. However, in the later years of the simulation, recharge from imported water return flows has reached the alluvial aquifer and riparian consumptive use is similar at about 2,500-2,600 afy in the both Baseline 1 and the Proposed Groundwater Supply Project (Tables 5-1 and 5-4).

5.2.6 Summary of Model Results

The model results demonstrate that typical volumes up to 5,700 afy, or 460 afm, can be pumped by the Proposed Groundwater Supply Project without significantly affecting the hydrology of the San Luis Rey Valley stream-aquifer system relative to conditions that would exist in the absence of imported water return flows, except near active pumping wells where groundwater level declines of up to 20 feet were simulated. Actual drawdowns near active wells will depend on the distance between the well and the groundwater level measurement point and will need to be assessed on a well-site-specific basis.

A capacity of 4,000 afy or 333 afm was selected for preliminary design purposes. As described in Section 5.2.1, these pumping rates are slightly less than evaluated in the model, because the modeled rates include some very wet years when simulated pumping rates were higher than would be implemented in the Proposed Groundwater Supply Project.

The impacts on the water quality were not evaluated in the modeling, but water quality would either be unchanged or improved with the project. This is because project pumping would create storage capacity in the aquifer, which would be available to accept recharge from precipitation. Without the project, this storage would not be available due to the aquifer being more frequently filled to capacity with imported water. The water quality improvement would occur because recharge from precipitation has much lower salinity then recharge from imported water return flows.



Notes:
1. Baseline 1 assumes no imported water return flows to the study area and is therefore not graphed.
2. Baseline 2 and the pumping scenario assume imported water return flows from RMWD, FPUD and VCMWD.
3. Baseline 3 assumes imported water return flows from FPUD and VCMWD.
4. The Proposed Groundwater Project assumes imported water return flows from RMWD, FPUD and VCMWD.

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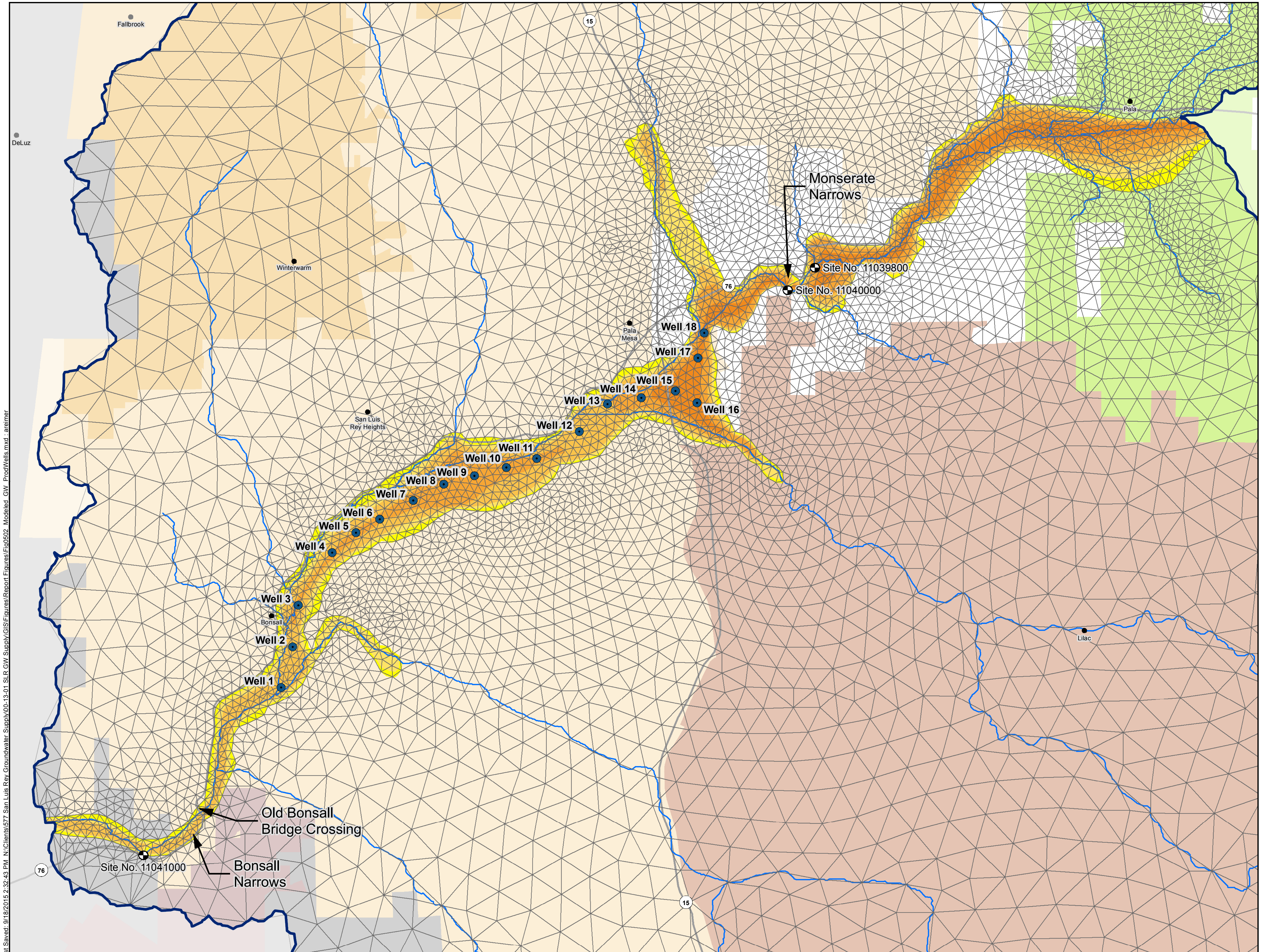
Figure 5-1

Projected Imported Water Return Flows

Rainbow Municipal Water District
Groundwater Supply Study



Notes:

1. Delineation of the San Luis Rey Valley Basin is based on DWR Bulletin 118 Groundwater Basin 9-7.
2. Subbasin delineations are based on descriptions provided in the San Diego County Basins report (2007) and the USGS Hydrologic and Salt Balance Investigations Utilizing Digital Models report (USGS WRIR 24-74).

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Figure 5-2

Modeled Groundwater Production Wells

Rainbow Municipal Water District
San Luis Rey Groundwater Supply

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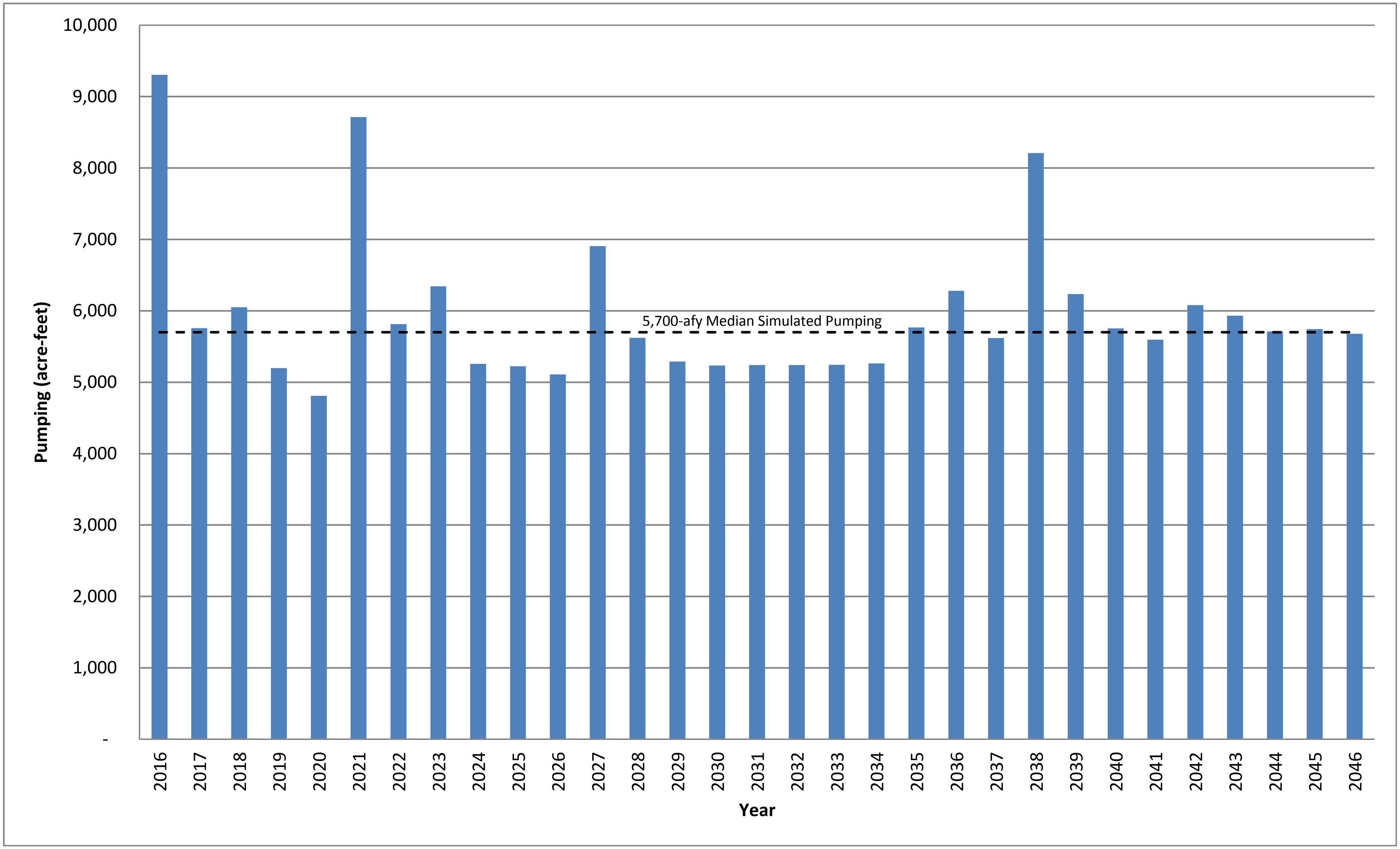

Figure 5-3

Simulated Potential Annual Pumping

Rainbow Municipal Water District
Groundwater Supply Study

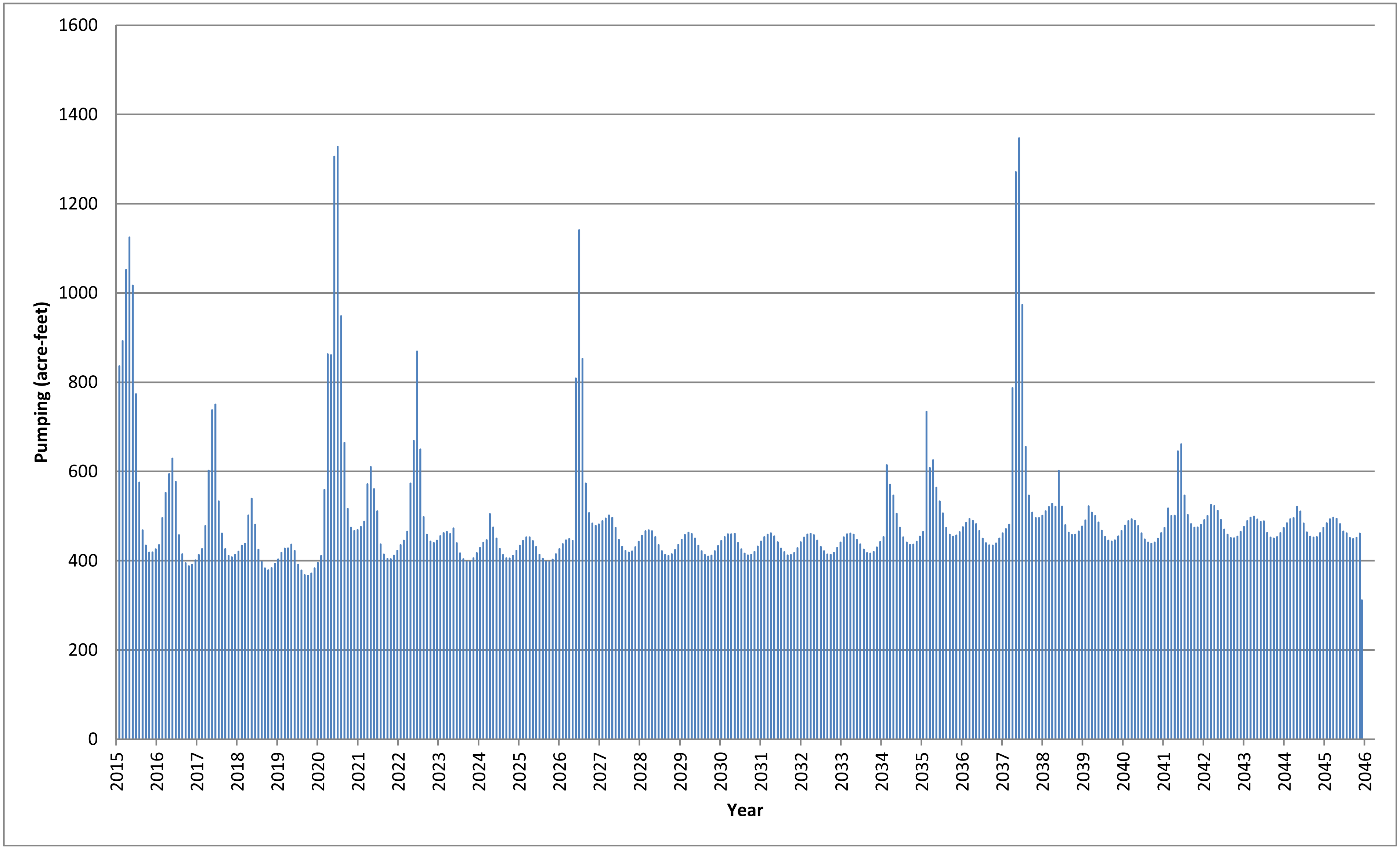

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Figure 5-4

Simulated Potential Monthly Pumping

Rainbow Municipal Water District
Groundwater Supply Study

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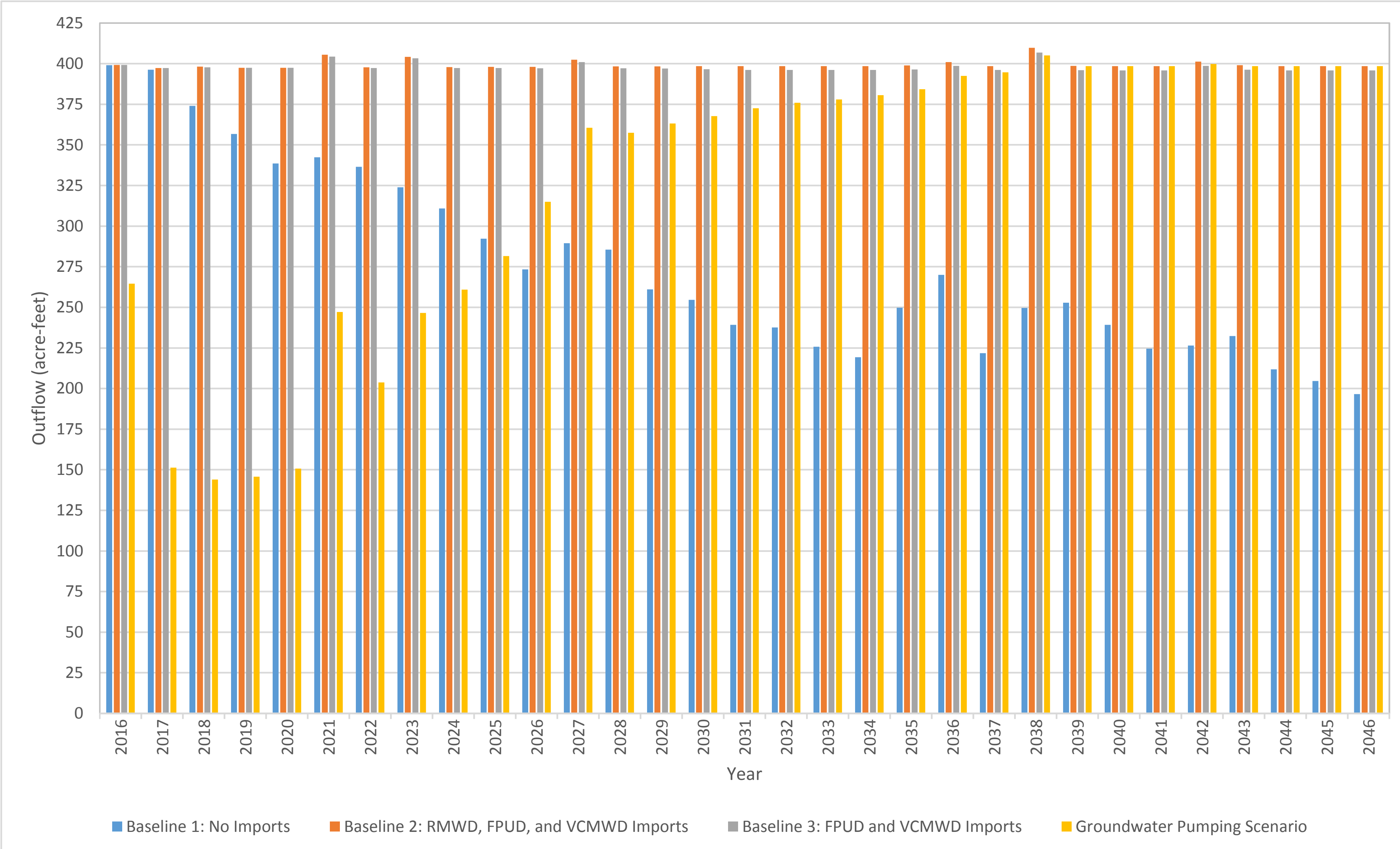
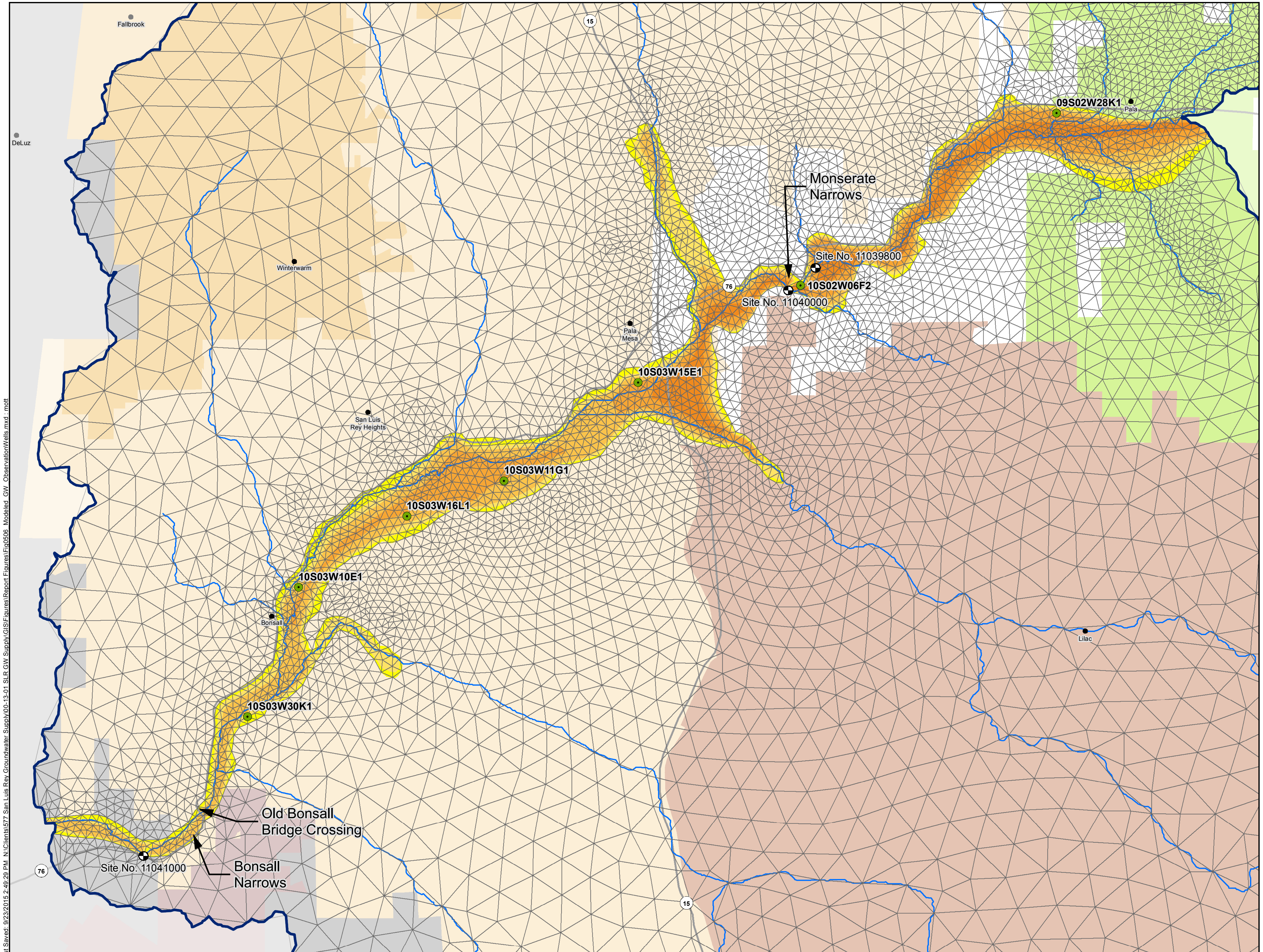


Figure 5-5

Alluvial Aquifer Groundwater Outflow

Rainbow Municipal Water District
Groundwater Supply Study



N

0 2,500 5,000
Scale in Feet

Symbology

- Groundwater Observation Wells
- USGS Stream Gauge
- City or Township
- Modeled Tributaries
- Study Area Boundary
- Model Elements
- County Boundary

San Diego County Water Districts

- Rainbow Municipal Water District
- Fallbrook Public Utility District
- Valley Center Municipal Water District
- Vista Irrigation District
- Other Water District
- Indian Reservation

Alluvial Thickness (feet)

- 1 - 20
- 20 - 40
- 40 - 60
- 60 - 80
- 80 - 100

Notes:
1. Delineation of the San Luis Rey Valley Basin is based on DWR Bulletin 118 Groundwater Basin 9-7.
2. Subbasin delineations are based on descriptions provided in the San Diego County Basins report (2007) and the USGS Hydrologic and Salt Balance Investigations Utilizing Digital Models report (USGS WRIR 24-74).

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Figure 5-6

Modeled Groundwater Observation Wells

Rainbow Municipal Water District
San Luis Rey Groundwater Supply

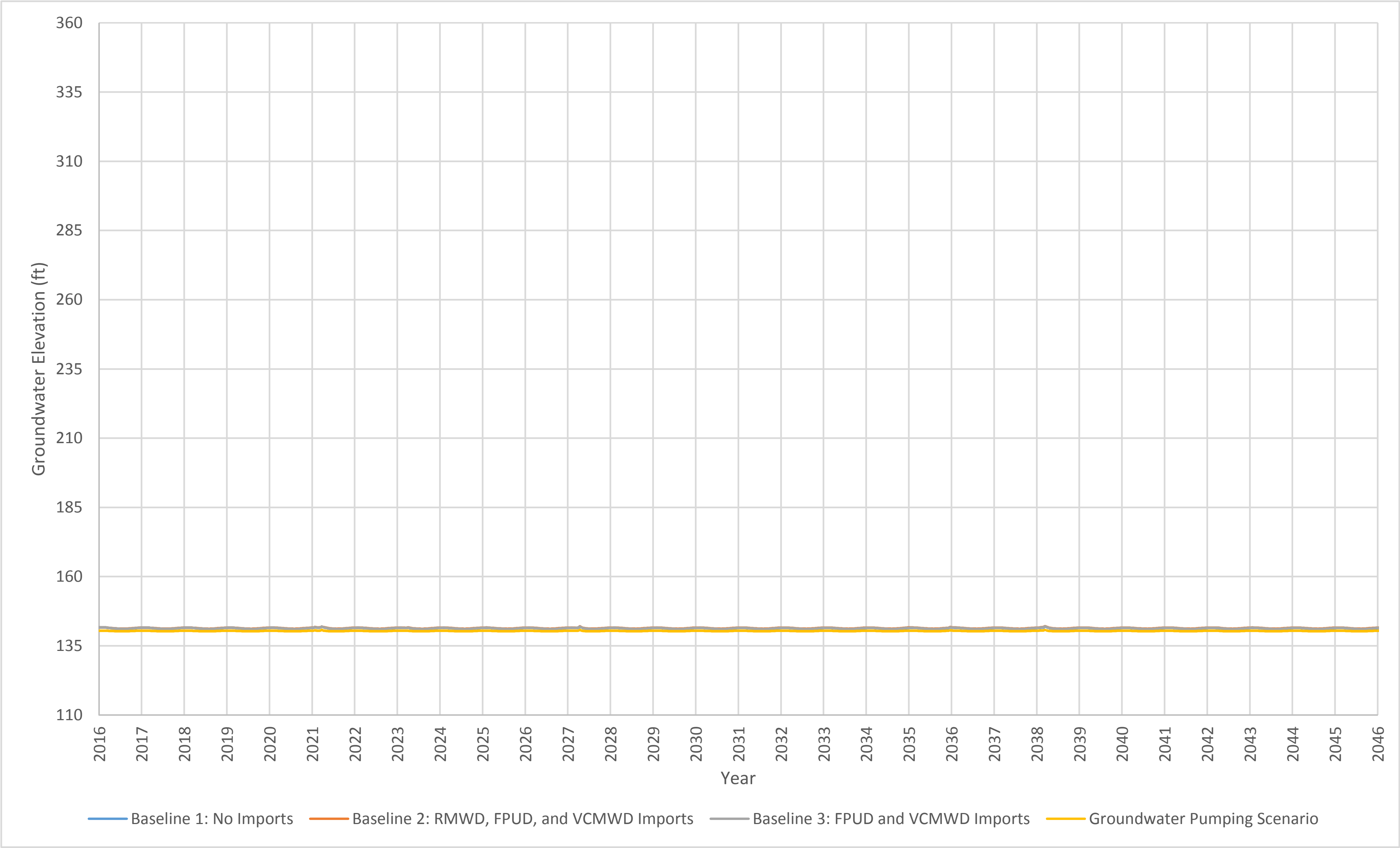

Figure 5-7

Groundwater Elevation for Observation Well 10S03W30K1

Rainbow Municipal Water District
Groundwater Supply Study

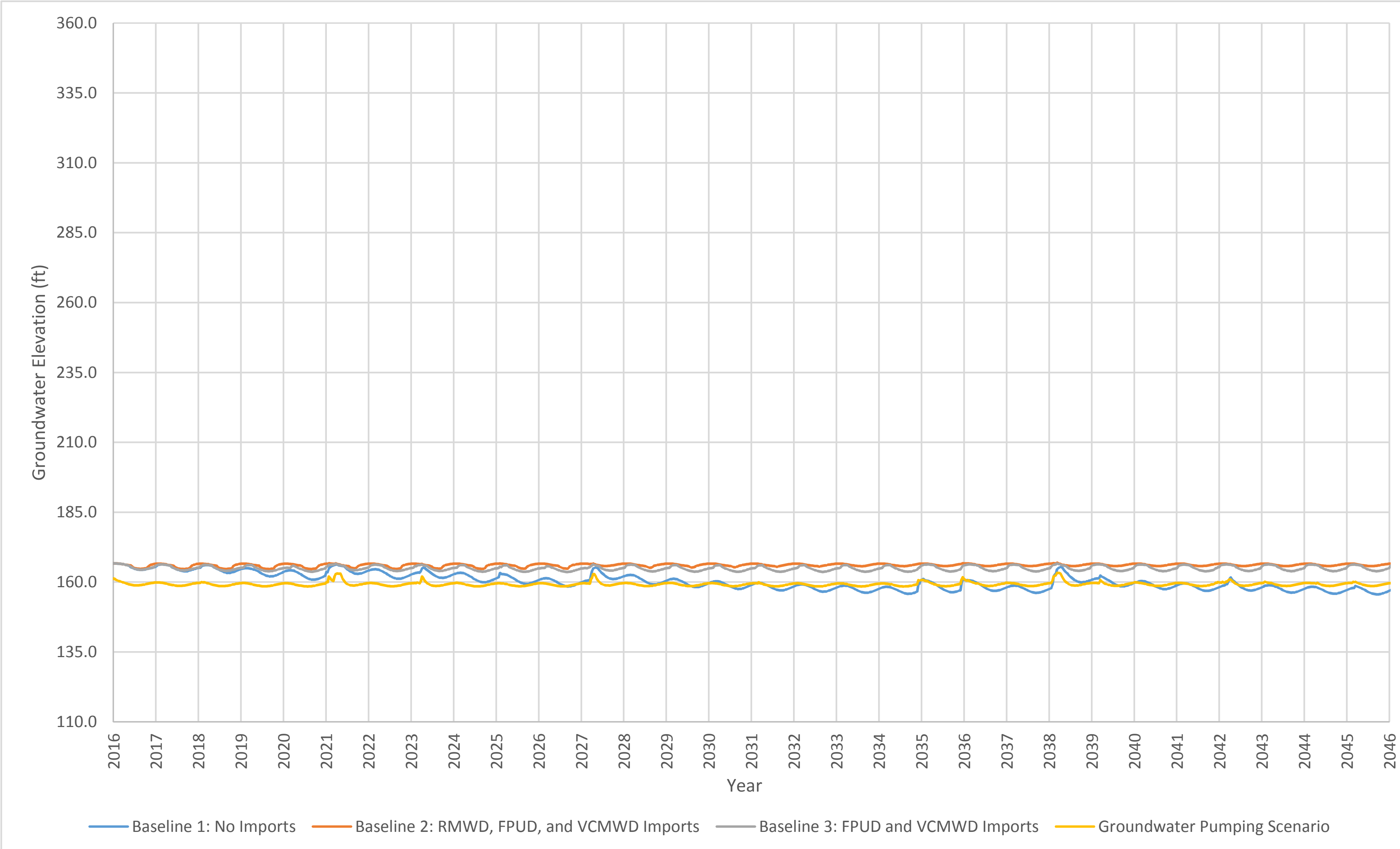

Figure 5-8

Groundwater Elevation for Observation Well 10S03W20E1

Rainbow Municipal Water District
Groundwater Supply Study

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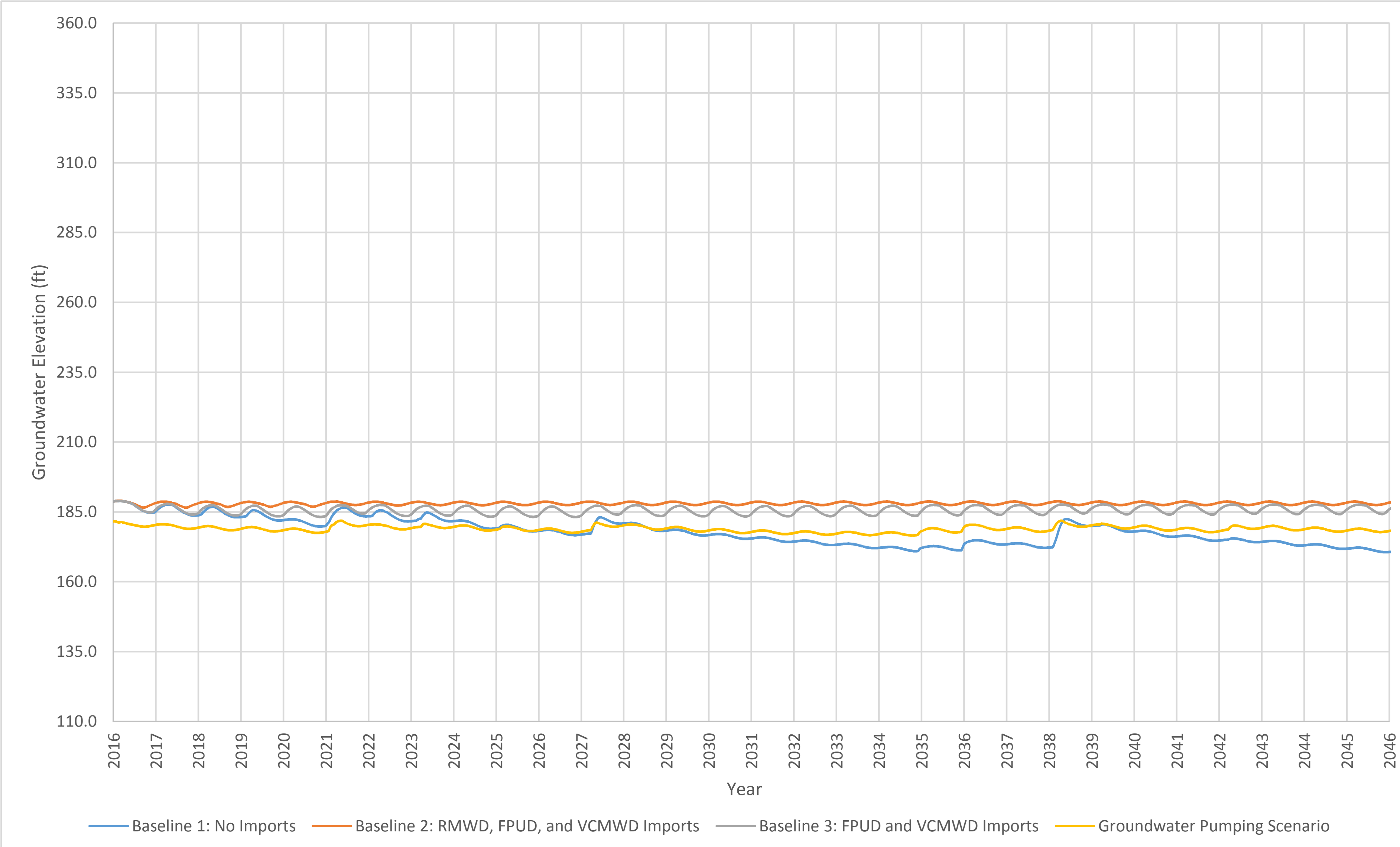

Figure 5-9

Groundwater Elevation for Observation Well 10S03W16L1

Rainbow Municipal Water District
Groundwater Supply Study

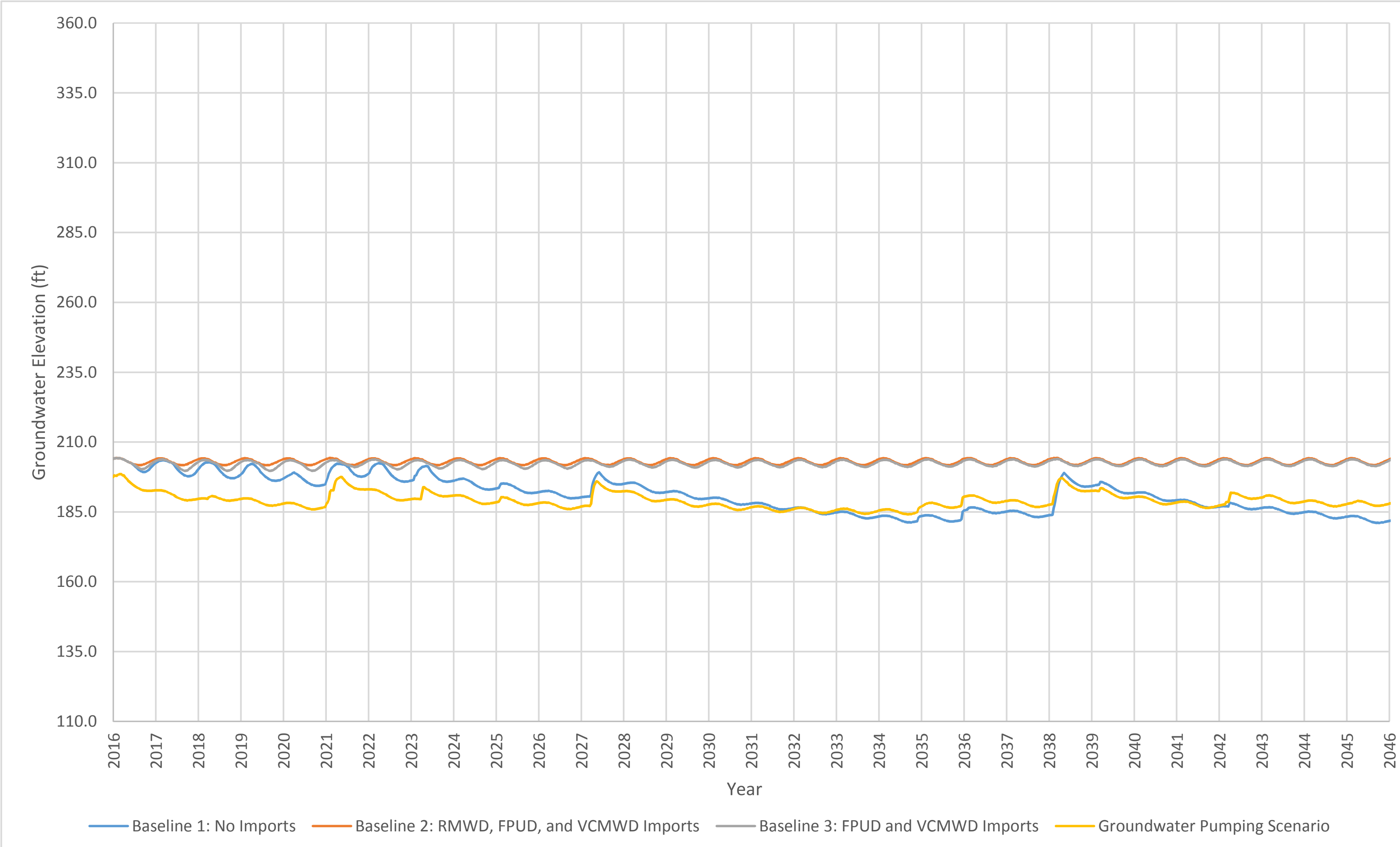

Figure 5-10

Groundwater Elevation for Observation Well 10S03W11G1

Rainbow Municipal Water District
Groundwater Supply Study

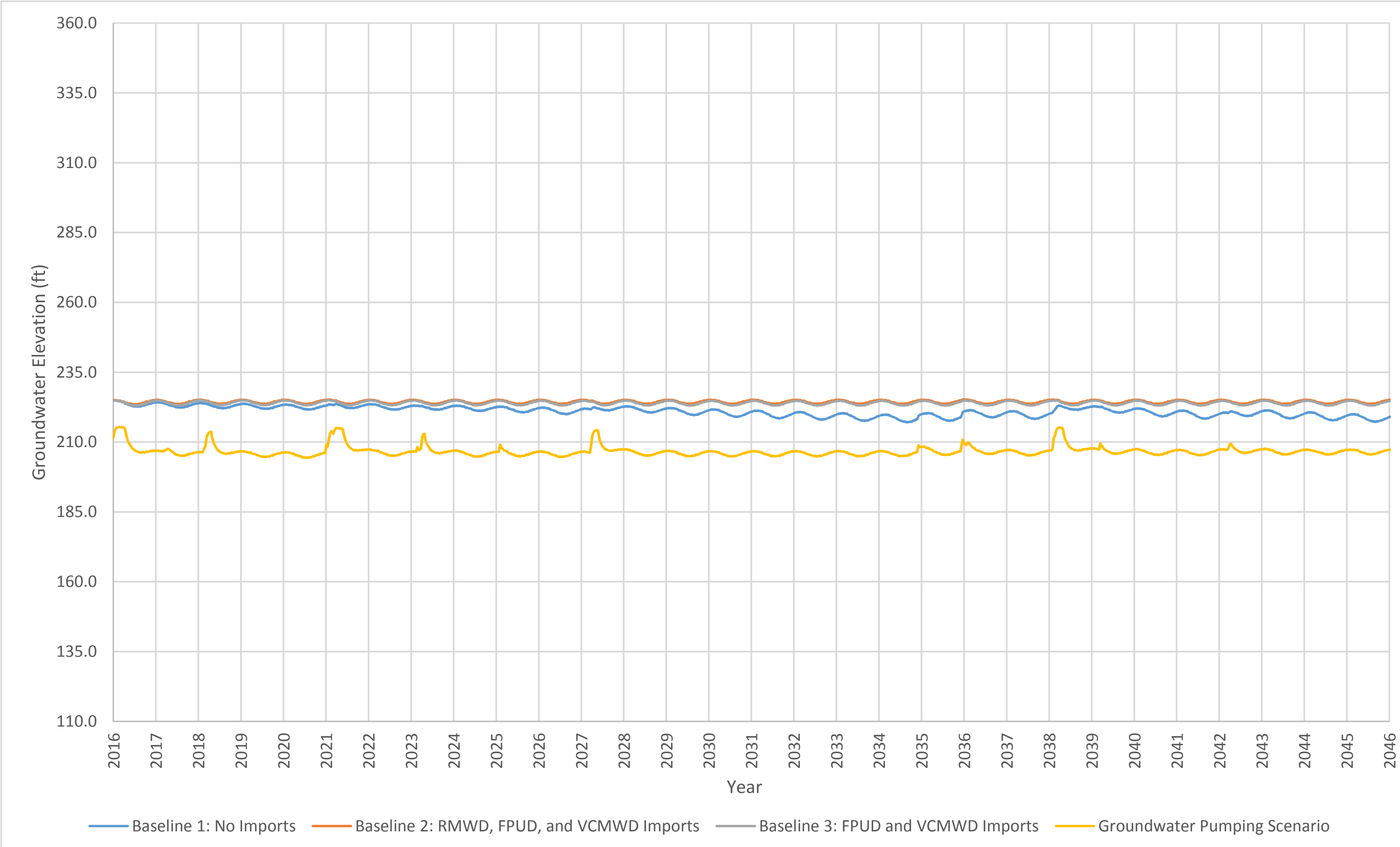

Figure 5-11

Groundwater Elevation for Observation Well 10S03W15E1

Rainbow Municipal Water District
Groundwater Supply Study

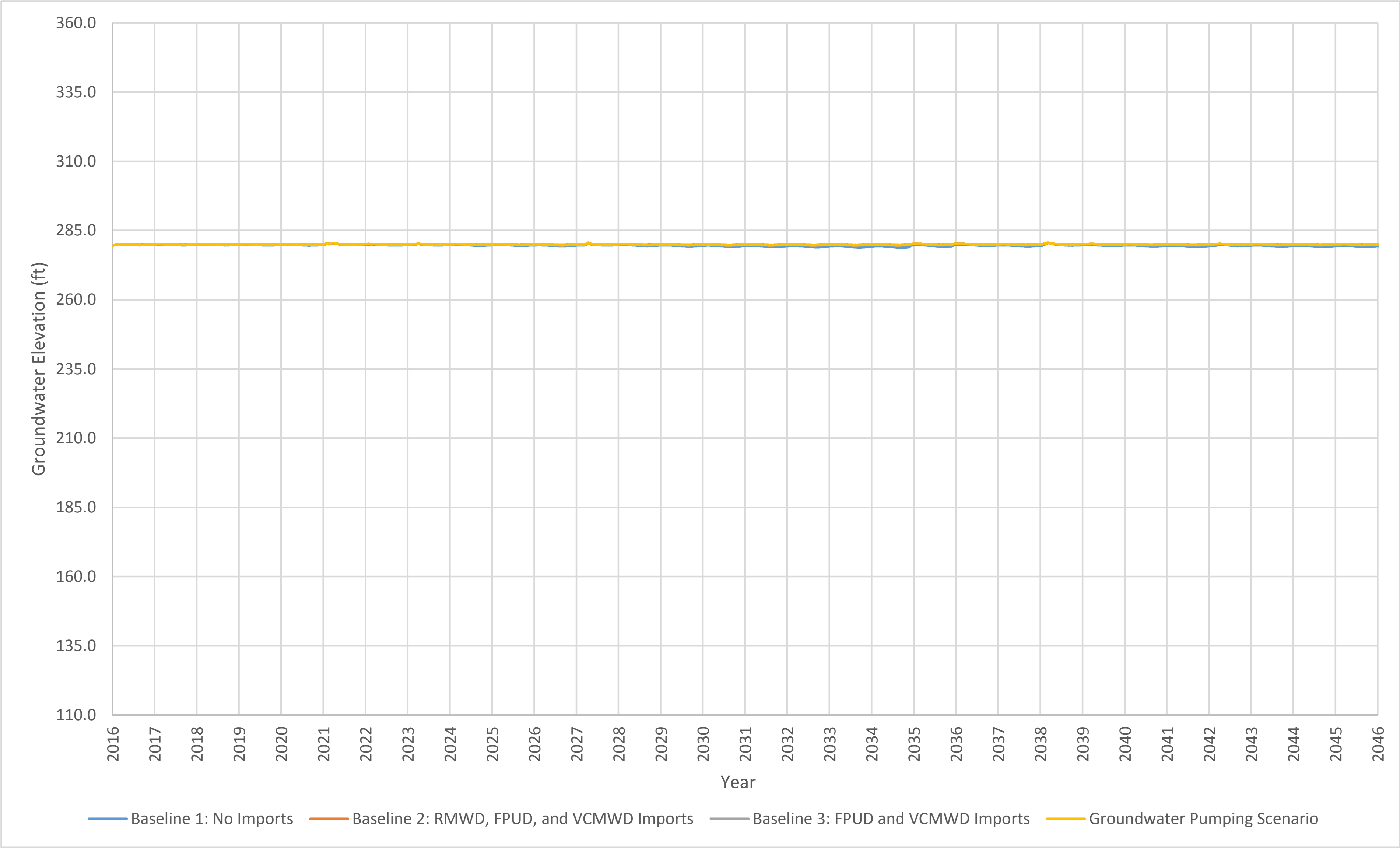

Figure 5-12

Groundwater Elevation for Observation Well 10S02W06F2

Rainbow Municipal Water District
Groundwater Supply Study

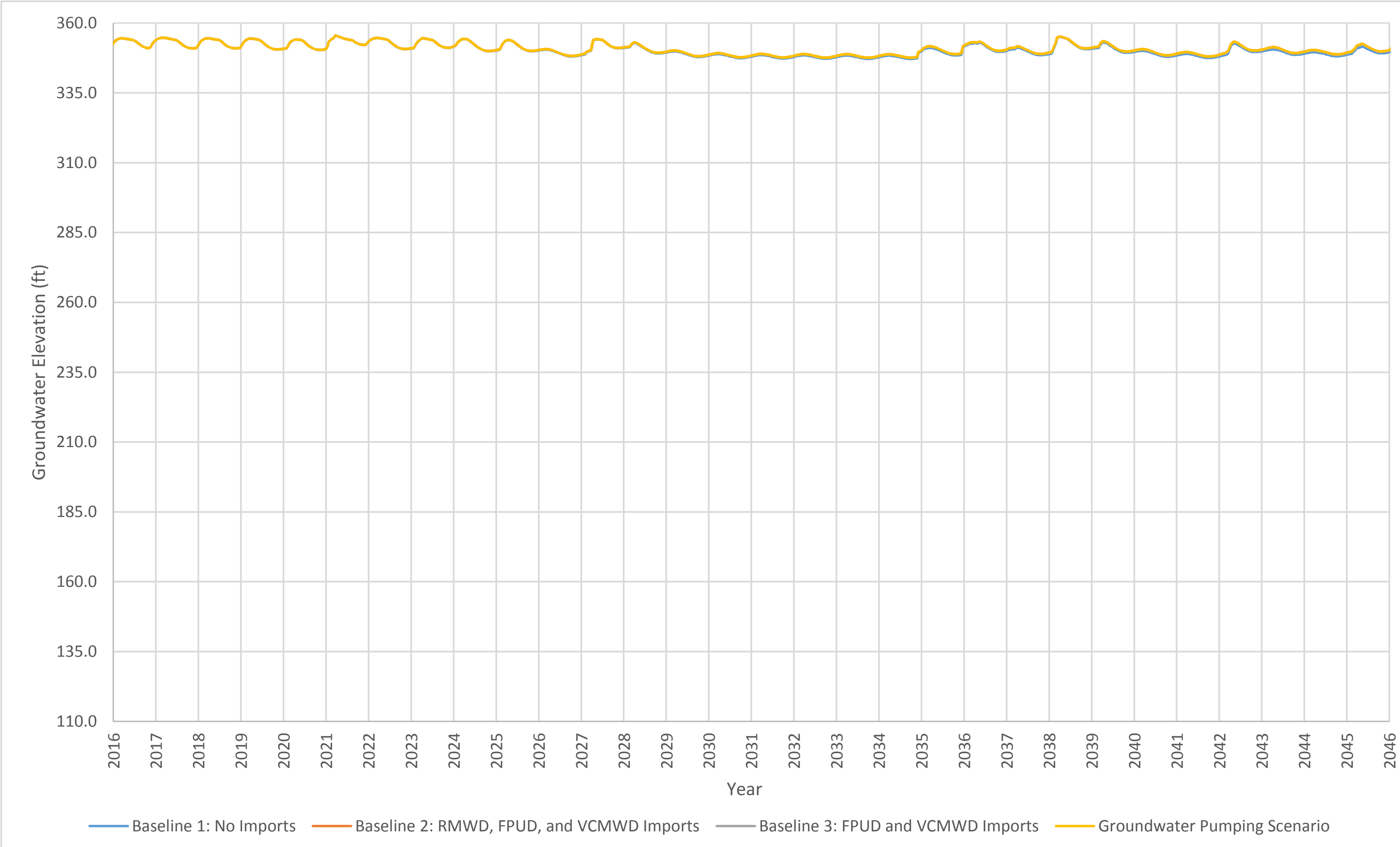

Figure 5-13

Groundwater Elevation for Observation Well 09S02W28K1

Rainbow Municipal Water District
Groundwater Supply Study

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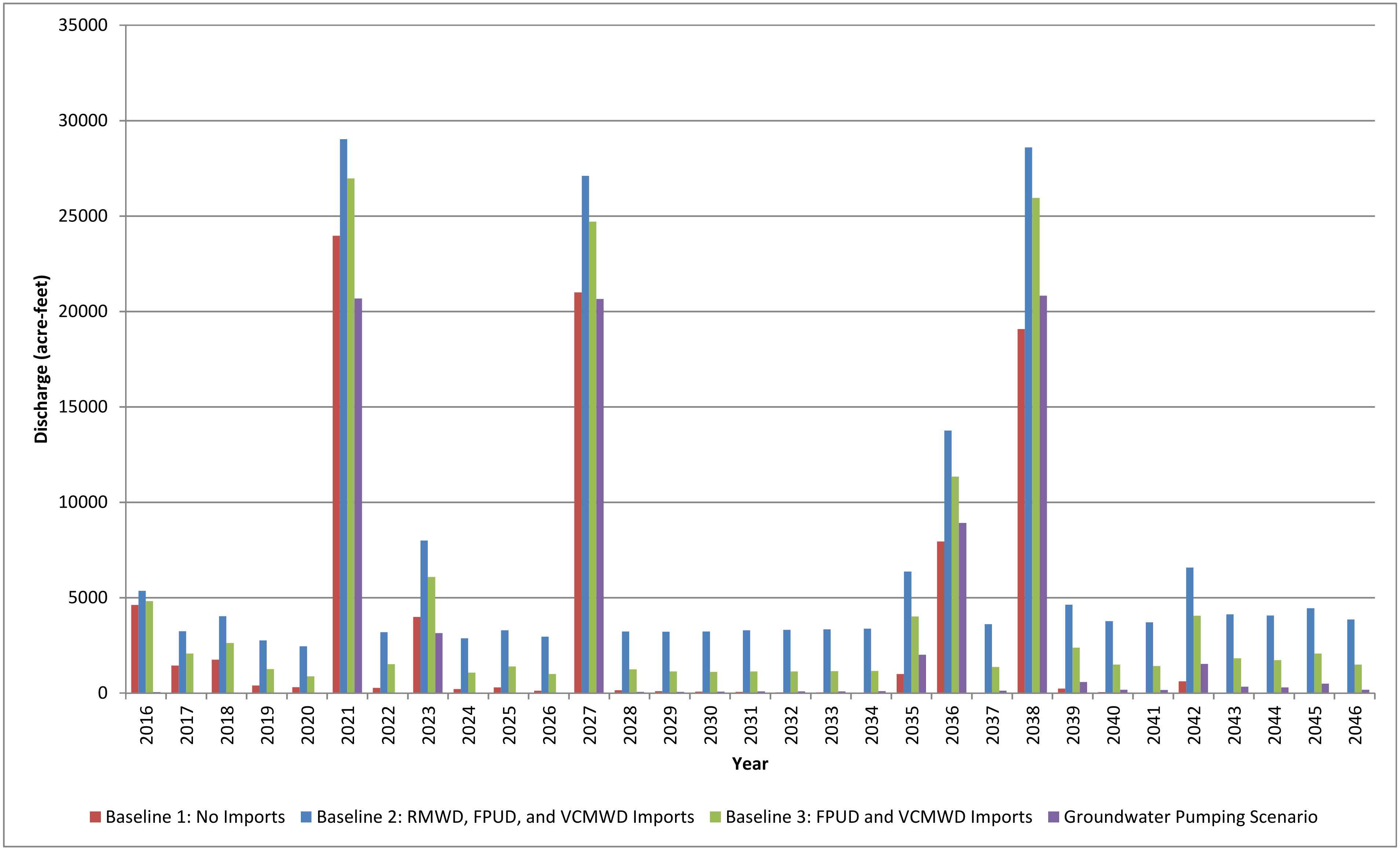

Figure 5-14

Simulated Annual San Luis Rey River Discharge: 1104100 Downstream

Rainbow Municipal Water District
Groundwater Supply Study

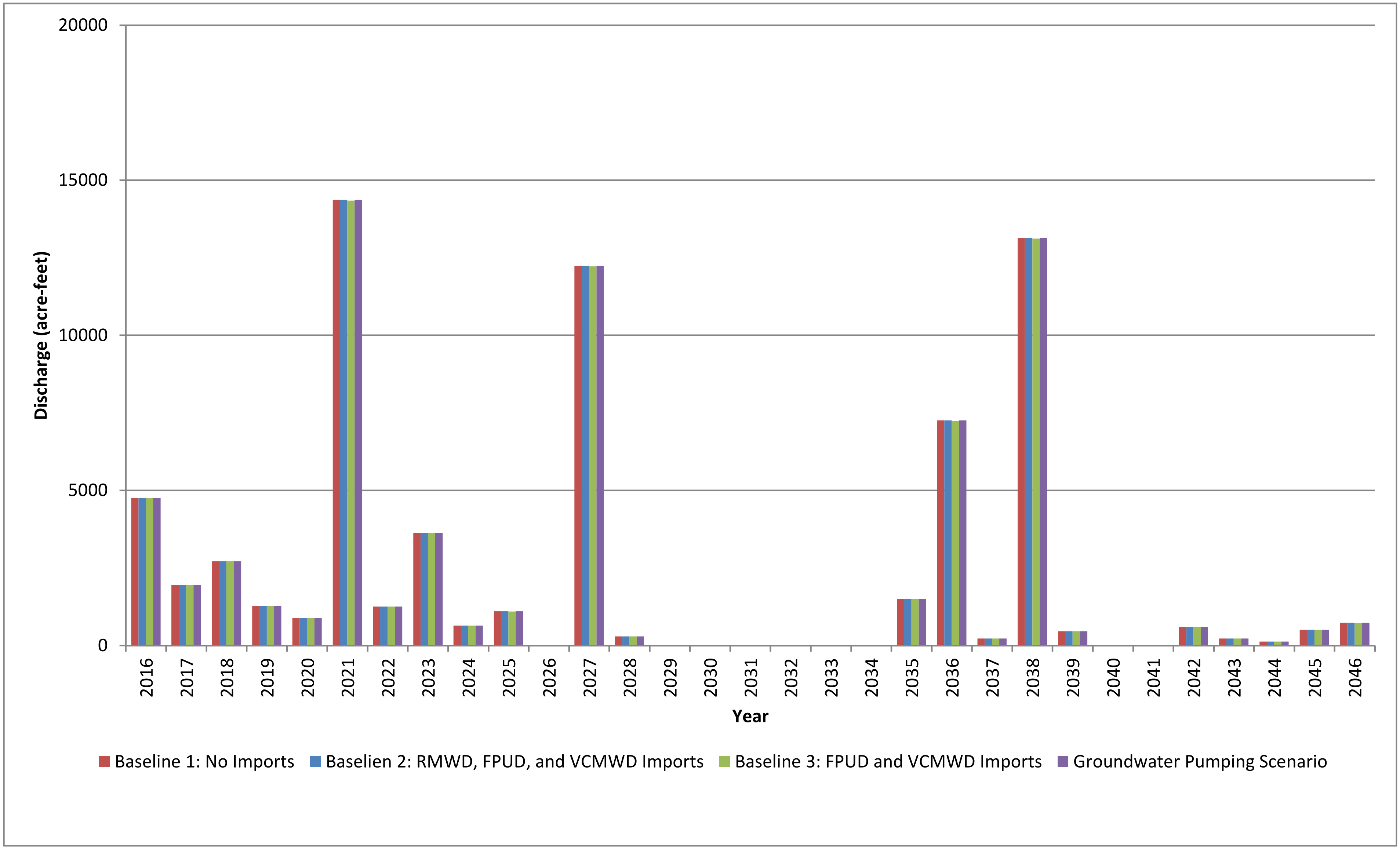

Figure 5-15

Simulated Annual San Luis Rey River Discharge: 1104000 Upstream

Rainbow Municipal Water District
Groundwater Supply Study

6.1 WATER RIGHTS OVERVIEW

This overview provides a basic understanding of the California water rights laws upon which the project legal analysis is predicated. The focused review of the conclusions from the hydrologic analysis presented in Chapter 5 is provided in Section 6.2 below.[1]

6.1.1 Surface Water Rights

There are two principal types of surface water rights recognized under California law: riparian rights and appropriative rights. In addition, there are several other types of water rights that may or will come into play; amongst them are return flow rights and spring water rights.

6.1.1.1 Riparian Water Rights

Generally, riparian rights authorize the diversion and use of water from a stream on land that is contiguous to the stream and located within the watershed of the stream. (Pleasant Valley Canal Co. v. Borror (1998) 61 Cal.App.4th 742, 774-775.) Riparian rights are limited to the natural flow of the stream, and do not authorize the diversion of "foreign water" that would not be present in the stream under natural conditions. (Bloss v. Rahilly (1940) 16 Cal.2d 70, 75-76.) In addition, water may not be stored from season to season under a riparian right. (City of Lodi v. East Bay Municipal Utility Dist. (1937) 7 Cal.2d 316, 335.) This is commonly referred to as "true" storage. Only temporary, or "regulatory" storage is allowed.

In most circumstances, a riparian right attaches only to the smallest parcel held under one title in the chain of title leading to the present owner. (Pleasant Valley Canal Co. v. Borror, supra, 61 Cal.App.4th at pp. 774-775.) When a riparian parcel is subdivided, such that a parcel is no longer contiguous to the stream, the riparian right formerly attached to the noncontiguous parcel is lost, absent proof of intent to retain the riparian right. (Anaheim Union Water Co. v. Fuller (1907) 150 Cal. 327, 331.) Such intent can be demonstrated by the persistence of a water distribution system that continues to deliver riparian water to the otherwise severed parcel, statements in a conveyancing document, and other mechanisms. If a riparian right is lost by severance, it cannot be resurrected by merging the parcels under common ownership or into a single parcel. (Ibid.)

Relative to other rights, riparian rights share in shortages of water on a correlative basis. When the natural flow of a stream is insufficient to satisfy all the riparian rights to use the waters of the stream, the riparian right holders must reduce their diversions proportionately. (Prather v. Hoberg (1994) 24 Cal.2d 549, 560.) Relative to an appropriative right, a riparian right has a priority date based on when the riparian parcel was patented. (Pleasant Valley Canal Co. v. Borror, supra, 61 Cal.App.4th at p. 774.)

[1] This chapter is for the purposes of this project, and no other purpose. The legal analysis relies upon and assumes the validity and sufficiency of the technical analysis.

Riparian rights are not lost through non-use. (In re Waters of Long Valley Creek Stream System (1979) 23 Cal.3d 339, 347, 358.) In other words, the water right holder may use the riparian right, and stop using it (or use a lesser amount) without losing or reducing the right itself. In one case, the State Water Resources Control Board (SWRCB) did subordinate the priority of unexercised riparian rights relative to otherwise junior water rights in the context of a stream adjudication (Id. at pp. 358-359.)

Municipalities and districts do not typically exercise riparian rights to provide water service, since such entities do not own or stand in the shoes of the owners of the riparian land upon which the water is to be used. Such entities can provide water to riparian lands they own, and there is a possibility that they can provide water service under a so-called "agency agreement" with riparian landowners[2]. The concept is that the water service provider is acting as an agent of the water right holder in diverting (or extracting) and delivering water to the land to be served, and to which the water right attaches. This is done in Rancho California Water District for customers' riparian and/or overlying groundwater rights.

If diversion under a riparian right is undertaken, a Statement of Water Diversion and Use must be filed with the SWRCB.

6.1.1.1.1 Riparian Property Owned by RMWD or its Customers

Property that vertically overlies the water interconnected with the San Luis Rey River may have riparian (assuming surface water classification) or overlying (assuming groundwater classification) rights. We discuss riparian rights here due to the above-described SWRCB classification. Hydrologic and chain of title information is necessary to assure "riparianism".

RMWD's property including that obtained from Sumac Mutual Water Company is likely to fall within this category. RMWD's ability to exercise its riparian rights would be somewhat limited, as described above. However, some of RMWD's larger customers may also have riparian rights, which RMWD may be able to exercise to make deliveries to those customers under an "agency" arrangement.

Riparian rights have disadvantages – constraints on the location and nature of use, for example – but they are also typically senior, existing rights.

6.1.1.2 Appropriative Water Rights

Appropriative rights are acquired by diverting water from a stream and applying it to beneficial use. Appropriative rights do not require use on land adjacent to the watercourse. They may authorize the use of water at locations distant from the watercourse, including outside of the watershed. (Crandell v. Woods (1857) 8 Cal. 136, 142; Miller v. Bay Cities Water Co. (1910) 157 Cal. 256, 280-281.)

[2] We have not yet located a judicial decision affirming this type of arrangement.

Appropriative rights are typically junior in priority to riparian rights. The maxim "first in time, first in right," governs the priority of appropriative rights as between themselves. The priority of an appropriative right is based on the date when the development of the right was initiated. When the stream flow is not sufficient to satisfy all of the appropriative rights, senior appropriators are entitled to fully satisfy their rights before junior appropriators may divert water under their rights. (City of Pasadena v. City of Alhambra (1949) 33 Cal.2d 908, 926.)

Unlike riparian rights, appropriative rights are not limited to the natural flow of the stream, water may be stored from season to season under an appropriative right, and the place of use is not limited to riparian land. (Bloss v. Rahilly, supra, 16 Cal.2d at pp. 75-76; City of Lodi v. East Bay Mun. Utility Dist., supra, 7 Cal.2d at p. 335.) The point of diversion, place of use, or purpose of use of an appropriative right may be changed, provided that the change does not amount to the initiation of a new water right, or result in injury to any other legal user of water. (Wat. Code, §§ 1701, 1702, 1706 [changes permissible subject to no injury rule]; Senior v. Anderson (1896) 115 Cal. 496, 501-504 [change in place of use permissible but appropriative right limited in quantity to amount of water used to irrigate original place of use].)

Before December 19, 1914[3], an appropriative right could be obtained in two different ways: by non-statutory and statutory appropriations. The non-statutory method entailed simply diverting water and applying it to beneficial use, after having made some sort of objective manifestation of the intent to appropriate the water. (See Nevada County & Sacramento Canal Co. v. Kidd (1869) 37 Cal. 282, 311-312.) The statutory method of obtaining a pre-1914 appropriative right entailed following the requirements of Civil Code sections 1410 through 1422, which were enacted in 1872. Civil Code section 1415 required the posting and recording of a notice that contained specified information about a proposed appropriation. Civil Code section 1416 required construction of the diversion works to be commenced within 60 days of posting the notice, and required the work to be conducted and completed with diligence.

Since December 19, 1914, obtaining a water right permit from the State Water Board (or its predecessor agency) pursuant to division 2 (commencing with section 1000) of the Water Code has been the exclusive means to acquire an appropriative water right. (Wat. Code § 1225; People v. Shirokow (1980) 26 Cal.3d 301, 308-309.)

Both pre-1914 and post-1914 appropriative rights are perfected by applying water to reasonable and beneficial use. The right is quantified as the amount of water actually applied to reasonable, beneficial use, not the amount of water listed in a notice of appropriation, the capacity of an appropriator's diversion works, the amount of water actually diverted, or the amount of water authorized to be diverted in a water right permit. (Haight v. Costanich (1920) 184 Cal. 426, 431; Trimble v. Heller (1913) 23 Cal.App. 436, 443-444; Akin v. Spencer (1937) 21 Cal.App.2d 325, 328; Wat. Code, §§ 1240, 1390, 1610.)

Appropriative rights must be developed and exercised with due diligence. (Maeris v. Bicknell (1857) 7 Cal. 261, 263; Wat. Code, §§ 1395, 1396, 1397; Cal. Code Regs., tit.23, § 840.) Under the doctrine of progressive use and development, the development of an appropriative right that

[3] This is the effective date of the Water Commission Act.

was initiated before December 14, 1914, may be completed after that date without obtaining a water right permit, provided that any increase in the diversion and use of water after December 14, 1914, is within the scope of the original plan of development, and the plan is carried out with due diligence. (Haight v. Costanich, supra, 184 Cal. at pp. 431-433.)

Appropriative rights are subject to forfeiture in whole or in part if water is not used under the right for a five-year period. (Smith v. Hawkins (1898) 110 Cal. 122, 1127-128; Erickson v. Queen Valley Ranch Co. (1971) 22 Cal.App.3d 578, 582.) The rules regarding forfeiture of pre-1914 appropriative water rights were recently restated in Millview County Water District v State Water Resources Control Board (2014).

6.1.2 Groundwater

California law recognizes essentially two categories of subsurface water: percolating groundwater and subterranean streams.

6.1.2.1 True or Percolating Groundwater

True, or "percolating" groundwater travels in a diffused manner through the substrata. Percolating groundwater is not subject to the SWRCB's permitting jurisdiction. Each pumper of groundwater for use on lands overlying the groundwater basin has an equal or "correlative" right to use groundwater for the enjoyment of his overlying land. Like riparians, the overlying users must share any shortage of supply proportionately. In this respect, the date of commencement of pumping is irrelevant.

California courts have not clearly defined how the boundaries of a groundwater basin are determined, or what constitutes overlying lands. In City of Pasadena v. City of Alhambra (1949) 33 Cal.2d 908, 925, cert. denied 339 U.S. 927 (1950), the court stated that "an overlying right ... is the right of the owner to take water from the ground underneath for use on his land within the basin or watershed ... " This statement implies that use on lands within the basin watershed is a legitimate overlying use, in addition to use on lands actually overlying the basin. Groundwater extracted over one part of a basin can be used in another part of the basin, as long as the place of use overlies the basin.[4]

Use of groundwater on non-overlying lands constitutes an "appropriation" of groundwater. Groundwater appropriators are entitled to use all of the surplus available after the overlying users' rights are satisfied. In the event of a shortage or overdraft of the groundwater basin, overlying users have priority over appropriative rights absent prescription. Katz v. Walkinshaw (1903) 141 Cal. 116, 135-136. The general rule that overlying users have priority over non-overlying users is altered where appropriators obtain prescriptive rights to non-surplus water. Prescriptive rights may be created by establishing that the diversion of non-surplus groundwater was actual, open and

[4] Certain uses on overlying land are considered non-overlying uses per se. Public use of percolating groundwater, even on overlying land, is generally not a proper overlying use. City of San Bernardino v. City of Riverside (1921) 186 Cal. 7, 24. A municipality or public entity may exercise an overlying right only to the extent that it is serving overlying land it owns, and possibly overlying landowners from whom the public entity acquired the right. City of San Bernardino, supra, 186 Cal. at 25-26. (See discussion of agency agreements, above.)

notorious, hostile and adverse to the overlying user, continuous and uninterrupted for five years, and under a claim of right. Prescription does not lie against public agencies.

Municipalities and districts are typically considered to be exercising appropriative rights to groundwater when delivering water to their customers, even if those customers are located on overlying land (unless the public agency owns the land.) This rule may be varied where the public agency acts as the agent of the customer-landowner who holds overlying water rights. Refer to Rancho California's form agency agreement, discussed.

6.1.2.2 Underground Water Classified as "Surface Water"

The second category of subsurface water is subterranean streams, which most commonly – but not necessarily – occurs as the subsurface flow of a surface stream.

The law applicable to subsurface stream flow (including underflow) is the same as that which applies to surface streams. Hanson v. McCue (1871) 42 Cal. 303, 308. For example, riparian rights extend to underground streams, Peabody v. City of Vallejo (1935) 2 Cal.2d 351, 375, and subsurface streams are subject to appropriation. Water Code § 1200.[5] Moreover, any right that a person has to a surface stream also extends to the stream's underflow, since both portions of the stream are considered to constitute one common supply. Rancho Santa Margarita v. Vail (1938) 11 Cal.2d 501, 555. Holders of riparian rights to underflow share the water supply on a pro rata basis with other underflow riparians, as well as with surface flow riparians.

There is a presumption in California law that groundwater is percolating groundwater, and any person asserting that groundwater is flowing in a known and defined underground stream has the burden of proof. Arroyo Ditch and Water Co. v. Baldwin (1909) 1 55 Cal. 280, 284.

In order for groundwater to be considered an underground stream, it must be "flowing through known and definite channels'. Water Code § 1200; accord, City of Pasadena v. City of Alhambra, 33 Cal.2d 908 at 934. The underground water course must be "contracted and bounded", City of Los Angeles v. Pomeroy (1899), 124 Cal. 597, 633, and must have a bed and banks, United States v. Fallbrook Public Utility Dist. (9th Cir. 1965) 347 F.2d 48, 56; with the bed consisting of any material that keeps the water from penetrating below a certain depth. City of Los Angeles v. Pomeroy, supra 124 Cal. At 623. Moreover, the underground stream must have a flow in connection with the surface flow and in the same general direction. Id. at 623-24. It should be noted, however, that the "subterranean stream" may extend a considerable distance, perhaps a mile or more, on either side of the trough in which the surface stream flows. Peabody v. City of Vallejo, supra, 2 Cal.2d at 375.

Whether particular groundwater is subject to the laws governing surface waters is typically established by evidence of a significant hydraulic connection between a well and the stream, such as the existence of a direct effect on the stream flow from pumping. Larsen v. Apollonio (1936) 5 Cal.2d 440, 443-444. The fact that a well is located in physical proximity to the surface stream and

[5] Water Code Section 1200 provides that "Whenever the terms stream, lake, or other body of water, or water occurs in relation to applications to appropriate water or permits or licenses issued pursuant to such applications, such term refers only to surface water, and to subterranean streams flowing through known and definite channels."

not at the fringes of an underground basin is also relevant. United States v. Fallbrook Public Utility Dist., supra, 34 7 F.2d at 56; City of San Bernardino v. City of Riverside (1921) 1 86 Cal. 7, 14. Similarly, the lack of a barrier such as a clay lense between what is clearly a subterranean flow connected with surface flow, and the well in question, may be relevant. Aguirre v. Fish & Game Commission (1957) 151 Cal.App.2d 469, 473.

The legal definition of percolating groundwater contrasts to that of underground stream flow. Percolating groundwater has been said to be water "combined with the earth, or passing through it, by percolation or filtration or chemical attraction" Cross v. Kitts (1886) 69 Cal. 217, 222. It is considered to be "part of the soil" rather than part of the body or flow of any surface or subterranean stream. Percolating waters may be rainwaters slowly infiltrating soil or water seeping from banks or the bed of a stream "which have so far left the bed and the other waters as to have lost their character as part of the flow." Vineland Irrigation Dist. v. Azusa Irrigating Co. (1899) 126 Cal. 486, 494.

This legal distinction ignores the hydrologic reality that virtually all surface and subsurface waters are connected, with variation in the nature and extent of that connection.[6] However, the legal fiction is very important from a regulatory standpoint. Hydrologic and geologic evaluation is generally necessary to determine which classification is most likely to adhere. The most common indication that the SWRCB will assert jurisdiction under Water Code Section 1200[7] is hydraulic connectivity such that pumping from an underground source is seen as significantly affecting surface flows.

6.1.2.2.1 Pauma Basin

In Decision 1645 (2002) regarding the San Luis Rey River, the SWRCB determined that extractions of water from the Pauma Basin were percolating ground water. In the same decision, it concluded that extractions from the downstream Pala Basin involved water flowing in a known and underground channel, and post-1914 appropriations require SWRCB permits. The earlier Decision 432 (1938) found that the underground waters further downstream in the Bonsall Basin were also legally "surface water".

6.1.2.2.2 Bonsall Basin

A portion of RMWD is within the San Luis Rey River's Bonsall Basin. A relatively recent SWRCB decision addressed the classification of water in basins upstream of Bonsall. That decision, Decision 645 (2002), also described the Bonsall Basin at page 3 as follows:

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[6] See, e.g., Review of the Laws Establishing the SWRCB's Permitting Authority Over Appropriations of Groundwater Classified as Subterranean Streams and the SWRCB's Implementation of those Laws, by Professor Joseph L. Sax, January 19, 2002.

[7] See, e.g., SWRCB Decision 432 (1938); Decision 968 (1960); Order WR 95- 10 (1995), and the 1999 draft decision in the Pauma and Pala case on the San Luis Rey River. This jurisdictional issue has often been highly controversial. In other locations, such as the Russian River, there has been extensive permitting of underground flow by the SWRCB, with limited objections.

> "Groundwater in the alluvial aquifer in the Bonsall Basin downstream of the Monserate Narrows was previously determined to be a subterranean stream flowing through known and definite channels (Decision 432 (1938)) of the Division of Water Resources of the State Department of Public Works (predecessor to the SWRCB), reaffirmed in Order of the State Water Rights Board dated June 26, 1962."

It concluded that the groundwater in the basin upstream of the Bonsall Basin, the Pala Basin, is a subterranean stream flowing through known and definite channels, and hence classified as surface water. (See pages 22 et. seq.)

The older Decision 432 referenced in Decision 645 specifically addresses the Bonsall Basin. It provides in part (pages 12-13):

> "A section at any point across the valley of the San Luis Rey River below Monserate narrows, except where the broad tributary canyons enter and make bays in the hills, would show the bedrock hills of granite or other material descending sharply to the trough and definitely marking the banks. In some places the width between the banks may not be more than a few hundred feet, in other places half mile a mile, and in the extreme this width may broaden to more than a mile, but with an average as previously noted. The same bedrock would be found to continue the bottom at depths varying from about 70 feet to over 200 feet below the alluvium which has filled the original channel. A short distance beneath the surface of the alluvium water is found from bank to bank, i.e., across the width of the canyon or valley from hill to hill. Its presence is indicated by the water loving vegetation growing in parts of the bottomlands but forming a continuous band or area from Monserate Narrows and above, almost to the ocean where salinity prevents its growth. The presence of underground flow is also proven by the wells which have been dug. The slope of the underground stream is about 10 feet per mile immediately above the ocean, gradually increasing upstream to about 14 feet per mile at Monserate Narrows. Movement downstream is very slow, but can be deduced from the fact that although the channel is dry above, water appears on the surface at Monserate Narrows, Old Bonsall Crossing, Bonsall narrows and Oceanside Narrows, and during the winter when evapotranspiration losses are small, forms a stream of considerable proportions which may again disappear below. The water thus appears because the underground channel at these points is too narrow to carry the flow which is moving through the wider and deeper channels above and below."

Based upon the foregoing, withdrawals from the alluvium in the Bonsall Basin are highly likely to be classified as surface water.[8] This can be challenged, and there may be site- specific variations. However, this is at the very least the starting assumption.

[8] The Fallbrook water right permit approved in this decision was never developed; the Carlsbad permit has been revoked; the City of Oceanside permit is active. Oceanside and other downstream water users/rights holders are likely to be concerned with any proposal to withdraw water from the system upstream from their points of diversion.

6.1.2.2.3 Rainbow Creek

This creek is part of the Santa Margarita River system, and is covered under Federal District Court Case 1247-SD-C. In Interlocutory Judgments 42 and 42A, the Court determined that the surface flows and subterranean waters, other than groundwater in the basement complex and residuum of the Rainbow Creek subbasin, are legally surface waters subject to the continuing jurisdiction of the court.

6.1.3 Spring Waters

A "spring" has been defined to mean:

> *"a damp, marshy, or boggy area, usually of small, but definite extent, wherein underground waters from a larger tract of land find their way to the surface thereof and make their presence known either by a definite outflow or by the surface presenting such a quantity thereof as will render practicable their assembling" [of the water into receptacles and conduits].*

Harrison v. Chaboya (1926) 198 Cal. 473, 476. Another definition of "spring" is:

Water rising to the surface of the earth from below and either flowing away in the form of a small stream or standing as a pool or small lake. De Wolfskill v. Smith (1907) 5 Cal.App. 175, 181.

Rights to spring water may derive from either surface or ground water law. If the spring water is flowing underground in a known and defined underground channel, or has surfaced and become surface flow, surface water law rules apply. Either riparian or appropriative rights may attach to it. De Wolfskill, supra, 5 Cal.App. at 181. Such rights have the same characteristics as they do with respect to other sources of surface water. For example, the riparian landowner on whose land the spring water has its source is entitled to a reciprocal share of the water for the benefit and enjoyment of his land, in common with others further down the stream. Gutierrez v. Wege (1905) 145 Cal. 730, 733; Eckel v. Springfield Tunnel & Development Co. (1927) 87 Cal. App. 617, 622, 624.

A spring water right classified as either overlying (if true groundwater) or riparian (if surface water) may be severed by a transfer or reservation of right separate from the real property to which it is appurtenant. Spring Valley Water Co. v. Alameda County (1927) 88 Cal.App.157, 167-168; Duckworth v. Watsonville Water & Light Co. (1907) 150 Cal. 520, 526. If the natural flow of the spring does not flow past the boundaries of the parcel on which it arises, all of the water may be used by the owner of the ground on which the spring rises. Simons, 48 Cal.App. at 542.

Groundwater law applies to subterranean spring water if it is not flowing through a "known and defined channel" (Water Code Section 1200, see discussion infra.) Spring water "passing through the soil, not in a stream but by way of filtration" is not subject to appropriation, but is considered part of the soil. De Wolfskill v. Smith (1907) 5 Cal.App. at 181.

The same regulatory requirements attach to surfaced spring waters as to other surface waters. Prior to the enactment of the Water Code, spring water constituting surface flow could be appropriated by actual use or via compliance with the procedures specified in the Civil Code. De Wolfskill, supra, 5 Cal.App. at 181-1 83. After 1914, appropriations can only occur via compliance with the procedures specified by the Water Code and the SWRCB.

6.1.4 Rights to Return Flow

The importance of rights to return flow depends substantially upon the location and upstream water practices. A classic example is the Sacramento River, where water diverted initially at the head of the valley is re-diverted and re-used multiple times as it descends southward. In this example, the water is largely natural except to the extent it was stored, such as in Shasta Reservoir. The general rule is that the downstream landowner has a right to the return flow of an upstream user.

6.1.5 Developed or Foreign Water

6.1.5.1 General

This category of water typically involves the addition of a new water supply due to the efforts or investment of a water user. The basic rule is that where a party develops a supply that augments the natural supply[9], he or she is entitled to use it provided that there is no injury to existing users. Because riparian and overlying rights do not attach to foreign water, the importer has broad discretion as long as the use is reasonable and beneficial. Under traditional water law, the one who develops a new supply such as foreign water may cease or change that practice without any obligation to downstream return flow users, including appropriators who may use developed or foreign water that is abandoned (usually the return flow from application of such water), but do not have a right to require continued abandonment. This may vary if, for example, there is injury to fish and wildlife (see, e.g., SWRCB WRO 95-9 (Deer Creek) or other water right users. A “no injury” rule applies. For example, in the event of basin spill, the foreign or imported water is considered to spill first to avoid impacting those with rights to the native water.

6.1.5.2 Imported Water

This is a sub-category of developed or foreign water. Water is imported when it is brought in from another watershed or groundwater basin, as is done extensively via projects such as the State Water Project (SWP), the Colorado River Project, and the Central Valley Project (CVP). After the water is applied to use, its return flow does not lose its character as imported or foreign water even when co-mingled with native water. City of Los Angeles v City of San Fernando (1975) 14 Cal.3d 199.

The importer has the right to recapture the imported water, provided that this can be done without injury to existing users, usually the users of water with which the imported return flow has commingled. No new water right is required for such recapture. In some instances the contract,

[9] Natural supply is water from rainfall, snowfall, channel infiltration, tributary runoff and other native sources not resulting from human intervention.

permit or other authorization for the first use of the imported water may contain conditions applicable to its recapture and re-use.

Imported or developed water which is not recaptured by the importer is available for appropriation by others, under the normal rules governing appropriation (e.g., a permit may be needed for surface waters.) The water is considered to have been abandoned or relinquished by the importer, and thus available for appropriation. Crane v Stevinson (1936) 5 Cal.2d 387.

The importer cannot recapture more than his input to the water system without complying with the rules of appropriation.

RMWD has for years imported a combination of Colorado River water and SWP water, which provides return flow to the basin. This creates the potential for recapture.

6.1.5.3 Treated Wastewater

The owner of a wastewater treatment plant has the exclusive right to the treated wastewater it produces as opposed to anyone who has supplied the water discharged into the wastewater collection and treatment system, including a person using water under a water service contract. (Water Code § 1210.) This rule can be varied by contractual arrangement. (Id.) If the volume of treated wastewater being discharged to a surface watercourse is proposed to be reduced, approval by the SWRCB is required. The no injury rule, and public trust considerations, will apply. Water Code §1211; SWRCB WRO 95-9 (Deer Creek).

It is important to note that these principles could arguably apply in the context of an imported water recapture project. Certainly the ‘no injury’ rule applies.

6.1.6 Fully Appropriated Stream Systems

The SWRCB can declare all or a portion of a stream system as “fully appropriated” pursuant to Water Code Section 1205(a). No new applications to appropriate will be accepted during the time, and within the geographic extent of, such declaration absent a showing that an exception is merited, such as that there is “new” water that is available for appropriation, the proposed use is non-consumptive, or area of origin rights are sought.

The neighboring Santa Margarita River system in its entirety is subject to a Declaration of Full Appropriation year round. The Study Area focus is the San Luis Rey River. Three tributaries of the San Luis Rey River are subject to a Declaration of Full Appropriation pursuant to SWRCB Order 98-08: Tickner Spring, Pilgrim Creek, and an unnamed spring. The mainstem does not appear to be included in this Order.

Declarations of full appropriation rely on prior decisions of the SWRCB. These can be quite old, and need to be examined to determine if they provide an adequate basis for the declaration.

6.1.7 Reasonable and Beneficial Use Requirement

The California Constitution, Article X Section 2 requires that all water uses and methods of diversion of water be reasonable and beneficial. Peabody v City of Vallejo (1935) 2 Cal.2d 351; SWRCB Decision 1644 *In the Matter of Fishery Resources and Water Right Issues of the Lower Yuba River* (2001.) It applies regardless of the priority of right. It has been used to require increased conservation, and to prohibit or modify diversions that adversely affect fish whether due to lack of sufficient remaining flow or lack of screening.

This doctrine applies to the wide range of water diversion and use, including recapture of imported water return flows.

6.2 FOCUSED DISCUSSION: RMWD RECAPTURE OF IMPORTED WATER RETURN FLOW

Water is imported when it is brought in from another watershed or groundwater basin, as is done extensively in the study area. After the imported water is applied to use, its return flow does not lose its character as imported water even when co-mingled with native water. In the area of the San Luis Rey River groundwater basin overlain by RMWD, the native water, including groundwater, is subject to riparian and appropriative rights.

Imported water is a type of developed or foreign water. Developed or foreign water is created when a new water supply is developed due to the efforts or investment of a party. The basic rule is that where a party develops a new supply that augments the natural supply, that party is entitled to use the new supply provided that there is no injury to existing users. For example, in the event of basin spill, the imported water is considered to spill first to avoid impacting those with rights to the native water.

The importer has the right to recapture the return flow from the water it imported, provided that this can be done without injury to existing users. Existing users include the users of native water with which the imported return flow has commingled; potentially the users of water imported by others, and potentially fish and wildlife (see discussion above of the Deer Creek case.) Secondary users of the importer's imported water return flow should not be able to require its continued abandonment; the importer has the right of recovery.

No new water right is required for such recapture. In some instances the contract, permit or other authorization for the first use of the imported water may contain conditions applicable to its recapture and re-use. The importer is entitled to recapture the importer's input to the water system; additional diversions beyond that amount could require a separate basis of water right. Mechanisms to address the need for a separate basis of right include some form of agreement to act as agent for another importing entity if the water is the other agency's imported water return flow, or potentially by compliance with the rules of appropriation (e.g., obtaining a permit).

Imported water that is not recaptured by the importer is considered to have been abandoned or relinquished by the importer and can be appropriated others. The intent of the importer is relevant in this context.

Other parties may use imported water that is abandoned by the importer (usually the return flow from application of such water). This is typically done by exercise of an appropriative right; riparian right holders are limited to use of native water.

As mentioned above, the appropriators of the abandoned water (sometimes referred to as "secondary users") do not have a right to require continued abandonment by the importer. The SWRCB has standard permit and license terms for post-1914 appropriative water rights issued by the SWRCB. Supplemental permit term 25 specifically addresses the right to imported water return flow. It makes the permit or license right to imported water return flow contingent due to the right of the importer to recapture. Supplemental permit term 25 is one of the "Mandatory Terms" reflected in the Water Right Template. It can be viewed at the SWRCB website:

http://www.waterboards.ca.gov/waterrights/water_issues/programs/permits/terms_20thru39.shtml

The term reads as follows:

> "*K. All rights are issued subject to available flows. Inasmuch as the source contains treated wastewater, imported water from another stream system, or return flow from other projects, there is no guarantee that such supply will continue.*"
>
> *(0000025)*

An example where this term was applied in the San Luis Rey River watershed is Permit 21280 (Application 29731), condition 10 on page 4, for a right to divert from Keys Creek tributary to the San Luis Rey River. Note that the SWRCB should, but does not always, include the standard mandatory terms in its past actions. This should not matter given the underlying 'black letter' water law. Whether others would take a different position is not known.

RMWD should consider routinely protesting filings at the SWRCB that may involve diversion of its imported water return flow to ensure the consistent inclusion of this term.

6.2.1 Discussion of Conclusions from the Hydrologic Analysis

The primary legal principles that govern an imported water return flow recapture project can be paraphrased as: (1) the importing agency has a right of recapture the imported water return flow it brought into the basin by its efforts; and (2) the recapture is not allowed to adversely affect native water and uses thereof.

The hydrologic analysis in Chapter 5 indicates that "groundwater" pumping dewatered the basin before the importation of surface water (Section 5.2.2.1, page 5-8). Some of this pumping may not have been legal; there is no entitlement to overdraft a basin. To the extent that such pumping continues, equaling or exceeding the available native water, it either needs to be abated to the extent it is pulling water to which the pumper(s) are not entitled, or an increment of imported water return flow may need be left to accommodate such continued pumping. The location of the pumping may influence its relevancy to the project. The hydrologic analysis in Chapter 5 also states that pre-existing pumping in the study area was assumed to continue at stated values (see Section 5.2.4, page 5-12). It further concludes that with the exception of localized effects in proximity to active project wells, groundwater levels are not significantly affected by Proposed

Groundwater Supply Project pumping, and stream discharge is maintained at levels higher than prior to importation of water by RMWD, FPUD and VCMWD. Both streamflow in and below the reach where project wells would be sited, and stream discharge at Bonsall Narrows downstream of the project wells, were modeled (See Section 5.2, pages 5-2 to 5-13). The Draft Report conclusions apply to both of these flows.

The Draft Report further states the conclusion as follows (Section 5.2.2.2, page 5-11):

> "These results demonstrate that the Proposed Groundwater Supply Project does not reduce groundwater levels below the level that would exist in the absence of imported water (Baseline 1), except at locations near proposed project wells. These effects can be mitigated by optimizing flow rates across the well network to balance pumping with the geographic availability of recharge."

The "Summary of Model Results" in Section 5.2.6 (page 5-13) similarly concludes that a recapture project can extract up to 5,700 afy without significantly affecting the hydrology that would exist absent imported water return flows, other than localized impacts that would need to be mitigated if they adversely impact another legal user of water.

Given the conclusion from the hydrologic analysis presented in Chapter 5, that there are no significant effects other than the localized impacts, are such effects permissible? The water law rule is that prior water rights are entitled to protection against material injury, or "substantial damage" (see, e.g., Peabody v City of Vallejo (1935) 2 Cal.2d 351, 374-375.) Such damage is also sometimes described as an appreciable diminution in the quantity or quality of water. The conclusion from the hydrologic analysis is that there is not an appreciable diminution of the quantity of water. The impacts on the water quality were not evaluated in the modeling, but water quality would either be unchanged or improved with the project. This is because project pumping would create storage capacity in the aquifer, which would be available to accept recharge from precipitation. Without the project, this storage would not be available due to the aquifer being more frequently filled to capacity with imported water. The water quality improvement would occur because recharge from precipitation has much lower salinity then recharge from imported water return flows (Section 5.2.6).

Such injury can only arise if the party causing the injury is exceeding the bounds of his water right. In this case, where RMWD is recapturing its imported water return flow, injury associated with depriving those who previously used abandoned RMWD imported water return flow (so-called secondary users) do not have a legally cognizable injury.

The volume of 4,000 afy was selected for preliminary design purposes, primarily for the purpose of infrastructure efficiency (see Section 5.2.6, page 5-13). Because this volume is less than the modeled volume, it may reduce or eliminate even insignificant impacts to the hydrology absent imported water return flows.

RMWD's right of recapture applies to return flows from water it has imported. There is also imported water return flow in this watershed from two other agencies. As between RMWD and third parties (other than the importing agency) who use the other importing agencies' imported water return flows, the relative rights are less well defined. RMWD should not assume that it has a prior right. If and to the extent that RMWD were to seek to recapture other's imported water return flow, the best course of action would be to either enter into agreements to authorize RMWD to recapture some or all of those return flows, or to avoid impacting them.

The hydrologic analysis analyzed the allocation of imported water return flow as between the three importing agencies. Per the input water budget (Section 4.6), RMWD's modeled imported water return flows averaged 2,645 afy for the modeled years 2035 through 2046. (Variations by month may also be relevant.) This means that some form of agreement with one or more of the other importing agencies would be appropriate for a 4,000 afy project.

6.2.2 Further Steps

The source of imported water to RMWD is based upon RMWD's purchase of water from the SDCWA, and the water rights holders for the Colorado River and SWP water which is imported. Water right terms and contractual arrangements can be the basis for limitations on imported water recapture. We are not aware of any constraint on recapture of the imported water once it has been delivered to the customer for use. This should be verified with SDCWA[10]. Typically, the wholesale agencies support imported water recapture projects due to the constraints on imported water availability.

If RMWD determines to proceed with a project for the recapture of imported water return flow, it should make its intent to cease abandonment, and to recapture, known by public notice to potentially affected parties. It would be useful to inform RMWD customers, the SDCWA, the City of Oceanside, and of course the other two importing agencies, and potentially the Santa Margarita River Watermaster, to update them regarding the proposed project and to identify any concerns.

Furthermore, RMWD should consider protesting water rights filings at the SWRCB that might involve diversion of its imported water return flow, to ensure inclusion of protective terms such as standard permit term 25 discussed above.

[10] We did not review the terms of RMWD's relationship with its retail customers.

7.1 INTRODUCTION

This chapter describes the analysis of engineering alternatives supporting the Proposed Groundwater Supply Project. Based on the hydrologic analysis and modeling results documented in Chapter 5, a capacity of 4,000 afy or 333 afm was selected for preliminary design purposes. As documented in this chapter, previous reports and groundwater quality sampling conducted for the current study show that groundwater produced by the Proposed Groundwater Supply Project will require treatment to meet applicable water quality standards. As discussed below, drinking water standards were determined to be the most likely standards for treatment, and these standards will require pretreatment to reduce iron and manganese (IM) concentrations followed by reverse osmosis (RO) treatment to reduce TDS concentrations.

Two treatment and conveyance alternatives were considered. Figure 7-1 shows the alternatives.

Alternative 1. Under the Alternative 1, groundwater would be pumped from a network of approximately 18 wells distributed along the portion of the San Luis Rey Valley Groundwater Basin within the study area. Figure 5-1 shows the locations of the 18 wells evaluated in the numerical model. For preliminary design purposes, these wells were assumed to operate at an average rate of approximately 140 gallons per minute (gpm) and would produce an average annual flow of 4,000 afy or 3.6 million gallons per day (MGD). These rates are based on the assumption that the water from the Proposed Groundwater Project will be used for ongoing supplemental supply to meet average day demands, and that peak demands will be met using other, existing sources and infrastructure.

Groundwater from the wells would be conveyed to the Proposed Bonsall Basin Groundwater Desalination Facility (GDF) located near the confluence of the San Luis Rey River and Moosa Canyon (Figure 7-1). Brine discharge from the RO treatment would be conveyed through a new, dedicated pump station and brine line to the City of Oceanside's San Luis Rey Water Reclamation Facility (WRF), and ultimately to the City of Oceanside's ocean outfall (Figure 7-1).

Alternative 2. Under Alternative 2, groundwater would be pumped from the same network of approximately 18 wells and conveyed as a raw water supply through a new, dedicated pump station and raw water pipeline to the City of Oceanside's Mission Basin Groundwater Purification Facility (GPF) for treatment. Brine discharge from the RO treatment would be conveyed through the existing Mission Basin GPF brine line to the City of Oceanside's ocean outfall (Figure 7-1).

7.2 CHAPTER OUTLINE

The following sections are included in this chapter:

- Section 7.3: This section provides a review of existing groundwater quality/treatment information for the alluvial portion of the San Luis Rey (SLR) River Basin. This included most relevantly a report prepared by CDM in 1996 on groundwater demineralization for RMWD and an alternative water source feasibility study prepared by Heden and Associates in 2013.

- Section 7.4: This section provides a detailed raw water quality characterization, including evaluation of the water quality based on groundwater sampling conducted for this project, additional sampling data from literature review, and relevant data obtained from the City of Oceanside. The water quality evaluation included development of the design basis raw water quality for the project.
- Section 7.5: This section documents the evaluation of the treated water quality goals used as the basis of design. The analysis included evaluation of regulatory requirements. At the same time, an analysis was performed of the viability of landscape irrigation and blending with imported water in addition to the option for use of SLR groundwater as a public drinking water supply.
- Section 7.6: This section states the recommendations for treatment under Alternative 1, including a conceptual estimate of cost for treatment. RO and greensand filtration were selected as the key components of the process train for Alternative 1, based on the water quality evaluation. The Alternative 1conceptual treatment process design is described in this section. This section also documents the estimated capital and O&M costs for treatment under Alternative 1.
- Section 7.7: This section describes the wells, pump stations, raw water pipelines and brine pipelines, and associated costs for Alternatives 1 and 2.
- Section 7.8: Summarizes the estimated costs for Alternatives 1 and 2.

Existing Water Quality and Treatment Technology Review

Water quality and treatment considerations from past work are presented below.

7.2.1 Water Quality Data from the Review of Past Work

The sources of data used for the water quality review include:

- Wellhead water quality data in the report prepared by CDM in 1996, "Demineralization of Groundwater within the Rainbow Municipal Water District (CDM Report)" The report presented the results of sampling wells in the vicinity of RMWD. The water quality of the Pope Well and the Vessels Ranch Well were discussed. TDS concentrations in the Pope Well and Vessels Ranch Well were 1080 mg/L and 2230 mg/L, respectively. Water quality parameters that exceeded State and Federal Primary Maximum Contaminant Levels (pMCLs) or Secondary MCLs (sMCLs) in the Pope Well were manganese, sulfate, and TDS. In the Vessels Ranch well, parameters that exceeded the MCLs were manganese, sulfate, TDS, chloride, iron, and color. The detailed water quality results are listed and compared in Table 7-1. Treatment with electrodialysis (ED) or RO was compared for treating groundwater from the Pope Well and Vessels Ranch well with the varied TDS levels shown above. Different capacity systems were evaluated ranging from 1250 to 3000 acre-ft/year. The report determined treatment by ED was less cost effective than RO.

Table 7-1. Summary of Water Quality from Sampled Wells and Other Available Sources

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
General Water Quality Parameters										
Calcium	mg/L	—	170	180	199	217	243	225	240	135
Magnesium	mg/L	—	82	76	73	78	88	82	115	55
Potassium	mg/L	—	6.7	7.9	9.5	8.8	9.7	9.8	9.0	9.0
Silica	mg/L	—	34	28	23.8	21.5	24.3	27.5	24	25
Sodium	mg/L	—	170	180	229	229	284	257	330	122
Hydroxide as OH	mg/L	—	ND	ND	—	—	—	—	<3	<3
Carbonate as CO3	mg/L	—	ND	ND	—	—	—	—	<3	<3
Total Hardness as CaCO3	mg/L	--	760	760	—	—	—	—	1080	567
Bicarb. Alkalinity as HCO3	mg/L	—	280	260	325	322	341	331	413	262
Ammonia Nitrogen	mg/L	—	ND	0.24	0.6	0.2	0.3	0.4	0.3	0.20
Alkalinity in CaCO3	mg/L	—	230	210	259	258	274	267	343	215
pH	Units	—	7.4	7.4	—	—	—	—	7.2	7.5
Orthophosphate as P	mg/L	—	0.031	0.034	<0.2	<0.2	<0.2	<0.2	—	—
Total Phosphorus as P	mg/L	—	—	—	0.16	0.09	0.09	0.09	0.050	0.100
Total Organic Carbon	mg/L	—	2.2	3.4	—	—	—	—	—	—
Surfactants	mg/L	—	ND	ND	—	—	—	—	—	—
Constituents with Primary Maximum Contaminant Levels (pMCLs)										
Inorganic Chemicals										
Aluminum	µg/L	1000	ND	ND	—	—	—	—	—	—
Antimony	µg/L	6	ND	ND	—	—	—	—	—	—
Arsenic	µg/L	10	ND	ND	--	—	—	—	—	—
Barium	mg/L	1	0.04	0.061	0.2	0.1	0.1	0.1	<0.1	<0.1
Beryllium	ug/L	4	ND	ND	—	--	—	—	—	—
Cadmium	µg/L	5	ND	ND	—	--	—	—	—	—
Chromium	µg/L	50	ND	ND	—	--	—	—	—	—
Fluoride	mg/L	2	0.45	0.26	0.3	0.5	0.4	0.3	0.40	0.40
Hexavalent chromium (Dissolved)	µg/L	10	ND	ND	—	—	—	—	—	—
Mercury	µg/L	2	ND	ND	—	—	--	--	—	—
Nickel	µg/L	100	ND	ND	—	—	--	--	—	—
Total Nitrate, Nitrite-N	mg/L	10	0.35	0.24	—	—	--	--	—	—
Nitrate as NO3	mg/L	45	1.6	1.1	0.17	0.04	0.18	0.22	—	—
Nitrite Nitrogen	mg/L	1	ND	ND	—	—	—	—	—	—
Perchlorate	µg/L	6	ND	ND	—	—	—	—	—	—
Selenium	µg/L	50	ND	ND	—	—	—	—	—	—
Thallium	µg/L	2	ND	ND	—	—	—	—	—	—
Radionuclides										
No Data Collected										
Volatile organic chemicals (VOCs)										
No Data Collected										
Disinfection Byproducts (DBPs)										
No Data Collected										
Constituents with Secondary Maximum Contaminant Levels (sMCLs)										
Aluminum	µg/L	200	ND	ND	—	—	—	—	—	—
Chloride	mg/L	250 Rec 500 Upper	320	320	440	499	587	513	600	190
Apparent Color	ACU	15	20	75	—	—	—	—	25	15
Specific Conductance 25 C	umho/cm	900 Rec 1600 Upper	2100	2200	—	—	—	—	3400	1520
Copper	µg/L	1000	ND	ND	—	—	—	—	—	—
Iron	mg/L	0.3	0.80	3.5	3.3	1.3	2	2.6	0.80	0.04
Manganese	mg/L	0.05	0.20	0.65	0.62	0.46	0.48	0.62	0.57	0.22
Odor	TON	3	2.0	2.0	—	—	—	—	—	—
Silver	µg/L	100	ND	ND	—	—	—	—	—	—
Sulfate	mg/L	250 Rec 500 Upper	480	520	415	462	454	427	760	370
Total Dissolved Solids	mg/L	500 Rec 1000 Upper	1400	1500	1577	1673	1888	1754	2230	1080
Turbidity	NTU	5	9.0	50	—	—	—	—	6.7	0.4
Zinc	µg/L	5000	ND	ND	—	—	—	—	—	—
Constituents with Notification Levels (NLs)										
1,2,3-Trichloropropane	µg/L	0.005	ND	ND	—	—	—	—	—	—
Boron	mg/L	1	0.11	0.11	0.19	0.17	0.17	0.19	—	—
Vanadium	µg/L	50	ND	ND	—	—	—	—	—	—
Constituents with Action Levels (ALs)										
Copper	µg/L	1300	ND	ND	—	—	—	—	—	—
Lead	µg/L	15	ND	ND	—	—	—	—	—	—
Microbiological Parameters										
No Data Collected										

[a] It should be noted that the groundwater quality in the Mission and San Luis Rey River Groundwater Basins is similar and amenable to treatment by similar processes. This is important because one alternative under consideration is to treat the groundwater from the San Luis Rey River Groundwater Basin at the Mission Desalter. It is especially important to note that the Fe and Mn concentrations are similar and presumably need similar treatment (oxidation-filtration treatment prior to RO).

- "Alternative Water Source Feasibility Study" (Feasibility Study) prepared by J.C. Heden and Associates, Inc., Jan. 2013 (Heden Report). No wells were sampled during the Feasibility Study. However, it was concluded that the Basin appears to have high TDS. It referred to groundwater samples obtained from shallow wells in 1989 indicating a range of 370 to 2330 mg/L TDS. It referenced samples from Rainbow Creek during low flow (base flow) conditions demonstrating a range of 793 to 1325 mg/L TDS. Nutrients, primarily nitrate and phosphate, were also evaluated in the Feasibility Study. Different sources indicated concentrations of nitrate as nitrogen peaked in the mid-1980s at 77 mg/L. The Heden Report discussed treatment with RO but did not provide a detailed description of the treatment train.
- Wellhead water quality data from 2009 to 2011 in the report, "Process Evaluation and Recommendations for the Mission Basin Groundwater Purification Facility," prepared by Trussell Technologies for the City of Oceanside, Nov. 2012. The wells evaluated in the City of Oceanside project are located in Mission Basin along the California Coastal Basin aquifers. Water quality data from the Mission Basin Desalter is presented in the detailed evaluation of raw water quality.

7.2.2 Existing Treatment Technology Review

RMWD has looked into groundwater desalination as a potentially viable local water supply over the past several years. The SLR Groundwater Supply is available to RMWD for such a project. This has resulted in the development of reports evaluating feasibility, as discussed above with respect to the water quality evaluation. A summary of reports and outcomes for past groundwater desalination treatment evaluations is provided below.

- CDM Report
  The report discussed groundwater quantity and quality, and the treatment alternatives to demineralize groundwater for two existing wells, Pope Well and Vessels Ranch Well. Electrodialysis and reverse osmosis were evaluated under different flow conditions, and capital cost and O&M costs were estimated for these scenarios as well.
- Heden Report
  In the Feasibility Study prepared by Heden, recapture of the imported water and treatment with reverse osmosis was listed as the top priority recommendation. Cost and payback periods were evaluated in the report.

7.3 RAW WATER QUALITY CHARACTERIZATION

7.3.1 Sampling

In June 2015, two wells were sampled with the assistance of RMWD staff. The entrance to the sampling area is located at 4141 Pala Rd, Fallbrook, CA 92028. The sampling area is currently used by Caltrans as a storage yard and is located roughly 2000 feet southeast of RMWD. The two wells sampled were being actively pumped at the time of the sampling event. The location of the two sampling wells is shown in Figure 7-2. The Pope Well and Vessels Ranch Well, for which sampling results were presented in the CDM report, are also shown in Figure 7-2, along with the

Mission Basin wells for the City of Oceanside. The methods used to analyze groundwater samples are listed in Appendix A.1.

7.3.2 Raw Water Quality Summary

A sampling event was conducted in June 2015, yielding water quality results for each of the two wells sampled (labeled Well 1 and Well 2). The water quality data from the CDM report included sampling results for two wells (Pope Well and Vessels Ranch Well). Data available from the City of Oceanside for four wells (Wells 1, 2, 3, and 9) was also included in the water quality evaluation. A summary of all water quality data evaluated is presented in Table 7-1. The wells sampled by RMWD in 2015 are labeled "Well 1" and "Well 2" to differentiate them (see Figure 7-2 for their locations). The wells sampled in 2015 are in close proximity to one another, with only 800 feet separating them. A detailed water quality evaluation is provided below that considers the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells. Water quality parameters most likely to be relevant to groundwater treatment for the San Luis Rey River area were included in the sampling on this project. Once a site is determined, a full round of sampling of regulated constituents and non-regulated constituents with health-based advisory levels is recommended.

7.3.3 Constituents with Primary Maximum Contaminant Levels

There is water quality data available for a limited number of constituents with pMCLs. The water quality data included for pMCLs represents either constituents most likely to be of potential concern or constituents relevant to the design of the treatment process. It should be noted that aluminum, antimony, arsenic, beryllium, cadmium, chromium, copper, lead, nickel, selenium, thallium, mercury, and perchlorate were not detected in either of sampled groundwater wells. Fluoride concentrations measured were 0.26 mg/L and 0.45 mg/L for the wells sampled in 2015. In comparison, fluoride concentrations measured for Pope Well and Vessels Ranch well were both 0.40 mg/L and Mission Basin well fluoride concentrations ranged from 0.3 to 0.5 mg/L. The fluoride concentrations showed consistency across all the well data evaluated. All fluoride measurements were below the state pMCL of 2 mg/L.

Barium concentrations measured for the wells sampled in 2015 were 0.040 mg/L and 0.061 mg/L. While these concentrations are lower than observed for the other wells evaluated (<0.1 to 0.2 mg/L), none of the concentrations observed are expected to be an issue for the treatment process. All measured barium values met the state MCL. Both samples of recently regulated hexavalent chromium were non-detect. Thus, no constituent with a pMCL is expected to be present at a concentration greater than its pMCL in the raw water for the design consideration. A full list of the regulatory pMCLs is shown in Appendix A.2.

7.3.4 Constituents with Secondary Maximum Contaminant Levels

Constituents with sMCLs are not of concern to public health. Instead, they cause the aesthetics of the water to be of concern to sensitive consumers. Constituents with sMCLs in most cases have a discrete MCL, but there are a limited number of constituents that the State Water Resources Control Board Division of Drinking Water (DDW) regulates with a recommended sMCL and an upper sMCL limit. In most cases, new water treatment plants will be required to meet the recommended sMCL for drinking water applications. The water quality data showed that several constituents with sMCLs exceeded their recommended sMCL in both sampled wells. The findings are consistent with water quality results in CDM report and with the Mission Basin wells. They are listed in Table 7-2. These constituents will require treatment to produce a water quality that meets the recommended sMCLs. A full list of constituents with sMCLs is provided in Appendix A.3.

Well 2 has a higher concentration of manganese and iron than Well 1. These results could be related to the higher apparent color observed in Well 2. The Pope Well, the Vessels Ranch Well, and the four Mission Basin Wells also had similar but slightly different results for TDS, conductivity, turbidity, and iron. The concentration of all of the constituents listed in Table 7-2 can be adequately reduced with a combination of IM and RO treatment.

There are some observations that should be made about the sampling results of certain constituents for the wells sampled in 2015 given their close proximity. The turbidity in Well 1 and Well 2 varied from 9 NTU (Well 1) to 50 NTU (Well 2). The turbidity of 50 for Well 2 is unrealistically high for groundwater. From a water quality perspective, there is no reason to expect the turbidity of two wells so close together to differ by a factor of five. The well condition may provide an explanation for the elevated turbidity. A well in need of rehabilitation or replacement may produce turbid water. Also, it is not uncommon to observe variations in iron levels in groundwater wells, because chemical reactions and biological activity occurring in the well structure affect the type of iron that is present in the dissolved form and the concentration level. Wide variations in iron concentrations are seen across the wells evaluated for the study (see TDS response below). The Vessels Ranch Well was measured at 0.8 mg/L, which is same as Well “1”. The Mission Basin wells were measured at 2-3.3 mg/L, which is the same order of magnitude as Well “2". The Pope Well had a very low concentration of 0.04 mg/L.

7.3.5 Constituents with Notification Levels

Of the measured parameters with notification levels (NLs), none exceeded the NL. Boron in both wells was measured at 0.11 mg/L, well below the notification level of 1 mg/L. Vanadium and 1,2,3-trichloropropane were not detected in either well sampled by RMWD in 2015. Therefore, no constituent with a NL is expected to be present at a concentration greater than its NL in the raw water. It should be noted that a complete round of sampling of all constituents with NLs was not conducted in this study, which focused on sampling the constituents most likely to be of concern for SLR groundwater.

Table 7-2. Constituents in Sampling Wells that Exceed the sMCLs

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
Chloride	mg/L	250 Rec 500 Upper	320	320	440	499	587	513	600	190
Apparent Color	ACU	15	20	75	—	—	—	—	25	15
Specific Conductance, 25 °C	umho/cm	900 Rec 1,600 Upper	2,100	2,200	--	--	--	--	3,400	1,520
Iron	mg/L	0.3	0.80	3.5	3.3	1.3	2	2.6	0.80	0.04
Manganese	mg/L	0.05	0.20	0.65	0.62	0.46	0.48	0.62	0.57	0.22
Sulfate	mg/L	250 Rec 500 Upper	480	520	415	462	454	427	760	370
Total Dissolved Solids	mg/L	500 Rec 1,000 Upper	1,400	1,500	1,577	1,673	1,888	1,754	2,230	1,080
Turbidity	NTU	5	9.0	50	—	—	—	—	6.7	0.4

7.3.6 Process Design Considerations

Some important points to consider in process design are shown below for:

- IM removal,
- RO sizing,
- RO recovery and pretreatment,
- Disinfection, and
- Product water stabilization.

IM treatment targets the elevated concentrations of iron and manganese in the raw water and reduces them. One of the advantages of IM treatment is the fact that it typically achieves removals in excess of what is typically required. Therefore, the IM system can be designed based on the range of iron and manganese levels observed. In this project, the maximum iron concentration is 3.5 mg/L, which is a typical iron concentration for wash water recovery systems. The presence of some iron is beneficial, because it promotes the settling of manganese and iron particulates in the backwash water.

If RO is employed to reduce the concentration of TDS, conductivity, chloride and other dissolved constituents, the sizing of the RO system will be determined by a combination of water quality goals including general mineral quality. The maximum recovery of RO will also be limited by constituents that may cause membrane scaling in the feed water, such as silica. In this case, antiscalant and/or pH adjustment may be required to increase the RO recovery.

It should be noted that the total organic concentration (TOC) levels observed in both wells sampled in 2015 (2.2 and 3.4 mg/L) are generally higher than typical groundwater, which commonly has TOC levels below 1 mg/L. The potential consequence is the fact that the level of disinfection byproducts (DBPs) may exceed the regulatory limit. Therefore, a bench scale Simulated Distribution System (SDS) test would be helpful to determine the potential for DBP formation for consideration in the design of the treatment facility.

Usually, raw water quality, including pH, temperature, alkalinity, and calcium, will affect the design of the product stabilization chemical system and the clearwell capacity. The blending of the RO permeate and the RO bypass (a portion of IM treated water that is not treated by RO) will help with these elements of the treatment train by increasing the mineral content of the product water from the treatment plant. The low mineral content and low buffer capacity in RO permeate must be adjusted to assure corrosion control and to provide a drinking water that meets aesthetic preferences of consumers.

7.3.7 Design Basis Water Quality

During this raw water quality characterization, the design concentrations of iron and manganese were established based on elevated levels of iron and manganese observed in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells. As a conservative approach the design basis TDS was determined based on the level observed in the Vessels Ranch

Well because the project team believes based on the groundwater modeling and hydrogeological analysis that the Vessels Ranch well is most typical of groundwater that would be pumped under the proposed project. The design basis TDS, iron, and manganese presented below are essentially a worst-case scenario. A worst-case scenario is commonly used as a design basis to assure the treatment plant can meet regulatory requirements during challenging conditions like drought.

The mineral water quality parameters required to size the RO are not constituents of concern with respect to the regulations. Therefore, their basis of design was determined based on averaging the water quality parameters in the wells sampled in 2015, the Pope Well, the Vessels Ranch well, and the Mission Basin wells. The raw water quality used as the design basis for the treatment technologies necessary to meet regulatory requirements are discussed below.

7.3.7.1 Constituents with Regulatory Limits and Health Advisory Levels

Almost all constituents with pMCLs were not detected in the raw water. The only constituents with pMCLs detected were fluoride and barium, but the concentrations were well below their pMCLs. Therefore, the recommended design basis assumes that constituents with pMCLs are not present at levels requiring treatment.

A limited number of constituents exceeded their sMCLs in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells were considered in establishing the basis for design. These parameters include manganese, iron (with the exception of Pope Well), chloride (with the exception of the Pope Well), sulfate, TDS, turbidity (with the exception of the Pope Well), specific conductance, and apparent color (Pope Well is at the limit of 15 ACU). These constituents require IM and RO treatment. Their recommended design basis for IM treatment is presented in Table 7-3. The recommended design basis for RO treatment is presented in Table 7-4. The remaining constituents with sMCLs were present at levels below their sMCLs and do not require treatment.

Constituents with NLs, including 1,2,3-trichloropropane and vanadium, were not detected in the wells sampled in June 2015. Other constituents with NLs, like boron, were tested and determined to be present at concentrations less than their NLs. Therefore, no constituents with NLs were present at levels requiring treatment.

The RO system design basis requires evaluation of general mineral quality. Constituents in the raw groundwater, including silica and various ions like sulfate, calcium, and phosphate can lead to fouling and scaling of the RO membranes. For this reason, general mineral quality was considered in the analysis. The design basis for mineral quality was established based on the average levels observed in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells.

7.3.7.2 Design Water Quality for IM and RO

The design basis for the constituents of concern that exceed regulatory MCLs was established based on maximum or near maximum levels, as discussed above. For other water quality parameters critical to the design, averaging of water quality parameters shown in Table 7-1 was performed.

The design basis water quality for the IM treatment system is presented in Table 7-3. The recommended design basis for iron and manganese based on the maximum observed level is 0.65 mg/L for manganese and 3.5 mg/L for iron. It was observed that other wells considered in the design had elevated levels of iron approaching 3 to 3.5 mg/L (see Table 7-1), providing additional justification for the design basis established. The same is true for manganese. Other factors may affect the IM design, including natural organic matter, pH, aluminum, and turbidity. For example, ammonia may affect the design due to its chlorine demand. In the IM filter, turbidity will be removed through the filter along with iron and manganese, contributing to the headloss. Additional discussion of these factors is provided below. The design basis for all of the water quality parameters that affect IM treatment is shown in Table 7-3.

Table 7-3. Recommended Design Basis Water Quality for Iron and Manganese Treatment System

Parameter	Units	Design Water Quality	Regulatory Limit	Water Quality from Table 7-1[a]		
				Min	Average	Max
Alkalinity in $CaCO_3$	mg/L	257	—	210	257	343
pH	Units	7.4	—	7.2	7.4	7.5
Aluminum	µg/L	ND[a]	1,000 pMCL 200 sMCL	ND	ND	ND
Ammonia Nitrogen	mg/L	0.6	—	ND	0.3	0.6
Chloride	mg/L	434	250 Rec 500 Upper	190	434	600
Apparent Color	ACU	34	15	15	34	75
Specific Conductance, 25 °C	umho/cm	2,305	900 Rec 1,600 Upper	1,520	2,305	3,400
Iron	mg/L	3.5	0.3	0.04	1.79	3.5
Manganese	mg/L	0.65	0.05	0.20	0.48	0.65
Total Dissolved Solids	mg/L	2,000	500 Rec 1,000 Upper	1,080	1,638	2,230
Turbidity	NTU	17	5	0.4	17	50

[a] Below detection limit.

Table 7-4. Recommended Design Basis Water Quality for Reverse Osmosis System Design

Parameter	Units	Design Water Quality	Regulatory Limit	Water Quality from Table 7-1[a]		
				Min	Average	Max
Aluminum	µg/L	ND[a]	1,000 pMCL 200 sMCL	ND	ND	ND
Barium	mg/L	0.1	1	0.04	0.10	0.20
Fluoride	mg/L	0.4	2	0.26	0.38	0.50
Nitrate as NO_3	mg/L	0.55	45	0.04	0.55	1.6
Chloride	mg/L	434	250 Rec 500 Upper	190	434	600
Specific Conductance, 25 °C	umho/cm	2,305	900 Rec 1,600 Upper	1,520	2,305	3,400
Iron	mg/L	0.1[b]	0.3	0.04	1.79	3.5
Manganese	mg/L	0.02[b]	0.05	0.20	0.48	0.65
Sulfate	mg/L	486	250 Rec 500 Upper	370	486	760
Total Dissolved Solids	mg/L	2,000	500 Rec 1,000 Upper	1,080	1,638	2,230
Turbidity	NTU	0.5[b]	5	0.4	17	50
Boron	mg/L	0.16	1	0.11	0.16	0.19
Calcium	mg/L	201	—	135	201	243
Magnesium	mg/L	81	—	55	81	115
Potassium	mg/L	8.8	—	6.7	8.8	9.8
Silica	mg/L	26	—	21.5	26	3.4
Sodium	mg/L	225	—	122	225	330
Ammonia Nitrogen	mg/L	0.3	—	ND	0.3	0.6
Alkalinity in $CaCO_3$	mg/L	257	—	210	257	343
pH	Units	7.4	—	7.2	7.4	7.5
Orthophosphate as P	mg/L	0.033	—	0.031	0.033	0.034
Total Organic Carbon	mg/L	2.8	—	2.2	2.8	3.4
Strontium[c]	mg/L	NA	—	—	—	—

(a) Below detection limit.
(b) Assumed IM treated water levels.
(c) Additional sampling is required in order to develop a recommended design water quality.

Ammonia affects the IM design by competing with iron and manganese to consume chlorine. Chlorine demand for ammonia is much higher than for iron and manganese, which is 8 chlorine: 1 ammonia. Therefore, maximum ammonia concentrations observed in all the wells was considered as the design water quality basis for ammonia. TOC, especially when associated with color, may indicate the formation of complex iron compounds. Complex iron compounds will likely decrease the chlorine oxidation efficiency in operation. The pH and alkalinity affect the oxidation reaction kinetics. High pH and high alkalinity are favorable for the IM oxidation process. The presence of aluminum will alter the precipitation structure in the IM filter, which will affect the manganese removal.

During IM treatment, iron and manganese concentrations are reduced by oxidation and filtration. In the meantime, turbidity will also be removed by the filtration. Therefore, lower iron, manganese, and turbidity in the IM treated water should be considered for RO design.

The design basis for the RO system is presented in Table 7-4. For the RO system design, specific water quality goals, such as chloride and TDS play an important role. RO elements have a certain salt rejection rate. The chloride concentration will affect the selection of the RO membrane, because the ability of the membrane to reject chloride is an important consideration. The RO membrane area is also an important design factor.

RO recovery is dictated by the solubility of specific dissolved inorganic constituents. Silica, and other dissolved scaling minerals have fouling potentials in RO systems. In Table 7-4, the iron and manganese concentrations reflect a typical removal through the IM system. Similarly, the turbidity value reflects an assumed IM system treated water turbidity.

In addition to IM and RO treatment, disinfection and product stabilization design will be affected by the availability of raw water quality data for important constituents. DBP data was not available from wells sampled in 2015 or the additional water quality data used in the analysis. Therefore, DBP formation potential testing should be conducted to confirm that DBP formation in the treatment processes and distribution system does not exceed pMCLs.

7.4 WATER QUALITY GOALS

7.4.1 Regulatory Requirements

For a groundwater, regulatory considerations begin with a determination on whether the source is groundwater or groundwater under the direct influence (GWUDI) of surface water. If the groundwater is classified as GWUDI, it will be required to meet the requirements of the Surface Water Treatment Rule (SWTR).

In this project, the appropriate way to classify the source water was evaluated. The classification of the source water will impact the treatment plant design criteria. With source water being groundwater, the utility would only have to demonstrate 4-log (99.99%) virus removal/inactivation at a new plant where removal/inactivation of viruses is implemented, as opposed to greater disinfection requirements for GWUDI. Disinfection requirements for GWUDI are greater than or equal to 4-log virus removal/inactivation, greater than or equal to 3-log giardia removal/inactivation, and greater than or equal to 2-log cryptosporidium removal/inactivation.

The groundwater classification presumes that there is not a significant occurrence of large pathogens in the groundwater (e.g., Cryptosporidium, Giardia), and that the groundwater is not subject to significant and rapid shifts in water quality characteristics due to changes in the water quality of the SLR River (e.g., with respect to turbidity, conductivity, pH). Consequently, groundwater has lower pathogen removal and/or inactivation requirements compared to GWUDI (purveyors of groundwater are required to conduct either triggered source water monitoring or demonstrate 4-log virus removal/inactivation).

Based on an evaluation of the SWTR Guidance Manual, the rationale to support the conclusion that the source water for RMWD is groundwater is as follows.

1. The water will be pumping out of a well (either horizontal or vertical).
2. Wells with perforations or screens less than 50 feet deep should be evaluated for direct surface water influence.

Based on discussion with West Yost on the implementation plan for the wells, it is expected that the source water will be pumped out from a well with roughly 80 feet in depth, but some of the wells may need to be evaluated for direct surface water influence. Therefore, the Groundwater Rule will be the regulatory approach that affects the design, not the SWTR.

The SWTR Guidance Manual suggests that well design criteria including the sanitary seal, the location of screens, etc., should be reviewed to confirm source water classification in the future design and construction. The casing or nearest collector lateral should be at least 200 feet from any surface water. Water quality records should indicate no record of total or fecal coliform contamination, no history of turbidity problems, and no known history of Giardia or Crypto outbreaks associated with the well.

7.4.1.1 pMCLs

The pMCLs are established at levels that, when exceeded, may have an adverse effect on the public health. They are legally enforceable limits. They are in place for inorganics, organics (volatile organic chemicals and synthetic organic chemicals), pathogens, radionuclides, and DBPs. The recommended water quality goal for constituents with pMCLs is to meet the pMCL. A summary of the constituents with pMCLs is shown Appendix A.2.

An evaluation of the raw water quality data showed that measured constituents with pMCLs were in no case detected at levels exceeding a pMCL.

Therefore, the only constituents with pMCLs worthy of further consideration in the project are those constituents impacted by the treatment process. The most important such constituents are DBPs. DBPs are discussed in more detail below.

7.4.1.2 sMCLs

Secondary MCLs (also known as Consumer Acceptance Levels) are based on aesthetics. Fixed sMCLs are provided by DDW for some constituents. A range of concentrations have been established for other constituents. The lower end of the range is termed a Recommended Level while the higher end of the range is termed an Upper Level. The recommended levels are more acceptable to consumers. The Upper Levels may be acceptable if it is not feasible or reasonable to deliver water that meets a lower level.

Review of the water quality data shows that treatment would be required to meet the fixed sMCL for color, iron, manganese, and turbidity, and the recommended sMCL for TDS, conductivity, chloride, and sulfate.

Iron and manganese can cause colored water episodes, detectable by consumers at concentrations below their sMCLs. Consumers can detect iron in the water at levels approaching 100 µg/L. Manganese can be detected by consumers at levels as low as 20 µg/L. IM treatment with greensand filtration is easily capable to meet these levels, which will be set as the water quality goals for this project. A summary of water quality goals that includes iron and manganese is shown in Section 7.4.4). A summary of constituents with sMCLs is shown in Appendix A.3.

RO treatment will reduce the chloride, sulfate, TDS, conductivity, and color concentrations to below their fixed and recommended sMCLs. The water quality goals for constituents with sMCLs are shown in Section 7.4.4. All other constituents with sMCL goals in Appendix A.3 do not require treatment.

7.4.1.3 Groundwater Rule

In this report, it is assumed that the groundwater is not under the direct influence of surface waters, as discussed above. Therefore, the Groundwater Rule applies to this system.

The Ground Water Rule requires sanitary surveys of groundwater systems. The Groundwater Rule also requires triggered source water monitoring or 4-log removal/inactivation of viruses. The triggered source water monitoring is based on monitoring of E. Coli, enterococci, or coliphage. With disinfection treatment, it is required to continuously demonstrate 4-log removal of viruses. Triggered source water monitoring can prove complicated, with the possibility that treatment would need to be added once the well is built. For that reason, chlorine disinfection will be included to assure compliance with the treatment requirements of the Groundwater Rule in the preliminary design (4-log virus removal/inactivation). If the well were operational, sampling could be conducted to assure that the groundwater system is likely to meet disinfection requirements of the Groundwater Rule through the triggered source water monitoring.

7.4.1.4 Disinfectants and Disinfection Byproducts Rule

The Stage 1 and 2 Disinfectants and Disinfection Byproduct Rules (D/DBPR) established pMCLs for several DBPs. DBPs with pMCLs including trihalomethanes (THMs), haloacetic acids (HAAs), bromate, and chlorite. There are four regulated THMs collectively termed total trihalomethanes (TTHM) including chloroform, bromoform, dibromochloromethane, and bromodichloromethane.

There are five regulated HAAs collectively called HAA5 including dichloroacetic acid, trichloroacetic acid, chloroacetic acid, bromoacetic acid, and dibromoacetic acid.

The D/DBPR also established maximum residual disinfect levels (MRDLs) for a number of oxidants. The oxidants include chlorine, chloramines, and chlorine dioxide. The Stage 1 D/DBPR established requirements for TOC removal using enhanced coagulation in filtration plants. The MRDLs for chlorine, chloramine, and chlorine dioxide are listed in Table 7-5. The recommended water quality goal for disinfectants and DBPs are not to exceed their pMCLs and MRDLs, as shown in Section 7.4.4.

Table 7-5. Maximum Residual Disinfectant Levels (MRDL)

Disinfectant	Units	MRDL
Chlorine	mg/L as Cl2	4.0
Chloramines	mg/L as Cl2	4.0
Chlorine Dioxide	mg/L	0.8

The generation of DBPs requires the coexistence of a disinfectant and DBP precursors. The principal DBP precursors in the water are large organic molecules and the bromide ion. Both of these precursors can be effectively removed by RO. Free chlorine is a more effective oxidant for virus inactivation than chloramine and chloramine requires a much longer contact time for the same level of virus removal. One the other hand, free chlorine decays rapidly in the distribution system and can form a significant concentration of THMs and HAA5, especially when precursors like natural organic matter are present at elevated levels. While chloramines will generate significantly lower levels of TTHM and HAA5, nitrosamines, such as N-nitrosodimethylamine (NDMA) are a byproduct of chloramine disinfection. NDMA has a DDW health-based advisory level (NL) of 10 ng/L. Despite the risk of NDMA formation, chloramines are the current industry standard for secondary disinfection where THM and HAA formation is a concern, or where chloramines are already used in the distribution system. MWD maintains a chloramine residual targeting 2.5 mg/L in the distribution system. They have done extensive testing of NDMA levels throughout their treatment plants and treated water pipelines. They have observed levels of NDMA in full compliance with the NDMA NL.

For this project, free chlorine will be used for primary disinfection to decrease the size of the clearwell. Chloramines will be used for secondary disinfection to ensure adequate residual disinfectant in the distribution system and reduce the potential for DBP formation in the distribution system. However, a fairly high level of TOC in the raw water will undergo chlorination prior to the IM treatment. Chlorine is fed upstream of IM treatment to provide optimal conditions for oxidation of manganese on the manganese dioxide surface of the greensand filters. This chlorination of the raw water could potentially form high levels of DBPs exceeding the regulatory limit. A DBP formation potential study is recommended in the future (SDS test, as discussed above).

7.4.2 Agricultural Considerations

7.4.2.1 Public Drinking Water Supply

RMWD is located in a major agricultural area. The presence of agriculture interests in the area will have a strong influence on the water quality goals established at the groundwater treatment plant. The water quality goals to be established need to consider the potential impact on plants and other agricultural crops for constituents like boron, sodium, and chloride, among others.

The SWRCB developed general water quality guidelines for standard agricultural practice in the interpretation of irrigation water quality. These guidelines were developed around the long-term influence of water quality on crop production. They represent a water quality that can be used without restriction or without special management practices. In the absence of site-specific information, these guidelines (Table 7-6) can offer direction for developing agricultural goals.

Table 7-6. Agriculture Industry Guidelines for Interpreting Irrigation Water Quality[a]

Parameter	Units	Degree of Restriction on Use[b]			Raw Groundwater Quality	Skinner WTP Treated Water Quality
		None	Slight to moderate	Severe		
EC	µS/cm	<700	700-3,000	>3,000	2,150	813
TDS	mg/L	<450	450-2,000	>2,000	2,000	482
Sodium	mg/L	<69	>69		175	79
Chloride	mg/L	<142	142-355	>355	320	85
Boron	mg/L	0.7	0.7-3.0	>3.0	0.11	—
Nitrate	mg/L-N	<5	5-30	>30	0.30	0.9
Bicarbonate	mg/L	<92	92-519	>519	—	133
pH		Normal range 6.5-8.4			8.21	8.21

(a) Adapted from Ayers 1977 and Ayers 1985. Assumes semi-arid to arid climate and low rainfall; sandy loam to clay loam soils with good internal drainage; no uncontrollable shallow water Table present within 2 meters of surface; a leaching fraction of 15-20% LF; infrequent irrigations; and a 40-30-20-10% root water uptake pattern.

(b) Full production capability of all crops, without use of special practice, is assumed when the guidelines indicate no restrictions on use. A "restriction on use" indicates that there may be a limitation in choice of crops, or special management may be needed to maintain full production capability. A "restriction on use" does not indicate that water is unsuitable for use.

The most common agricultural crops in RMWD are avocados, citrus, nursery, and nut crops. Based on research, it is known that these plants are sensitive to chloride. A previous project conducted by members of the project team found that avocado trees might develop tip burn when excessive chloride accumulates in their leaves. Other chloride sensitive plants include fruit crops (almond, apricot, banana, citrus, grapes, mango, peach), berries (including strawberry), vegetables (lettuce, onions, sweet pepper), field crops (potato, tobacco), coffee, and flowers. The SWRCB guidelines specific to particularly sensitive crops are presented in Table 7-7.

Table 7-7. Guidelines Specific to Particularly Sensitive Crops[a]

Parameter, units	Limit	Sensitive Species
EC (µS/cm)	600	Turnip
	700	Beans, Carrot, Strawberry
	800	Radish, Onion
Chloride (mg/L)	118	Avocado, Shasta Strawberry, Indian Summer Raspberry
Boron (mg/L)	0.5	Lemon, Blackberry

(a) Adapted from Ayers 1977 and Ayers 1985. Assumes semi-arid to arid climate and low rainfall; sandy loam to clay loam soils with good internal drainage; no uncontrollable shallow water table present within 2 meters of surface; a leaching fraction of 15-20% LF; infrequent irrigations; and a 40-30-20-10% root water uptake pattern.

Comparing the water quality to the guidelines, treatment will not be needed for pH, boron, and nitrate so water quality goals will be set at the agricultural guideline. On the other hand, EC, TDS, sodium, and chloride would require treatment to meet SWRCB guidelines for agricultural crops.

To appropriately develop a reasonable water quality goal for constituents requiring treatment, the imported water quality was compared with the water quality for agricultural considerations. The conductivity, TDS, sodium, and chloride in the treated water from Robert A. Skinner Water Treatment Plant (Skinner WTP) were obtained from RMWD by averaging monthly data from January 2012 to April 2015. Chloride in the imported water meets both the general guideline and guideline for sensitive crops. With respect to EC, TDS, and sodium, the imported water does not meet the general guidelines. But the imported water is already being successfully used for agricultural purposes in the area, in this case, the recommendation is to match the imported water quality. Therefore, the agricultural water quality goal for electrical conductivity, TDS, sodium, and chloride is set to match the imported water quality. The water quality goals for agriculture are presented in Table 7-8.

Table 7-8. General Agricultural Water Quality Goals

Parameter, units	Goal	Rationale
Electrical Conductivity (µS/cm)	813	Imported Water Quality
Total Dissolved Solids (mg/L)	482	Imported Water Quality
Sodium (mg/L)	79	Imported Water Quality
Chloride (mg/L)	85	Imported Water Quality
Boron (mg/L)	0.7	General Agricultural Guideline
Nitrate	5	General Agricultural Guideline
pH	6.5-8.4	General Agricultural Guideline

Based on the tables above, it was determined that the water quality goals established to meet agricultural requirements are in this case more stringent than the requirements for drinking water. For example, sodium and chloride levels are 69 and 118 for agriculture, versus no limit and 250 (recommended) for drinking water. This observation means a higher level of RO treatment would be required for agricultural considerations than for drinking water supply. However, in this project, specific water quality parameters were taken into account in developing the water quality goals to assure the treated water quality meets the requirements both for a drinking water supply and for assurance that crops will not incur incremental damage in comparison to the imported water supply.

7.4.2.2 Landscape Irrigation

Another possible application of groundwater considered on the project is landscape irrigation. As discussed above, similar levels of RO treatment will be needed to meet water quality goals for agriculture as compared to drinking water. According to California Water Code Sections 13550-13557, "the use of potable domestic water for nonpotable uses, including, but not limited to, cemeteries, golf courses, parks, highway landscaped areas, and industrial and irrigation uses, is a waste or an unreasonable use of the water…if recycled water is available." Therefore, landscape irrigation is not considered a viable alternative to application as a public drinking water supply and was not analyzed further.

7.4.3 Blending with Imported Water for Delivery to RMWD

It is important to understand the application to set appropriate water quality goals. Treated SLR groundwater blending with imported water for delivery to RMWD is a potential way to increase the water supply with a possible reduction in level of treatment required based on dilution provided by the imported water supply. It is essential to understand the imported water quality to determine the possible benefit of blending with imported water.

The primary source of water for the RMWD is imported water through the SDCWA, which provides water from MWD, through the Skinner WTP. MWD typically supplies this plant with large amounts of both SWP and Colorado River Water (CRW). The water quality of Skinner WTP treated water from 2012 to 2015 was obtained from RMWD (see Appendix A.4). The concentrations in Skinner WTP treated water were evaluated with respect to the product water quality goals proposed for the new SLR groundwater treatment facilities (see Section 7.4.4). The average concentration in Skinner WTP treated water for constituents related to aesthetic concerns is shown in Table 7-9.

Table 7-9. Average Water Quality of Skinner WTP from 2012 to 2015			
Constituents	Units	MCLs	Monthly Average from January 2012 to April 2015
Silica	mg/L	—	8.9
Calcium	mg/L	—	54
Magnesium	mg/L	—	20
Sodium	mg/L	—	79
Potassium	mg/L	—	4.0
Carbonate	mg/L	—	0.21
Bicarbonate	mg/L	—	133
Sulfate	mg/L	250 Rec 500 Upper	162
Chloride	mg/L	250 Rec 500 Upper	85
Nitrate	mg/L	45	0.9
Fluoride	mg/L	2	0.8
TDS	mg/L	500 Rec 1,000 Upper	482
Total Hardness as CaCO3	mg/L	—	220
Total Alkalinity as CaCO3	mg/L	—	109
Free Carbon Dioxide	mg/L	—	1.5
pH	—	—	8.21
Specific Conductance	µS/cm	900 Rec 1,600 Upper	813
Color	CU	15	1
Turbidity	NTU	5	0.06
Temperature	°C	—	22
Saturation Index	—	—	0.53

The summary in Table 7-9 shows that the average imported water quality is slightly below than the MCLs. However, the SWP portion has decreased substantially since 2014 due to extreme drought conditions. This has increased the amount of CRW in the Skinner WTP treated water. Certain minerals are elevated in the CRW supply compared to the SWP, as evidenced by higher TDS levels, for example. For this reason, various constituents that may be of aesthetic concern have been increasing in concentration, even exceeding the sMCLs in the case of TDS and EC. Such constituents include TDS, EC, sulfate, and chloride. The water quality changes over time are shown in Figure 7-3 and Figure 7-4. The changes in chloride, sodium, and sulfate levels are shown in Figure 7-3. There are no sMCL exceedances, but small increases in sodium and chloride have been observed, with a larger increase in sulfate concentrations. This is not surprising as CRW is known to have elevated levels of sulfate compared to SWP. The most recent sulfate data is approaching, but just under the sMCL. The same trend observed for sulfate is seen for TDS and EC, as shown on Figure 7-4. The difference is that the TDS and EC levels have exceeded sMCLs in multiple samples over the past several months. TDS has exceeded 600 mg/L, a level considerably above the sMCL of 500 mg/L. A probability plot of Skinner WTP treated water TDS concentration from Jan. 2012 to Apr. 2015 is shown in Figure 7-6. Figure 7-5 shows a median TDS level of 520 mg/L based on 2012-15 data.

As discussed above, the imported water had an average of 482 mg/L TDS concentration based on data from January 2012 to April 2015. As discussed above, the persistent drought in recent years has caused elevated levels of TDS. As shown in Figure 7-4, the TDS concentration increased as high as 639 mg/L. It is uncertain what the future TDS will be in imported water, thus 90 percent level for TDS concentration in imported water was considered in the evaluation of blending. As shown in Figure 7-5, the 90 percent level is 624 mg/L.

A blending analysis was conducted as part of the study. RO capacity was evaluated as a function of the amount of imported water required for blending and the RO bypass capacity. The analysis was performed on blending different amounts of imported water with RO permeate and/or RO bypass. The goal of this evaluation is to assure compliance with TDS MCLs. Therefore, only the TDS concentration was considered in the analysis. There were four blending scenarios considered in the analysis, involving 5, 10, 15, and 19 MGD of imported water. The 19 MGD scenario represents the projection of normal year water demand for RMWD by 2020, according to the RMWD 2010 Urban Water Management Plan. The case with no blending is also included in the results for comparison. In Table 7-10, the amount of RO design capacity and RO bypass capacity to meet the TDS sMCL are presented for the different blending scenarios.

To blend with 19 MGD imported water, a RO system has to be designed at 6.68 MGD without bypass capacity to comply with the TDS MCL. Compared with the zero blending option, the size of the RO is 0.33 to 3.76 MGD larger depending on imported water flow. Therefore, the blending option will require a higher RO design capacity and associated cost. For this reason, the blending with imported water option was not considered further.

Table 7-10. Scenario Projection of Blending Product Water with Imported Water to Comply with TDS sMCL

Blending with Imported Water, MGD	RO Capacity, MGD	RO Bypass, MGD
19	6.68	0
15	5.29	0
10	3.58	0.02
5	3.25	0.35
0	2.92	0.68

7.4.4 Recommended Water Quality Goals

Based on the discussion above, the recommended water quality goals are summarized in Table 7-11. The levels of iron and manganese, in addition to TDS and chloride, when compared to the design basis raw water quality (see Table 7-3 and Table 7-4) will require IM and RO treatment. A detailed discussion of the treatment processes required for iron and manganese removal and groundwater desalination is provided in Section 7.5.3.

Table 7-11. Design Water Quality Goals[a]

Constituent	Goal	Rationale
Iron (mg/L)	0.1	Aesthetics (colored water), Goal Set at Detection Limit
Manganese (mg/L)	0.020	Aesthetics (colored water), Goal Set at Detection Limit
TDS (mg/L)	482	Aesthetics, Goal Set to Match Imported Water Quality
Conductivity (µS/cm)	813	Aesthetics, Goal Set to Match Imported Water Quality
Chloride (mg/L)	85	Agriculture, Goal Set to Match Imported Water Quality
Sodium (mg/L)	79	Agriculture, Goal Set to Match Imported Water Quality
Sulfate (mg/L)	250	Aesthetics, Goal set at sMCL
Boron (mg/L)	0.7	Agriculture, Goal Set to SWRCB General Guideline
Apparent Color (CU)	15	Aesthetics, Goal Set at sMCL
Turbidity (NTU)	5	Aesthetics, set at sMCL

[a] Treatment goals for iron and manganese are based on consumer expectations and exceed the percentage reductions needed to meet the sMCLs.

7.5 WATER TREATMENT

Based on the design basis water quality and the water quality goals, process selection will be accomplished by identification and selection of treatment technologies that allow for the treatment train to meet water quality goals. Following process selection, a conceptual design of the treatment processes selected will be prepared. Conceptual Level Capital and O&M costs will also be presented. This Section includes (1) Process Selection, (2) Process Flow Diagram and Description, (3) Iron and Manganese Treatment, (4) RO Treatment System, (5) Disinfection and Stabilization, (6) Chemical Feed and Storage, (7) Treated Water Quality, (8) Simplified Conceptual Layout, and (9) Conceptual Level Capital and O&M Costs.

7.5.1 Process Selection

Constituents requiring treatment for the feed design basis water quality shown in Table 7-3 and Table 7-4 to meet the water quality goals in Table 7-11 are presented in Table 7-12. Based on Table 7-12, iron and manganese need greater than 95 percent removal to meet the water quality goal. Chloride and TDS need more than 75 percent removal. Sodium, conductivity, and sulfate need significant removal as well. For target constituents that need to be removed, a selection of treatment technologies was made, with a detailed explanation provided below.

Table 7-12. Design Water Quality Compared to Water Quality Goals

Constituent	Design Basis	WQ Goal	% Removal to Meet WQ Goal
Iron (mg/L)	3.5	0.1	97.1
Manganese (mg/L)	0.65	0.02	96.9
TDS (mg/L)	2,000	482	75.9
Conductivity (μS/cm)	2,305	813	64.7
Chloride (mg/L)	434	85	80.4
Sodium (mg/L)	225	79	64.9
Sulfate (mg/L)	486	250	48.6
Boron (mg/L)	0.16	0.7	0
Apparent Color (ACU)	34	15	55.9
Turbidity (NTU)	5.4	5	7.4

For iron and manganese, treatment options include sequestering, ion exchange, oxidation followed by conventional filters, and oxidation followed by greensand filters. Sequestering prevents the iron and manganese from oxidizing but does not remove them. This is only an option if the iron is in the form of ferrous iron and manganese as manganous ion. Ion exchange softening is another option for the removal of iron and manganese. However, ion exchange is only an option when the iron and manganese combined concentration is low, preferably lower than 2 mg/L. This is because high concentrations may cause precipitated iron residue buildup on the resin, decreasing the efficiency of the ion exchange process.

Another common removal process is oxidation followed by filtration with conventional granular media filters (GMF). Iron and manganese are oxidized by free chlorine upstream of the GMF. After the oxidation, soluble ferrous iron (Fe^{2+}) is oxidized to insoluble ferric iron (Fe^{3+}) and soluble manganese (Mn^{2+}) is oxidized to insoluble manganese (Mn^{4+}). The addition of chlorine will also result in the formation of a manganese oxide surface on the conventional filter that will catalyze the removal of manganese in the conventional filters. However, it is not as effective as greensand filtration, especially for manganese, and has an acclimation period to build the manganese oxide surface.

Greensand filtration has been used for decades for IM treatment. In recent years, the greensand media has been improved, resulting in lower O&M burden as the greensand does not need to be regenerated regularly. Like with the free chlorine/GMF alternative above, Fe^{2+} is oxidized to Fe^{3+} readily forming the insoluble iron hydroxide complex $Fe(OH)_3$, and Mn^{2+} is oxidized to Mn^{4+} forming insoluble manganese dioxide (MnO_2). The insoluble metals can be precipitated out in a settling tank or removed by filtration.

There are different preoxidation approaches available, such as aeration, chlorine oxidation and potassium permanganate oxidation. The oxidation reactions with aeration are too slow to be viable, as pH greater than 9.5 would be required. Potassium permanganate oxidation is more expensive than the alternatives, and it needs more operational attention due to the fact that overdosing will leave a pink tinge in the water. Free chlorine oxidation is another commonly used approach and has proven very effective with greensand filtration. Given the concentration of iron and manganese in the groundwater and the pH of the raw water, chlorine oxidation is the best and most cost effective approach for this project. Based on the rationale above, the treatment process for iron and manganese will be:

- Oxidation by Free Chlorine
- Greensand Filtration

The oxidation step will be referred to as "preoxidation" because it occurs upstream of the greensand filtration process it drives. Technologies considered for TDS and mineral removal included RO, NF, and electrodialysis. Electrodialysis was ruled out as it was shown to be less cost effective than RO in the CDM report and it is not commonly used in the water industry for groundwater desalination. Nanofiltration (NF) is more common and requires lower pressure than RO membranes, but it does not reject salt particularly well or remove monovalent ions effectively. For this reason, NF was ruled out given the high level of TDS in the groundwater and the low chloride water quality goal.

For the TDS and other inorganic constituents, RO is the most commonly used treatment process among the best available technologies for inorganic chemicals listed in California Regulations Related to Drinking Water. It is a pressure-driven membrane separation process that removes dissolved contaminants from water. It is driven by the passage of a solvent (e.g., water) through a semi-permeable membrane from a solution of higher concentration to a solution of lower concentration against the concentration gradient. This is achieved by applying pressure greater than the osmotic pressure to the more concentrated solution. RO treatment is more expensive than conventional treatment processes. Only the amount of water required to meet the TDS/mineral treatment target in RO will be fed to the RO. The rest will bypass the RO.

RO permeate is low in minerals, soft, and low in alkalinity. Waters of similar quality have been shown to be aggressive towards the distribution system. Blending with RO bypass water will improve the water quality and its corrosivity, but blending alone is typically not sufficient because the percentage of blending water is too low. Stabilization needs to be considered as part of the treatment train to prevent corrosion in the distribution system.

Disinfection, as discussed above, is needed to achieve 4-log virus removal/inactivation to comply with groundwater regulations. Therefore, disinfection will be included in the treatment train to be described below.

7.5.2 Process Flow Diagram and Description

The process flow diagram is shown in Figure 7-6. An overview of equipment capacity and purpose is shown in Table 7-13 below. Design raw water quality and treatment goals are summarized in Sections 7.3 and 7.4. The water treatment plant equipment was sized based on the water quality goals presented in Section 7.4. The maximum groundwater that the plant can treat is 4,000 afy (3.6 MGD), which was determined from groundwater modeling. The treatment plant includes the following major components:

- Iron and manganese removal with sodium hypochlorite oxidation and greensand filtration;
- Desalination via:
  - Low pressure, low energy RO membrane;
- Disinfection and stabilization in the clearwell using:
  - Sodium hypochlorite for primary disinfection, followed by addition of ammonium hydroxide to form chloramines for secondary disinfection, caustic (NaOH) addition prior to distribution to minimize the potential corrosion in distribution pipeline.

Feed water is stored in an EQ tank at low flows to provide sufficient volume for downstream processes. After oxidation with sodium hypochlorite, the IM filters are able to remove iron and manganese. The iron and manganese system rejects stream flow to a waste wash water (WWW) recovery tank for solids separation. In the WWW tank, the liquid is decanted and returned to the feed water pipe to the iron and manganese treatment process while the solids slurry is separately pumped to sludge drying beds.

Downstream of the IM filter, RO treatment is designed to reduce the TDS in the water. To maximize the RO treatment process efficiency, the treated water from the IM plant is split into three lines prior to desalination. Some of the flow bypasses the RO unit and receives no further treatment until post-treatment disinfection in the clearwell. The maximum flow of this RO bypass is 0.46 MGD due to the high salinity of raw water. The remaining flow of 3.1 MGD is fed to the RO. The RO permeate flow rate of 2.51 MGD is determined using the RO membrane salt rejection capability and the water recovery of 80 percent. The blended flow ensures a final TDS concentration of 385 mg/L (80 percent of TDS goal), and the concentration of chloride is 68 mg/L (80 percent of chloride goal) for agricultural application. All other parameters are within regulatory requirements in the clearwell as well.

Primary free chlorine disinfection is designed to obtain at least 4 log virus inactivation removal credit. Secondary disinfection with chloramines is designed to maintain the disinfectant residual in the distribution system. The product water was evaluated for corrosion control. Caustic soda (NaOH) addition is designed to ensure the Langelier Saturation Index (LSI) of the treatment plant product water is in the acceptable range for distribution. Based on LSI, the blended RO product water quality will be compatible with the existing system water quality.

Design criteria are presented below for equipment associated with IM and RO treatment. Design criteria for chemical feed and storage are presented after the sections on IM and RO treatment.

Table 7-13. Major Components in the Treatment System

Equipment	Capacity	Purpose
Equalization (EQ) Tank	50,000 gal	Provides storage to assist with controlled startup and shutdown of the plant.
IM Treatment Vessels	3 IM vessels, 10 feet in diameter by 32 feet long	Removes iron and manganese from raw water.
RO Feed Tank	125,000 gal	1. Provides additional contact time for the dechlorination of the IM treated water to protect the downstream RO membranes from oxidation. 2. Provides wash water storage volume for backwashing the IM media.
IM Vessel Back Wash Pump	1,920 gpm per cell	Provides water to perform backwashing of the IM vessels.
WWW Recovery Tank	80,000 gal	1. Holds the IM backwash volume of one vessel. 2. Provides sludge settling time.
Treated WWW Recycle Pump	375 gpm	Recycles the supernatant from the WWW recovery tank to the front of the EQ tank.
WWW Sludge Pump	50 gpm	Transfer sludge to the drying beds.
RO Membrane System	3.14 MGD 3 trains	Reduces TDS and chloride.

7.5.3 Iron and Manganese Treatment

The purpose of the IM treatment is to lower the concentrations of iron and manganese to levels that prevent the aesthetic disadvantages associated with these inorganics. IM pretreatment also will prevent iron and manganese related fouling and scaling of the RO membranes located downstream.

In the natural geology of an aquifer, the interaction between the iron-bearing soil and water dissolves the iron into the groundwater, where it is present in the reduced or ferrous form [Fe^{2+}]. Similar to iron, manganese is found in the reduced or manganous form [Mn^{2+}]. The presence of iron and manganese in water supplies can lead to several aesthetic problems. They impart a metallic taste, promote the growth of microorganisms in reservoirs and distribution systems, and stain laundry and fixtures. Iron and manganese are perceptible to consumers as colored water at low levels, as low as 0.02 mg/L for Mn.

To produce aesthetically acceptable water, the sMCLs limit iron and manganese to 0.3 mg/L and 0.05 mg/L, respectively. Unfortunately, iron and manganese can be easily detected by the pallet of some consumers and via color observed in the water at concentrations below their sMCLs. Iron is detectable at concentrations down to 0.10 mg/L and manganese is detectable at concentrations down to 0.020 mg/L. Therefore, the goal for removing these constituents should be based on their respective detection limits. The raw water quality and treatment goals necessary to design the IM treatment are summarized in Table 7-14.

Table 7-14. Design Water Quality and Recommended Goals for IM Process

Parameter	Units	Design Water Quality	Treatment Goals
Iron	mg/L	3.5	<0.1
Manganese	mg/L	0.65	<0.02

7.5.3.1 IM Treatment Process Flow

The IM treatment system consists of an oxidation system that provides continuous chlorination upstream of the pressurized media filters, as well as an equalization tank, backwashing, waste wash water recovery, sludge handling, and chemical feed and storage. The process flow diagram is shown in Figure 7-6 and an overview of equipment capacity and purpose is summarized in Table 7-13.

7.5.3.2 Equalization Tank

The EQ tank will be designed based on the maximum flow of 3.6 MGD feeding into RMWD treatment plant and a recycle flow of 0.5 MGD of supernatant. The tank will provide about 17 minutes of storage at 4.1 MGD. The storage provides contact time between the chlorine and raw water prior to the IM media and retains water in case of controlled startups and plant shutoffs. Table 7-15 shows the design criteria for the EQ tank.

Table 7-15. EQ Tank Design Criteria

Parameter	Units	Value
Storage Capacity[(a)]	gal	50,000
Height	ft	8
Diameter	ft	33
[(a)] Design based on maximum flow rate of 3.6 MGD and 0.5 MGD of supernatant.		

7.5.3.3 Preoxidation System

All of the common Fe and Mn removal processes utilize the oxidation process upstream of filtration. Oxidation changes the dissolved iron and manganese present in groundwater supplies to insoluble forms that are removed by filter media. Ferrous iron, Fe^{2+}, is oxidized to ferric iron, Fe^{3+}, which readily forms the insoluble iron hydroxide. Similarly, reduced manganese, Mn^{3+}, is oxidized to Mn^{4+}, which forms insoluble manganese oxide. The most commonly used oxidants are chlorine, potassium permanganate, ozone, chlorine dioxide, oxygen, and hydrogen peroxide. The oxidant of choice for the RMWD IM treatment is chlorine because stronger oxidants form colloidal manganese particles that are harder to remove through filtration. The oxidizing strength trend is:

$$O_3 > H_2O_2 > ClO_2 > KMnO_4 > Cl_2 > O_2$$

In the case of permanganate, dissolved Mn is actually produced from the quenching of residual permanganate, which can lead to RO fouling downstream. Sodium hypochlorite (NaOCl) is the form of free chlorine that will be dosed in the raw water upstream of the filters and EQ tank. A residual of 0.5 mg/L Cl_2 will be carried through the filters, such that 0.5 mg/L is present at the outlet of the filters.

Oxidation of ferrous iron with chlorine is effective and occurs rapidly, whereas the reaction between Mn^{2+} and free chlorine is more difficult and slower. To remove manganese, a manganese oxide coated surface is commonly employed as a filter media. This approach is called greensand filtration named after the media that is used. First, the dissolved manganese adsorbs to the oxide-coated filter media, which is gradually oxidized to manganese dioxide on the surface by the chlorine residual passing through the filter. The precipitant acts as a new surface for further adsorption of reduced Mn.

Based on corresponding redox reactions, the required free chlorine dose for the oxidation of iron is 0.64 mg Cl_2/mg Fe^{2+} and 1.30 mg Cl_2/mg Mn^{2+} for the oxidation of manganese. The presence of ammonia in the raw water will reduce the rate of oxidation for both iron and manganese, since it consumes chlorine to form chloramines. Therefore, ammonia has to be considered when determining the necessary dose of sodium hypochlorite. Table 7-16 shows the chlorine demand of the bulk water. Note that the free chlorine demand factors are higher than would be expected based on stoichiometry as a result of applying a safety factor.

Table 7-16. Free Cl2 Requirements for Oxidation of Iron, Manganese, and Ammonia

Parameter	Raw Water Quality, mg/L	Free Cl2 Demand Factor	Chlorine Demand, mg/L
Fe	3.5	1	3.5
Mn	0.65	2	1.3
NH_3-N	0.6	8	4.8

In groundwater supplies where reduced iron forms strong complexes with natural organic matter, free chlorine is unable to oxidize the iron. Fortunately, chlorine doses as 5 mg/L have been used effectively in the oxidation of organically bound iron.

A common storage facility will be used to store all the sodium hypochlorite solution utilized in the oxidation of the raw water, disinfection of the RO treated water, regeneration of the IM vessels, and removal of sodium bisulfite from the RO feed tank. During storage, sodium hypochlorite degrades to salt and oxygen. The rate of degradation is a function of solution concentration, temperature, light, pH, and concentration of heavy metals. The faster the oxidant is used, the less time it will have to degrade, so storage time should be limited. For this project, the sodium hypochlorite will be stored for 15 days. The design criteria for the oxidant dose and sodium hypochlorite storage tank are shown in Table 7-17.

7.5.3.4 Pressure Vessels

Filtration will be employed downstream of preoxidation with free chlorine. The most common IM treatment technology is greensand filtration. For this technology, an oxidant (free chlorine) residual is carried through dual media filters (see below for discussion of the media). For this project, horizontal pressure vessels will be used. Advantages of pressure filters are their ability to operate at higher terminal headloss, which translates to longer filter runs and reduced backwash requirements. Table 7-18 summarizes the design parameters of the pressure vessels obtained from the IM original equipment manufacturer (OEM) Hungerford & Terry.

The design of the vessels was based on the maximum flow of 3.6 MGD, with three filters operating at all times. In addition, the working pressure is recommended to have filters with robust shells and to protect against negative pressures.

Table 7-17. Design Criteria of Oxidant and Sodium Hypochlorite Storage

Parameter	Units	Value
Oxidant	—	Sodium Hypochlorite
Typical Dose[(a)]	mg/L	10
Oxidant feed pump capacity[(b)]	gph	10
Oxidant feed pump turndown[(b)]	—	10:1
Sodium hypochlorite strength	%	12.5
Sodium hypochlorite storage volume[(c)]	gal	5,000
Height	ft	13
Diameter	ft	8.5

(a) Pilot testing would provide confirmation of oxidant dosage and choice of oxidant.
(b) Based on the 10:1 ratio, pump can feed as low as 1 gph to 10 gph.
(c) Provides 15 days of storage.

Table 7-18. Design Criteria of Pressure Vessels[a]

Parameter	Units	Value
Number of vessels (n+1)	#	3
Pressure vessel configuration	—	Horizontal
Cells per vessel	#	2
Vessel diameter	ft	10
Vessel length	ft	32
Active surface area[b]	ft^2	320
Working Pressure	psi	100
Underdrain configuration	—	Common underdrain
Filtration rate[c]	gpm/ ft^2	2.6
Filter flow rate	gpm	832
Backwashing filtration rate[d]	gpm/ ft^2	3.9
Backwashing filter flow rate	gpm	1248
Filter run length	hr	18

(a) It should be noted that flow control is needed on the pressure vessels to assure the same flow is delivered to each vessel.
(b) Active surface area is the average filter surface area of GreensandPlus media over the depth of the media.
(c) All filters online.
(d) One filter backwashing or one filter offline.

7.5.3.5 Filter Media

Manganese greensand is a common filter medium used in the removal of iron and manganese through pressure filtration. Greensand is a processed material consisting of glauconite that is coated with manganese oxide. This surface promotes the removal of dissolved manganese through the reactions that occur between the adsorbed reduced manganese and the oxidized manganese oxide coating. Supply of greensand became difficult for Hungerford and Terry (then Inversand) in recent years. At the same time, a superior replacement product was introduced that is an exact match for existing greensand system. Numerous systems across the country have switched to GreensandPlus™ in recent years (e.g., the City of San Juan Capistrano). Greensand Plus is a processed material consisting of a special density silica sand coated with manganese oxide. Unlike greensand, which often required potassium permanganate as the preoxidant, GreensandPlus works well with free chlorine as the oxidant, which offers several advantages as discussed above. It also does not require continuous regeneration. For this project, the GreensandPlus media supplied by Hungerford and Terry will be installed in the pressure vessels. Table 7-19 shows the design criteria for the anthracite cap, GreensandPlus, and gravel support. The purpose of the anthracite cap is to remove precipitated iron present due to reactions with the preoxidant upstream of the greensand media and prevent blockage of the manganese oxide sites.

Table 7-19. GreensandPlus Filter Media Design Criteria

Parameter [a]	Value	Units
Anthracite (top layer)		
Depth	inch	12
Effective size (d_{10})	mm	0.6 - 0.8
Specific gravity	—	1.6
GreensandPlus (middle layer)		
Depth	inch	24
Effective size (d_{10})	mm	0.30 - 0.35
Specific gravity	—	2.4
Gravel Support (bottom layer)		
Depth	inch	16

[a] Other important design considerations for SBS feed are installation of a rapid mixer downstream of SBS addition and a free chlorine analyzer at the outlet of IM treatment to control SBS dosing, and free chlorine analyzer and ORP analyzer downstream of SBS dosing to confirm quenching.

7.5.3.6 Quenching System

A quenching system is necessary upstream of the reverse osmosis treatment to remove the free chlorine residual from the IM treated water to prevent damage to the RO membranes. The IM treated water will be dechlorinated with sodium bisulfite and fed to an RO feed tank, which provides contact time between the chemical and the treated water.

The stoichiometric ratio of 1.5 mg $NaHSO_3$/mg Cl_2 was utilized to determine the amount of sodium bisulfite necessary to quench the free chlorine residual in the treated water. When dechlorination is applied upstream of an RO process, a safety factor of 2 is applied to the ratio.

The design criteria for the quenching system are summarized in Table 7-20.

Table 7-20. Quenching System Design Criteria

Parameter	Units	Value
Quenching Chemical	—	Sodium bisulfite
Typical Dose[a]	mg/L	1.5[d]
Quench feed pump capacity[b]	gph	0.875
Quench feed pump turndown	—	10:1
Sodium bisulfite storage volume[c]	gal	200
Sodium bisulfite strength	%	38

[a] Designed for 3.0 MGD of treated water containing 0.5 mg/L of free chlorine.
[b] Pump can feed as low as 0.0875 gph.
[c] Based on 15 days storage.
[d] Includes a safety factor of 2 to protect the RO membranes.

7.5.3.7 RO Feed Tank

The RO feed tank contains water that has been dechlorinated upstream to avoid the possibility of oxidizing the RO membranes. The RO feed tank will be designed to continuously feed the RO system at 3.0 MGD during the span of the backwash and to have enough water to perform a complete air/water wash and backwash for a whole vessel. Before backwashing a vessel, the stored water will be chlorinated to remove residual sodium bisulfite. The elements of the backwashing system are discussed below. The design criteria for the RO feed tank are shown in Table 7-21.

Table 7-21. RO Feed Tank Design Criteria

Parameter	Units	Value
Storage Capacity	gal	125,000
Height (ft)	ft	24
Diameter (ft)	ft	30
BW chlorination chemical	—	Sodium Hypochlorite
Typical dose [a]	mg/L	1.5
Chlorine BW pump capacity	gph	0.875
Chlorine BW pump turndown	--	10:1
Chemical storage volume	gal	Same as preoxidant[b]
Chemical strength	%	Same as preoxidant[b]

[a] Designed to dose 160,000 gal of backwash water in a day that includes 4 backwashes and 1 air/water wash.
[b] See Table 7-17.

7.5.3.8 Backwashing System

The backwashing system consists of daily backwash sequences and weekly air scouring (air/water wash). The air scouring system increases the efficiency of backwashes and is recommended to prevent the formation of manganese mudballs. The presence of SBS is detrimental to the IM filters. Therefore, backwash water will be supplied from the RO feed tank, which will be chlorinated to remove any residual sodium bisulfite present due to the free chlorine quenching system upstream of the RO feed tank.

The volume of water required for every backwash will be sized to backwash a single filter, one cell at a time. While one cell is being backwashed, the other cell will not be able to filter. The IM filters will operate in a staggered manner, meaning that instead of performing consecutive backwashes once the filter run time is reached, only one of the three operating filters will be backwashed. During this time, the filtration rate will increase from 2.6 to 3.9 gpm/ft^2 to compensate for the offline (backwashing) filter.

The staggering approach will require less water demand during backwashes, which translates to a smaller WWW tank footprint. A filter will have a run time of about 18 hours, meaning that approximately four backwashes will occur in a period of 24 hours, each backwash consisting of 38,400 gallons.

On a day when a filter requires its weekly air scouring, the total water volume required will be approximately 44,800 gallons, which consists of 6,400 gal of air/water wash (AWW) and 38,400 gal of backwash. This is assuming that no more than one air/water wash can occur in a day. After a backwash, the wash water will drain to the WWW recovery tank. The design criteria for a backwash, in conjunction with air scouring, are presented in Table 7-22.

Table 7-22. IM Vessel Backwash System Design Criteria

Parameter	Units	Value
Backwash		
Backwash rate per cell	gpm/ ft^2	12
BW pump capacity[(a)]	gpm	1,920
BW run time[(b)]	min	20
Backwash volume	gal	38,400
Air/water wash		
Air wash rate	cfm/ ft^2	2
AWW rate per cell	gpm/ ft^2	4
AWW pump capacity[(a)]	gpm	640
AWW run time[(b)]	min	10
AWW volume	gal	6,400

(a) Pump capacity for each cell.
(b) Run time per vessel.

7.5.3.9 Regeneration System

In IM treatment, the greensand's oxide coating is continuously regenerated. A residual will pass through the vessels during filtration and backwashing. For safety reasons, a regeneration facility is recommended if the GreensandPlus media is ever exhausted, which is not anticipated. Table 7-23 lists the design criteria for the regeneration system. The regeneration process involves draining the pressure vessel, filling the vessel with chlorinated water from the RO feed tank, soaking the media for at least four hours, draining the waste to a regeneration tank, and rinsing the media before startup.

During the regeneration, an unusually high sodium hypochlorite dose of approximately 1,000 mg/L will be injected into the backwash line. The chlorine will be fed from the same storage tank as the preoxidant. An equalization tank is needed to slowly discharge the regeneration waste to a sewer.

Table 7-23. Design Criteria for Regeneration System

Parameter	Units	Value
Regeneration chemical	—	Sodium Hypochlorite
Typical Dose	mg/L	1,000
Chemical pump capacity	gph	600
Chemical feed pump turndown	na	10:1
Chemical storage volume2	gal	Same as preoxidant[a]
Chemical strength	%	Same as preoxidant[a]
Regeneration EQ tank volume	gal	12,500

[a] See Table 7-17.

7.5.3.10 Waste Wash Water Recovery Tank

The WWW recovery tank will be designed to account for the total volume of water used in one backwash. This will include the amount of water used during an air/water wash and backwash sequence, as well as the unfiltered water required to rinse a filter. The total capacity of the WWW recovery tank will include a depth of 2 feet for sludge accumulation.

After an assumed settling time of 3.5 hours, 98.9 percent of the water in the tank will be recycled to the front of the EQ tank, while the remaining 1.1 percent of volume will be settled as sludge. In similar waters that are low in iron but high in manganese, addition of ferric chloride is required to help the slow settling. The water from the SLR river basin contains a high content of iron, so ferric chloride is not required in the treatment process. The WWW recovery tank design criteria are shown in Table 7-24.

Table 7-24. WWW Recovery Tank Design Criteria

Parameter	Units	Value
Storage Capacity[a]	gal	80,000
Height	ft	18
Diameter	ft	28
Supernatant volume	gal	48,016
Recycle time	hr	2.1
Reclaim pump	gpm	375
Settled sludge volume	gal	534
Sludge removal time	hr	0.2
Sludge removal pump	gpm	50

[a] Allows for 3 hours of settling time, 2.1 hours of recycling time, and 54 minutes of backwashing and air water wash, based on a backwash interval of 6 hours (based on filter run times of 18 hours).

7.5.3.11 Drying Beds

The sludge volume will be pumped to drying beds, a natural dewatering process that treats the sludge through natural evaporation (Kawamura, 2000 and McGivney and Kawamura, 2008). This process requires a large footprint and the operation depends on climatic conditions. Spreading the sludge from the WWW recovery tank on drying beds will produce a dry, solid sludge that can either be disposed of or reused.

A drying bed with a concrete bed bottom is recommended to allow mechanical equipment to clean the bed and to prevent groundwater pollution through percolation. The factor that determines the feasibility of dewatering at any given site is the size of the paved bed, since the process of draining is inhibited. As a result, the only way that water will be removed is through evaporation.

Each drying bed will consist of two channels and require cleaning every 17 months. The design criteria of the beds are shown in Table 7-25.

Table 7-25. Sludge Drying Bed Design Criteria

Parameters	Units	Value
Number of drying beds	#	4
Number of independent channels per drying bed	#	2
Drying bed length	ft	175
Channel width	ft	20
Drying bed depth	ft	1.5

7.5.4 RO Treatment System

As discussed above, RO treatment is necessary to reduce the salinity of the groundwater and meet water quality goals for TDS and chloride. This design strategy will allow the treatment process to also meet water quality goals for other mineral constituents for which goals have been established (see Table 7-11). Not all of the water will require RO treatment to meet water quality goals. Therefore, a portion of the water treated by IM treatment will bypass the RO.

High RO recovery indicates a larger amount of permeate extracted from the feed water, therefore conserving the raw water resource and minimizing the amount of concentrate required for discharge. But RO recovery is often limited by the potential for fouling and scaling of the RO membranes and by the concentration of soluble salts in brine streams (concentrate). In this project, a design recovery of 80 percent is proposed based on the projection after addition of antiscalant.

Three identical trains have been designed to accommodate the potential for various groundwater well extraction flow rates. Low pressure polyamide RO membranes are recommended to achieve the balance of energy savings and treated product water volume. The blended product water quality has been evaluated. All water quality parameters achieve the water quality goals discussed in Section 7.4. The details of the RO conceptual design are shown below.

7.5.4.1 RO Bypass

The IM treated water will be split into two streams to achieve maximum production while meeting the design water quality goals. A portion of the IM treated water will be pumped into a reverse osmosis treatment system to remove any soluble salt. The treated water (RO permeate) will be blended with another portion of the IM treated water not treated by RO (RO bypass). For this project, RO bypass was set up as 15 percent to achieve 80 percent of the water quality goal in Table 7-11. The RO treatment flow paradigm is shown in Table 7-26.

Table 7-26. RO Flow Design Criteria

Parameter	Flowrate, MGD
RO Feed	3.14
RO Bypass	0.46
RO Recovery	80%
RO Permeate	2.51
RO Concentrate	0.63
Product Water	2.97

7.5.4.2 RO Feed Water Pretreatment

As the water flows through the RO membranes, dissolved constituents are left behind in a decreasing volume of water that is concentrated. There are certain constituents when present in the groundwater that create conditions for fouling and/or scaling of RO membranes. These constituents include divalent ions like sulfate, calcium, and magnesium in addition to less common constituents like strontium and barium. Another such constituent is silica, present in the groundwater at levels near 30 mg/L. IM treatment will not have an impact on the silica concentration, it requires the addition of acid and inhibitors to control the fouling and scaling problem. Table 7-27 provides the design criteria for acid and antiscalant dosing.

Typically, the addition of sulfuric acid will keep LSI in the concentrate to levels below 1.8, the level proven by practical experience to prevent problems with calcium carbonate precipitation. Silica is another inhibitor that will limit recovery. Initial process modeling of RO treatment showed that with 74 percent recovery of permeate, the silica concentration in the concentrate would be near saturation. This consideration will factor into the choice of antiscalant to be added to the process. With proper sulfuric acid and antiscalant addition, recovery of 80 percent can be achieved based on the raw water quality. The detailed projection from the antiscalant vendor after antiscalant addition is included in Appendix A.5.

Table 7-27. Acid and Antiscalant Dosing Design Criteria

Parameter	Units	Value
Acid	—	Sulfuric Acid
Estimated Dose	mg/L	36
Sulfuric Acid Storage Volume	gal	1,000
Sulfuric Acid Strength	%	100
Antiscalant	—	e.g., Vitec 1400
Estimated Dose	mg/L	1.8
Antiscalant Storage Volume	gal	300
Antiscalant Strength	%	100

7.5.4.3 RO System Design

It is understood that the groundwater extraction wells will be operated at various extraction capacities. To accommodate the potential capacity range, it is necessary to configure the system with multiple, identical, independent RO membrane trains. In this project, three identical trains will be beneficial as it allows the maintenance of two thirds of the RO system treatment capacity with any single train out of service. A 90 percent online factor is considered in the RO capacity design.

For RO element selection, a high rejection, a low- pressure membrane is recommended. Higher rejection by the RO membrane results in lower chloride levels. This allows for reduced RO sizing and system cost. Low pressure operation uses less energy.

Another important design parameter for the RO system is flux. Flux is the effective loading rate on the membranes expressed as gpd/ft^2 or gfd. The advantage of higher flux design is better RO permeate water quality and smaller RO membrane area. This comes with a tradeoff of higher energy consumption and higher fouling potential. For this project, a design flux of 12 gfd is recommended to achieve the balance of energy savings and capital cost.

To achieve the target flux in the system, multiple elements must be combined in pressure vessels. The elements are loaded in series. For this project, seven elements per pressure vessel were used. The recovery that can be achieved by a seven-element vessel is between 40 to 60 percent. Higher recoveries will require staging, which sends the brine from the first seven-element vessel to a second group of pressure vessels. For the 80 percent recovery, a 2-stage system is recommended. In addition to the membrane elements and pressure vessels, each train will be equipped with a feed pump that provides the driving pressure for the process and a concentrate control valve that controls the brine flow leaving the train and thus the recovery. The RO membrane train design criteria are provided in Table 7-28.

Table 7-28. RO Treatment System Design Criteria

Parameter	Unit	Value
Trains	—	3
Capacity per train	MGD	1.05
Online factor	%	90
Stages	—	2
Elements per vessel	—	7
No. of Vessels for Stage 1[(a)]	—	16
No. of Vessels for Stage 2[(a)]	—	8
Target flux	gfd	12

(a) Based on RO treatment projection using Toray model.

7.5.4.4 Membrane Process Selection

There are different types of membranes that were considered and analyzed for this project. They are NF membranes, low-pressure RO membranes, and brackish water RO membranes. The advantages and disadvantages for these three types of membrane are listed in Table 7-29.

NF membranes have the lowest salt rejection because they do not reject monovalent ions very well. Given the high TDS concentration in the raw water and the low chloride water quality goal in product water, NF was ruled out.

For RO membrane elements, the standard diameter and length of spiral wound membrane elements in municipal systems are 8 inches and 40 inches, respectively. Widely installed membrane elements have 400 square feet of membrane area. Therefore, a standard 8-inch diameter and 40-inch length 400 square feet membrane area element was proposed.

Six different RO elements from two major manufacturers were considered. The membrane manufacturers were Hydranautics and Toray. Their RO membrane performance was evaluated using membrane projection software, IMSDesign by Hydranautics and Toray DS2 by Toray. Several RO treatment projections were carried out with the membrane manufacturers' models. The full projection summary information is listed in Table 7-30.

Table 7-29. Characterization of Three Types of Membrane Considered for the Project

Membrane Type	Salt Rejection	Advantages	Disadvantages
Nanofiltration Membrane	91%-97%	Low feed pressure, lower post treatment requirement	High feed water pretreatment required, lower bypass rate, lower recovery
Low Energy RO Membrane	99.2%-99.6%	Moderate recovery, relatively low feed pressure	Lower recovery than high rejection RO membrane
High Rejection RO Membrane	99.7%	Highest recovery due to highest bypass rate	High feed pressure, require more intense post-treatment

Table 7-30. Overall Projection of Different RO Elements

Element	Total elements required	Average flux, gfd	Feed pressure, psi	Max RO Bypass by TDS, percent	Chloride Conc. at Max. Bypass, mg/L
TMH2-A-400C	175	11.9	93.8	15	85.7
TMG20-400	**175**	**11.9**	**108**	**17**	**85.0**
TM720D-400	175	11.9	161	19	84.1
ESPA4-LD	168	11.6	107	9.0	76.0
ESPA2-LD	168	11.6	156	17	86.1
CPA5-LD	168	11.6	206	18	85.0

The selected criteria for evaluation of the RO elements were the number of membrane elements per train, the allowable bypass flow, and the feed pressure. Based on the projection results in Table 7-30, ESPA4-LD was excluded from selection due to its low salt rejection and resultant decrease in the amount of bypass flow. When the maximum bypass volume was set for compliance with the TDS sMCL, TMH2-A-400C and ESPA2-LD were unable to comply with the chloride water quality goal. Therefore, they were excluded from selection. The remaining elements, TMG 20-400, TM720D-400, and CPA5-LD, had similar maximum RO bypass flow and resultant chloride concentration. The remaining RO elements can be differentiated by feed pressure, with higher feed pressure meaning higher energy use. TMG20-400 required substantially lower feed pressure compared to TM720D-400 and CPA5-LD. Therefore, TMG 20-400 was selected among the six options for application in the conceptual design. The RO modeling results for TMG20-400 is presented in Appendix A.6.

7.5.4.5 Clean-In-Place System

Periodic cleaning of the RO membrane is required to maintain membrane salt rejection and flux. A clean-in-place cleaning system is typical, wherein the membranes are cleaned within the pressure vessels by circulating cleaning solutions through them. For mineral scales, low pH, cleaners such as citric acid at a 2 percent concentration are typically employed. For silica, custom cleaning agents are available from vendors in both low and high pH forms.

7.5.5 Disinfection and Stabilization

The treatment plant product water will require disinfection and stabilization prior to distribution. A clearwell was designed to provide detention time and baffling to achieve the disinfection contact time (CT) requirement. The clearwell size is 35,000 gallons.

The disinfection system is divided into two major categories: primary disinfection for virus removal and secondary disinfection to maintain a residual in the distribution system. Primary disinfection of groundwater often is conducted in a clearwell with free chlorine. The contact time in the clearwell and the chlorine residual at the outlet of the clearwell need to be calculated to achieve 4 logs of virus inactivation, according to the Environmental Protection Agency (EPA) CT tables. Sodium hypochlorite will be drawn from the same sodium hypochlorite storage system that is used for preoxidation upstream of the IM vessels, backwash chlorination and regeneration. Following the sodium hypochlorite dosing, rapid mixing is required to ensure adequate disinfection. For this project, disinfection design criteria are listed in Table 7-31.

For groundwater, free chlorine alone is often used for disinfection due to the low TOC in typical groundwater. For this project, chloramines were selected for secondary disinfection. This was due to elevated TOC levels observed in the limited amount of sampling conducted for groundwater in the study area that could lead to THM formation. It was also selected to match the approach used by MWD, which provides imported water to RMWD at present. Chloramine often is used for secondary disinfection due to its stability in the distribution pipeline. Another benefit is fewer THMs and HAAs are formed with chloramines. Chloramines can be formed by the addition of aqueous ammonia (ammonium hydroxide) to the outlet of the clearwell, which will react with the residual free chlorine to form chloramines.

It is recommended that a chlorine analyzer be used for both the clearwell feed and outlet. The chlorine analyzer for the clearwell feed allows for determination of chlorine decay through the clearwell. The clearwell outlet chlorine analyzer is used for controlling the chlorine and ammonia dose and for CT compliance reporting.

Stabilization is required to minimize corrosion of the distribution piping and household plumbing, given the corrosivity of the product water, which is a blend of RO permeate and groundwater high in hardness, alkalinity, and minerals. Stabilization could be achieved with caustic soda and/or orthophosphate addition. A degasifier can be considered if needed to reduce caustic requirements. The caustic dose can be controlled by downstream pH measurement, with rapid mixing in between the dose point and the analyzer sample location to achieve a low coefficient of variation (e.g., 5 percent). If orthophosphate addition is needed, it will be flow paced, with the dose set by the operator.

Table 7-31. Disinfection Design Criteria

Parameter	Units	Value
Temperature	°C	>= 15
pH	—	6-9
T10/T baffling efficiency	—	0.3
Safety factor	—	0.2
Residual free chlorine	mg/L	1.0
Flowrate	MGD	2.99
Disinfectant Chemical	—	Sodium Hypochlorite
Estimated Typical Dose, disinfectant	mg/L	2.5
Assumed Disinfectant Strength	%	12.5
Disinfectant Storage Volume	gal/year	Same as preoxidant feed
Clearwell Volume for CT, minimum	gal	35,000
Ammonia	—	Aqua Ammonia
Estimated Typical Dose, ammonia	mg/L	0.6
Assumed Ammonia Strength	%	19
Ammonia Storage Volume	gal	250

Note: Chlorine decay varies with flowrate. Therefore, chlorine dosing and ammonia dosing will be controlled using the effluent chlorine analyzer.

Standard practice for minimizing corrosion of cement mortar lined steel pipelines is to maintain calcium carbonate saturation. This can be accomplished by maintaining a slightly positive LSI. Thus, an LSI goal of 0.1 to 0.5 is recommended for stabilization of the product water. At the same time, calcium carbonate precipitation potential is often used (CCPP) and a CCPP in the range of 4-10 mg/L was targeted in line with industry practice.

For this project, caustic soda was designed for stabilization. The addition of caustic soda raises the pH and increases LSI. Numerous water utilities with waters of similar quality use pH adjustment alone to meet corrosion control water quality goals. The RO permeate water quality used in the analysis is based on the TMG20-400 modeling result, which was discussed in Section 7.5.4.4. The design criteria for the stabilization system are shown in Table 7-32.

7.5.6 Pumps and Tanks

Design criteria for pumps and tanks are presented. This includes general pumps and tanks. It also includes chemical feed pumps and storage tanks.

Table 7-32. Stabilization System Chemical Dosing Design Criteria

Parameter	Units	Value
Alkali chemical	—	Sodium Hydroxide
Estimated Dose	mg/L	45
Sodium Hydroxide Storage Volume	gal	6,000
Sodium Hydroxide Strength	%	25

7.5.6.1 General Pumps and Tanks

The design criteria and materials of construction recommended for pumps discussed above are summarized in Table 7-33. This includes the raw groundwater freed pump, pumps associated with IM treatment, and the treated water distribution pumps. The design criteria for tanks discussed above are presented in Table 7-34.

7.5.6.2 Chemical Feed Pumps and Chemical Storage Tanks

The properties of chemicals used in the treatment plant conceptual design are listed in Table 7-35. To design the chemical storage, 15 days of storage time was assumed. In addition to chemical consumption and chemical storage, chemical feed rate was evaluated. The design criteria for chemical feed pump sizing are summarized in Table 7-36. The parameters that serve of as the basis of sizing the chemical storage tanks are summarized in Table 7-37 along with the tank sizes. The chemical dosing shown in Table 7-37 is broken down by stage in the treatment train. For this reason, certain chemicals are fed at different doses throughout the treatment train and appear multiple times in Table 7-37 (e.g., NaOCl). For this reason, the tank sizes shown in the last column of Table 7-37 may look larger than what is required for an individual location, as they are based on the sum total of the chemical storage volume required for all dosing locations.

Table 7-33. Sizing for Pumps

	Total Flow, gpm	Pressure, psi	Qty[a]	Flow/Pump, gpm	Power, Hp
Plant Raw Groundwater Feed Pump					
Feed Pump	2500	30	2	1250	26
Feed Pump VFD			2		26
Pumps for IM Treatment					
IM Backwash Pump	1920	30	2	960	20
IM Backwash Pump VFD			2		20
IM Regeneration Pump	640	30	1	640	13
IM Regeneration Pump VFD			1		13
WWW Recycle Pump	Included in IM OEM				
WWW Recycle Pump VFD			2		4
WWW Sludge Pump	230	30	1	230	5
WWW Sludge Pump VFD			1		5
Treated Water Distribution Pump					
Distribution Pump	2100	120	4	525	44
Distribution Pump VFD			4		44

[a] Redundancy in pumping equipment was not considered at this level of design but was included in the cost estimating contingency.

Table 7-34. Summary of Sizing and Materials for Tanks

Tanks	Capacity, gallon	Quantity	Material
Tanks for IM Treatment			
EQ Tank	50,000	1	Bolted Steel
RO Feed Tank	125,000	1	Bolted Steel
WWW Storage Tank	80,000	1	Bolted Steel
Regeneration Tank	12,500	1	HDPE

Table 7-35. Properties of Chemicals Used in the Design

Chemical Data	$NaOCl$	NH_4OH	$NaHSO_3$	H_2SO_4	$NaOH$
Name	Sodium Hypochlorite	Aqua Ammonia	Sodium Bisulfite	Sulfuric Acid	Sodium Hydroxide
Concentration[a]	12.5%	19%	40%	98%	25%
Unit Weight, lb/gal	10.1	7.76	10.8	15.3	12.8
Basis for Dosage	as Cl_2	as NH_3-N	as $NaHSO_3$	as H_2SO_4	as $NaOH$
Unit Price, $/gal	1.10	1.05	2.35	2.39	0.66
Unit Price, $/lb	0.87	0.71	0.55	0.160	0.21

[a] Dilution of sodium hypochlorite stock solution to avoid problems with chlorate formation, especially at elevated temperatures.

Table 7-36. Summary of Sizing of Chemical Feed Pumps

Chemical	Flow, MGD	Dose, mg/L	Feed, gph	Pressure, psi	Quantity[a]
IM Preoxidation					
$NaOCl$	3.6	10.0	9.9	50	1
IM Quenching					
$NaHSO_3$	3.14	1.5	0.39	50	1
IM Backwash					
$NaOCl$	2.8	1.5	1.14	50	1
IM Regeneration					
$NaOCl$	0.9	1000	248	50	1
RO Pretreatment					
Antiscalant	3.14	1.83	0.7	—	—
H_2SO_4	3.14	18.96	2.7	—	—
RO Post Treatment					
$NaOH$	2.97	45	14.5	50	1
Disinfection					
$NaOCl$	2.97	2.5	2.0	50	1
NH_4OH	2.97	0.6	0.4	—	—

[a] Redundancy in pumping equipment was not considered at this level of design but was included in the cost estimating contingency.

Table 7-37. Summary of Sizing for Chemical Storage Tanks

Chemical	Flow, mgd	Dose, mg/L	Chemical Required, gpd	Days of Storage	Storage Volume Required, gal	Tank Size, gal	Tank Material
IM Preoxidation							
NaOCl	3.6	10.0	238	15	3570	5000	HDPE
IM Quenching System							
$NaHSO_3$	3.14	1.5	9.5	15	143	200	HDPE
IM Backwashing System							
NaOCl	2.8	1.5	1.5	15	23	5000	Same
IM Regeneration System							
NaOCl	0.9	1000	5.5	15	83	5000	Same
RO Pretreatment							
Antiscalant	3.14	1.8	15	15	223	275	Totes
H_2SO_4	3.14	36	33	15	957	1000	Mild Steel
Disinfection and Stabilization							
NaOH	2.97	45	347	15	5207	6250	HDPE
NaOCl	2.97	2.5	48	15	715	5000	Same as above
NH_4OH	2.97	0.6	13	15	202	250	Carbon Steel

7.5.7 Treated Water Quality

The product water quality after stabilization represents the treated water quality from the IM/RO groundwater treatment plant. The finished water quality from the clearwell for those constituents removed by RO is summarized in the last column of Table 7-38 below. As discussed iron, manganese, and TDS also meet their water quality goals. Therefore, the treated water quality meets all water quality goals.

Table 7-38. Water Quality Before and After Stabilization

Parameters	Units	Before Stabilization			Treated Water Quality After Stabilization
		RO Bypass	RO permeate	Blend	
Calcium	mg/L	201	4.0	34	34
Magnesium	mg/L	81	1.6	14	14
Sodium	mg/L	220	12	44	70
Potassium	mg/L	8.8	0.73	1.9	1.9
Ammonia-N	mg/L	0.3	0.02	0.07	0.07
Chloride	mg/L	434	15	77	77
Sulfate	mg/L	490	9.3	81	81
Nitrate as NO_3	mg/L	0.55	0.06	0.13	0.13
Fluoride	mg/L	0.4	0.04	0.08	0.08
pH	pH units	7.4	5.55	6.24	8.20
LSI	pH units	0.49	-4.14	-1.97	0.35

7.5.8 Simplified Conceptual Layout

A simplified conceptual layout is presented in Figure 7-7. The footprint was evaluated by considering the footprint of equipment associated with each individual treatment process. The estimated footprint for each piece of equipment in square feet is listed in Table 7-39.

Table 7-39. Estimated Required Footprint by Main Treatment Components

Treatment Components	Area Required, Estimated Net, ft^2
EQ Tank	1,200
IM Vessels	2,000
Wash Water Recovery Tank	900
Regeneration Tank	200
RO Feed Tank	1,000
RO Treatment Skid	3,000
Clearwell	1,000
Solid Drying Bed	35,000
Chemical Storage Area	7,500
Total	51,800 (1.2 acres)

7.5.9 Conceptual Level Capital and O&M Costs

Conceptual level capital and O&M costs were estimated. An opinion of probable construction cost (OPCC) was developed based on the treatment train described in Section 7.5. An O&M cost was developed in consideration of what it takes to operate the treatment train. It is broken down according to the IM treatment system including both preoxidation, greensand filtration, and ancillary processes. Costs for RO treatment and associated items are presented. There are also prices included for pumps.

7.5.9.1 Conceptual Capital Cost

The OPCC for IM treatment, RO treatment, and post treatment (disinfection and stabilization) is in shown in Table 7-40. The preliminary equipment cost from Hungerford and Terry (H&T) for the parts of IM treatment used in development of the OPCC is included in Appendix A.7. The preliminary equipment cost for RO treatment from Biwater used in development of the OPCC is included in Appendix A.8. The cost of additional items presented earlier in Section 7.5 and not included in the IM treatment and RO treatment quotes were determined based on cost estimating practice.

Table 7-40. OPCC for the SLR GW Treatment Plant

Treatment Process	OPCC, dollars
IM Treatment	4,190,000
RO Treatment	3,340,000
Post Treatment	331,000
Total	$7,860,000

7.5.10 Operation and Maintenance Cost

Chemical, power and maintenance costs were estimated for the treatment train. The detailed O&M estimation is listed in Table 7-41. The maintenance cost refers to maintenance of the equipment. Only those components listed in Table 7-41 were included in the O&M cost estimate. Chemical costs are annual costs estimated based the chemical feed design criteria provided above. Power costs were estimated based on pump size. Maintenance includes greensand media replacement, mechanical items associated with IM treatment, and RO membrane element replacement.

Table 7-41. O&M Cost Estimate for the Treatment System				
Treatment Process	Annual Chemical Costs, dollars	Annual Power Costs, dollars	Maintenance Costs, dollars	Subtotal, dollars
IM Treatment	99,000	73,000	41,000	213,000
RO Treatment	75,500	314,000	50,400	440,204
Post Treatment	108,000	113,000	—	220,436
Total Chemical and Power Costs	282,392	500,000	91,000	874,000

7.6 WELLS, PUMP STATIONS, AND CONVEYANCE PIPELINES

This section describes the wells pump stations, raw water conveyance pipelines and brine disposal pipelines for Alternatives 1 and 2.

7.6.1 Wells

Under Alternatives 1 and 2, groundwater would be pumped from a network of approximately 18 wells distributed along the portion of the San Luis Rey Valley Groundwater basin within the study area tapping into the alluvial aquifer. Figure 5-2 shows the locations of the 18 wells evaluated in the numerical model. For preliminary design purposes, these wells were assumed to operate at an average rate of approximately 140 gpm or 0.2 MGD per well and would produce an average annual flow of 4,000 afy or 3.6 million gallons per day (MGD) over the entire wellfield. The 18 wells were assumed to have 8-inch inside diameter PVC casing constructed in 16-inch diameter borings drilled to an average depth of approximately 70 feet. The wells will be equipped with submersible pumps with five-inch bowls.

Table 7-42 provides the estimated construction cost for the wells.

Table 7-42. Summary of Estimated Well Construction Costs		
	Per Well, dollars	18 Wells, dollars
Estimated Construction Cost	250,000	4.5 M

7.6.2 Alternative 1: Proposed Bonsall Basin Groundwater Desalination Plant

This alternative transports brackish groundwater from the well field described in Chapter 5 along the San Luis Rey River area to be treated at a Proposed Bonsall Basin GDF. It is estimated that 18 wells would each produce a brackish groundwater flow of approximately 200,000 gpd (140 gpm per well). The total flow rate to be conveyed to the GDP is 3.6 MGD. After treatment at the GDP, a booster pump station will pump approximately 0.63 MGD of brine solution to the San Luis Rey WRF.

7.6.3 Preliminary Alignment

The general alignment of Alternative 1 is shown on Figure 7-1. The alignment consists of two segments: approximately 30,000 feet (5.75 miles) of raw water pipeline (brackish groundwater) and approximately 45,000 feet (8.5 miles) of brine line. The raw water pipeline segment begins at the San Luis Rey River, just east of I-15 and proceeds westerly generally along the Highway 76 corridor approximately 5.75 miles to the Proposed Bonsall Basin GDF generally located near Highway 76 and Camino Del Rey. After treatment, the resulting brine solution would be pumped westerly generally along the Highway 76 corridor for approximately 5.7 miles to College Boulevard, than northerly along College Boulevard approximately 0.85 miles to North River Road, then westerly along North River Road approximately 1.95 miles to the San Luis Rey WRF.

This preliminary evaluation did not review or determine permitting requirements, the need for temporary or permanent easements, or discussions with Caltrans to determine feasibility of pipeline construction within Caltrans right of way. Alternative alignments parallel to the Caltrans Highway 76 corridor are available, if necessary due to possible restrictions for construction within Caltrans Highway 76.

7.6.4 Hydraulics

Flow in the raw water pipeline increases with each well connection starting with the first well pumping 0.2 MGD up to a maximum flow of 3.6 MGD. For this study, the 30,000-foot long segment was subdivided into multiple smaller pipe segments ranging in size from 4 inches in diameter to serve the first well site up to a 14 inches in diameter to convey the maximum combined well flow of 3.6 MGD.

The 45,000-foot brine pipeline will convey 0.63 MGD. A single booster pump station located at the GDP will pump the brine solution through an 8-inch diameter pipeline to the San Luis Rey WRF.

Table 7-43 presents a summary of the pipeline segments for Alternative 1.

Figure 7-8 presents a simplified hydraulic profile of the Alternative 1 alignment. The elevations are preliminary based on Google Earth™ data and will vary depending on the final pipeline alignment.

Table 7-43. Summary of Alternative 1 Pipeline Segments

Diameter	Flow, MGD	Velocity, fps	Length, feet	Headloss, feet
Raw Water Pipeline Segment				
14	3.60	5.21	5,000	33
14	3.00	4.34	5,000	23
12	2.40	4.73	5,000	33
12	1.80	3.55	5,000	19
10	1.20	3.40	5,000	22
8	0.60	2.66	2,000	7
6	0.36	2.84	2,000	11
4	0.18	3.19	1,000	11
Brine Line Segment				
8	0.63	2.79	45,000	179

7.6.5 Pipeline Construction

Pipe Material. It is assumed that pipeline materials for all pipeline segments would consist of either polyvinylchloride (PVC) (AWWA C-900), or HDPE (AWWA C-906). PVC pipeline joints can be either push-on gasketed joints or fused PVC joints, and HDPE pipeline is typically installed with fused joints. Both pipe materials are cost-effective and can accommodate the anticipated pressures and water quality.

Construction Methods. Most of the pipeline would be constructed using traditional open cut (trench) construction methods. Trenchless construction methods would be required for crossing streams, major intersections, and other large utilities. It is anticipated that bore and jack construction would be used for small crossings (generally shorter than 200 feet long) and horizontal directional drill (HDD) construction would be used for longer crossings. For instance, HDD would likely be the most practical method for crossing under the San Luis Rey River.

Appurtenances. For pipeline isolation and maintenance, a main live gate valve will be installed along the pipeline alignment at half-mile intervals. Air and vacuum valves will be installed at major high points and blow offs will be installed at major low points to dewater the pipeline for maintenance or repairs.

7.6.6 Alternative 2: Proposed Bonsall Basin Groundwater Conveyance Facilities

This alternative transports brackish groundwater from the well fields along the San Luis Rey River area all the way to the Mission Basin GPF for treatment. As in Alterative 1, it is estimated that 18 wells would each produce a brackish groundwater flow of 200,000 gpd (140 gpm per well). The total flow rate to be conveyed is 3.6 MGD. Due to the length of the pipeline a booster pump station is required.

7.6.7 Preliminary Alignment

The general alignment of Alternative 2 is shown on Figure 7-1. The alignment consists of two segments: approximately 30,000 feet (5.75 miles) of raw water pipeline located near the proposed well field and approximately 50,000 feet (9.5 miles) of raw water conveyance pipeline. The entire raw water pipeline begins at the San Luis Rey River, just east of I-15 and proceeds westerly generally along the Highway 76 corridor approximately 13.1 miles to Mission Avenue, then westerly along Mission Avenue approximately 1.5 miles to Fireside Street, than northwesterly approximately 0.5 miles to the Mission Basin GPF.

This preliminary evaluation did not review or determine permitting requirements, the need for temporary or permanent easements, or discussions with Caltrans to determine feasibility of pipeline construction within Caltrans right of way. Alternative alignments parallel to the Caltrans Highway 76 corridor are available, if necessary due to possible restrictions for construction within Caltrans Highway 76.

7.6.8 Hydraulics

Flow in the raw water pipeline segment located within the well field area increases with each well connection starting with the first well pumping 0.2 MGD up to a maximum flow of 3.6 MGD. For this study, the 30,000-foot long segment was subdivided into multiple smaller pipe segments ranging in size from 4 inches in diameter to serve the first well site up to a 14 inches in diameter to convey the maximum 3.6 MGD flow.

After the well field segment, a 16-inch diameter conveyance pipeline will convey the 3.6 MGD flow approximately 50,000 feet to the Mission Basin GPF. Due to the long length (and resulting head loss), a single booster pump station is required. The pump station should be located generally near the midpoint of the overall alignment; the exact location can vary somewhat.

Table 7-44 presents a summary of the pipeline segment for Alternative 2.

Figure 7-8 presents a simplified hydraulic profile of the Alternative 2 alignment. The elevations are preliminary based on Google Earth data and will vary depending on the final pipeline alignment.

Table 7-44. Summary of Alternative 2 Pipeline Segments

Diameter	Flow, MGD	Velocity, fps	Length, feet	Headloss, feet
Raw Water Pipeline Segment				
14	3.60	5.21	5,000	33
14	3.00	4.34	5,000	23
12	2.40	4.73	5,000	33
12	1.80	3.55	5,000	19
10	1.20	3.40	5,000	22
8	0.60	2.66	2,000	7
6	0.36	2.84	2,000	11
4	0.18	3.19	1,000	11
Raw Water Conveyance Pipeline Segment				
16	3.60	3.99	50,000	170

7.6.9 Pipeline Construction

The pipe materials, construction methods, and appurtenances for Alternative 1 would be the same as those described in Alternative 1.

7.6.10 Construction Cost Estimate—Alternatives 1 and 2

Table 7-45 presents a summary of the estimated construction costs for Alternatives 1 and 2. Detailed cost estimates for each alternative are presented in Appendix B. The estimates includes allowances for contractor mobilization, traffic control, sheeting and shoring, valves, air valves, blow offs, trench paving, and well connections. In addition, several trenchless construction crossings using HDD and bore and jack methods were estimated. The cost estimate includes a 30 percent construction contingency based on the preliminary evaluation. The cost estimate does not include allowances for temporary or permanent easements or for asphalt concrete pavement overlay which may be a requirement of construction by the local agency.

Table 7-45. Summary of Estimated Construction Costs

	Alternative 1, dollars	Alternative 2, dollars
Estimated Construction Cost	18.7 M	26.1 M

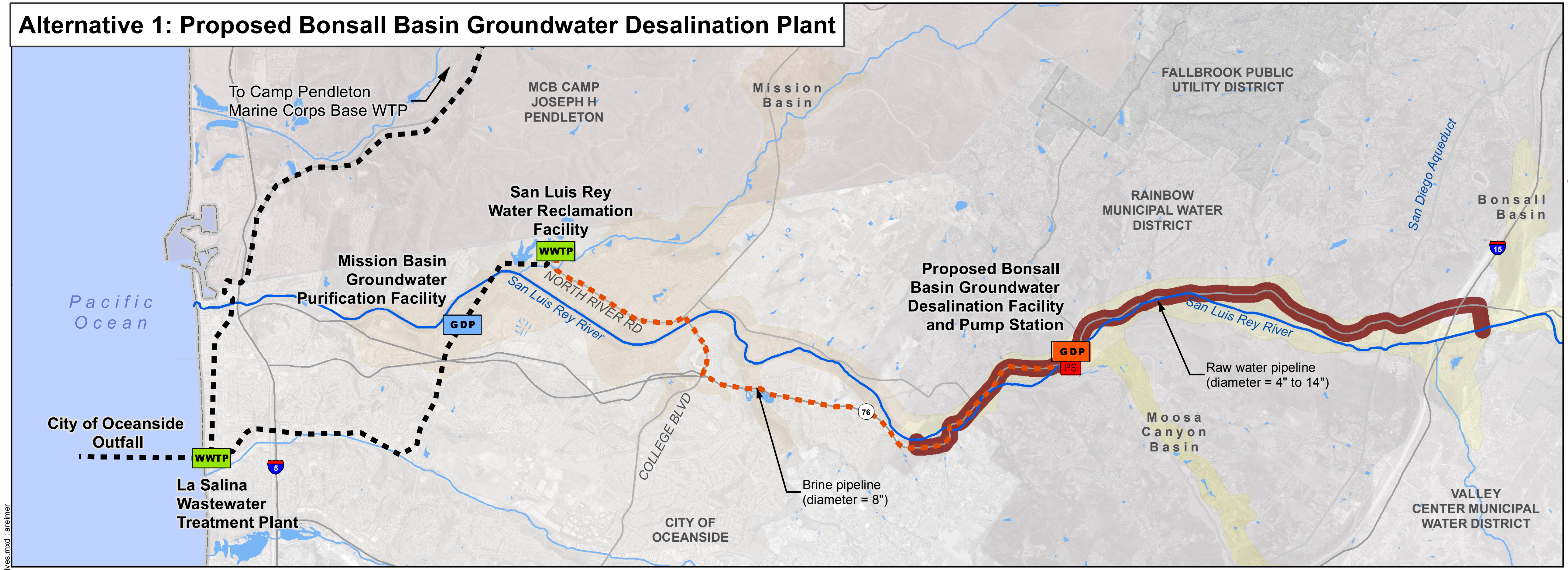
7.7 ESTIMATED CAPITAL AND O&M COSTS

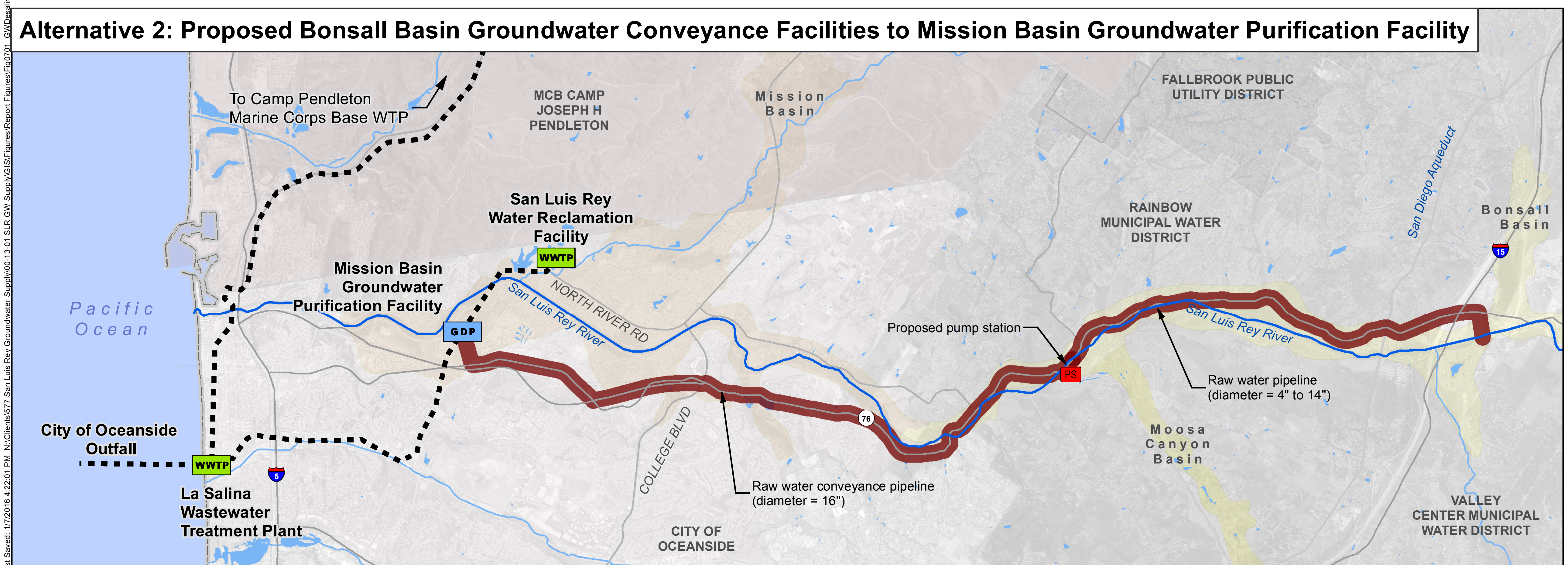
Table 7-46 summarizes the capital and O&M costs for Alternatives 1 and 2 of the Proposed Groundwater Supply Project.

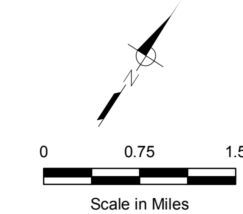
Table 7-46. Engineer's Opinion of Probable Cost[(a)]

	Alternative 1	Alternative 2
Treatment Plant Construction Cost[(b)]	$13,488,000	$0
Well Construction Cost[(c)]	$7,722,000	$7,722,000
Raw Water and Brine Disposal Pipeline Construction Cost[(d)]	$32,089,000	$44,788,000
Estimated Project Capital Cost[(a)]	$53,299,000	$52,510,000
Estimated Annual O&M Cost[(e)]	$874,000	$787,000
Present Value Over 30-year Period[(f,g)]	$110,377,826	$105,738,814
Proposed Groundwater Project 30-year Yield, acre-feet	120,000 af	120,000 af
Unit Cost, dollars per acre-feet	$920/af	$881/af

(a) All costs include a 20 percent estimating contingency, a 10 percent construction contingency, and a 30 percent allocation for other project costs such as administration, construction management, and engineering services during construction. Costs are based on the September 2015 ENR 20-City CCI of 10,065.
(b) Refer to Table 7-40, plus contingencies and other project fees.
(c) Refer to Table 7-42, plus contingencies and other project fees.
(d) Refer to Table 7-45, plus contingencies and other project fees.
(e) Assumes O&M costs, including payments to the City of Oceanside, are 90 percent of Alternative 1 O&M costs.
(f) Includes maintenance and replacement cost of 1 percent of capital cost per year.
(g) Present value of future O&M costs factored by net discount rate of 2 percent per year.







Alternative Alignments and
- Proposed Raw Water Conveyance Pipeline
- Proposed Brine Line
- PS Proposed Pump Station
- GDP Proposed Bonsall Basin Groundwater Desalination Plant

Water Treatment and Conveyance
- WWTP Wastewater Treatment Plant
- GDP Existing Groundwater Desalination Plant
- Existing Brine Line and Outfall

Groundwater Basins
- Bonsall Basin
- Mission Basin
- Moosa Canyon Basin
- Pala Basin

Hydrologic Features
- San Luis Rey River
- Escondido Canal
- San Diego Aqueducts
- Vista Canal
- Natural Waterway

Notes:
1. Under Alternative 1 brackish groundwater would be treated at a proposed new groundwater desalination plant located in the Bonsall Basin. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin and conveyed to the proposed Bonsall Basin Groundwater Desalination Facility for treatment. Brine disposal would be via a proposed new brine line extending from the proposed Bonsall Basin Groundwater Desalination Facility to the City of Oceanside San Luis Rey Water Reclamation Facility, then through the existing brine line to the City of Oceanside Ocean Outfall.
2. Under Alternative 2 brackish groundwater would be treated at the existing City of Oceanside Mission Basin Groundwater Purification Facility. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin, and conveyed to the Mission Basin Groundwater Purification Facility for treatment. Brine disposal would be via the existing brine line to the City of Oceanside Ocean Outfall.

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Figure 7-1

Groundwater Conveyance and Desalination Alternatives

Rainbow Municipal Water District
Groundwater Supply Study

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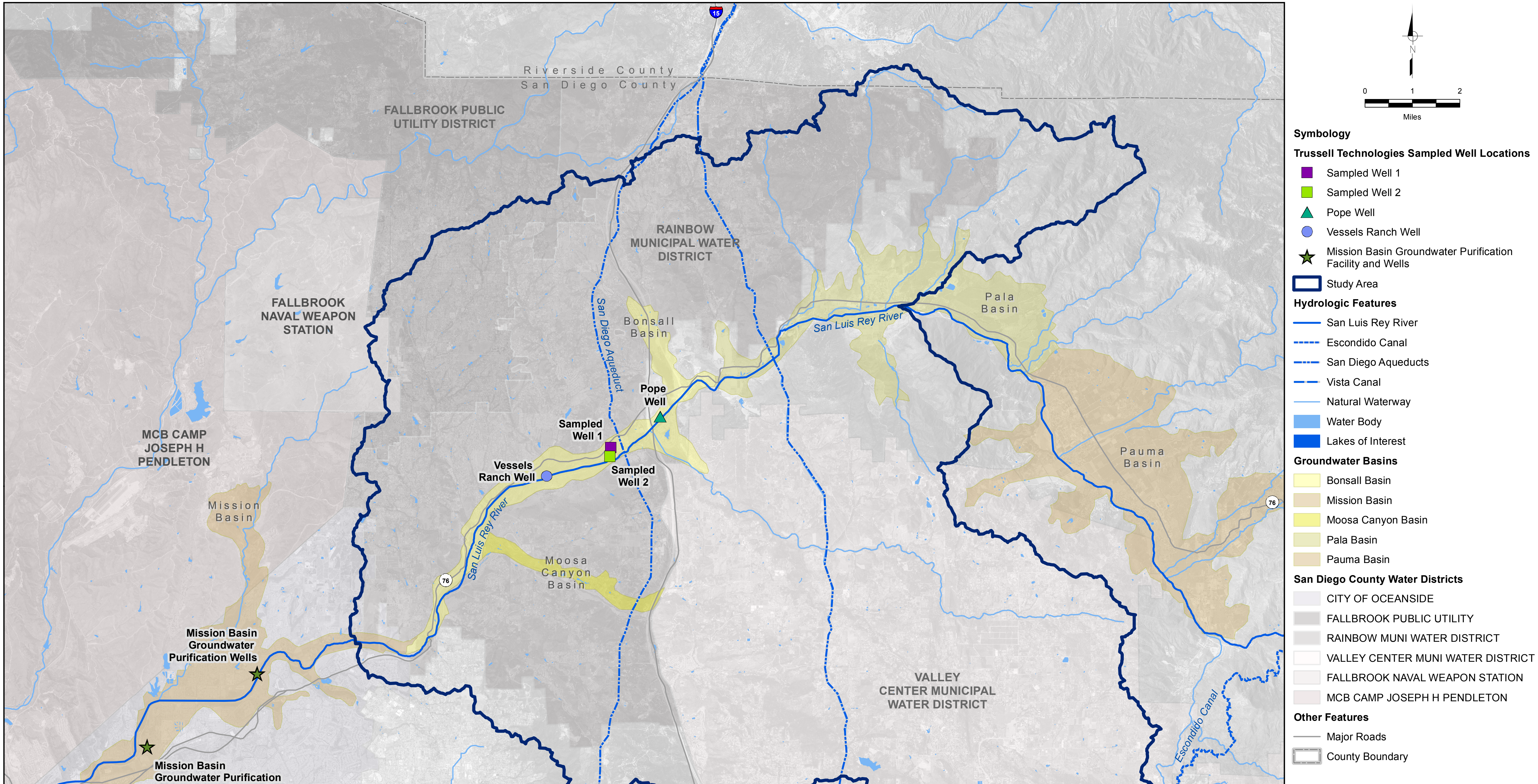

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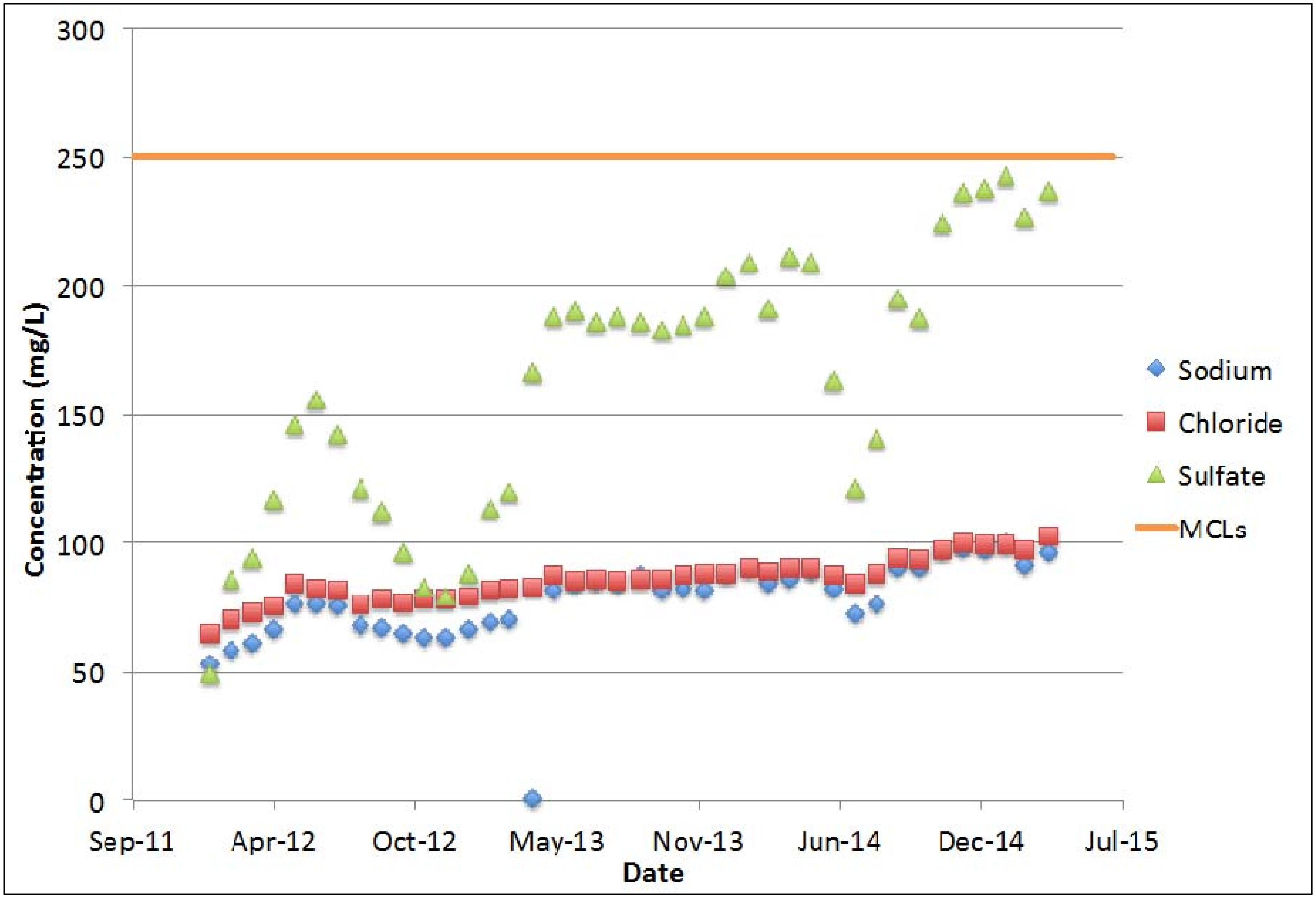
Figure 7-2

Groundwater Sampling Locations

Rainbow Municipal Water District
San Luis Rey Groundwater Supply



RAINBOW Municipal Water District Committed to Excellence

WEST YOST ASSOCIATES Consulting Engineers

Figure 7-3

Imported Water Sodium, Chloride, Sulfate Monthly Data and MCLs

Rainbow Municipal Water District
Groundwater Supply Study

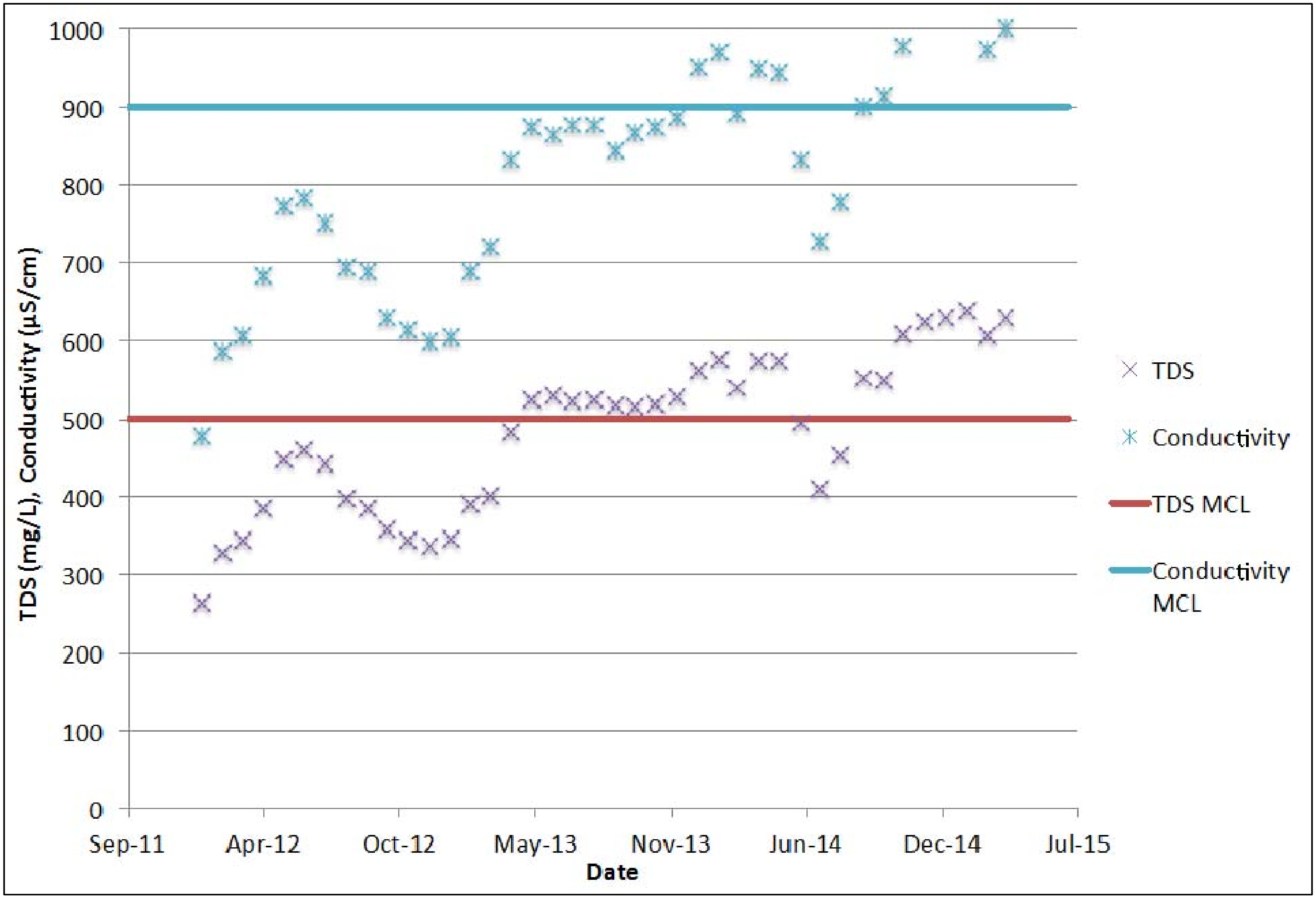


Figure 7-4

Imported Water TDS, Conductivity Monthly Data and MCLs

Rainbow Municipal Water District
Groundwater Supply Study

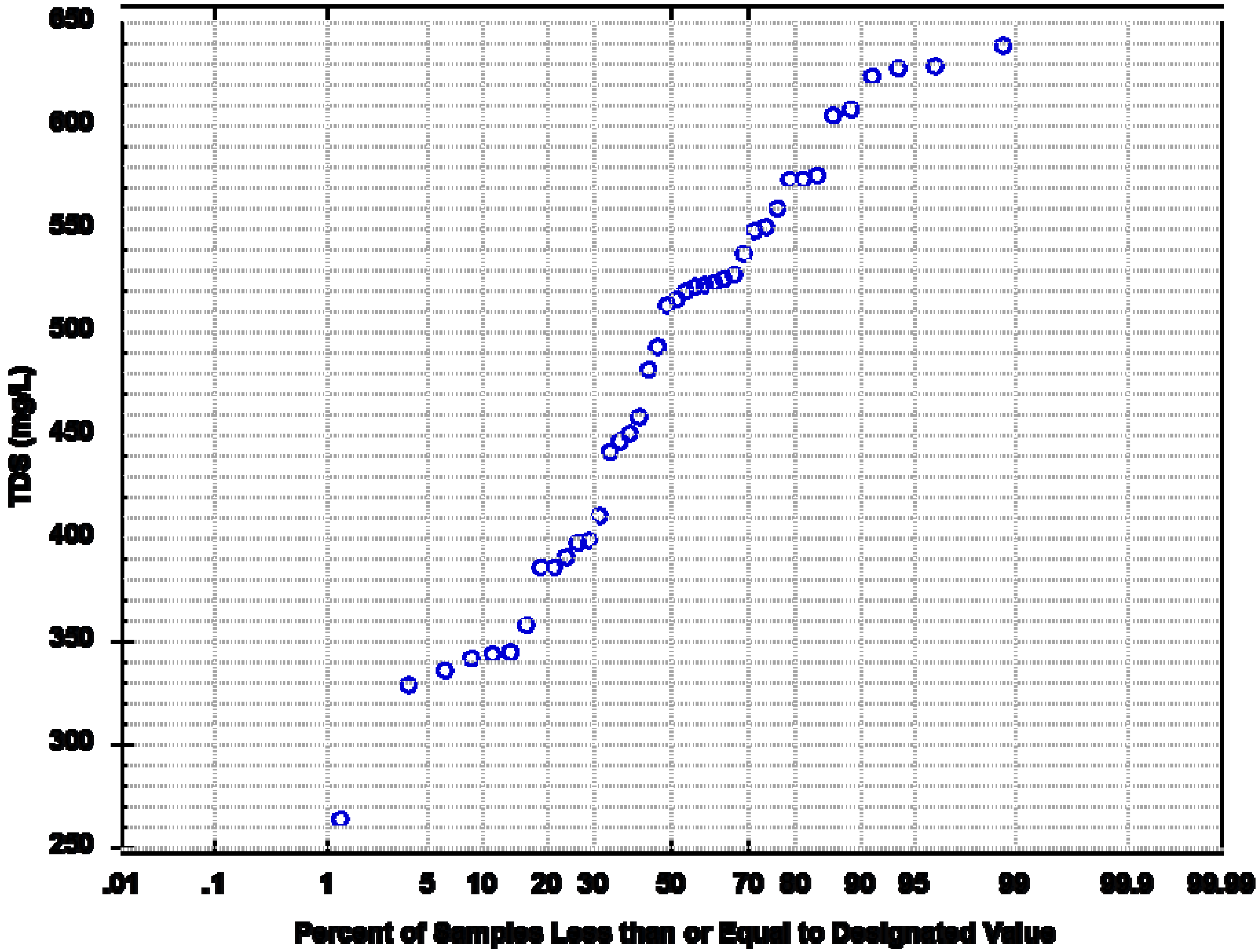


Figure 7-5

Probability Plot of Skinner WTP Treated Water TDS Concentration from January 2012 to April 2015

Rainbow Municipal Water District
Groundwater Supply Study

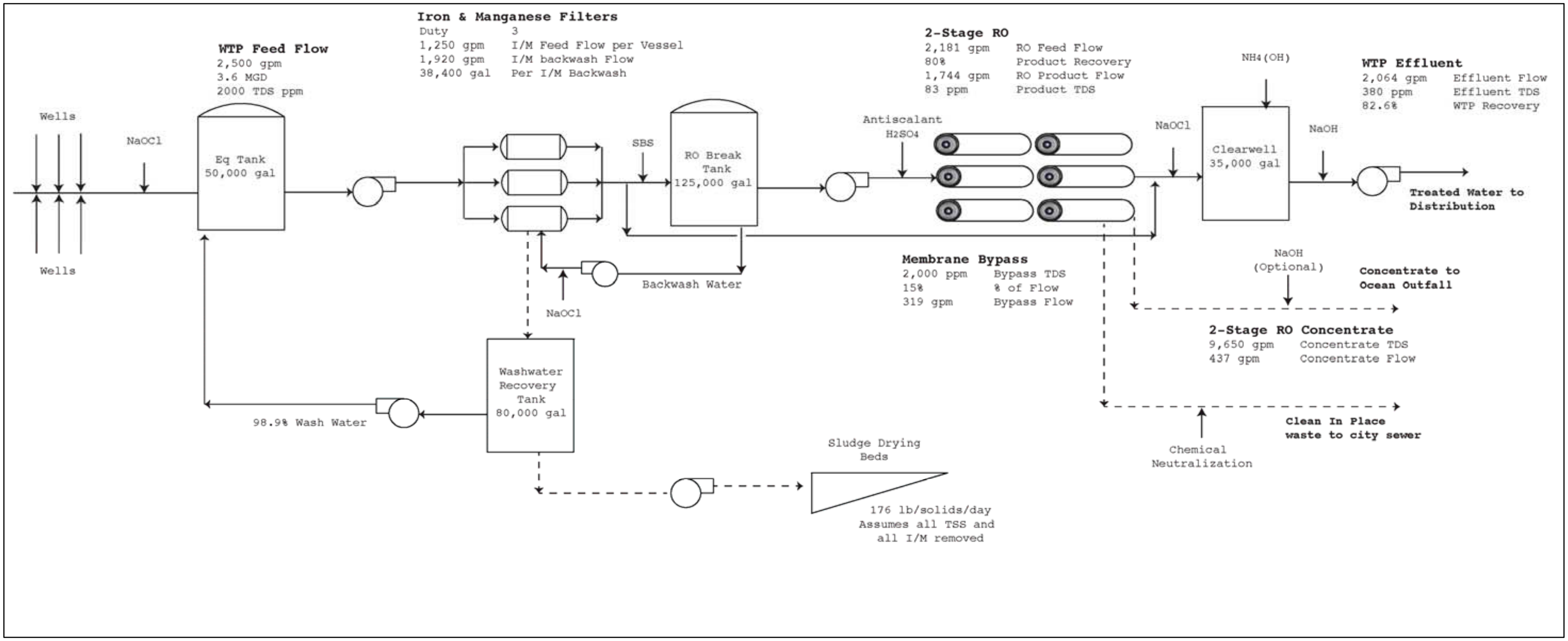


Figure 7-6

Process Flow Diagram

Rainbow Municipal Water District
Groundwater Supply Study

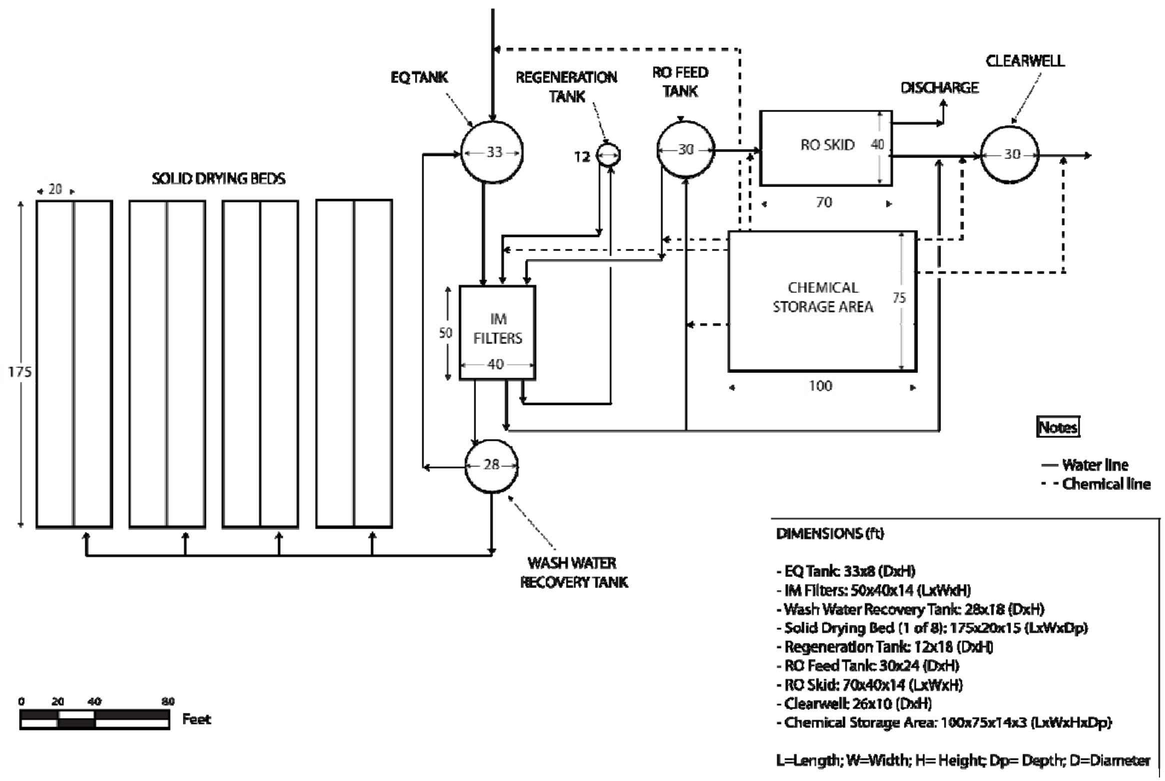
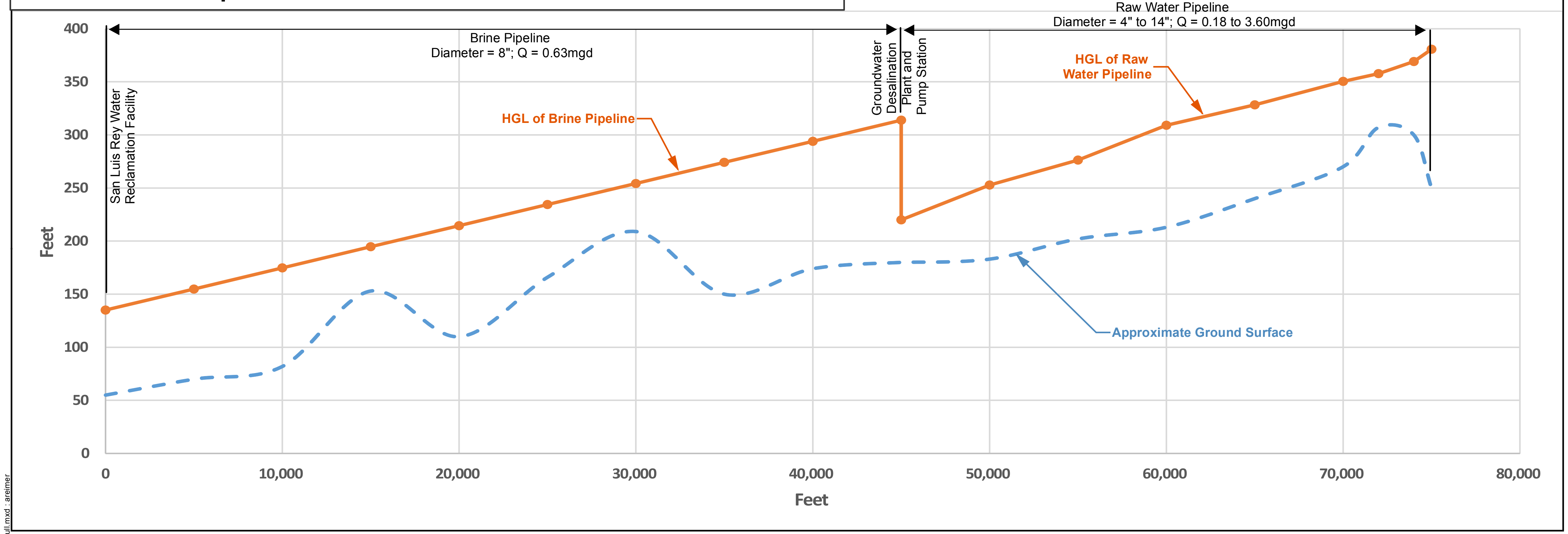


Figure 7-7

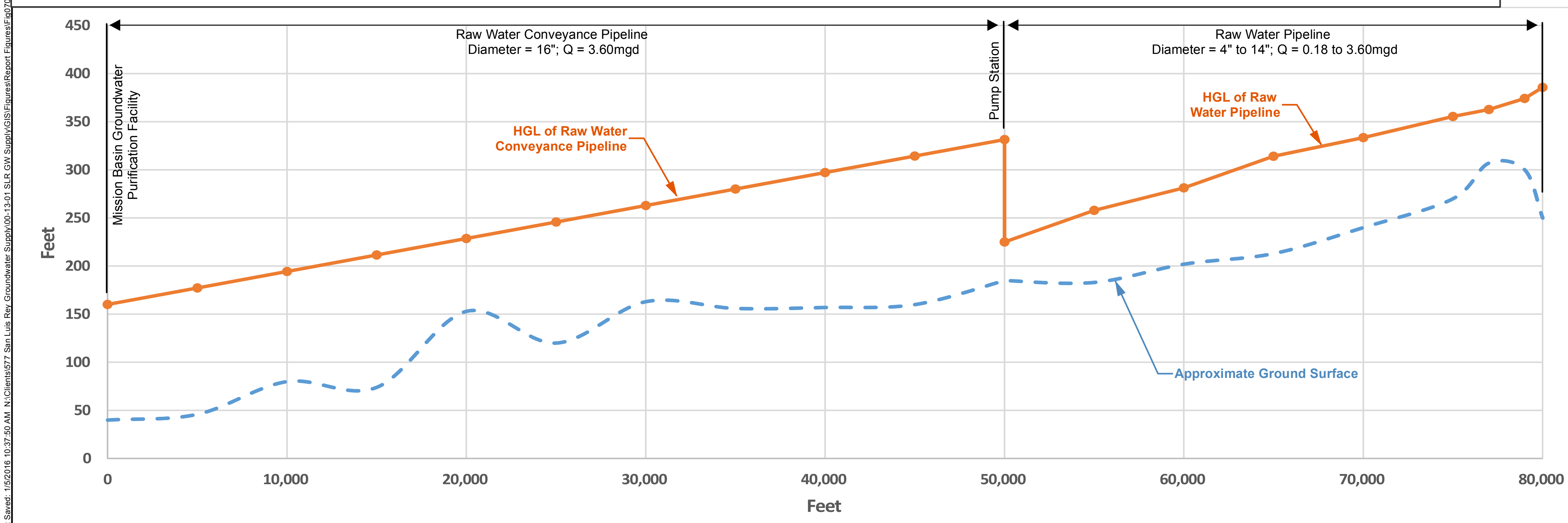
Simplified Conceptual Layout

Rainbow Municipal Water District
Groundwater Supply Study

Alternative 1: Proposed Bonsall Basin Groundwater Desalination Plant



Alternative 2: Proposed Bonsall Basin Groundwater Conveyance Facilities to Mission Basin Groundwater Purification Facility



Notes:
1. The pipeline profile views correspond to Figure 6-7 Draft Groundwater Conveyance and Desalination Alternatives.
2. Under Alternative 1 brackish groundwater would be treated at a proposed new groundwater desalination plant located in the Bonsall Basin. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin and conveyed to the proposed Bonsall Basin Groundwater Desalination Facility for treatment. Brine disposal would be via a proposed new brine line extending from the proposed Bonsall Basin Groundwater Desalination Facility to the City of Oceanside San Luis Rey Water Reclamation Facility, then through the existing brine line to the City of Oceanside Ocean Outfall.
3. Under Alternative 2 brackish groundwater would be treated at the existing City of Oceanside Mission Basin Groundwater Purification Facility. Brackish groundwater would be pumped from wells distributed along the alluvial aquifer of the Bonsall groundwater basin, and conveyed to the Mission Basin Groundwater Purification Facility for treatment. Brine disposal would be via the existing brine line to the City of Oceanside Ocean Outfall.

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Figure 7-8

Groundwater Conveyance and Desalination Alternatives Profile Views

Rainbow Municipal Water District
Groundwater Supply Study

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This chapter presents West Yost's conclusions and recommendations for the Proposed Groundwater Supply Project.

8.1 CONCLUSIONS

The findings of this preliminary analysis of the Proposed Groundwater Supply Project are favorable in terms of hydrology, water quality and treatment, and disposal of RO concentrate.

The hydrologic analysis conducted to support development of the hydrologic model determined that projected imported water return flows to the study area for water years 2016 through 2046 ranged from approximately 7,200 to 7,600 afy.

The results of modeling of the study area portion of the San Luis Rey River Valley Groundwater Basin indicated that median annual pumping rates up to 5,700 afy, or 460 afm, could be supported without significant impacts to the hydrologic conditions that would exist in the absence of imported water. Groundwater pumping of this scale under the Proposed Groundwater Supply Project would have only limited effects on the projected future hydrology of the study area portion of the groundwater basin. These effects were limited to localized drawdowns near active pumping wells. These effects can be limited by siting wells at locations away from neighboring wells and by adjusting pumping rates during project operations to limit potentially adverse drawdown in neighboring wells.

A capacity of 4,000 afy or 333 afm was selected for preliminary design purposes. These pumping rates are slightly less than evaluated in the model, because the modeled rates include some very wet years when simulated pumping rates were higher than would be implemented in the Proposed Groundwater Supply Project. Groundwater levels in the study area change quickly in response to stream flow events and groundwater pumping, because of the relatively large size of the study area watershed in relation to the relatively limited extent and thickness of the alluvial aquifer. Project infrastructure would not be designed to capture relatively infrequent and unpredictable events.

Drinking water standards were found to be the applicable water quality standards, and treatment to reduce iron, manganese and TDS concentrations was found to be necessary to comply with these standards, based on the results of the treatment analysis, which included review of available water quality data, consideration of alternative uses for the groundwater supply, and applicable water quality goals.

The alternatives considered were treatment of 3.6 mgd (approximately 4,000 afy) of raw water at:

Alternative 1: A Proposed Bonsall Basin Groundwater Desalination Facility using preoxidation with free chlorine followed by greensand filtration and RO;

Alternative 2: Conveyance of 3.6 mgd of raw water to the City of Oceanside Mission Basin Groundwater Purification Facility for treatment.

Under both alternatives, brine would be disposed via the City of Oceanside Ocean Outfall.

Estimated probable costs in present day dollars per acre-feet of water treated over the 30-year period from 2016 through 2046 were relatively similar at $920 per acre-foot for the Proposed Bonsall Basin Groundwater Desalination Facility, and $881 per acre-foot for treatment at the City of Oceanside Mission Basin Groundwater Purification Facility. The estimated probable costs include a 20 percent estimating contingency, a 10 percent construction contingency, and a 30 percent allocation for other project costs such as administration, construction management, and engineering services during construction.

8.2 RECOMMENDATIONS

The following are recommended next steps for further evaluation of the Proposed Groundwater Supply Project.

8.2.1 Information Gathering

Stakeholder agencies should be contacted to share information on Proposed Groundwater Supply Project and to assess opportunities for cooperation. The following specific steps are recommended.

- The City of Oceanside should be contacted to assess available capacity at the Mission Basin Groundwater Purification Facility and brine line, permitting requirements, and connection fees and ongoing treatment costs. This information should be used to refine the estimated cost of Alternative 2, Treatment at the Mission Basin Groundwater Purification Facility.
- FPUD and VCMWD should be engaged to assess interest and opportunities for conducting the Proposed Groundwater Supply Project as a multi-agency project.
- Caltrans should be contacted to further assess the feasibility of pipeline construction within Caltrans rights of way.

8.2.2 Water Rights

The source of imported water to RMWD is based upon RMWD's purchase of water from the SDCWA, and the water rights holders for the Colorado River and SWP water, which is imported. Water right terms and contractual arrangements can be the basis for limitations on imported water recapture. Typically, the wholesale agencies support imported water recapture projects due to the constraints on imported water availability. RMWD should verify that there are no water right terms and contractual arrangements limiting imported water recapture.

If RMWD determines to proceed with a project for the recapture of imported water return flow, it should make its intent to cease abandonment, and to recapture, known by public notice to potentially affected parties, including RMWD customers, the City of Oceanside, FPUD, VCMWD, and potentially the Santa Margarita River Watermaster, to update them regarding the Proposed Groundwater Supply Project and to identify any concerns.

RMWD should consider protesting water rights filings at the SWRCB that might involve diversion of its imported water return flow, to ensure inclusion of protective terms such as standard permit term 25, which specifically addresses the right to imported water return flow.

8.2.3 Siting Studies

Siting studies are recommended to identify and prioritize potential well locations, the site of the Proposed Bonsall Basin Groundwater Desalination Facility, and pipeline alignments. The well siting studies should be designed to allow identification and prioritization of sites that:

1. Have adequate yield
2. Do not have unacceptable environmental impacts, including on neighboring wells
3. Avoid classification of the pumped groundwater as groundwater under the influence of surface water (GWUDI) by DDW

Water sources classified as GWUDI are required to provide water treatment that satisfies requirements in the Surface Water Treatment Rule, Interim Enhanced Surface Water Treatment Rule, and Long Term 2 Surface Water Treatment Rule. If groundwater from one or more of the wells is classified as GWUDI and this water is blended with water from the other wells prior to treatment, all of the water will have to be treated as a surface water supply. These requirements should be avoided by ensuring that the wells are sited at locations which produce groundwater that is not classified as GWUDI.

8.2.4 Treatment Studies

The following next steps are recommended for the development of the treatment process:

- Conceptual level water quality and treatment evaluation should be expanded to the level of a ten percent design.
- Additional sampling should be conducted at representative locations within the proposed wellfield. Samples should be collected quarterly for all constituents with primary MCLs, secondary MCLs, unregulated Notification Levels, as well as general mineral and physical parameters needed to define the treatment train. The design basis water quality should be re-evaluated after completing the four rounds of sampling.
- Sizing of the iron and manganese treatment should be reevaluated based on the results of the additional iron and manganese sampling.
- Sizing of the RO treatment system should be reevaluated based on the TDS and chloride concentrations observed in the additional sampling.
- Additional suppliers of pressure filters for iron and manganese treatment should be considered in the development of the ten percent design to assure the opportunity for a competitive bid.
- Additional suppliers of RO membranes (e.g., Dow in addition to already included Toray and Hydranautics) should be considered in the ten percent design to assure the opportunity for a competitive bid.
- A life cycle cost evaluation should be performed as part of the ten percent design.

8.2.5 Expected Permitting and Regulatory Approvals

This section presents the anticipated permitting and regulatory approvals needed for the Proposed Groundwater Supply Project. Table 8-1summarizes the expected permitting and regulatory approvals needed to support development of the Proposed Groundwater Supply Project, potential regulatory complications, permit and regulatory-driven scheduling constraints, milestones and estimated costs. The following permitting and regulatory issues are addressed in Table 8-1:

- Public notification of the intent to cease abandonment of imported water return flows
- Appropriative water right filings by others
- Sustainable Groundwater Management Act requirements
- CEQA requirements
- Caltrans and San Diego County encroachment permitting
- Well construction permits
- DDW permits to operate
- Brine discharge permit
- Agreements with the City of Oceanside

Additional details regarding the requirements associated with obtaining a DDW water treatment plant operating permit, a RO waste brine discharge permit, and agreements with the City of Oceanside for potential RO treatment and brine discharge are provided below.

Table 8-1. Anticipated Permitting and Regulatory Requirements			
Anticipated Requirement	Issues/Challenges/Scheduling Constraints	Schedule Milestones	Conceptual Cost Range(a)
Public notification of intent to cease abandonment of imported water return flows	Imported water return flows not recaptured by the importer are considered abandoned and can be appropriated by others until the importer issues public notices to potentially affected parties of the intent to cease abandoning the water and implements a project to recapture imported water return flows. The timing and approach to issuing public notices requires further technical and legal analyses but might be structured in a phased approach in which initial notices are published stating the intent to cease abandonment contingent upon the development of a project to recapture the imported water return flows. Re-noticing would then occur at milestone events such as initiating a CEQA EIS, public review of the EIS and after adoption, certification and issuance of the Notice of Determination for the EIS.	2017 Issue initial public notice of intent to cease abandonment 2022 Issue second public notice concurrent with CEQA NOP 2023 Issue third public notice concurrent with public draft EIS 2024 Issue final public notice concurrent with CEQA NOD	$20K-$40K
SWRCB Appropriative Water Right Filings by Others	Others may file appropriative water rights applications with the SWRCB to recover imported water return flows abandoned by RMWD. RWMD should consider protesting water rights filings at the SWRCB that might involve diversion of RMWD-imported water return flow, to ensure inclusion of protective terms such as standard permit term 25. Term 25 makes the permit or license right to divert imported water return flows contingent upon the right of the importer to recapture the imported water return flows.	On-going activity	$0-$20K annually
Sustainable Groundwater Management Act Requirements	The 2014 Sustainable Groundwater Management Act (SGMA) requires the formation of groundwater sustainability agencies (GSAs) in high- and medium-priority groundwater basins. DWR has designated the San Luis Rey Valley Groundwater Basin as medium priority. A GSA or GSAs must be established for the basin and a groundwater sustainability plan (GSP) or GSPs must be prepared that consider all beneficial uses and users of groundwater in the basin. The RMWD service area encompasses most of the Bonsall subbasin of the San Luis Rey Valley Groundwater Basin. RMWD should participate in discussion of GSA formation in the San Luis Rey Valley Groundwater Basin and specifically in the Bonsall subbasin and should consider either becoming a GSA, or participating in a GSA along with other local agencies or San Diego County.	January 1, 2017 Alternative to GSP due to DWR June 30, 2017 Deadline for establishing GSA and seeking to resolve boundary overlap issues with adjacent GSAs July 1, 2017 Deadline for San Diego County to affirm or disaffirm responsibility for GSA if no GS has been established. Jan. 31, 2022 Basin must be managed under GSP April 1, 2022 First annual GSP progress report due to DWR	GSA Formation $10K-$100K GSP Preparation $250K-$500K GSP Implementation $10-$50K annually
CEQA EIS for groundwater project, including wells, treatment plant, conveyance pipelines, and brine discharge pipeline	CEQA analysis must be completed before a groundwater project can be designed, permitted, constructed and operated. The CEQA analysis cannot be undertaken until a detailed project description has been prepared, which will require further analysis of the proposed groundwater project. This includes identification and site-specific testing of individual well sites; updated hydrologic and regulatory analysis based on the site-specific studies; detailed alignment studies for pipelines; and land acquisition studies, including the need for easements. The potential for agreements with other local agencies whose imported water return flows are comingled with RMWD imported water return flows must be evaluated and alternative agreements identified. The potential for agreements with City of Oceanside for groundwater treatment, brine discharge and wheeling of treated groundwater also need to be evaluated and alternative agreements identified.	2017 Implement detailed studies of groundwater project alternatives 2022 Prepare project description and issue CEQA NOP 2023 Issue draft EIS for public review 2024 Adopt and certify EIR	Groundwater Project Alternatives Studies $500K-$1MM CEQA EIS $250K-$500K
Caltrans and County of San Diego encroachment permits for pipelines	Encroachment permits will be needed for raw water, RO brine disposal and treated water pipelines. Pipeline alternatives should be further evaluated and included in the CEQA analysis as described above. Encroachment permits should be obtained as part of the design of the groundwater project.	2024 Initiate design of groundwater project and encroachment permitting 2030 Complete design of groundwater project and encroachment permitting	Encroachment Permits $100K-$200K
Well construction permits from San Diego County Department of Environmental Health	Well construction permits from the San Diego County Department of Environmental Health will be required for construction of test production wells, monitoring wells and production wells ultimately constructed to provide source water for the groundwater project. These administrative permits can be obtained by RMWD or the well construction contractor shortly before well construction.	2017-2022 Obtain well construction permits for monitoring and test production wells 2030 Obtain well construction permits for production wells(b)	Monitoring and Test Production Wells $2K-$5K Production Wells $20K
DDW permits to operate for individual wells and water treatment plant	Public water systems that plan to construct new water supplies, such as wells or water treatment facilities, must submit an application for an operating permit or an amended permit for the proposed water system improvements to DDW for review and approval. The operating permit application should include an operation and maintenance plan (Plan) for the new facilities. A simulated distribution system (SDS) study to determine if the treated water is vulnerable to producing regulated disinfection byproducts that exceed the MCLs in the Stage 1 and Stage 2 Disinfectants and Disinfection Byproducts Rules is also recommended.	2031 Prepare and submit operating permit application and O&M plan to DDW. Conduct SDS study. 2032 Obtain DDW operating permit(c)	DDW operating permit application and O&M plan $100K-$200K SDS Study $70K-$100K

Table 8-1. Anticipated Permitting and Regulatory Requirements

Anticipated Requirement	Issues/Challenges/Scheduling Constraints	Schedule Milestones	Conceptual Cost Range[a]
New Regional Board permit for discharge to OOO	If Alternative 1 is pursued, a separate discharge permit would be needed for discharge of brine to the Land Outfall, and ultimately the Combined Oceanside Ocean Outfall (OOO). A permit application (Report of Waste Discharge) would need to be prepared detailing the proposed discharge. The application would need to provide adequate detail regarding the capacity of the Land Outfall to accommodate the additional flow.	2017 Begin discussions with City of Oceanside regarding available capacity in the Land Outfall 2018 Assuming improvements are needed, begin technical studies needed to identify appropriate improvements. 2022 Complete technical studies and incorporate Land Outfall improvements into the overall Project 2030 Prepare permit application for new discharge to the Land Outfall. 2032 Obtain discharge permit for RO brine	Studies to evaluate Land Outfall capacity improvements $50K - >$250K Application for new permit $25K-$50K
Modification of Regional Board Oceanside Permit for discharge of additional brine to OOO	If Alternative 2 is pursued, the existing Oceanside Permit will likely need to be modified to accommodate additional brine from the Mission Basin Groundwater Purification Facility. Under this scenario, the City of Oceanside would need to seek a permit modification, and it is assumed that the studies and permit application process would be combined with the agreement with the City to treat groundwater for the District.		
Agreement with the City of Oceanside for Brine Disposal at the OOO.	Two public and one private entities have existing contracts with the City of Oceanside for discharge of municipal effluent and/or brine waste to the OOO. Under either alternative, a similar agreement would be needed with the City of Oceanside for discharge of the proposed groundwater project brine to the OOO. The City of Oceanside needs to be consulted to determine how flows are controlled, and whether additional flow can be accommodated in the Land Outfall.	2017 Begin discussions with City of Oceanside regarding available capacity in the Land Outfall 2018 Assuming improvements are needed, begin technical studies needed to identify appropriate improvements. 2022 Complete technical studies and incorporate Land Outfall improvements into the overall groundwater Project. 2022 Begin negotiations with City for an agreement to discharge, including identification of how costs will be shared (as appropriate) for increasing the capacity of the Land Outfall 2024 Finalize agreement with City.	Studies to evaluate Land Outfall capacity improvements $50K - >$250K Negotiations with City of Oceanside $150K->$1.0M[d]
Agreement with the City of Oceanside for Groundwater Treatment at the Mission Basin Groundwater Purification Facility	Under Alternative 2, treatment of project groundwater would occur at the City of Oceanside's Mission Basin Groundwater Purification Facility. An agreement with the City of Oceanside would be needed for treatment and brine disposal. The City needs to be consulted to determine if additional treatment capacity is available at the facility.	2017 Begin discussions with City of Oceanside regarding available capacity in the Land Outfall and Mission Basin Groundwater Purification Facility 2018 Assuming improvements are needed, begin technical studies needed to identify appropriate improvements. 2020 Begin negotiations with City for an agreement to treat groundwater and discharge brine 2022 Complete technical studies and incorporate Land Outfall and Mission Basin Groundwater Purification Facility improvements into the overall Project. 2022 Finalize agreement with City for an agreement to treat groundwater and discharge brine.	Studies to evaluate Land Outfall capacity improvements $50K - >$250K Negotiations with City of Oceanside $1.0M->$2.0M[e]

[a] 2016 dollars.
[b] Assumes that groundwater project construction is initiated by 2030.
[c] Assumes that groundwater project construction and testing is completed by 2032.
[d] The level of effort associated with developing an agreement with the City of Oceanside for discharge to the Land Outfall is highly dependent on the technical feasibility of discharging to the Land Outfall and the operational requirements that will fall on the City to control the discharges within the limitations of the discharge permit.
[e] The level of effort associated with developing an agreement with the City of Oceanside for treatment of groundwater at the City's facility is highly dependent on the capacity of the existing system, the technical feasibility of discharging to the Land Outfall, and the costs associated with requirements that will fall on the City to control the discharges within the limitations of the discharge permit.

8.2.5.1 Water Treatment Plant Operating Permit

Public water systems that plan to construct new water supply facilities, such as wells or water treatment facilities, must submit an application for an operating permit or an amended permit for the proposed water system improvements to DDW for review and approval. The operating permit application should include an Operation and Maintenance Plan for the new facilities. The permit application for the new facilities should be submitted to DDW 6-12 months prior to the facilities' scheduled start-up date to provide adequate time for DDW to review and comment, the water system to revise its permit application, and resubmit the application for final approval of the permit by DDW.

As part of the DDW permitting process, a simulated distribution system (SDS) study should be conducted to determine if the treated water is vulnerable to producing regulated disinfection byproducts that exceed the MCLs in the Stage 1 and Stage 2 Disinfectants and Disinfection Byproducts Rules. Planning, obtaining DDW review, scheduling and performing a SDS DBP study, and preparing draft and final DBP study report will require about three months to complete. This task could also be performed during the wells' and water treatment facility's initial operating period.

8.2.5.2 RO Waste Brine Discharge Permit

The current discharge permits for the City of Oceanside's Oceanside Ocean Outfall (OOO) were reviewed to determine potential permitting constraints for discharging the RO waste brine from the Proposed Groundwater Supply Project. There are four separate discharge permits that have been issued by the San Diego Regional Water Quality Control Board (Regional Board) for discharges to the OOO. These include a combined permit – Regional Board Order No. R9-2014-0108 (Oceanside Permit). The Oceanside Permit regulates the discharges from the three City of Oceanside facilities that discharge to the OOO: San Luis Rey Water Reclamation Facility, La Salina Wastewater Treatment Plant, and Mission Basin Groundwater Purification Facility. In addition, Genentech, Camp Pendleton, and the Fallbrook Public Utility District have individual permits for discharge of various wastes to the OOO.

In addition to regulating the individual discharges from the City's three facilities, the Oceanside Permit limits the combined discharge flows from two locations: the Land Outfall and the Combined OOO. The Land Outfall conveys flows from the San Luis Rey Water Reclamation Facility, La Salina Wastewater Treatment Plant, Mission Basin Groundwater Purification Facility and Genetech, Inc. facilities to the Combined OOO. The Combined OOO outfall conveys all the flows from the Land Outfall plus flows from Camp Pendleton, and FPUD to the discharge point in the Pacific Ocean. The flow limitations applied to these two outfalls are based on conveyance capacity constraints that were defined by the City of Oceanside through engineering studies.

The Oceanside Permit also details that improvements have been identified that would increase the capacity of the Land Outfall. Accordingly, the Oceanside Permit allows for an increase in flow to the Land Outfall based on the capacity defined by these previous studies. However, such an increase would have to be approved by the Regional Board. Similarly, the Oceanside Permit allows for an increase in flow to the Combined OOO with the completion of improvements that were previously identified and approval by the Regional Board.

The type of discharge and permitted capacity for each of these facilities and the combined discharge locations are summarized in Table 8-2. Also shown is the allowable permitted capacity following the improvements identified in the Oceanside Permit and approval by the Regional Board. As shown, the permitted discharge capacity for the Land Outfall is less than the permitted capacity of the four individual facilities that discharge to this outfall – even after the improvements are made. The Oceanside Permit does not detail what measures are used to control discharges to the Land Outfall below the permitted flow of the individual facilities that contribute to this outfall. Conversely, Table 8-2 demonstrates that the permitted discharge capacity for the Combined OOO is greater than the combined permitted discharge capacity for the six facilities that contributes to this outfall.

Table 8-2. Summary of Currently Permitted Oceanside Ocean Outfall Discharges

Facility	Permitted Discharge Type	Permitted Average Monthly Discharge Flow, MGD	Permitted Discharge Flow Following Outfall Improvements and Approval by Regional Board, MGD	Contracted Flow with Oceanside, MGD
San Luis Rey Water Reclamation Facility	Treated municipal effluent (wastewater)	13.5	15.4	N/A
La Salina Wastewater Treatment Plant	Treated municipal effluent (wastewater)	5.5	5.5	N/A
Mission Basin Groundwater Purification Facility	Brine waste	2.0	2.0	N/A
Genentech, Inc.	Brine waste	0.155	0.155	0.85
Combined Discharge to Land Outfall		**16.6[a]**	**18.4[a]**	**N/A**
Camp Pendleton	Treated municipal effluent (wastewater) and brine waste	3.6	3.6	3.6
Fallbrook Public Utility District Wastewater Treatment Plant No. 1	Treated municipal effluent (wastewater)	2.7	2.7	2.4
Unallocated Combined OOO Capacity[b]		14.1	15.7	N/A
Total Combined Discharge to the OOO		**41.5**	**45.0**	**N/A**

(a) The Oceanside Permit limits the combined flow from the San Luis Rey Water Reclamation Facility, La Salina Wastewater Treatment Plant, Mission Basin Groundwater Purification Facility and Genetech, Inc. to an average monthly flow limit that is lower than the limits applied to the individual facilities. Although the Oceanside Permit does indicate that the facilities have effluent storage, it is silent on how the individual facilities control their discharges to meet the combined limit.

(b) The Unallocated Combined OOO Capacity is the difference between the sum of the individually permitted flows to the OOO and the total permitted discharge flow of the OOO. Note that improvements to the Land Outfall would be necessary for the San Luis Rey Water Reclamation Facility, La Salina Wastewater Treatment Plant, Mission Basin Groundwater Purification Facility and Genetech facilities to exercise their maximum discharge capacity potential.

Both of the Proposed Groundwater Supply Project alternatives call for discharging to the Land Outfall, where hydraulic constraints are identified in the Oceanside Permit. Therefore, it is not clear whether discharge to the system is technically feasible without additional infrastructure. Discussions with City of Oceanside staff will be necessary to gain clarification on this issue. However, assuming the hydraulic constraints in the Land Outfall can be addressed, there is adequate permitted and hydraulic capacity in the Combined OOO to accommodate additional flows.

For Alternative 1, the RO brine discharge would enter the Land Outfall near the Mission Basin Groundwater Purification Facility. Under this condition, a separate discharge permit would be needed, similar to the Genentech permit for brine discharge. Because it will be incumbent on the City to demonstrate capacity is available in the Land Outfall for the brine discharge, the costs and level of effort associated with developing a permit application is dependent on the technical issues associated with the available capacity of the Land Outfall. Barring the need to address the capacity issues, the process for developing a permit application and obtaining a discharge permit is estimated to require about 12 to 16 months. Engineering services to complete the permit application and review a draft permit are estimated to require between $25,000 and $50,000.

For Alternative 2, the Proposed Groundwater Supply Project RO brine would be discharged from the Mission Basin Groundwater Purification Facility. Under this scenario, the Oceanside Permit would likely need to be amended to account for the additional treatment capacity of the Mission Basin Groundwater Purification Facility. Assuming a treatment capacity and discharge flow increase is necessary, the application process would be similar to the individual permit process. However, it is likely that the efforts would be wrapped into the agreement with the City.

8.2.5.3 Agreement with City of Oceanside

Similar to the permitting process, the costs and level of effort associated with establishing an agreement with the City of Oceanside will depend on the level of effort needed to confirm the technical feasibility of increasing discharge to the Land Outfall and how these costs are transferred to the RMWO. Given the likely technical constraints, discussions with the City of Oceanside should begin as soon as possible.

CHAPTER 9
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APPENDIX A

Water Quality and Treatment Technology Analysis for
Rainbow Municipal Water District San Luis Rey Groundwater Supply
Technical Memorandum, August 28, 2015

TECHNICAL MEMORANDUM NO. 1 (TM 1)

Water Quality and Treatment Assessment
for the San Luis Rey Groundwater Supply
Prepared for Rainbow Municipal Water District/West Yost Associates

Draft Date: August 28, 2015
Final Date:

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Subject: Water Quality and Treatment Technology Analysis for Rainbow Municipal Water District San Luis Rey Groundwater Supply

EXECUTIVE SUMMARY

Rainbow Municipal Water District (RMWD) purchases its entire water supply from the San Diego County Water Authority. (SDCWA). The District is pursuing local water supply options to reduce its dependency on imported water. This project evaluated San Luis Rey River groundwater as a potential source of drinking water for the District. A detailed analysis of water quality and treatment was prepared and is presented in this document, broken down into five Sections:

1. Introduction
2. Existing Water Quality and Treatment Review
3. Raw Water Quality Characterization
4. Water Quality Goals
5. Water Treatment

The results of the analysis are summarized in the Executive Summary.

Existing Water Quality and Treatment Review

Past reports by CDM and Heden for RMWD and by Trussell Tech for City of Oceanside were evaluated. In the CDM Report, the water quality of the Pope Well and the Vessels Ranch well were evaluated, with TDS of 1080 and 2230, respectively. The report compared treatment with electrodialyis to treatment with RO for different capacity scenarios for each wellfied and determined RO to be more cost effective. The Heden Report had more limited discussion of water quality and treatment but discussed TDS levels up to 2330 mg/L for shallow wells sampled in 1989 and up to 1325 mg/L for Rainbow Creek. The report discussed treatment with RO at a high level. None of the past RMWD reports included iron and manganese treatment. In the City of Oceanside evaluation prepared by Trussell Tech, water quality data is presented for the Mission Basin wells along the California Coastal Basin aquifer, which are relevant to the project as water quality is expected to be similar to the water quality of the wells evaluated.

Raw Water Quality Characterization

Sampling was conducted for two wells selected by RMWD in June 2015. The two wells sampled were being actively pumped during the time of the sampling event. The location of the wells sampled in 2015 is compared on an aerial map to the Pope Well, the Vessels Ranch well, and the Mission Basin wells. Geographically, the upstream to downstream order of the wells is Pope Well, wells sampled by RMWD in 2015, Vessels Ranch Well, and the Mission Basin (see Figure 1). Water quality data from each well was used in the water quality evaluation and is shown in Table 1. The water quality analysis showed there were no constituents with exceedances of primary MCLs (pMCLs) or unregulated notification levels (NLs). The constituents with exceedances of secondary (sMCLs) are shown in Table 2. The critical constituents whose raw water quality will drive treatment considerations from a regulatory perspective (see Sections 3 and 4) are TDS, iron, and manganese. The raw water quality for the various sources for these constituents are shown in Table ES - 1.

Table ES - 1 - Raw Water Quality for Constituents That Drive Treatment Considerations from a Regulatory Perspective

Parameter	Units	Regul-atory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
Total Dissolved Solids	mg/L	500 Rec 1,000 Upper	1400	1500	1577	1673	1888	1754	2230	1080
Iron	mg/L	0.3	0.80	3.5	3.3	1.3	2	2.6	0.80	0.04
Manganese	mg/L	0.05	0.20	0.65	0.62	0.46	0.48	0.62	0.57	0.22

The treatment of iron and manganese (IM treatment) is accomplished with preoxidation followed by greensand filtration. An advantage of IM treatment is that it typically achieves removals greater than what is required, making it more straightforward to treat elevated levels of iron and manganese to low levels. It also allows for a design that can treat the range of iron and manganese levels shown in Table ES - 1. An RO system to treat TDS will require sizing that also considers additional minerals most notably chloride and also general mineral quality. The amount of recovery that can be achieved is limited by the potential for fouling and scaling by constituents (e.g., silica) present at levels that are not completely controllable by antiscalants.

The design basis water quality was determined for constituents that exceed regulatory sMCLs was determined based on maximum or near maximum levels. The TDS design basis was determined near the maximum level observed in the Vessels Ranch well water quality because the project team believes based on the groundwater modeling and hydrogeological analysis that the Vessels Ranch well is most typical of groundwater that would be pumped under the proposed project. The iron and manganese design basis was determined as a worst case of the water quality shown in Table ES - 1. For other water quality parameters critical to the design, averaging of water quality parameters shown in Table 1. was performed. The design basis water quality is shown in Table 3 for IM treatment and Table 4 for RO. The design basis water quality for constituents of particular interest are presented in Table ES - 2.

Table ES - 2 – Design Basis Water Quality for Key Constituents

Parameter	Units	Design Water Quality	Regulatory Limit	Water Quality from Table 1.		
				Min	Average	Max
Iron	mg/L	3.5	0.3	0.04	1.79	3.5
Manganese	mg/L	0.65	0.05	0.20	0.48	0.65
Total Dissolved Solids	mg/L	2,000	500 Rec 1,000 Upper	1,080	1,638	2,230
Chloride	mg/L	434	250 Rec 500 Upper	190	434	600
Silica	mg/L	26	--	21.5	26	3.4

Water Quality Goals

Water quality goals were determined based on regulatory requirements, agricultural considerations, and the potential for benefit of blending with imported water. The recommended water quality goals are to meet MCLs (pMCLs and sMCLs), with the exception of iron and manganese. Iron and manganese can be noticed as aesthetically displeasing by consumers at levels below their sMCLs (0.3 and 0.05 mg/L). Given that IM treatment can easily reduce iron and manganese below detection, the water quality goals for iron and manganese will be set at 0.1 and 0.02 mg/L, respectively.

Disinfection with free chlorine to achieve 4-log virus removal/inactivation is recommended to assure compliance with the Groundwater Rule and avoid the complexities of triggered source water monitoring. Given the levels of TOC measured in the raw water were higher than typical levels, it is recommended to conduct testing for DBP formation as the project moves along.

RMWD is located in a major agricultural area. The water quality goals need to consider impacts on plants and agriculture (e.g. avacados). Based on research, it is known plants in the area are sensitive to chloride. A previous project conducted by members of the project team found avocado trees might develop tip burn when excessive chloride accumulates in their leaves. State Water Resources Control Board water quality guidelines for agricultural applications were compared with imported water quality from the Skinner WTP. It was recommended the water quality goal for constituents that impact agriculture be set to match the water quality from the Skinnner WTP, with the chloride water quality goal set at 85 mg/L, for example.

Blending imported water with groundwater was considered as a possible way to reduce the treatment requirement through dilution. The State has cut back on its State Water Project supply in recent years and more water has been delivered from the Colorado River, which is higher in minerals like sulfate and chloride, as well as TDS. The impact of blending with Skinner WTP imported water and its impact on RO treatment system size was evaluated. The elevated TDS in the imported water (~620 mg/L is the 90% level) results in a larger and more costly RO plant required for the scenarios employing blending with imported water, with the effect more pronounced with increased imported water flow. For this reason, blending with imported water was not considered further on the project.

The design water quality goals for iron, manganese, TDS, and chloride are summarized in Table ES - 3. TDS and chloride water quality goals were set to match Skinner WTP water quality based on 2012 through 2015 data.

Table ES - 3 – Design Water Quality Goals for Key Constituents

Constituent or Property	Goal	Rationale
Iron (mg/L)	0.1	Aesthetics (colored water), Goal Set at Detection Limit
Manganese (mg/L)	0.020	Aesthetics (colored water), Goal Set at Detection Limit
TDS (mg/L)	482	Aesthetics, Goal Set to Match Imported Water Quality
Chloride (mg/L)	85	Agriculture, Goal Set to Match Imported Water Quality

Water Treatment

Conceptual level design of water treatment process and conceptual level capital and O&M costs were presented. The analysis was broken down into sections on process selection; a process flow diagram; conceptual design for iron and manganese (IM) treatment, RO treatment, post treatment, chemical feed and storage; treated water quality; a simplified conceptual layout; and conceptual level O&M costs. The design flow for the plant was determined to be 3.6 million gallons per day (mgd) based on a safe yield for the basin of 4,000 acre-feet/year (AFY) from groundwater modeling and hydrogeological analysis.

Process Selection and Process Flow Diagram

Removal requirements for key constituents that drive the design are summarized in Table ES - 4. It is shown that about 97 percent removal of iron and manganese is needed, with about 75 percent TDS removal and 80 percent chloride removal. Processes that can meet these requirements were considered.

Table ES - 4 Design Basis, Water Quality Goals and %Removal for Key Constituents that Drive the Conceptual Design

Constituent	Design Basis	WQ Goal	%Removal to Meet WQ Goal
Iron (mg/L)	3.5	0.1	97.1
Manganese (mg/L)	0.65	0.02	96.9
TDS (mg/L)	2,000	482	75.9
Chloride (mg/L)	434	85	80.4

A list of IM treatment options considered and a summary of why processes were selected or ruled out are presented in Table ES - 5. Both conventional filtration and greensand filtration approaches involve a manganese oxide surface on the filter to catalyze the removal of iron and manganese. The process selected was **Preoxidation with Free Chlorine followed by Greensand Filtration**. Technologies considered for treatment of TDS and minerals along with there rationale for selection or rejection are also presented in Table ES - 5. **The process selected was RO.** The process flow diagram is shown in Section 5 in Figure 5. The elements of the treatment train will be discussed below.

Table ES - 5 – IM and Mineral Treatment Technologies Evaluated and Rationale for Selection or Rejection

IM Treatment	Rationale
Sequestration	Ruled out as only prevents Fe/Mn oxidation without removal
Ion Exchange	Ruled out as not effective at high concentration as precipitated Fe can build up on the resin & reduces process efficiency
Preoxidation + Conventional Filtration	Ruled out because it requires an acclimation period to coat manganese oxide service and less effective than greensand
Preoxidation + Greensand Filtration	There is new greensand filter media available that does not require regular regeneration. It comes coated with a manganese dioxide surface. It is more effective in removing Fe/Mn than any other process. **Preoxidation + Greensand Filtration Selected.**
Preoxidants	Aeration ruled due to slow reaction times at pH of treatment Permanganate ruled out due to high cost and risk of overdosing **Free Chlorine selected as works well with new greensand**
TDS/Cl Treatment	**Rationale**
Electrodialysis	Ruled due to high cost and lack of water industry use
Nanofiltration (NF)	Ruled out due to poor salt rejection and inability reject monovalent ions effectively
RO	RO can easily meet the TDS/chloride requirements. Low pressure RO membrane will allow for efficient sizing, lower energy required, and lower cost. **RO was selected** for these reasons.

IM Treatment

The major components in the IM Treatment portion of the treatment train and their purpose are summarized in Table ES - 6. As shown on the PFD, the raw groundwater is dosed with sodium hypochlorite (preoxidation step), pumped into an equalization tank, and fed into iron and manganese filters. Downstream of the IM filters, the water is quenched with sodium bisulfite (SBS) to remove excess free chlorine that would damage the RO membranes and fed to an RO feed tank that feeds the RO and also provides water for backwashing the IM filters. While the full flow is treated by IM filters, only a portion of the flow is treated by RO, with a partial bypass. The IM filter backwash water is dosed with free chlorine prior to the filter backwashing to optimize conditions for the greensand filters. After backwashing the filters, the backwash water is fed to a waste wash water (WWW) recovery tank where sludge is separated and supernatant is recycled to the front of the plant with sludge pumped to drying beds. Design criteria for each component of the IM treatment system are presented in Section 5.3.

Table ES - 6 - Major Components in the IM Treatment System[a]

Equipment	Purpose
Filter media	GreensandPlus media has a manganese oxide surface that optimizes the reactions that remove Fe/Mn
IM Treatment Vessels	Remove iron and manganese precipitants after preoxidation with free chlorine and additional oxidation and removal on manganese oxide surface.
RO Feed Tank	Provide dechlorination contact time and provide wash water storage volume for backwashing the IM media.
IM Vessel Back Wash Pump	Provides water to perform backwashing of the IM vessels.
Waste Wash Water (WWW) Recovery Tank	Holds the IM backwash volume of one vessel and provides sludge settling time.
Treated WWW Recycle Pump	Recycles the supernatant from the WWW recovery tank to the front of the EQ tank.
WWW Sludge Pump	Transfer sludge from the WWW to the drying beds.

[a]Chemical feed and storage including preoxidant feed discussed in Section 5.6.

RO Treatment and Post Treatment Disinfection & Stabilization

As discussed above, low-pressure RO membranes were selected. As pretreatment, sulfuric acid and antiscalant are dosed to control fouling and scaling in the feed water. It was determined based on the level of pretreatment, the silica level, and the concentrations of other minerals with scaling potential that 80% recovery was appropriate. Based on the level of removal of TDS and chloride required, it was determined that 85 percent of the raw groundwater required RO treatment, with 15 percent bypass. RO modeling was used to compare six RO elements from two manufacturers, Hydranautics and Toray, with selection criteria including number of elements per train, the allowable bypass flow when achieving water quality goals, and feed pressure. Important design criteria for the RO membrane system are summarized in Table ES - 7.

Table ES - 7 – Important RO System Design Criteria[a]

Parameter	Unit	Value
RO recovery	percent	80
Target flux	gfd	12
No. of Trains	--	3
No. of Stages	--	2
RO elements[1]	--	Toray TMG20-400
No. of Elements per vessel	--	7
No. of Vessels for Stage 1[1]	--	16
No. of Vessels for Stage 2[1]	--	8
RO feed pressure (psi)[1]	--	108
RO element surface area	square feet	400

[a] Flows (mgd): Feed=3.14; Bypass=0.46; Permeate=2.51; Concentrate=0.63; Product=2.97
[1] Determined based on RO modeling

Disinfection with free chlorine is included for Groundwater Rule compliance with a baffled clearwell designed to get the detention time required to achieve CT. Waters treated by RO and blended with raw groundwater high in minerals are typically stabilized to assure corrosion control water quality goals are achieved (often corrosion indicators like Langelier Saturation Index and/or Calcium Carbonate Precipitation Potential). Caustic soda and orthophosphate were considered for corrosion control, with caustic soda selected due to ease of operation and the it's common use in water industry practice. Disinfection and stabilization are discussed in Section 5.5. Design criteria for pumps and tanks are discussed in Section 5.6.

Treated Water Quality and Simplified Conceptual Layout

Treated water quality data for key constituents compared to water quality goals is summarized in Table ES - 8. All constituents meet their water quality goals. Chloride is the limiting parameter.

Table ES - 8 - Treated water quality compared to water quality goals for key constituents

Constituent	Water Quality Goal (mg/L)	Treated Water Quality (mg/L)
Iron	0.1	<0.1
Manganese	0.020	<0.020
TDS	482	351
Chloride	85	77

A simplified conceptual layout is presented in Section 5.8. The layout is depicted in Figure 6. The rough estimate of area required is 1.2 acres.

Conceptual Level Capital and O&M Costs

An opinion of probable construction cost (OPCC) was developed. It was developed based on cost quotes for portions of the IM treatment system provided by OEM Hungerford and Terry and for portions of the RO treatment system provided by OEM Biwater, as well as cost estimating practice for items not included in the quotes. O&M costs were developed and considered chemical costs, power costs, and maintenance costs. The breakdown of the OPCC for capital costs and O&M costs for IM treatment, RO, and post treatment is summarized in Figure 6.

Table ES - 9 – Capital and O&M Cost Summary

Treatment Process	Capital Costs (OPCC)	Annual O&M Costs
IM Treatment	$4,190,000	$213,000
RO Treatment	$3,340,000	$440,204
Post Treatment	$331,000	$220,436
TOTAL	**$7,860,000**	**$874,000**

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1 INTRODUCTION

Rainbow Municipal Water District (RMWD) purchases its entire water supply from the San Diego County Water Authority (SDCWA), which acts as a wholesale importer for its 24 member agencies. SDCWA's principal sources consist of water purchased from Metropolitan Water District of Southern California (MWD), water transfers from Imperial Irrigation District that are wheeled through MWD's conveyance facilities, and short-term water transfers needed to offset dry-year reductions in water supply from MWD. The principal source areas for these supplies are the State Water Project and Colorado River watersheds. These imported water supplies are increasingly expensive and over allocated. The District is pursuing alternative supplies to provide a level of autonomy over water supply decisions and to develop lower cost local water sources.

Trussell Tech completed the following tasks to meet the project objectives:

- Reviewed existing groundwater quality/treatment information for the alluvial portion of the San Luis Rey (SLR) River Basin. This included most relevantly a report prepared by CDM in 1996 on groundwater demineralization for RMWD and an alternative water source feasibility study prepared by Heden and Associates in 2013. This is presented in Section 2.
- Prepared a detailed raw water quality characterization. Evaluated the water quality based on groundwater sampling conducted for this project, additional sampling data from the literature review, and relevant data obtained from the City of Oceanside. The water quality evaluation included development of the design basis raw water quality for the project. This is presented in Section 3.
- The treated water quality goals used as the basis of design were evaluated. The analysis included evaluation of regulatory requirements. At the same time, an analysis was performed of the viability of landscape irrigation and blending with imported water in addition to the option for use of SLR groundwater as a public drinking water supply. The evaluation of water quality goals is presented in Section 4.
- Prepared recommendations for treatment including a conceptual estimate of cost for treatment alternatives. Reverse osmosis and greensand filtration were selected as the key components of the process train based on the water quality evaluation. The conceptual treatment process design is shown in Section 5. The estimate of capital and O&M costs is also shown in Section 5.

2 EXISTING WATER QUALITY AND TREATMENT TECHNOLOGY REVIEW

Water quality and treatment considerations from past work are presented below.

2.1 WATER QUALITY DATA FROM THE REVIEW OF PAST WORK

The sources of data used for the water quality review include:

- Wellhead water quality data in the report prepared by CDM in 1996, "Demineralization of Groundwater within the Rainbow Municipal Water District (CDM Report)" The report presented the results of sampling wells in the vicinity of RMWD. The water quality of the Pope Well and the Vessels Ranch Well were discussed. The total dissolved solids (TDS) concentrations in the Pope Well and Vessels Ranch Well were 1080 mg/L and 2230 mg/L, respectively. Water quality parameters that exceeded State and Federal Primary Maximum Contaminant Levels (pMCLs) or Secondary MCLs (sMCLs) in the Pope Well were manganese, sulfate, and TDS. In the Vessels Ranch well, parameters that exceeded the MCLs were manganese, sulfate, TDS, chloride, iron, and color. The detailed water quality results are listed and compared in Table 1. Treatment with electrodialysis (ED) or RO was compared for treating groundwater from the Pope Well and Vessels Ranch well with the varied TDS levels shown above. Different capacity systems were evaluated ranging from 1250 to 3000 acre-ft/year. The report determined treatment by ED was less cost effective than RO.
- "Alternative Water Source Feasibility Study" (Feasibility Study) prepared by J.C. Heden and Associates, Inc., Jan. 2013 (Heden Report). No wells were sampled during the Feasibility Study. However, it was concluded that the Basin appears to have high TDS. It referred to groundwater samples obtained from shallow wells in 1989 indicating a range of 370 to 2330 mg/L TDS. It referenced samples from Rainbow Creek during low flow (base flow) conditions demonstrating a range of 793 to 1325 mg/L TDS. Nutrients, primarily nitrate and phosphate, were also evaluated in the Feasibility Study. Different sources indicated concentrations of nitrate as nitrogen peaked in the mid 1980s at 77 mg/L. The Heden Report discussed treatment with RO but did not provide a detailed description of the treatment train.
- Wellhead water quality data from 2009 to 2011 in the report, "Process Evaluation and Recommendations for the Mission Basin Groundwater Purification Facility," prepared by Trussell Technologies for the City of Oceanside, Nov. 2012. The wells evaluated in the City of Oceanside project are located in Mission Basin along the California Coastal Basin aquifers. Water quality data from the Mission Basin Desalter is presented in the detailed evaluation of raw water quality.

2.2 EXISTING TREATMENT TECHNOLOGY REVIEW

RMWD has looked into groundwater desalination as a potentially viable local water supply over the past several years. The SLR Groundwater Supply is available to RMWD for such a project. This has resulted in the development of reports evaluating feasibility, as discussed above with respect to the water quality evaluation. A summary of reports and outcomes for past groundwater desalination treatment evaluations is provided below.

- CDM Report
  The report discussed groundwater quantity and quality, and the treatment alternatives to demineralize the groundwater. In the CDM report, two existing wells, Pope Well and Vessels Ranch Well were sampled and tested for the water quality. In the second phase of the project, electrodialysis and reverse osmosis were evaluated for two different areas (Pope and Vessels Ranch) under different flow conditions. The capital cost and O&M costs were estimated for these scenarios as well.
- Heden Report
  In the Feasibility Study prepared by Heden, recapture of the imported water and treatment with reverse osmosis was listed as the top priority recommendation. Cost and payback periods were evaluated in the report.

3 RAW WATER QUALITY CHARACTERIZATION

3.1 SAMPLING

In June 2015, two wells were sampled with the assistance of RMWD staff. The entrance to the sampling area is located at 4141 Pala Rd, Fallbrook, CA 92028. The sampling area is currently used by Caltrans as storage yard and is located roughly 2000 feet southeast of RMWD. The two wells sampled were being actively pumped at the time of the sampling event. The location of the two sampling wells is shown in Figure 1. The Pope Well and Vessels Ranch Well, for which sampling results were presented in the CDM report, are also shown in Figure 1, along with the Mission Basin wells for the City of Oceanside. The methods used to analyze groundwater samples are listed in Appendix 1.

Figure 1. Aerial Map Showing the Well Locations for the Water Quality Data Evaluated in the Study

3.2 RAW WATER QUALITY SUMMARY

A sampling event was conducted in June 2015, yielding water quality results for each of the two wells sampled. The water quality data from the CDM report included sampling results for two wells (Pope Well and Vessels Ranch Well). Data available from the City of Oceanside for four wells (Wells 1, 2, 3, and 9) was also included in the water quality evaluation. A summary of all water quality data evaluated is presented in Table 1. The wells sampled by RMWD in 2015 are labeled “Well 1” and “Well 2” to differentiate them (see Figure 1 for their locations). The wells sampled in 2015 are in close proximity to one another, with only 800 feet separating them. A detailed water quality evaluation is provided below that considers the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells. Water quality parameters most likely to be relevant to groundwater treatment for the San Luis Rey River area were included in the sampling on this project. Once a site is determined, a full round of sampling of regulated constituents and non-regulated constituents with health-based advisory levels is recommended.

Table 1. Summary of water quality from sampled wells and other available sources

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
General Water Quality Parameters										
Calcium	mg/L	--	170	180	199	217	243	225	240	135
Magnesium	mg/L	--	82	76	73	78	88	82	115	55
Potassium	mg/L	--	6.7	7.9	9.5	8.8	9.7	9.8	9.0	9.0
Silica	mg/L	--	34	28	23.8	21.5	24.3	27.5	24	25
Sodium	mg/L	--	170	180	229	229	284	257	330	122
Hydroxide as OH	mg/L	--	ND	ND	--	--	--	--	<3	<3
Carbonate as CO_3	mg/L	--	ND	ND	--	--	--	--	<3	<3
Total Hardness as $CaCO_3$	mg/L	--	760	760	--	--	--	--	1080	567
Bicarb. Alkalinity as HCO_3	mg/L	--	280	260	325	322	341	331	413	262
Ammonia Nitrogen	mg/L	--	ND	0.24	0.6	0.2	0.3	0.4	0.3	0.20
Alkalinity in $CaCO_3$	mg/L	--	230	210	259	258	274	267	343	215
pH	Units	--	7.4	7.4	--	--	--	--	7.2	7.5
Orthophosphate as P	mg/L	--	0.031	0.034	<0.2	<0.2	<0.2	<0.2	--	--
Total Phosphorus as P	mg/L	--	--	--	0.16	0.09	0.09	0.09	0.050	0.100
Total Organic Carbon	mg/L	--	2.2	3.4	--	--	--	--	--	--
Surfactants	mg/L	--	ND	ND	--	--	--	--	--	--
Constituents with Primary Maximum Contaminant Levels (pMCLs)										
Inorganic Chemicals										
Aluminum	µg/L	1000	ND	ND	--	--	--	--	--	--
Antimony	µg/L	6	ND	ND	--	--	--	--	--	--

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
Arsenic	µg/L	10	ND	ND	--	--	--	--	--	--
Barium	mg/L	1	0.04	0.061	0.2	0.1	0.1	0.1	<0.1	<0.1
Beryllium	ug/L	4	ND	ND	--	--	--	--	--	--
Cadmium	µg/L	5	ND	ND	--	--	--	--	--	--
Chromium	µg/L	50	ND	ND	--	--	--	--	--	--
Fluoride	mg/L	2	0.45	0.26	0.3	0.5	0.4	0.3	0.40	0.40
Hexavalent chromium (Dissolved)	µg/L	10	ND	ND	--	--	--	--	--	--
Mercury	µg/L	2	ND	ND	--	--	--	--	--	--
Nickel	µg/L	100	ND	ND	--	--	--	--	--	--
Total Nitrate, Nitrite-N	mg/L	10	0.35	0.24	--	--	--	--	--	--
Nitrate as NO_3	mg/L	45	1.6	1.1	0.17	0.04	0.18	0.22	--	--
Nitrite Nitrogen	mg/L	1	ND	ND	--	--	--	--	--	--
Perchlorate	µg/L	6	ND	ND	--	--	--	--	--	--
Selenium	µg/L	50	ND	ND	--	--	--	--	--	--
Thallium	µg/L	2	ND	ND	--	--	--	--	--	--
Radionuclides										
No Data Collected										
Volatile organic chemicals (VOCs)										
No Data Collected										
Disinfection Byproducts (DBPs)										
No Data Collected										
Constituents with Secondary Maximum Contaminant Levels (sMCLs)										

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
Aluminum	µg/L	200	ND	ND	--	--	--	--	--	--
Chloride	mg/L	250 Rec 500 Upper	320	320	440	499	587	513	600	190
Apparent Color	ACU	15	20	75	--	--	--	--	25	15
Specific Conductance 25 C	umho/cm	900 Rec 1600 Upper	2100	2200	--	--	--	--	3400	1520
Copper	µg/L	1000	ND	ND	--	--	--	--	--	--
Iron	mg/L	0.3	0.80	3.5	3.3	1.3	2	2.6	0.80	0.04
Manganese	mg/L	0.05	0.20	0.65	0.62	0.46	0.48	0.62	0.57	0.22
Odor	TON	3	2.0	2.0	--	--	--	--	--	--
Silver	µg/L	100	ND	ND	--	--	--	--	--	--
Sulfate	mg/L	250 Rec 500 Upper	480	520	415	462	454	427	760	370
Total Dissolved Solids	mg/L	500 Rec 1000 Upper	1400	1500	1577	1673	1888	1754	2230	1080
Turbidity	NTU	5	9.0	50	--	--	--	--	6.7	0.4
Zinc	µg/L	5000	ND	ND	--	--	--	--	--	--
Constituents with Notification Levels (NLs)										
1,2,3-Trichloropropane	µg/L	0.005	ND	ND	--	--	--	--	--	--
Boron	mg/L	1	0.11	0.11	0.19	0.17	0.17	0.19	--	--
Vanadium	µg/L	50	ND	ND	--	--	--	--	--	--
Constituents with Action Levels (ALs)										
Copper	µg/L	1300	ND	ND	--	--	--	--	--	--
Lead	µg/L	15	ND	ND	--	--	--	--	--	--
Microbiological Parameters										

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
No Data Collected										

[1] It should be noted that the groundwater quality in the Mission and San Luis Rey River Groundwater Basins is similar and amenable to treatment by similar processes. This is important because one alternative under consideration is to treat the groundwater from the San Luis Rey River Groundwater Basin at the Mission Desalter. It is especially important to note that the Fe and Mn concentrations are similar and presumably need similar treatment (oxidation-filtration treatment prior to RO).

3.3 CONSTITUENTS WITH PRIMARY MAXIMUM CONTAMINANT LEVELS

There is water quality data available for a limited number of constituents with primary MCLs (pMCLs). The water quality data included for pMCLs represents either constituents most likely to be of potential concern or constituents relevant to the design of the treatment process. It should be noted that aluminum, antimony, arsenic, beryllium, cadmium, chromium, copper, lead, nickel, selenium, thallium, mercury, and perchlorate were not detected in either of sampled groundwater wells. Fluoride concentrations measured were 0.26 mg/L and 0.45 mg/L for the wells sampled in 2015. In comparison, fluoride concentrations measured for Pope Well and Vessels Ranch well were both 0.40 mg/L and Mission Basin well fluoride concentrations ranged from 0.3 to 0.5 mg/L. The fluoride concentrations showed consistency across all the well data evaluated. All fluoride measurements were below the state pMCL of 2 mg/L.

Barium concentrations measured for the wells sampled in 2015 were 0.040 mg/L and 0.061 mg/L. While these concentrations are lower than observed for the other wells evaluated (<0.1 to 0.2 mg/L), none of the concentrations observed are expected to be an issue for the treatment process. All measured barium values met the state MCL. Both samples of recently regulated hexavalent chromium were non-detect. Thus, no constituent with a pMCL is expected to be present at a concentration greater than its pMCL in the raw water for the design consideration. A full list of the regulatory pMCLs is shown in Appendix 2.

3.4 CONSTITUENTS WITH SECONDARY MAXIMUM CONTAMINANT LEVELS

Constituents with secondary MCLs (sMCLs) are not of concern to public health. Instead, they cause the aesthetics of the water to be of concern to sensitive consumers. Constituents with sMCLs in most cases have a discrete MCL, but there are a limited number of constituents that the State Water Resources Control Board Division of Drinking Water (DDW) regulates with a recommended sMCL and an upper sMCL limit. In most cases, new plants will be required to meet the recommended sMCL for drinking water applications. The water quality data showed that several constituents with sMCLs exceeded their recommended sMCL in both sampled wells. The findings are consistent with water quality results in CDM report and with the Mission Basin wells. They are listed in Table 2. These constituents will require treatment to produce a water quality that meets the recommended sMCLs. A full list of constituents with sMCLs is provided in Appendix 3.

Well 2 has a higher concentration of manganese and iron than Well 1. These results could be related to the higher apparent color observed in Well 2. The Pope Well, the Vessels Ranch Well, and the four Mission Basin Wells also had similar but slightly different results for TDS, conductivity, turbidity, and iron. The concentration of all of the constituents listed in Table 2 can be adequately reduced with a combination of iron and manganese (IM) and reverse osmosis (RO) treatment.

There are some observations that should be made about the sampling results of certain constituents for the wells sampled in 2015 given their close proximity. The turbidity in Well 1 and Well 2 varied from 9 NTU (Well 1) to 50 NTU (Well 2). The turbidity of 50 for Well 2 is unrealistic for a groundwater. From a water quality perspective, there is no reason to expect the turbidity of two wells so close together to differ by a factor of 5. The well condition may provide an explanation for the elevated turbidity. A well in need of rehabilitation or replacement may produce turbid water. Also, it is not uncommon to observe variations in iron levels in groundwater wells, because chemical reactions and biological activity occurring in the well structure affect the type of iron that is present in the dissolved form and the level. Wide variations in iron concentrations are seen across the wells evaluated for the study (see TDS response below). The Vessels Ranch Well was measured at 0.8 mg/L, which is same as Well “1”. The Mission Basin wells were measured at 2-3.3 mg/L, which is the same order of magnitude as Well “2". The Pope Well had a very low concentration of 0.04 mg/L.

3.5 CONSTITUENTS WITH NOTIFICATION LEVELS

Of the measured parameters with notification levels (NLs), none exceeded the NL. Boron in both wells was measured at 0.11 mg/L, well below the notification level of 1 mg/L. Vanadium and 1,2,3-trichloropropane were not detected in either well sampled by RMWD in 2015. Therefore, no constituent with a NL is expected to be present at a concentration greater than its NL in the raw water. It should be noted that a complete round of sampling of all constituents with NLs was not conducted in this study, which focused on sampling the constituents most likely to be of concern for SLR groundwater.

3.6 PROCESS DESIGN CONSIDERATIONS

Some important points to consider in process design are shown below for:

- IM removal
- RO sizing
- RO recovery and pretreatment
- Disinfection, and
- Product water stabilization

IM treatment targets the elevated concentrations of iron and manganese in the raw water and reduces them. One of the advantages of IM treatment is the fact that it typically achieves removals in excess of what is typically required. Therefore, the IM system can be designed based on the range of iron and manganese levels observed. In this project, the maximum iron concentration is 3.5 mg/L, which is a typical iron concentration for wash water recovery systems. The presence of some iron is beneficial, because it promotes the settling of manganese and iron particulates in the backwash water.

Table 2. Constituents in Sampling Wells that exceed the sMCLs

Parameter	Units	Regulatory Limit	San Luis Rey River Valley Wells, June 2015		Mission Basin Wells, 2009-2011				CDM Wells, 1996	
			1	2	1	2	3	9	Vessels Ranch Well	Pope Well
Chloride	mg/L	250 Rec 500 Upper	320	320	440	499	587	513	600	190
Apparent Color	ACU	15	20	75	--	--	--	--	25	15
Specific Conductance, 25 °C	umho/cm	900 Rec 1,600 Upper	2,100	2,200	--	--	--	--	3,400	1,520
Iron	mg/L	0.3	0.80	3.5	3.3	1.3	2	2.6	0.80	0.04
Manganese	mg/L	0.05	0.20	0.65	0.62	0.46	0.48	0.62	0.57	0.22
Sulfate	mg/L	250 Rec 500 Upper	480	520	415	462	454	427	760	370
Total Dissolved Solids	mg/L	500 Rec 1,000 Upper	1,400	1,500	1,577	1,673	1,888	1,754	2,230	1,080
Turbidity	NTU	5	9.0	50	--	--	--	--	6.7	0.4

If RO is employed to reduce the concentration of TDS, conductivity, chloride and other dissolved constituents, the sizing of the RO system will be determined by a combination of water quality goals including general mineral quality. The maximum recovery of RO will also be limited by constituents that may cause membrane scaling in the feed water, such as silica. In this case, antiscalant and/or pH adjustment may be required to increase the RO recovery.

It should be noted that the total organic concentration (TOC) levels observed in both wells sampled in 2015 (2.2 and 3.4 mg/L) are generally higher than typical groundwater, which commonly has TOC levels below 1 mg/L. The potential consequence is the fact that the level of disinfection byproducts (DPBs) may exceed the regulatory limit. Therefore, a bench scale Simulated Distribution System (SDS) test would be helpful to determine the potential for DBP formation for consideration in the design of the treatment facility.

Usually, raw water quality, including pH, temperature, alkalinity, and calcium, will affect the design of the product stabilization chemical system and the clearwell capacity. The blending of the RO permeate and the RO bypass (a portion of IM treated water that is not treated by RO) will help with these elements of the treatment train by increasing the mineral content of the product water from the treatment plant. The low mineral content and low buffer capacity in RO permeate must be adjusted to assure corrosion control and to provide a drinking water that meets aesthetic preferences of consumers.

3.7 DESIGN BASIS WATER QUALITY

During this raw water quality characterization, the design concentrations of iron and manganese were established based on elevated levels of iron and manganese observed in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells. As a conservative approach the design basis TDS was determined based on the level observed in the Vessels Ranch Well because the project team believes based on the groundwater modeling and hydrogeological analysis that the Vessels Ranch well is most typical of groundwater that would be pumped under the proposed project. The design basis TDS, iron, and manganese presented below are essentially a worst-case scenario. A worst-case scenario is commonly used as a design basis to assure the treatment plant can meet regulatory requirements during challenging conditions like drought.

The mineral water quality parameters required to size the RO are not constituents of concern with respect to the regulations. Therefore, their basis of design was determined based on averaging the water quality parameters in the wells sampled in 2015, the Pope Well, the Vessels Ranch well, and the Mission Basin wells. The raw water quality used as the design basis for the treatment technologies necessary to meet regulatory requirements are discussed below.

3.7.1 Constituents with Regulatory Limits and Health Advisory Levels

Almost all constituents with pMCLs were not detected in raw water. The only constituents with pMCLs detected were fluoride and barium, but the concentrations were well below their pMCLs. Therefore, the recommended design basis assumes that constituents with pMCLs are not present at levels requiring treatment.

A limited number of constituents exceeded their sMCLs in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells considered in establishing the basis for design. These parameters include manganese, iron (with the exception of Pope Well), chloride (with the exception of the Pope Well), sulfate, TDS, turbidity (with the exception of the Pope Well), specific conductance, and apparent color (Pope Well is at the limit of 15 ACU). These constituents require IM and RO treatment. Their recommended design basis is presented in the Table 3 and Table 4. The remaining constituents with sMCLs were present at levels below their sMCLs and do not require treatment.

Constituents with NLs, including 1,2,3-trichloropropane and vanadium, were not detected in the wells sampled in June 2015. Other constituent with NLs, like boron, were tested and determined to be present at concentrations less than their NLs. Therefore, no constituents with NLs were present at levels requiring treatment.

The RO system design basis requires evaluation of general mineral quality. Constituents in the raw groundwater, including silica and various ions like sulfate, calcium, and phosphate can lead to fouling and scaling of the RO membranes. For this reason, general mineral quality was considered in the analysis. The design basis for mineral quality was established based on the average levels observed in the wells sampled in 2015, the Pope Well, the Vessels Ranch Well, and the Mission Basin wells.

3.7.2 Design Water Quality for IM and RO

The design basis for the constituents of concern that exceed regulatory MCLs was established based on maximum or near maximum levels, as discussed above. For other water quality parameters critical to the design, averaging of water quality parameters shown in Table 1 was performed.

The design basis water quality for the IM treatment system is presented in Table 3. The recommended design basis for iron and manganese based on the maximum observed level is 0.65 mg/L for manganese and 3.5 mg/L for iron. It was observed that other wells considered in the design had elevated levels of iron approaching 3 to 3.5 mg/L (see Table 1), providing additional justification for the design basis established. The same is true for manganese. Other factors may affect the IM design, including natural organic matter, pH, aluminum, and turbidity. For example, ammonia may affect the design due to its chlorine demand. In the IM filter, turbidity will be removed through the filter along with iron and manganese, contributing to the headloss. Additional discussion of these factors is provided below. The design basis for all of the water quality parameters that affect IM treatment is shown in Table 3.

Ammonia affects the IM design by competing with iron and manganese to consume chlorine. Chlorine demand for ammonia is much higher than for iron and manganese, which is 8 chlorine:1 ammonia. Therefore, maximum ammonia concentrations observed in all the wells was considered as the design water quality basis for ammonia. TOC, especially when associated with color, may indicate the formation of complex iron compounds. Complex iron compounds will likely decrease the chlorine oxidation efficiency in operation. The pH and alkalinity affect the oxidation reaction kinetics. High pH and high alkalinity are favorable for the IM oxidation process. The presence of aluminum will alter the precipitation structure in the IM filter, which will affect the manganese removal.

During IM treatment, iron and manganese concentrations are reduced by oxidation and filtration. In the meantime, turbidity will also be removed by the filtration. Therefore, lower iron, manganese, and turbidity in the IM treated water should be considered for RO design.

The design basis for the RO system is presented in Table 4. For the RO system design, specific water quality goals, such as chloride and TDS play an important role. RO elements have a certain salt rejection rate. The chloride concentration will affect the selection of the RO membrane, because the ability of the membrane to reject chloride is an important consideration. The RO membrane area is also an important design factor.

RO recovery is dictated by the solubility of specific dissolved inorganic constituents. Silica, and other dissolved scaling minerals have fouling potentials in RO systems. In Table 4, the iron and manganese concentrations reflect a typical removal through the IM system. Similarly, the turbidity value reflects an assumed IM system treated water turbidity.

In addition to IM and RO treatment, disinfection and product stabilization design will be affected by the availability of raw water quality data for important constituents. DBP data was not available from wells sampled in 2015 or the additional water quality data used in the analysis. Therefore, DBP formation potential testing should be conducted to confirm that DBP formation in the treatment processes and distribution system does not exceed pMCLs.

Table 3. Recommended Design Basis Water Quality for Iron and Manganese Treatment System

Parameter	Units	Design Water Quality	Regulatory Limit	Water Quality from Table 1		
				Min	Average	Max
Alkalinity in $CaCO_3$	mg/L	257	--	210	257	343
pH	Units	7.4	--	7.2	7.4	7.5
Aluminum	µg/L	ND[1]	1,000 pMCL 200 sMCL	ND	ND	ND
Ammonia Nitrogen	mg/L	0.6	--	ND	0.3	0.6
Chloride	mg/L	434	250 Rec 500 Upper	190	434	600
Apparent Color	ACU	34	15	15	34	75
Specific Conductance, 25 °C	umho/cm	2,305	900 Rec 1,600 Upper	1,520	2,305	3,400
Iron	mg/L	3.5	0.3	0.04	1.79	3.5
Manganese	mg/L	0.65	0.05	0.20	0.48	0.65
Total Dissolved Solids	mg/L	2,000	500 Rec 1,000 Upper	1,080	1,638	2,230
Turbidity	NTU	17	5	0.4	17	50

[1] Below detection limit

Table 4. Recommended Design Basis Water Quality for Reverse Osmosis System Design

Parameter	Units	Design Water Quality	Regulatory Limit	Water Quality from Table 1		
				Min	Avg	Max
Aluminum	µg/L	ND[1]	1,000 pMCL 200 sMCL	ND	ND	ND
Barium	mg/L	0.1	1	0.04	0.10	0.20
Fluoride	mg/L	0.4	2	0.26	0.38	0.50
Nitrate as NO_3	mg/L	0.55	45	0.04	0.55	1.6
Chloride	mg/L	434	250 Rec 500 Upper	190	434	600
Specific Conductance, 25 °C	umho/cm	2,305	900 Rec 1,600 Upper	1,520	2,305	3,400
Iron	mg/L	0.1[2]	0.3	0.04	1.79	3.5
Manganese	mg/L	0.02[2]	0.05	0.20	0.48	0.65
Sulfate	mg/L	486	250 Rec 500 Upper	370	486	760
Total Dissolved Solids	mg/L	2,000	500 Rec 1,000 Upper	1,080	1,638	2,230
Turbidity	NTU	0.5[2]	5	0.4	17	50
Boron	mg/L	0.16	1	0.11	0.16	0.19
Calcium	mg/L	201	--	135	201	243
Magnesium	mg/L	81	--	55	81	115
Potassium	mg/L	8.8	--	6.7	8.8	9.8
Silica	mg/L	26	--	21.5	26	3.4
Sodium	mg/L	225	--	122	225	330
Ammonia Nitrogen	mg/L	0.3	--	ND	0.3	0.6
Alkalinity in $CaCO_3$	mg/L	257	--	210	257	343
pH	Units	7.4	--	7.2	7.4	7.5
Orthophosphate as P	mg/L	0.033	--	0.031	0.033	0.034
Total Organic Carbon	mg/L	2.8	--	2.2	2.8	3.4
Strontium[3]	mg/L	NA	--	--	--	--

[1] Below detection limit

[2] Assumed IM treated water levels

[3] Addition sampling is required in order to develop a recommended design water quality

4 WATER QUALITY GOALS

4.1 REGULATORY REQUIREMENTS

For a groundwater, regulatory considerations begin with a determination on whether the source is groundwater or groundwater under the direct influence (GWUDI) of surface water. If the groundwater is classified as GWUDI, it will be required to meet the requirements of the Surface Water Treatment Rule (SWTR).

In this project, the appropriate way to classify the source water was evaluated. The classification of the source water will impact the treatment plant design criteria. With source water being groundwater, the utility would only have to demonstrate 4-log (99.99%) virus removal/inactivation at a new plant where removal/inactivation of viruses is implemented, as opposed to greater disinfection requirements for GWUDI. Disinfection requirements for GWUDI are greater than or equal to 4-log virus removal/inactivation, greater than or equal to 3-log giardia removal/inactivation, and greater than or equal to 2-log cryptosporidium removal/inactivation.

The groundwater classification presumes that there is not a significant occurrence of large pathogens in the groundwater (e.g., Cryptosporidium, Giardia), and that the groundwater is not subject to significant and rapid shifts in water quality characteristics due to changes in the water quality of the SLR River (e.g., with respect to turbidity, conductivity, pH). Consequently, groundwater has lower pathogen removal and/or inactivation requirements compared to GWUDI (purveyors of groundwater are required to conduct either triggered source water monitoring or demonstrate 4-log virus removal/inactivation).

Based on an evaluation of the SWTR Guidance Manual, the rationale to support the conclusion that the source water for RMWD is ground water is as follows.

1. The water will be pumping out of a well (either horizontal or vertical).
2. Wells with perforations or screens less than 50 feet deep should be evaluated for direct surface water influence.

Based on discussion with West Yost on the implementation plan for the wells, it is expected that the source water will be pumped out from a well with roughly 80 feet in depth, but some of the wells may need to be evaluated for direct surface water influence. Therefore, the Groundwater Rule will be the regulatory approach that affects the design, not the SWTR.

The SWTR Guidance Manual suggests that well design criteria including the sanitary seal, the location of screens, etc., should be reviewed to confirm source water classification in the future design and construction. The casing or nearest collector lateral should be at least 200 feet from any surface water. Water quality records should indicate no record of total or fecal coliform contamination, no history of turbidity problems, and no known history of Giardia or Crypto outbreaks associated with the well.

4.1.1 pMCLs

The pMCLs are established at levels that may have an adverse effect on the public health. They are legally enforceable limits. They are in place for inorganics, organics (volatile organic chemicals and synthetic organic chemicals), pathogens, radionuclides, and DBPs. The recommended water quality goal for constituents with pMCLs is to meet the pMCL. A summary of the constituents with pMCLs is shown Appendix 2.

An evaluation of the raw water quality data showed that measured constituents with pMCLs were in no case detected at levels exceeding a pMCL. Therefore, the only constituents with pMCLs worthy of further consideration in the project are those constituents impacted by the treatment process. The most important such constituents are DBPs. DBPs are discussed in more detail below.

4.1.2 sMCLs

Secondary MCLs (also known as Consumer Acceptance Levels) are based on aesthetics. Fixed sMCLs are provided by DDW for some constituents. A range of concentrations have been established for other constituents. The lower end of the range is termed a Recommended Level while the higher end of the range is termed an Upper Level. The recommended levels are more acceptable to consumers. The Upper Levels may be acceptable if it is not feasible or reasonable to deliver water that meets a lower level.

Review of the water quality data shows that treatment would be required to meet the fixed sMCL for color, iron, manganese, and turbidity, and the Recommended sMCL for TDS, conductivity, chloride, and sulfate.

Iron and manganese can cause colored water episodes, detectable by consumers at concentrations below their sMCLs. Consumers can detect iron in the water at levels approaching 100 µg/L. Manganese can be detected by consumers at levels as low as 20 µg/L. IM treatment with greensand filtration is easily capable to meet these levels, which will be set as the water quality goals for this project. A summary of water quality goals that includes iron and manganese is shown in Section 4.4). A summary of constituents with sMCLs is shown in Appendix 3.

RO treatment will reduce the chloride, sulfate, TDS, conductivity, and color concentrations to below their fixed and recommended sMCLs. The water quality goals for constituents with sMCLs are shown in Section 4.4. All other constituents with sMCL goals in Appendix 3 do not require treatment.

4.1.3 Groundwater Rule

In this report, it is assumed that the groundwater is not under the direct influence of surface waters, as discussed above. Therefore, the Groundwater Rule applies to this system.

The Ground Water Rule requires sanitary surveys of groundwater systems. The Groundwater Rule also requires triggered source water monitoring or 4-log removal/inactivation of viruses. The triggered source water monitoring is based on monitoring of E. Coli, enterococci, or coliphage. With disinfection treatment, it is required to continuously demonstrate 4-log removal of viruses. Triggered source water monitoring can prove complicated, with the possibility that treatment would need to be added once the well is built. For that reason, chlorine disinfection will be included to assure compliance with the treatment requirements of the Groundwater Rule in the preliminary design (4-log virus removal/inactivation). If the well were operational, sampling could be conducted to assure that the groundwater system is likely to meet disinfection requirements of the Groundwater Rule through the triggered source water monitoring.

4.1.4 Disinfectants and Disinfection Byproducts Rule

The Stage 1 and 2 Disinfectants and Disinfection Byproduct Rules (D/DBPR) established pMCLs for several DBPs. DBPs with pMCLs including trihalomethanes (THMs), haloacetic acids (HAAs), bromate, and chlorite. There are four regulated THMs collectively termed total trihalomethanes (TTHM) including chloroform, bromoform, dibromochloromethane, and bromodichloromethane. There are five regulated HAAs collectively called HAA5 including dichloroacetic acid, trichloroacetic acid, chloroacetic acid, bromoacetic acid, and dibromoacetic acid.

The D/DBPR also established maximum residual disinfect levels (MRDLs) for a number of oxidants. The oxidants include chlorine, chloramines, and chlorine dioxide. The Stage 1 D/DBPR established requirements for TOC removal using enhanced coagulation in filtration plants. The MRDLs for chlorine, chloramine, and chlorine dioxide are listed in Table 5. The recommended water quality goal for disinfectants and DBPs are not to exceed their pMCLs and MRDLs, as shown in Section 4.4.

Table 5. Maximum Residual Disinfectant Levels (MRDL)

Disinfectant	Units	MRDL
Chlorine	mg/L as Cl_2	4.0
Chloramines	mg/L as Cl_2	4.0
Chlorine Dioxide	mg/L	0.8

The generation of DBPs requires the coexistence of a disinfectant and DBP precursors. The principal DBP precursors in the water are large organic molecules and the bromide ion. Both of these precursors can be effectively removed by RO. Free chlorine is a more effective oxidant for virus inactivation than chloramine and chloramine requires a much longer contact time for the same level of virus removal. One the other hand, free chlorine decays rapidly in the distribution system and can form a significant concentration of THMs and HAA5, especially when precursors like natural organic matter are present at elevated levels. While chloramines will generate significantly lower levels of TTHM and HAA5, nitrosamines, such as N-nitrosodimethylamine (NDMA) are a byproduct of chloramine disinfection. NDMA has a DDW health-based advisory level (NL) of 10 ng/L. Despite the risk of NDMA formation, chloramines are the current industry standard for secondary disinfection where THM and HAA formation is a concern, or where chloramines are already used in the distribution system. MWD maintains a chloramine residual targeting 2.5 mg/L in the distribution system. They have done extensive testing of NDMA levels throughout their treatment plants and treated water pipelines. They have observed levels of NDMA in full compliance with the NDMA NL.

For this project, free chlorine will be used for primary disinfection to decrease the size of the clearwell. Chloramines will be used for secondary disinfection to ensure adequate residual disinfectant in the distribution system and reduce the potential for DBP formation in the distribution system. However, a fairly high level of TOC in the raw water will undergo chlorination prior to the IM treatment. Chlorine is fed upstream of IM treatment to provide optimal conditions for oxidation of manganese on the manganese dioxide surface of the greensand filters. This chlorination of the raw water could potentially form high levels of DBPs exceeding the regulatory limit. A DBP formation potential study is recommended in the future (SDS test, as discussed above).

4.2 AGRICULTURAL CONSIDERATIONS

4.2.1 Public Drinking Water Supply

RMWD is located in a major agricultural area. The presence of agriculture interests in the area will have a strong influence on the water quality goals established at the groundwater treatment plant. The water quality goals to be established need to consider the potential impact on plants and other agricultural crops of constituents like boron, sodium, and chloride, among others.

The SWRCB developed general water quality guidelines for standard agricultural practice in the interpretation of irrigation water quality. These guidelines were developed around the long-term influence of water quality on crop production. They represent a water quality that can be used without restriction or without special management practices. In the absence of site-specific information, these guidelines (Table 6) can offer direction for developing agricultural goals.

Table 6. Agriculture Industry Guidelines for Interpreting Irrigation Water Quality[1]

Parameter	Units	Degree of restriction on use [2]			Raw Groundwater Quality	Skinner WTP Treated Water Quality
		None	Slight to moderate	Severe		
EC	µS/cm	<700	700-3,000	>3,000	2,150	813
TDS	mg/L	<450	450-2,000	>2,000	2,000	482
Sodium	mg/L	<69	>69		175	79
Chloride	mg/L	<142	142-355	>355	320	85
Boron	mg/L	0.7	0.7-3.0	>3.0	0.11	--
Nitrate	mg/L-N	<5	5-30	>30	0.30	0.9
Bicarbonate	mg/L	<92	92-519	>519	--	133
pH		Normal range 6.5-8.4			8.21	8.21

1. Adapted from Ayers 1977 and Ayers 1985. Assumes semi-arid to arid climate and low rainfall; sandy loam to clay loam soils with good internal drainage; no uncontrollable shallow water table present within 2 meters of surface; a leaching fraction of 15-20% LF; infrequent irrigations; and a 40-30-20-10% root water uptake pattern.
2. Full production capability of all crops, without use of special practice, is assumed when the guidelines indicate no restrictions on use. A "restriction on use" indicates that there may be a limitation in choice of crops, or special management may be needed to maintain full production capability. A "restriction on use" does not indicate that water is unsuitable for use.

The most common agricultural crops in RMWD are avocados, citrus, nursery, and nut crops. Based on research, it is known that these plants are sensitive to chloride. A previous project conducted by members of the project team found avocado trees might develop tip burn when excessive chloride accumulates in their leaves. Other chloride sensitive plants include fruit crops (almond, apricot, banana, citrus, grapes, mango, peach), berries (incl. strawberry), vegetables (lettuce, onions, sweet pepper), field crops (potato, tobacco), coffee and flowers. The SWRCB guidelines specific to particularly sensitive crops are presented in Table 7.

Table 7. Guidelines Specific to Particularly Sensitive Crops[1]

Parameter (units)	Limit	Sensitive Species
EC (µS/cm)	600	Turnip
	700	Beans, Carrot, Strawberry
	800	Radish, Onion
Chloride (mg/L)	118	Avocado, Shasta Strawberry, Indian Summer Raspberry
Boron (mg/L)	0.5	Lemon, Blackberry

1. Adapted from Ayers 1977 and Ayers 1985. Assumes semi-arid to arid climate and low rainfall; sandy loam to clay loam soils with good internal drainage; no uncontrollable shallow

water table present within 2 meters of surface; a leaching fraction of 15-20% LF; infrequent irrigations; and a 40-30-20-10% root water uptake pattern.

Comparing the water quality to the guidelines, treatment will not be needed for pH, boron, and nitrate so water quality goals will be set at the agricultural guideline. On the other hand, EC, TDS, sodium, and chloride would require treatment to meet SWRCB guidelines for agricultural crops.

To appropriately develop a reasonable water quality goal for constituents requiring treatment, the imported water quality was compared with the water quality for agricultural considerations. The conductivity, TDS, sodium, and chloride in the treated water from Robert A. Skinner Water Treatment Plant (Skinner WTP) were obtained from RMWD by averaging monthly data from January 2012 to April 2015. Chloride in the imported water meets both the general guideline and guideline for sensitive crops. With respect to EC, TDS, and sodium, the imported water does not meet the general guidelines. But the imported water is already being successfully used for agricultural purposes in the area, in this case, the recommendation is to match the imported water quality. Therefore, the agricultural water quality goal for electrical conductivity, TDS, sodium, and chloride is set to match the imported water quality. The water quality goals for agriculture are presented in Table 8.

Table 8. General Agricultural Water Quality Goals

Parameter (units)	Goal	Rationale
Electrical Conductivity (µS/cm)	813	Imported Water Quality
Total Dissolved Solids (mg/L)	482	Imported Water Quality
Sodium (mg/L)	79	Imported Water Quality
Chloride (mg/L)	85	Imported Water Quality
Boron (mg/L)	0.7	General Agricultural Guideline
Nitrate	5	General Agricultural Guideline
pH	6.5-8.4	General Agricultural Guideline

Based on the tables above, it was determined that the water quality goals established to meet agricultural requirements are in this case more stringent than the requirements for drinking water. For example, sodium and chloride levels are 69 and 118 for agriculture, versus no limit and 250 (recommended) for drinking water. This observation means a higher level of RO treatment would be required for agricultural considerations than for drinking water supply. However, in this project, specific water quality parameters were taken into account in developing the water quality goals to assure the treated water quality meets the requirements both for a drinking water supply and for assurance that crops will not incur incremental damage in comparison to the imported water supply.

4.2.2 Landscape Irrigation

Another possible application of groundwater considered on the project is landscape irrigation. As discussed above, similar levels of RO treatment will be needed to meet water quality goals for agriculture as compared to drinking water. According to California Water Code Section 13550-13557, "the use of potable domestic water for nonpotable uses, including, but not limited to, cemeteries, golf courses, parks, highway landscaped areas, and industrial and irrigation uses, is a waste or an unreasonable use of the water ……if recycled water is available". Therefore, landscape irrigation is not considered a viable alternative to application as a public drinking water supply and was not analyzed further.

4.3 BLENDING WITH IMPORTED WATER FOR DELIVERY TO RMWD

It is important to understand the application to set appropriate water quality goals. Treated SLR groundwater blending with imported water for delivery to RMWD is a potential way to increase the water supply with a possible reduction in level of treatment required based on dilution provided by the imported water supply. It is essential to understand the imported water quality to determine the possible benefit of blending with imported water.

The primary source of water for the RMWD is imported water through the SDCWA, which provides water from MWD, through the Skinner WTP. MWD typically supplies this plant with large amounts of both State Project Water (SWP) and Colorado River Water (CRW). The water quality of Skinner WTP treated water from 2012 to 2015 was obtained from RMWD (see Appendix 4). The concentrations in Skinner WTP treated water were evaluated with respect to the product water quality goals proposed for the new SLR groundwater treatment facilities (see Section 4.4). The average concentration in Skinner WTP treated water for constituents related to aesthetic concerns is shown in Table 9.

The summary in Table 9 shows that the average imported water quality is slightly below than the MCLs. However, the SWP portion has decreased substantially since 2014 due to extreme drought conditions. This has increased the amount of CRW in the Skinner WTP treated water. Certain minerals are elevated in the CRW supply compared to the SWP, as evidenced by higher TDS levels, for example. For this reason, various constituents that may be of aesthetic concern have been increasing in concentration, even exceeding the sMCLs in the case of TDS and EC. Such constituents include TDS, EC, sulfate, and chloride. The water quality changes over time are shown in Figure 2 and Figure 3. The changes in chloride, sodium, and sulfate levels are shown in Figure 2. There are no sMCL exceedances, but small increases in sodium and chloride have been observed, with a larger increase in sulfate concentrations. This is not surprising as CRW is known to have elevated levels of sulfate compared to SWP. The most recent sulfate data is approaching, but just under the sMCL. The same trend observed for sulfate is seen for TDS and EC, as shown on Figure 3. The difference is that the TDS and EC levels have exceeded sMCLs in multiple samples over the past several months. TDS has exceeded 600 mg/L, a level considerably above the sMCL of 500 mg/L. A probability plot of Skinner WTP treated water TDS concentration from Jan. 2012 to Apr. 2015 is shown in Figure 4. Figure 4 shows a median TDS level of 520 mg/L based on 2012-15 data. As discussed above, the imported water had an average of 482 mg/L TDS concentration based on data from January 2012 to April 2015. As discussed above, the persistent drought in recent years has caused elevated levels of TDS. As shown in Figure 3, the TDS concentration increased as high as 639 mg/L. It is uncertain what the future TDS will be in imported water, thus 90 percent level for TDS concentration in imported water was considered in the evaluation of blending. As shown in Figure 4, the 90 percent level is 624 mg/L.

Table 9. Average Water Quality of Skinner WTP from 2012 to 2015

Constituents	Units	MCLs	Monthly Average from January 2012 to April 2015
Silica	mg/L	--	8.9
Calcium	mg/L	--	54
Magnesium	mg/L	--	20
Sodium	mg/L	--	79
Potassium	mg/L	--	4.0
Carbonate	mg/L	--	0.21
Bicarbonate	mg/L	--	133
Sulfate	mg/L	250 Rec 500 Upper	162
Chloride	mg/L	250 Rec 500 Upper	85
Nitrate	mg/L	45	0.9
Fluoride	mg/L	2	0.8
TDS	mg/L	500 Rec 1000 Upper	482
Total Hardness as $CaCO_3$	mg/L	--	220
Total Alkalinity as $CaCO_3$	mg/L	--	109
Free Carbon Dioxide	mg/L	--	1.5
pH	--	--	8.21
Specific Conductance	µS/cm	900 Rec 1600 Upper	813
Color	CU	15	1
Turbidity	NTU	5	0.06
Temperature	°C	--	22
Saturation Index	--	--	0.53

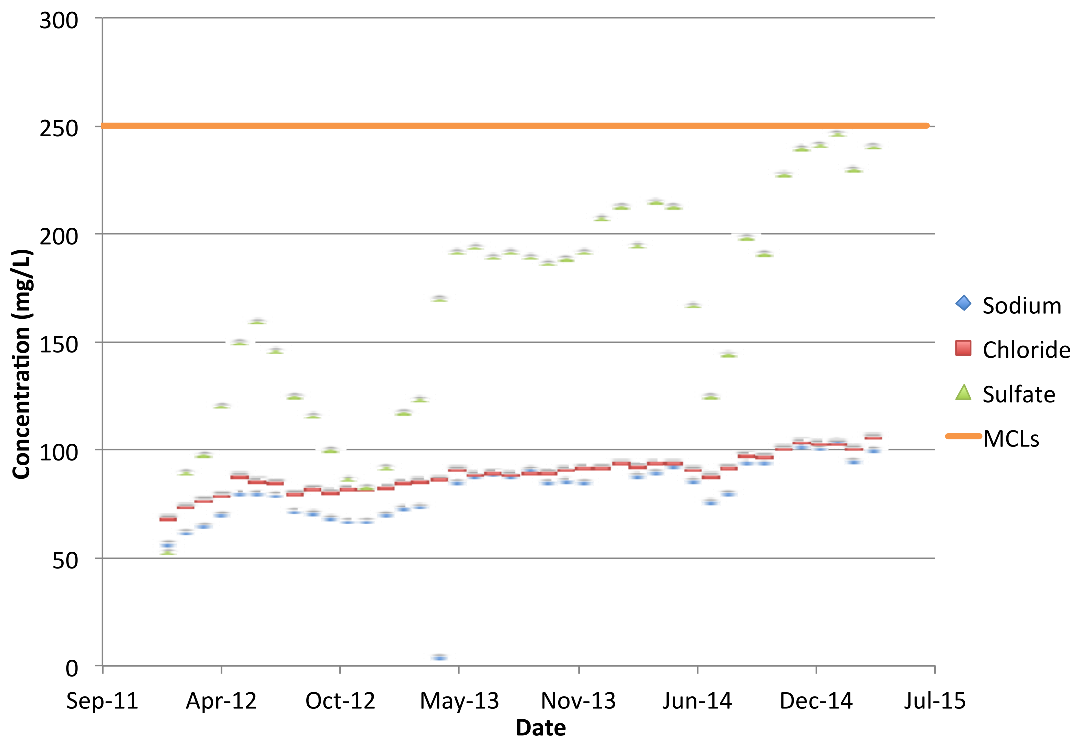

Figure 2. Imported Water Sodium, Chloride, Sulfate Monthly Data and MCLs

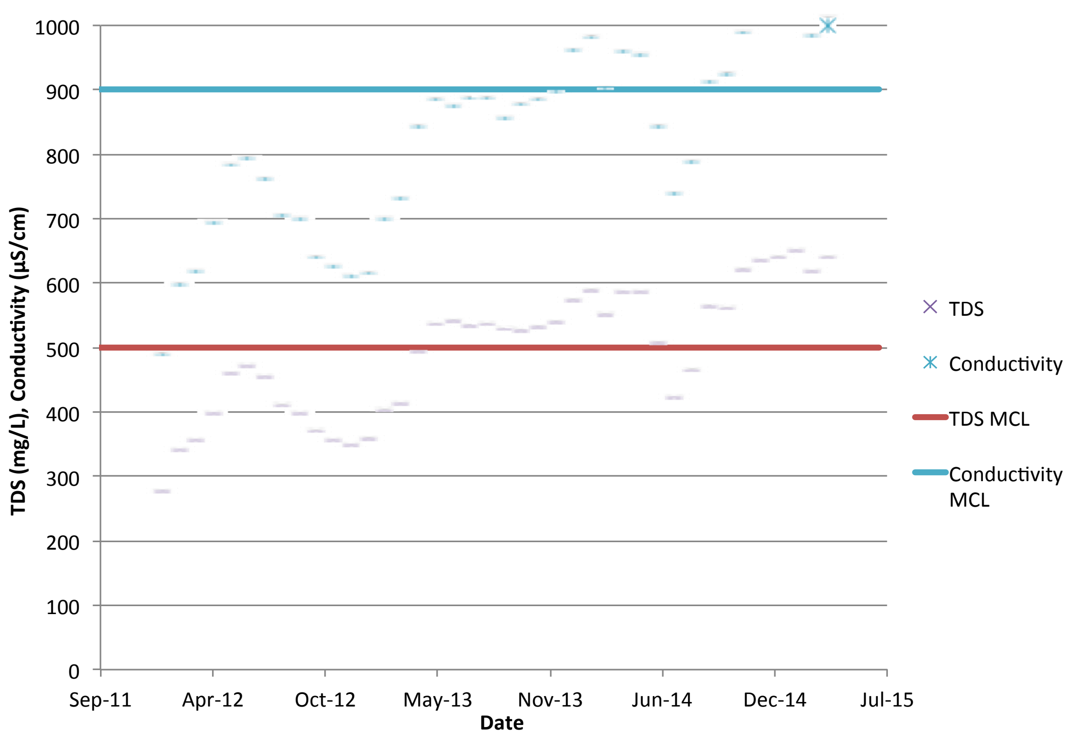

Figure 3. Imported Water TDS, Conductivity Monthly Data and MCLs

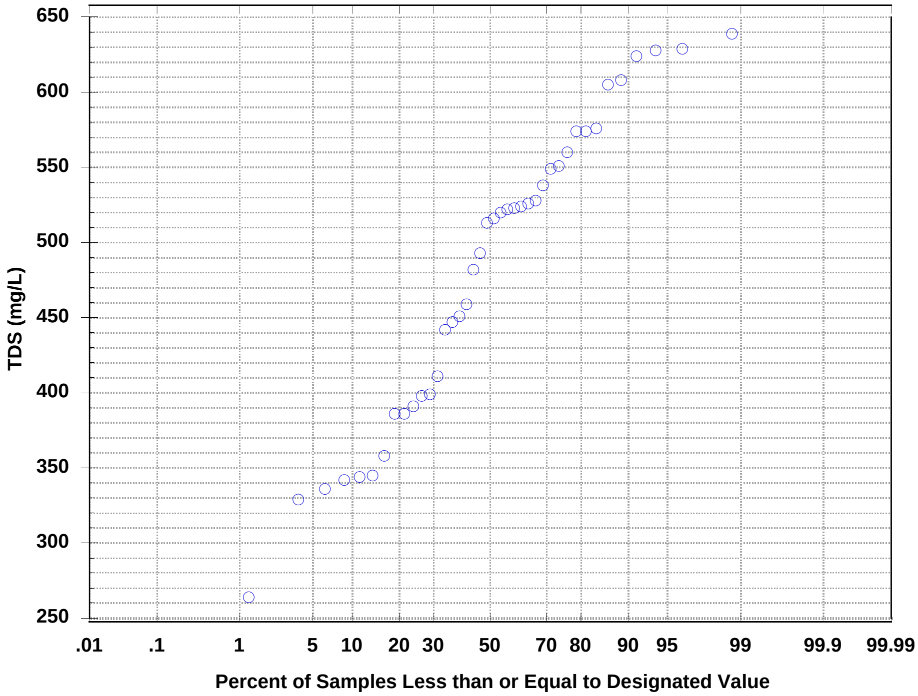

Figure 4. Probability Plot of Skinner WTP Treated Water TDS Concentration from January 2012 to April 2015

A blending analysis was conducted as part of the study. RO capacity was evaluated as a function of the amount of imported water required for blending and the RO bypass capacity. The analysis was performed on blending different amounts of imported water with RO permeate and/or RO bypass. The goal of this evaluation is to assure compliance with TDS MCLs. Therefore, only the TDS concentration was considered in the analysis. There were four blending scenarios considered in the analysis, involving 5, 10, 15, and 19 mgd of imported water. The 19 mgd scenario represents the projection of normal year water demand for RMWD by 2020, according to the RMWD 2010 Urban Water Management Plan. The case with no blending is also included in the results for comparison. In Table 10, the amount of RO design capacity and RO bypass capacity to meet the TDS sMCL are presented for the different blending scenarios.

To blend with 19 mgd imported water, a RO system has to be designed at 6.68 mgd without bypass capacity to comply with the TDS MCL. Compared with the zero blending option, the size of the RO is 0.33 to 3.76 mgd larger depending on imported water flow. Therefore, the blending option will require a higher RO design capacity and associated cost. For this reason, the blending with imported water option was not considered further.

Table 10. Scenario Projection of Blending Product Water with Imported Water to Comply with TDS sMCL

Blending with Imported Water (mgd)	RO Capacity (mgd)	RO Bypass (mgd)
19	6.68	0
15	5.29	0
10	3.58	0.02
5	3.25	0.35
0	2.92	0.68

4.4 RECOMMENDED WATER QUALITY GOALS

Based on the discussion above, the recommended water quality goals are summarized in Table 11. The levels of iron and manganese, in addition to TDS and chloride, when compared to the design basis raw water quality (see Table 3 and Table 4) will require IM and RO treatment. A detailed discussion of the treatment processes required for iron and manganese removal and groundwater desalination is provided in Section 5.

Table 11. Design Water Quality Goals[a]

Constituent	Goal	Rationale
Iron (mg/L)	0.1	Aesthetics (colored water), Goal Set at Detection Limit
Manganese (mg/L)	0.020	Aesthetics (colored water), Goal Set at Detection Limit
TDS (mg/L)	482	Aesthetics, Goal Set to Match Imported Water Quality
Conductivity (µS/cm)	813	Aesthetics, Goal Set to Match Imported Water Quality
Chloride (mg/L)	85	Agriculture, Goal Set to Match Imported Water Quality
Sodium (mg/L)	79	Agriculture, Goal Set to Match Imported Water Quality
Sulfate (mg/L)	250	Aesthetics, Goal set at sMCL
Boron (mg/L)	0.7	Agriculture, Goal Set to SWRCB General Guideline
Apparent Color (CU)	15	Aesthetics, Goal Set at sMCL
Turbidity (NTU)	5	Aesthetics, set at sMCL

[a] Treatment goals for iron and manganese are based on consumer expectations and exceed the percentage reductions needed to meet the sMCLs

5 WATER TREATMENT

Based on the design basis water quality and the water quality goals, process selection will be accomplished by identification and selection of treatment technologies that allow for the treatment train to meet water quality goals. Following process selection, a conceptual design of the treatment processes selected will be prepared. Conceptual Level Capital and Operations and Maintenance (O&M) costs will also be presented. This section includes (1) Process Selection, (2) Process Flow Diagram and Description, (3) Iron and Manganese Treatment, (4) RO Treatment System, (5) Disinfection and Stabilization, (6) Chemical Feed and Storage, (7) Treated Water Quality, (8) Simplified Conceptual Layout, and (9) Conceptual Level Capital and O&M Costs.

5.1 PROCESS SELECTION

Constituents requiring treatment for the feed design basis water quality shown in Table 3 and Table 4 to meet the water quality goals in Table 11 are presented in Table 12. Based on Table 12, iron and manganese need greater than 95% removal to meet the water quality goal. Chloride and TDS need more than 75% removal. Sodium, conductivity, and sulfate need some extent of removal as well. For target constituents that need to be removed, a selection of treatment technologies was made, with a detailed explanation provided below.

Table 12. Design Water Quality Compared to Water Quality Goals

Constituent	Design Basis	WQ Goal	%Removal to Meet WQ Goal
Iron (mg/L)	3.5	0.1	97.1
Manganese (mg/L)	0.65	0.02	96.9
TDS (mg/L)	2,000	482	75.9
Conductivity (µS/cm)	2,305	813	64.7
Chloride (mg/L)	434	85	80.4
Sodium (mg/L)	225	79	64.9
Sulfate (mg/L)	486	250	48.6
Boron (mg/L)	0.16	0.7	0
Apparent Color (CU)	34	15	55.9
Turbidity (NTU)	5.4	5	7.4

For iron and manganese, treatment options include sequestering, ion exchange, oxidation followed by conventional filters, and oxidation followed by greensand filters. Sequestering only prevents the iron and manganese from oxidizing but does not remove them. This is only an option if the iron is in the form of ferrous iron and manganese as manganous ion. Ion exchange softening is another option for the removal of iron and manganese. However, ion exchange is only an option when the iron and manganese combined concentration is low, preferably lower than 2 mg/L. This is because high concentrations may cause precipitated iron residue buildup on the resin, decreasing the efficiency of the ion exchange process.

Another common removal process is oxidation followed by filtration with conventional granular media filters (GMF). Iron and manganese are oxidized by free chlorine upstream of the GMF. After the oxidation, soluble ferrous iron (Fe^{2+}) is oxidized to insoluble ferric iron (Fe^{3+}) and soluble Mn(II) is oxidized to insoluble Mn(IV). The addition of chlorine will also result in the formation of a manganese oxide surface on the conventional filter that will catalyze the removal of manganese in the conventional filters. However, it is not as effective as greensand filtration, especially for manganese, and has an acclimation period to build the manganese oxide surface.

Greensand filtration has been used for decades for IM treatment. In recent years, the greensand media has been improved, resulting in lower O&M burden as the greensand does not need to be regenerated regularly. Like with the free chlorine/GMF alternative above, Fe(II) is oxidized to Fe(III) readily forming the insoluble iron hydroxide complex $Fe(OH)_3$, and Mn(II) is oxidized to Mn(IV) forming insoluble manganese dioxide (MnO_2). The insoluble metals can be precipitated out in a settling tank or removed by filtration.

There are different preoxidation approaches available, such as aeration, chlorine oxidation and potassium permanganate oxidation. The oxidation reactions with aeration are too slow to be viable, as pH greater than 9.5 would be required. Potassium permanganate oxidation is more expensive than the alternatives, and it needs more operational attention due to the fact that overdosing will leave a pink tinge in the water. Free chlorine oxidation is another commonly used approach and has proven very effective with greensand filtration. Given the concentration of iron and manganese in the groundwater and the pH of the raw water, chlorine oxidation is the best and most cost effective approach for this project. Based on the rationale above, the treatment process for iron and manganese will be:

- **Oxidation by Free Chlorine**
- **Greensand Filtration**

The oxidation step will be referred to as "preoxidation" because it occurs upstream of the greensand filtration process it drives. Technologies considered for TDS and mineral removal included RO, NF, and electrodialysis. Electrodialysis was ruled out as it was shown to be less cost effective than RO in the CDM report and it is not commonly used in the water industry for groundwater desalination. NF is more common and requires lower pressure than RO membranes, but it does not reject salt particularly well or remove monovalent ions effectively. For this reason, NF was ruled out given the high level of TDS in the groundwater and the low chloride water quality goal.

For the TDS and other inorganic constituents, RO is the most commonly used treatment process among the best available technologies (BAT) for inorganic chemicals listed in California Regulations Related to Drinking Water. It is a pressure driven membrane separation process that removes dissolved contaminants from water. It is driven by the passage of a solvent (e.g., water) through a semi-permeable membrane from a solution of higher concentration to a solution of lower concentration against the concentration gradient. This is achieved by applying pressure greater than the osmotic pressure to the more concentrated solution. RO treatment is more expensive than conventional treatment processes. Only the amount of water required to meet the TDS/mineral treatment target in RO will be fed to the RO. The rest will bypass the RO.

RO permeate is low in minerals, soft, and low in alkalinity. Waters of similar quality have been shown to be aggressive towards the distribution system. Blending with RO bypass water will improve the water quality and its corrosivity, but blending alone is typically not sufficient because the percentage of blending water is too low. Stabilization needs to be considered as part of the treatment train to prevent corrosion in the distribution system.

Disinfection, as discussed above is needed to achieve 4-log virus removal/inactivation to comply with groundwater regulations. Therefore, disinfection will be included in the treatment train to be described below.

5.2 PROCESS FLOW DIAGRAM AND DESCRIPTION

The process flow diagram (PFD) is shown in Figure 5. An overview of equipment capacity and purpose is shown in Table 13 below. Design raw water quality and treatment goals are summarized in Sections 3 and 4. The water treatment plant equipment was sized based on the water quality goals presented in Section 4. The maximum groundwater that the plant can treat is 4,000 AF/Y (3.6MGD), which was determined from groundwater modeling. The treatment plant includes the following major components:

- Iron and manganese removal with sodium hypochlorite oxidation and greensand filtration
- Desalination via:
  Low pressure, low energy RO membrane;
- Disinfection and stabilization in the clearwell using:
  Sodium hypochlorite for primary disinfection, followed by addition of ammonium hydroxide to form chloramines for secondary disinfection,

caustic (NaOH) addition prior to distribution to minimize the potential corrosion in distribution pipeline.

Feed water is stored in an EQ tank at low flows to provide sufficient volume for downstream processes. After oxidation with sodium hypochlorite, the IM filters are able to remove iron and manganese. The iron and manganese system rejects stream flow to a waste wash water (WWW) recovery tank for solids separation. In the WWW tank, the liquid is decanted and returned to the feed water pipe to the iron and manganese treatment process while the solids slurry is separately pumped to sludge drying beds.

Downstream of the IM filter, RO treatment is designed to reduce the TDS in the water. To maximize the RO treatment process efficiency, the treated water from the IM plant is split into two (2) lines prior to desalination. Some of the flow bypasses the RO unit and receives no further treatment until post-treatment disinfection in the clearwell. The maximum flow of this RO bypass is 0.46 MGD due to the high salinity of raw water. The remaining flow of 3.1 MGD is fed to the RO. The RO permeate flow rate of 2.51 MGD is determined using the RO membrane salt rejection capability and the water recovery of 80 percent. The blended flow ensures a final TDS concentration of 385 mg/L (80 percent of TDS goal), and the concentration of chloride is 68 mg/L (80 percent of chloride goal) for agricultural application. All other parameters are within regulatory requirements in the clearwell as well.

Primary free chlorine disinfection is designed to obtain at least 4 log virus inactivation removal credit. Secondary disinfection with chloramines is designed to maintain the disinfectant residual in the distribution system. The product water was evaluated for corrosion control. Caustic soda (NaOH) addition is designed to ensure the Langelier Saturation Index (LSI) of the treatment plant product water is in the acceptable range for distribution. Based on LSI, the blended RO product water quality will be compatible with the existing system water quality.

Design criteria are presented below for equipment associated with IM and RO treatment. Design criteria for chemical feed and storage are presented after the sections on IM and RO treatment.

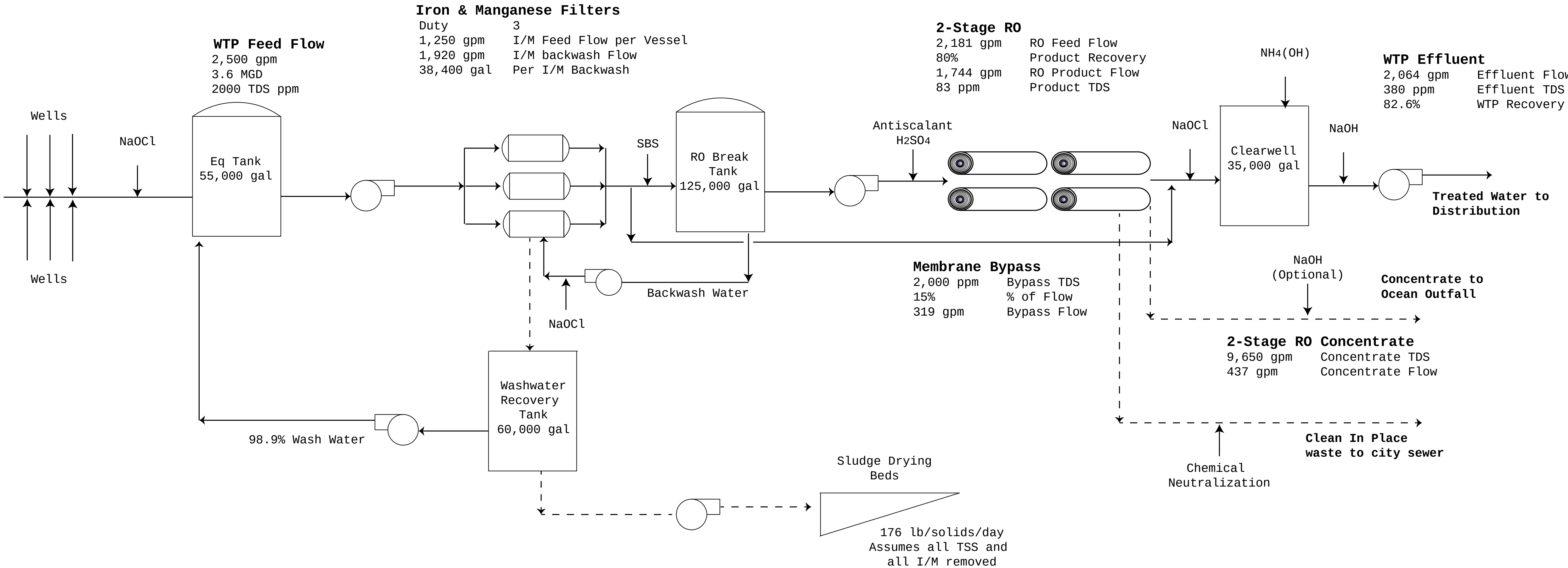

Figure 5. Process Flow Diagram

Table 13. Major Components in the Treatment System

Equipment	Capacity	Purpose
Equalization (EQ) Tank	50,000 gal	Provides storage to assist with controlled startup and shutdown of the plant.
IM Treatment Vessels	3 IM vessels, 10 feet in diameter by 32 feet long	Removes iron and manganese from raw water.
RO Feed Tank	125,000 gal	1. Provides additional contact time for the dechlorination of the IM treated water to protect the downstream RO membranes from oxidation. 2. Provides wash water storage volume for backwashing the IM media.
IM Vessel Back Wash Pump	1,920 gpm per cell	Provides water to perform backwashing of the IM vessels.
Waste Wash Water (WWW) Recovery Tank	80,000 gal	1. Holds the IM backwash volume of one vessel. 2. Provides sludge settling time.
Treated WWW Recycle Pump	375 gpm	Recycles the supernatant from the WWW recovery tank to the front of the EQ tank.
WWW Sludge Pump	50 gpm	Transfer sludge to the drying beds.
RO Membrane System	3.14 mgd 3 trains	Reduces TDS and chloride

5.3 IRON AND MANGANESE TREATMENT

The purpose of the IM treatment is to lower the concentrations of iron and manganese to levels that prevent the aesthetic disadvantages associated with these inorganics. IM pretreatment also will prevent iron and manganese related fouling and scaling of the RO membranes located downstream.

In the natural geology of an aquifer, the interaction between the iron-bearing soil and water dissolves the iron into the groundwater, where it is present in the reduced or ferrous form [Fe(II)]. Similar to iron, manganese is found in the reduced or manganous form [Mn(II)]. The presence of iron and manganese in water supplies can lead to several aesthetic problems. They impart a metallic taste, promote the growth of microorganisms in reservoirs and distribution systems, and stain laundry and fixtures. Iron and manganese are perceptible to consumers as colored water at low levels, as low as 0.02 mg/L for Mn.

To produce aesthetically acceptable water, the sMCLs limit iron and manganese to 0.3 mg/L and 0.05 mg/L, respectively. Unfortunately, iron and manganese can be easily detected by the pallet of some consumers and via color observed in the water at concentrations below their sMCLs. Iron is detectable at concentrations down to 0.10 mg/L and manganese is detectable at concentrations down to 0.020 mg/L. Therefore, the goal for removing these constituents should be based on their respective detection limits. The raw water quality and treatment goals necessary to design the IM treatment are summarized in Table 14.

Table 14. Design Water Quality and Recommended Goals for IM Process

Parameter	Units	Design Water Quality	Treatment Goals
Iron	mg/L	3.5	<0.1
Manganese	mg/L	0.65	<0.02

5.3.1 IM Treatment Process Flow

The IM treatment system consists of an oxidation system that provides continuous chlorination upstream of the pressurized media filters, as well as an equalization tank, backwashing, waste wash water recovery, sludge handling, and chemical feed and storage. The process flow diagram is shown in Figure 5 and an overview of equipment capacity and purpose is summarized in Table 13.

5.3.2 Equalization Tank

The EQ tank will be designed based on the maximum flow of 3.6 MGD feeding into RMWD treatment plant and a recycle flow of 0.5 MGD of supernatant. The tank will provide about 17 minutes of storage at 4.1 MGD. The storage provides contact time between the chlorine and raw water prior to the IM media and retains water in case of controlled startups and plant shutoffs. Table 15 shows the design criteria for the EQ tank.

Table 15. EQ tank design criteria

Parameter	Units	Value
Storage Capacity[1]	gal	50,000
Height	ft	8
Diameter	ft	33

[1]Design based on maximum flow rate of 3.6 MGD and 0.5 MGD of supernatant.

5.3.3 Preoxidation System

All of the common Fe and Mn removal processes utilize the oxidation process upstream of filtration. Oxidation changes the dissolved iron and manganese present in groundwater supplies to insoluble forms that are removed by filter media. Ferrous iron, Fe(II), is oxidized to ferric iron, Fe(III), which readily forms the insoluble iron hydroxide. Similarly, reduced manganese, Mn(II), is oxidized to Mn(IV), which forms insoluble manganese oxide. The most commonly used oxidants are chlorine, potassium permanganate, ozone, chlorine dioxide, oxygen, and hydrogen peroxide. The oxidant of choice for the RMWD IM treatment is chlorine because stronger oxidants form colloidal manganese particles that are harder to remove through filtration. The oxidizing strength trend is:

$$O_3 > H_2O_2 > ClO_2 > KMnO_4 > Cl_2 > O_2$$

In the case of permanganate, dissolved Mn is actually produced from the quenching of residual permanganate, which can lead to RO fouling downstream. Sodium hypochlorite (NaOCl) is the form of free chlorine that will be dosed in the raw water upstream of the filters and EQ tank. A residual of 0.5 mg/L Cl_2 will be carried through the filters, such that 0.5 mg/L is present at the outlet of the filters.

Oxidation of ferrous iron with chlorine is effective and occurs rapidly, whereas the reaction between Mn(II) and free chlorine is more difficult and slower. To remove manganese, a manganese oxide coated surface is commonly employed as a filter media. This approach is called greensand filtration named after the media that is used. First, the dissolved manganese adsorbs to the oxide-coated filter media, which is gradually oxidized to manganese dioxide on the surface by the chlorine residual passing through the filter. The precipitant acts as a new surface for further adsorption of reduced Mn.

Based on corresponding redox reactions, the required free chlorine dose for the oxidation of iron is 0.64 mg Cl_2/mg Fe^{2+} and 1.30 mg Cl_2/mg Mn^{2+} for the oxidation of manganese. The presence of ammonia in the raw water will reduce the rate of oxidation for both iron and manganese, since it consumes chlorine to form chloramines. Therefore, ammonia has to be considered when determining the necessary dose of sodium hypochlorite. Table 16 shows the chlorine demand of the bulk water. Note that the free chlorine demand factors are higher than would be expected based on stoichiometry as a result of applying a safety factor.

Table 16. Free Cl_2 requirements for oxidation of iron, manganese, and ammonia

Parameter	Raw Water Quality (mg/L)	Free Cl_2 Demand Factor	Chlorine Demand (mg/L)
Fe	3.5	1	3.5
Mn	0.65	2	1.3

NH3-N	0.6	8	4.8

In groundwater supplies where reduced iron forms strong complexes with natural organic matter (NOM), free chlorine is unable to oxidize the iron. Fortunately, chlorine doses as 5 mg/L have been used effectively in the oxidation of organically bound iron.

A common storage facility will be used to store all the sodium hypochlorite solution utilized in the oxidation of the raw water, disinfection of the RO treated water, regeneration of the IM vessels, and removal of sodium bisulfite from the RO feed tank. During storage, sodium hypochlorite degrades to salt and oxygen. The rate of degradation is a function of solution concentration, temperature, light, pH, and concentration of heavy metals. The faster the oxidant is used, the less time it will have to degrade, so storage time should be limited. For this project, the sodium hypochlorite will be stored for 15 days. The design criteria for the oxidant dose and sodium hypochlorite storage tank are shown in Table 17.

5.3.4 Pressure Vessels

Filtration will be employed downstream of preoxidation with free chlorine. The most common IM treatment technology is greensand filtration. For this technology, an oxidant (free chlorine) residual is carried through dual media filters (see below for discussion of the media). For this project, horizontal pressure vessels will be used. Advantages of pressure filters are their ability to operate at higher terminal headloss, which translates to longer filter runs and reduced backwash requirements. Table 18 summarizes the design parameters of the pressure vessels obtained from the IM original equipment manufacturer (OEM) Hungerford & Terry.

The design of the vessels was based on the maximum flow of 3.6 MGD, with three filters operating at all times. In addition, the working pressure is recommended to have filters with robust shells and to protect against negative pressures.

Table 17. Design Criteria of Oxidant and Sodium Hypochlorite Storage

Parameter	Units	Value
Oxidant	--	Sodium Hypochlorite
Typical Dose[1]	mg/L	10
Oxidant feed pump capacity[2]	gph	10
Oxidant feed pump turndown[2]	--	10:1
Sodium hypochlorite strength	%	12.5
Sodium hypochlorite storage volume[3]	gal	5,000
Height (ft)	ft	13
Diameter (ft)	ft	8.5

[1] Pilot testing would provide confirmation of oxidant dosage and choice of oxidant.

[2] Based on the 10:1 ratio, pump can feed as low as 1 gph to 10 gph.

[3] Provides 15 days of storage.

5.3.5 Filter Media

Manganese greensand is a common filter media used in the removal of iron and manganese through pressure filtration. Greensand is a processed material consisting of glauconite that is coated with manganese oxide. This surface promotes the removal of dissolved manganese through the reactions that occur between the adsorbed reduced manganese and the oxidized manganese oxide coating. Supply of greensand became difficult for Hungerford and Terry (then Inversand) in recent years. At the same time, a superior replacement product was introduced that is an exact match for existing greensand system. Numerous systems across the country have switched to GreensandPlus™ in recent years (e.g. the City of San Juan Capistrano). Greensand Plus is a processed material consisting of a special density silica sand coated with manganese oxide. Unlike greensand, which often required potassium permanganate as the preoxidant, GreensandPlus works well with free chlorine as the oxidant, which offers several advantages as discussed above. It also does not require continuous regeneration. For this project, the GreensandPlus media supplied by Hungerford and Terry will be installed in the pressure vessels. Table 19 shows the design criteria for the anthracite cap, GreensandPlus, and gravel support. The purpose of the anthracite cap is to remove precipitated iron present due to reactions with the preoxidant upstream of the greensand media and prevent blockage of the manganese oxide sites.

Table 18. Design Criteria of Pressure Vessels[a]

Parameter	Units	Value
Number of vessels (n+1)	#	3
Pressure vessel configuration	--	Horizontal
Cells per vessel	#	2
Vessel diameter	ft	10
Vessel length	ft	32
Active surface area[1]	ft^2	320
Working Pressure	psi	100
Underdrain configuration	--	Common underdrain
Filtration rate[2]	gpm/ ft^2	2.6
Filter flow rate	gpm	832
Backwashing filtration rate[3]	gpm/ ft^2	3.9
Backwashing filter flow rate	gpm	1248
Filter run length	hr	18

[a]It should be noted that flow control is needed on the pressure vessels to assure the same flow is delivered to each vessel

[1] Active surface area is the average filter surface area of GreensandPlus media over the depth of the media.

[2] All filters online

[3] One filter backwashing or one filter offline.

5.3.6 Quenching System

A quenching system is necessary upstream of the reverse osmosis treatment to remove the free chlorine residual from the IM treated water to prevent damage to the RO membranes. The IM treated water will be dechlorinated with sodium bisulfite and fed to an RO feed tank, which provides contact time between the chemical and the treated water.

The stoichiometric ratio of 1.5 mg $NaHSO_3$/mg Cl_2 was utilized to determine the amount of sodium bisulfite necessary to quench the free chlorine residual in the treated water. When dechlorination is applied upstream of an RO process, a safety factor of 2 is applied to the ratio.

The design criteria for the quenching system are summarized in Table 20.

Table 19. GreensandPlus Filter Media Design Criteria

Parameter	Value	Units
Anthracite (top layer)		
Depth	inch	12
Effective size (d_{10})	mm	0.6 - 0.8
Specific gravity	--	1.6
GreensandPlus (middle layer)		
Depth	inch	24
Effective size (d_{10})	mm	0.30 - 0.35
Specific gravity	--	2.4
Gravel Support (bottom layer)		
Depth	inch	16

[a]Other important design considerations for SBS feed are installation of a rapid mixer downstream of SBS addition and a free chlorine analyzer at the outlet of IM treatment to control SBS dosing, and free chlorine analyzer and ORP analyzer downstream of SBS dosing to confirm quenching.

Table 20. Quenching System Design Criteria

Parameter	Units	Value
Quenching Chemical	--	Sodium bisulfite
Typical Dose[1]	mg/L	1.5[4]
Quench feed pump capacity[2]	gph	0.875
Quench feed pump turndown	--	10:1
Sodium bisulfite storage volume[3]	gal	200
Sodium bisulfite strength	%	38

[1] Designed for 3.0 MGD of treated water containing 0.5 mg/L of free chlorine.

[2] Pump can feed as low as 0.0875 gph.

[3] Based on 15 days storage.

[4] Includes a safety factor of 2 to protect the RO membranes

5.3.7 RO Feed Tank

The RO feed tank contains water that has been dechlorinated upstream to avoid the possibility of oxidizing the RO membranes. The RO feed tank will be designed to continuously feed the RO system at 3.0 MGD during the span of the backwash and to have enough water to perform a complete air/water wash and backwash for a whole vessel. Before backwashing a vessel, the stored water will be chlorinated to remove residual sodium bisulfite. The elements of the backwashing system are discussed below. The design criteria for the RO feed tank are shown in Table 21.

Table 21. RO Feed Tank Design Criteria

Parameter	Units	Value
Storage Capacity	gal	125,000
Height (ft)	ft	24
Diameter (ft)	ft	30
BW chlorination chemical	--	Sodium Hypochlorite
Typical dose[1]	mg/L	1.5
Chlorine BW pump capacity	gph	0.875
Chlorine BW pump turndown	--	10:1
Chemical storage volume	gal	Same as preoxidant[2]
Chemical strength	%	Same as preoxidant[2]

[1] Designed to dose 160,000 gal of backwash water in a day that includes 4 backwashes and 1 air/water wash.

[2] See Table 17

5.3.8 Backwashing System

The backwashing system consists of daily backwash sequences and weekly air scouring (air/water wash). The air scouring system increases the efficiency of backwashes and is recommended to prevent the formation of manganese mudballs. The presence of SBS is detrimental to the IM filters. Therefore, backwash water will be supplied from the RO feed tank, which will be chlorinated to remove any residual sodium bisulfite present due to the free chlorine quenching system upstream of the RO feed tank.

The volume of water required for every backwash will be sized to backwash a single filter, one cell at a time. While one cell is being backwashed, the other cell will not be able to filter. The IM filters will operate in a staggered manner, meaning that instead of performing consecutive backwashes once the filter run time is reached, only one of the three operating filters will be backwashed. During this time, the filtration rate will increase from 2.6 to 3.9 gpm/ft^2 to compensate for the offline (backwashing) filter.

The staggering approach will require less water demand during backwashes, which translates to a smaller WWW tank footprint. A filter will have a run time of about 18 hours, meaning that approximately four backwashes will occur in a period of 24 hours, each backwash consisting of 38,400 gallons.

On a day when a filter requires its weekly air scouring, the total water volume required will be approximately 44,800 gallons, which consists of 6,400 gal of air/water wash (AWW) and 38,400 gal of backwash. This is assuming that no more than one air/water wash can occur in a day. After a backwash, the wash water will drain to the WWW recovery tank. The design criteria for a backwash, in conjunction with air scouring, are presented in Table 22.

Table 22. IM Vessel Backwash System Design Criteria

Parameter	Value	Units
Backwash		
Backwash rate per cell	gpm/ ft^2	12
BW pump capacity[1]	gpm	1,920
BW run time[2]	min	20
Backwash volume	gal	38,400
Air/water wash		
Air wash rate	cfm/ ft^2	2
AWW rate per cell	gpm/ ft^2	4
AWW pump capacity[1]	gpm	640
AWW run time[2]	min	10
AWW volume	gal	6,400

[1] Pump capacity for each cell.

[2] Run time per vessel.

5.3.9 Regeneration System

In IM treatment, the greensand's oxide coating is continuously regenerated. A residual will pass through the vessels during filtration and backwashing. For safety reasons, a regeneration facility is recommended if the GreensandPlus media is ever exhausted, which is not anticipated. The regeneration process involves draining the pressure vessel, filling the vessel with chlorinated water from the RO feed tank, soaking the media for at least four hours, draining the waste to a regeneration tank, and rinsing the media before startup.

During the regeneration, an unusually high sodium hypochlorite dose of approximately 1,000 mg/L will be injected into the backwash line. The chlorine will be fed from the same storage tank as the preoxidant. An equalization tank is needed to slowly discharge the regeneration waste to a sewer.

Table 23. Design Criteria for Regeneration System

Parameter	Units	Value
Regeneration chemical	--	Sodium Hypochlorite
Typical Dose	mg/L	1,000
Chemical pump capacity	gph	600
Chemical feed pump turndown	na	10:1
Chemical storage volume[2]	gal	Same as preoxidant[1]
Chemical strength	%	Same as preoxidant[1]
Regeneration EQ tank volume	gal	12,500

[1] See Table 17

5.3.10 Waste Wash Water Recovery Tank

The WWW recovery tank will be designed to account for the total volume of water used in one backwash. This will include the amount of water used during an air/water wash and backwash sequence, as well as the unfiltered water required to rinse a filter. The total capacity of the WWW recovery tank will include a depth of 2 feet for sludge accumulation.

After an assumed settling time of 3.5 hours, 98.9% of the water in the tank will be recycled to the front of the EQ tank, while the remaining 1.1% of volume will be settled as sludge. In similar waters that are low in iron but high in manganese, addition of ferric chloride is required to help the slow settling. The water from the SLR river basin contains a high content of iron, so ferric chloride is not required in the treatment process. The WWW recovery tank design criteria are shown in Table 24.

Table 24. WWW Recovery Tank Design Criteria

Parameter	Units	Value
Storage Capacity[1]	gal	80,000
Height	ft	18
Diameter	ft	28
Supernatant volume	gal	48,016
Recycle time	hr	2.1
Reclaim pump	gpm	375
Settled sludge volume	gal	534
Sludge removal time	hr	0.2
Sludge removal pump	gpm	50

[1] Allows for 3 hours of settling time, 2.1 hours of recycling time, and 54 minutes of backwashing and air water wash, based on a backwash interval of 6 hours (based on filter run times of 18 hours).

5.3.11 Drying Beds

The sludge volume will be pumped to drying beds, a natural dewatering process that treats the sludge through natural evaporation (Kawamura, 2000 and McGivney and Kawamura, 2008). This process requires a large footprint and the operation depends on climatic conditions. Spreading the sludge from the WWW recovery tank on drying beds will produce a dry, solid sludge that can either be disposed of or reused.

A drying bed with a concrete bed bottom is recommended to allow mechanical equipment to clean the bed and to prevent groundwater pollution through percolation. The factor that determines the feasibility of dewatering at any given site is the size of the paved bed, since the process of draining is inhibited. As a result, the only way that water will be removed is through evaporation.

Each drying bed will consist of two channels and require cleaning every 17 months. The design criteria of the beds are shown in Table 25.

Table 25. Sludge Drying Bed Design Criteria

Parameter	Units	Value
Number of drying beds	#	4
Number of independent channels per drying bed	#	2
Drying bed length	ft	175
Channel width	ft	20
Drying bed depth	ft	1.5

5.4 RO TREATMENT SYSTEM

As discussed above, RO treatment is necessary to reduce the salinity of the groundwater and meet water quality goals for TDS and chloride. This design strategy will allow the treatment process to also meet water quality goals for other mineral constituents for which goals have been established (see Table 11). Not all of the water will require RO treatment to meet water quality goals. Therefore, a portion of the water treated by IM treatment will bypass the RO.

High RO recovery indicates a larger amount of permeate extracted from the feed water, therefore conserving the raw water resource and minimizing the amount of concentrate required for discharge. But RO recovery is often limited by the potential for fouling and scaling of the RO membranes and by the concentration of soluble salts in brine streams (concentrate). In this project, a design recovery of 80% is proposed based on the projection after addition of antiscalant.

Three identical trains have been designed to accommodate the potential for various groundwater well extraction flow rates. Low pressure polyamide RO membranes are recommended to achieve the balance of energy savings and treated product water volume. The blended product water quality has been evaluated. All water quality parameters achieve the water quality goals discussed in Section 4. The details of the RO conceptual design are shown below.

5.4.1 RO Bypass

The IM treated water will be split into two streams to achieve maximum production while meeting the design water quality goals. A portion of the IM treated water will be pumped into a reverse osmosis treatment system to remove any soluble salt. The treated water (RO permeate) will be blended with another portion of the IM treated water not treated by RO (RO bypass). For this project, RO bypass was set up as 15 percent to achieve 80 percent of the water quality goal in Table 11. The RO treatment flow paradigm is shown in Table 26.

Table 26. RO Flow Design Criteria

Parameter	Flowrate (mgd)
RO Feed	3.14
RO Bypass	0.46
RO Recovery	80%
RO Permeate	2.51
RO Concentrate	0.63
Product Water	2.97

5.4.2 RO Feed Water Pretreatment

As the water flows through the RO membranes, dissolved constituents are left behind in a decreasing volume of water that is concentrated. There are certain constituents when present in the groundwater that create conditions for fouling and/or scaling of RO membranes. These constituents include divalent ions like sulfate, calcium, and magnesium in addition to less common constituents like strontium and barium. Another such constituent is silica, present in the groundwater at levels near 30 mg/L. IM treatment will not have an impact on the silica concentration, it requires the addition of acid and inhibitors to control the fouling and scaling problem.

Typically, the addition of sulfuric acid will keep Langelier Saturation Index (LSI) in the concentrate to levels below 1.8, the level proven by practical experience to prevent problems with calcium carbonate precipitation. Silica is another inhibitor that will limit recovery. Initial process modeling of RO treatment showed that with 74 percent recovery of permeate, the silica concentration in the concentrate would be near saturation. This consideration will factor into the choice of antiscalant to be added to the process. With proper sulfuric acid and antiscalant addition, recovery of 80 percent can be achieved based on the raw water quality. The detailed projection from the antiscalant vendor after antiscalant addition is included in Appendix 5.

5.4.3 RO System Design

It is understood that the groundwater extraction wells will be operated at various extraction capacities. To accommodate the potential capacity range, it is necessary to configure the system with multiple, identical, independent RO membrane trains. In this project, three identical trains will be beneficial as it allows the maintenance of two thirds of the RO system treatment capacity with any single train out of service. A 90 percent online factor is considered in the RO capacity design.

Table 27. Acid and Antiscalant Dosing Design Criteria

Parameter	Units	Value
Acid	--	Sulfuric Acid
Estimated Dose	mg/L	36
Sulfuric Acid Storage Volume	gal	1,000
Sulfuric Acid Strength	%	100
Antiscalant	--	eg. Vitec 1400
Estimated Dose	mg/L	1.8
Antiscalant Storage Volume	gal	300
Antiscalant Strength	%	100

For RO element selection, a high rejection, a low- pressure membrane is recommended. Higher rejection by the RO membrane results in lower chloride levels. This allows for reduced RO sizing and system cost. Low pressure operation uses less energy.

Another important design parameter for the RO system is flux. Flux is the effective loading rate on the membranes expressed as gpd/ft^2 or gfd. The advantage of higher flux design is better RO permeate water quality and smaller RO membrane area. This comes with a tradeoff of higher energy consumption and higher fouling potential. For this project, a design flux of 12 gfd is recommended to achieve the balance of energy savings and capital cost.

To achieve the target flux in the system, multiple elements must be combined in pressure vessels. The elements are loaded in series. For this project, seven elements per pressure vessel were used. The recovery that can be achieved by a seven-element vessel is between 40 – 60 percent. Higher recoveries will require staging, which sends the brine from the first seven-element vessel to a second group of pressure vessels. For the 80 percent recovery, a 2-stage system is recommended. In addition to the membrane elements and pressure vessels, each train will be equipped with a feed pump that provides the driving pressure for the process and a concentrate control valve that controls the brine flow leaving the train and thus the recovery. The RO membrane train design criteria are provided in Table 28.

Table 28. RO Treatment System Design Criteria

Parameter	Unit	Value
Trains	--	3
Capacity per train	mgd	1.05
Online factor	%	90
Stages	--	2
Elements per vessel	--	7
No. of Vessels for Stage 1[1]	--	16
No. of Vessels for Stage 2[1]	--	8
Target flux	gfd	12

[1] Based on RO treatment projection using Toray model

5.4.4 Membrane Process Selection

There are different types of membranes that were considered and analyzed for this project. They are nanofiltration (NF) membranes, low- pressure RO membranes, and brackish water RO membranes. The advantages and disadvantages for these three types of membrane are listed in Table 29.

NF membranes have the lowest salt rejection because they do not reject monovalent ions very well. Given the high TDS concentration in the raw water and the low chloride water quality goal in product water, NF was ruled out.

For RO membrane elements, the standard diameter and length of spiral wound membrane elements in municipal systems are 8 inches and 40 inches, respectively. Widely installed membrane elements have 400 square feet of membrane area. Therefore, a standard 8 in diameter and 40 inch length 400 square feet membrane area element was proposed.

Six different RO elements from two major manufacturers were considered. The membrane manufacturers were Hydranautics and Toray. Their RO membrane performance was evaluated using membrane projection software, IMSDesign by Hydranautics and Toray DS2 by Toray. Several RO treatment projections were carried out with the membrane manufacturers's models. The full projection summary information is listed in Table 30.

Table 29. Characterization of Three Types of Membrane Considered for the Project

Membrane Type	Salt Rejection	Advantages	Disadvantages
Nanofiltration Membrane	91%-97%	Low feed pressure, lower post treatment requirement	High feed water pretreatment required, lower bypass rate, lower recovery
Low Energy RO Membrane	99.2%-99.6%	Moderate recovery, relatively low feed pressure	Lower recovery than high rejection RO membrane
High Rejection RO Membrane	99.7%	Highest recovery due to highest bypass rate	High feed pressure, require more intense post-treatment

The selected criteria for evaluation of the RO elements were the number of membrane elements per train, the allowable bypass flow, and the feed pressure. Based on the projection results in Table 30, ESPA4-LD was excluded from selection due to its low salt rejection and resultant decrease in the amount of bypass flow. When the maximum bypass volume was set for compliance with the TDS sMCL, TMH2-A-400C and ESPA2-LD were unable to comply with the chloride water quality goal. Therefore, they were excluded from selection. The remaining elements, TMG 20-400, TM720D-400, and CPA5-LD, had similar maximum RO bypass flow and resultant chloride concentration. The remaining RO elements can be differentiated by feed pressure, with higher feed pressure meaning higher energy use. TMG20-400 required substantially lower feed pressure compared to TM720D-400 and CPA5-LD. Therefore, TMG 20-400 was selected among the six options for application in the conceptual design. The RO modeling results for TMG20-400 is presented in Appendix 6.

Table 30. Overall Projection of Different RO Elements

Element	Total elements required	Average flux (gfd)	Feed pressure (psi)	Max RO Bypass by TDS (percent)	Chloride Conc. at Max. Bypass (mg/L)
TMH2-A-400C	175	11.9	93.8	15	85.7
TMG20-400	**175**	**11.9**	**108**	**17**	**85.0**
TM720D-400	175	11.9	161	19	84.1
ESPA4-LD	168	11.6	107	9.0	76.0
ESPA2-LD	168	11.6	156	17	86.1
CPA5-LD	168	11.6	206	18	85.0

5.4.5 Clean-In-Place System

Periodic cleaning of the RO membrane is required to maintain membrane salt rejection and flux. A clean-in-place cleaning system is typical, wherein the membranes are cleaned within the pressure vessels by circulating cleaning solutions through them. For mineral scales, low pH, cleaners such as citric acid at a 2 percent concentration are typically employed. For silica, custom cleaning agents are available from vendors in both low and high pH forms.

5.5 DISINFECTION AND STABILIZATION

The treatment plant product water will require disinfection and stabilization prior to distribution. A clearwell was designed to provide detention time and baffling to achieve the disinfection contact time (CT) requirement. The clearwell size is 35,000 gallons.

The disinfection system is divided into two major categories: primary disinfection for virus removal and secondary disinfection to maintain a residual in the distribution system. Primary disinfection of groundwater often is conducted in a clearwell with free chlorine. The contact time in the clearwell and the chlorine residual at the outlet of the clearwell need to be calculated to achieve 4 logs of virus inactivation, according to the Environmental Protection Agency (EPA) CT tables. Sodium hypochlorite will be drawn from the same sodium hypochlorite storage system that is used for preoxidation upstream of the IM vessels, backwash chlorination and regeneration. Following the sodium hypochlorite dosing, rapid mixing is required to ensure adequate disinfection. For this project, disinfection design criteria are listed in Table 31.

For groundwater, free chlorine alone is often used for disinfection due to the low TOC in typical groundwater. For this project, chloramines were selected for secondary disinfection. This was due to elevated TOC levels observed in the limited amount of sampling conducted for groundwater in the study area that could lead to THM formation. It was also selected to match the approach used by MWD, which provides imported water to RMWD at present. Chloramine often is used for secondary disinfection due to its stability in the distribution pipeline. Another benefit is fewer THMs and HAAs are formed with chloramines. Chloramines can be formed by the addition of aqueous ammonia (ammonium hydroxide) to the outlet of the clearwell, which will react with the residual free chlorine to form chloramines.

It is recommended that a chlorine analyzer be used for both the clearwell feed and outlet. The chlorine analyzer for the clearwell feed allows for determination of chlorine decay through the clearwell. The clearwell outlet chlorine analyzer is used for controlling the chlorine and ammonia dose and for CT compliance reporting.

Stabilization is required to minimize corrosion of the distribution piping and household plumbing, given the corrosivity of the product water, which is a blend of RO permeate and groundwater high in hardness, alkalinity, and minerals. Stabilization could be achieved with caustic soda and/or orthophosphate addition. A degasifier can be considered if needed to reduce caustic requirements. The caustic dose can be controlled by downstream pH measurement, with rapid mixing in between the dose point and the analyzer sample location to achieve a low coefficient of variation (e.g., 5 percent). If orthophosphate addition is needed, it will be flow paced, with the dose set by the operator.

Standard practice for minimizing corrosion of cement mortar lined steel pipelines is to maintain calcium carbonate saturation. This can be accomplished by maintaining a slightly positive LSI. Thus, an LSI goal of 0.1 to 0.5 is recommended for stabilization of the product water. At the same time, calcium carbonate precipitation potential is often used (CCPP) and a CCPP in the range of 4-10 mg/L was targeted in line with industry practice.

For this project, caustic soda was designed for stabilization. The addition of caustic soda raises the pH and increases LSI. Numerous water utilities with waters of similar quality use pH adjustment alone to meet corrosion control water quality goals. The RO permeate water quality used in the analysis is based on the TMG20-400 modeling result, which was discussed in section 5.3.4. The design criteria for the stabilization system are shown in Table 32. The water quality before and after caustic soda addition is presented in Table 38.

5.6 PUMPS AND TANKS

Design criteria for pumps and tanks are presented. This includes general pumps and tanks. It also includes chemical feed pumps and storage tanks.

5.6.1 General Pumps and Tanks

The design criteria and materials of construction recommended for pumps discussed above are summarized in Table 33. This includes the raw groundwater freed pump, pumps associated with IM treatment, and the treated water distribution pumps. The design criteria for tanks discussed above are presented in Table 34.

5.6.2 Chemical Feed Pumps and Chemical Storage Tanks

The properties of chemicals used in the treatment plant conceptual design are listed in Table 35. To design the chemical storage, 15 days of storage time was assumed. In addition to chemical consumption and chemical storage, chemical feed rate was evaluated. The design criteria for chemical feed pump sizing are summarized in Table 36. The parameters that serve of as the basis of sizing the chemical storage tanks are summarized in Table 37 along with the tank sizes. The chemical dosing shown in Table 37 is broken down by stage in the treatment train. For this reason, certain chemicals are fed at different doses through out the treatment train and appear multiple times in Table 37 (e.g., NaOCl). For this reason, the tank sizes shown in the last column of Table 37 may look larger than what is required for an individual location, as they are based on the sum total of the chemical storage volume required for all dosing locations.

Table 31. Disinfection Design Criteria

Parameter	Units	Value
Temperature	°C	>= 15
pH	--	6-9
T_{10}/T baffling efficiency	--	0.3
Safety factor	--	0.2
Residual free chlorine	mg/L	1.0
Flowrate	mgd	2.99
Disinfectant Chemical	--	Sodium Hypochlorite
Estimated Typical Dose, disinfectant	mg/L	2.5
Assumed Disinfectant Strength	%	12.5
Disinfectant Storage Volume	gal/year	Same as preoxidant feed
Clearwell Volume for CT, minimum	gal	35,000
Ammonia	--	Aqua Ammonia
Estimated Typical Dose, ammonia	mg/L	0.6
Assumed Ammonia Strength	%	19
Ammonia Storage Volume	gal	250
Note: Chlorine decay varies with flowrate. Therefore, chlorine dosing and ammonia dosing will be controlled using the effluent chlorine analyzer.		

Table 32. Stabilization System Chemical Dosing Design Criteria

Parameter	Units	Value
Alkali chemical	--	Sodium Hydroxide
Estimated Dose	mg/L	45
Sodium Hydroxide Storage Volume	gal	6,000
Sodium Hydroxide Strength	%	25

Table 33. Sizing for Pumps

	Total Flow (gpm)	Pressure (psi)	Qty	Flow/Pump (gpm)	Power (Hp)
Plant Raw Groundwater Feed Pump					
Feed Pump	2500	30	2	1250	26
Feed Pump VFD			2		26
Pumps for IM Treatment					
IM Backwash Pump	1920	30	2	960	20
IM Backwash Pump VFD			2		20
IM Regeneration Pump	640	30	1	640	13
IM Regeneration Pump VFD			1		13
WWW Recycle Pump	Included in IM OEM				
WWW Recycle Pump VFD			2		4
WWW Sludge Pump	230	30	1	230	5
WWW Sludge Pump VFD			1		5
Treated Water Distribution Pump					
Distribution Pump	2100	120	4	525	44
Distribution Pump VFD			4		44

Table 34. Summary of Sizing and Materials for Tanks

Tanks	Capacity (gallon)	Quantity	Material
Tanks for IM Treatment			
EQ Tank	50,000	1	Bolted Steel
RO Feed Tank	125,000	1	Bolted Steel
WWW Storage Tank	80,000	1	Bolted Steel
Regeneration Tank	12,500	1	HDPE

Table 35. Properties of Chemicals Used in the Design

Chemical Data	NaOCl	NH_4OH	$NaHSO_3$	H_2SO_4	NaOH
Name	Sodium Hypochlorite	Aqua Ammonia	Sodium Bisulfite	Sulfuric Acid	Sodium Hydroxide
Concentration[1]	12.5%	19%	40%	98%	25%
Unit Weight, lb/gal	10.1	7.76	10.8	15.3	12.8
Basis for Dosage	as Cl_2	as NH_3-N	as $NaHSO_3$	as H_2SO_4	as NaOH
Unit Price, $/gal	1.10	1.05	2.35	2.39	0.66
Unit Price, $/lb	0.87	0.71	0.55	0.160	0.21

[1]Dilution of sodium hypochlorite stock solution to avoid problems with chlorate formation, especially at elevated temperatures.

Table 36. Summary of Sizing of Chemical Feed Pumps

Chemical	Flow (mgd)	Dose (mg/L)	Feed (gph)	Pressure (psi)	Quantity
IM preoxidation					
NaOCl	3.6	10.0	9.9	50	1
IM Quenching					
$NaHSO_3$	3.14	1.5	0.39	50	1
IM Backwash					
NaOCl	2.8	1.5	1.14	50	1
IM regeneration					
NaOCl	0.9	1000	248	50	1
RO pretreatment					
Antiscalant	3.14	1.83	0.7	--	--
H_2SO_4	3.14	18.96	2.7	--	--
RO post treatment					
NaOH	2.97	45	14.5	50	1
Disinfection					
NaOCl	2.97	2.5	2.0	50	1
NH_4OH	2.97	0.6	0.4	--	--

Table 37. Summary of Sizing for Chemical Storage Tanks

Chemical	Flow (mgd)	Dose (mg/L)	Chemical Required (gpd)	Days of Storage	Storage Volume Required (gal)	Tank Size (gal)	Tank Material
IM Preoxidation							
NaOCl	3.6	10.0	238	15	3570	5000	HDPE
IM Quenching System							
$NaHSO_3$	3.14	1.5	9.5	15	143	200	HDPE
IM Backwashing System							
NaOCl	2.8	1.5	1.5	15	23	5000	Same
IM Regeneration System							
NaOCl	0.9	1000	5.5	15	83	5000	Same
RO pretreatment							
Antiscalant	3.14	1.8	15	15	223	275	Totes
H_2SO_4	3.14	36	33	15	957	1000	Mild Steel
Disinfection and Stabilization							
NaOH	2.97	45	347	15	5207	6250	HDPE
NaOCl	2.97	2.5	48	15	715	5000	Same as above
NH_4OH	2.97	0.6	13	15	202	250	Carbon Steel

5.7 TREATED WATER QUALITY

The product water quality after stabilization represents the treated water quality from the IM/RO groundwater treatment plant. The finished water quality from the clearwell for those constituents removed by RO is summarized in the last column of Table 38 below. As discussed iron, manganese, and TDS also meet their water quality goals. Therefore, the treated water quality meets all water quality goals.

Table 38. Water Quality before and after Stabilization

Parameters	Units	Before Stabilization			Treated Water Quality After Stabilization
		RO Bypass	RO permeate	Blend	
Calcium	mg/L	201	4.0	34	34
Magnesium	mg/L	81	1.6	14	14
Sodium	mg/L	220	12	44	70
Potassium	mg/L	8.8	0.73	1.9	1.9
Ammonia-N	mg/L	0.3	0.02	0.07	0.07
Chloride	mg/L	434	15	77	77
Sulfate	mg/L	490	9.3	81	81
Nitrate as NO_3	mg/L	0.55	0.06	0.13	0.13
Fluoride	mg/L	0.4	0.04	0.08	0.08
pH	pH units	7.4	5.55	6.24	8.20
LSI	pH units	0.49	-4.14	-1.97	0.35

5.8 SIMPLIFIED CONCEPTUAL LAYOUT

A simplified conceptual layout is presented in Figure 6. The footprint was evaluated by considering the footprint of equipment associated with each individual treatment process. The estimated footprint for each piece of equipment in square feet is listed in Table 39.

Table 39. Estimated Required Footprint by Main Treatment Components

Treatment Components	Area Required, Estimated Net (ft^2)
EQ Tank	1,200
IM Vessels	2,000
Wash Water Recovery Tank	900
Regeneration Tank	200
RO Feed Tank	1,000
RO Treatment Skid	3,000
Clearwell	1,000
Solid Drying Bed	35,000
Chemical Storage Area	7,500
Total	***51,800 (1.2 acres)***

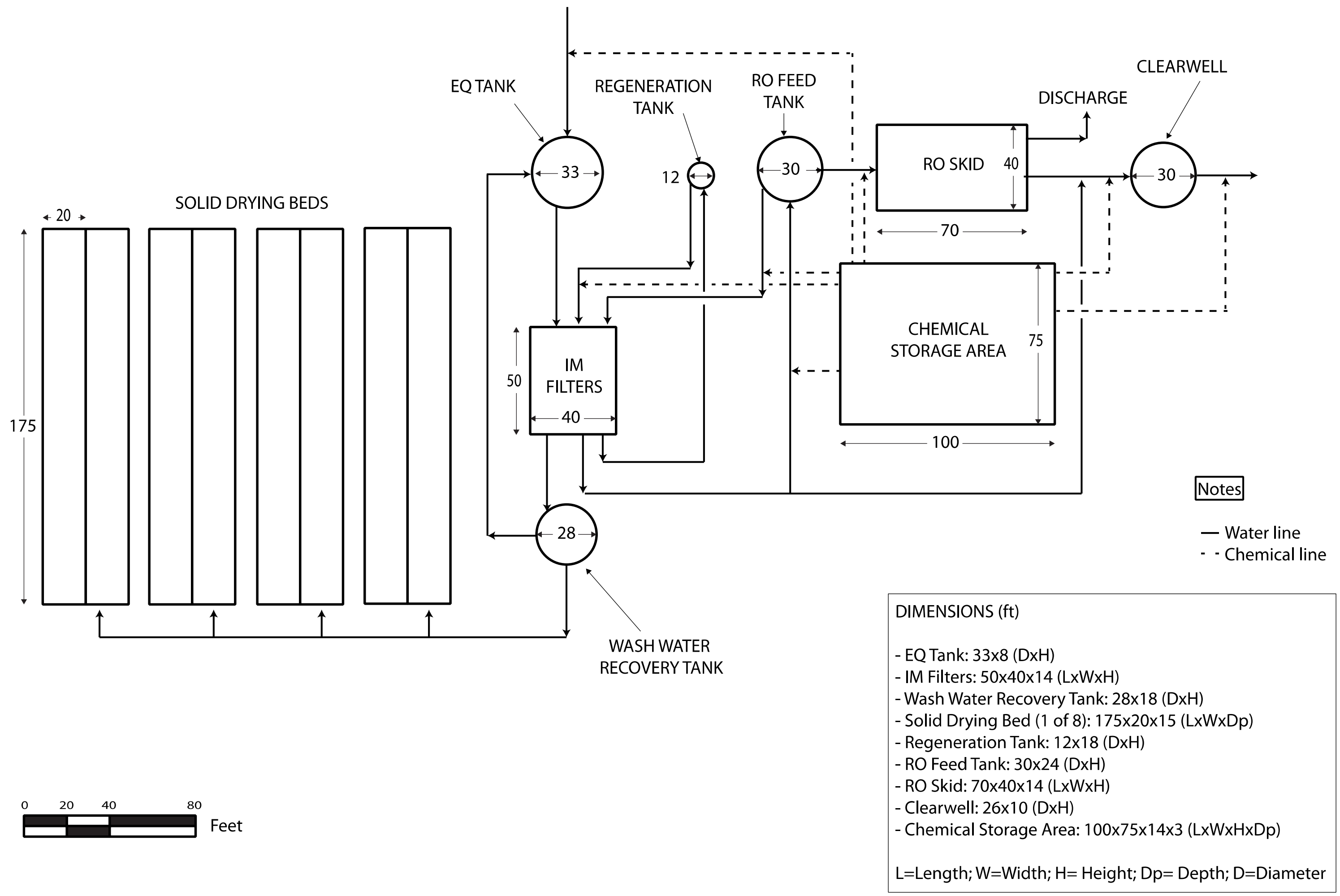

Figure 6. Simplified Conceptual Layout (does not include mechanical/electrical/civil, piping, or actual clearances)

5.9 CONCEPTUAL LEVEL CAPITAL AND O&M COSTS.

Conceptual level capital and O&M costs were estimated. An opinion of probable construction cost (OPCC) was developed based on the treatment train described in Section 5. An O&M cost was developed in consideration of what it takes to operate the treatment train. It is broken down according to the IM treatment system including both preoxidation, greensand filtration, and ancillary processes. Costs for RO treatment and associated items are presented. There are also prices included for pumps.

5.9.1 Conceptual Capital Cost

The OPCC for IM treatment, RO treatment, and post treatment (disinfection and stabilization) is in shown in Table 39. The preliminary equipment cost from Hungerford and Terrry (H&T) for the parts of IM treatment used in development of the OPCC is included in Appendix 7. The preliminary equipment cost for RO treatment from Biwater used in development of the OPCC is included in Appendix 8. The cost of additional items presented earlier in Section 5 and not included in the IM treatment and RO treatment quotes were determined based on cost estimating practice.

Table 40. OPCC for the SLR GW Treatment Plant

Treatment Process	OPCC
IM Treatment	$4,190,000
RO Treatment	$3,340,000
Post Treatment	$331,000
TOTAL	**$7,860,000**

5.10 Operation And Maintenance Cost

Chemical, power and maintenance cost were estimated for the treatment train. The detailed O&M estimation is listed in Table 41. The maintenance cost refers to maintenance of the equipment. Only those components listed in Table 41 were included in the O&M cost estimate. Chemical costs are annual costs estimated based the chemical feed design criteria provided above. Power costs were estimated based on pump size. Maintenance includes greensand media replacement, mechanical items associated with IM treatment, and RO membrane element replacement.

Table 41. O&M Cost Estimate for the Treatment System

Treatment Process	Annual Chemical Costs	Annual Power Costs	Maintenance Costs	Subtotal
IM Treatment	$99,000	$73,000	$41,000	$213,000
RO Treatment	$75,500	$314,000	$50,400	$440,204
Post Treatment	$108,000	$113,000	--	$220,436
Total Chemical and Power Costs	**$282,392**	**$500,000**	**$91,000**	**$874,000**

6 REFERENCES

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Trussell Technologies (2012). Process Evaluation and Recommendations for the Mission Basin Groundwater Purification Facility. Technical Memorandum prepared for the City of Oceanside, Nov., 2012.

CA Water Code, 2014. Section 79171, Article 2, Chapter 9, Division 26, of California Water Code.

7 APPENDIX

Appendix 1 – Analytical Methods for the Assessment of Groundwater Quality

Analyte	Units	MRL	Method	Analyte	Units	MRL	Method
Aluminum	µg/L	40	EPA 200.8	Mercury	µg/L	0.2	EPA 245.1
Antimony	µg/L	2	EPA 200.8	Total Organic Carbon	mg/L	0.3	SM5310C/E4 15.3
Arsenic	µg/L	2	EPA 200.8	Hydroxide as OH, Calculated	mg/L	2	SM2330B
Barium	µg/L	4	EPA 200.8	pH of CaCO3 Saturation(60°C)	Units	0.1	SM 2330B
Beryllium	µg/L	2	EPA 200.8	Carbon Dioxide, Free (25°C), Calculated	mg/L	2	SM4500-CO2-D
Cadmium	µg/L	1	EPA 200.8	Langelier Index, 25°C	None		SM 2330B
Chromium	µg/L	2	EPA 200.8	Carbonate as CO3, Calculated	mg/L	2	SM2330B
Copper	µg/L	4	EPA 200.8	Total Hardness as CaCO3, Calculated	mg/L	3	SM 2340B
Lead	µg/L	1	EPA 200.8	Anion Sum, Calculated	meq/L	0.001	SM 1030E
Manganese	µg/L	4	EPA 200.8	Cation Sum, Calculated	meq/L	0.001	SM 1030E
Nickel	µg/L	10	EPA 200.8	pH of CaCO3 Saturation (25°C)	Units	0.1	SM 2330B
Selenium	µg/L	10	EPA 200.8	Bicarb Alkalinity as HCO3, Calculated	mg/L	2	SM2330B
Silver	µg/L	0.5	EPA 200.8	Aggressiveness Index, Calculated	None	0.1	SM 2330
Thallium	µg/L	2	EPA 200.8	Cation/Anion Difference	%		SM 1030E
Vanadium	µg/L	6	EPA 200.8	Hexavalent Chromium, Dissolved	µg/L	0.02	EPA 218.6
Zinc	µg/L	40	EPA 200.8	Nitrate as Nitrogen	mg/L	0.13	EPA 300.0
Boron	mg/L	0.05	EPA 200.7	Nitrate as NO3, Calculated	mg/L	0.55	EPA 300.0
Calcium	mg/L	1	EPA 200.7	Nitrite Nitrogen	mg/L	0.13	EPA 300.0
Iron	mg/L	0.02	EPA 200.7	Total Nitrate, Nitrite-N, Calculated	mg/L	0.1	EPA 300.0
Magnesium	mg/L	0.1	EPA 200.7	Chloride	mg/L	10	EPA 300.0
Potassium	mg/L	1	EPA 200.7	Sulfate	mg/L	5	EPA 300.0

Silica	mg/L	0.43	EPA 200.7	Perchlorate	µg/L	2	EPA 331.0
Analyte	**Units**	**MRL**	**Method**	**Analyte**	**Units**	**MRL**	**Method**
Sodium	mg/L	1	EPA 200.7	Ammonia Nitrogen	mg/L	0.05	EPA 350.1
1,2,3-Trichloropropane	µg/L	0.005	CASRL 524M-TCP	pH	Units	0.1	SM4500-HB
Odor at 60°C (TON)	TON	1	SM 2150B	Surfactants	mg/L	0.05	SM 5540C/EPA 425.1
Fluoride	mg/L	0.05	SM 4500F-C	Turbidity	NTU	0.05	EPA 180.1
Alkalinity in $CaCO_3$ units	mg/L	2	SM 2320B	Specific Conductance, 25°C	µmho/cm	2	SM2510B
Total Dissolved Solids (TDS)	mg/L	10	E160.1/SM2540C				

Appendix 2 - Constituents with Federal and California DDW pMCLs

Contaminant	U.S. EPA MCL (mg/L)	California MCL (mg/L)
Inorganics		
Antimony	0.006	0.006
Arsenic	0.05 0.010	0.05 0.010
Asbestos	7 MFL[b]	7 MFL[b]
Barium	1 2	1
Beryllium	0.004	0.004
Cadmium	0.010 0.005	0.010 0.005
Chromium	0.05 0.1	0.05
Cyanide	0.2	0.2 0.15
Fluoride	4 2[a]	2
Hexavalent Chromium	-	0.010
Lead	0.05[d] 0.015[c]	0.05 [d] 0.015[c]
Mercury	0.002	0.002
Nickel	Remanded	0.1
Nitrate	(as N) 10	(as NO3) 45
Nitrite (as N)	1	1
Total Nitrate/Nitrite (as N)	10	10
Perchlorate	-	0.006
Selenium	0.01 0.05	0.01 0.05
Thallium	0.002	0.002
Radionuclides		
Uranium	30 ug/L	20 pCi/L 20 pCi/L
Combined Radium - 226+228	5 pCi/L	5 pCi/L 5 pCi/L
Gross Alpha particle activity (excluding radon & uranium)	15 pCi/L	15 pCi/L 15 pCi/L
Gross Beta particle activity	4 millirem/yr	50 pCi/L[e] 4 millirem/yr
Strontium-90	8 pCi/L	8 pCi/L[e] 8 pCi/L[e]
Contaminant	**U.S. EPA MCL (mg/L)**	**California MCL (mg/L)**
Tritium	20,000 pCi/L	20,000 pCi/L[e] 20,000 pCi/L[e]

VOCS		
Benzene	0.005	0.001
Carbon Tetrachloride	0.005	0.0005
1,2-Dichlorobenzene	0.6	0.6
1,4-Dichlorobenzene	0.075	0.005
1,1-Dichloroethane	-	0.005
1,2-Dichloroethane	0.005	0.0005
1,1-Dichloroethylene	0.007	0.006
cis-1,2-Dichloroethylene	0.07	0.006
trans-1,2-Dichloroethylene	0.1	0.01
Dichloromethane	0.005	0.005
1,3-Dichloropropene	-	0.0005
1,2-Dichloropropane	0.005	0.005
Ethylbenzene	0.7	0.68 0.7 0.3
Monochlorobenzene	0.1	0.03 0.07
Styrene	0.1	0.1
1,1,2,2-Tetrachloroethane	-	0.001
Tetrachloroethylene	0.005	0.005
Toluene	1	0.15
1,2,4 Trichlorobenzene	0.07	0.07 0.005
1,1,1-Trichloroethane	0.200	0.200
1,1,2-Trichloroethane	0.005	0.032 0.005
Trichloroethylene	0.005	0.005
Trichlorofluoromethane	-	0.15
1,1,2-Trichloro-1,2,2-Trifluoroethane	-	1.2
Vinyl chloride	0.002	0.0005
Xylenes	10	1.750
SOCS		
Alachlor	0.002	0.002
Atrazine	0.003	0.003 0.001
Bentazon	-	0.018
Benzo(a) Pyrene	0.0002	0.0002
Contaminant	**U.S. EPA MCL (mg/L)**	**California MCL (mg/L)**
Carbofuran	0.04	0.018

Chlordane	0.002	0.0001
Dalapon	0.2	0.2
Dibromochloropropane	0.0002	0.0001 0.0002
Di(2-ethylhexyl)adipate	0.4	0.4
Di(2-ethylhexyl)phthalate	0.006	0.004
2,4-D	0.1 0.07	0.1 0.07
Dinoseb	0.007	0.007
Diquat	0.02	0.02
Endothall	0.1	0.1
Endrin	0.0002 0.002	0.0002 0.002
Ethylene Dibromide	0.00005	0.00002 0.00005
Glyphosate	0.7	0.7
Heptachlor	0.0004	0.00001
Heptachlor Epoxide	0.0002	0.00001
Hexachlorobenzene	0.001	0.001
Hexachlorocyclopentadiene	0.05	0.05
Lindane	0.004 0.0002	0.004 0.0002
Methoxychlor	0.1 0.04	0.1 0.04 0.03
Molinate	-	0.02
Oxamyl	0.2	0.2 0.05
Pentachlorophenol	0.001	0.001
Picloram	0.5	0.5
Polychlorinated Biphenyls	0.0005	0.0005
Simazine	0.004	0.010 0.004
Toxaphene	0.005 0.003	0.005 0.003
2,3,7,8-TCDD (Dioxin)	$3x10^{-8}$	$3x10^{-8}$
2,4,5-TP (Silvex)	0.01 0.05	0.01 0.05
Disinfection Byproducts		
TTHM	0.100 0.080	0.100 0.080
HAA5	0.060	0.060
Contaminant	**U.S. EPA MCL (mg/L)**	**California MCL (mg/L)**

Bromate	0.010	0.010
Chlorite	1.0	1.0
Treatment Technique		
Acrylamide	TT[f]	TT[f]
Epichlorohydrin	TT[f]	TT[f]

a. Secondary MCL.
b. MFL = million fibers per liter, with fiber length > 10 microns.
c. Regulatory Action Level; if system exceeds, it must take certain actions such as additional monitoring, corrosion control studies and treatment, and for lead, a public education program; replaces MCL
d. The MCL for lead was rescinded with the adoption of the regulatory action level described in footnote c.
e. Gross beta MCL is 4 millirem/year annual dose equivalent to the total body or any internal organ; Sr-90 MCL = 4 millirem/year to bone marrow; tritium MCL = 4 millirem/year to total body
f. TT = treatment technique, because an MCL is not feasible.

Appendix 3 - Constituents with Fixed Consumer Acceptance Levels and Consumer Acceptance Level Ranges (sMCLs)

Parameters (units)	sMCL	
Aluminum (mg/L)	0.2	
Color (color units)	15	
Copper (mg/L)	1	
Foaming Agents (MBAS) (mg/L)	0.5	
Iron (mg/L)	0.3	
Manganese (mg/L)	0.05	
Methyl-tert-butyl ether (MTBE)(mg/L)	0.005	
Odor Threshold (order unit)	3	
Silver (mg/L)	0.1	
Thiobencarb (mg/L)	0.001	
Turbidity (NTU)	5	
Zinc (mg/L)	5.0	
Parameters (units)	**Recommended sMCL**	**Upper sMCL**
Total Dissolved Solids (mg/L)	500	1,000
Specific Conductance (µS/cm)	900	1,600
Chloride (mg/L)	250	500
Sulfate (mg/L)	250	500

Appendix 4 - Skinner WTP Treated Water Quality Summary from 2012 to 2015

Constituents	Units	Jan-12	Feb-12	Mar-12	Apr-12	May-12	Jun-12	Jul-12	Aug-12	Sep-12	Oct-12	Nov-12	Dec-12
Silica	mg/L	10.0	10.0	10.0	8.6	9.0	8.6	9.4	9.3	9.2	9.9	10.9	11.7
Calcium	mg/L	21	33	34	41	47	49	47	42	40	34	33	31
Magnesium	mg/L	12	14	14	17	18	19	19	17	16	15	15	14
Sodium	mg/L	53	58	61	66	76	76	75	68	67	65	63	63
Potassium	mg/L	2.5	3.2	3.3	3.6	3.7	3.7	3.7	3.5	3.6	3.4	3.3	3.4
Carbonate	mg/L	0	0	0	0	0	0	1	2	1	1	1	1
Bicarbonate	mg/L	98	105	105	112	122	126	124	115	116	110	106	105
Sulfate	mg/L	49	86	94	117	146	156	142	121	112	96	83	79
Chloride	mg/L	65	70	73	75	84	82	81	76	78	77	78	78
Nitrate	mg/L	1.1	1.2	1.0	0.9	1.3	1.1	0.7	0.6	0.4	0.5	0.8	1.0
Fluoride	mg/L	0.9	0.8	0.9	0.8	0.9	0.9	0.8	0.8	0.8	0.8	0.8	0.9
Total Dissolved Solids (TDS)	mg/L	264	329	344	386	447	459	442	398	386	358	342	336
Total Hardness as $CaCO_3$	mg/L	104	146	146	173	197	203	197	174	170	150	141	140

Total Alkalinity as $CaCO_3$	mg/L	80	86	86	92	100	103	104	98	97	92	89	88
Free Carbon Dioxide	mg/L	0.8	1.0	1.2	1.2	1.3	0.9	0.9	0.8	0.8	0.8	0.8	0.8
pH	pH	8.29	8.23	8.18	8.20	8.18	8.37	8.38	8.41	8.39	8.36	8.35	8.35
Specific Conductance	µS/cm	476	587	606	683	771	783	749	693	688	628	612	599
Color	CU	2			1			1			1		
Turbidity	NTU	0.06	0.05	0.06	0.07	0.07	0.06	0.06	0.07	0.06	0.06	0.07	0.06
Temperature	°C	16	16	17	20	23	25	27	28	27	24	20	18
Saturation Index	--	0.10	0.23	0.20	0.35	0.47	0.72	0.73	0.70	0.66	0.5	0.40	0.35
State Project Water	%	76	73	74	66	51	43	46	54	56	66	77	78

Constituents	**Units**	**Jan-13**	**Feb-13**	**Mar-13**	**Apr-13**	**May-13**	**Jun-13**	**Jul-13**	**Aug-13**	**Sep-13**	**Oct-13**	**Nov-13**	**Dec-13**
Silica	mg/L	12.1	10.5	10.2	6.9	7.3	8.5	8.7	8.3	7.8	6.1	6.6	8.1
Calcium	mg/L	31	40	40	56	60	60	58	60	55	59	60	61
Magnesium	mg/L	14	16	16	20	22	23	22	22	22	21	22	23
Sodium	mg/L	66	69	70	78	81	84	85	84	87	81	82	81

Potassium	mg/L	3.4	3.5	3.5	3.9	4.0	4.1	4.0	4.0	4.1	4.3	4.2	4.2
Carbonate	mg/L	0	0	0	0	0	0	1	0	0	0	0	0
Bicarbonate	mg/L	100	111	112	133	143	144	140	143	134	142	143	143
Sulfate	mg/L	88	114	120	166	188	190	186	188	186	183	185	188
Chloride	mg/L	79	81	82	83	87	85	86	85	86	86	87	88
Nitrate	mg/L	0.7	0.9	0.9	1	1.2	1.0	0.7	0.3	0.4	0.6	0.6	0.8
Fluoride	mg/L	0.9	0.9	0.9	0.9	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.9
Total Dissolved Solids (TDS)	mg/L	345	391	399	482	523	528	522	524	516	513	520	526
Total Hardness as CaCO3	mg/L	140	168	168	227	244	250	242	246	233	240	246	252
Total Alkalinity as CaCO3	mg/L	82	91	92	109	117	118	117	117	110	116	117	117
Free Carbon Dioxide	mg/L	0.8	0.9	1.1	1.5	1.2	1.0	0.9	0.9	1.0	1.4	2.1	2.2
pH	pH	8.33	8.30	8.23	8.18	8.31	8.39	8.43	8.41	8.37	8.22	8.06	8.04

Specific Conductance	µS/cm	604	687	720	832	873	864	875	876	844	866	874	887
Color	CU	1			2			1			1		
Turbidity	NTU	0.06	0.06	0.06	0.07	0.06	0.06	0.06	0.06	0.06	0.06	0.06	0.06
Temperature	°C	16	16	17	20	23	25	27	28	27	24	20	18
Saturation Index	--	0.25	0.37	0.34	0.51	0.74	0.88	0.94	0.92	0.81	0.66	0.46	0.39
State Project Water	%	80	65	64	36	22	18	20	18	21	18	15	12
Constituents	**Units**	**Jan-14**	**Feb-14**	**Mar-14**	**Apr-14**	**May-14**	**Jun-14**	**Jul-14**	**Aug-14**	**Sep-14**	**Oct-14**	**Nov-14**	**Dec-14**
Silica	mg/L	8.3	7.9	8.4	8.0	7.9	8.9	9.0	9.0	9.0	9.2	9.5	8.9
Calcium	mg/L	67	70	64	70	70	57	43	49	63	65	71	72
Magnesium	mg/L	24	25	23	25	24	21	17	19	23	24	26	26
Sodium	mg/L	88	90	84	86	89	82	72	76	90	90	97	98
Potassium	mg/L	4.3	4.3	4.1	4.3	4.3	4.0	3.6	3.9	4.4	4.5	4.7	4.7

Carbonate	mg/L	0	0	0	0	0	0	0	0	0	0	0	0
Bicarbonate	mg/L	150	156	146	155	156	137	121	128	143	150	155	152
Sulfate	mg/L	204	209	191	211	209	163	121	141	195	187	224	236
Chloride	mg/L	88	90	89	90	90	87	84	88	94	93	97	100
Nitrate	mg/L	1.0	1.1	1.0	1.3	1.2	0.9	0.5	0.4	0.8	0.7	0.9	1.2
Fluoride	mg/L	0.8	0.9	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.9
Total Dissolved Solids (TDS)	mg/L	560	576	538	574	574	493	411	451	551	549	608	624
Total Hardness as CaCO3	mg/L	264	278	261	276	276	226	180	202	247	264	286	284
Total Alkalinity as CaCO3	mg/L	123	128	120	127	128	112	99	105	117	123	127	125
Free Carbon Dioxide	mg/L	2.0	2.3	2.0	2.2	2.2	1.9	1.6	1.8	2.0	2.0	2.0	2.1
pH	pH	8.09	8.06	8.09	8.07	8.08	8.08	8.09	8.07	8.08	8.10	8.10	8.08
Specific Conductance	µS/cm	951	971	890	947	944	831	726	777	902	913	978	1010

Color	CU	1			1			1			1		
Turbidity	NTU	0.06	0.06	0.06	0.05	0.05	0.06	0.05	0.05	0.06	0.05	0.05	0.06
Temperature	°C	16	17	19	21	24	26	26	28	29	26	22	18
Saturation Index	--	0.47	0.49	0.51	0.53	0.61	0.50	0.36	0.44	0.59	0.61	0.56	0.51
State Project Water	%	10	0	15	6	8	32	55	46	25	17	5	0
Constituents	**Units**	**Jan-15**	**Feb-15**	**Mar-15**	**Apr-15**								
Silica	mg/L	8.4	8.1	8.2	7.8								
Calcium	mg/L	74	76	73	75								
Magnesium	mg/L	27	27	24	25								
Sodium	mg/L	97	100	91	96								
Potassium	mg/L	4.8	4.7	4.6	4.7								
Carbonate	mg/L	0	0	0	0								

Bicarbonate	mg/L	156	159	159	159										
Sulfate	mg/L	238	243	226	237										
Chloride	mg/L	99	99	97	102										
Nitrate	mg/L	1.1	1.4	1.0	1.5										
Fluoride	mg/L	0.9	0.8	0.8	0.8										
Total Dissolved Solids (TDS)	mg/L	628	639	605	629										
Total Hardness as CaCO3	mg/L	296	297	286	290										
Total Alkalinity as CaCO3	mg/L	128	130	130	130										
Free Carbon Dioxide	mg/L	2.1	2.0	2.0	2.0										
pH	pH	8.10	8.11	8.13	8.11										
Specific Conductance	µS/cm	1020	1020	972	1000										
Color	CU	1			1										

Turbidity	NTU	0.05	0.05	0.05	0.05								
Temperature	°C	16	18	20	22								
Saturation Index	--	0.53	0.57	0.60	0.63								
State Project Water	%	0	0	0	6								

Appendix 5 - Projection from Antiscalant Vendor

Avista TECHNOLOGIES

Avista Advisor

Project Details

Project: Trussell Technologies - Yan - Rainbow
Permeate Flowrate: 1875USGPM
System Recovery: 80%

Antiscalant Projection

The projection is based on the following feed water analysis. The adjusted feed is the analysis after pH correction, and any ions have been added to balance the analysis. The concentrate analysis has been calculated based on the adjusted feed, using typical rejections of a High Rejection polyamide membrane.

Ion	Feed Water	Adjusted Feed	Concentrate	
Sodium	225.00	243.73	1207.27	mg/l
Potassium	8.80	8.80	43.47	mg/l
Calcium	201.00	201.00	1003.54	mg/l
Magnesium	81.00	81.00	404.28	mg/l
Iron	0.10	0.10	0.50	mg/l
Manganese	0.02	0.02	0.10	mg/l
Barium	0.10	0.10	0.50	mg/l
Strontium	0.00	0.00	0.00	mg/l
Aluminium	0.00	0.00	0.00	mg/l
Chloride	434.00	434.00	2151.84	mg/l
Sulfate	486.00	515.36	2573.04	mg/l
Bicarbonate	313.64	277.09	1361.84	mg/l
Nitrate	0.55	0.55	2.64	mg/l
Fluoride	0.00	0.00	0.00	mg/l
Phosphate	0.03	0.03	0.15	mg/l
Silica	26.00	26.00	128.87	mg/l
CO2	20.94	58.52	58.52	mg/l
TDS		1787.78	8878.05	
pH	7.40	6.90	7.54	

Water Source: Unknown
Water Temperature: 68º F

Product Choice

Vitec Choice: Vitec 1400
Dosage: 1.83mg/l
Usage: 51.41 lb per day.
There is one dosing pump and chemical tank per membrane train.
With 1 trains, each pump will deliver 5.36USGPD

Application

Dosed Solution Strength: 100%
Pump Rate: 5.36USGPD
14.11ml/m

pH Correction

Chemical choice: Sulfuric acid
Dosage: 29.66ppm 100% H_2SO_4

140 Bosstick Blvd
San Marcos, CA 92069
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Fax: +1 (760) 744 0619

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Livingston, EH54 5GG, UK
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Fax: +44 131 449 5599

03-Sep-2015 7:56
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Project Details

Project:	Rainbow - Trussell Technologies
Permeate Flowrate:	1875USGPM
System Recovery:	80%

Scaling Potential.

Langelier Saturation Index (LSI)

The reject stream has a LSI of 2.14.
Vitec 1400 has a limit of 3.00

Calcium Carbonate Precipitation Potential (CCPP)

The concentrate has a CCPP of 672mg/l.
This is within the limits of Vitec 1400.

Calcium Sulfate

The concentrate has a calcium sulphate saturation of 126.60%.
This is within the limits of Vitec 1400.

Barium Sulfate

The concentrate has a barium sulphate saturation of 5171.78%.
This is within the limits of Vitec 1400.

Strontium Sulfate

No Strontium was included in the feed water analysis.
At these design conditions and product dose, 86.250mg/l could be controlled.

Calcium Fluoride

No fluoride was included in the feed water analysis.
At these design conditions and product dose, 9.904mg/l could be controlled.

Silica

The concentrate has a silica level of 128.87mg/l.
This is within the limits of Vitec 1400.

Magnesium Hydroxide

The concentrate has a magnesium hydroxide saturation of 0.01%.

Calcium Phosphate

The concentrate has a calcium phosphate saturation of 0.00%.
This is within the limits of Vitec 1400.

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20-Aug-2015 10:09
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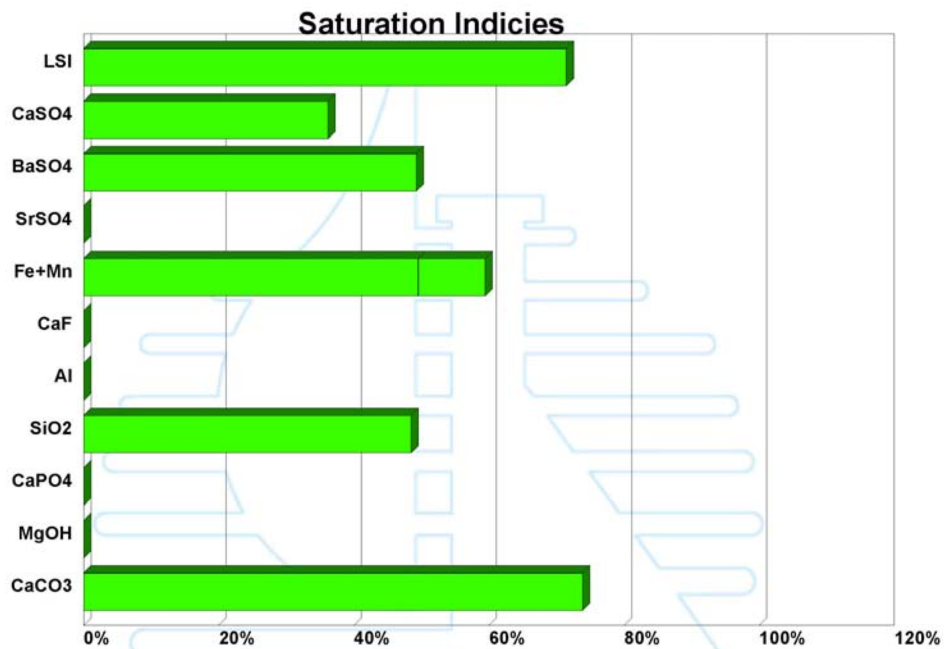
Avista Advisor

Project Details

Project:	Rainbow - Trussell Technologies
Permeate Flowrate:	1875USGPM
System Recovery:	80%



Product Choice

Vitec Choice:	Vitec 1400
Dosage:	1.83mg/l
Usage:	51.41 lb per day.

Application

Dosed Solution Strength: 100%
Pump Rate: 4.94USGPD
12.98ml/m

There is one dosing pump and chemical tank per membrane train.
With 1 trains, each pump will deliver 4.94USGPD

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20-Aug-2015 10:09
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Appendix 6 – RO Modeling Projection of TMG20-400

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TORAY Innovation by Chemistry

System Overview Report

Project	89 Rainbow SLR
Case	3 Toray TMG20-400
Revision	6 T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SPI(Elem1)=0.10, Brackish Well, Feed: 726.0 gal/min, TDS: 1559.8, Perm: 580.8, TDS: 65, Tot Elem: 168, 1st Elem: TMH20A-400
Feed Water Type	Brackish Well, Note: Auto Balance is ON
Warnings and Errors	Warnings:18, Errors:0. See Important Notes at end /E
Database Info :	Project Database : \\vmware-host\Shared Folders\Documents\TorayDS2\App_Data\DS2.sdf Membrane Database (V.20133) :

		Overall	Pass 1
Raw water TDS	mg/l	1,801.7	1,792.6
Feed EC @25C / @20.00C	uS	2,705 / 2,399.5	2,720.6 / 2,413.3
Feed Pressure	psi	0.0	107.7
Temperature	deg C	20.00	
Total DP	psi	26.72	26.72
Brine Pressure	psi	121.0	121.0
Fouling Max	7.00 yrs		0.755
SP % Increase (Max)	7.00 yrs		94.87%
Recovery	%	80.00%	80.0%
Feed Flow	gal/min	726.0	726.0
Product Flow	gal/min	580.8	580.8
Average Flux	gfd	11.900	11.900
Concentrate Flow	gal/min	145.2	145.2
Product TDS	mg/l	59.60	59.60
Concentrate TDS	mg/l	8,732	8,732
Primary HP Pump kW	kilowatt	50.58	50.58
Power Consumption	kWh/m^3	0.383	0.383

Ions		Feed	Net Feed	Conc	Product	RO Permeate
Ca	mg/l	201.0	201.0	988.8	4.029	4.029
Mg	mg/l	81.00	81.00	398.5	1.624	1.624
Na	mg/l	245.6	245.6	1,180	11.818	11.818
K	mg/l	8.800	8.800	41.08	0.730	0.730
Ba	mg/l	0.100	0.100	0.492	0.002	0.002
Sr	mg/l	0.0	0.0	0.0	0.0	0.0
NH4	mg/l	0.300	0.300	1.400	0.0249	0.0249
Fe	mg/l	0.100	0.100	0.500	0.0	0.0
HCO3	mg/l	317.0	272.7	1,304	16.067	16.067
CO3	mg/l	0.680	0.185	4.760	0.0007	0.0007
CO2	mg/l	18.796	51.13	49.75	50.03	50.03
Cl	mg/l	434.0	434.0	2,112	14.553	14.553
SO4	mg/l	486.0	521.7	2,571	9.309	9.309
NO3	mg/l	0.550	0.550	2.523	0.0568	0.0568
F	mg/l	0.400	0.400	1.857	0.0357	0.0357
Br	mg/l	0.0	0.0	0.0	0.0	0.0
PO4	mg/l	0.03	0.03	0.149	0.0004	0.0004
SiO2	mg/l	26.00	26.00	125.1	1.214	1.214
B(Boron)	mg/l	0.160	0.160	0.268	0.133	0.133
TDS	mg/l	1,802	1,793	8,732	59.60	59.60
Feed EC @25C / @20.00C	uS	2,705 / 2,400	2,721 / 2,413	11,644 / 10,357	107.8 / 95.6	107.8 / 95.6
pH	pH	7.400	6.900	7.530	5.548	5.548
Osmotic Press (DS1 / Pitzer)	psi	14.647 / 13.18	14.523 / 13.28	69.263 / 59.38	0.531 / 1.11	0.531 / 1.11
LSI / SDSI		0.50 / 0.45	-0.07 / -0.12	1.71 / 1.37	-3.84 / -4.03	-3.84 / -4.03
CaSO4 / SrSO4 %	%	15.9% / 0.0%	17.0% / 0.0%	134.0% / 0.0%	0.0% / 0.0%	0.0% / 0.0%
BaSO4 / SiO2 %	%	771.8% / 22.6%	823.3% / 27.8%	5324.4% / 108.8%		
Pitzer % Solubility	Calcite/Dolomite	154% / 626%	42% / 46%	2,214% / 128,663%		
Pitzer % Solubility	CaSO4/SrSO4	17% / 0%	18% / 0%	128% / 0%		

Toray TDS2: Rainbow SLR, T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SPI(Elem1)=0.10, Brackish Well, Feed: Page 1 / 8

Stage/Bank Data	Pass1	Stage 1	Stage 2
Lead Element Type		TMG20-400	TMG20-400
Last Element Type		TMG20-400	TMG20-400
Total Elements	175	112	63
Total Vessels	25	16	9
Elements per Vessel		7	7
Feed Flow	gal/min	726.0	350.7
Product Flow	gal/min	375.3	205.5
Average Flux	gfd	12.013	11.698
Brine Flow	gal/min	350.7	145.2
Recovery %	%	51.69 %	58.60 %
Feed Pressure	psi	107.7	132.5
dP Elements	psi	15.247	11.477
Boost Pressure	psi	0.0	40.00
Piping Loss	psi	0.0	0.0
Net (Boost - dP piping)	psi	0.0	40.00
Brine Pressure	psi	92.46	121.0
Permeate Pressure	psi	0.0	0.0
Feed TDS	mg/l	1,793	3,679
Perm TDS	mg/l	32.93	108.3
Lead Element	Pass1	Stage 1	Stage 2
Feed Flow	gal/min	45.38	38.97
Product Flow	gal/min	3.916	4.167
Product TDS	mg/l	18.580	50.00
Flux	gfd	14.040	14.941
Last Element	Pass1	Stage 1	Stage 2
Product Flow	gal/min	2.770	2.262
Product TDS	mg/l	59.06	239.6
Brine/Product Ratio	ratio	7.915	7.132
Brine Flow	gal/min	21.92	16.135
Net Driving Pressure	psi	63.07	52.55
Beta		1.111	1.116

Chemicals 100%. Disclaimer: These estimated dose rates are provided as a courtesy to Toray DS2 users and are not guaranteed.

Feed pretreat: Sulfuric Acid, 36.43 mg/l, 144.17 kg/day

Warnings

1. TMG20-400. Stage 1, Element 1: This Element will be phased out soon. Please check availability with Toray sales personnel.
2. TMG20-400. Stage 1, Element 2: This Element will be phased out soon. Please check availability with Toray sales personnel.
3. TMG20-400. Stage 1, Element 3: This Element will be phased out soon. Please check availability with Toray sales personnel.
4. TMG20-400. Stage 1, Element 4: This Element will be phased out soon. Please check availability with Toray sales personnel.
5. TMG20-400. Stage 1, Element 5: This Element will be phased out soon. Please check availability with Toray sales personnel.
6. TMG20-400. Stage 1, Element 6: This Element will be phased out soon. Please check availability with Toray sales personnel.
7. TMG20-400. Stage 1, Element 7: This Element will be phased out soon. Please check availability with Toray sales personnel.
8. TMG20-400. Stage 2, Element 1: This Element will be phased out soon. Please check availability with Toray sales personnel.
9. TMG20-400. Stage 2, Element 2: This Element will be phased out soon. Please check availability with Toray sales personnel.
10. TMG20-400. Stage 2, Element 3: This Element will be phased out soon. Please check availability with Toray sales personnel.
11. TMG20-400. Stage 2, Element 4: This Element will be phased out soon. Please check availability with Toray sales personnel.
12. TMG20-400. Stage 2, Element 5: This Element will be phased out soon. Please check availability with Toray sales personnel.
13. TMG20-400. Stage 2, Element 6: This Element will be phased out soon. Please check availability with Toray sales personnel.
14. TMG20-400. Stage 2, Element 7: This Element will be phased out soon. Please check availability with Toray sales personnel.
15. Conc Stiff Davis Index = 1.37 Warning - the Stiff Davis Index (SDSI) is greater than 0. Scale inhibitor required.
16. Conc CaSO4 % Sat'n = 134.01 Warning - concentrate calcium sulfate exceeds saturation.
17. Conc BaSO4 % Sat'n = 5324.41 Warning - concentrate barium sulfate exceeds saturation.
18. Conc SiO2 % Sat'n = 108.81 Warning - concentrate silica exceeds saturation.

Errors

Disclaimer :
The program is intended to be used by persons having technical skill, at their own discretion and risk. The projections, obtained with the program, are the expected system performance, based on the average, nominal element-performance and are not automatically guaranteed. Toray shall not be liable for any error or miscalculation in the program. The obtained results cannot be used to raise any claim for liability or warranty. It is the users responsibility to make provisions against fouling, scaling and chemical attacks, to account for piping and valve pressure losses, feed pump suction pressure and permeate backpressure. For questions please contact us:

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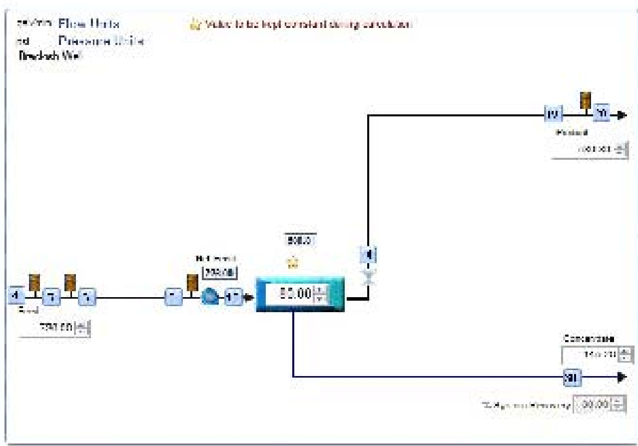
Zone B, Tianzhu Airport Industrial Zone, Beijing 101318, China

http://www.toraywater.com/

Date/Time :	8/18/2015 12:25:11 PM
Project	89:Rainbow SLR
Case :	3:Toray TMH20A-400C
Revision :	6:T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SP(Elem1)=0.10, Brackish Well, Feed: 728.0 gal/min, TDS: 1566.8, Perm: 580.8, TDS: 65, Tot Elem: 168, 1st Elem: TMH20A-400
User name :	WIN-D0V8V9JKF97\Yan Qu
Prepared for :	RMWD
Notes :	
Membrane Database	
Version Number:	20133
ReleaseDate:	5/6/2014
UpdateBy:	PFM
Toray DS2 version :	2.0.1.98

Flow Diagram:



Stream Details					
Stream Number	Flow	Pressure	TDS	Est uS	pH
20. Final Product	580.8	0.0	59.80	107.8	5.548
4. Feed Net	728.0	0.0	1,801.69	2,706	7.400
5. Treated Feed	728.0	0.0	1,792.57	2,720.8	6.500
6. Treated Feed 2	728.0	0.0	1,792.57	2,720.8	6.500
10. Feed to Pass 1	728.0	107.7	1,792.57	2,720.8	6.500
19. Permeate with blend	580.8	0.0	59.80	107.8	5.548
30. Conc to brine	146.2	121.0	8,732.02	11,844.2	7.530

Element Details in Pass 1

Pass 1 Stage 1	Element 1	Element 2	Element 3	Element 4	Element 5
Model	TMG20-400	TMG20-400	TMG20-400	TMG20400	TMG20-400

Toray DS2: Rainbow SLR, T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SP(Elem1)=0.10, Brackish Well, Feed: Page 3 / 8

Area m^2 / dia inch	37.30 / 8	37.30 / 8	37.30 / 8	37.30 / 8	37.30 / 8
Age	7	7	7	7	7
SPI %/yr	10	10	10	10	10
SPI Applied	94.87	94.87	94.87	94.87	94.87
Fouling	0.755	0.755	0.755	0.755	0.755
Recovery %	8.630	8.990	9.387	9.817	10.276
Feed Flow(gal/min)	45.38	41.46	37.73	34.19	30.83
Perm Flow(gal/min)	3.916	3.727	3.542	3.357	3.168
Conc Flow(gal/min)	41.46	37.73	34.19	30.83	27.67
Flux(gfd)	14.040	13.364	12.699	12.035	11.361
Beta	1.092	1.094	1.098	1.101	1.105
Feed Press(psi)	107.7	104.6	101.9	99.46	97.33
DP(psi)	3.086	2.742	2.424	2.130	1.860
Conc Press(psi)	104.6	101.9	99.46	97.33	95.47
Perm Press(psi)	0.0	0.0	0.0	0.0	0.0
Pi_Feed(psi)	14.523	15.870	17.404	19.168	21.21
Pi_Memb(psi)	16.556	18.173	20.03	22.18	24.67
Pi_Conc(psi)	15.857	17.391	19.150	21.18	23.54
Pi_Perm(psi)	0.166	0.195	0.229	0.277	0.339
Net Press(psi)	89.88	85.40	81.02	76.67	72.29

Pass 1 Stage 1	Element 6	Element 7
Model	TMG20-400	TMG20-400
Area m^2 / dia inch	37.30 / 8	37.30 / 8
Age	7.000	7.000
SPI %/yr	10.000	10.000
SPI Applied	94.87	94.87
Fouling	0.755	0.755
Recovery %	10.750	11.217
Feed Flow(gal/min)	27.67	24.69
Perm Flow(gal/min)	2.974	2.770
Conc Flow(gal/min)	24.69	21.92
Flux(gfd)	10.664	9.931
Beta	1.108	1.111
Feed Press(psi)	95.47	93.85
DP(psi)	1.614	1.390
Conc Press(psi)	93.85	92.46
Perm Press(psi)	0.0	0.0
Pi_Feed(psi)	23.57	26.33
Pi_Memb(psi)	27.57	30.96
Pi_Conc(psi)	26.30	29.51
Pi_Perm(psi)	0.420	0.526
Net Press(psi)	67.78	63.07

Perm mg/l Pass 1 Stage 1	Element 1	Element 2	Element 3	Element 4	Element 5
Ca	1.215	1.431	1.696	2.064	2.539
Mg	0.490	0.577	0.684	0.832	1.023
Na	3.598	4.235	5.015	6.097	7.492
K	0.226	0.265	0.314	0.381	0.467
Ba	0.0006	0.0007	0.0008	0.001	0.0013
Sr	0.0	0.0	0.0	0.0	0.0
NH4	0.0077	0.009	0.0107	0.013	0.0159
Fe	0.0	0.0	0.0	0.0	0.0
HCO3	5.171	6.005	6.954	8.374	10.234
Cl	4.420	5.202	6.162	7.491	9.207
SO4	2.809	3.308	3.920	4.768	5.865
NO3	0.0177	0.0208	0.0245	0.0298	0.0365
F	0.0111	0.013	0.0154	0.0187	0.0229
Br	0.0	0.0	0.0	0.0	0.0
B	0.103	0.108	0.113	0.119	0.126
SiO2	0.513	0.589	0.680	0.791	0.928
PO4	0.0001	0.0001	0.0001	0.0002	0.0002
CO3	4.04E-05	5.51E-05	7.41E-05	0.0001	0.0002
CO2	51.13	50.73	50.69	50.32	50.12

Toray TDS2: Rainbow SLR, T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SPI(Elem1)=0.10, Brackish Well, Feed: Page 4 / 8

pH	5.252	5.320	5.382	5.465	5.552
TDS	18.580	21.76	25.59	30.98	37.96

Perm mg/l Pass 1 Stage 1	Element 6	Element 7	Stage 1
Ca	3.158	3.979	2.195
Mg	1.273	1.604	0.885
Na	9.308	11.714	6.479
K	0.580	0.728	0.404
Ba	0.0016	0.002	0.0011
Sr	0.0	0.0	0.0
NH4	0.0198	0.0248	0.0138
Fe	0.0	0.0	0.0
HCO3	12.695	15.860	8.931
Cl	11.443	14.405	7.963
SO4	7.294	9.191	5.072
NO3	0.0452	0.0567	0.0316
F	0.0284	0.0356	0.0198
Br	0.0	0.0	0.0
B	0.133	0.140	0.119
SiO2	1.101	1.323	0.816
PO4	0.0003	0.0003	0.0002
CO3	0.0003	0.0004	0.0001
CO2	50.09	49.74	50.45
pH	5.645	5.742	5.433
TDS	47.08	59.06	32.93

Feed mg/l Pass 1 Stage 1	Element 1	Element 2	Element 3	Element 4	Element 5
Ca	201.0	219.9	241.4	266.3	295.0
Mg	81.00	88.60	97.30	107.3	118.9
Na	245.6	268.4	294.5	324.5	359.2
K	8.800	9.610	10.533	11.592	12.812
Ba	0.100	0.109	0.120	0.132	0.147
Sr	0.0	0.0	0.0	0.0	0.0
NH4	0.300	0.328	0.359	0.395	0.437
Fe	0.100	0.109	0.120	0.133	0.147
HCO3	272.7	298.5	327.5	361.2	399.9
Cl	434.0	474.6	520.9	574.3	636.0
SO4	521.7	570.7	626.7	691.3	766.0
NO3	0.550	0.600	0.658	0.723	0.799
F	0.400	0.437	0.479	0.527	0.582
Br	0.0	0.0	0.0	0.0	0.0
B	0.160	0.165	0.171	0.177	0.183
SiO2	26.00	28.41	31.16	34.31	37.96
PO4	0.03	0.0328	0.0361	0.0398	0.0441
CO3	0.185	0.189	0.231	0.287	0.359
CO2	51.13	50.73	50.69	50.32	50.12
pH	6.900	6.937	6.975	7.017	7.060
TDS	1,792.57	1,960.70	2,152.31	2,373.15	2,628.42

Feed mg/l Pass 1 Stage 1	Element 6	Element 7	Stage 1
Ca	328.5	367.7	201.0
Mg	132.4	148.2	81.00
Na	399.5	446.4	245.6
K	14.226	15.869	8.800
Ba	0.163	0.183	0.100
Sr	0.0	0.0	0.0
NH4	0.485	0.541	0.300
Fe	0.164	0.184	0.100
HCO3	444.5	496.9	272.7
Cl	707.7	791.6	434.0
SO4	853.1	954.9	521.7
NO3	0.886	0.987	0.550
F	0.646	0.720	0.400
Br	0.0	0.0	0.0
B	0.190	0.197	0.160

SiO2	42.20	47.15	26.00
PO4	0.0491	0.055	0.03
CO3	0.451	0.579	0.185
CO2	50.09	49.74	51.13
pH	7.102	7.150	6.900
TDS	2,925.15	3,272.31	1,792.57

Pass 1 Stage 2	Element 1	Element 2	Element 3	Element 4	Element 5
Model	TMG20-400	TMG20-400	TMG20-400	TMG20-400	TMG20-400
Area m^2 / dia inch	37.30 / 8	37.30 / 8	37.30 / 8	37.30 / 8	37.30 / 8
Age	7	7	7	7	7
SPI %/yr	10	10	10	10	10
SPI Applied	94.87	94.87	94.87	94.87	94.87
Fouling	0.755	0.755	0.755	0.755	0.755
Recovery %	10.692	11.196	11.680	12.100	12.393
Feed Flow(gal/min)	38.97	34.80	30.91	27.30	23.99
Perm Flow(gal/min)	4.167	3.897	3.610	3.303	2.974
Conc Flow(gal/min)	34.80	30.91	27.30	23.99	21.02
Flux(gfd)	14.941	13.972	12.944	11.843	10.662
Beta	1.113	1.117	1.120	1.123	1.123
Feed Press(psi)	132.5	130.0	127.8	126.0	124.4
DP(psi)	2.504	2.158	1.848	1.571	1.329
Conc Press(psi)	130.0	127.8	126.0	124.4	123.1
Perm Press(psi)	0.0	0.0	0.0	0.0	0.0
Pi_Feed(psi)	29.55	33.00	37.04	41.79	47.34
Pi_Memb(psi)	34.71	38.99	44.01	49.84	56.56
Pi_Conc(psi)	32.96	37.00	41.73	47.26	53.67
Pi_Perm(psi)	0.446	0.556	0.703	0.902	1.179
Net Press(psi)	97.24	90.81	84.04	76.82	69.12

Pass 1 Stage 2	Element 6	Element 7
Model	TMG20-400	TMG20-400
Area m^2 / dia inch	37.30 / 8	37.30 / 8
Age	7.000	7.000
SPI %/yr	10.000	10.000
SPI Applied	94.87	94.87
Fouling	0.755	0.755
Recovery %	12.482	12.297
Feed Flow(gal/min)	21.02	18.397
Perm Flow(gal/min)	2.624	2.262
Conc Flow(gal/min)	18.397	16.135
Flux(gfd)	9.408	8.112
Beta	1.121	1.116
Feed Press(psi)	123.1	121.9
DP(psi)	1.121	0.946
Conc Press(psi)	121.9	121.0
Perm Press(psi)	0.0	0.0
Pi_Feed(psi)	53.78	61.11
Pi_Memb(psi)	64.15	72.45
Pi_Conc(psi)	60.97	69.07
Pi_Perm(psi)	1.571	2.123
Net Press(psi)	60.96	52.55

Perm mg/l Pass 1 Stage 2	Element 1	Element 2	Element 3	Element 4	Element 5
Ca	3.373	4.217	5.350	6.900	9.057
Mg	1.360	1.700	2.156	2.781	3.650
Na	9.915	12.381	15.687	20.20	26.48
K	0.615	0.766	0.969	1.245	1.626
Ba	0.0017	0.0021	0.0027	0.0034	0.0045
Sr	0.0	0.0	0.0	0.0	0.0
NH4	0.021	0.0261	0.033	0.0424	0.0554
Fe	0.0	0.0	0.0	0.0	0.0
HCO3	13.480	16.808	21.29	27.18	35.58
Cl	12.199	15.236	19.313	24.89	32.64

SO4	7.792	9.739	12.356	15.939	20.93
NO3	0.0479	0.0596	0.0753	0.0967	0.126
F	0.0301	0.0375	0.0474	0.0609	0.0795
Br	0.0	0.0	0.0	0.0	0.0
B	0.133	0.141	0.151	0.161	0.173
SiO2	1.034	1.238	1.503	1.855	2.331
PO4	0.0003	0.0004	0.0005	0.0006	0.0008
CO3	0.0003	0.0005	0.0007	0.0012	0.0021
CO2	49.56	49.18	48.99	49.00	49.15
pH	5.675	5.772	5.875	5.977	6.090
TDS	50.00	62.35	78.94	101.4	132.7

Perm mg/l Pass 1 Stage 2	Element 6	Element 7	Stage 2
Ca	12.100	16.425	7.376
Mg	4.876	6.619	2.972
Na	35.30	47.82	21.56
K	2.161	2.915	1.325
Ba	0.006	0.0082	0.0037
Sr	0.0	0.0	0.0
NH4	0.0737	0.0994	0.0452
Fe	0.0	0.0	0.0
HCO3	47.64	64.24	29.10
Cl	43.56	59.06	26.58
SO4	27.97	37.99	17.045
NO3	0.168	0.226	0.103
F	0.106	0.143	0.0648
Br	0.0	0.0	0.0
B	0.186	0.199	0.159
SiO2	2.988	3.903	1.939
PO4	0.0011	0.0014	0.0006
CO3	0.0038	0.0071	0.0018
CO2	49.41	49.65	49.26
pH	6.212	6.335	5.900
TDS	177.1	239.6	108.3

Feed mg/l Pass 1 Stage 2	Element 1	Element 2	Element 3	Element 4	Element 5
Ca	413.7	462.8	520.6	588.8	668.9
Mg	166.7	186.5	209.8	237.3	269.6
Na	501.4	560.2	629.3	710.4	805.4
K	17.782	19.838	22.24	25.06	28.33
Ba	0.206	0.230	0.259	0.293	0.333
Sr	0.0	0.0	0.0	0.0	0.0
NH4	0.606	0.676	0.758	0.854	0.966
Fe	0.207	0.232	0.261	0.296	0.336
HCO3	557.8	623.4	700.0	789.6	894.0
Cl	889.8	994.9	1,118.39	1,263.74	1,434.28
SO4	1,074.42	1,202.12	1,352.45	1,529.68	1,738.06
NO3	1.105	1.231	1.379	1.551	1.752
F	0.807	0.900	1.008	1.136	1.284
Br	0.0	0.0	0.0	0.0	0.0
B	0.204	0.213	0.222	0.231	0.241
SiO2	52.94	59.16	66.46	75.05	85.13
PO4	0.0619	0.0693	0.078	0.0882	0.100
CO3	0.746	0.958	1.237	1.609	2.104
CO2	49.56	49.18	48.99	49.00	49.15
pH	7.197	7.245	7.292	7.340	7.387
TDS	3,678.52	4,113.49	4,624.51	5,225.67	5,930.84

Feed mg/l Pass 1 Stage 2	Element 6	Element 7	Stage 2
Ca	762.2	869.2	413.7
Mg	307.2	350.3	166.7
Na	915.6	1,041.18	501.4
K	32.11	36.38	17.782
Ba	0.379	0.432	0.206
Sr	0.0	0.0	0.0

Toray TDS2: Rainbow SLR, T=20.0 deg C, Recov=80.0%, FF(Elem1)=0.75, SPI(Elem1)=0.10, Brackish Well, Feed: Page 7 / 8

NH4	1.095	1.240	0.606
Fe	0.384	0.438	0.207
HCO3	1,014.73	1,151.86	557.8
Cl	1,632.56	1,859.19	889.8
SO4	1,980.96	2,259.50	1,074.42
NO3	1.981	2.240	1.105
F	1.454	1.646	0.807
Br	0.0	0.0	0.0
B	0.250	0.259	0.204
SiO2	96.84	110.2	52.94
PO4	0.114	0.130	0.0619
CO3	2.763	3.632	0.746
CO2	49.41	49.65	49.56
pH	7.435	7.482	7.197
TDS	6,750.65	7,687.86	3,678.52

Appendix 7- Proposal from IM Treatment OEM

HUNGERFORD & TERRY, INC.
Manufacturers of Water Treating Equipment

P.O. BOX 650
CLAYTON, NEW JERSEY 08312-0650
856-881-3200
FAX 856-881-6859
E-Mail: fcaligiuri@hungerfordterry.com

August 7, 2015

Trussell Technologies, Inc.
232 N. Lake Ave., Suite 300
Pasadena, CA 91101

Attention: Mr. Israel Monroy

Reference: Iron and Manganese Removal Filter Plant
Hungerford & Terry Budgetary Proposal No. BDJ-0400a

Dear Mr. Monroy:

In response to your request, Hungerford & Terry, Inc. is pleased to submit for your consideration the attached ***REVISED*** Budgetary Proposal No. BDJ-0400a. This proposal defines an assumed scope of materials and services offered for your consideration in connection with the subject project.

We very much appreciate the opportunity to submit this proposal, and we hope that it meets your immediate requirements. Should you have any questions or need for additional information, please feel free to contact our local representative at the address and telephone number listed below, or the undersigned at our home office in Clayton, New Jersey.

Very truly yours,
HUNGERFORD & TERRY, INC.

Frank J. Caligiuri
Frank J. Caligiuri
Vice President, Sales

REPRESENTATIVE:
Charles P. Crowley Co.
15861 Business Center Drive
Irwindale, CA 91706
Telephone No.: (626) 856-5656
Fax No.: (626) 856-5658

Enclosure

Bdj0400a Trussell Tech CA

PROPOSAL

No.: BDJ-0400a

Date: August 7, 2015

Iron and Manganese Removal Filter Plant

For:

Trussell Technologies

The equipment offered in this proposal is based on information received, and our quotation is subject to revision after 30 days from the date shown above.

HUNGERFORD & TERRY, INC.
Clayton, New Jersey

PROPOSAL

HUNGERFORD & TERRY, INC.
Manufacturers of Water Treating Equipment
Clayton, New Jersey

To: Trussell Technologies

Proposal: BDJ-0400a
Date: 8-7-15

Subject: Iron and Manganese Removal Filter Plant

We wish to submit the following quotation for your consideration, subject to the conditions printed herein.

HUNGERFORD AND TERRY IS PLEASED TO QUOTE THE FOLLOWING MATERIALS AND SERVICES:

Horizontal GreensandPlus Filtration System:

Filter Tanks:

Three (3) 120 inch OD x 32 foot straight shell, 2-cell horizontal filter tanks designed in accordance with the following:

- 100 PSI design pressure.
- ASME code section VIII construction with stamp.
- Two (2) 14 inch x 18 inch manholes in tank shell.
- One (1) internal partition plate with stiffeners and adequate support to withstand a 12 psi differential across the plate
- Necessary flanged pad or nozzle type connections.
- Four (4) lifting lugs.
- Two (2) structural steel saddle type supports.
- Tank interiors will be white metal sandblasted (SSPC-SP10) and lined with two (2) coats (6.0-8.0 mils dft) of Tnemec Series N140 potapox epoxy-polyamide potable water tank coating system.
- Tank exteriors will be commercial sandblasted (SSPC-SP6) and painted with one (1) coat (3.0-5.0 mils DFT) of Tnemec 66-1211 primer.

Notes:

1) Finish painting of the filter tank exteriors and all piping provided herein shall be furnished and field applied by the installer.

2) Anchor bolts are considered part of the foundation equipment and shall be furnished and installed by the installer.

Tank Internal Distributors:

Six (6) Single pipe type inlet distributor/waste collectors with schedule 80 PVC pipe manifolds with 2 inch orifices on 6 inch maximum centers.

Six (6) Gravel retaining screens with 304 stainless steel support angles and flats, 8-mesh #304 stainless steel screen, and stainless steel nelson studs required to fasten the screen to the tank and supports.

Six (6) Header-lateral air wash distributors of Schedule 80 PVC construction complete with 3 inch manifolds and 1/2 inch drilled laterals wrapped with two (2) layers of mesh and stainless steel supports.

Three (3) Header-lateral underdrain distributors with Schedule 80 PVC manifolds and laterals, and 5 inch square stainless steel expansive port sandvalves installed on 15 inch maximum centers.

Notes:

1.) The tank inlet and underdrain distributors shall be shop installed prior to shipment. The air wash distributors and gravel retaining screens shall be field installed by the installer as the media is placed into the units.

2.) The sandvalve underdrains require a concrete sub-fill to fill the void area in the lower tank, up through the bottom of the sand valve nozzles. All concrete shall be furnished and installed by the installer.

Filter Media:

Three (3) 16 inch beds of graded gravel support beds.

Three (3) 24 inch beds of GreensandPlus.

Three (3) 12 inch beds of Anthracite.

Notes:

1) The GreensandPlus will meet the following criteria:

a)	Specific gravity:	approx. 2.4
b)	Effective size:	0.30 – 0.35 mm
c)	Uniformity coefficient:	less than 1.6
d)	Screen grading:	18 x 60 mesh

2) The Anthracite will meet the following criteria:

a)	Specific gravity:	approx 1.6
b)	Effective size:	0.6 – 0.8 mm
c)	Uniformity coefficient:	less than 1.6

3) All filter media is to be field installed by the installer.

4) H&T will provide GreensandPlus in one-half cubic foot bags, and anthracite in one cubic foot bags.

Filter Exteriors:

Three (3) Fully automatic valve nest exteriors (with differential pressure override) consisting of:

Bray series 30 or equal butterfly control valves with wafer style cast iron bodies, ductile iron nylon coated discs, metal reinforced EPDM seats, with Bray series 70 electric actuators with auxiliary limit switches, and anti-condensation heaters for:

- Cell-1 Inlet (6 inch)
- Cell-2 Inlet (6 inch)
- Tank common outlet (8 inch)
- Tank common backwash inlet (8 inch)
- Cell-1 Backwash outlet (8 inch)
- Cell-2 Backwash outlet (8 inch)
- Tank common rinse (6 inch)
- Cell-1 Air wash (4 inch)
- Cell-2 Air wash (4 inch)
- Tank common air pressurization (4 inch)
- Tank common drain down (6 inch)
- Tank common slow refill (6 inch)

Bray series 30 equal manual butterfly valves with wafer style cast iron bodies, ductile iron nylon coated discs, and manual operators for:

- Tank-1 Inlet isolating (8 inch)
- Tank-2 Inlet isolating (8 inch)
- Tank-3 Inlet isolating (8 inch)
- Tank-1 Outlet isolating (8 inch)
- Tank-2 Outlet isolating (8 inch)
- Tank-3 Outlet isolating (8 inch)

Apollo series 70 or equal bronze ball valve with manual lever operator for:

- Tank drain (6 inch)

Apco model 200A or equal automatic air vent valves with threaded cast iron bodies and stainless steel floats for:

- Tank air vent

System Face Piping:

Schedule 40 Steel Piping:

Schedule 40 steel pipe with 150# class flat faced welded flanges and 150# butt weld fittings:

- Filter face piping
- Filter inlet header
- Filter outlet header
- Filter waste header

Schedule 40, 304 stainless steel pipe with stainless steel threaded flanges and fittings for:

- Filter air inlet header
- Air line from blower

Notes:

1) All pipe supports for the interconnecting piping that are not otherwise not located on a skid-mounted unit are to be furnished by the purchaser. Skid mounted piping will be supported from the skid and tank.

2) All system face and interconnecting piping will be furnished with the required bolts, studs, nuts, and gaskets as follows:

Bolts: ASTM A193 stainless steel
Studs: ASTM A193 stainless steel
Nuts: ASTM A194 stainless steel hex
Gaskets: Garlock style 98206

Start-Up Chemicals:

For every 100 cubic feet of GreensandPlus, 20 gallons of 15% bleach, or 50 gallons of 6% bleach is required for conditioning of the media and disinfecting of the tanks.

Notes:

1) All start-up chemicals are to be supplied by the purchaser or installer.

2) Disinfection of the entire filter system shall be performed by the purchaser or installer.

Filter Pressure Equipment:

One (1) Ashcroft model D-400 snap acting differential pressure switch with a NEMA 4 enclosure for:

- Tanks 1, 2, and 3

Nine (9) Ashcroft model 1279AS pressure gauges with 4.5 inch diameter dials and bronze bourdon tubes for:

- Cell inlets
- Tank common outlets

Ten (10) Sets of polypropylene supply tubing

Ten (10) Sets of isolating valves

Filter Flow Equipment:

Four (4) Rosemount model 3051 DP smart transmitters with stainless steel wetted parts, integeral square root extractors, linear meters and 4-20 mA outputs. Transmitters shall be furnished for the GreensandPlus filtration system common inlet and common waste. Each transmitter shall be furnished with #316 stainless steel orifice plate and polypropylene spacer type orifice union, a #316 stainless steel 3-valve manifold, and Flowtek manual #316 stainless steel ball valves for transmitter isolation, for

- Tank Inlet flow
- Common waste flow

Filter Air Blower:

One (1) Roots model URAI air blower rated 320 cfm at 5 PSI complete with flexible connections, v-belt drive with guard, inlet filter/silencer, weight type pressure relief valve, motor slide rails, common steel baseplate, and a 460 volt, 3-phase, 60 hertz, TEFC motor.

One (1) Check-Rite flange insert type check valve of steel construction:

Analytical Equiment:

One (1) Hach model CL 17 chlorine analyzer with 4-20 mA output.

- Filter effluent

One (1) Hach model 1720 E turbidity analyzer with SC200 controller for.

- Common outlet.

Filter Control Panel:

One (1) NEMA 4 free-standing/floor mount electrical control panel of ANSI 61 grey polyester powder painted steel construction complete with an Allen Bradley CompactLogix L33ER programmable controller, Allen Bradley 1769 I/O, Allen Bradley PV(+) 1250 Color/Touch OIT, N-Tron #104TX Ethernet Switch, all required, nameplates, Phoenix Contact #UT series terminal blocks, internal THHN wire, Phoenix Contact 120VAC surge suppressor, Sixnet Ethernet Surge suppressor, 24VDC power supply, Allen Bradley #700-HK36A1 interposing relays, C3 Controls door-mounted power disconnect switch, UPS (with 5 minute backup of PLC and OIT), UL-508 label, etc.

Notes:

1.) The control panel will be completely shop wired, tested, and finish painted prior to shipment.

2.) All interconnecting wiring, conduit, and wire terminations between the control panel and all local mounted electrical equipment is to be furnished and installed by the contractor.

3.) All control, status and indication shall be via the PanelView Plus OIT color/touch screens. All manual control shall be via the "soft" manual selectors on the OIT screens. No hardwired lights or switches will be supplied.

4.) H&T will provide one (1) spare Ethernet port for communication with SCADA via Ethernet/IP.

5.) Programming integral to the operation of the equipment supplied by Hungerford & Terry, Inc. will be performed by Hungerford & Terry, Inc. Should the customer choose to modify the original factory program, a laptop computer and a registered copy of Allen Bradley RSLOGIXS PLC & Allen Bradley Factory Talk OIT software will be required. Hungerford & Terry will provide a quotation to furnish the programming software and accessory hardware defined above upon request.

Shop Fabrication:

All filter tank internal distributors shall be completely shop fabricated prior to shipment.

All filter tank exteriors piping will be shop fabricated to the proper lengths as shown on our construction drawings, and will be furnished with all required flanges installed.

All equipment and materials will be shipped for jobsite installation by the purchaser unless specifically mentioned above to be installed prior to shipment.

Hungerford and Terry, Inc. Standard Surface Preparation & System Painting:

Lined Tanks:

The interiors of all lined tanks will be sandblasted to near white metal in accordance with SSPC-SP10. Lining will be as specified in the general description of the tanks.

Tank Exteriors:

The exteriors of all steel tanks will be commercial sandblasted in accordance with SSPC-SP6. Painting will be as specified in the general description of the tanks.

Misc. Equipment:

The exteriors surfaces of all misc. valves, pumps, etc. will be furnished with the manufacturer's standard prime coat painting or standard prime and finish painting.

Finish Painting:

Finish painting of all equipment furnished by Hungerford & Terry, Inc. is to be furnished and field applied by the installer.

Price: BUDGET - $975,000.00

BUDGET - $975,000.00

F.O.B. Shipping Points: With full motor freight allowed to the jobsite: Pasadena, CA
Payable in US currency, plus any applicable Municipal, State or Federal Taxes.

Payment Terms:

Unless otherwise specified, 90 % of the contract amount due within 30 days of equipment delivery. 10% of the contract amount due within 30 days of start-up, not to exceed 120 days following shipment. A service or interest charge of 1½% per month (18% per annum) will be assessed on all amounts which become past due.

A copy of the payment bond (if applicable) will be required as part of the credit approval process. Purchaser agrees to make pro rata payments for partial shipments and further agrees that if shipment of material is delayed by any act or omission on part of purchaser, payment shall become due within thirty (30) days after the material is ready for shipment

Shipment:

Will advise

Supervision:

For on-site start-up supervision, please refer to the "Field Service Page" provided.

Title and Possession:

The title and right of possession of above described articles shall remain vested in Hungerford & Terry, Inc. until Purchaser shall have made full payment thereof in cash and this right shall not be waived by attachment of said articles to the real estate. Upon Purchaser's failure to make above agreed payments or any part thereof, Hungerford & Terry, Inc. is to retain any and all partial payments which may have been made as liquidated damages, and shall be entitled to take immediate possession of said materials.

Acceptance:

This proposal, of which the Conditions of Sale are an integral part, shall not become a contract or become binding until it has been approved and signed by a representative of Hungerford & Terry, Inc. at its home office, Clayton, NJ. Persons signing on behalf of purchaser hereby represent that they are legally authorized to enter into this contract.

Acceptance by Purchaser

____

____

Date______________________

By______________________

HUNGERFORD & TERRY, INC.

By______________________
Frank Caligiuri, Vice President, Sales
This proposal is hereby accepted by:
Hungerford & Terry, Inc. August 7, 2015
(date)

CONDITIONS

Prices are based on present day labor and material costs and subject to revision after thirty days from date of quotation. They do not include any Federal, State, Municipal or other tax or Government charge applicable to the sale, shipment or use of equipment quoted on.

Deliveries are contingent upon strikes, accidents, delays in manufacture and other causes beyond our control.

Any typographical or clerical errors in the prices or specifications are subject to correction.

Order shall be made out to Hungerford & Terry, Inc., Clayton, NJ, and shall be subject to acceptance by us at Clayton, NJ. After acceptance, orders may be cancelled only with our written consent and on terms that will indemnify us against loss. Equipment on material cannot be returned except by special permission and when so returned will be subject to discount.

The Company will, free of charge, replace or repair, after receipt f.o.b. its factory promptly and within one year from shipment by it, any part of equipment which, under normal or proper use proves to be defective in workmanship or material. In no event shall the Company be liable for consequential damages.

The Company shall not be liable for failure to perform or delay in performing any obligation if such failure or delay shall be caused directly or indirectly by invasion, insurrection, riot, war, military authority, or by fire, flood, strike, or labor difficulty or by any other cause, whether of the same or different nature from those enumerated, beyond our reasonable control.

From the time said machinery of apparatus or any part thereof arrives on the premises, and until Hungerford & Terry, Inc., for an amount equal to the unpaid portion of the purchase price of the same; such loss or damage to be payable to Hungerford & Terry, Inc., as its interests may appear. All losses by fire or other casualties for which Hungerford & Terry, Inc., is not indemnified and paid under such policies of insurance, shall be borne by the Purchaser on and after the arrival of said machinery or apparatus, or any part thereof on Purchaser's premises.

HUNGERFORD & TERRY, INC.

FIELD SERVICE CHARGES

Hungerford & Terry, Inc., will furnish a Field Supervisor at USD-$ 1,500.00 per weekday of 8 consecutive hours or USD-$187.50 per hour, coinciding with the Purchaser's regular business hours during the normal work week of Monday through Friday including traveling time, plus living and traveling expenses from date of departure from Clayton, NJ, to destination and return. All time in excess of 8 hours shall be charged at 1-1/2 times the daily rate and all traveling and living expenses will be charged at cost. Meal charge is USD-$50.00 per day.

Saturday time, including traveling time, will be charged at 1-1/2 times the daily rate whereas Sunday and holiday (National) time including traveling time will be charged at 2 times the daily rate.

PLEASE NOTE

The Purchaser will be charged for the services of the Field Supervisor at the jobsite when service cannot be rendered because of delay or conditions beyond Hungerford & Terry's control. In cases of undue delay, Hungerford & Terry reserves the right to recall the supervisor.

ACCEPTED: ____________________

PER: ____________________

TITLE: ____________________

DATE: ____________________

HUNGERFORD & TERRY, INC.

PER: ____________________
Frank Caligiuri, Vice President, Sales

DATE: August 7, 2015

Alternate Section:

Alternate Item No. 1:

For Hungerford & Terry to provide a backwash reclaim system consisting of one (1) 126,000 gallon bolted steel tank (erection of the tank is included), two (2) reclaim pumps, associated valves and piping, please add the following to the base bid price:

$400,000.00

Appendix 8- Proposal from RO OEM

Biwater

BIWATER's ref:	Budget Price for 2.97 MGD RO System	Rev. 0:	Aug 6, 2015
Project:	Rainbow WMD		Page 1 of 3

To:	Trussell Technologies Inc.	Tel:	626. 463. 0402
Attn:	Yan Qu	Fax:	626. 486. 0571

Biwater Inc. (herein referred as Biwater) is pleased to submit herewith our budget offer for the design, supply, installation coordination and supervision, dry and wet test commissioning, performance testing services and training for a 2.97 MGD RO system as more detailed herein.

Raw water will be provided from wells and deliver 3.6 MGD to the RO System. The raw water will be pretreated prior to the reverse osmosis process for Iron and Manganese. The RO system will include three process trains. Each train will receive feed water where it will then split into a by-pass blend stream and the RO feed stream. Total flow will be; blend = 0.46 MGD, RO feed = 3.14 MGD. The RO system will recover ~80% of the feed water and produce 2.51 MGD of RO permeate. This permeate will blend with the by-pass blend stream to produce 2.97 MGD of blended water with ~60 mg/l of Chlorides and ~300 mg/l of TDS. Overall recovery of the pretreated well water will be ~82.5%. Higher recoveries may be possible once a maximum well TDS and individual contaminant constituents are identified for final design purposes. Post treatment for drinking water stabilization and sanitization includes a Degasifier, Chlorine and Ph adjustment chemical feed systems This design is based on the well water analyses provided to Biwater, which once balanced, equates to ~1750 mg/l.

1.0 SCOPE OF SUPPLY

Each RO train and skid will be pre-assembled and tested prior to shipping to the fullest extent possible to minimize field assembly. Once the system components are delivered to site, a Mechanical Contractor will be necessary to install the equipment per Biwater's instructions and supervision.
Biwater will complete the Plant Commissioning and Operator Training.

1.1 RO MEMBRANE SYSTEM

3 train RO system including the following main items	➢ Pretreatment equipment of acid and antiscalant chemical feeds. ➢ 1 micron Cartridge Filtration (3 units) ➢ HP Pumps and HP piping and instrumentation up to the membrane skids ➢ Membrane Skids including 3 interstage booster pumps, 20 FRP PV's for each RO Skid, total = 60. Key spec: 300 psi, 8" dia, 7-elements/PV, multi-port design. ➢ 400 sq. ft. RO membranes, total = 420 membranes. ➢ Degasifer Tower with Blower. ➢ Caustic or Soda Ash and Chlorine post treatment chemical feed systems. ➢ CIP System (1)

Biwater Inc.
190 East Arrow Highway, San Dimas, CA 91773, USA
tel: +1 (909) 599-4129 fax: +1 (909) 599-4017 www.biwater.com

Biwater

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	➢ Permeate Flush System (1) ➢ Motor Control Center ➢ Instrumentation and PLC Control and SCADA operation system

2.0 EXECUTION

2.1 **BIWATER Services for Installation SUPERVISION, Start-up, Testing and Training**
Site services are included in BIWATER's budget price.

2.2 **Warranty**

Membrane Elements
The membrane element warranty is pro-rated for 36 months from system start-up or (TBD) months from shipment, whichever occurs first. The warranty will cover materials and workmanship and performance.
Equipment and Components
Equipment and Components are warranted for a period of twelve (12) months from system start-up or 18 months from shipment, whichever occurs first.

2.3 **RO Skid Frame Structural Calculation**
Structural calculations for the RO frames (including skid anchorage requirements) endorsed by a registered Professional Engineer is included.

3.0 **BUDGET QUOTATION**
3.1 **Price**

1	Material and Site Services:	USD $1,600,000
	BASE GRAND TOTAL	**USD $1,600,000**
Delivery	FOB Job-Site	
Bonds	Not included	
Sales Tax	**NOT INCLUDED**	

4.0 **EXCLUSIONS**

The following equipment, materials and services are **excluded** from BIWATER scope of work and supply:

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1. Power, water, chemicals and consumables.
2. Water supply and disposal during setup, testing and operation.
3. All laboratory analyses cost during testing and operation.
4. All civil works
5. All equipment, labor and tools as required including heavy machinery rentals (i.e.: overhead cranes, forklifts, etc.).
6. All local permits and licenses.
7. All installation and assembly works at job-site.
8. Well Equipment and Sand Strainers if required.
9. Finished Water Tank and Water Distribution System
10. Bulk Chemical Storage for Acid if desired
11. Low pressure interconnecting plastic piping
12. Interconnecting power and control wiring between components
13. Service water system
14. Any Transformers as may be required or in-coming line sections (Power Distribution Panel)

We trust our budget proposal is acceptable to you and we look forward to working with you on this project.

Sincerely,
For and on behalf of Biwater Inc.

Jorg Menningmann
President
Cell: (626) 222-8335
Office: 909-599-4129
Email: jorg.menningmann@biwater.com

APPENDIX B

Detailed Cost Estimates for Alternatives 1 and 2

Table B-1. Groundwater Convenyance and Desalination Alternatives
Preliminary Estimated Construction Cost Alternative 1

Item No.	Item Description	Unit of Measure	Estimated Quantity	Unit Cost	Total Cost
1	Mobilization	Lump sum	1	$1,000,000	$1,000,000
2	Traffic control	Lump sum	1	$250,000	$250,000
3	Sheeting, shoring, and bracing	Lump sum	1	$250,000	$250,000
Raw Water Pipeline					
4	8-inch water main by open cut [(b)(c)]	Lin. Feet	3,000	$120	$360,000
5	10-inch water main by open cut [(b)(c)]	Lin. Feet	5,000	$150	$750,000
6	12-inch water main by open cut [(b)(c)]	Lin. Feet	15,000	$180	$2,700,000
7	14-inch water main by open cut [(b)(c)]	Lin. Feet	5,000	$210	$1,050,000
8	Well connection [(a)]	Each	20	$20,000	$400,000
9	8-inch main line gate valve	Each	2	$3,000	$6,000
10	10-inch main line gate valve	Each	2	$4,000	$8,000
11	12-inch main line gate valve	Each	6	$5,000	$30,000
12	14-inch main line gate valve	Each	2	$6,000	$12,000
13	2-inch air & vacuum valve assemblies	Each	30	$4,000	$120,000
14	4-inch blow off	Each	10	$5,000	$50,000
15	HDD Crossing (16" casing/10" carrier)	Lin. Feet	2,000	$750	$1,500,000
16	HDD Mobilization (setup, pits)	Each	1	$100,000	$100,000
17	Bore and Jack Crossing, L=~200 ft (24"casing plus carrier pipe)	Each	3	$100,000	$300,000
20	Bore and Jack mobilization (including pits)	Each	3	$30,000	$90,000
21	Trench Repavement [(d)]	Lin. Feet	28,000	$30	$840,000
Brine Pipeline					
22	8-inch water main by open cut [(b)(c)]	Lin. Feet	39,000	$120	$4,680,000
23	8-inch main line gate valve	Each	20	$3,000	$60,000
24	2-inch air & vacuum valve assemblies	Each	40	$4,000	$160,000
25	4-inch blow off	Each	10	$5,000	$50,000
26	HDD Crossing, L=~2,000 ft. (8" HDPE)	Each	2	$1,000,000	$2,000,000
27	HDD Mobilization (setup, pits)	Each	2	$100,000	$200,000
28	Bore and Jack Crossing, L=~200 ft (24"casing plus carrier pipe)	Each	3	$100,000	$300,000
29	Bore and Jack mobilization (including pits)	Each	3	$30,000	$90,000
30	Trench Repavement [(d)]	Lin. Feet	39,000	$30	$1,170,000
Pump Station at GDP					
31	Booster Pump Station at GDP (Q=500 gpm; TDH=150 ft)	Each	1	$200,000	$200,000
				Subtotal	**$18,726,000**
				Construction Contingency (30%)	$5,617,800
				Estimated Construction Cost	**$24,343,800**

[(a)] Well connection includes tee, 6-inch valve, and 200' of 6-inch piping.
[(b)] Costs do not include ROW acquistion costs.
[(c)] Water main material is assumed to be either HDPE or PVC with fused joints.
[(d)] Estimate does not include pavement overlay. It may be required to provide grind and pavement overlay for full land width.

Table B-2. Groundwater Convenyance and Desalination Alternatives
Preliminary Estimated Construction Cost Alternative 2

Item No.	Item Description	Unit of Measure	Estimated Quantity	Unit Cost	Total Cost
1	Mobilization	Lump sum	1	$1,000,000	$1,000,000
2	Traffic control	Lump sum	1	$250,000	$250,000
3	Sheeting, shoring, and bracing	Lump sum	1	$250,000	$250,000
Raw Water Pipeline (I-15 to Camino Del Rey)					
4	8-inch water main by open cut [(b)(c)]	Lin. Feet	3,000	$120	$360,000
5	10-inch water main by open cut [(b)(c)]	Lin. Feet	5,000	$150	$750,000
6	12-inch water main by open cut [(b)(c)]	Lin. Feet	15,000	$180	$2,700,000
7	14-inch water main by open cut [(b)(c)]	Lin. Feet	5,000	$210	$1,050,000
8	Well connection [(a)]	Each	20	$20,000	$400,000
9	8-inch main line gate valve	Each	2	$3,000	$6,000
10	10-inch main line gate valve	Each	2	$4,000	$8,000
11	12-inch main line gate valve	Each	6	$5,000	$30,000
12	14-inch main line gate valve	Each	2	$6,000	$12,000
13	2-inch air & vacuum valve assemblies	Each	30	$4,000	$120,000
14	4-inch blow off	Each	10	$5,000	$50,000
15	HDD Crossing (16" casing/10" carrier)	Lin. Feet	2,000	$750	$1,500,000
16	HDD Mobilization (setup, pits)	Each	1	$100,000	$100,000
17	Bore and Jack Crossing, L=~200 ft (24"casing plus carrier pipe)	Each	3	$100,000	$300,000
20	Bore and Jack mobilization (including pits)	Each	3	$30,000	$90,000
21	Trench Repavement [(d)]	Lin. Feet	28,000	$30	$840,000
Raw Water Conveyance Pipeline (Camino Del Rey to Groundwater Purification Facility)					
22	16-inch water main by open cut [(b)(c)]	Lin. Feet	48,000	$240	$11,520,000
23	16-inch main line gate valve	Each	20	$8,000	$160,000
24	2-inch air & vacuum valve assemblies	Each	40	$4,000	$160,000
25	4-inch blow off	Each	10	$5,000	$50,000
26	HDD Crossing, L=~2,000 ft. (16" HDPE)	Each	1	$1,500,000	$1,500,000
27	HDD Mobilization (setup, pits)	Each	1	$100,000	$100,000
28	Bore and Jack Crossing, L=~200 ft (30"casing plus carrier pipe)	Each	3	$100,000	$300,000
29	Bore and Jack mobilization (including pits)	Each	3	$30,000	$90,000
30	Trench Repavement [(d)]	Lin. Feet	48,000	$30	$1,440,000
Pump Station at GDP					
31	Booster Pump Station at GDP (Q=2,500 gpm; TDH=175 ft)	Each	1	$1,000,000	$1,000,000
				Subtotal	**$26,136,000**
				Construction Contingency (30%)	$7,840,800
				Estimated Construction Cost	**$33,976,800**

[(a)] Well connection includes tee, 6-inch valve, and 200' of 6-inch piping.
[(b)] Costs do not include ROW acquistion costs.
[(c)] Water main material is assumed to be either HDPE or PVC with fused joints.
[(d)] Estimate does not include pavement overlay. It may be required to provide grind and pavement overlay for full land width.