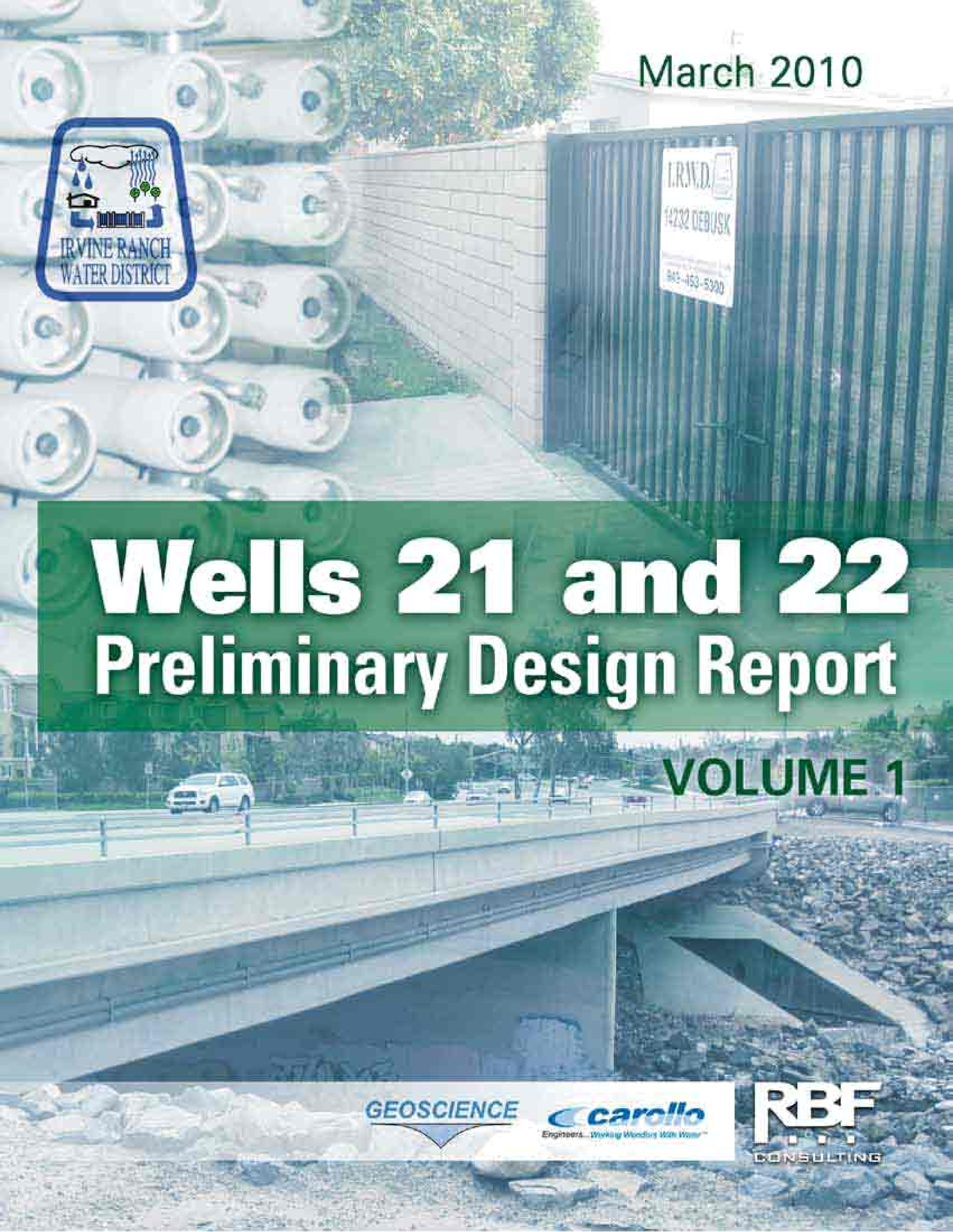
March 2010




# Wells 21 and 22 Preliminary Design Report

## VOLUME 1

# Table of Contents

## VOLUME 1

## List of Figures

## List of Tables

This page left intentionally blank.

RBF CONSULTING

# Executive Summary

# Executive Summary

## 1. General

This report presents a summary of the rehabilitation and testing of the Irvine Ranch Water District's (IRWD) Wells 21 and 22, evaluates and recommends alternatives for inclusion of the wells into domestic service, and presents a preliminary design, economic analysis, and implementation schedule for all components of the proposed project.

## 2. Introduction

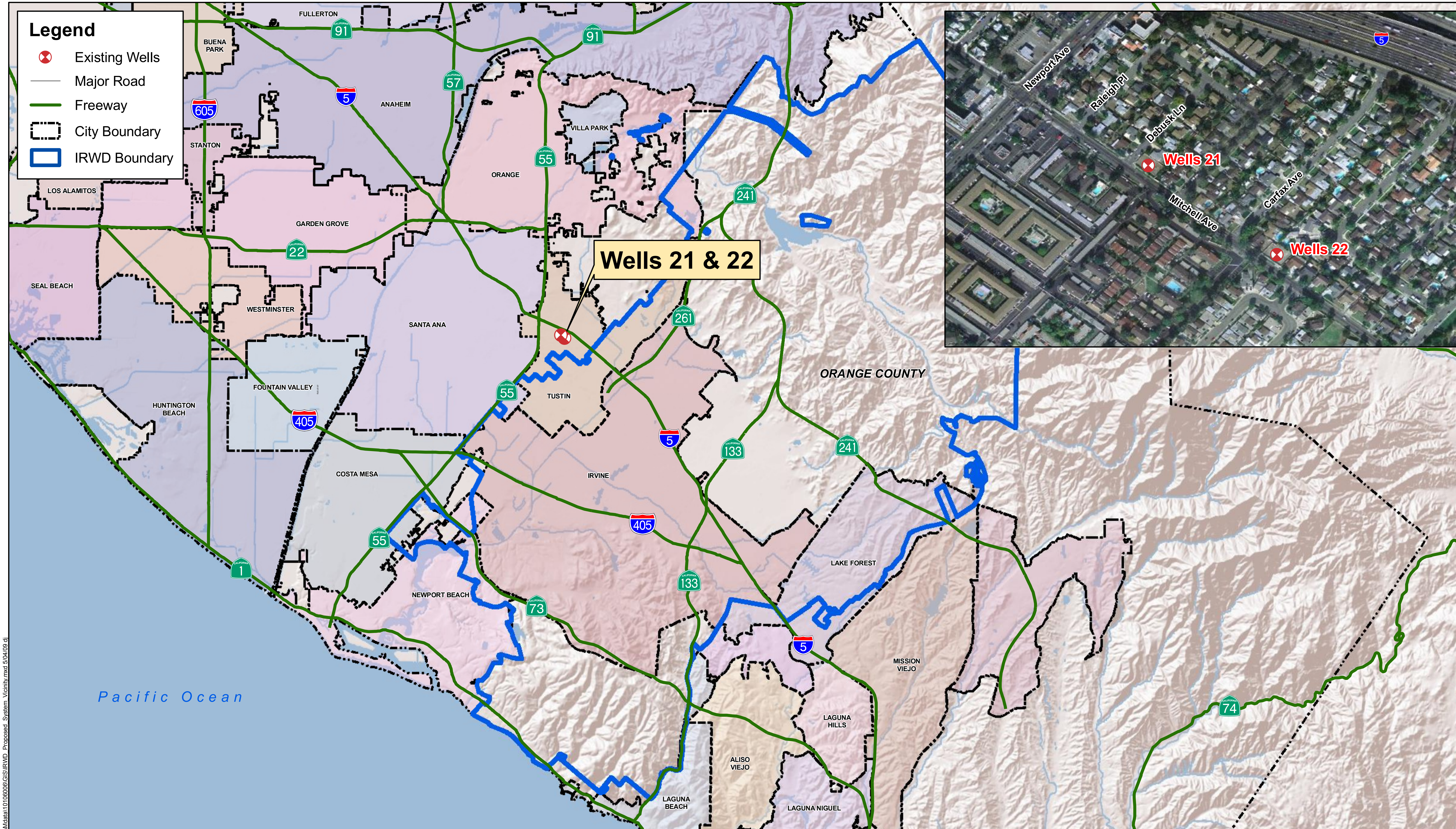
Wells 21 and 22 were originally constructed in 1992 on two sites, located in the City of Tustin, which previously housed two abandoned irrigation supply wells. The vicinity and location map of Wells 21 and 22 are shown on Figure ES-1. In 1992, the estimated combined capacity of the wells was approximately 4,250 gallons per minute (gpm): 3000 gpm for Well 21 and 1,250 gpm for Well 22. The water quality in both wells contained nitrate, total dissolved solids (TDS) and total hardness levels above regulated and/or IRWD standards. In the original Wells 21 and 22 preliminary engineering study, completed for IRWD in 1993, reverse osmosis (RO) membranes were recommended as the preferred method of treatment to reduce these constituents to acceptable levels for domestic water use. Wells 21 and 22 were never equipped and have sat idle for the past 16+ years due to the high estimated cost for treatment and conveyance.

As imported supplies continue to be stressed by drought and environmental constraints, IRWD has opted to take an updated look at the potential of integrating Wells 21 and 22 into their locally based domestic water supply. Treatment options have progressed since 1993 making various treatment processes, including membrane treatment, more efficient and cost effective.

## 3. Rehabilitation of Wells 21 and 22

Section 2 of this report provides comprehensive details and results of the rehabilitation, redevelopment and testing procedures that were conducted for Wells 21 and 22 in late 2008 through early 2009.

This page left blank intentionally




RBF CONSULTING GIS

0 2 4 Miles

Source: ESRI Online, IRWD, OCSD

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure ES-1 Vicinity Map

This page left intentionally blank.

The results of the rehabilitation and testing process showed the following:

Well 21:

- Well 21 was redeveloped to exceed its original capacity and with improved efficiency in spite of standing idle for more than 16 years. The design discharge rate following rehabilitation is 3,300 gpm with an expected 79 feet of drawdown after 1 year of continuous pumping, and with a well efficiency of 95 percent.

- The November 2008 water quality results show that the nitrate concentration (as $NO_3^-$) has increased since April 1992 when it was reported to be 43 mg/L (and nearly the state's primary maximum contaminant level (MCL) of 45 mg/L) to 67 mg/L (as $NO_3^-$), now exceeding the primary MCL. Additionally, total dissolved solids have increased slightly in that length of time from 678 mg/L to 740 mg/L, exceeding the state's recommended level of 500 mg/L. Total hardness levels have increased from 450 mg/l to 500 mg/l.

Well 22:

- Well 22 was restored above its original capacity to a design discharge rate of 1,600 gpm with 109 feet of drawdown expected after 1 year of continuous pumping at a well efficiency of 88 percent.

- Water quality results showed that nitrate concentrations (as $NO_3^-$) have increased since May 1992 when it was reported to be 47 mg/L—which is slightly above the state's primary maximum contaminant level (MCL) of 45 mg/L, to 50 mg/L in January 2009. Additionally, total dissolved solids have increased slightly from 620 mg/L to 650 mg/L, remaining above the California Department of Public Health (CDPH) recommended level of 500 mg/L. Total hardness levels have increased from 381 mg/l to 430 mg/l.

The complete results of the water quality testing are included in Appendix A.

## 4. Key Project Developments

A draft of this PDR was completed in May 2009 that provided an evaluation of the required facilities to integrate Wells 21 and 22 into the IRWD domestic distribution system. It included well head equipment, pipeline alignments and treatment facilities location and preliminary design. Since this time, several key project developments have occurred that required re-evaluation of a portion of the facilities originally proposed, the overall economics of the project and the timeline for final design and construction. These developments and their impacts are discussed below:

1. *$11.6M Grant Funding Award*

   IRWD received confirmation that the Wells 21 and 22 project has been awarded $11.6 million in grant funding from the Bureau of Reclamation (BOR) Title XVI program through the American Recovery and Reinvestment Act (ARRA). The grant funding contains stipulations regarding the time frame in which the money must be spent to receive the funding. To collect the full allotment of funding (up to 25 percent of the total project cost) the project must be substantially complete by September 2011. Based on the initial project schedule developed prior to this award, this timeline would not be met, therefore, a revised final design and construction program was needed to create an accelerated schedule.

2. *Acquisition of the former Marine Corps Air Station (MCAS) site identified for the treatment facility deemed infeasible*

   In the May 2009 Draft PDR, the MCAS site was identified as the preferred site for the treatment facility. From an engineering perspective, the MCAS site was preferred for the following reasons:

   - Vacant property.
   - Adequate space for the treatment plant, a new well and potential plant expansion.
   - Improvement plans around the site provided access off a major thoroughfare.
   - Perceived minimal visual and social impacts
   - Site is located centrally to the existing wells and the existing distribution system connection point, reducing untreated water conveyance and product water pipeline lengths.

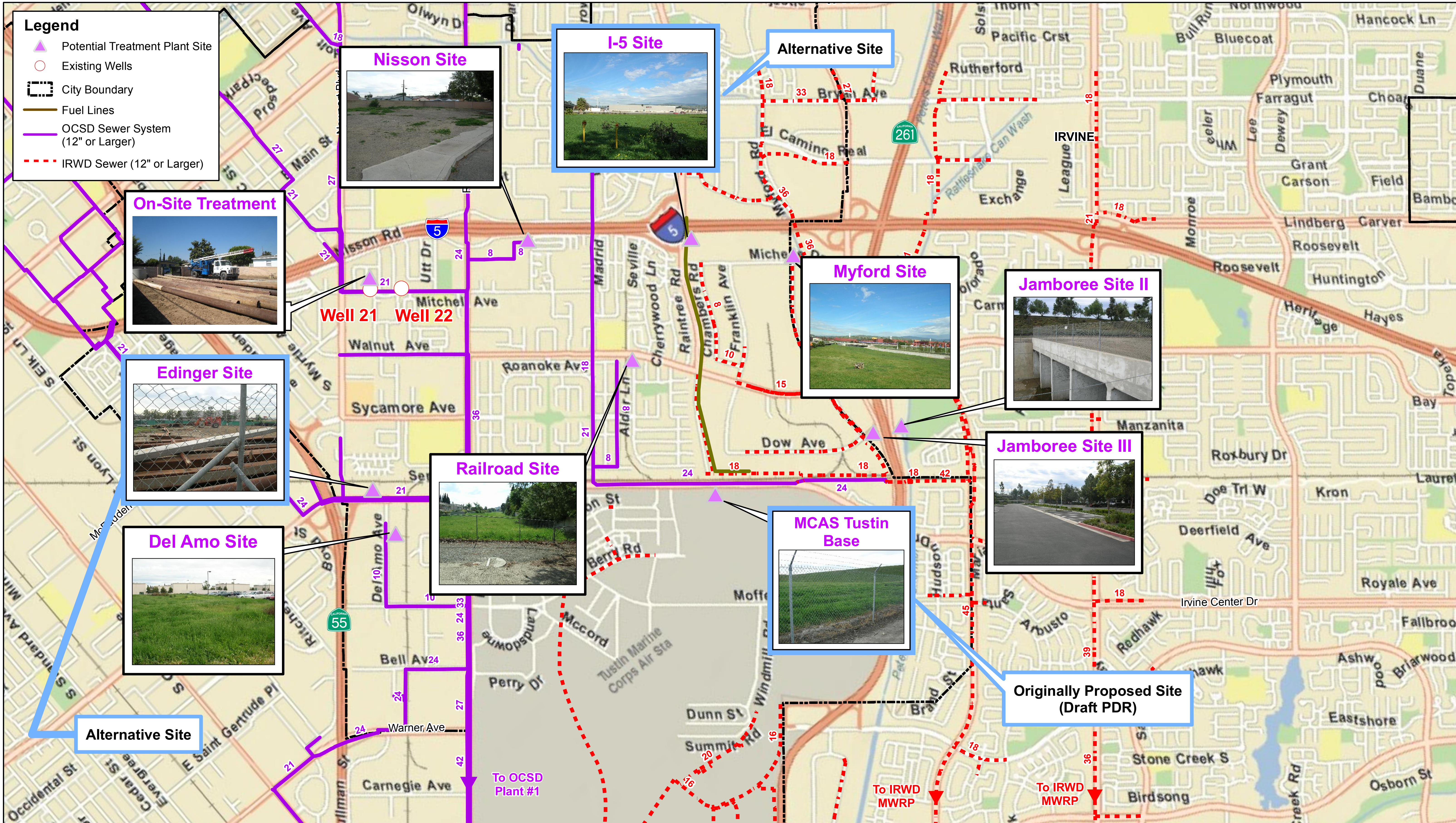
Given the required acceleration of the project schedule, due to the grant funding requirements, IRWD conducted preliminary discussions with the City of Tustin to confirm viability of acquisition and general acceptance of the proposed MCAS site land use, with the intent to begin negotiations for acquisition of the site. It became evident through these discussions that the site would be difficult to acquire and permit impacting the timing of the project.

Included in the Draft PDR were recommendations for alternative sites should negotiations fail for the preferred MCAS site. The alternative sites were located near the I-5 and Tustin Ranch Road intersection (I-5 Site) and near the 55 freeway at Edinger Avenue (Edinger Site). Figure ES-2 depicts the potential treatment plant sites originally identified in the May 2009 Draft PDR.

IRWD began a real estate search in these general areas of both vacant and occupied parcels that would be of adequate size to house the treatment facilities and were currently available for acquisition to meet the new project timeline. The sites identified during this focused treatment plant site analysis resulted with the acquisition of 1221 Edinger Avenue (Site D), a 1.88 acre site located near the 55 freeway. The site is located east of and adjacent to the Edinger Avenue Site that was originally presented in the Draft PDR as a potential treatment site. Figure ES-3 depicts the acquired site for the treatment plant.

This final draft incorporates these key developments into the project; therefore, detailed analysis that was done in preparation of the May 2009 Draft PDR that is specific to the MCAS treatment plant location is relocated in this Final report to Appendix B for preservation. This includes site layouts, pipeline alignments to and from the site, alternatives to the base project that were developed and cost analyses.

This page left blank intentionally




Source: ESRI Online

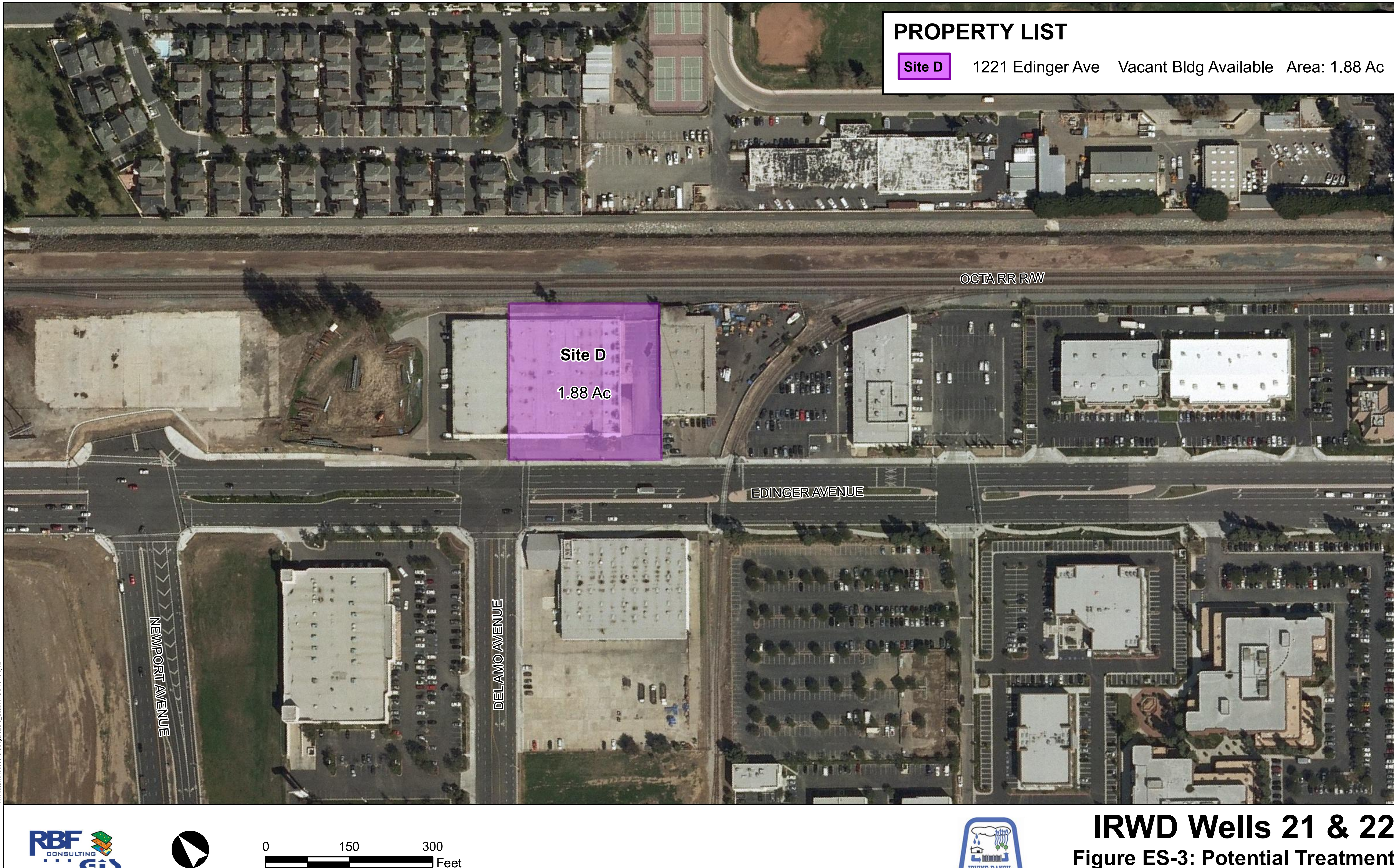
0 0.5 1 Miles

# IRWD Wells 21 & 22

## Figure ES-2: Alternative Treatment Sites

M:\Mdata\10106006\GIS\FigureES-2_new.mxd 2/16/10 dj ko

This page left intentionally blank.




Source: Eagle Aerial, 2009

# IRWD Wells 21 & 22

## Figure ES-3: Potential Treatment Plant - Site D

This page left intentionally blank.

## 5. Proposed System

*Base Project*

An evaluation of the water quality, treatment processes and required project facilities is included in Sections 3 and 4. Preliminary design, based on this analysis, is presented for the Wells 21 and 22 Base Project in Section 5. The key project developments were taken into account. The Wells 21 and 22 Base Project includes the following elements:

- Wells 21 and 22 wellhead equipping with submersible pumps and motors.
- Untreated water conveyance pipeline from the well sites to the treatment plant site.
- Wells 21 and 22 treatment plant located at the acquired 1221 Edinger Avenue site
- Brine disposal pipeline from the treatment plant to the proposed sewer connection at the intersection of Red Hill Avenue and Warner Ave.
- Product water pipeline from the treatment plant to IRWD's Zone 1 distribution pipeline in Harvard Avenue.

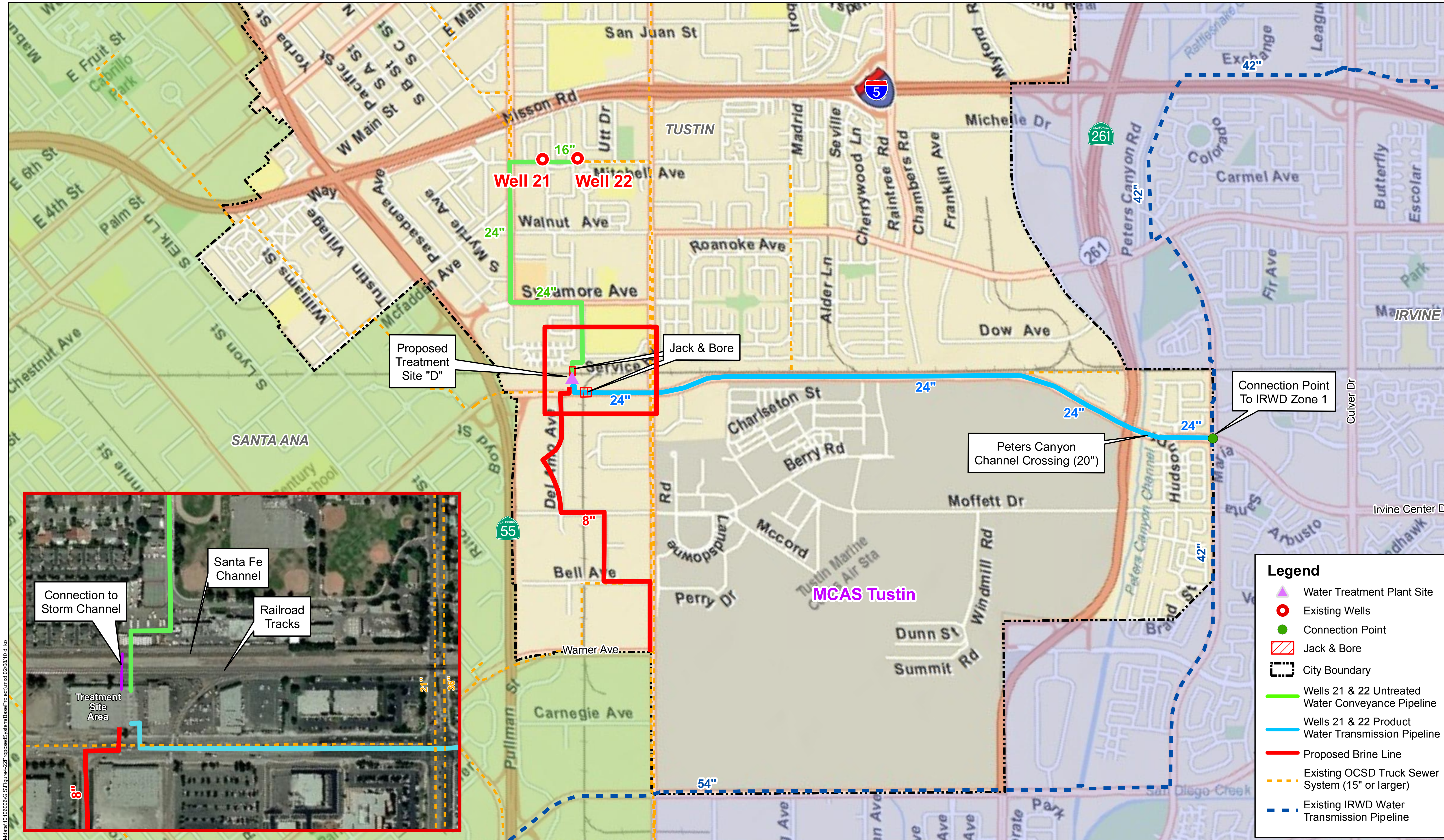
*Alternative Project*

As an option to the Base Project, an Alternative Project was evaluated, which increased the Product Water Pipeline and Brine Disposal Pipeline diameters to create additional capacity for a potential future expansion at, or near, the Wells 21 and 22 treatment plant site. The assumptions for treatment plant space allocation and upsizing of pipelines are as follows:

- An assumed increase of 4,000 gpm of groundwater supply
- An overall treatment plant recovery rate of 89% (including untreated bypass)
- The treatment plant design will also allocate space for an additional treatment system, assumed at this time to be a membrane based process

These assumptions increased the product water pipeline from 24-inch to 30-inch diameter and increased the size of the brine disposal pipeline from 8-inch to 10-inch. The proposed Wells 21 and 22 Base Project and Alternative Project facilities are shown on Figures ES-4 and ES-5, respectively.

This page left blank intentionally




RBF CONSULTING GIS

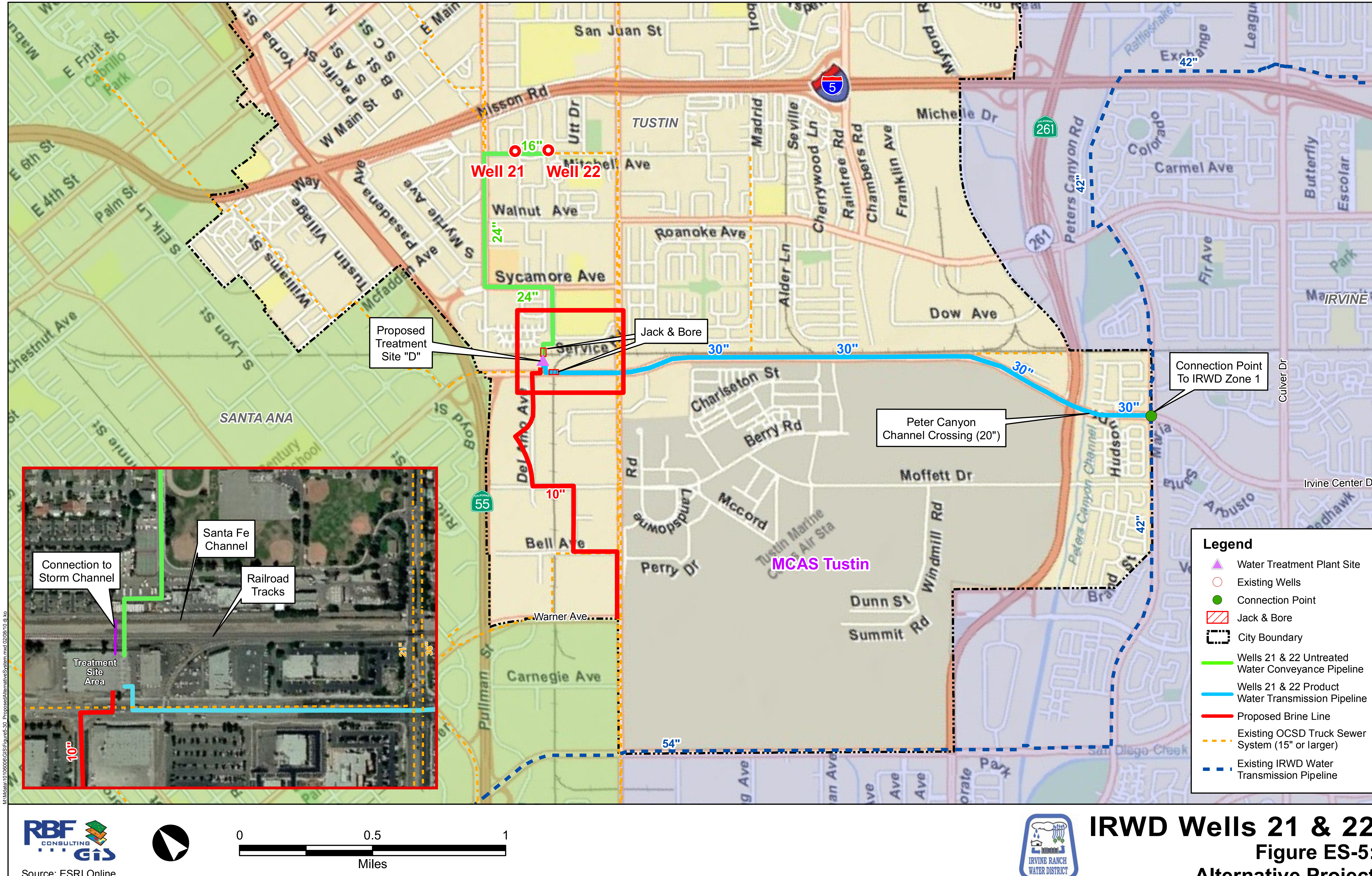
Source: ESRI Online

0 0.5 1
Miles

# IRWD Wells 21 & 22

## Figure ES-4: Proposed System (Base Project)

This page left intentionally blank.




Source: ESRI Online

**IRWD Wells 21 & 22**
**Figure ES-5:**
**Alternative Project**

This page left intentionally blank.

## 6. Economic Evaluation

For the Wells 21 and 22 Base Project and Alternative A, an economic evaluation was conducted. All costs are presented in February 2010 dollars (ENR Index = 8660). The capital costs approximate Class 4 budget estimates as defined by the Association for the Advancement of Cost Engineering (AACE) with associated accuracy of -15 percent to +30 percent. Class 4 level estimates are intended for study or feasibility purposes.

### Capital Cost

An estimate of construction cost has been prepared for the Base and Alternative Projects. Construction costs were converted to capital cost by including contingencies and non-construction project related cost to the estimate. A summary of these estimates is provided in Table ES-1. Detailed cost estimates are included in Section 6 and Appendix C of this report.

**Table ES-1 Capital Cost Estimates Summary**

| **Cost Item** | **Project Alternative** | |
|---|---|---|
| | Wells 21/22 Base Project | Alternative Project |
| Wells 21/22 Wellhead Equipping | $2,770,000 | $2,770,000 |
| Raw Water Conveyance Pipeline | $2,080,000 | $2,080,000 |
| Water Treatment Plant | $20,860,000 | $20,860,000 |
| Finished Water Pipeline | $3,310,000 | $4,400,000 |
| Brine Disposal Pipeline | $793,000 | $793,000 |
| *Subtotal Construction Cost* | $29,810,000 | $30,900,000 |
| *Contingincies @ 20%* | $5,960,000 | $6,180,000 |
| *Preliminary Design* | $1,052,000 | $1,052,000 |
| *Engineering* | $4,313,500 | $4,510,000 |
| District Costs | $1,910,000 | $1,910,000 |
| **Total Estimated Capital Cost** | **$43,050,000** | **$44,550,000** |

## Operation and Maintenance Cost

Operation and maintenance (O&M) costs have been developed from several data sources and engineering calculations. Table ES-2 contains a summary of the estimated annual O&M cost for the Base and Alternative Project components. A breakdown of the O&M costs is given in Section 6.

**Table ES-2 O&M Annual Cost Summary (2010)**

| Annual Project O & M Costs | Base Project | Alternative Project |
|---|---|---|
| Well Pump Energy Costs ($/yr) | $475,000 | $475,000 |
| Treatment Plant/Miscellaneous O&M ($/yr) | $1,845,000 | $1,876,000 |
| Brine Disposal Cost ($/yr) | $621,000 | $621,000 |
| Replenishment Assessment Cost ($/yr) | $1,575,000 | $1,575,000 |

## Annualized Cost Estimates

Methodology

Annualized capital cost (amortized over 25 years @ 4 percent) and annual O&M costs, including OCWD's replenishment assessment fee, were converted to $/acre-foot of water unit cost based on anticipated water volume production. The wells were assumed at 90 percent utilization throughout the year. Future projections, in $/acre-foot of water, to the year 2028 for the Wells 21 and 22 project were developed based on inflation assumptions and anticipated escalation of fees as described in Section 6. Two scenarios were developed to compare the Wells 21 and 22 water costs to anticipated MWDOC Tier 1, Tier 2 and a blend of Tier 1 (90%) and Tier 2 (10%) imported water cost:

- Scenario 1 - No subsidies from outside sources other than the $11.6 million BOR Title XVI/ARRA funding awarded
- Scenario 2 - An additional $250 per acre-foot subsidy from MWD through the Local Resources Program (LRP)

These two scenarios provide a "bookend" high and low cost of water for the project. The wells 21 and 22 project is believed to be a candidate to receive the MWD LRP subsidy, however, the subsidy is discretionary and can range anywhere from $0 to $250 per acre-foot.

## Economic Evaluation Results

Scenario 1

For the Base Project and Alternative Project under Scenario 1, water rates in 2010 dollars were calculated at $1,029 and $1,049 per acre-foot, respectively. It was determined, through the long term water cost projection, and as reflected in Figure ES-6, that the unit cost of water for the Wells 21 and 22 Base and Alternative Project, without additional subsidies, would be approximately 32 percent higher than MWDOC Tier 1 imported water cost at the start of the project.

Wells 21 and 22 water, under Scenario 1, is anticipated to become more competitive with imported water rates in the future. The projection shows that in 2028, the cost is estimated at 2 percent higher than Tier 1 imported water and less expensive than Tier 2 water starting in 2021. This is due to the anticipated increases in import water cost outpacing the assumed escalation of Wells 21 and 22 water costs, as described in Section 6.

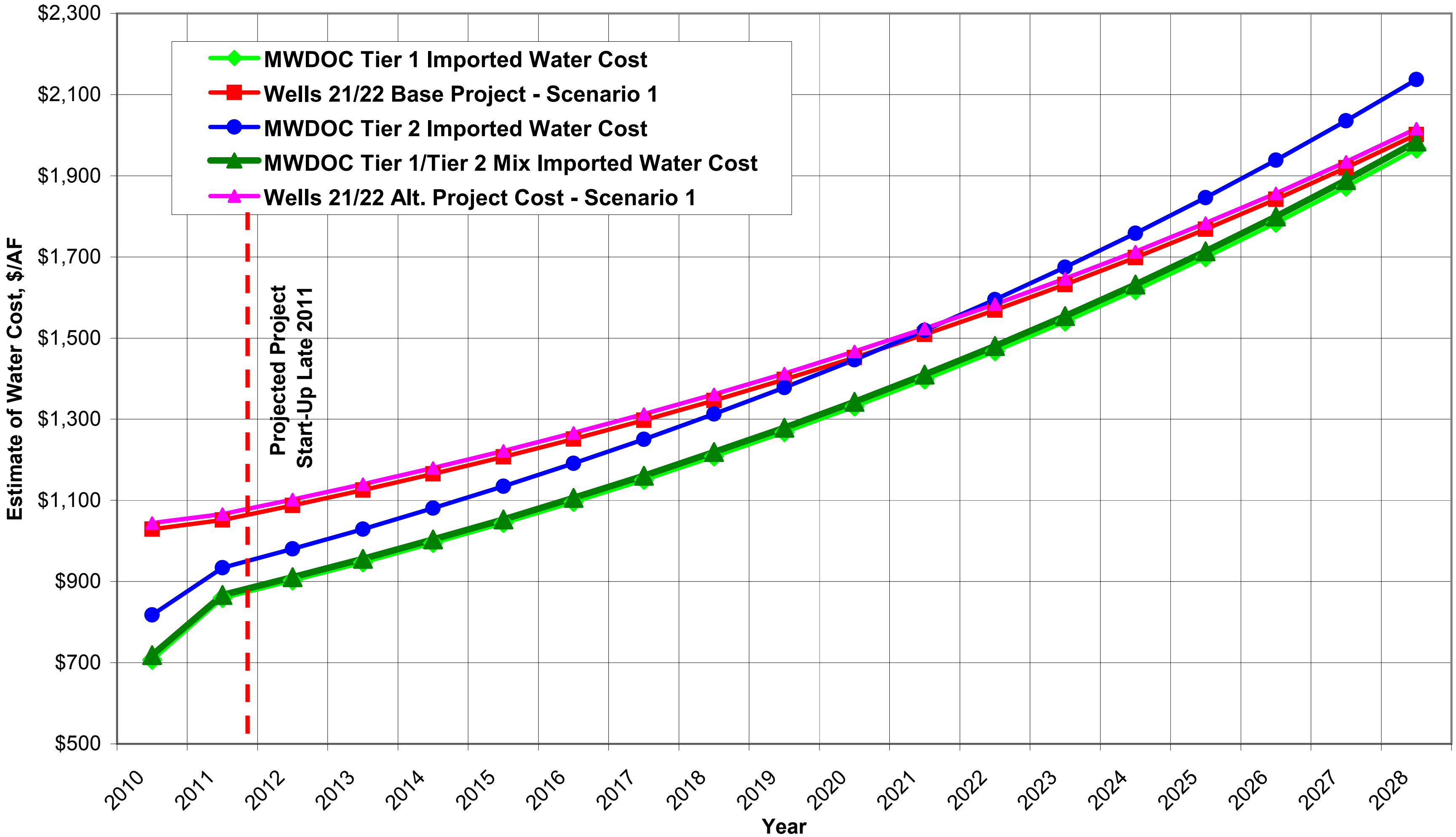
Scenario 2

For the Base Project and Alternative Project under Scenario 2, water rates in 2010 dollars were calculated at $779 and $799 per acre-foot, respectively. It was determined, through the long term water cost projection, and as reflected in Figure ES-7, with a maximum MWD LRP subsidy of $250 per acre-foot, the cost of Wells 21 and 22 water becomes less expensive than tier 1, tier 2 and the assumed tier 1/tier 2 mix imported water from 2011 through the life of the projection.

The MWD LRP program will not subsidize water that is cheaper than imported water, therefore, the actual MWD LRP subsidy, if granted to the project, would be subject to yearly audits and adjustment. Based on the long term water cost projections in this report the maximum funding available from would range from $207/AF (in 2011) to $34/AF (in 2028) to bridge the gap between Tier 1 and Wells 21 and 22 water cost.

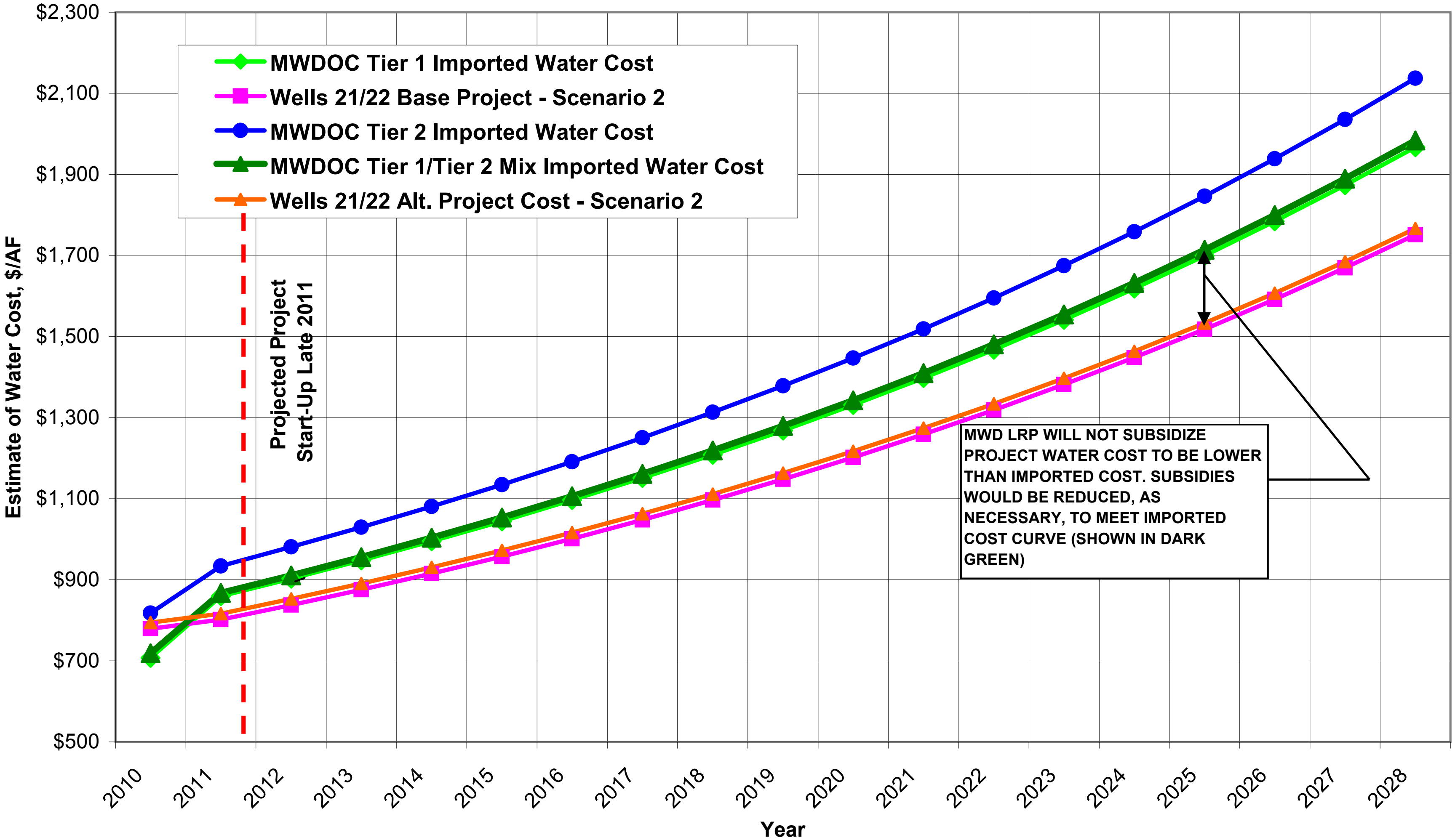
This page left blank intentionally

**FIGURE ES-6**
**Cost of Water Comparison - Scenario 1**
**Imported Water (MWDOC)**
**vs.**
**Wells 21/22 (With Title XVI Funding, Without MWD LRP Funding)**



This page left intentionally blank.

**FIGURE ES-7**
**Cost of Water Comparison - Scenario 2**
**Imported Water (MWDOC)**
**vs.**
**Wells 21/22 (With Title XVI Funding and $250/AF MWD LRP Subsidy)**



This page left intentionally blank.

## 7. Accelerated Project Implementation Schedule

A project schedule was developed to accelerate the overall timeline to design and construct the Wells 21 and 22 facilities. This was done to take advantage of the grant funding through BOR Title XVI, made available by the ARRA. The schedule assumes the following program to implement the project:

1. Wells 21 and 22 Treatment Plant—The treatment plant will be a design-build project to shorten the overall time frame that would be associated with a standard design-bid-build program.

2. Wells 21 and 22 Equipping and Pipelines—The equipping and pipeline contract(s) will be a standard design-bid-build project(s) as construction is anticipated to be complete prior to September 2011.

Figure ES-8 enumerates the major activities and timeframe for the Wells 21 and 22 treatment plant design-build project and the Wells 21 and 22 well equipping and pipelines project. Milestones for the project implementation schedules are shown in Table ES-3 below:

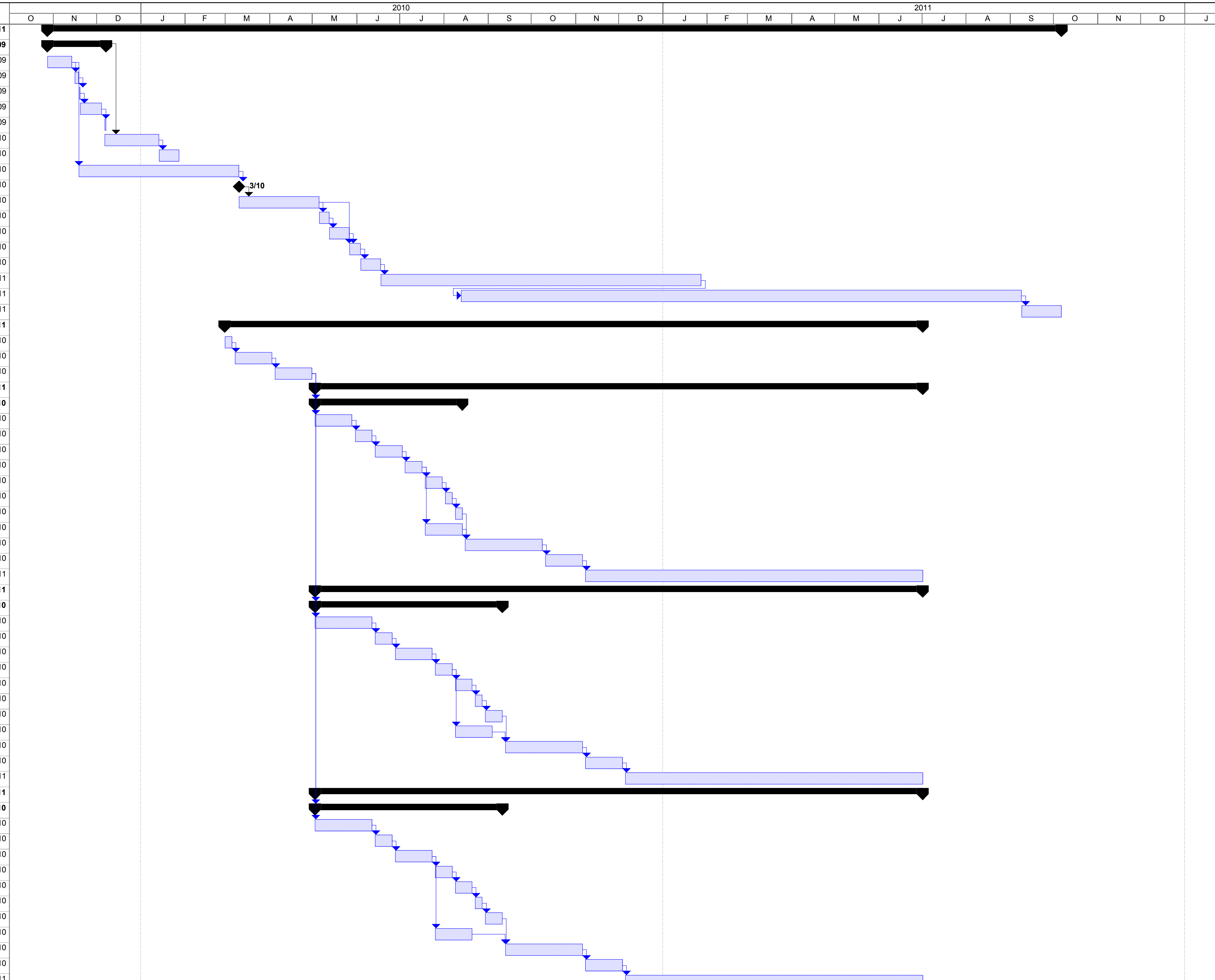
**Table ES-3 Wells 21 & 22 Implementation Schedule Summary**

| **Treatment Plant Design Build (D-B)** | **Start Date** | **Finish Date** |
|---|---|---|
| Prepare and Issue D-B RFP, Plans and Specifications | In Progress | March 10, 2010 |
| Proposal Preparation and Submittal by D-B Firms | March 10, 2010 | May 5, 2010 |
| Owner D-B Proposal Review | May 5, 2010 | May 12, 2010 |
| Award D-B Contract | May 12, 2010 | June 3, 2010 |
| Negotiations/Notice of Award/Issue Notice to Proceed | June 3, 2010 | June 17, 2010 |
| Treatment Plant Design | June 18, 2010 | January 27, 2011 |
| Treatment Plant Construction/Start-Up | August 13, 2010 | September 8, 2011 |
| **Well 21 and 22 Equipping and Pipelines** | **Start Date** | **Finish Date** |
| Prepare Design RFP and Issue | In Progress | March 5, 2010 |
| Receive Design Proposals | March 5, 2010 | April 2, 2010 |
| Review Proposals, Issue Notice to Proceed for Design | April 5, 2010 | April 30, 2010 |
| Well Equipping/Pipelines Design | May 3, 2010 | September 10, 2010 |
| Well Equipping/Pipelines Advertise, Bidding & Award | September 10, 2010 | December 3, 2010 |
| Well Equipping/Pipelines Construction and Start-Up | December 3, 2010 | July 1, 2011 |

This page left blank intentionally

**Figure ES-8**
**Wells 21 and 22 - Accelerated Implementation Schedule**

| ID | Task Name | Duration | Start | Finish |
|---|---|---|---|---|
| 1 | **Contract 1 - Treatment Facilities (Design-Build)** | **507 days** | **Wed 10/28/09** | **Thu 10/6/11** |
| 2 | **Prepare RFQ** | **29 days** | **Wed 10/28/09** | **Mon 12/7/09** |
| 3 | RBF Submits Draft RFQ to IRWD | 13 days | Wed 10/28/09 | Fri 11/13/09 |
| 4 | IRWD to Review Draft RFQ | 3 days | Mon 11/16/09 | Wed 11/18/09 |
| 5 | RBF Revises RFQ | 1 day | Thu 11/19/09 | Thu 11/19/09 |
| 6 | IRWD Reviews and Accepts Final RFQ | 11 days | Fri 11/20/09 | Fri 12/4/09 |
| 7 | Distribution of RFQ to D-B Firms | 1 day | Mon 12/7/09 | Mon 12/7/09 |
| 8 | SOQ Preparation and Submittal by D-B Firms | 28 days | Mon 12/7/09 | Wed 1/13/10 |
| 9 | Review SOQ's and Short List D-B Firms | 10 days | Thu 1/14/10 | Wed 1/27/10 |
| 10 | Prepare D-B RFP Plans and Specs | 16 wks | Thu 11/19/09 | Wed 3/10/10 |
| 11 | Issue RFP to Short Listed D-B Firms | 0 days | Wed 3/10/10 | Wed 3/10/10 |
| 12 | Proposal Preparation and Submittal by D-B Firms | 2 mons | Thu 3/11/10 | Wed 5/5/10 |
| 13 | Review, Evaluate and Rank D-B Proposals | 1 wk | Thu 5/6/10 | Wed 5/12/10 |
| 14 | Negotiate with Top Ranked D-B Firm | 2 wks | Thu 5/13/10 | Wed 5/26/10 |
| 15 | Award of Contract to D-B Firm | 6 days | Thu 5/27/10 | Thu 6/3/10 |
| 16 | Notice of Award/Notice to Proceed | 2 wks | Fri 6/4/10 | Thu 6/17/10 |
| 17 | Design | 8 mons | Fri 6/18/10 | Thu 1/27/11 |
| 18 | Construction and Start-Up | 14 mons | Fri 8/13/10 | Thu 9/8/11 |
| 19 | Commissioning | 1 mon | Fri 9/9/11 | Thu 10/6/11 |
| 20 | **Contract 2 - Well Equipping and Pipelines (Design-Bid-Build)** | **350 days** | **Mon 3/1/10** | **Fri 7/1/11** |
| 21 | Issue RFP for Well Equipping and Pipelines Design | 1 wk | Mon 3/1/10 | Fri 3/5/10 |
| 22 | Receive Consultant Proposals | 4 wks | Mon 3/8/10 | Fri 4/2/10 |
| 23 | Design Contract Award | 4 wks | Mon 4/5/10 | Fri 4/30/10 |
| 24 | **Wells 21 & 22 Wellhead Equipping** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 25 | **Final Design** | **75 days** | **Mon 5/3/10** | **Fri 8/13/10** |
| 26 | Prepare 30% Design Submittal | 4 wks | Mon 5/3/10 | Fri 5/28/10 |
| 27 | IRWD Review | 2 wks | Mon 5/31/10 | Fri 6/11/10 |
| 28 | Prepare 70% Design Submittal | 3 wks | Mon 6/14/10 | Fri 7/2/10 |
| 29 | IRWD Review | 2 wks | Mon 7/5/10 | Fri 7/16/10 |
| 30 | Prepare 100% Design Submittal | 2 wks | Mon 7/19/10 | Fri 7/30/10 |
| 31 | IRWD Review | 1 wk | Mon 8/2/10 | Fri 8/6/10 |
| 32 | Prepare/Issue Final Bid Documents | 1 wk | Mon 8/9/10 | Fri 8/13/10 |
| 33 | Complete Permits Acquisition | 1 mon | Mon 7/19/10 | Fri 8/13/10 |
| 34 | Advertising & Bidding | 8 wks | Mon 8/16/10 | Fri 10/8/10 |
| 35 | Contract Award | 4 wks | Mon 10/11/10 | Fri 11/5/10 |
| 36 | Construction, Start-Up and Testing | 8.5 mons | Mon 11/8/10 | Fri 7/1/11 |
| 37 | **Raw Water Conveyance Pipeline** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 38 | **Final Design** | **95 days** | **Mon 5/3/10** | **Fri 9/10/10** |
| 39 | Prepare 30% Design Submittal | 6 wks | Mon 5/3/10 | Fri 6/11/10 |
| 40 | IRWD Review | 2 wks | Mon 6/14/10 | Fri 6/25/10 |
| 41 | Prepare 70% Design Submittal | 4 wks | Mon 6/28/10 | Fri 7/23/10 |
| 42 | IRWD Review | 2 wks | Mon 7/26/10 | Fri 8/6/10 |
| 43 | Prepare 100% Design Submittal | 2 wks | Mon 8/9/10 | Fri 8/20/10 |
| 44 | IRWD Review | 1 wk | Mon 8/23/10 | Fri 8/27/10 |
| 45 | Prepare/Issue Final Bid Documents | 2 wks | Mon 8/30/10 | Fri 9/10/10 |
| 46 | Complete Permits Acquisition | 1 mon | Mon 8/9/10 | Fri 9/3/10 |
| 47 | Advertising & Bidding | 8 wks | Mon 9/13/10 | Fri 11/5/10 |
| 48 | Contract Award | 4 wks | Mon 11/8/10 | Fri 12/3/10 |
| 49 | Construction, Start-Up and Testing | 7.5 mons | Mon 12/6/10 | Fri 7/1/11 |
| 50 | **Finished Water Transmission Pipeline** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 51 | **Design** | **95 days** | **Mon 5/3/10** | **Fri 9/10/10** |
| 52 | Prepare 30% Design Submittal | 6 wks | Mon 5/3/10 | Fri 6/11/10 |
| 53 | IRWD Review | 2 wks | Mon 6/14/10 | Fri 6/25/10 |
| 54 | Prepare 70% Design Submittal | 4 wks | Mon 6/28/10 | Fri 7/23/10 |
| 55 | IRWD Review | 2 wks | Mon 7/26/10 | Fri 8/6/10 |
| 56 | Prepare 100% Design Submittal | 2 wks | Mon 8/9/10 | Fri 8/20/10 |
| 57 | IRWD Review | 1 wk | Mon 8/23/10 | Fri 8/27/10 |
| 58 | Prepare/Issue Final Bid Documents | 2 wks | Mon 8/30/10 | Fri 9/10/10 |
| 59 | Complete Permits Acquisition | 1 mon | Mon 7/26/10 | Fri 8/20/10 |
| 60 | Advertising & Bidding | 8 wks | Mon 9/13/10 | Fri 11/5/10 |
| 61 | Contract Award | 4 wks | Mon 11/8/10 | Fri 12/3/10 |
| 62 | Construction, Start-Up and Testing | 7.5 mons | Mon 12/6/10 | Fri 7/1/11 |




Project: Schedule 2.17.10
Date: Mon 2/22/10

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

This page left intentionally blank.

# Section 1

RBF CONSULTING

# Introduction

This page left intentionally blank.

# Section 1 Introduction

## 1.1 Project Objectives

The objective for the Wells 21 and 22 preliminary design report is to define a system that:

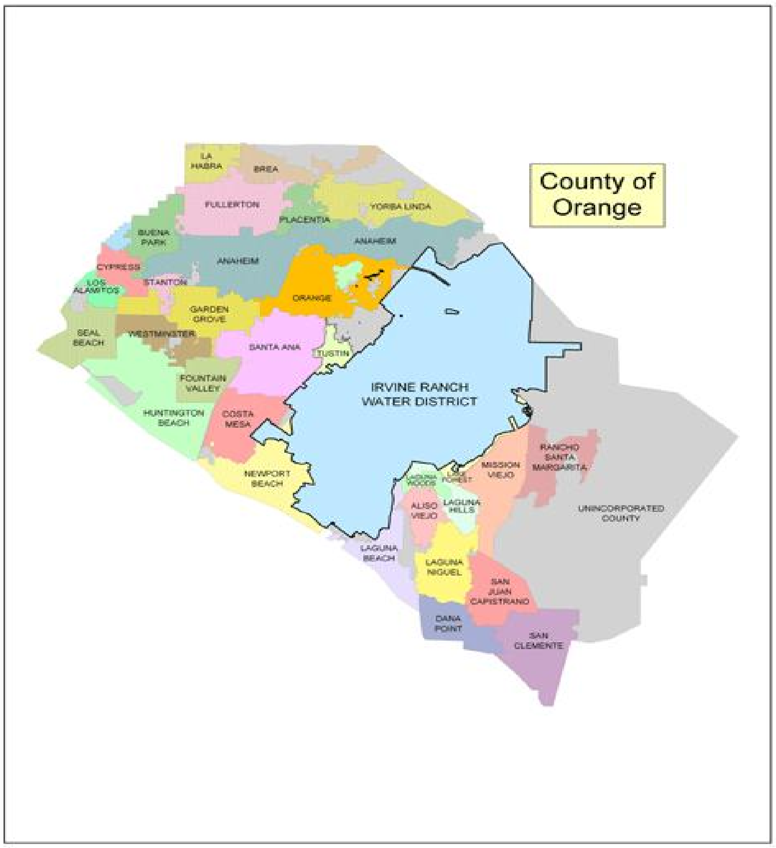
- Creates a new, non-dependent, cost effective local source of potable water supply
- Increases IRWD's domestic water system flexibility and reliability by integrating Wells 21 and 22 into the IRWD distribution system
- Allows IRWD to meet both current and future domestic water demands
- Is implementable within the time constraints of BOR funding, that is, on-line by September 2011

## 1.2 Project Background

### 1.2.1 Irvine Ranch Water District

Irvine Ranch Water District (IRWD), a public agency, provides water, reclaimed water and sewer service to Southern Central Orange County communities including all of the City of Irvine, parts of the cities of Tustin, Costa Mesa, Newport Beach, Lake Forest, and Orange, and unincorporated portions of Orange County. IRWD has been an integral part of a rapidly growing Southern California community since its inception in 1961 as a California Water District. Figure 1-1 depicts the IRWD service area, which encompasses an area of approximately 179 square miles and serves a population of over 300,000.

**Figure 1-1 IRWD Service Area**




## 1.2.2 Existing Water Supplies and Distribution System

For many years, IRWD relied heavily on imported water supplies from Metropolitan Water District of Southern California (MWD). Imported water is supplied through the Colorado River via the Colorado River Aqueduct (CRA) and from Northern California through the State Water Project (SWP). In the past three decades, IRWD has made a concerted effort to develop a reliable, cost-effective local supply to alleviate itself from dependence on imported sources. Through these efforts, more than 65 percent of the current supplies are now developed locally. This local water is extracted through groundwater wells located within and around the District's boundary.

IRWD maintains a domestic water distribution system that includes the following facilities:

- Nine pressure zones ranging in elevation from sea level to 1,400 feet MSL
- Over 90,000 customer connections
- 39 reservoirs with over 153 million gallons of storage
- Multiple distribution system pump stations

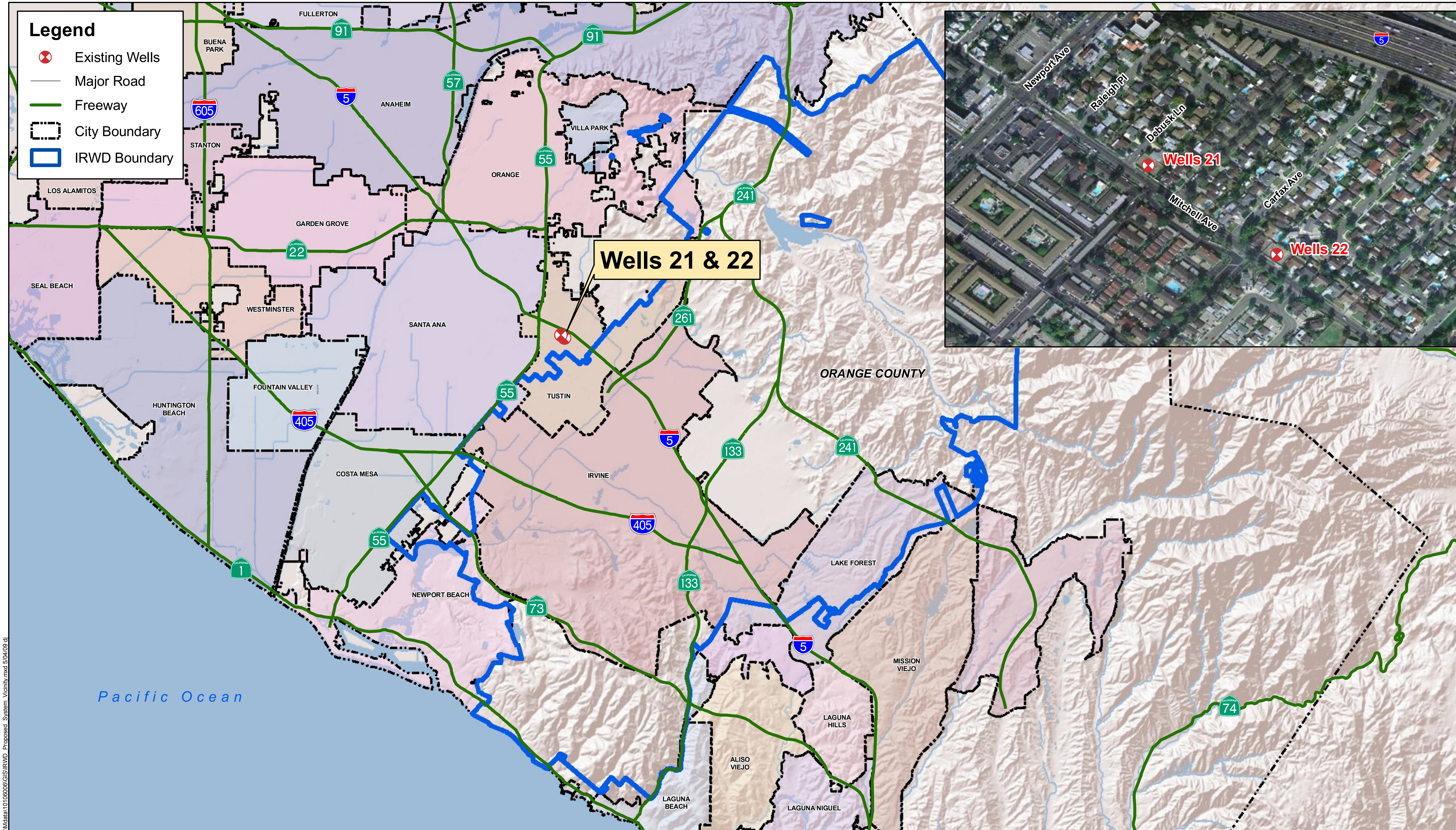
### 1.2.3 Historical Overview of Wells 21 and 22

Originally constructed by the District in 1992, Wells No. 21 and 22 have been dormant for the past 16+ years due to above-standard nitrate, total dissolved solids (TDS), and total hardness levels. The wells are located in the City of Tustin, Orange County, California, just west of IRWD's service area near the I-5 and I-55 freeways (Figure 1-2). Well 21 is located at 14232 Debusk Lane, near the intersection (northern corner) of Debusk Lane and Mitchell Avenue. Well 22 is located at 1251 Mitchell Avenue, near the intersection of Carfax Avenue and Mitchell Avenue.

During the initial drilling and construction in 1992, the wells were tested for both production capacity and water quality. The estimated capacities of Wells 21 and 22 were 3,000 gpm and 1,250 gpm respectively. While the capacity of the wells was sufficient to constitute a relatively large and reliable water supply source, the water quality results were less than desirable. The nitrate levels exceeded state and federal maximum concentration limit (MCL) of 45 mg/l (as $NO_3^-$), TDS concentrations were above the secondary standard upper limit of 500 mg/l, and hardness levels were above an acceptable concentration for the District.

A preliminary engineering analysis was conducted in 1992/1993 by Boyle Engineering to ascertain the feasibility of equipping Wells 21 and 22 and treating the water for supply into IRWD's domestic water system. The report evaluated available treatment sites, viable treatment technologies, potential for additional wells, and brine disposal options. It was concluded in the report that the best option for treatment was membrane separation by reverse osmosis (RO). IRWD decided, at that time, the project was not feasible due to the high cost of the water in comparison to imported sources.

This page left intentionally blank.




RBF CONSULTING GIS

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure 1-2: Vicinity Map

Source: ESRI Online, IRWD, OCSD

This page left intentionally blank.

## 1.3 Project Scope

The main components of the Wells 21 and 22 preliminary design project are:

- Rehabilitation and testing of Wells 21 and 22
- Development and evaluation of alternatives for required project facilities
- Preliminary design
- Permits and environmental constraints analysis
- Economic evaluation
- Grant funding research
- Development of a project schedule

### 1.3.1 Rehabilitation and Testing of Wells 21 and 22

The initial step in the Wells 21 and 22 project was the rehabilitation and testing of the wells. This was deemed necessary after sitting idle for nearly 17 years. This was a multi-staged process that included the following major tasks:

- Well inspection, pre and post rehabilitation, by video
- Mechanical and chemical cleaning of the well screens
- Well re-development by pumping
- Well pump testing
- Water quality sampling and testing

Detailed descriptions and the results of these procedures are given in Section 2 of this report. A key element in this initial stage of the project was to re-evaluate capacity and water quality of the wells.

### 1.3.2 Development and Evaluation of Alternatives for Project Facilities

The second phase of the project, once the results from the rehabilitation and testing of the wells were available, was to evaluate the following project elements required to place Wells 21 and 22 into service:

- Based on review of historic water quality from Wells 21 and 22, recent water quality data from surrounding wells, and water quality information from the testing of Wells 21 and 22 as part of the rehabilitation effort, an evaluation of:
  - The possibilities for blending of the untreated well water with both local and imported supplies to meet the District's water quality goals

    or;
  - Treatment technologies used alone or in combination to meet the District's water quality goals.
- Options for disposal of brine waste streams associated with the various water treatment processes.
- Potential sites for treatment and/or blending facilities.
- Potential alignments for an untreated water conveyance pipeline from Wells 21 and 22 to the potential treatment sites.
- Available connection points to the existing IRWD Zone 1 potable water distribution system.
- Potential alignments for a product water pipeline from the treatment sites to the potential Zone 1 connection points.

A detailed analysis of the development and evaluation of alternatives for the project facilities is discussed in Sections 3 and 4 of this report.

### 1.3.3 Preliminary Design

Based upon the results of the analyses of alternatives, and with close coordination with IRWD on the results of these analyses, the engineering team has identified the preferred treatment option and pipeline routing and advanced this alternative to preliminary design for the purpose of preparing cost estimates of sufficient detail to compare various alternatives and determine a proposed project. The proposed system and details on preliminary design elements are given in Section 5. The following items have been prepared as part of the preliminary design:

1.3.3.1 Wells 21 and 22 Facilities

- Well head equipping site layouts
- System hydraulic calculations
- Preliminary sizing of well pumps and motors
- Wellhead process flow diagram
- Electrical load requirements

1.3.3.2 Treatment Plant Site

- Treatment process description and design criteria
- Overall treatment plant site layout including:
    - Pre-treatment
    - RO trains
    - Post-treatment
    - Chemical feed systems
    - Electrical and instrumentation facilities
    - Clearwell and product water pump station
    - Control Room and miscellaneous storage areas
- Brine discharge piping layout and details
- Treatment process diagram
- Electrical load requirements

1.3.3.3 Untreated Water Conveyance Pipeline

- Proposed pipe size, type and alignment, with alternatives, depicting existing right of ways and street sections with existing utilities based on preliminary utility research
- Standard trench detail
- Pipe design details

1.3.3.4 Product Water Pipeline

- Proposed pipe size, type and alignment, with alternatives, depicting existing right of ways and street sections with major utilities
- Standard trench detail
- Pipe design details
- Peters Canyon Channel bridge crossing support detail

### 1.3.4 Economic Analysis

A detailed economic analysis was conducted for the Wells 21 and 22 Base and Alternative Project. This analysis is included in Section 6. Preliminary estimates for capital facilities cost, operation and maintenance costs, and associated fees were evaluated through 2028. The volume cost of water from Wells 21 and 22 was compared to current and projected costs of MWDOC imported water.

### 1.3.5 Grant Funding Research

DDB Engineering researched and compiled potential funding opportunities for the IRWD Wells 21 and 22 Project from local, state, and federal sources. The engineering team, along with DDB Engineering, assisted IRWD in preparation of the application for funding under Bureau of Reclamation (BOR) Title XVI and the Safe Drinking Water Revolving Fund (SDWSRF).

The following programs were identified as the possible sources of outside project funding:

- California Department of Public Health (CDPH) Safe Drinking Water State Revolving Fund
- Proposition 84, Chapter 2 "Small Community Infrastructure Improvements for Chemical and Nitrate Contaminants" and "Public Water Systems – Prevention and Reduction of Groundwater Contamination"
- Proposition 82 New Local Water Supply Construction Loans
- I-Bank Infrastructure State Revolving Fund Loan Program
- BOR Challenge Grants: Water Marketing and Efficiency Grants for the ARRA of 2009
- BOR Title XVI Funding - Water Reclamation and Reuse Program Under the ARRA of 2009
- Economic Development Administration (EDA) Recovery Act Funding

The applications that were filed during preparation of the Draft PDR resulted in a significant grant award from the BOR Title XVI funding made available through the American Recovery and Reinvestment Act (ARRA). This impacted the economics of the project significantly and will be discussed in more detail in Section 6.

#### 1.3.6 Permits and Environmental Constraints Analysis

A concept level environmental due diligence (EDD) was provided for the well sites improvements, treatment plant site and pipeline alignment locations for the project. The goal of this task is to identify key environmental issues, permits and required approvals that could significantly constrain the project as proposed.

The (EDD) analysis included a list of required environmental documentation, including the preparation of an Initial Study/Mitigated Negative Declaration (IS/MND), and other appropriate documentation under the California Environmental Quality Act (CEQA) and National Environmental Policy Act (NEPA).

To expedite the project, a mitigated negative declaration (MND) - Initial Study/Environmental Assessment (IS/EA) was completed by IRWD for the proposed project in February 2010. The document is required to meet both California Environmental Quality Act (CEQA) and the National Environmental Policy Act (NEPA). The document details environmental factors potentially affected by the project and incorporates the required mitigation measures. The entire document is available at http://www.irwd.com/AboutIRWD/board_CEQA.php.

### 1.3.7 Development of a Project Implementation Schedule

An implementation schedule has been developed and is shown in Section 7 of this report. The schedule breaks the Wells 21 and 22 Project into two separate contracts as follows:

- Treatment Plant (Design-Build)
- Wells 21 and 22 Wellhead Equipping and Pipelines (Design-Bid-Build)

# Section 2

# Rehabilitation of Wells 21 and 22

Section 2
Rehabilitation of Wells 21 and 22

This page left intentionally blank.

# Section 2 Rehabilitation of Wells 21 and 22

This section provides comprehensive details and results of the rehabilitation, redevelopment and testing procedures for Wells 21 and 22. In addition to standard well and aquifer testing, Wells 21 and 22 were tested using an inflatable packer to isolate and test by pumping selected intervals of screen within the well. This section includes a summary of the water quality analyses conducted on the samples collected during the 2008/2009 pumping tests, and a comparison of analyses results with previous results conducted during the construction of Wells 21 and 22 in 1992.

## 2.1 Background

Previously located at the Well 21 and 22 sites were the Francis Mutual Water Company (FMWC) Wells 17 and 18, respectively. Wells 17 and 18 were destroyed in February 1992 prior to drilling Wells 21 and 22. Records show the Francis Mutual wells were drilled in 1943. Well 17 was perforated from 250 to 388 feet and 596 to 1,000 feet bgs. Well 18 was perforated from 252 to 1,014 feet bgs.

Wells 21 and 22 were drilled and constructed from February to May 1992, by Beylik Drilling, Inc. of La Habra, California, and were designed and supervised during field activities by Camp Dresser & McKee, Inc. (CDM) of Irvine, California. Both wells were constructed using 20-inch diameter by 3/8-inch wall thickness mild steel casing. The casing for Wells 21 and 22 extended from ground surface to a depth of 278 feet bgs and 291 feet bgs, respectively.

At this point, the casing telescoped (i.e., reduced) from 20 to 16 inches in diameter. The reducing section is approximately 5 ft in length and is mild steel. Below the reducing section, a short length of mild steel casing is attached to a 304 stainless steel adaptor (located at 285 ft bgs at Well21, and 296 ft bgs at Well 22). All casing and screen materials from these depths and continuing to the bottom of the well consist of 16- inch diameter (5/16-inch wall thickness) type 304 stainless steel for both wells. Colorado Silica Sand fills the annular space (with an 8 x 16 gradation) from total depth of 1,110 ft bgs to 248 ft bgs at Well 21, and from 1,028 ft to 252 ft bgs at Well 22.

Following the construction and development of Wells 21 and 22, a series of well tests were conducted in April and May of 1992, which resulted in the design discharge rates and specific capacities shown in Table 2-1. A drawing showing Well 21 and Well 22 construction details are shown on Figures 2-1 and 2-2, respectively.

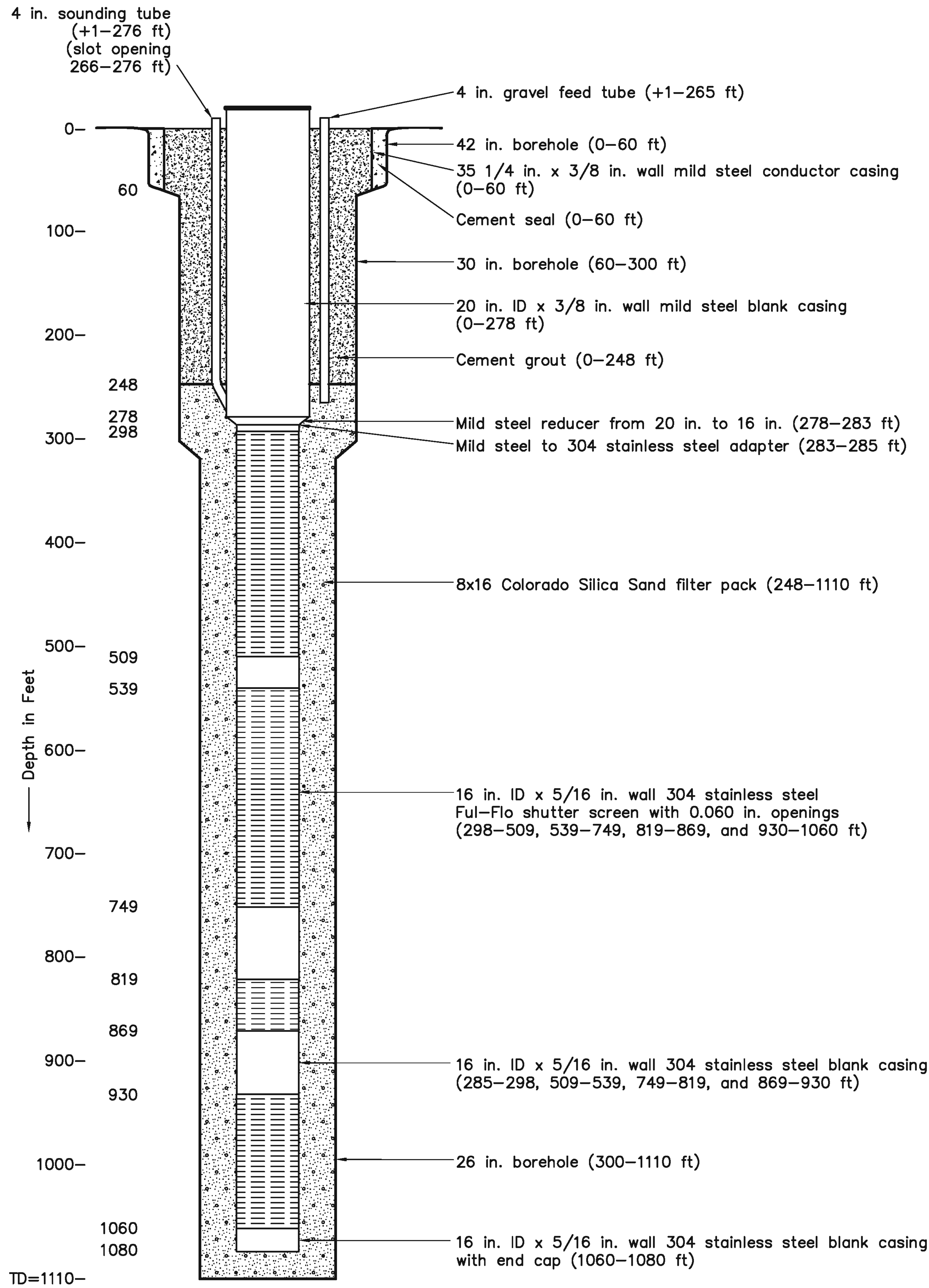
**Table 2-1 1992 Well Test Results**

| Well | Design Discharge Rate (gpm) | Specific Well Capacity (gpm/foot) |
|---|---|---|
| Well 21 | 3,000 | 72 |
| Well 22 | 1,250 | 19 |

Wells 21 and 22 were not equipped at the time of construction in 1992 primarily due to elevated nitrate levels that exceeded the State of California's Primary Drinking Water Standard and total dissolved solids concentrations that exceeded California Department of Public Health (CDPH) lower secondary standard of 500 mg/l. Additionally, hardness values for Well 21 were considerably elevated, and although there is no drinking water standard for hardness, aesthetically this level is unacceptable. A complete Title 22 water quality report was not prepared at the time of well construction, so some constituents have limited information available.

As a result of increased growth in Southern California since the time of drilling these wells, and because of the need to increase its local water supply, IRWD decided to rehabilitate Wells 21 and 22 to determine current water quality and production capability as part of their evaluation process to establish the most cost effective way to meet water quality goals and demands.

The primary steps undertaken during the rehabilitation and testing process for both wells are summarized in Table 2-2. Appendix D of this report contains the chronology of events that occurred during rehabilitation and testing of Wells 21 and 22.



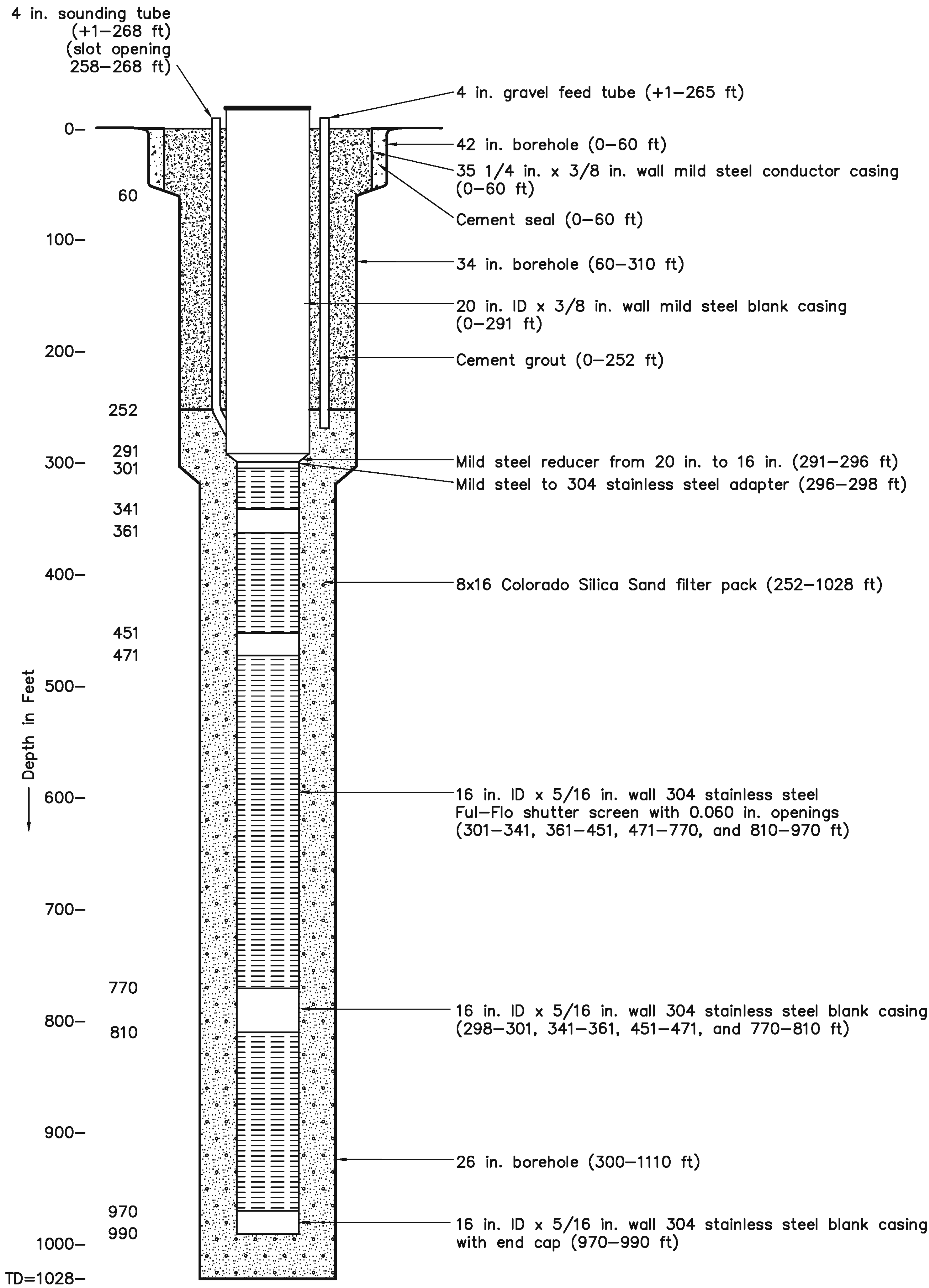

## WELL CROSS SECTION

NOTE: Depths are based on 1992 as–built information from CDM, Exhibit "B" Well Construction Report Irvine Ranch Water District Well Nos. 21 and 22, 1992

| Figure 2–1 | Drawn: PLP | RBF CONSULTING / IRVINE RANCH WATER DISTRICT | GEOSCIENCE |
|---|---|---|---|
| | Checked: | CONSTRUCTION DETAILS | GEOSCIENCE Support Services, Incorporated |
| | Approved: | IRWD WELL NO. 21 | P.O. Box 220, Claremont, CA 91711 |
| | Date: 18–MAR–09 | | Tel: (909)451–6650 Fax: (909)451–6638 www.gssiwater.com |

rbf\irwd\wells 21_22 rehab\rehab report well 21\figures\well 21 asbuilt.dwg

This page left intentionally blank.




## WELL CROSS SECTION

NOTE: Depths are based on 1992 as–built information from CDM, Exhibit "B" Well Construction Report Irvine Ranch Water District Well Nos. 21 and 22, 1992

| Figure 2–2 | Drawn: PLP | RBF CONSULTING / IRVINE RANCH WATER DISTRICT | GEOSCIENCE |
|---|---|---|---|
| | Checked: | CONSTRUCTION DETAILS<br>IRWD WELL NO. 22 | GEOSCIENCE Support Services, Incorporated<br>P.O. Box 220, Claremont, CA 91711<br>Tel: (909)451–6650 Fax: (909)451–6638<br>www.gssiwater.com |
| | Approved: | | |
| | Date: 18–MAR–09 | | |

rbf\irwd\wells 21_22 rehab\rehab report well 22\figures\well 22 asbuilt.dwg

This page left intentionally blank.

The initial downhole video survey was conducted January 17, 2008 to evaluate the condition of the well casing and screens prior to rehabilitation. The pre-rehabilitation report, as well as subsequent video survey reports, are found in Appendix E. The January 2008 video survey showed the depth to static water level in Well 21 to be approximately 97 feet below ground surface (bgs). The depth to static water level at Well 22 was measured at approximately 96 feet bgs. The video survey also showed iron bacteria tubercles in the wells that adhered primarily to the welds and heat affected zone within the stainless steel portion of the well screens.

Following the video survey, side wall samples of the tubercles were collected by Pacific Surveys and were analyzed by Dr. Leo Truttman and technical specifications for rehabilitation were developed incorporating this data. A copy of Dr. Truttman's report is found in Appendix F. Following preparation of the technical specifications and negotiating initial pricing for the work with Bakersfield Well & Pump Company (BW&P), significant delays were incurred as the storm drain in the vicinity of Wells 21 and 22 was found to be in very poor condition and would not accommodate the required flow rates (of more than 3,000 gpm) during well development and testing. Over the next several months, a temporary sewer lateral with a large-diameter manhole was constructed on the site and the necessary permits from Orange County Sanitation District (OCSD) were obtained. Actual rehabilitation work began in September 2008 and was completed in February 2009 for both the Well 21 and Well 22 sites.

**Table 2-2 Tasks Undertaken for the Rehabilitation and Testing Wells 21 and 22**

| | Task | Well 21 | Well 22 |
|---|---|---|---|
| 1 | Initially inspecting the well by video survey on January 17, 2008 | √ | √ |
| 2 | Preparation of technical specifications for rehabilitation based on the findings of the January 2008 video survey | √ | √ |
| 3 | Construction of a temporary sewer lateral onto the well site as a result of the existing storm drain being in very poor condition (completed in late August 2008) | √ | √ |
| 4 | Mobilization of equipment to the site in September 2008 | √ | √ |
| 5 | Installation of sound barriers | √ | √ |
| 6 | Brushing the well casings and screen using a nylon bristle brush to remove bulk encrustants and pipe scale | √ | √ |
| 7 | Bailing accumulated sediment from the bottom of the well | √ | √ |
| 8 | Injecting Aqua-Clear PFD, a dispersant for clays, throughout the screened interval | N/A | √ |
| 9 | Certifying the accuracy of the 4-inch flow meter | √ | √ |
| 10 | Redeveloping the well by swabbing and airlifting in 10 ft intervals | √ | √ |
| 11 | Collection of water quality samples for selenium analysis as per OCSD requirements | √ | √ |
| 12 | Inspecting the condition of well following brushing and airlifting by downhole video | √ | √ |
| 13 | Installing a vertical line-shaft test pump with an inflatable packer installed above the pump bowl assembly | √ | √ |

| | Task | Well 21 | Well 22 |
|---|---|---|---|
| 14 | Certifying the accuracy of the 12-inch flow meter | √ | √ |
| 15 | Redeveloping the well using pumping and surging techniques | √ | √ |
| 16 | Testing the well to determine yield, drawdown and sand production, as well as to determine aquifer characteristics | √ | √ |
| 17 | Collection of full Title 22 and California Unregulated water quality samples for analysis | √ | √ |
| 18 | Inflating the packer to isolate selected screen intervals below the pump intake | √ | √ |
| 19 | Pumping the selected intervals to determine basic water quality and quantity | √ | √ |
| 20 | Collection of water quality samples for analysis of general minerals and physical properties, volatile organic compounds, total organic carbon and perchlorate from Zone 1 | √ | √ |
| 21 | Lowering the pump bowls and packer assembly to the second zone for testing | √ | √ |
| 22 | Inflating the packer to isolate selected screen intervals below the pump intake | √ | √ |
| 23 | Pumping the intervals determine basic water quality and quantity | √ | √ |
| 24 | Collection of water quality samples for analysis of general minerals and physical properties, volatile organic compounds, total organic carbon and perchlorate from Zone 2 | √ | √ |
| 25 | Removing the test pumping equipment and packer assembly | √ | √ |
| 26 | Inspecting the well by video to document post-rehabilitation conditions | √ | √ |
| 27 | Removing additional accumulated sediment | √ | √ |
| 28 | Performing final disinfection | √ | √ |
| 29 | Securing the top of well with welded cover | √ | √ |
| 30 | Demobilizing all equipment from the site in February 2009 and leveling the ground surface | √ | √ |

As a part of the rehabilitation process, updated pumping tests were performed to determine whether well performance had been restored, to evaluate current well and aquifer parameters, and to update water quality data. In addition to water quality samples that were collected at the end of the constant rate pumping test, two zone isolation tests were conducted by inflating a packer at selected depths and pumping from below the packer. It was found that by inflating the packer at Well 21, the water level below the packer dropped significantly and flow rates from the isolated zones were quite low, contrary to what was found in Well 22. By inflating the packer at Well 22, the water level below the packer dropped significantly, but flow rates from the isolated zones were quite high.

During each pumping test, water quality samples were collected that were analyzed by E.S. Babcock & Sons of Riverside, California, and all water quality results are found in Appendix A. Contractor's notes for well rehabilitation redevelopment and testing are found in Appendix G and details of the pumping tests and zone isolation tests that were conducted following re-development are found in Appendix H. A number of photos were taken at the site during the rehabilitation process to document specific events and selected photos are found in Appendix I. The Department of Water Resources (DWR) Well Completion Report for Well 21 (completed in April 1992) is found in Appendix J. It should be noted that some of the details on the DWR report are different than the data contained in the 1992 CDM report. For the purpose of continuity, GEOSCIENCE has used the depths that were reported by CDM in 1992 for this report, as they appear to be consistent with recent video survey information.

The results of the rehabilitation and testing process showed the following:

<u>Well 21</u>:

- Well 21 was redeveloped to exceed its original capacity and with improved efficiency in spite of standing idle for more than 16 years. The design discharge rate following rehabilitation is 3,300 gpm with an expected 79 feet of drawdown after 1 year of continuous pumping, and with a well efficiency of 95 percent. Interference from Well 22 pumping at 1,600 gpm will be 16 feet after 1 year of continuous pumping.

- The November 2008 water quality results show that the nitrate concentration (as NO3) has increased since April 1992 when it was reported to be 43 mg/L and nearly the state's primary maximum contaminant level (MCL) of 45 mg/L to 67 mg/L (as NO3), now exceeding the primary MCL. Additionally, total dissolved solids have increased slightly in that length of time from 678 mg/L to 740 mg/L, exceeding the state's recommended level of 500 mg/L. Total hardness levels have increased from 450 mg/l to 500 mg/l.

- Perchlorate information is not available from the 1992 testing. However, in water quality samples collected at the end of the constant rate pumping test (on November 12, 2008), perchlorate was detected at 4.3 μg/L, which is less than CDPH's MCL of 6.0 μg/L for this contaminant. Perchlorate was not detected in water quality samples collected from either the Zone 1 or Zone 21 isolation tests indicating that it is coming from the shallow interval (298-509 ft bgs), and not from the deeper intervals.

<u>Well 22</u>:

- Well 22 was redeveloped to exceed original capacity in spite of standing idle for nearly 16 years. The design discharge rate for Well 22 is 1,600 gpm with 109 feet of drawdown expected after 1 year of continuous pumping at a well efficiency of 88 percent.

- Water quality results showed that nitrate concentrations (as NO3) have increased since May 1992 when it was reported to be 47 mg/L—which is slightly above the state's primary maximum contaminant level (MCL) of 45 mg/L, to 50 mg/L in January 2009. Additionally, total dissolved solids have increased slightly from 620 mg/L to 650 mg/L, remaining above the California Department of Public Health (CDPH) recommended level of 500 mg/L. Total hardness levels have increased from 381 mg/l to 430 mg/l. Perchlorate was not analyzed in 1992 and was not detected in January 2009 at the laboratory reportable detection limit (RDL) of 4.0 µg/L.

## 2.2 Rehabilitation and Redevelopment Procedures for Wells 21 and 22

### 2.2.1 Initial Brushing and Bailing of Wells 21 and 22

Work within Wells 21 and 22 was initiated using a cable tool rig that was set up with a nylon bristle brush attached to a weighted core and to the sand line. The cable tool rig was capable of allowing the bristle brush to move up and down in a coordinated manner as cable was slowly reeled off the spool to ensure thorough brushing of the interior of the casing and screen. The upper 20-inch diameter casing was brushed using a larger diameter brush than was the lower 16-inch portion of the well.

Each well was brushed for a total of 16 hours using 20 ½- and 16 ¼-inch diameter spirally wound nylon bristle brushes to dislodge loose scale and other bulk encrusting material from the wetted portions of the well casing and screen. Table 2-3 below summarizes the well characteristics for Wells 21 and 22 as depicted in Figures 2-1 and 2-2.

**Table 2-3 Well Characteristics**

| | Well 21 | Well 22 |
|---|---|---|
| **Screened Portion** | 298 to 509 ft bgs<br>539 to 749 ft bgs<br>819 to 869 ft bgs<br>930 to 1,060 ft bgs | 301 to 341 ft bgs<br>361 to 451 ft bgs<br>471 to 770 ft bgs<br>810 to 970 ft bgs |
| **Screen** | Type 304 stainless steel ful-flo horizontal louvers with 0.060 inch openings | Type 304 stainless steel ful-flo horizontal louvers with 0.060 inch openings |

Table 2-4 summarizes the brushing procedure performed at Well 21 (from September 29 and October 1, 2008) and Well 22 (October 3 to October 6, 2008).

**Table 2-4 Brushing and Bailing of Wells 21 and 22**

| | | Well 21 | Well 22 |
|---|---|---|---|
| No. of Passes [1] | | 4 Passes | 4 Passes |
| Accumulated Material after Brushing | | | |
| *Depth* | | *7 feet* | *24 feet* |
| *Removed on* | | *October 2, 2008* | *October 6 &7, 2008* |
| *Type* | | | |
| (1) | Very dark & rust-colored sediment containing small chips of rust and pipe scale | √ | √ |
| (2) | Large sections of a cast iron float plate as well as several large diameter washers and nuts [2] | √ | - |
| (3) | Large pieces of the tubercles | - | √ |
| (4) | Silt, fine sand and some Colorado silica gravel pack | √ | √ |
| Depth Reached with Bailer | | 1,080 feet | 990 feet |

[1] Passes were made in both the 16-inch and the 20-inch portions of the wells

[2] See Appendix G

### 2.2.2 Initial Redevelopment of Well 21 by Airlifting and Swabbing

A total of 72 hours of airlifting and swabbing of Well 21 screened intervals[1] was conducted from October 3 to October 17, 2008. The swabbing tool consisted of two 16-inch diameter rubber discs that were spaced 10 feet apart on a 6-inch diameter section of perforated steel pipe. This swabbing tool was installed in the well on threaded and coupled 4-inch diameter drill pipe that was moved vigorously up and down in the well using the cable tool rig to move the entire assembly in 20 foot strokes. Compressed air was injected into the 4-inch eductor pipe through a 1-inch diameter PVC air line, aerating the water column and causing the fluid to rise to the surface and carrying sediment with it. The aerated water was discharged into a 1,000 gal temporary storage tank before being pumped to the first of two 21,000 gallon "Baker" tanks. The Baker tanks allowed the sediment to settle before being discharged to the OCSD sewer system[2].

1 The screened intervals are located at 298-509 ft, 539-749 ft, 819- 869 ft and 930- 1,060 ft bgs.

2 All discharges during the airlifting and swabbing procedure were measured using a 4-inch McCrometer propeller-style flowmeter that was certified by McCall's Meters of Hemet, California on October 2, 2008.

The rubber discs of the swabbing tool isolated individual 10 foot intervals of the screen, and the up-and-down stroking action of the pipe served to agitate and loosen sediment in the screen and filter pack. Sediment that accumulated in the bottom of the well as a result of the swabbing and airlifting activities was removed using a bailer at the completion of the process. Two passes were made through the screens with the airlifting and swabbing apparatus in the 72 hours of airlift development time. Copies of BW&P daily notes that were prepared during brushing, airlifting and swabbing are found in Appendix H.

### 2.2.3 Mid-Rehabilitation Verification Video Survey of Well 21

Following completion of the airlift and swabbing portion of redevelopment, Pacific Surveys of Claremont, California, conducted a “dual-cam” video survey on October 20, 2008 to verify the effectiveness of initial redevelopment work. The video survey showed that the static water level was approximately 97 feet bgs and a large amount of suspended debris was suspended in the water column. The casing was observed to be in good condition with no visible pitting, however dark stains were observed on the casing and screen in areas where the tubercles had been removed. Most of these stains appeared to be streaming downward from the spiral weld and heat affected zone of the casing. Debris was observed lodged in the louvers that increased with depth, particularly from 930 to 1,060 feet bgs; however, the video log verified that overall the swabbing and airlifting procedure was effective.

Copies of the one page report generated by Pacific Surveys during each downhole video survey is included in Appendix E.

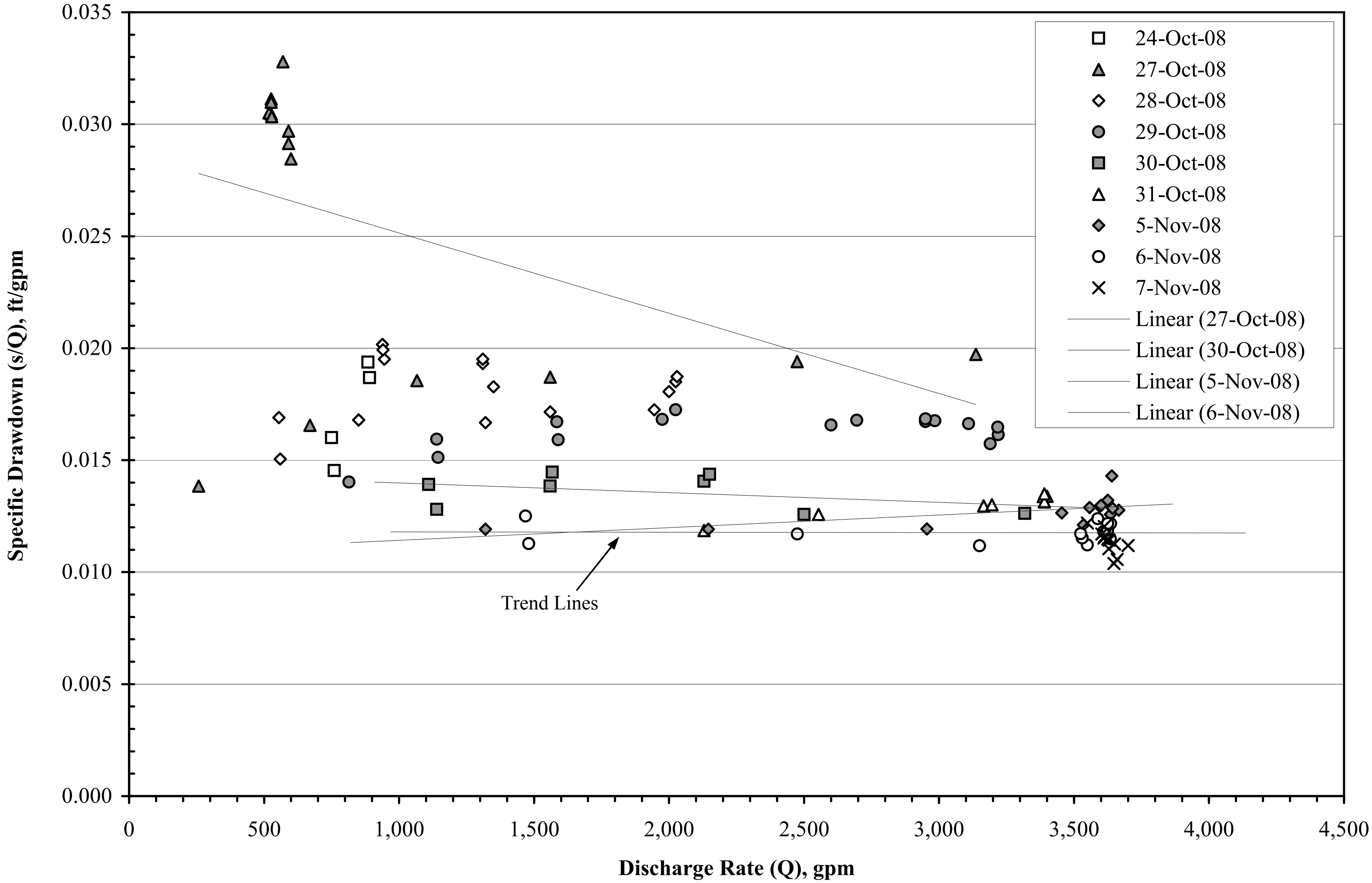
### 2.2.4 Final Redevelopment by Pumping of Well 21

Final redevelopment of Well 21 was conducted from October 24 to November 7, 2008 using a vertical turbine test pump. Development efforts initially consisted of pumping the well at low discharge rates (without surging), until sand concentrations declined to acceptable levels. The pumping rate was gradually increased until the maximum rate was achieved. Once the maximum discharge rate was reached, repeated cycles of pumping and surging (pulling the pump out of gear and allowing the water column to back-flush within the well) were performed until clear water with minimal sand was produced with a maximum specific capacity (i.e., the discharge rate divided by the drawdown). When the specific capacity approached a maximum, and the turbidity and sand concentration were minimal, well redevelopment was considered complete. Figure 2-3 shows the daily progress of final development by pumping. It should be noted that during the second day of development pumping, a significant amount of fine sand was observed in the discharge (over 1,200 parts per million (ppm)); however, with continuous pumping at a decreased discharge rate, the sand content was reduced to acceptable levels within a few hours.

Discharge rates were measured using a 12-inch in-line McCrometer propeller-style flowmeter that was certified for accuracy. On October 27, 2008 the meter was certified as installed onsite by McCall's Meters to measure accurately to within 5 percent of the indicated amount at each of three flow rates spread across the range of the meter. The flowmeter was linked to an electronic circular chart recording device that monitored instantaneous flow rates. Additionally, totalizer readings were recorded frequently by BW&P personnel throughout each day of pumping. During redevelopment and testing, water level measurements from the well were collected using an electric sounder, while sand concentrations were measured using a centrifugal Rossum Sand Tester.

This page left intentionally blank

**Figure 2-3: Specific Drawdown Plot during Development**
**IRWD Well No. 21**




GEOSCIENCE Support Services, Inc.

During redevelopment pumping, a maximum short-term discharge rate of 3,700 gallons per minute (gpm) was achieved with 35 feet of drawdown. The short-term specific capacity on the final day of redevelopment was 83 gpm per foot of drawdown at 3,612 gpm. The sand content within the discharge was monitored throughout redevelopment and ranged from trace amounts to 10 ppm, with the exception of the one incident where the sand content increased to a maximum of 1,267 ppm. The high sand concentrations were observed only in the beginning stages of redevelopment and quickly declined after several hours of continuous pumping. Because of the very high sand content that was experienced, development by pumping proceeded cautiously. Copies of BW&P's development by pumping notes are included in Appendix G.

### 2.2.5 Initial Redevelopment of Well 22 by Airlifting and Swabbing

On October 21, 2008, BW&P injected a total of 40 gallons of Baroid Aqua-Clear PFD through the swabbing tool throughout the screened interval from 301 to 970 feet bgs. Aqua-Clear PFD is a polymer dispersant that thins bentonite drilling mud and clay to makes them more fluid and therefore more easily removed from the filter pack and near-well zone. From October 24 to November 13, 2008, airlifting and swabbing of the screened intervals[3] was completed. After two passes through the screen, the lower portion from 620-760 feet bgs remained very muddy with a large quantity of what appeared to be bentonite clay. GEOSCIENCE recommended an additional treatment with Aqua-Clear followed by more airlifting and swabbing in the interval that appeared to have the greatest amount of clay. Subsequently, an additional 41 hours were added to the time spent in airlifting and swabbing the lower portions of the well screen for a total of 113 hours and a total of 50 gallon of Aqua-Clear PFD.

Similar to the airlifting and swabbing procedure performed at Well 21, the swabbing tool was installed in Well 22 on threaded and coupled 4-inch diameter drill pipe that was moved vigorously up and down in the well using a cable tool drilling rig. Compressed air was injected into the 4-inch pipe through a 1-inch diameter PVC air line, aerating the water column to draw in fluid from the 10-foot perforated interval located between the two rubber discs and cause it to rise to the surface. The aerated column of

[3] The screened intervals in Well 22 are located at 301-341 ft, 361-451 ft, 471-770 ft and 810-970 ft bgs.

water then carried high turbidity fluid and sediment to the surface, and was discharged into a nearby 1,000 gal temporary storage tank. From the temporary holding tank, the fluid was pumped to the first of two 21,000 gallon Baker tank to allow sediment to settle out before being discharged to the OCSD sewer system.

The fill generated in Well 22 by the swabbing and airlifting activities was removed from the bottom of the well using a bailer at the completion of the process. Three passes were made through the Well 22 screens with the airlifting and swabbing apparatus in the 141 hours of airlift development time. Copies of BW&P daily notes that were made during brushing, airlifting, and swabbing are found in Appendix G.

### 2.2.6 Mid-Rehabilitation Verification Video Survey of Well 22

Following the airlift and swabbing procedure at Well 22, Pacific Surveys conducted a "dual-cam" video survey on November 17, 2008 to verify the effectiveness of initial redevelopment work. The video survey showed that the static water level was approximately 100 feet bgs and a large amount of suspended debris was observed in the water column above the top of the screened interval at 301 feet, but cleared once the screen was entered.

The 20-inch casing from the static water level to the stainless steel adaptor at 296 feet bgs has a significant amount of debris adhering to it and there were remains of some tubercle deposits within the uppermost screen from 301-341 feet bgs. Otherwise, the casing was observed to be in good condition with no visible pitting; however staining was observed on the casing and screen in locations where the tubercles had been removed. Most of these stains appeared to be streaming downward from the spiral weld of the casing and the associated heat affected zone. Light colored feathery debris was observed on the louvers that increased with depth, particularly below 460 feet bgs. Copies of the one page reports generated by Pacific Surveys during each downhole video survey are included in Appendix E.

Based on the observations made during the video survey, GEOSCIENCE recommended additional time be spent in brushing the upper section of the well (both 16- and 20-inch diameter) before using a bailer to "fan" the lower screened interval in an attempt to dislodge additional loose debris. On November 19, 2008, BW&P personnel returned to the well site with the cable tool rig and on November 20-21, 2008 completed additional brushing and bailing work.

### 2.2.7 Final Redevelopment of Well 22 by Pumping

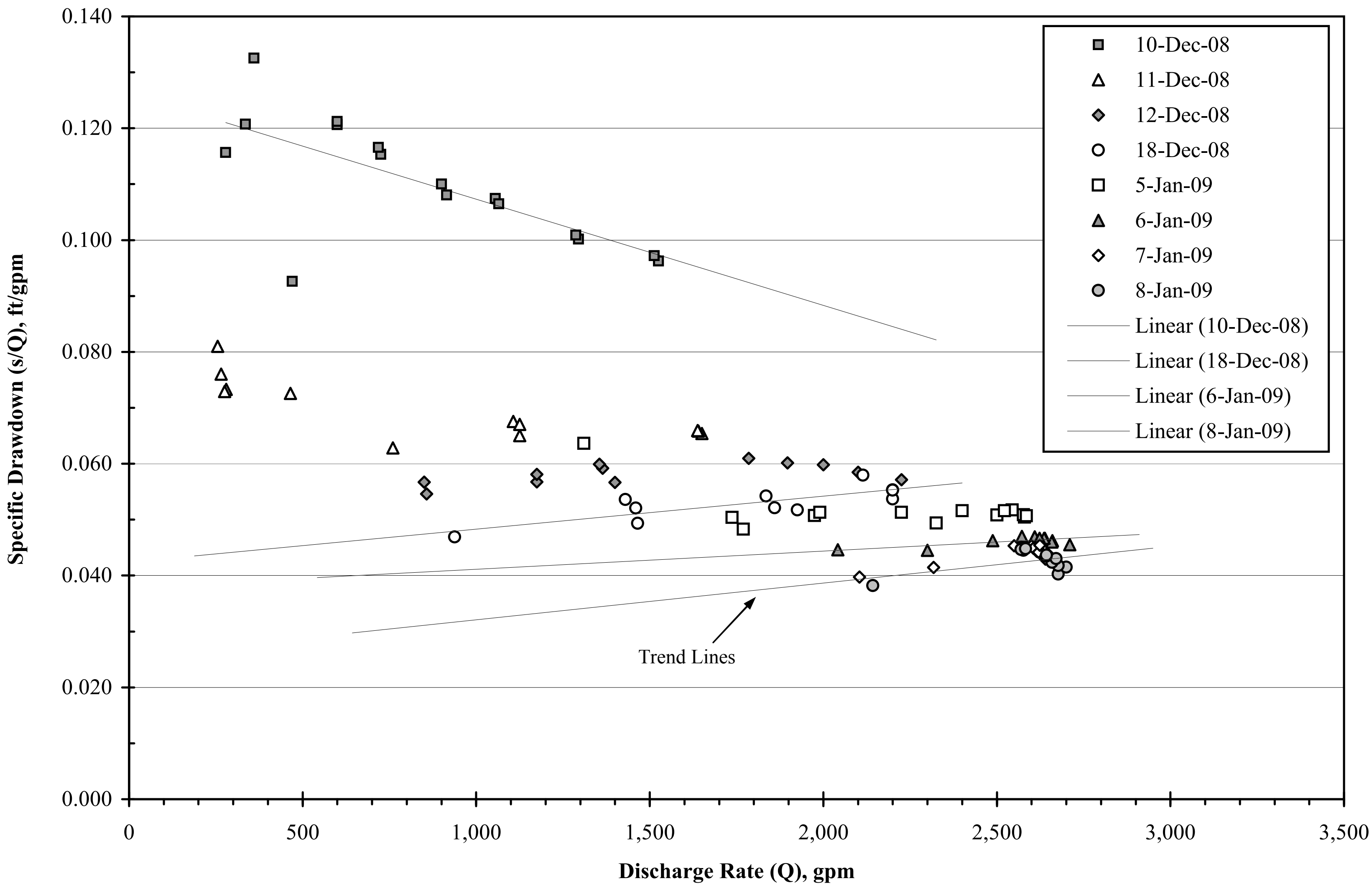
The test pump and packer were installed in Well 22 from December 4 to December 9, 2008. Final redevelopment for Well 22 was conducted using a vertical turbine test pump from December 10, 2008 to January 8, 2009. Some delays to the work during this time were caused by heavy rainfall that precluded discharge to the OCSD sewer system and because of the Christmas holiday.

Development efforts initially consisted of pumping the well at low discharge rates (without surging) until sand concentrations declined to acceptable levels. The well was then pumped at gradually increasing rates until the maximum discharge rate was achieved. Once the maximum discharge rate was reached, repeated cycles of pumping and surging were performed until clean water with minimal sand was being produced, and maximum specific capacity was achieved. When the specific capacity (i.e., the discharge rate divided by the drawdown) approached a maximum, and the turbidity and sand concentration remained constant, well rehabilitation was considered complete. Figure 2-4 shows the daily progress of development by pumping conducted by BW&P.

Discharge rates were measured using a 12-inch in-line McCrometer propeller-style flowmeter that was certified for accuracy. On December 11, 2008, the meter was certified as installed onsite by McCall's Meters to measure accurately to within 5 percent of the indicated amount at each of three flow rates spread across the range of the meter. The flowmeter was linked to an electronic circular chart recording device that monitored instantaneous flow rates. Additionally, totalizer readings were recorded frequently by BW&P personnel throughout each day of pumping. During redevelopment and testing, water level measurements from the well were collected using an electric sounder, while sand concentrations were measured using a centrifugal Rossum Sand Tester.

This page left intentionally blank

**Figure 2-4: IRWD Well No. 22 Development**
**Specific Drawdown Plot**




GEOSCIENCE Support Services, Inc.

During redevelopment pumping, a maximum discharge rate of 2,710 gallons per minute (gpm) was achieved with 155 feet of drawdown. The short-term specific capacity on the final day of redevelopment was 23 gpm per foot of drawdown at 2,613 gpm. The sand content within the discharge was monitored throughout redevelopment and ranged from approximately trace amounts to 105 ppm. Copies of BW&P's development by pumping notes are included in Appendix G.

## 2.3 Pumping Test Procedures

### 2.3.1 Basic Assumptions Used in Analysis of Pumping Test Data

The purpose of a pumping test is to obtain field data, which when substituted into an equation or set of equations, will yield estimates of well and aquifer properties. Certain assumptions have been used to derive these equations and it is important to consider or control these factors during testing. These assumptions are:

- The aquifer material is assumed to consist of porous media, with flow velocities being laminar and obeying Darcy's law.
- The aquifer is considered to be homogeneous, isotropic, of infinite aerial extent, and of constant thickness throughout.
- Water is released from (or added to) internal aquifer storage instantaneously upon change in water level.
- No storage occurs in the semi-confining layers of leaky aquifers.
- The storage in the well is negligible.
- The pumping well penetrates the entire aquifer and receives water from the entire thickness by horizontal flow.
- The slope of the water table or piezometric surface is assumed to be flat during the test with no natural (or other) recharge occurring, which would affect test results.
- The pumping rate is assumed constant during the entire time period of pumping during a constant-rate test, and constant during each discharge step in a variable-rate test.

### 2.3.2 Pumping Test Data Analysis Methods

#### 2.3.2.1 Step Drawdown Test Method

Step drawdown testing is used to determine formation losses, well losses, and well efficiency; all necessary in determining the final pump design. In an actively pumping well, the total drawdown in the well is composed of both laminar and turbulent head loss components. Laminar losses generally occur away from the borehole (where approach velocities are low), while turbulent losses are confined to the area in and around the immediate vicinity of the well screen and within the well bore. The total drawdown in a pumping well may be expressed as:

$$s_w = BQ + CQ^2 \quad \text{“Drawdown in a Pumping Well”} \qquad (1)$$

where:
$s_w$ = Total drawdown measured in the well, [ft]
B = Formation or aquifer loss coefficient, [ft/gpm]
Q = Discharge rate of the well, [gpm]
C = Well loss coefficient, [ft/gpm$^2$]

The first and second terms in equation (1) are referred to as formation, or aquifer loss[4] (BQ) and well loss[5] ($CQ^2$), respectively. Formation (i.e., aquifer) loss and well loss coefficients are determined from the step drawdown test. The test procedure involves pumping the well at multiple (at least three) discharge rates with each "step" being a fraction of the maximum discharge. Analysis of the step drawdown data requires plotting the "specific drawdown" ($s_w$/Q) for each step against discharge rate. The formation loss coefficient (B) is the y-intercept of the best-fit straight line through the specific drawdown data points. The slope of the line is equal to the well loss coefficient (C).

4 Aquifer loss is the head loss measured at the interface between the aquifer and the filter pack. The magnitude of the aquifer loss can be found from consideration of radial flow into the well and can be calculated, for example, using Jacob's equation.

5 Well losses are turbulent flow losses which are head losses associated with the entrance of water into and through the well screen including those losses incurred as the flow moves axially towards the pump intake. These losses vary as the square of the velocity.

Well Efficiency (E) is defined as the ratio of the formation (i.e. aquifer) loss component (BQ) to the total drawdown measured in the well ($s_w$) and is expressed as a percent:

$$E = 100\frac{BQ}{s_w} = \frac{100}{1 + CQ/B}$$ "Well Efficiency" (2)

where:
E = Well Efficiency, [%]
B = Formation or aquifer loss coefficient, [ft/gpm]
Q = Discharge rate of the well, [gpm]
C = Well loss coefficient, [ft/gpm$^2$]

2.3.2.2 Constant Rate Test Methods

Calculation of aquifer parameters from pumping test data is based on analytical solutions of the basic differential equation of ground water flow that can be derived from fundamental laws of physics. One of the most widely used solutions of this equation for non-steady radial flow to well is the "Theis Equation":

$$s(r,t) = \frac{114.6Q}{T}W(u)$$ "Theis Equation" (3)

where:
s(r,t) = Drawdown in the vicinity of an artesian well, [ft]
r = Distance from pumping well, [ft]
Q = Discharge rate of pumping well, [gpm]
T = Transmissivity of aquifer, [gpd/ft]
W(u) = "Well function of Theis"
u = 1.87 x $r^2$ x S / (T x t)

where:
S = Storativity, [fraction]
t = Time after pumping started, [days]

2.3.2.3 Jacob's Straight-Line (Modified Theis Non-Equilibrium) Method

According to Jacob (1950), for small values of "u" (u < 0.05), the Theis equation may be approximated by Jacob's equation:

$$s(r,t) = \frac{264Q}{T}\log\left(\frac{0.3\,Tt}{r^2\,S}\right)$$ "Jacob's Equation" (4)

Jacob's equation is valid for use for most hydrogeologic problems of practical interest, is easier to use than the Theis equation, and involves a simple graphical procedure to calculate transmissivity and storativity. This method (D 4105) is summarized by ASTM (1994).

Transmissivity (T, in gpd/ft) can be calculated as:

$$T = \frac{264Q}{\Delta s} \qquad (5)$$

where:
Q = Pumping rate, [gpm]
$\Delta s$ = Change in drawdown over one log cycle of time, [ft]

Storativity can be calculated as:

$$S = \frac{0.3Tt_0}{r^2} \qquad (6)$$

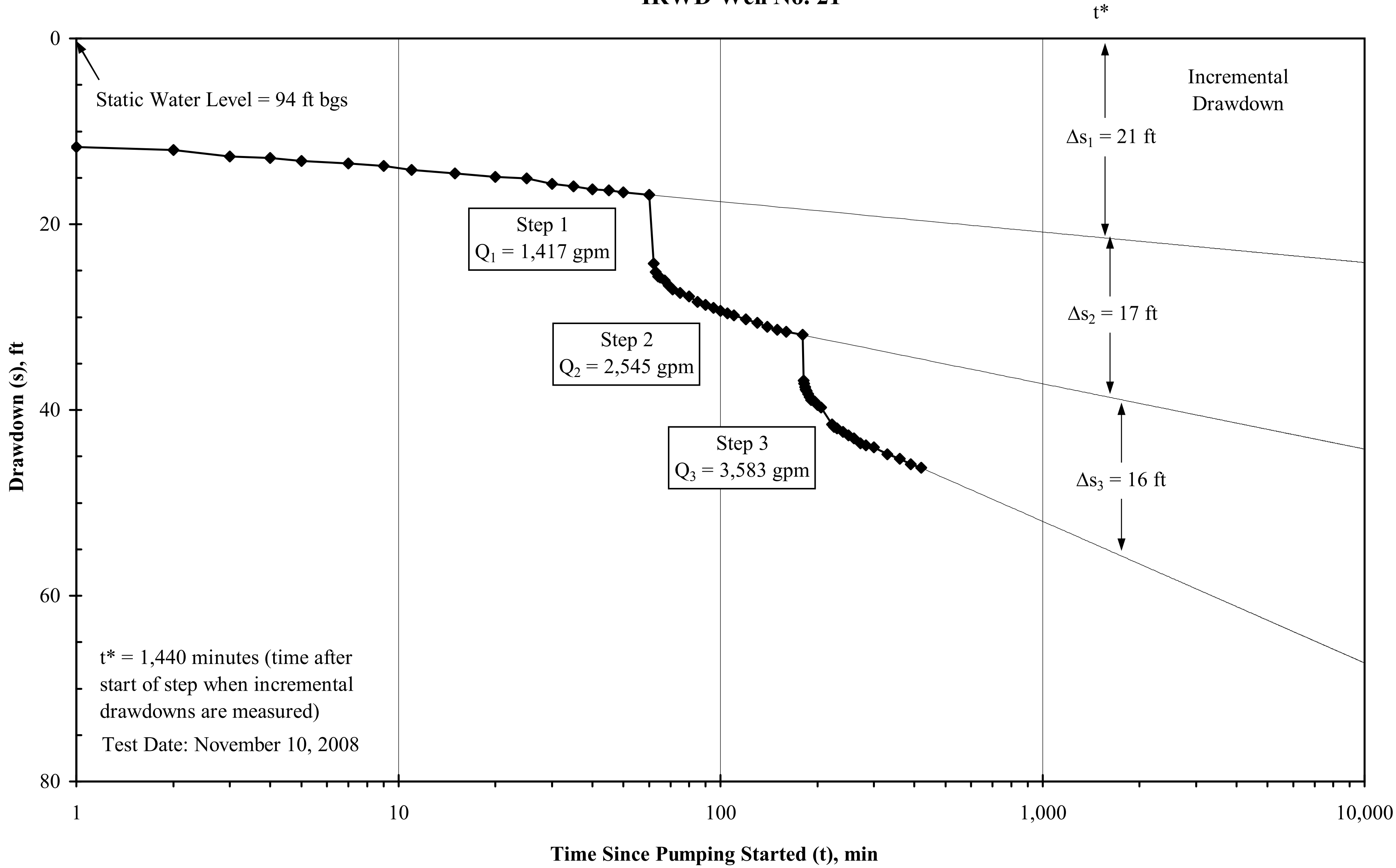
where:
T = Transmissivity, [gpd/ft]
$t_0$ = Time at the zero-drawdown intercept, [days]
r = Radial distance from the pumping well, [ft]

## 2.4 Well 21 Pumping Test Data, Analysis, and Results

### 2.4.1 Well 21 Step Drawdown Test

A step drawdown test was conducted on November 10, 2008 at discharge rates of 1,417 gpm, 2,545 gpm, and 3,583 gpm with incremental drawdowns of 21, 17 and 16 gpm/feet respectively. The static water level at the beginning of the test was approximately 94 feet bgs. Figure 2-5 is a plot of the step drawdown test data and shows the time-drawdown curve for each step. The specific drawdown for each step is shown in Table 2-5. It should be noted that the values shown on this table are extrapolated to reflect the value at 1,440 minutes after the start of each step, and are not the values at the end of each step.

## Figure 2-5: Step Drawdown Pumping Test
## IRWD Well No. 21




GEOSCIENCE Support Services, Inc.

**Table 2-5 Summary of Step Drawdown Test Data for Well 21 – Nov. 2008**

| Step m | Discharge Rate $Q_m$ (gpm) | Incremental Drawdown[1] $\Delta_m$ (ft) | Cumulative Drawdown [1] $s_m$ (ft) | Specific Drawdown $(s/Q)_m$ (ft/gpm) |
|---|---|---|---|---|
| 1 | 1,417 | 21 | 21 | 0.0148 |
| 2 | 2,545 | 17 | 38 | 0.0149 |
| 3 | 3,583 | 16 | 54 | 0.0151 |

[1] Extrapolated to 1,440 minutes after the start of each step.

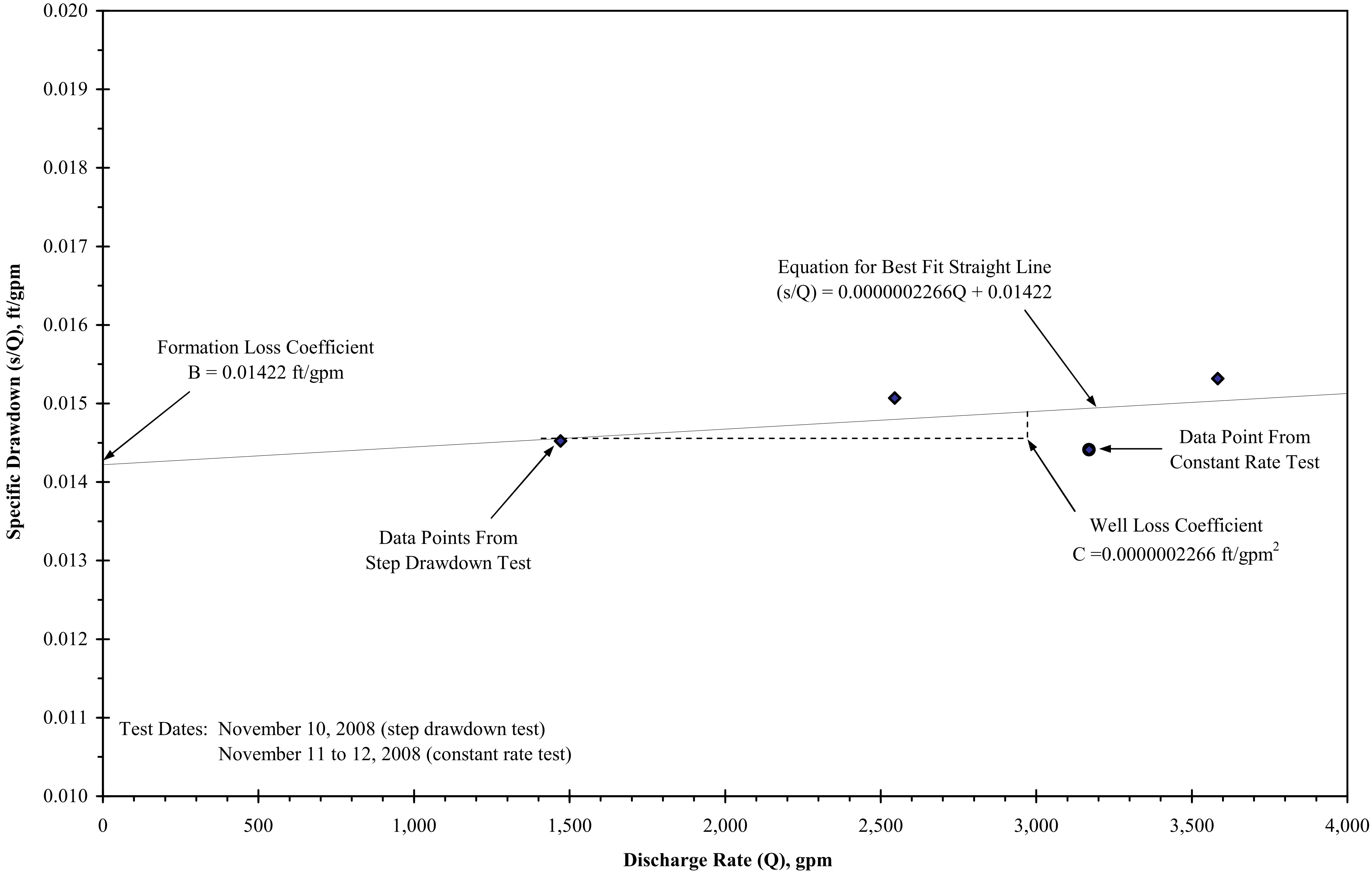
The specific drawdown plot (Figure 2-6) shows the relationship between specific drawdown (s/Q) and the discharge rate (Q). The testing showed a formation loss coefficient (B) of 0.01422 ft/gpm and a well loss coefficient (C) of 0.0000002266 ft/gpm$^2$.

The specific capacity and efficiency diagram (Figure 2-7) shows the estimated drawdown and well efficiency for Well 21. Estimated drawdown and well efficiency may also be calculated from equations (1) and (2) discussed in Section 2.3.2.1 (Step Drawdown Test Method). Based on a design discharge rate of 3,300 gpm the short-term drawdown is expected to be 49 feet and long-term drawdown is expected to be 79 feet with a well efficiency of 95 percent.

The performance of Well 21 was improved over its original performance in 1992 through the recent rehabilitation process. This can be seen by comparing specific drawdown values obtained from the 1992 post-construction step drawdown tests to the values obtained in the 2008 post-rehabilitation step drawdown test. Table 2-6 and Figure 2-8 compare the post-rehabilitation step test data to the post-construction data.

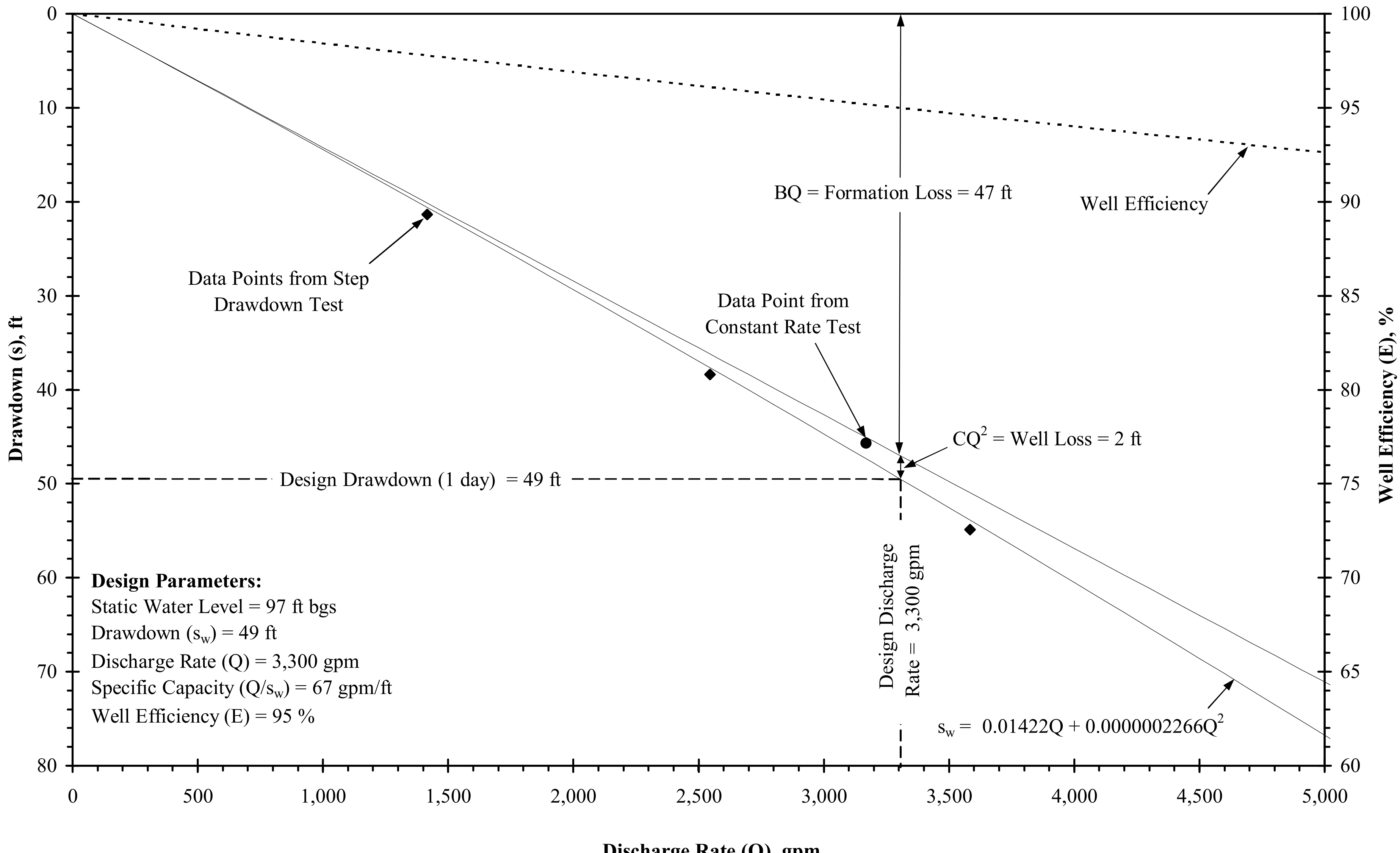
This page left intentionally blank

**Figure 2-6: Specific Drawdown Chart**
**IRWD Well No. 21**



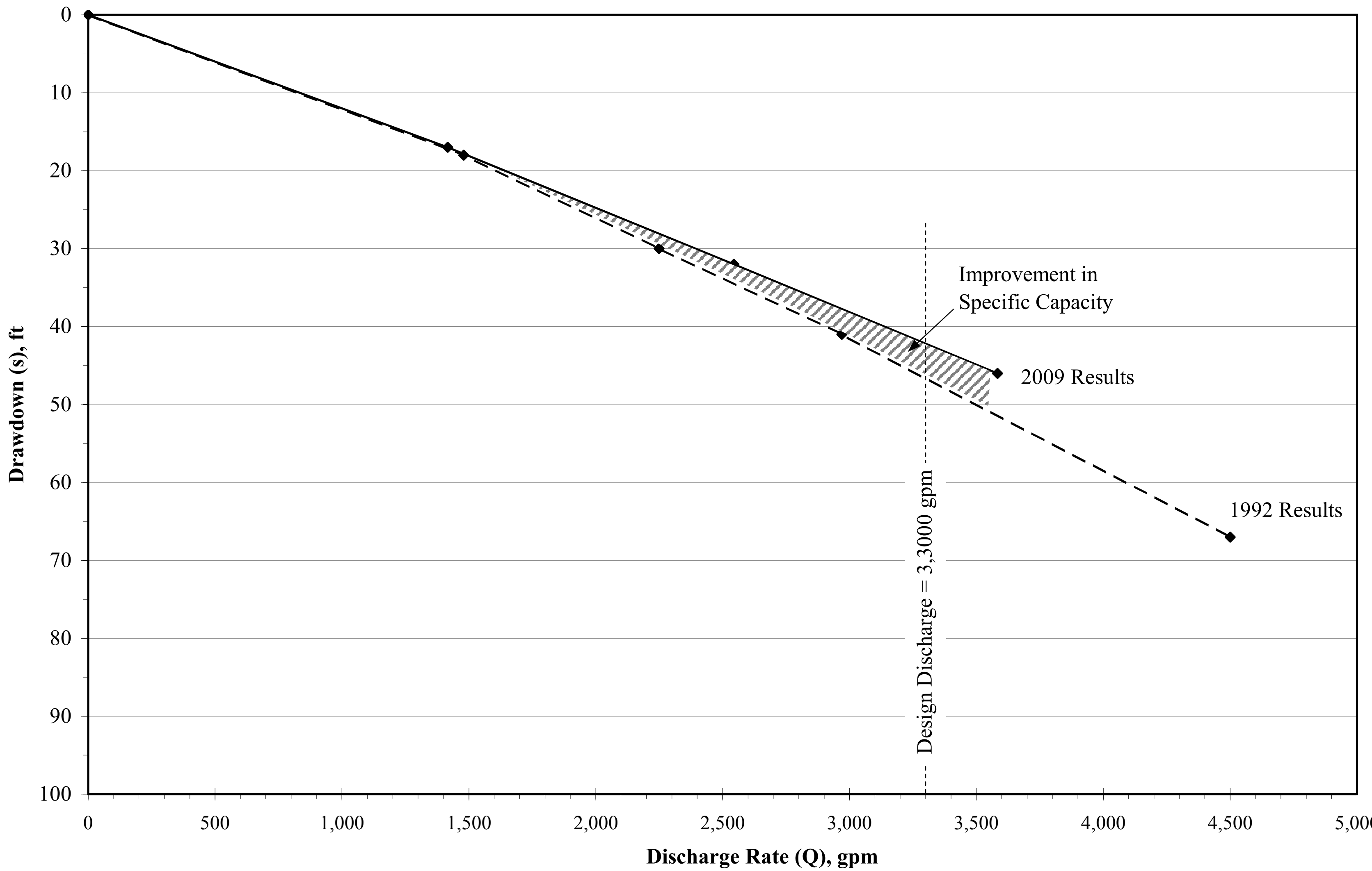

**GEOSCIENCE Support Services, Inc.**

**Figure 2-7: Specific Capacity and Well Efficiency Diagram**
**IRWD Well No. 21**




**GEOSCIENCE Support Services, Inc.**

**Figure 2-8: Improved Specific Capacity**
**IRWD Well No. 21**




**GEOSCIENCE Support Services, Inc.**

**Table 2-6 Comparison of 1992 Post-Construction and 2008 Post-Rehabilitation Step Drawdown Test Data – IRWD Well 21**

| Step m | April 1992 Discharge Rate $Q_m$ [gpm] | April 1992 Drawdown at End of Step $s_m$ [ft] | April 1992 Specific Capacity $(Q/s)_m$ [gpm/ft] | November 2008 Discharge Rate $Q_m$ [gpm] | November 2008 Drawdown at End of Step $s_m$ [ft] | November 2008 Specific Capacity $(Q/s)_m$ [gpm/ft] |
|---|---|---|---|---|---|---|
| 1 | 1,480 | 18.23 | 81.2 | 1,417 | 16.83 | 84.2 |
| 2 | 2,250 | 29.86 | 75.4 | 2,545 | 31.93 | 79.7 |
| 3 | 2,970 | 41.46 | 71.6 | 3,583 | 46.23 | 77.5 |
| 4 | 4,500 | 67.2 | 66.9 | - | - | - |

As shown on the Table 2-6 above, the specific capacity at the end of each step is higher following the recent rehabilitation than was originally measured following well construction in 1992. Additionally, it should be noted that with the exception of Step 1, each step conducted in November 2008 was at a higher rate of discharge than its corresponding April 1992 rate, and with increased specific capacity in spite of the higher flow. In a fully developed well, the specific capacity will decrease as the discharge rate is increased.

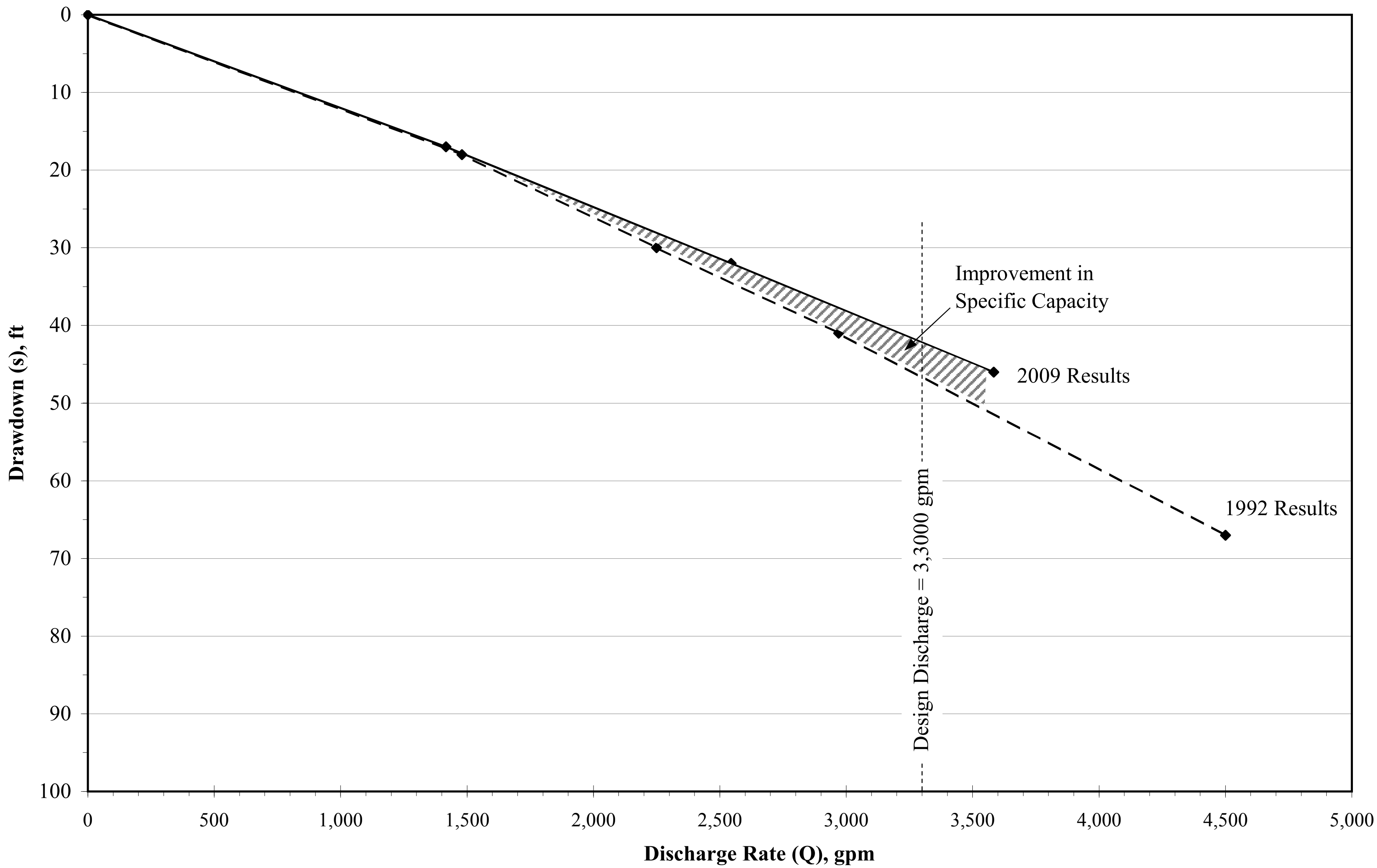
### 2.4.2 24-Hour Constant Rate Test for Well 21

Following recovery from the step drawdown test, a 24-hour constant rate pumping test was conducted on November 11 and 12, 2008 (see Figure 2-9). The static water level at the start of the test was approximately 97 feet bgs. At the average discharge rate of 3,170 gpm, the resulting water level drawdown was 9 foot per log cycle. By using the Jacob's straight-line method to analyze the time-drawdown data, results show a transmissivity of approximately 93,000 gpd/feet (see Figure 2-9).

During the 24-hour constant rate test, water level measurements were taken at Well 22 (Figure 2-10), which served as an observation well as it is located approximately 684 feet to the southeast of Well 21. The static water level within the observation well was approximately 102 feet bgs at the start of the test. A water level drawdown of 10 feet per log cycle was measured in Well 22 while pumping Well 21 at a discharge rate of 3,170 gpm. By analyzing the time-drawdown data using the Jacob's straight-line method, transmissivity was estimated at approximately 84,000 gpd/feet (Figure 2-10). At the end of the constant rate pumping test, water quality samples were collected and delivered to E.S. Babcock & Sons, Inc. for analysis (Appendix A).

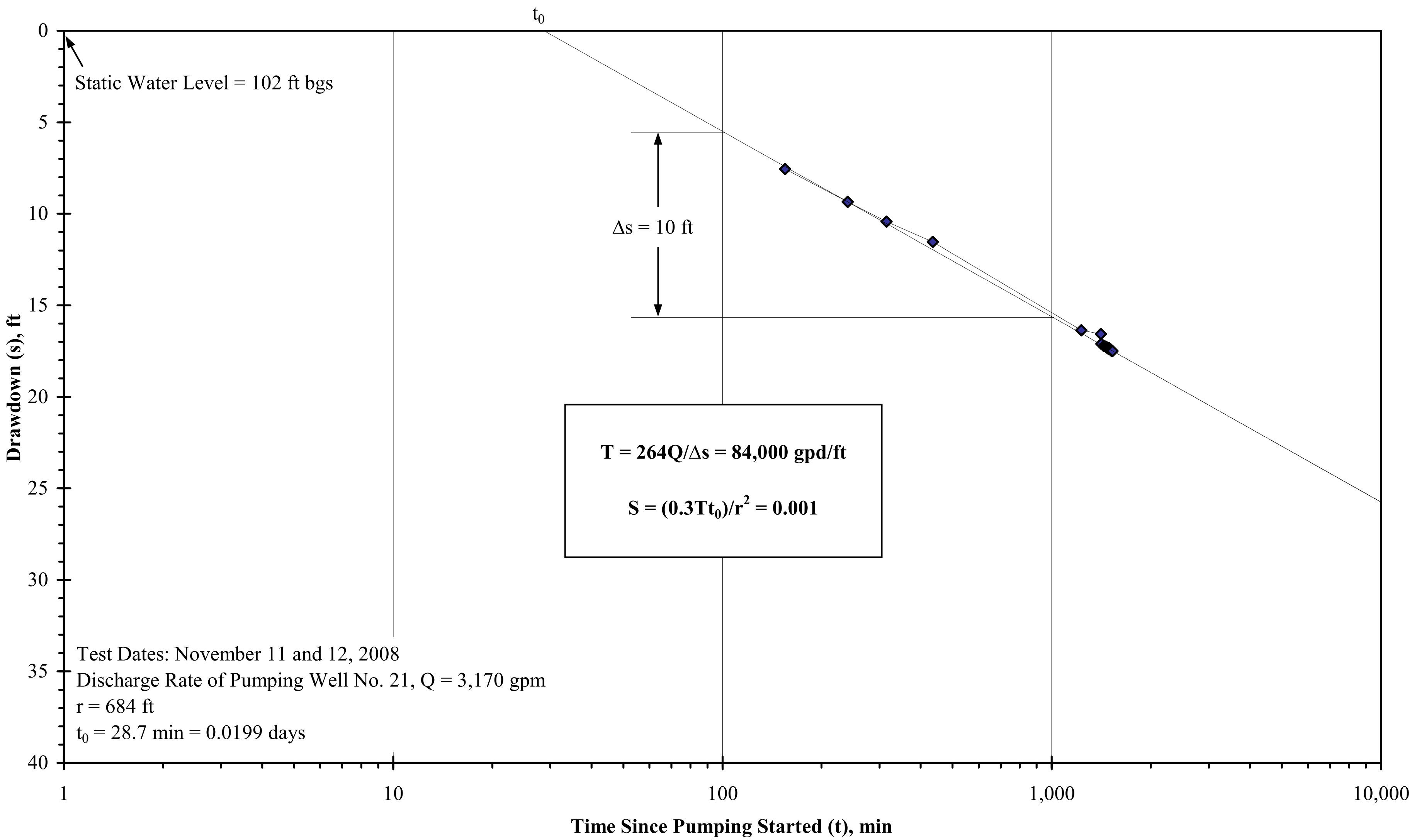
This page left intentionally blank

## Figure 2-8: Improved Specific Capacity
## IRWD Well No. 21




GEOSCIENCE Support Services, Inc.

**Figure 2-10: Observation Well Data**
**IRWD Well No. 22 (IRWD Well No. 21 Pumping)**



In addition, a silt density index (SDI) test was conducted in the field prior to shutting down the pump. Test results indicated an SDI value of 1.1 at Well 21.

### 2.4.3 Well 21 Zone Isolation Pumping Tests

Because of the potential for treatment, two zone isolation tests were planned to determine whether there were any changes in the water quality within the aquifers that were screened. The Zone 1 isolation test was conducted on November 13, 2008 following recovery from the constant rate pumping test. The Zone 2 isolation test was conducted on December 1, 2008. The delay between the two tests was due to failure of the packer to inflate after the test pump had been lowered to the second zone, requiring removal and replacement of the packer and then reinstallation of the test pump.

#### 2.4.3.1 Well 21 Zone 1 Isolation Pumping Test

The Zone 1 isolation test was conducted with the inflatable packer installed from 526 to 528 feet bgs (see Figures 2-11 and 2-12), and with the pump intake located at approximately 538 feet bgs. The static water level in the well was approximately 100 feet bgs prior to starting the test.

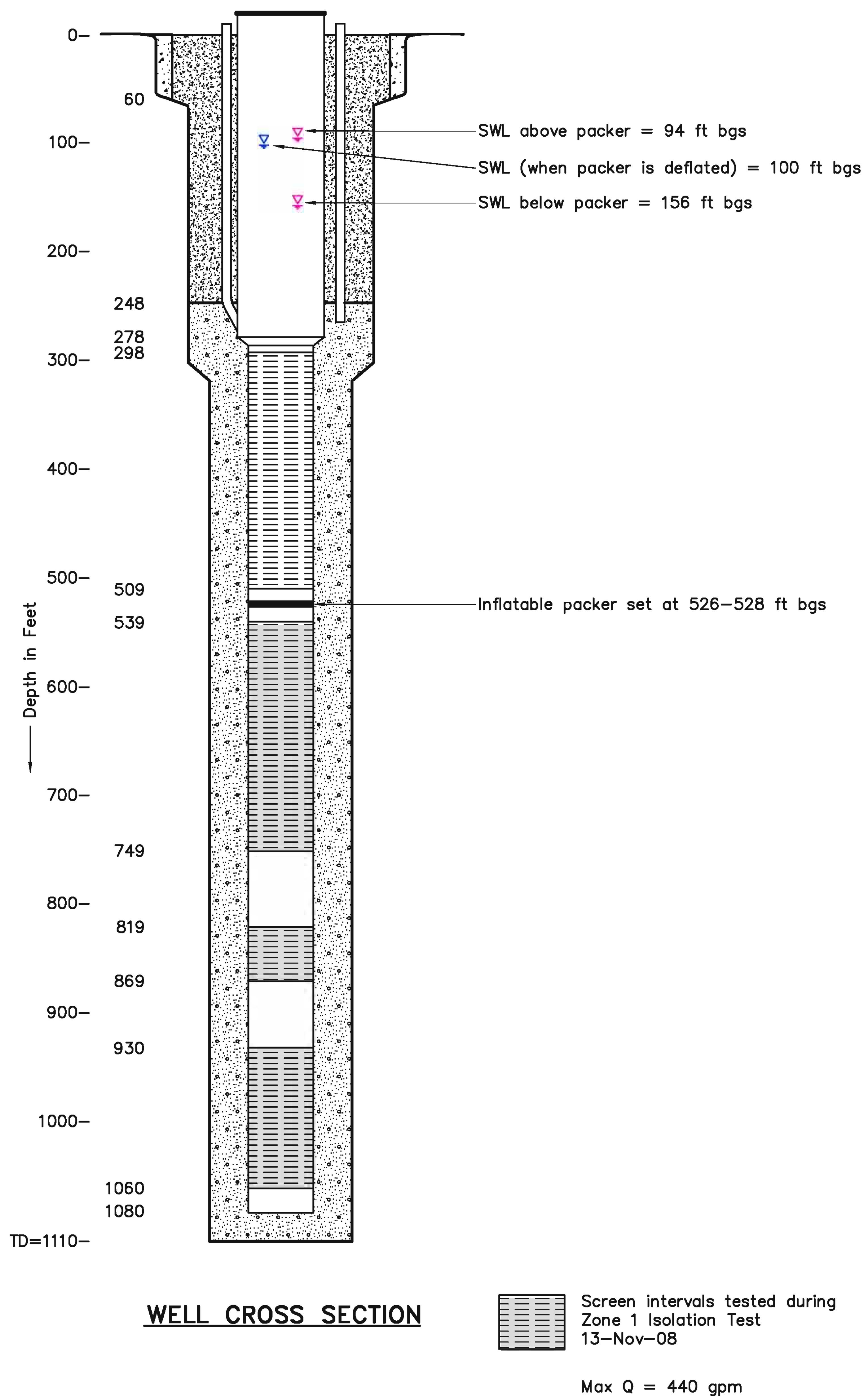
The Zone 1 isolation test consisted of inflating the packer to 240 pounds per square inch (psi), which is approximately 55 psi greater than background pressure[6], in order to test water quality and amount of contribution from the screened intervals located at 539-749 feet, 819-869 feet and 930-1,060 feet bgs (totaling 390 feet). The packer removed the uppermost screen (located at 298-509 feet bgs) from production during the test.

After inflating the packer to the required pressure, water levels were allowed to stabilize. The static water level below the packer fell to approximately 156 feet bgs, while the water level above the packer was elevated slightly to 94 feet bgs (see Figure 2-12). The zone was initially pumped at a low rate averaging approximately 225 gpm that was gradually increased to 440 gpm as the discharge cleared and the zone

[6] Background pressure is calculated by subtracting the static water level from the packer depth divided by 2.31 ft/psi, or (528-100) ÷2.31=185 psi. The packer pressure was increased by 55 psi for proper sealing.

developed. Water levels and discharge rates were monitored frequently during the testing. At the maximum discharge rate of 440 gpm, the drawdown was 53 feet with a resulting specific capacity of 8 gpm/ft.

After 6 hours of continuous pumping, water quality samples were collected for analysis of general mineral and physical properties, metals (i.e., inorganics), VOCs, perchlorate and total organic carbon (TOC) before the pump was turned off and recovering water levels were monitored. Once the zone had fully recovered, the packer was deflated and water levels returned to their pre-inflation levels. At the time of sample collection the field turbidity of the water was 0.68 nephelometric turbidity units (NTU). The complete results of the Zone 1 water quality analyses are shown in Appendix A.



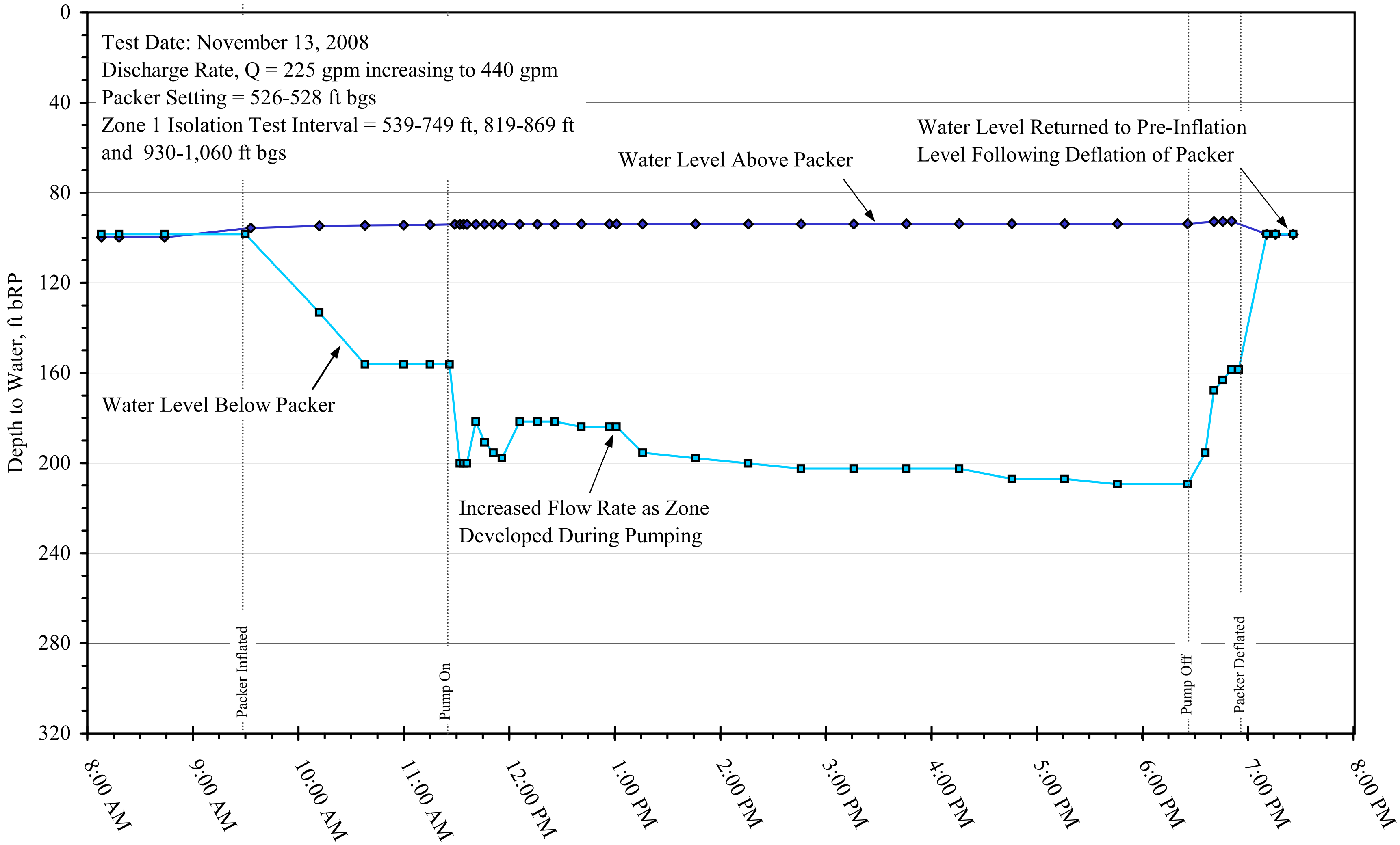

WELL CROSS SECTION

| | | | |
|---|---|---|---|
| Figure 2–11 | Drawn: PLP | RBF CONSULTING / IRVINE RANCH WATER DISTRICT | GEOSCIENCE |
| | Checked: | ZONE 1 ISOLATION TEST | GEOSCIENCE Support Services, Incorporated P.O. Box 220, Claremont, CA 91711 |
| | Approved: | IRWD WELL NO. 21 | Tel: (909)451-6650 Fax: (909)451-6638 |
| | Date: 18–MAR–09 | | www.gssiwater.com |

rbf\irwd\wells 21_22 rehab\rehab report well 21\figures\well 21 zone 1.dwg

This page left intentionally blank.

**Figure 2-12: Isolated Aquifer Zone Testing Timeline Analysis**
**Zone 1 Isolation Test - IRWD Well No. 21**




GEOSCIENCE Support Services, Inc.

### 2.4.3.2 Well 21 Zone 2 Isolation Pumping Test

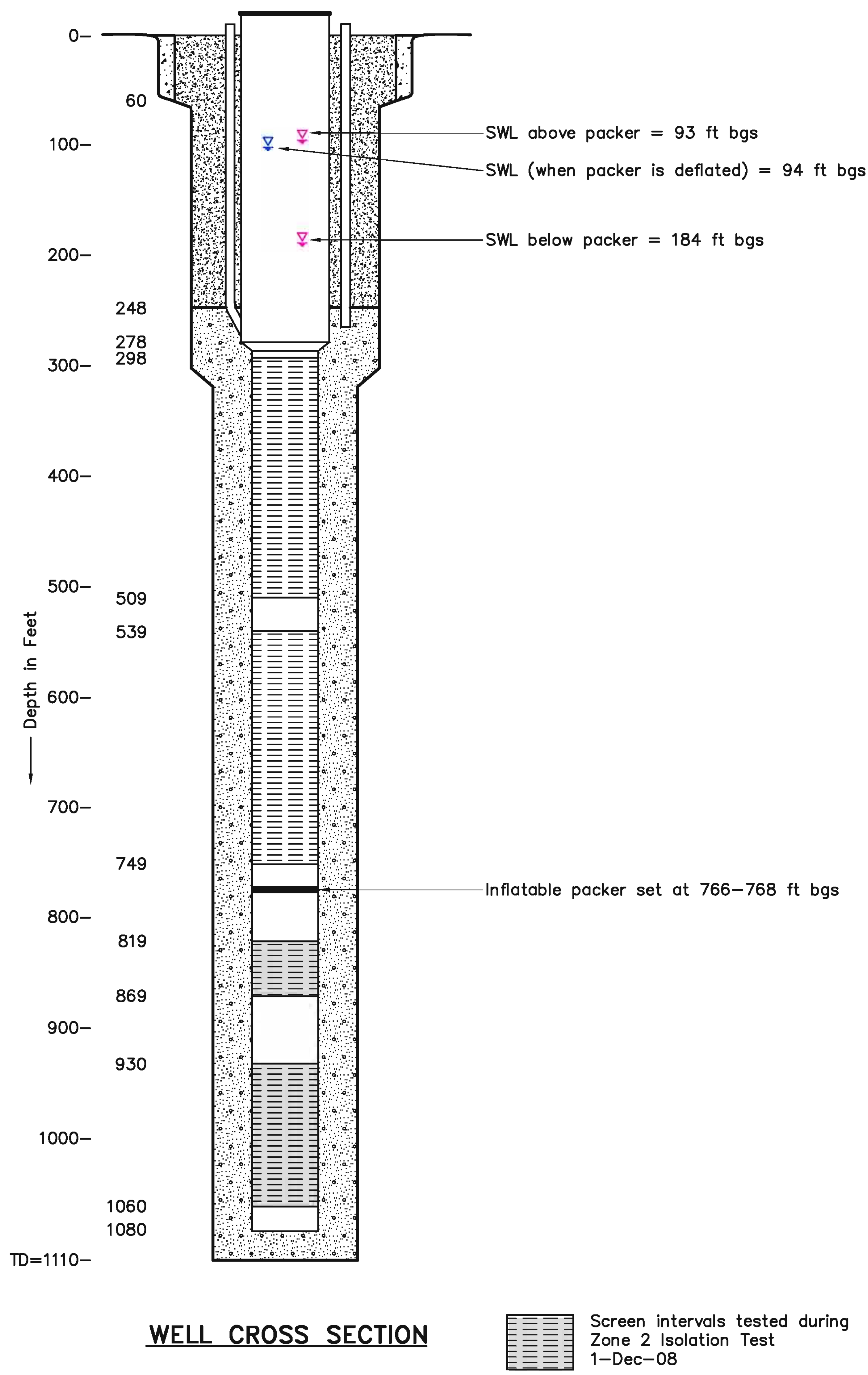
From November 14 to November 18, 2008, the packer and test pump were lowered within the well to 766-768 feet bgs. On November 19, 2008, the packer failed to inflate requiring removal of the entire test pump and packer assembly from the well. A new packer and the test pump was subsequently installed to the same depth on November 26, 2008. Due to laboratory scheduling constraints and the potential to exceed the shorter holding times if a sample were taken past mid-day, the site was secured for the Thanksgiving holiday.

The Zone 2 isolation test was conducted December 1, 2008 with the inflatable packer installed at 766-768 feet bgs and with the pump intake located at 778 feet bgs (see Figures 2-13 and 2-14). The static water level in the well was approximately 94 feet prior to starting the test. Testing consisted of inflating the packer to 355 psi[7], at least 60 psi above the background pressure. The screen intervals tested were 819-869 feet and 930-1,060 feet bgs (totaling 180 feet), removing the uppermost screens (298-509 feet and 539-749 feet bgs) from production.

Immediately following inflation of the packer, the water level in the zone beneath the packer dropped nearly 90 feet before stabilizing at 184 feet, while the water level above the packer did not change significantly. The initial discharge rate of approximately 90 gpm was increased to a maximum of 236 gpm as the zone developed. Initially, the discharge contained a large amount of suspended fine sand, silt and clay that cleared as pumping continued. After 6 hours of continuous pumping, the drawdown at the maximum discharge rate was more than 92 feet, resulting in a specific capacity of less than 3 gpm/feet.

[7] Background pressure for Zone 2 is calculated by subtracting the static water level from the depth of the packer divided by 2.31 ft/psi, or (768 - 94) ÷ 2.31 = 292 psi. The packer pressure was then increased by at least 60 psi to ensure proper sealing of the packer within the well casing at this depth.

This page left intentionally blank

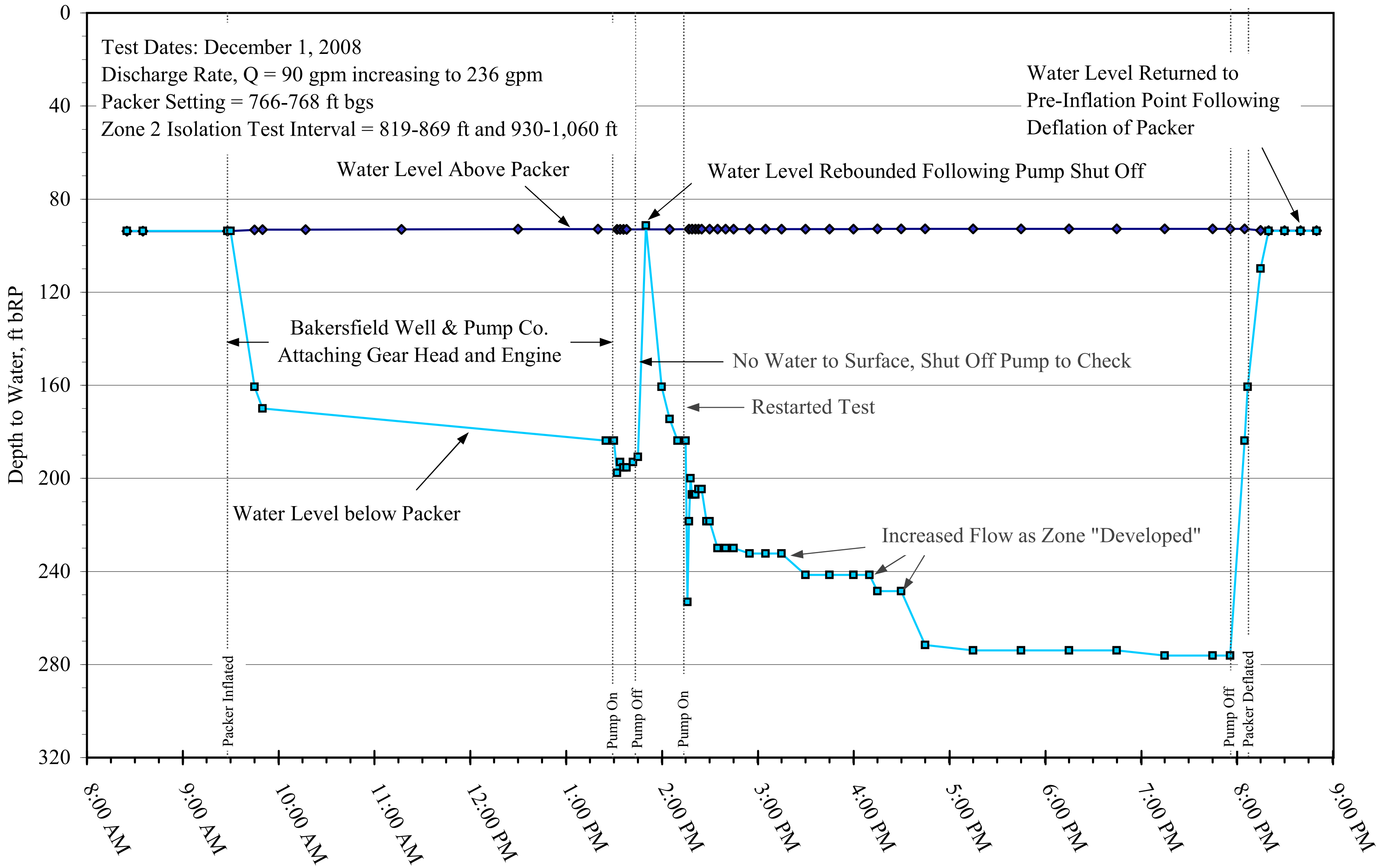



| | | | |
|---|---|---|---|
| Figure 2–13 | Drawn: PLP | RBF CONSULTING / IRVINE RANCH WATER DISTRICT | GEOSCIENCE |
| | Checked: | ZONE 2 ISOLATION TEST<br>IRWD WELL NO. 21 | GEOSCIENCE Support Services, Incorporated<br>P.O. Box 220, Claremont, CA 91711<br>Tel: (909)451-6650 Fax: (909)451-6638<br>www.gssiwater.com |
| | Approved: | | |
| | Date: 18–MAR–09 | | |

rbf\irwd\wells 21_22 rehab\rehab report well 21\figures\well 21 zone 2.dwg

This page left intentionally blank.

**Figure 2-14: Isolated Aquifer Zone Testing Timeline Analysis**
**Zone 2 Isolation Test - IRWD Well No. 21**




GEOSCIENCE Support Services, Inc.

Water quality samples were again collected for general mineral and physical properties, inorganics, volatile organic compounds, total organic carbon (TOC) and perchlorate before the pump was turned off and recovery levels were monitored. Once the zone had fully recovered, the packer was deflated and water levels returned to their pre-inflation point.

The complete results of the Zone 2 water quality analyses are shown in Appendix A. At the time of sample collection, the water was still slightly cloudy and was measured at 10.7 NTU in the field. The elevated level of suspended solids likely explains the high concentration of iron (590 µg/L) that was reported for this zone.

2.4.3.3 Analysis of Zone Isolation Pumping Test

Based on analysis of the isolated pumping tests, it is concluded that there is communication between the upper and lower aquifers, with an estimated 200-300 gpm being lost to the lower zones when the well is not being pumped (i.e., when it is idle). The amount of loss will diminish during pumping, particularly as the pumping water level approaches the static water level of the lower zone.

The lower screened intervals within Well 21 are very poor producers as indicated by the low pumping rates during zone isolation testing of 440 gpm for the middle and lower zones (539-749 feet , 819-869 feet and 930-1,060 feet bgs), and 236 gpm for the lowermost zones (819-869 feet and 930-1,060 feet bgs).

### 2.4.4 Review of 1992 Depth Specific Sampling and Spinner Survey Data

At the end of the April 28 to 30, 1992 constant rate pumping test, a spinner survey (using both down run and stop count data) was conducted by Welenco at a discharge rate of 2,970 gpm. The results showed that 92.1 percent of the total flow was being produced by the interval 298-509 feet bgs (211 feet), while 4.3 percent was produced by the interval 539-749 feet bgs (210 feet) and 3.6 percent was produced by the interval 819-869 feet bgs (50 feet). No flow was detected in the interval 930-1,060 feet bgs.

At this time, depth specific samples were obtained from the well at depths of 307 feet, 573 feet and 837 feet bgs and nitrate concentrations (as NO3) were recorded from the well depths at 44, 35 and 35 mg/L respectively, in addition to a composite sample reported as 43 mg/L (as N) from the full discharge. The results of the April 1992 depth specific water quality samples show that the water quality had slightly higher nitrates in the upper portion of the well (at 307 feet bgs), while the corresponding TDS in that interval was lower, reported to be 582 mg/L versus approximately 700 mg/L below 573 feet bgs. The composite water sample quality is most similar to the results from the upper depth specific samples (at 307 feet bgs), which indicates that most of the flow is coming from the uppermost screen.

A review of the 1992 spinner survey down runs that were performed at the end of the constant rate pumping test showed no flow being contributed from zones below 869 feet bgs. The most significant flow contribution in the well is from 298-350 feet, an interval that corresponds to a zone of very high resistivity on the electric log that suggests highly permeable material. Smaller amounts of flow are contributed from the 350-410 feet interval, also corresponding to intervals of high resistivity on the electric log.

The stop counts obtained in 1992 show that the most significant flow contribution is from the interval 300-360 feet bgs (the upper portion of the uppermost screened interval). No flow was detected below 820 feet, which is at the top of the lowermost screen interval. Intervals showing substantial flow contribution corresponded to the higher resistivity zones on the electric log, as was observed with the down run log.

In summary, following the recent well rehabilitation and development, the results of the aquifer testing conducted by GEOSCIENCE show an improvement over the 1992 testing. Specific capacities, flow rates and well efficiency are all increased over the 1992 results despite the well remaining idle for more than 16 years.

It should be noted that a dynamic spinner survey with stop counts could not be performed during the November 2008 constant rate pumping test due to the inflatable packer that was installed on the pump column within the well. The outside diameter of the packer measured 14 inches in diameter while the well casing and screen measures 16 inches in diameter. The resulting 1 inch of annular space was not enough room for the 1-11/16 inch diameter spinner tool to pass.

## 2.5 Well 21 Pump Design Recommendations

Based on the pumping test data found in Appendix H, a design discharge rate of 3,300 gpm is recommended for Well 21. At this discharge rate, and assuming current water level conditions, a short-term (i.e., 1 day) drawdown of approximately 49 feet is expected with a total lift (to the land surface) of 153 feet including 7 feet of estimated interference from Well 22 (located 684 feet to the southeast) when pumping at an estimated discharge rate of 1,600 gpm (Table 2-7). The long-term drawdown (i.e., after 1 year of *continuous* pumping) is estimated to be approximately 79 feet with a total lift to the land surface of 192 feet including an estimated 16 feet of interference from Well 22 when pumping at 1,600 gpm.

With the current static water level of 97 feet bgs, the specific capacity of Well 21 is approximately 67 gpm/feet at the design discharge rate of 3,300 gpm (see Figures 2-7 and 2-9). It is recommended that the pump intake be set at a depth of approximately 270 feet bgs, within the 20-inch diameter blank section of well casing (i.e., located from ground surface to 298 feet bgs). Table 2-7 summarizes the recommended pump design based on the current depth to static water level of 97 feet bgs.

**Table 2-7 IRWD Well 21 - Pump Design Recommendations**

| Parameters | Short Term (1 day) | Long Term (1 year) |
|---|---|---|
| Design Pumping Rate | 3,300 gpm | 3,300 gpm |
| Design Drawdown | 49 ft | 79 ft |
| Design Well Efficiency | 95% | 95% |
| Pump Setting | 270 ft bgs | 270 ft bgs |
| Estimated Interference When IRWD Well 22 is Pumping at 1,600 gpm[8] | 7 ft | 16 ft |
| Static Water Level Depth (without Interference) | 97 ft bgs | 97 ft bgs |
| Total Lift to Surface (does not include regional decline in static water level) | **153 ft** | **192 ft** |

8 Interference estimated for IRWD Well 22 pumping with an assumed discharge rate of 1,600 gpm and assuming average well field parameters.

## 2.6 Well 21 Ground Water Quality

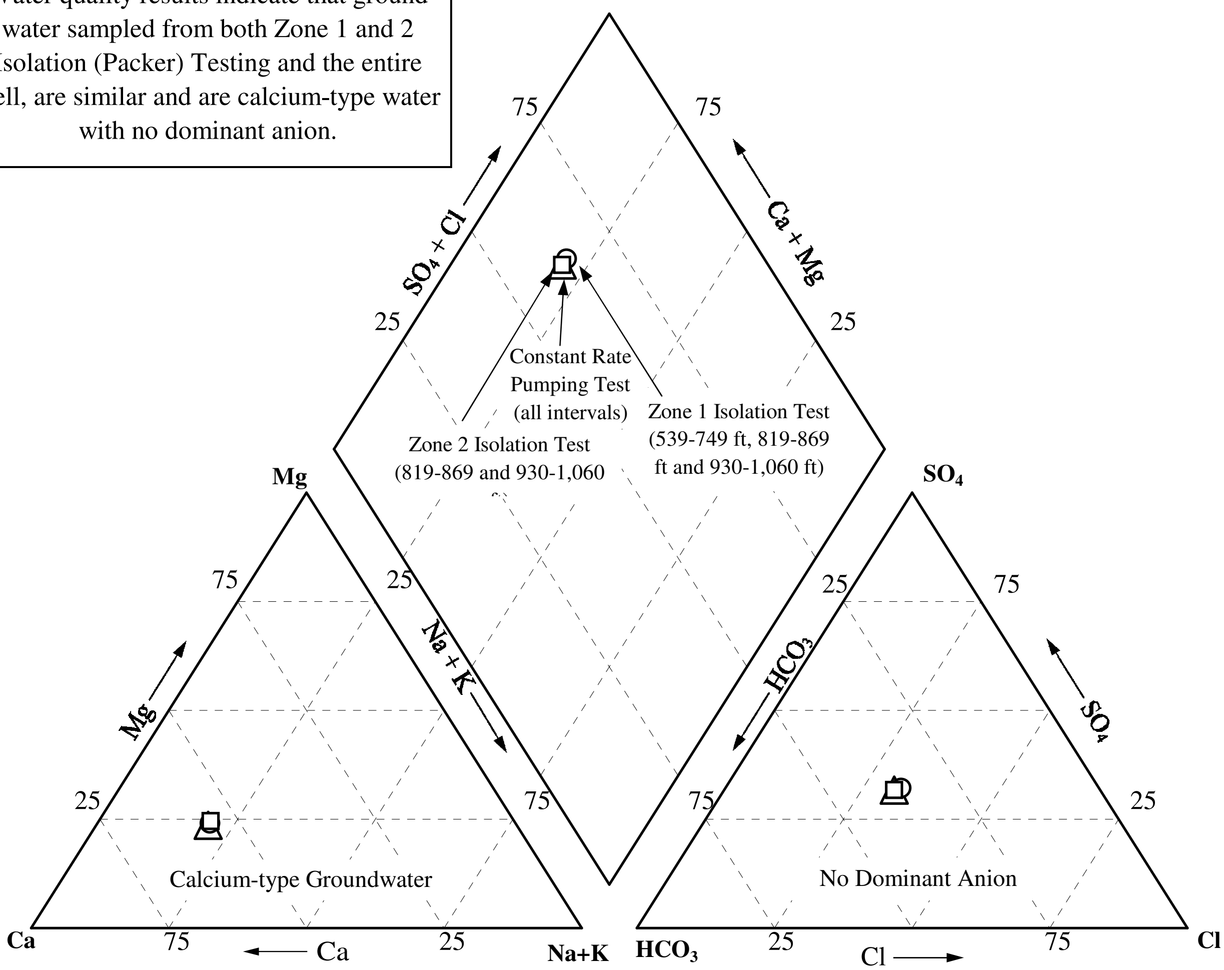
Water quality results of the constant rate test, and the Zone 1 and Zone 2 isolation tests conducted on both wells indicate that ground water from all three sampling events are calcium-type groundwater with no dominant anion. The results are presented in the tri-linear diagram shown in Figure 2-15.

Well nitrate (as $NO_3^-$) was detected in the constant rate test (i.e., Title 22 sampling event), and the Zone 1 and Zone 2 sampling events at concentrations of 67, 72, and 66 mg/L respectively. These concentrations exceed the State of California Department of Public Health (CDPH) maximum contaminate level (MCL) of 45 mg/L for nitrate. Perchlorate was detected during the constant rate test at a concentration of 4.3 micrograms per liter (µg/L), which is below the current CDPH MCL of 6.0 µg/L for perchlorate, however the Zone 1 and Zone 2 isolation tests showed a result of non-detect (i.e., less than the reportable detection limit, or RDL of 4.0 µg/L). Perchlorate was not detected in water quality samples collected from either the Zone 1 or Zone 2 isolation tests indicating that it is coming from the shallow interval (298-509 ft bgs), and not from the deeper intervals.

Iron was detected within Zone 2 at a concentration of 590 µg/L; significantly exceeding the CDPH secondary MCL of 300 µg/L) and is likely the result of the elevated turbidity of that zone at the time of sampling. The constant rate test and the Zone 1 test showed a result of non-detect (i.e., less than the RDL of 100 µg/L). Total hardness was detected in the constant rate test as well as Zone 1 and 2 at concentrations of 500, 510 and 520 µg/L respectively. Although hardness currently has no MCL, these concentrations are significantly elevated over that which is typically found in acceptable water sources. Total dissolved solids (TDS) were detected at elevated concentrations during the constant rate pumping test as well as the Zone 1 and Zone 2 pumping tests at concentrations of 740, 740 and 730 mg/L, respectively. These values exceed the lower recommended limit for the secondary MCL of 500 mg/L, but are below the maximum allowable limit of 1,000 mg/L.

**Figure 2-15: Trilinear Diagram**
**Water Quality Data - IRWD Well No. 21**
**Constant Rate Pumping Test and Zone Isolation (Packer) Tests**

Water quality results indicate that ground water sampled from both Zone 1 and 2 Isolation (Packer) Testing and the entire well, are similar and are calcium-type water with no dominant anion.




Water Quality Results from E.S. Babcock & Sons, Inc. of Riverside, California

At the end of the constant rate pumping test, the SDI was measured as 1.1 and the corresponding total silica was reported as 24 mg/L. All other constituents analyzed were found to be within the recommended drinking water standards. The complete results of the constituents analyzed are contained in Appendix A; however, selected results are summarized in Table 2-8.

Unfortunately, data collected during 1992 was incomplete for full comparison of the water quality; however, some constituents such as nitrate and sulfate are known to have increased. A tri-linear diagram that compares cation and anion data from the Zone 2 isolated aquifer zone test (565-580 feet) that was performed within the open borehole in 1992 appears to be consistent with the recent water quality data collected from the entire well and the zone isolation tests following rehabilitation and testing of Well 21 (see Figure 2-16).

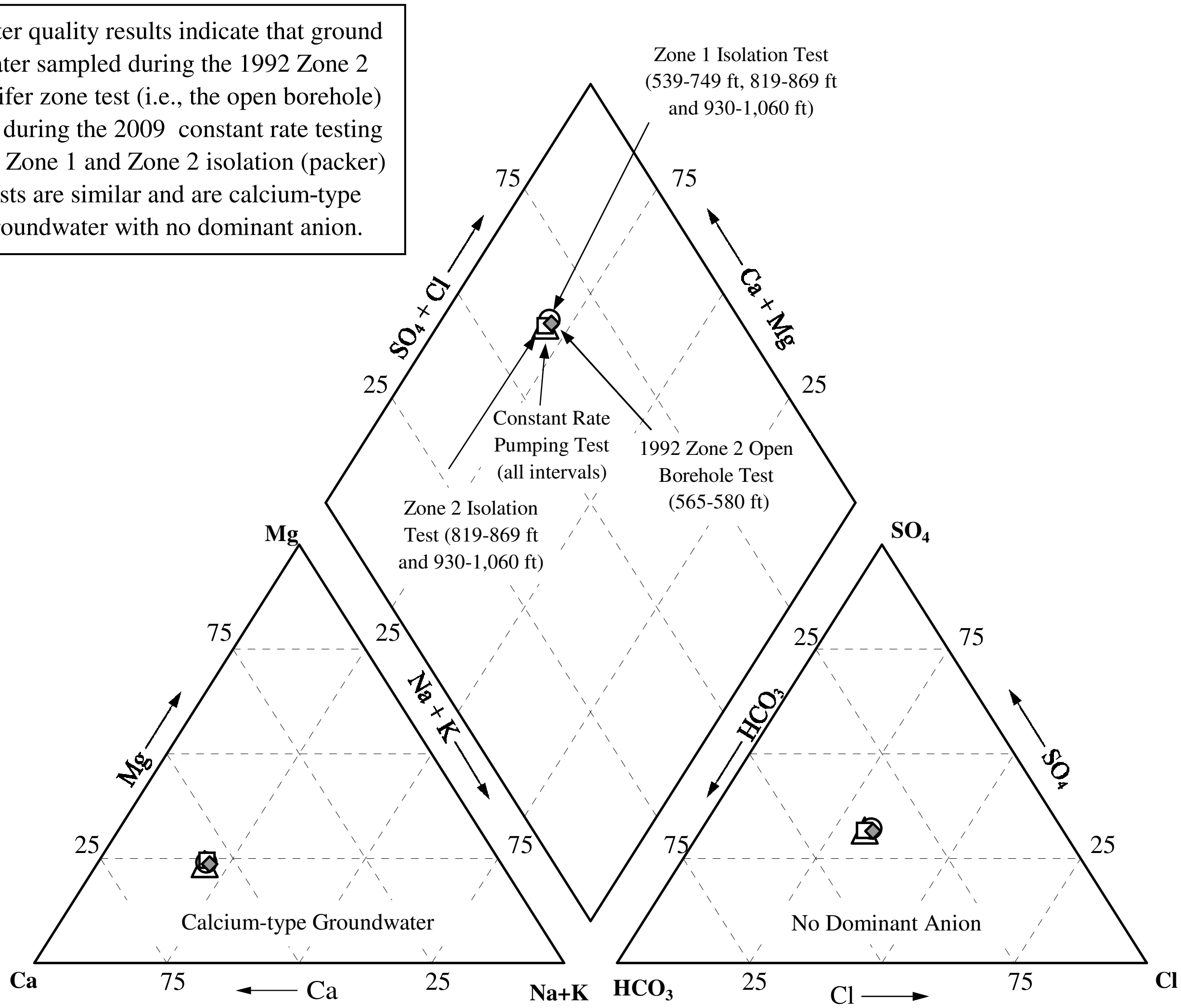
## Table 2-8 IRWD Well 21 - Summary of Title 22 Water Quality Analyses

| **Constituents** | | **Constant Rate Test**<br>Screened interval: 298-509, 539-749, 819-869, 930-1,060 [ft bgs] | **Isolated Zone 1**<br>Screened interval: 539-749, 819-869, 930-1,060 [ft bgs] | **Isolated Zone 2**<br>Screened interval: 819-869, 930-1,060 [ft bgs] | **Drinking Water Regulatory Standards** |
|---|---|---|---|---|---|
| **Arsenic** | [µg/L] | < 2 | - | - | 10[1] |
| **Boron** | [µg/L] | 140 | - | - | 1,000[3] |
| **Chromium (Total)** | [µg/L] | 8.2 | - | - | 50[1] |
| **Chloride** | [mg/L] | 130 | 140 | 130 | 250-500[2] |
| **Fluoride (Total)** | [mg/L] | 0.3 | - | - | 2.0[1] |
| **Iron** | [µg/L] | < 100 | < 100 | **590** | 300[2] |
| **Manganese** | [µg/L] | ND | ND | ND | 50[2] |
| **Nitrate (as $NO_3$)** | [mg/L] | **67** | **72** | **66** | 45[1] |
| **Perchlorate (EPA Method 314.0)** | [µg/L] | 4.3 | < 4.0 | < 4.0 | 6.0[1] |
| **Specific Conductance** | [umhos/cm] | 1,200 | 1,200 | 1,200 | 900-1,600[2] |
| **Sulfate (as $SO_4$)** | [mg/L] | 180 | 190 | 180 | 250-500[2] |
| **1,2,3-Trichloropropane** | [µg/L] | <0.005 | - | - | 5[3] |
| **Total Dissolved Solids, TDS** | [mg/L] | **740** | **740** | **730** | 500 - 1,000[2] |
| **Total Organic Carbon (TOC)** | [mg/L] | < 0.30 | < 0.30 | 0.35 | NA[6] |
| **Total Silica** | [mg/L] | 24 | - | - | NA[6] |
| **Total Hardness** | [µg/L] | **500** | **510** | **520** | NA[6] |
| **Vanadium** | [µg/L] | 3.2 | - | - | 50[3] |
| **Volatile Organic Chemicals (EPA Method 524.2)** | [µg/L] | ND | ND | ND | Varies with Compound |
| **Gross Alpha** | [pCi/L] | 1.37 | - | - | 15[1] |
| **Color** | [color unit] | < 3 | < 3 | < 3 | 15[2] |
| **Odor** | [TON] | < 1 | < 1 | < 1 | 3[2] |
| **pH** | [std. units] | 7.4 | 7.3 | 7.4 | 6.5 - 8.5[5] |
| **Turbidity** | [NTU] | 0.37 | 0.65 | **7.5** | 5[2] |
| **Silt Density Index, SDI** | - | 1.11 | - | - | NA[6] |

1 California Department of Public Health (CDPH) Primary Maximum Contaminant Levels (MCL).
2 CDPH Secondary MCLs.
3 CDPH notification level for unregulated chemicals requiring monitoring.
4 Hexavalent chromium is currently regulated under the 50-microgram per liter (□g/L) MCL for total chromium.
5 USEPA recommended range for pH.
6 Not applicable, no current MCL.
ND Not detected – used only regarding VOCs not located in the above table.
**BOLD** Equal to or above current CDPH MCLs or notification levels.

**Figure 2-16: Trilinear Diagram**
**Comparison of 2008 and 1992 Water Quality Results**
**IRWD Well No. 21**

Water quality results indicate that ground water sampled during the 1992 Zone 2 aquifer zone test (i.e., the open borehole) and during the 2009 constant rate testing and Zone 1 and Zone 2 isolation (packer) tests are similar and are calcium-type groundwater with no dominant anion.




2008 Water Quality Results from E.S. Babcock & Sons, Inc. of Riverside, California. 1992 water quality results are from IRWD Michelson Water Reclamation Plant Laboratory.

## 2.7 Final Downhole Video Survey and Testing of Well 21

On December 8, 2008, Pacific Surveys completed the final video survey to document the post-rehabilitation condition of the well. The video survey showed that in the upper portions of the well a moderate amount of debris remained suspended in water column. The water column cleared below 293 feet bgs, particularly when the top of the screened interval was reached at 298 feet bgs. The well was observed to be structurally sound and the screened intervals were observed to be clean and open. Some light to moderate staining was observed on the well casing and screen at some of the spiral welds and heat affected zone within the casing, particularly where tubercular deposits were removed during the rehabilitation process. Appendix E contains a copy of the post-rehabilitation video survey report that was provided by Pacific Surveys.

In addition to the post-rehabilitation video, a pre-rehabilitation video survey was conducted on January 17, 2008 and a post-brushing, swabbing, and airlifting video survey was conducted on October 20, 2008. Copies of each video survey report are found in Appendix E.

On February 11, 2009, the remaining 30 feet of fill material was bailed from the bottom of the well. The material removed consisted of very fine sand and silt with approximately 5 percent Colorado silica sand.

On February 13, 2009, Well 21 was disinfected by adjusting the pH in the water column to 5 pH units using 12 gallons of NW-410 product before injecting 15 gallons of 12.5 percent sodium hypochlorite through a tremie pipe at 100 foot intervals. The water column was thoroughly agitated using a bailer before testing the chlorine concentration at 82 mg/L and 86 mg/L.

During final demobilization and clean up BW&P personnel welded the casing extensions back on the top of the well casing as well as the gravel feed pipe and sounding tube. At the conclusion of the work, a cover was welded on top of the 20-inch well casing and threaded caps were tightly fastened to the tops of the gravel feed pipe and the sounding tube.

## 2.8 Well 21 Conclusions and Recommendations

### 2.8.1 Conclusions

Well 21 is a multi-aquifer completed well (i.e., the well is screened in multiple aquifers) and based on the testing and analysis, vertical flow may be occurring from the shallow to the deeper zones. The amount of flow can be quantified (if necessary) with a static (non-pumping) flowmeter survey. The amount of downward flow of 200 to 300 gpm was predicted based on results of isolated zone (packer) testing and transmissivity of aquifers.

The design discharge rate is increased over that of the 1992 recommendations and is 3,300 gpm versus 3,000 gpm (increase of 300 gpm). The efficiency of the well is 95 percent with 2 feet of the total drawdown attributed to well losses and 47 feet attributed to formation losses. Rehabilitation has improved the well over its original efficiency.

The specific capacity of Well 21 following rehabilitation was improved over the 1992 results (after construction of the well) as shown in Table 2-9.

**Table 2-9 IRWD Well 21 - Improved Specific Capacity Following Rehabilitation**

| 2008 Result Following Rehabilitation (at end of each step) | | | 1992 Result Following Initial Construction, Development and Testing (at end of each step) | | |
|---|---|---|---|---|---|
| Discharge Rate, Q [gpm] | Drawdown [ft] | Specific Capacity, Q/s, [gpm/ft] | Discharge Rate, Q [gpm] | Drawdown [ft] | Specific Capacity, Q/s, [gpm/ft] |
| **1,417** | **16.83** | **84** | **1,480** | **18.23** | **81.2** |
| **2,545** | **31.93** | **80** | **2,250** | **29.86** | **75.4** |
| **3,583** | **46.23** | **79** | **2,970** | **41.46** | **71.6** |
| **-** | **-** | **-** | **4,500** | **67.2** | **67.0** |
| Design = 3,300 | **49** | **67** | Design = 3,000 | **49.1** | **61** |

The large mounds of tubercles that were observed in January 17, 2008 video survey have been completely removed from within the screened intervals and blank sections of casing within the well.

Well 21 was observed to be in sound structural condition after more than 16 years of idle time. Sand production during the pumping tests was minimal.

Nitrate and TDS concentrations in Well 21 have increased since the well was drilled in 1992 (Table 2-10), exceeding their respective primary and secondary MCLs. Hardness is also unacceptably high. The perchlorate concentration was reported to be 4.3 µg/L in Well 21, which is slightly above the laboratory's reportable detection limit (RDL) but is less than the State's MCL of 6.0 µg/L. Perchlorate was not detected in either of the zone isolation tests at the laboratory RDL of 4.0 µg/L.

Basic water chemistry in terms of cations and anions has remained unchanged from 1992 to 2008 (see Figure 2-16); however, nitrate concentrations have increased in that time period from 43 mg/L to 67 mg/L. TDS also increased from 600 mg/L to 740 mg/L in the same time period. Hardness was not consistently analyzed in 1992; however, it is likely that hardness has increased at approximately the same rate as TDS.

The amount of interference created by Well 22 pumping at 1,600 gpm was estimated as 16 feet under long-term pumping conditions and is based on average well field parameters. In effect, the static water level in Well 21 will be lowered by this amount when Well 22 is being pumped at the design discharge rate. The silt density index (SDI) was measured in the field at the end of the constant rate pumping test as 1.1.

**Table 2-10 IRWD Well 21 - Comparison of 2008 and 1992 Depth Specific Water Quality Results**

| 2008 Water Quality Results | | | | 1992 Water Quality Results | | | |
|---|---|---|---|---|---|---|---|
| Event/Depth | Nitrate (NO3) [mg/L] | TDS [mg/L] | Hardness [mg/L] | Event/Depth | Nitrate (NO3) [mg/L] | TDS [mg/L] | Hardness [mg/L] |
| - | - | - | - | Zone 1 Open Borehole 830-845 ft | 6.6 | 376 | 125 |
| - | - | - | - | Zone 2 Open Borehole 565-580 ft | 44.3 | 754 | 480 |
| - | - | - | - | Zone 3 Open Borehole 300-315 ft | 42.5 | 712 | 462 |
| Constant Rate Pumping Test (298-1,060 ft) | 67 | 740 | 500 | Constant Rate Pumping Test (298-1,060 ft) | 43.0 | 600 | - |
| Zone 1 Isolation Test (539-1,060 ft) | 72 | 740 | 510 | Depth Specific Sampling at 307 ft | 43.9 | 582 | - |
| Zone 2 Isolation Test (819-1,060 ft) | 66 | 730 | 520 | Depth Specific Sampling at 573 ft | 35.4 | 700 | - |
| | - | - | - | Depth Specific Sampling at 837 ft | 35.0 | 678 | - |

## 2.9 Recommendations

An operational schedule of 24 hours per day, 7 days per week, for a period of 11 months per year to maximize production is acceptable; however, it is important that the well is turned off for a short time each year to allow full recovery. It is not desirable for a production well to be operated continuously year after year without being allowed to recover.

If Well 21 is to remain idle and unequipped for more than a period of 5 years, a limited rehabilitation program is recommended prior to placing the well in service. As a minimum, the well should be videoed and thoroughly brushed before being equipped. Brushing should be accomplished using a new nylon or polypropylene bristle brush and should take place for at least 1 minute per wetted foot of screen, and 1/2 minute per wetted foot of blank casing with the brush measuring approximately 1/2 inches larger in diameter than the casing. A weighted core will ensure passage downward through the screen. The use of a "wire" brush should not be allowed. Accumulated fill or sediment should be bailed prior to installing a test pump.

The well should be thoroughly re-developed by alternating pumping with surging (starting at low intensity and working upward), before conducting step drawdown and constant rate pumping tests for comparison with previous testing to ensure that no loss in specific capacity has occurred. Should there be a significant reduction in the specific capacity, additional redevelopment efforts may be required. All work should be followed by a video survey and final disinfection.

If a well is idle 10 or more years, it should be brushed as described above followed by airlifting and swabbing to stress each 10 foot interval of screen. Airlifting and swabbing should take place for a minimum of one-half hour per 10 foot interval. If sand, silt, or cloudy water is produced, the length of time spent in airlifting and swabbing should be increased until clear or nearly clear water is achieved. A test pump is recommended to be installed for redevelopment and testing as described above, with test results compared against previous results. If the specific capacity is less than desirable, further rehabilitation steps (such as focused airlifting and swabbing using dispersants and surfactants followed by additional development pumping) should undertaken to restore capacity. All work within the well should be followed by a video survey and final disinfection. A summary of the recommended actions are provided in Table 2-11.

**Table 2-11 Recommended Redevelopment Based on 2, 5 and 10 Year Idle Periods**

| Description of Work | After 2 Years Idle | After 5 Years Idle | After 10 Years Idle |
|---|---|---|---|
| Perform Pre- Video Survey | Not Recommended | √ | √ |
| Brushing with Nylon or Polypropylene Brush Followed by Bailing | Not Recommended | √ | √ |
| Airlifting and Swabbing at 10 ft Intervals Throughout Screen | Not Recommended | Not Recommended | √ |
| Installation of Test Pump Followed by Development Pumping and Removal of Test Pump Equipment | Not Recommended | √ | √ |
| Perform Step Drawdown and Constant Rate Pumping Tests | Not Recommended | √ | √ |
| Perform Post- Video Survey | Not Recommended | √ | √ |
| Perform Final Disinfection | Not Recommended | √ | √ |

## 2.10 Well 22 Pumping Test Data, Analysis and Results

### 2.10.1 Well 22 Step Drawdown Test

A step drawdown test was conducted on January 9, 2009 at discharge rates of 850 gpm, 1,613 gpm, and 2,227 gpm with incremental drawdowns of 40, 39 and 37 gpm/feet respectively. The static water level at the beginning of the test was approximately 128 feet bgs. Figure 2-17 is a plot of the step drawdown test data and shows the time-drawdown curve for each step. The specific drawdown for each step is shown in Table 2-12 below. It should be noted that the values shown on this table are extrapolated to reflect the value at 1,440 minutes after the start of each step, and are not the values at the end of each step.

**Table 2-12 Summary of Step Drawdown Test Data for IRWD Well 22**

| Step $m$ | Discharge Rate $Q_m$ [gpm] | Incremental Drawdown[1] $\Delta_m$ [ft] | Cumulative Drawdown* $s_m$ [ft] | Specific Drawdown $(s/Q)_m$ [ft/gpm] |
|---|---|---|---|---|
| 1 | 850 | 40 | 40 | 0.0471 |
| 2 | 1,613 | 39 | 79 | 0.0490 |
| 3 | 2,227 | 37 | 116 | 0.0521 |

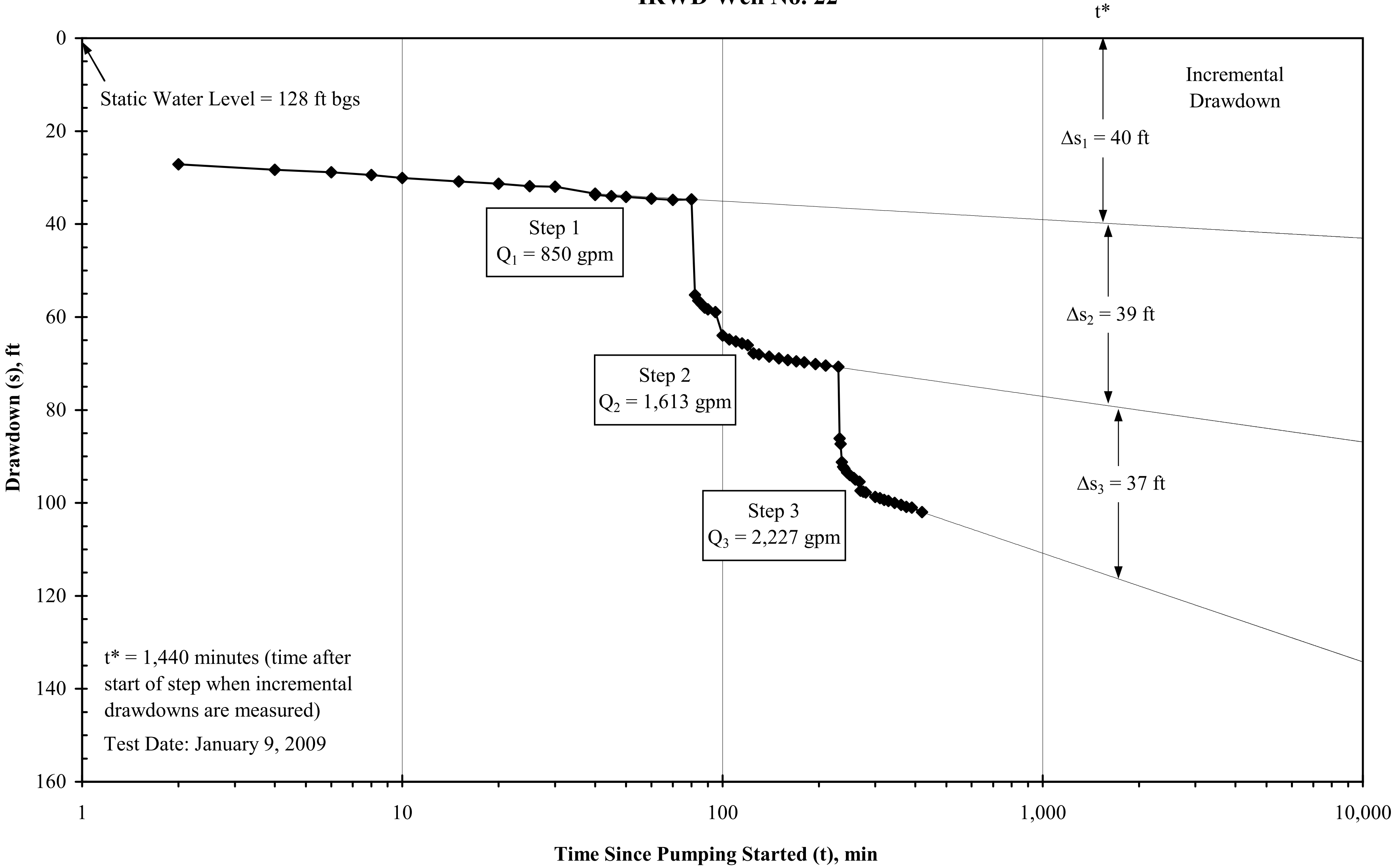
* Extrapolated to 1,440 minutes after the start of each step.

The specific drawdown plot (see Figure 2-18) shows the relationship between specific drawdown (s/Q) and the discharge rate (Q). The testing showed a formation loss coefficient (B) of 0.04332 feet/gpm and a well loss coefficient (C) of 0.000003596 feet/gpm$^2$.

The specific capacity and efficiency diagram (see Figure 2-19) shows estimated drawdown and well efficiency. Estimated drawdown and well efficiency may also be calculated from equations (1) and (2) discussed in Section 3.2.1 Step Drawdown Test Method. Based on a design discharge rate of 1,600 gpm short-term drawdown is expected to be 79 feet and long-term drawdown is expected to be 109 feet with a well efficiency of 88 percent.

The performance of Well 22 was restored through the recent rehabilitation process as seen by comparing specific capacity data obtained from the 1992 post-construction step drawdown tests to the data obtained from the 2009 post-rehabilitation step drawdown test. Table 2-13 and Figure 2-20 compares the 2009 post-rehabilitation step test data to the 1992 post-construction step drawdown test data.

**Figure 2-17: Step Drawdown Pumping Test**
**IRWD Well No. 22**




GEOSCIENCE Support Services, Inc.

**Figure 2-18: Specific Drawdown Plot**
**IRWD Well No. 22**

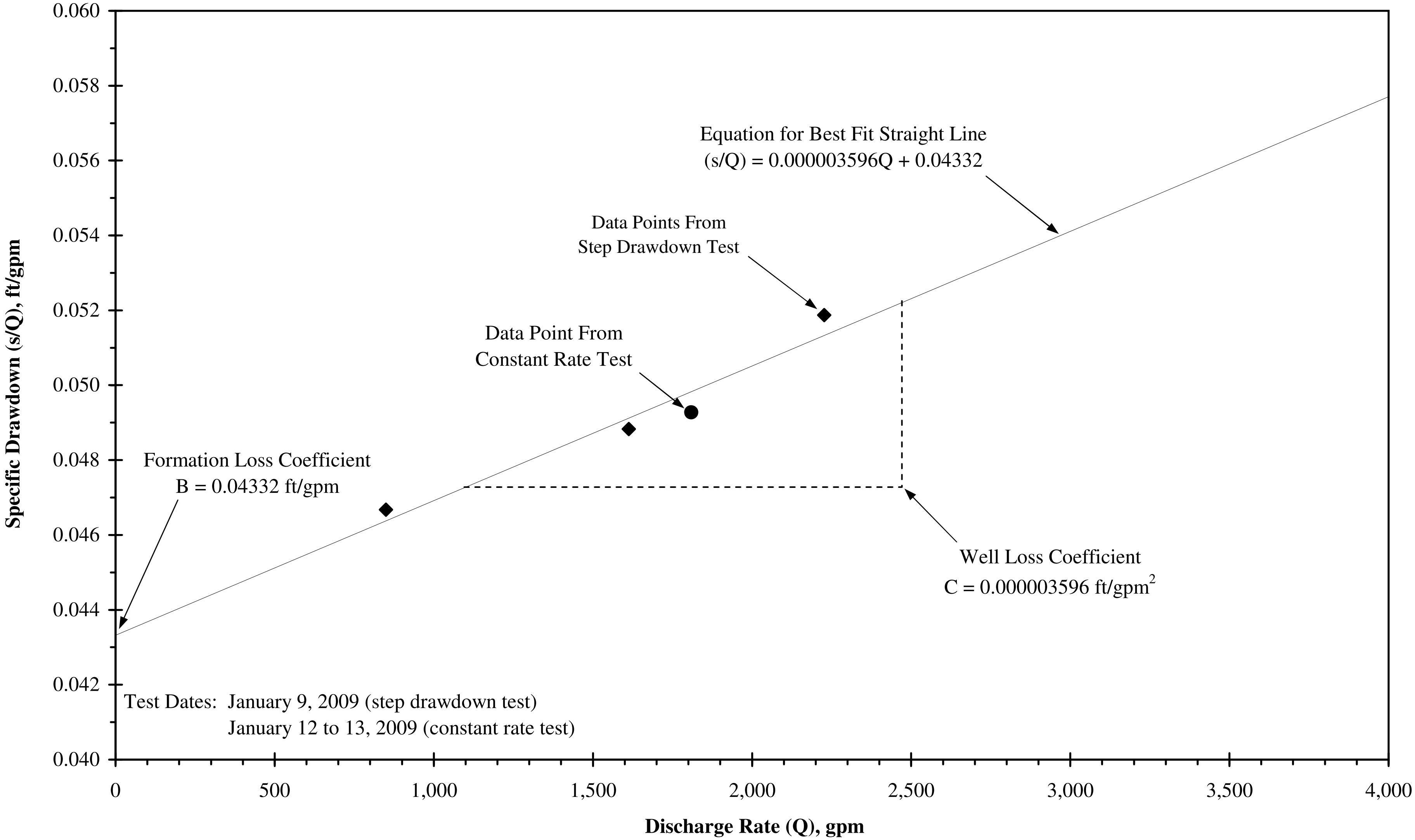


**Figure 2-19: Specific Capacity and Well Efficiency Diagram**
**IRWD Well No. 22**

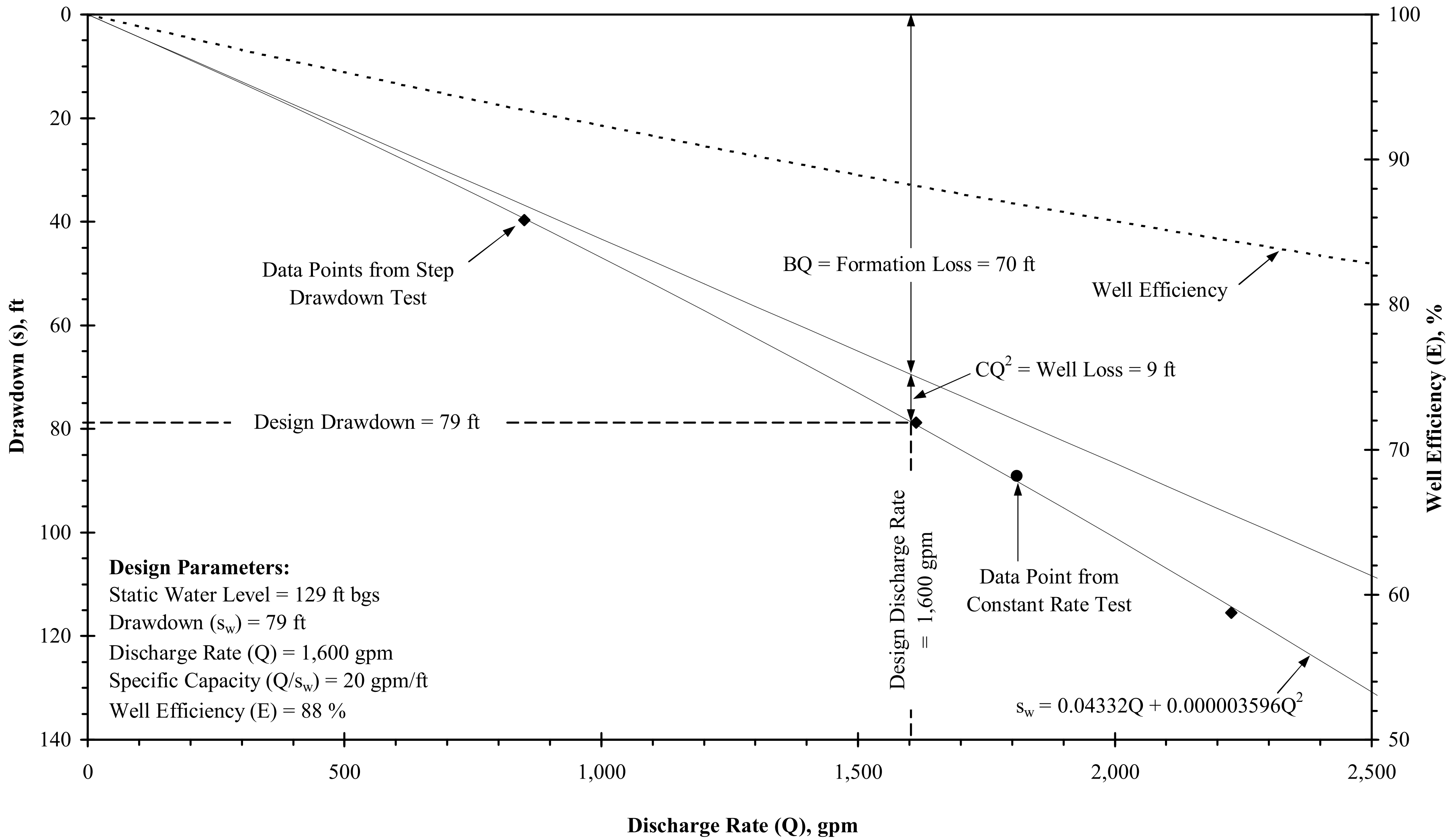
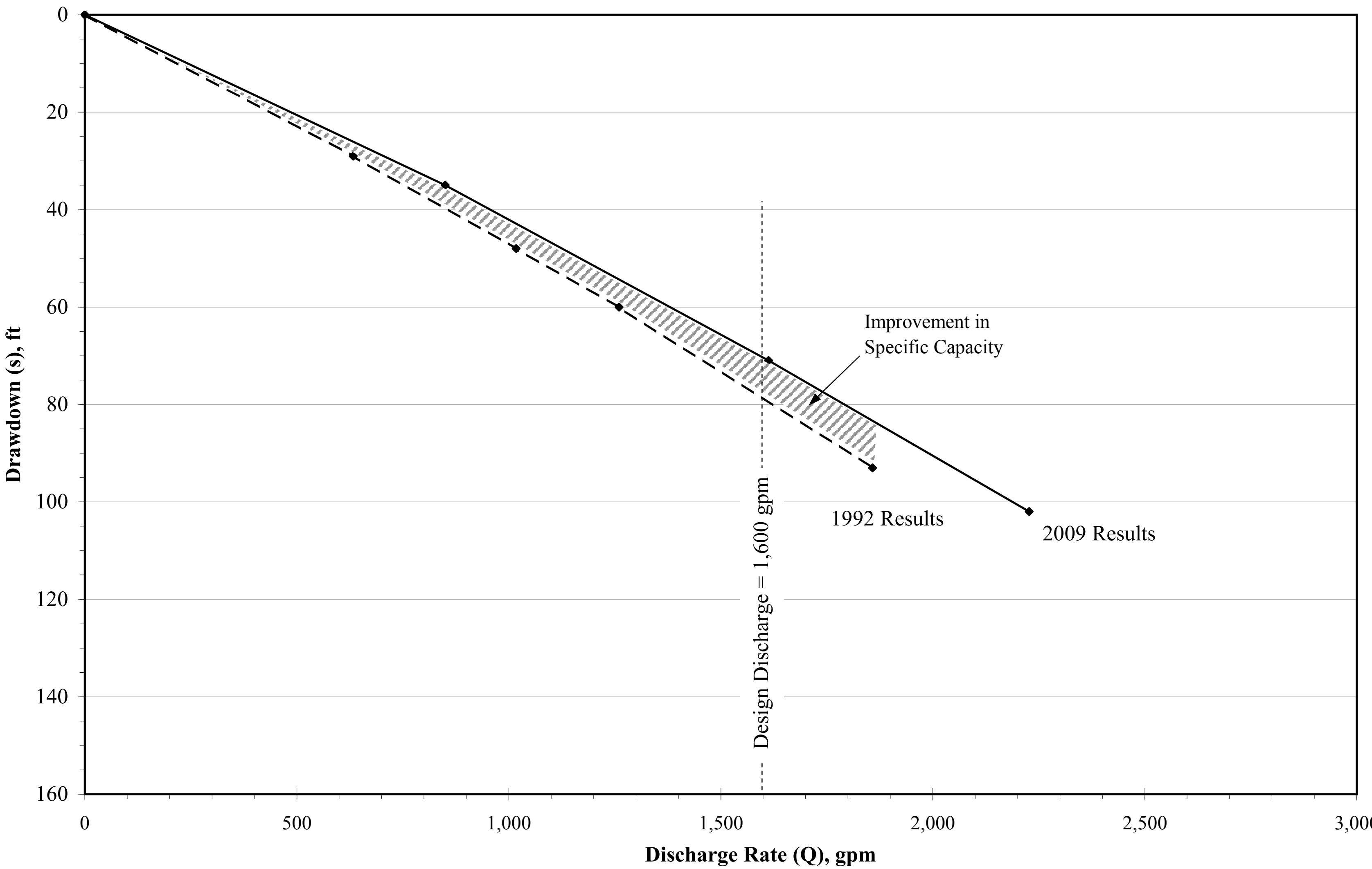


**Figure 2-20: Improved Specific Capacity**
**IRWD Well No. 22**




GEOSCIENCE Support Services, Inc.

**Table 2-13 IRWD Well 22 - Comparison of 1992 Post-Construction and 2009 Post-Rehabilitation Step Drawdown Test Data**

| Step m | May 1992 Discharge Rate $Q_m$ [gpm] | May 1992 Drawdown at End of Step $s_m$ [ft] | May 1992 Specific Capacity $(Q/s)_m$ [gpm/ft] | January 2009 Discharge Rate $Q_m$ [gpm] | January 2009 Drawdown at End of Step $s_m$ [ft] | January 2009 Specific Capacity $(Q/s)_m$ [gpm/ft] |
|---|---|---|---|---|---|---|
| 1 | 633 | 29.40 | 21.5 | 850 | 34.67 | 24.5 |
| 2 | 1,017 | 47.60 | 21.4 | 1,613 | 70.72 | 22.8 |
| 3 | 1,260 | 60.43 | 20.8 | 2,227 | 101.82 | 21.9 |
| 4 | 1,858 | 92.58 | 20.1 | - | - | - |

As shown on the table above, following the specific capacity at the end of each step is higher following the recent rehabilitation than was originally measured following well construction in 1992. It is known that in a fully developed well the specific capacity decreases as the discharge rate increases, so although 2009 testing was performed at a slightly higher rate, the corresponding specific capacity is also slightly higher.

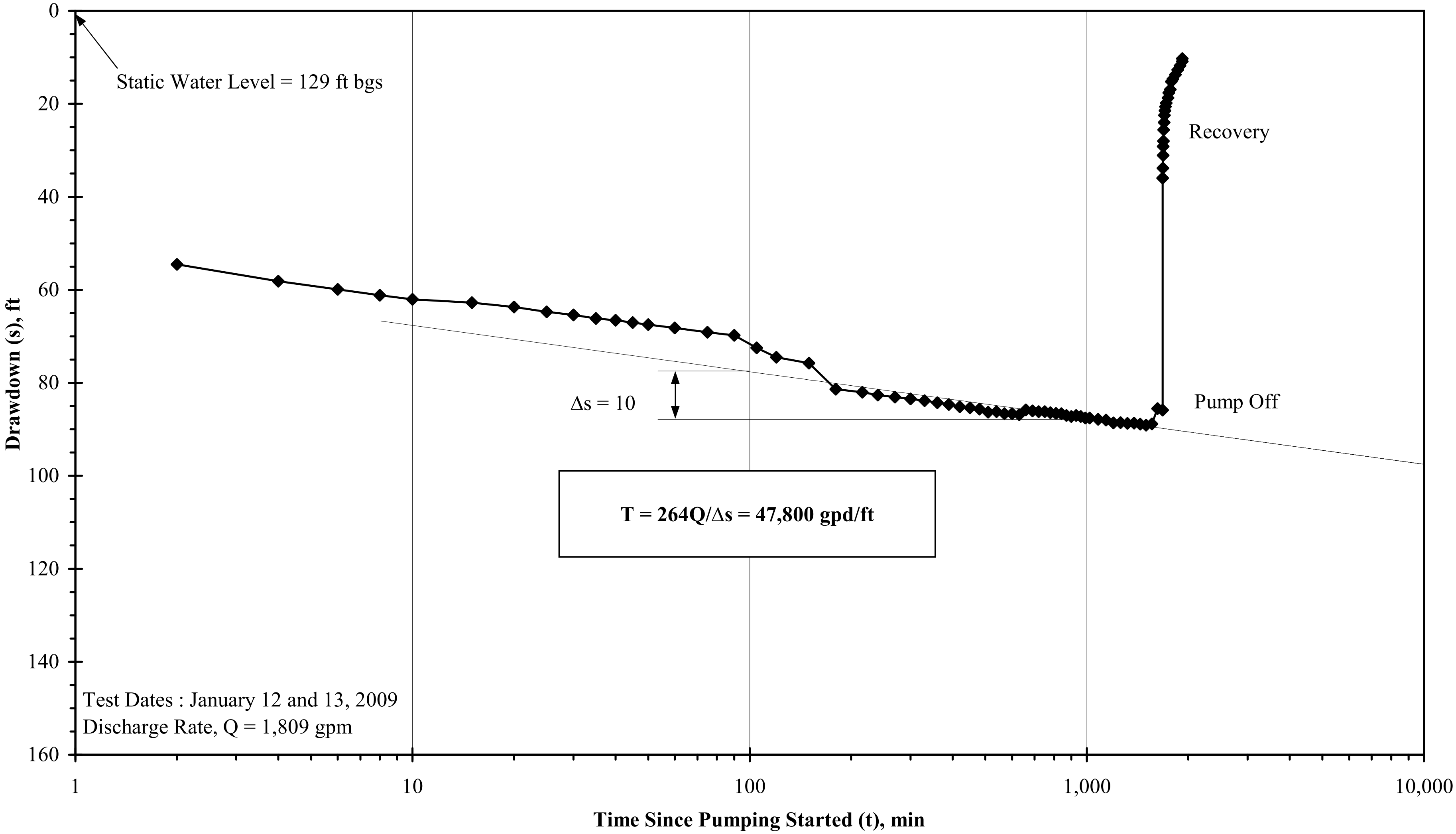
### 2.10.2 24-Hour Constant Rate Test for Well 22

Following recovery from the step drawdown test, a 24-hour constant rate pumping test was conducted from January 12 and 13, 2009 (see Figure 2-21). The static water level at the start of the test was approximately 129 feet bgs, and the average discharge rate was 1,809 gpm. Water level drawdown was 10 feet per log cycle. Jacob's straight-line method was used to analyze the time-drawdown data and results show a transmissivity of approximately 47,800 gpd/feet (see Figure 2-21).

During the 24-hour constant rate test, water level measurements were taken at Well 21 (see Figure 2-22), which served as an observation well approximately 684 feet to the northwest of Well 22. The static water level within the observation well at the start of the test was approximately 95 feet bgs. A water level drawdown of 3.3 feet per log cycle was measured in Well 21 while pumping Well 22 at 1,809 gpm. The Jacob's straight-line method was used to analyze the time-drawdown data and results show a transmissivity of approximately 145,000 gpd/feet (see Figure 2-22). At the end of the constant rate pumping test, water quality samples were collected for full Title 22 and California Unregulated analysis, including silica. The water quality samples were delivered to E.S. Babcock & Sons, Inc. (see Appendix A). In addition, a silt density index (SDI) test was conducted in the field prior to shutting down the pump, resulting in an SDI value of 0.98.

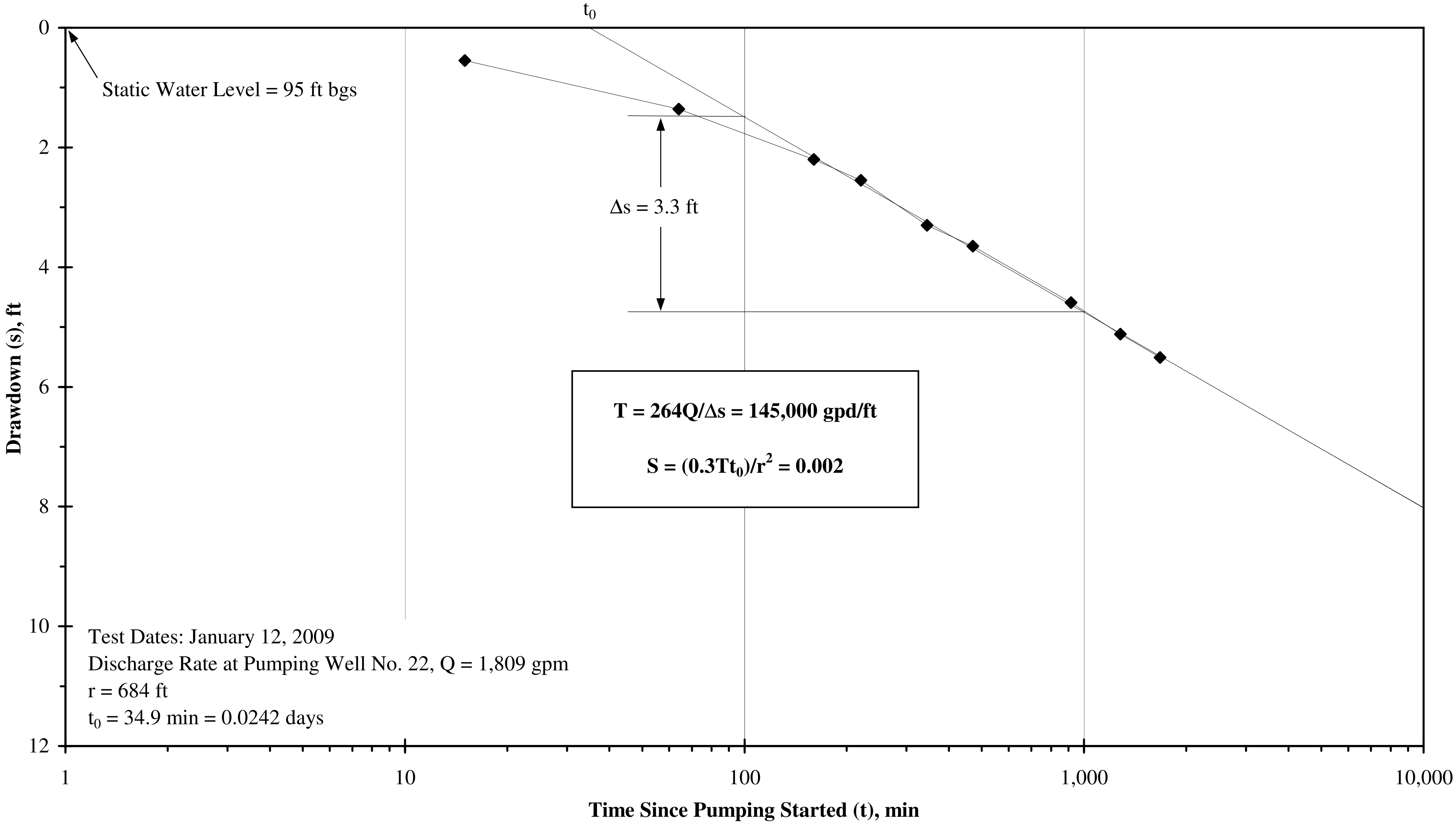
This page left intentionally blank

**Figure 2-21: Constant Rate Pumping Test Chart**
**IRWD Well No. 22**




GEOSCIENCE Support Services, Inc.

**Figure 2-22: Observation Well Data**
**IRWD Well No. 21 (IRWD Well No. 22 Pumping)**




**GEOSCIENCE Support Services, Inc.**

### 2.10.3 Well 22 Zone Isolation Pumping Tests

Because of the potential for treatment, two zone isolation tests were planned to test for changes in the water quality within the aquifers screened. The Zone 1 isolation test was conducted January 14, 2009 following recovery from the constant rate pumping test, while the Zone 2 isolation test was conducted January 19, 2009.

#### 2.10.3.1 Zone 1 Isolation Pumping Test

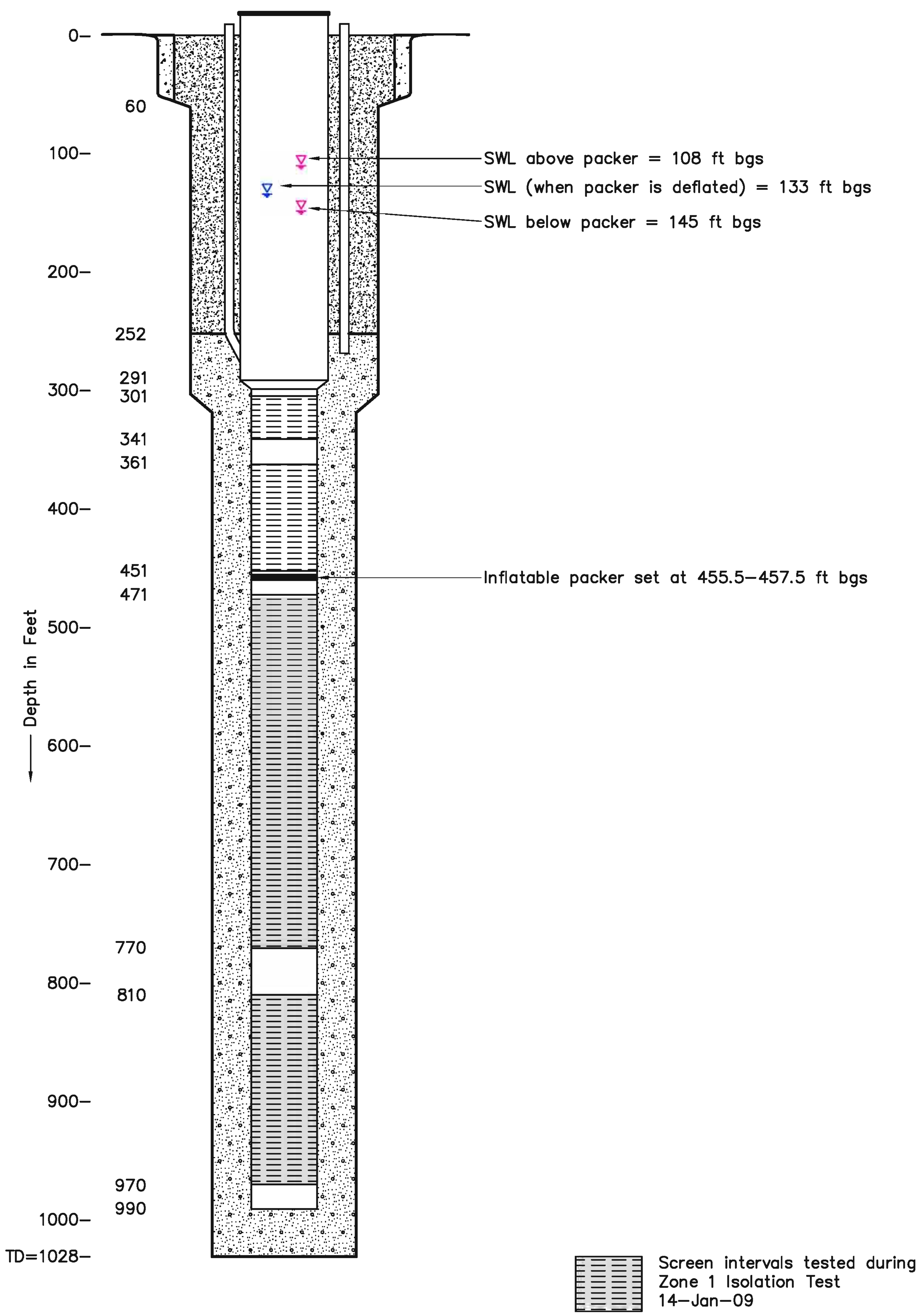
The Zone 1 isolation test was conducted with the inflatable packer installed at 455.5-457.5 feet bgs (see Figures 2-23 and 2-24), and with the pump intake located at approximately 467 feet bgs. The static water level in the well was approximately 133 feet below ground surface (bgs) prior to starting the test.

The Zone 1 isolation test consisted of inflating the packer to 180 pounds per square inch (psi), approximately 40 psi greater than background pressure[9] to test water quality and quantity in the screened intervals located at 471-770 feet and 810-970 feet bgs, totaling 459 feet. The packer removed the uppermost screens located at 301-341 feet and 361-451 feet bgs from production during the test.

After inflating the packer and allowing water levels to stabilize, the static water level below the packer fell to approximately 145 feet bgs, while the water level above the packer was elevated to 108 feet bgs (see Figure 2-24). The zone was initially pumped at a low discharge rate of approximately 300 gpm that was gradually increased to 1,400 gpm. Water levels and discharge rates were monitored frequently during the testing. At the maximum discharge rate, the drawdown was measured at 90 feet, with a resulting specific capacity of 15 gpm/feet.

9 Background pressure is calculated by subtracting the static water level from the depth of the packer divided by 2.31 ft/psi, or (455-129)÷2.31=141 psi. The packer pressure was then increased by approximately 40 psi to ensure proper sealing of the packer within the well casing.

This page left intentionally blank



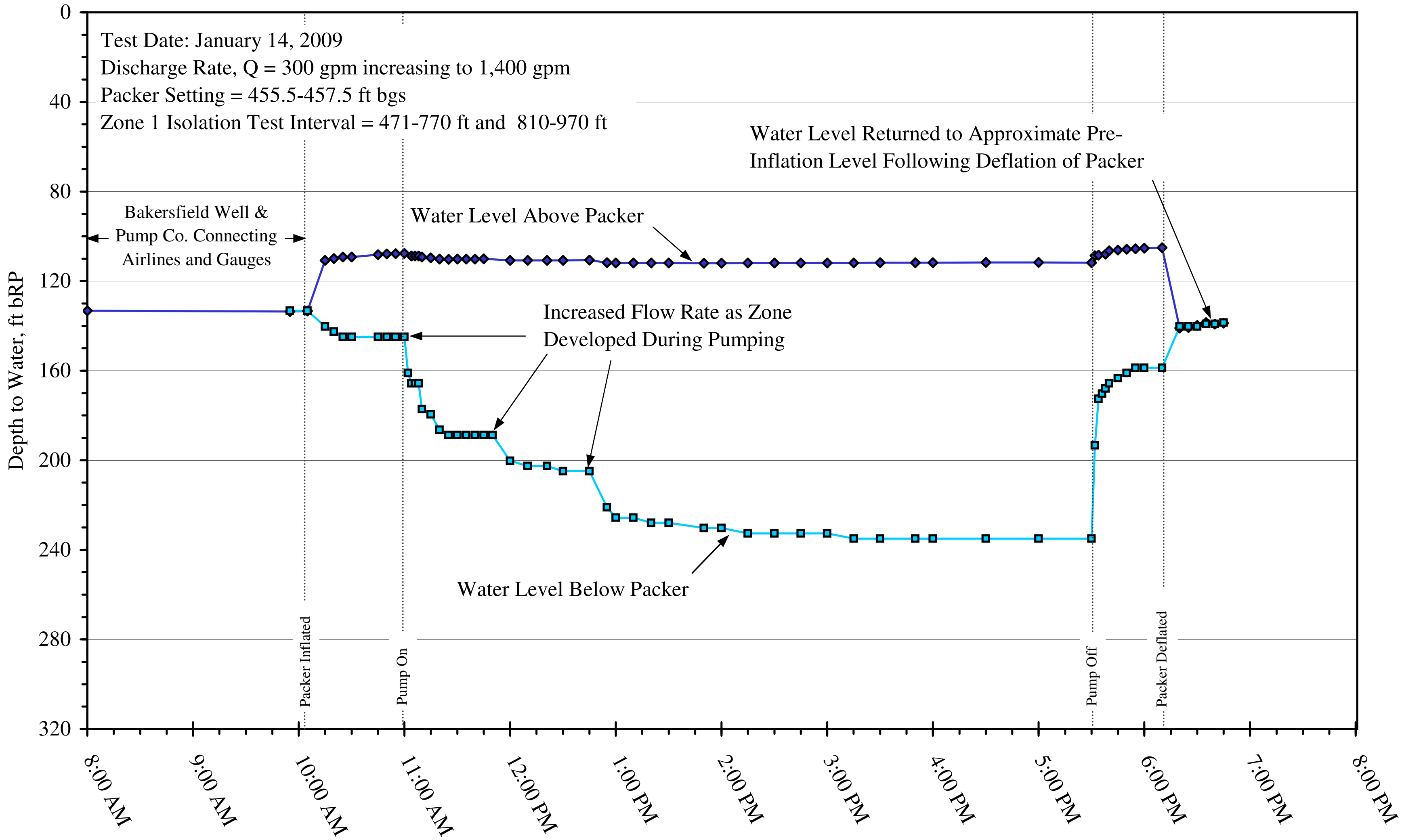

WELL CROSS SECTION

| | | | |
|---|---|---|---|
| Figure 2–23 | Drawn: PLP | RBF CONSULTING / IRVINE RANCH WATER DISTRICT | GEOSCIENCE |
| | Checked: | ZONE 1 ISOLATION TEST | GEOSCIENCE Support Services, Incorporated P.O. Box 220, Claremont, CA 91711 Tel: (909)451–6650 Fax: (909)451–6638 www.gssiwater.com |
| | Approved: | IRWD WELL NO. 22 | |
| | Date: 18–MAR–09 | | |

rbf\irwd\wells 21_22 rehab\rehab report well 22\figures\well 22 zone 1.dwg

This page left intentionally blank.

**Figure 2-24: Isolated Aquifer Zone Testing Timeline Analysis**
**Zone 1 Isolation Test - IRWD Well No. 22**




GEOSCIENCE Support Services, Inc.

After 6 hours of continuous pumping, water quality samples were collected for analysis of general mineral and physical properties, metals (i.e., inorganics), VOCs, perchlorate and total organic carbon (TOC) before the pump was turned off and recovering water levels were monitored. Once the zone had fully recovered, the packer was deflated and water levels returned to their pre-inflation levels. At the time of sample collection, the field turbidity of the water was 0.45 nephelometric turbidity units (NTU). The complete results of the Zone 1 water quality analyses are shown in Appendix A.

#### 2.10.3.2 Zone 2 Isolation Pumping Test

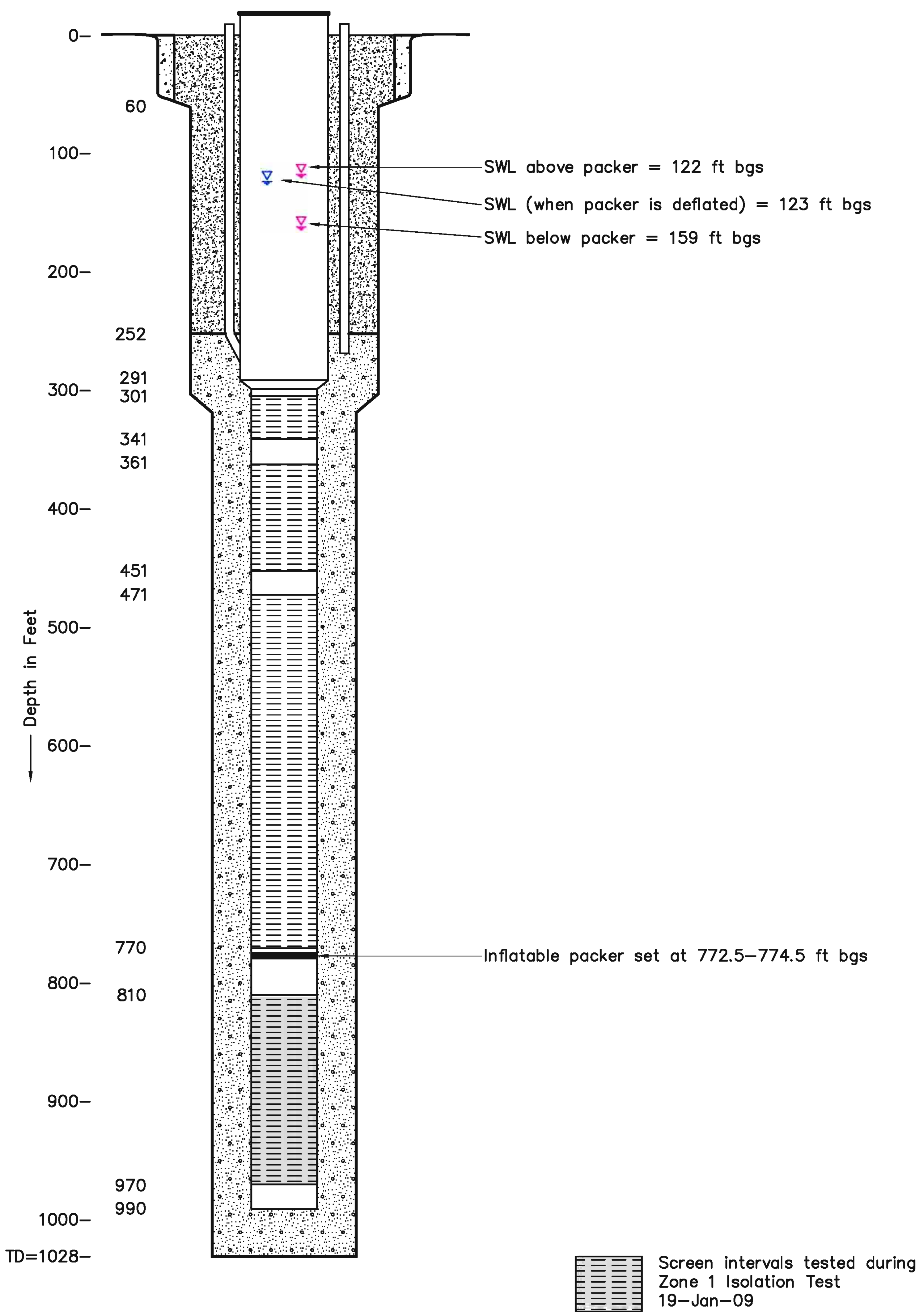
From January 15 to January 18, 2009, the packer and test pump was lowered within the well to a depth of 772.5-774.5 feet bgs and with the pump intake located at 784 feet bgs (see Figures 2-25 and 2-26). The Zone 2 isolation test was conducted January 19, 2009 and testing consisted of inflating the packer to 330 psi[10], approximately 50 psi above the background pressure. The screen interval tested was 810-970 feet bgs (totaling 160 feet), removing the uppermost screens (301-341 feet, 361-451 feet and 471-770 feet bgs) from production.

Immediately following inflation of the packer. the water level in the zone beneath the packer dropped nearly 35 feet before stabilizing at 159 feet bgs, while the static water level above the packer did not change appreciably at 123 feet bgs. The initial discharge rate of 250 gpm was increased to a maximum of 300 gpm due to significant drawdown on the zone. A targeted discharge rate of 300 gpm was held for the remainder of the test. Initially, the discharge contained a large amount of suspended fine sand, silt, and clay that cleared as pumping continued. After 6 hours of continuous pumping, the drawdown at the targeted discharge rate was more than 104 feet, resulting in a calculated specific capacity of less than 3 gpm/foot. Water quality samples were again collected for general mineral and physical properties, inorganics, volatile

10 Background pressure for Zone 2 is calculated by subtracting the static water level from the depth of the packer divided by 2.31 ft/psi, or (772-123) ÷2.31=281 psi. The packer pressure was then increased by at least 60 psi to ensure proper sealing of the packer within the well casing at this depth.

organic compounds, total organic carbon (TOC), and perchlorate before the pump was turned off and recovery levels were monitored. Once the zone had fully recovered, the packer was deflated and water levels returned to their pre-inflation point.

The complete results of the Zone 2 water quality analyses are shown in Appendix A. At the time of sample collection, the water was clear and with a field turbidity of 1.31 nephelometric turbidity units (NTU).




WELL CROSS SECTION

Figure 2–25

| | |
|---|---|
| Drawn: | PLP |
| Checked: | |
| Approved: | |
| Date: | 18–MAR–09 |

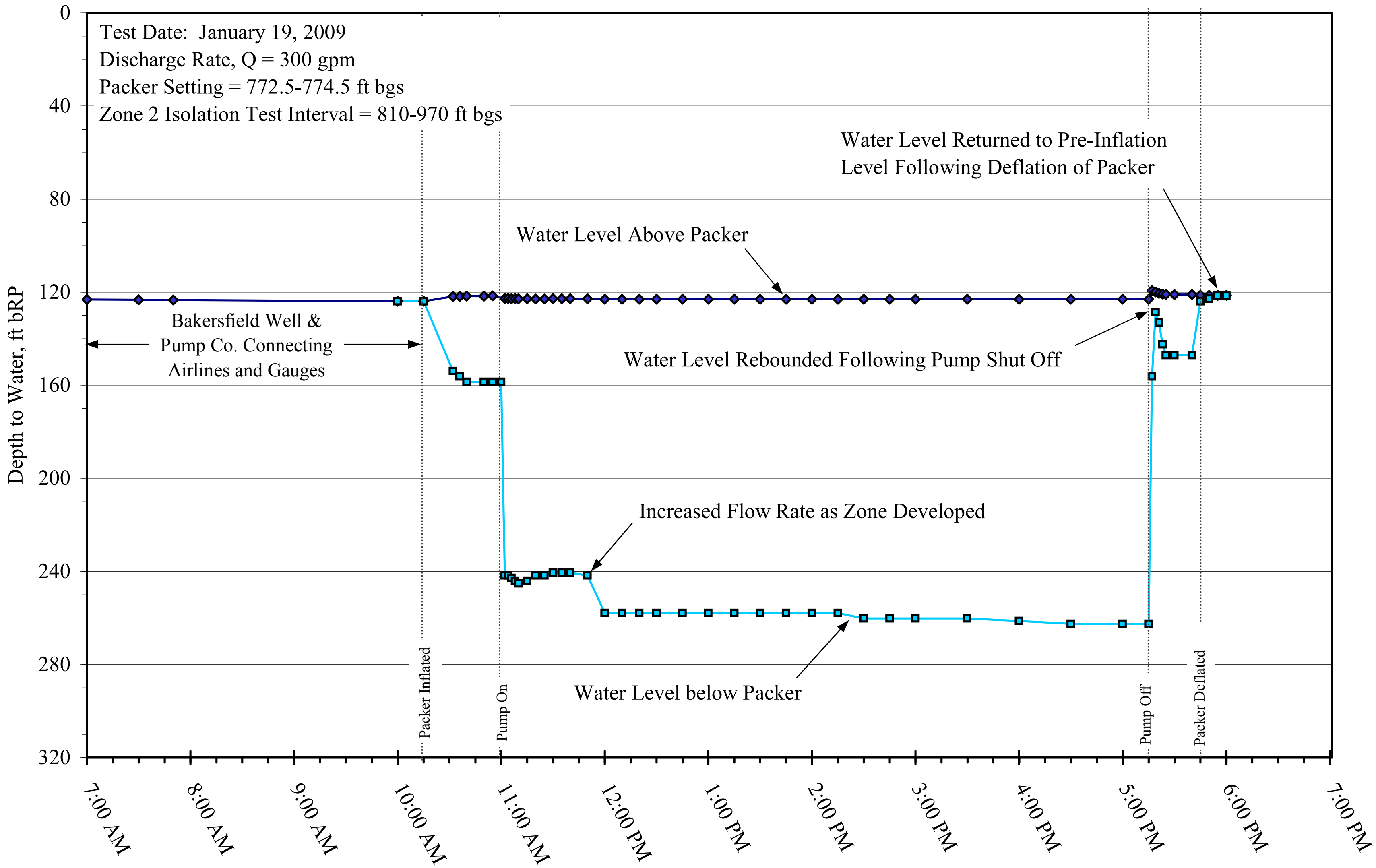
RBF CONSULTING / IRVINE RANCH WATER DISTRICT

**ZONE 2 ISOLATION TEST**
**IRWD WELL NO. 22**

GEOSCIENCE
GEOSCIENCE Support Services, Incorporated
P.O. Box 220, Claremont, CA 91711
Tel: (909)451–6650 Fax: (909)451–6638
www.gssiwater.com

This page left intentionally blank.

**Figure 2-26: Isolated Aquifer Zone Testing Timeline Analysis**
**Zone 2 Isolation Test - IRWD Well No. 22**




GEOSCIENCE Support Services, Inc.

2.10.3.3 Analysis of Zone Isolation Pumping Test

Based on analysis of the isolated pumping tests, it is concluded that there is communication between the upper and lower aquifers with 200-300 gpm being lost to the lower zones while the well is not being pumped (i.e., when it is idle.) The amount of loss will diminish during pumping, particularly as the pumping water level approaches the static water level of the lower zone.

The middle and lower screens combined (i.e., 471-770 and 810-970 feet bgs) produce a substantial amount of the flow contribution to the well as they were pumped during the Zone 1 isolation packer test at more than 1,400 gpm. The constant rate pumping test averaged 1,809 gpm for the entire screened interval in the well.

2.10.4 Review of 1992 Spinner Survey Data

At the end of the May 15-17, 1992 constant rate pumping test, a spinner survey (using both down run and stop count data) was conducted by Welenco at a discharge rate of 1,248 gpm. The results showed that 38.4 percent of the total flow was being produced by the interval 301-341 feet, 18.4 percent was produced by the interval 361-451 feet bgs, 36.0 percent was produced by the interval 471-770 feet bgs, and 7.2 percent was produced by the interval 810-970 feet bgs. Depth specific samples were also obtained from the well at depths of 305 feet, 517 feet, and 907 feet bgs, and nitrate concentrations (as NO3) were recorded from the well depths at 52, 45 and 52 mg/L respectively, in addition to a composite sample reported as 47 mg/L (as NO3) from the full discharge. The results of the May 1992 depth specific water quality samples showed the water quality to be consistent in all zones throughout the well. In contrast, the Zone 3 aquifer isolation test (295-310 feet bgs) performed in the open borehole was reported to have a very low nitrate concentration of 16 mg/L (as NO3) as compared to nearly 47 mg/L (as NO3) reported in the completed well.

A review of the 1992 spinner survey down runs that were performed at the end of the constant rate pumping test showed very little flow being contributed from zones below 720 feet bgs. The most significant flow contribution in the well is from the 301-315 foot interval that corresponds to a zone of very high resistivity on the electric log, suggesting highly permeable material. Smaller amounts of flow are contributed from the 350-370 foot and 390-410 foot intervals, also corresponding to intervals of increased resistivity on the electric log.

The stop counts obtained in 1992 show the most significant flow contribution is from the interval 300-320 feet bgs (the top of the uppermost screened interval). At depth, small amounts of flow were detected in the intervals from 420-430 feet, 500-530 feet, and 620-630 feet. A small amount of flow may have been leaving the well at a depth of 720-730 feet indicated by declining stop counts reflecting a loss of approximately 150 gpm; however, this has not been verified. No flow was detected below 820 feet (i.e., below the top of the lowermost screen interval). Intervals showing substantial flow contribution corresponded to the higher resistivity zones on the electric log, as was observed in the spinner survey down run log.

In summary, following the recent well rehabilitation and development, the results of the aquifer testing conducted by GEOSCIENCE are more favorable than that of the 1992 testing. Specific capacities, flow rates, and well efficiency are all increased over the 1992 results despite the well remaining idle for more than 16 years.

It should be noted that a dynamic spinner survey with stop counts could not be performed during the January 2009 constant rate pumping test due to the inflatable packer that was installed on the pump column within the well. The outside diameter of the packer measured 14 inches in diameter while the well casing and screen measures 16 inches in diameter. The resulting 1 inch of annular space was not enough room for the 1-11/16 inch diameter spinner tool to pass.

## 2.11 Well 22 Pump Design Recommendations

Based on the pumping test data found in Appendix A, a design discharge rate of 1,600 gpm is recommended for Well 22. At this discharge rate, and assuming current water level conditions, a short-term (i.e., 1 day) drawdown of approximately 79 feet is expected with a total lift (to the land surface) of 226 feet including 18 feet of estimated interference from Well 21 pumping at an estimated discharge rate of 3,300 gpm (see Table 2-14). The long-term drawdown (i.e., after 1 year of continuous pumping) is estimated to be approximately 109 feet with a total lift to the land surface of 289 feet including an estimated 51 feet of interference from Well 21 when pumping at 3,300 gpm.

With the current static water level of 129 feet bgs, the specific capacity of IRWD Well 22 is approximately 20 gpm/feet at the design discharge rate of 1,600 gpm (see Figures 2-19 and 2-21). It is recommended that the pump intake be set at a depth of approximately 460 feet bgs, within a blank section between screens in the 16-inch diameter portion of the well. Table 2-14 summarizes the recommended pump design based on the current depth to static water level of 129 feet bgs.

**Table 2-14 IRWD Well 22 - Pump Design Recommendations**

| Parameters | Short Term (1 day) | Long Term (1 year) |
|---|---|---|
| Design Pumping Rate | 1,600 gpm | 1,600 gpm |
| Design Drawdown | 79 ft | 109 ft |
| Design Well Efficiency | 88% | 88% |
| Pump Setting | 460 ft bgs | 460 ft bgs |
| Estimated Interference When IRWD Well 21 is Pumping at 3,300 gpm[11] | 18 ft | 51 ft |
| Static Water Level Depth (without Interference) | 129 ft bgs | 129 ft bgs |
| Total Lift to Surface (does not include regional decline in static water level) | 226 ft | 289 ft |

## 2.12 Well 22 Ground Water Quality

Water samples were collected toward the end of the 24-hour constant rate pumping test and were submitted by GEOSCIENCE Support Services, Inc. personnel to E.S. Babcock & Sons, Inc. of Riverside, California, for State of California Code of Regulations Title 22.

Additionally, water quality samples were collected at the end of the 6 hours of continuous pumping in Zones 1 and 2 and were submitted to E.S. Babcock & Sons for analysis of general mineral and physical properties, VOCs, metals, TOC, and perchlorate.

As is the case with Well 21, water quality results of the constant rate test, and the Zone 1 and Zone 2 isolation tests conducted on Well 22, indicate that ground

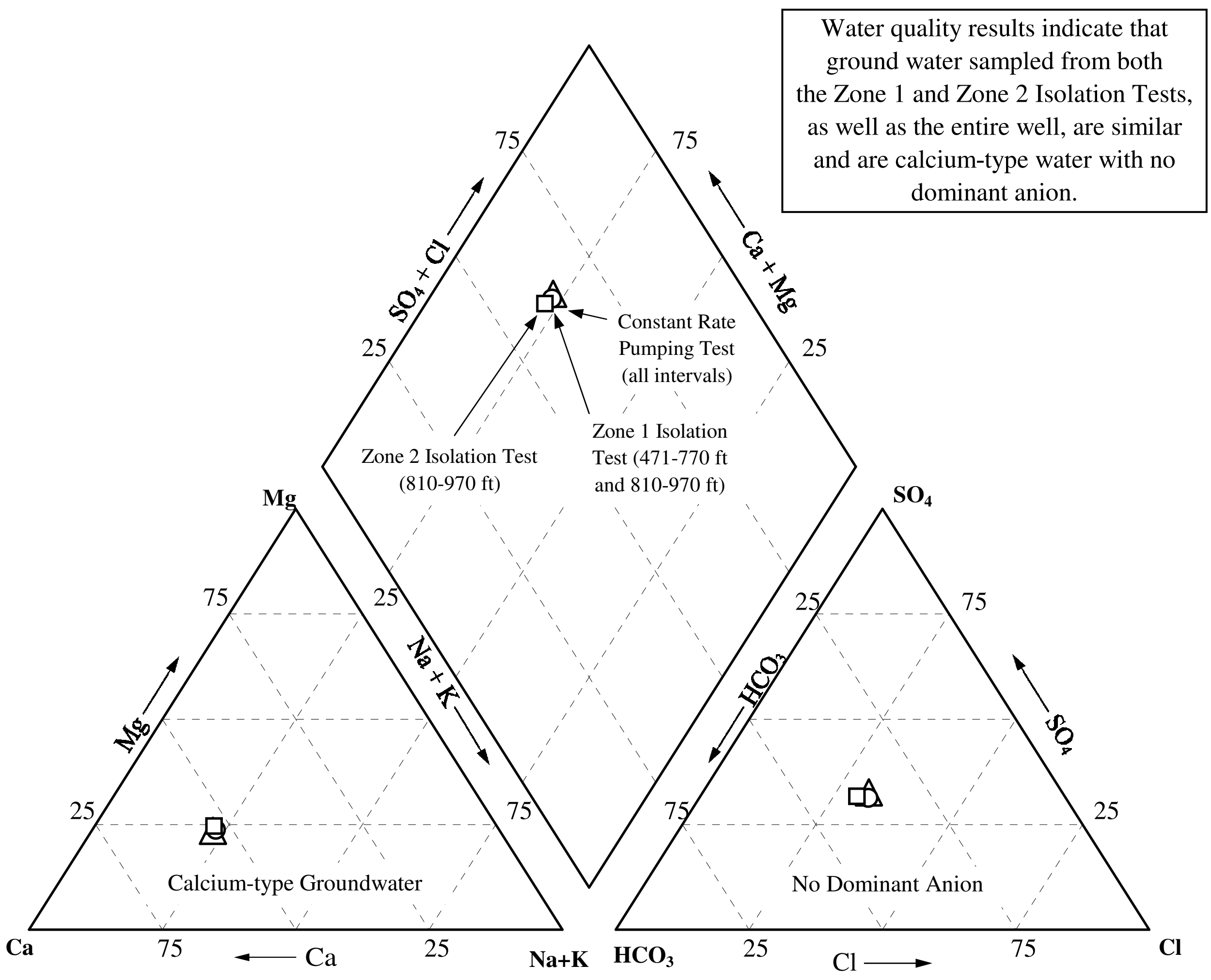
11 Interference estimated for IRWD Well 21 pumping with an assumed discharge rate of 3,300 gpm and assuming average well field parameters.

water from all three sampling events are calcium-type groundwater with no dominant anion. The results are presented in the tri-linear diagram shown as Figure 2-27.

Nitrate (as $NO_3^-$) at Well 22 was detected in the constant rate test (i.e., Title 22 sampling event), and the Zone 1 and Zone 2 sampling events at concentrations of 50, 49, and 54 mg/L respectively. These concentrations exceed the State of California Department of Public Health (CDPH) maximum contaminate level (MCL) of 45 mg/L for nitrate. Perchlorate showed a result of non-detect (i.e., less than the reportable detection limit, or RDL of 4.0 µg/L) in all three sampling events. The iron concentrations in all three sampling events showed a result of non-detect (i.e., less than the RDL of 100 µg/L). Total hardness was detected in the constant rate test as well as Zone 1 and 2 at concentrations of 430, 410, and 440 µg/L respectively. Although hardness currently has no MCL, these concentrations are significantly elevated over what is typically found in acceptable water sources. Total dissolved solids (TDS) was detected at elevated concentrations during the constant rate test and Zone 1 and Zone 2 pumping test at concentrations of 650, 760, and 710 mg/L, respectively, exceeding the lower limit for the CDPH secondary MCL of 500 mg/L but below the maximum CDPH secondary MCL of 1,000 mg/L.

At the end of the Well 22 constant rate pumping test, the SDI was measured as 0.98 and the corresponding total silica was reported as 26 mg/L. All other constituents analyzed were found to be within the recommended drinking water standards. The complete results of the constituents analyzed are contained in Appendix A; however, selected results are summarized in Table 2-15.

**Figure 2-27: Trilinear Diagram**
**Water Quality Data - IRWD Well No. 22**
**Constant Rate Pumping Test and Zone Isolation (Packer) Tests**




Water Quality Results from E.S. Babcock & Sons, Inc. of Riverside, California

**Table 2-15 IRWD Well 22 - Summary of Title 22 Water Quality Analyses**

| Constituents | | Constant Rate Test<br><br>Screened interval: 301-341, 361-451, 471-770, 810-970 [ft bgs] | Isolated Zone 1<br><br>Screened interval: 471-770, 810-970 [ft bgs] | Isolated Zone 2<br><br>Screened interval: 810-970 [ft bgs] | Drinking Water Regulatory Standards |
|---|---|---|---|---|---|
| Arsenic | [µg/L] | < 2 | - | - | 10[1] |
| Boron | [µg/L] | 110 | - | - | 1,000[3] |
| Chromium (Total) | [µg/L] | 2.6 | - | - | 50[1] |
| Chloride | [mg/L] | 120 | 120 | 110 | 250-500[2] |
| Fluoride (Total) | [mg/L] | 0.2 | - | - | 2.0[1] |
| Hardness | [mg/L] | 430 | 410 | 440 | NA[6] |
| Iron | [µg/L] | < 100 | < 100 | < 100 | 300[2] |
| Manganese | [µg/L] | ND | ND | ND | 50[2] |
| Nitrate (as $NO_3$) | [mg/L] | 50 | 49 | 54 | 45[1] |
| Perchlorate | [µg/L] | < 4.0 | < 4.0 | < 4.0 | 6.0[1] |
| Specific Conductance | [umhos/cm] | 1,100 | 1,000 | 1,100 | 900-1,600[2] |
| Sulfate (as $SO_4$) | [mg/L] | 170 | 160 | 160 | 250-500[2] |
| 1,2,3-Trichloropropane | [µg/L] | <0.005 | - | - | 5[3] |
| Total Dissolved Solids, TDS | [mg/L] | 650 | 760 | 710 | 500 - 1,000[2] |
| Total Organic Carbon (TOC) | [mg/L] | 0.31 | < 0.30 | < 0.30 | NA[6] |
| Total Silica | [mg/L] | 26 | - | - | NA[6] |
| Total Hardness | [µg/L] | 430 | 410 | 440 | NA[6] |
| Vanadium | [µg/L] | < 3.0 | - | - | 50[3] |
| Volatile Organic Chemicals (EPA Method 524.2) | [µg/L] | ND | ND | ND | Varies with Compound |
| Gross Alpha | [pCi/L] | 1.49 | - | - | 15[1] |
| Color | [color unit] | < 3 | < 3 | < 3 | 15[2] |
| Odor | [TON] | < 1 | < 1 | < 1 | 3[2] |
| pH | [std. units] | 7.5 | 7.5 | 7.4 | 6.5 - 8.5[5] |
| Turbidity | [NTU] | 0.26 | 0.25 | 0.83 | 5[2] |
| Silt Density Index, SDI | - | 0.98 | - | - | NA[6] |

1 California Department of Public Health (CDPH) Primary Maximum Contaminant Levels (MCL).
2 CDPH Secondary MCLs.
3 CDPH notification level for unregulated chemicals requiring monitoring.
4 Hexavalent chromium is currently regulated under the 50-microgram per liter (□g/L) MCL for total chromium.
5 USEPA recommended range for pH.
6 Not applicable, no current MCL.
ND Not detected – used only regarding VOCs not located in the above table.
BOLD Equal to or above current CDPH MCLs or notification levels.

Data collected during the May 1992 tests is compared to 2009 water quality results on the tri-linear diagram shown on Figure 2-28. The general water chemistry is very similar between that of the 1992 open borehole zone testing and the 2009 constant rate pumping and zone isolation tests.

## 2.13 Well 22 Final Downhole Video Survey and Disinfection

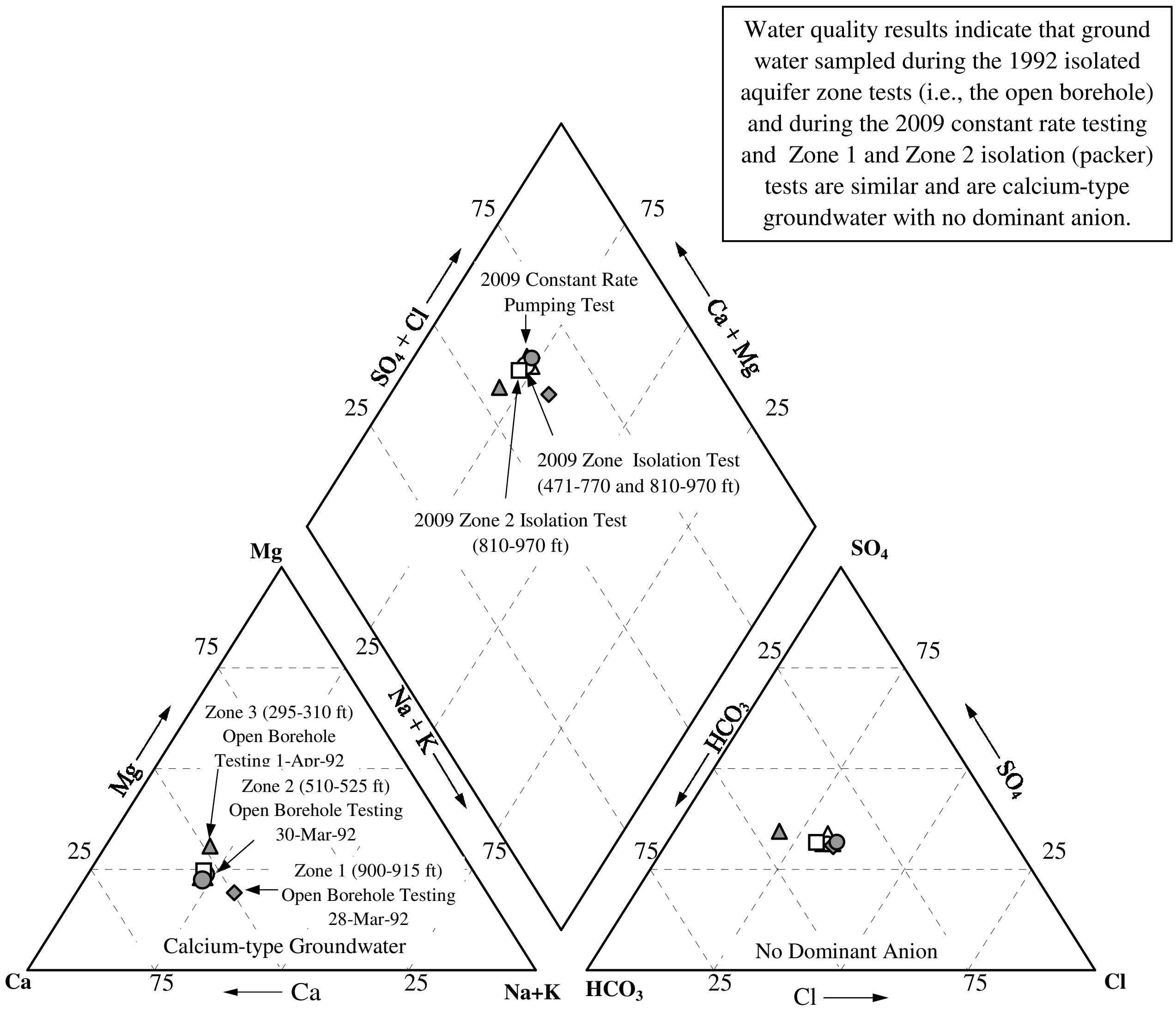
On January 30, 2009, Pacific Surveys completed a final video survey to document the post-rehabilitation condition of the well. The video survey showed that in the upper portions of the well a small amount of particles were settling out on the transition section between the 20- and 16-inch casings located from 291-296 feet bgs. The water column was clear throughout the well. The well was observed to be structurally sound and the screened intervals were observed to be very clean and open. Some light to moderate staining was observed on the well casing and screen at some of the spiral welds and heat affected zone within the casing, particularly where tubercular deposits were removed during the rehabilitation process. At 340 feet, water downward movement was observed that slowed at a depth of approximately 488 feet. At a depth of 670 feet, there was a slight amount of pitting observed where a weld on the exterior of the casing had caused some corrosion. At 986 feet, brown staining appearing to be a “silt ring” was observed on the interior of the sump that is likely where sediment had remained in contact with the casing for an extended period of time. Appendix B contains a copy of the post-rehabilitation video survey report that was provided by Pacific Surveys.

In addition to the post-rehabilitation video, a pre-rehabilitation video survey was conducted on January 17, 2008, and a post-brushing, swabbing, and airlifting video survey was conducted on November 17, 2008. Copies of each video survey report are found in Appendix E.

On February 13, 2009, Well 22 was disinfected by adjusting the pH in the water column to 5 pH units using 15 gallons of NW-410 product before injecting 15 gallons of 12.5 percent sodium hypochlorite through a tremie pipe at 100 foot intervals. The water column was thoroughly agitated using a bailer before testing the chlorine concentration at 100 mg/L.

During final demobilization and clean up of the site, BW&P personnel welded the casing extensions back on the top of the well casing as well as the gravel feed pipe and sounding tube. At the conclusion of the work, a cover was welded on top of the 20-inch well casing and threaded caps were tightly fastened to the tops of the gravel feed pipe and the sounding tube.

**Figure 2-28: Trilinear Diagram**
**Comparison of 2009 and 1992 Water Quality Results**
**IRWD Well No. 22**




2009 Water Quality Results are from E.S. Babcock & Sons, Inc. of Riverside, California. 1992 Results are from IRWD Michelson Water Reclamation Plant Laboratory.

## 2.14 Well 22 Conclusions and Recommendations

### 2.14.1 Conclusions

Well 22 is a multi-aquifer completed well (i.e., the well is screened in multiple aquifers) and based on the testing and analysis, vertical flow may be occurring from the shallow to the deeper zones. The amount of flow can be quantified (if necessary) with a static (non-pumping) flowmeter survey. The amount of downward flow of 200 to 300 gpm was predicted based on results of isolated zone (packer) testing and transmissivity of aquifers.

The design discharge rate was increased over that of the 1992 recommendations and is 1,600 gpm versus 1,250 gpm (increase of 350 gpm). The efficiency of the well is 88 percent with 9 feet of the total drawdown attributed to well losses and 70 feet attributed to formation losses. Rehabilitation has restored the well to its original capacity.

The specific capacity of Well 22 following rehabilitation was improved over the 1992 results (after construction of the well) as shown in Table 2-16.

**Table 2-16 IRWD Well 22 - Restored Specific Capacity Following Rehabilitation**

| 2009 Result Following Rehabilitation (at end of each step) | | | 1992 Result Following Initial Construction, Development and Testing (at end of each step) | | |
|---|---|---|---|---|---|
| Discharge Rate, Q [gpm] | Drawdown [ft] | Specific Capacity, Q/s, [gpm/ft] | Discharge Rate, Q [gpm] | Drawdown [ft] | Specific Capacity, Q/s, [gpm/ft] |
| **850** | **34.67** | **25** | **633** | **29.4** | **21.5** |
| **1,613** | **70.72** | **23** | **1,017** | **47.6** | **21.4** |
| **2,227** | **101.82** | **22** | **1,260** | **60.43** | **20.9** |
| **-** | **-** | **-** | **1,858** | **92.58** | **20.1** |
| **Design = 1,600** | **79** | **21** | **Design = 1,250** | **66** | **18.9** |

The large masses of tubercles that were observed in January 17, 2008 video survey have been completely removed from within the screened intervals and blank sections of casing within the well.

Well 22 was observed to be in sound structural condition after nearly 17 years of idle time. Sand production during the pumping tests was minimal and a large quantity of bentonite mud and other fine grained sediments were removed during the rehabilitation process.

Nitrate and TDS concentrations in Well 22 have not increased significantly since the well was drilled in 1992 although both constituents currently exceed their respective MCLs (Table 2-17). Hardness is currently also unacceptably high.

The perchlorate concentration was reported as non-detect in all tests. The laboratory's reportable detection limit (RDL) was 4.0 µg/L.

Basic water chemistry in terms of cations and anions has remained the same from 1992 to 2008 (see Figure 2-28).

**Table 2-17 IRWD Well 22 - Comparison of 2008 and 1992 Depth Specific Water Quality Results**

| 2009 Water Quality Results | | | | 1992 Water Quality Results | | | |
|---|---|---|---|---|---|---|---|
| **Event/Depth** | **Nitrate ($NO_3$) [mg/L]** | **TDS [mg/L]** | **Hardness [mg/L]** | **Event/Depth** | **Nitrate ($NO_3$) [mg/L]** | **TDS [mg/L]** | **Hardness [mg/L]** |
| - | - | - | - | Zone 1 Open Borehole 830-845 ft | 45.6 | 718 | - |
| - | - | - | - | Zone 2 Open Borehole 565-580 ft | 56.7 | 724 | - |
| - | - | - | - | Zone 3 Open Borehole 300-315 ft | 16.4 | 712 | - |
| Constant Rate Pumping Test (301-970 ft) | 50 | 650 | 430 | Constant Rate Pumping Test (301-970 ft) | 47.4 | 620 | - |
| Zone 1 Isolation Test (539-1,060 ft) | 49 | 760 | 410 | Depth Specific Sampling at 305 ft | 47.4 | 620 | - |
| Zone 2 Isolation Test (819-1,060 ft) | 54 | 710 | 440 | Depth Specific Sampling at 517 ft | 44.7 | 588 | - |
| - | - | - | - | Depth Specific Sampling at 907 ft | 52.3 | 640 | - |

The amount of interference created by Well 21 pumping at 3,300 gpm was estimated as 51 feet under long-term pumping conditions and is based on average well field parameters. In effect, the static water level in Well 22 will be lowered by this amount when Well 21 is being pumped at the design discharge rate.

The silt density index (SDI) was measured in the field at the end of the constant rate pumping test as 0.98.

### 2.14.2 Recommendations

An operational schedule of 24 hours per day, 7 days per week for a period of 11 months per year to maximize production is acceptable; however, it is important that the well is turned off for short time each year to allow full recovery. It is not desirable for a production well to be operated continuously year after year without being allowed to recover.

If Well 22 is to remain idle and unequipped for more than a period of 5 years, a limited rehabilitation program is recommended prior to placing the well in service. As a minimum, the well should be videoed and thoroughly brushed before being equipped. Brushing should be accomplished using a new nylon or polypropylene bristle brush and should take place for at least 1 minute per wetted foot of screen, and 1/2 minute per wetted foot of blank casing with the brush measuring approximately 1/2 inches larger in diameter than the casing. A weighted core will ensure passage downward through the screen. The use of a “wire” brush should not be allowed. Accumulated fill or sediment should be bailed prior to installing a test pump.

The well should be thoroughly re-developed by alternating pumping with surging (starting at low intensity and working upward), before conducting step drawdown and constant rate pumping tests for comparison with previous testing to ensure that no loss in specific capacity has occurred. Should there be a significant reduction in the specific capacity additional re-development efforts may be required. All work should be followed by a video survey and final disinfection.

If well is idle 10 or more years, it should be brushed as described above followed by airlifting and swabbing to stress each 10-foot interval of screen. Airlifting and swabbing should take place for a minimum of 1/2 hour per 10-foot interval. If sand, silt, or cloudy water is produced, the length of time spent in airlifting and swabbing should be increased until clear or nearly clear water is achieved. A test pump is recommended to be installed for re-development and testing as described above, with test results compared against previous results. If the specific capacity is less than desirable, further rehabilitation steps (such as focused airlifting and swabbing using dispersants and surfactants followed by additional development pumping) should undertaken to restore capacity. All work within the well should be followed by a video survey and final disinfection. A summary of the recommended actions for Well 22 is provided in Table 2-18.

**Table 2-18 Recommended Redevelopment Based on 2, 5 and 10 Year Idle Periods**

| Description of Work | After 2 Years Idle | After 5 Years Idle | After 10 Years Idle |
|---|---|---|---|
| Perform Pre- Video Survey | Not Recommended | √ | √ |
| Brushing with Nylon or Polypropylene Brush Followed by Bailing | Not Recommended | √ | √ |
| Airlifting and Swabbing at 10 ft Intervals Throughout Screen | Not Recommended | Not Recommended | √ |
| Installation of Test Pump Followed by Development Pumping and Removal of Test Pump Equipment | Not Recommended | √ | √ |
| Perform Step Drawdown and Constant Rate Pumping Tests | Not Recommended | √ | √ |
| Perform Post- Video Survey | Not Recommended | √ | √ |
| Perform Final Disinfection | Not Recommended | √ | √ |

# Section 3

# Blending and Treatment Evaluations

This page left intentionally blank.

# Section 3 Blending and Treatment Evaluations

## 3.1 General

Section 2 describes the means and methods used to rehabilitate Wells 21 and 22, their recommended capacities, and the general water quality gathered during pump testing. This section evaluates the results from this sampling, along with the historical water quality from Wells 21 and 22, along with several nearby wells to gain an understanding of the current water quality and potential future trends in contaminant concentrations. This will be compared to state and federal maximum contaminant limits (MCL) as well as IRWD's water quality goals.

Those constituents that do not meet regulated standards or IRWD's water quality goals will require some method of treatment prior to introduction into IRWD's system. This section will evaluate blending with existing potable supply sources and the treatment processes available to reduce the targeted contaminant concentrations to meet the stated water quality goals.

## 3.2 Water Quality Evaluation

### 3.2.1 Test Protocol and Water Quality Analysis

During pump testing in late 2008/early 2009, water samples were collected from Wells 21 and 22 near the end of the 24-hour constant rate pumping tests and were submitted by GEOSCIENCE Support Services, Inc. personnel to E.S. Babcock & Sons, Inc. of Riverside, California, for State of California Code of Regulations Title 22 testing. Title 22 requires analysis for the following constituents:

- Cations
- Anions
- Aggregate properties
- Solids
- General physical
- Surfactants
- General inorganics
- Nutrients
- Metals and metalloids
- Asbestos
- Gross alpha (radioactivity)
- VOCs (EPA Method 524.2 and MTBE)
- EDB & DBCP (EPA Method 504.1)

- Nitrogen-phosphate pesticides (EPA Method 507)
- Organic pesticides and PCBs as DCP (EPA Method 508)
- Chlorinated herbicides (EPA Method 515.3)
- Semivolatile compounds (EPA Method 525.2)
- Carbamates (EPA Method 531.1)
- Glyphosate (EPA Method 547)
- Endothall (EPA Method 548.1)
- Diquat (EPA Method 549.2); and
- Dioxin (2, 3, 7, 8 - TCDD) (EPA Method 1613)

Also, additional analysis included:

- 1,2,3-Trichloropropane (1,2,3-TCP) by purge and trap GC/MS
- Boron
- Hexavalent chromium (CrVI)
- Perchlorate
- Total silica
- Asbestos
- Vanadium

Additionally, water quality samples were collected at the end of the 6 hours of continuous pumping in Zones 1 and 2 isolation tests and were submitted to E.S. Babcock & Sons for analysis of general mineral and physical properties, VOCs, metals, TOC, and perchlorate.

The detailed results for all water quality constituents tested under this protocol are included in Appendix A. A summary of the key constituents' concentrations from the composite well samples taken at Wells 21 and 22 are shown in Table 3-1.

**Table 3-1 Summary of Key Constituents' Concentrations**

| Constituents | MCL | Sources [3] | |
|---|---|---|---|
| | | Well 21 [4] | Well 22 [5] |
| Total Hardness | N/A | 500 | 430 |
| TDS | 500 mg/L [1][2] | 740 | 650 |
| Nitrate (as $NO_3^-$) | 45 mg/L | 67 | 50 |

[1] Constituent concentrations ranging to the upper contaminant level are acceptable if it is neither reasonable nor feasible to provide more suitable waters.

[2] The upper contaminant level for TDS is 1,000 mg/L.

[3] Values are in mg/L (unless noted otherwise).

[4] Based on water quality results of the Well 21 composite sample, which was collected during the Well 21 constant-rate pumping test (November 2008).

[5] Based on water quality results of the Well 22 composite sample, which was collected during the Well 22 constant-rate pumping test (January 2009).

### 3.2.2 Historical Water Quality and Prediction Analysis

The water quality in this basin was analyzed over time by reviewing Wells 21 and 22 and other nearby wells' historical data for key constituents. The ground water in the vicinity of IRWD Wells 21 and 22 has periodically exceeded the MCL of several water quality parameters. The constituents of greatest concern in the area are nitrate (reported as $NO_3^-$), total dissolved solids (TDS), perchlorate, and hardness (as $CaCO_3$). In general, the concentrations of these chemicals have not changed significantly over time with the exception of nitrate, which has increased in several of the wells over time. The purpose of this analysis is to examine ground water quality trends in the area to assess future ground water quality trends at Wells 21 and 22.

#### 3.2.2.1 Well Selection for Water Quality Trends

Historical ground water quality and ground water level data were provided by the Orange County Water District (OCWD) for 13 wells in the vicinity of Wells 21 and 22 (see Figure 3-1). Of these 13 wells, five were selected for being most representative of the ground water quality in the area. These five wells include:

- T-MS3 (City of Tustin Main Street Well No. 3)
- T-MS4 (City of Tustin Main Street Well No. 4)

- T-PANK (City of Tustin Pankey Well)
- T-WALN (City of Tustin Walnut Well)
- SA-31 (City of Santa Ana Well No. 31)

These wells were selected because they collectively represent the general area surrounding Wells 21 and 22 and have both the longest and the most recent periods of record. All wells are active with the exception of Well 21, Well 22, and T-PANK. All of these wells are located within a 1-mile radius of Wells 21 and 22 with the exception of SA-31, which is approximately 1 ¼ miles to the northwest. In addition to these five selected wells, the data received for all of the wells in the vicinity of Wells 21 and 22 was examined for information that supported or opposed the analysis in this report and is included graphically in Appendix K in the form of hydrographs and water quality trends. This also contains additional hydrographs and water quality trends for wells in the area that were not the primary focus of this investigation.

#### 3.2.2.2 Historical Water Quality Prediction Analysis

##### 3.2.2.2.1 Nitrate

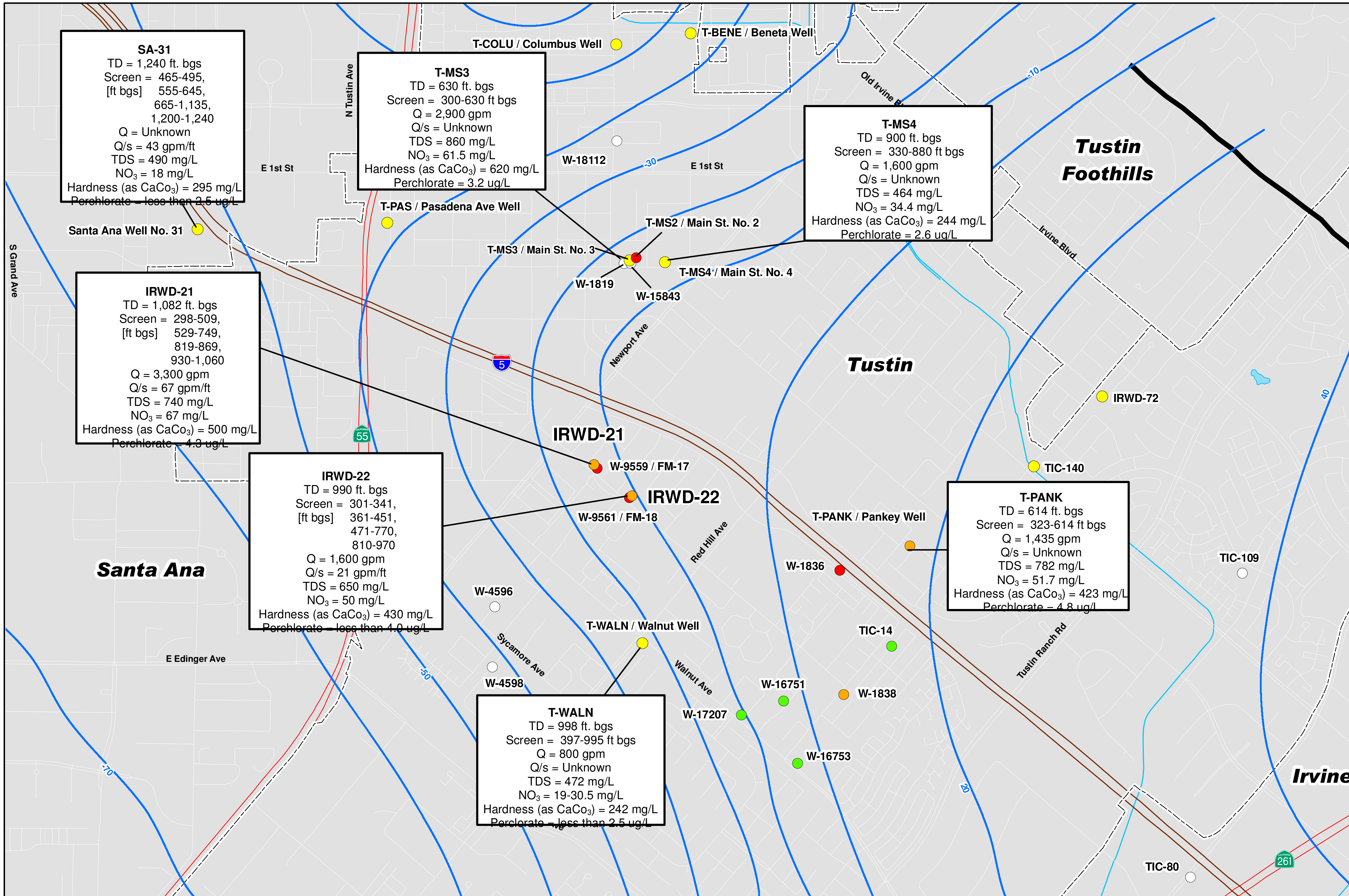
The primary MCL for $NO_3^-$ is 45 mg/L[1]. Two nearby wells to Wells 21 and 22 have experienced an increase in $NO_3^-$ concentrations during their periods of record while three wells have remained stable. Several nearby wells have nitrate levels that are approximate to or exceed the MCL.

T-WALN (located 2,600 feet south of Well 21) experienced increases in nitrate concentrations (as NO3) from approximately 18 mg/L in mid-1987 to 30 mg/L in September 2008, before declining to 19 mg/L in November 2008. During this time period, a maximum concentration of 38 mg/L was recorded in August 2003.

[1] California Department of Public Heath (CDPH) Primary Maximum Contaminate Level.

RBF CONSULTING /
IRVINE RANCH WATER DISTRICT

HISTORICAL WATER QUALITY AND WATER LEVEL
TRENDS IN THE VICINITY OF IRWD WELL NOS. 21 AND 22




T.5 S
R.9 W.

LOACTION AND STATUS OF WELLS INCLUDING WATER QUALITY AND GROUNDWATER ELEVATIONS

~~EXPLANATION~~

Well Classification

- Active
- Inactive
- Abandoned
- Destroyed
- Other or Unknown

10 Groundwater Contour OCWD, June 2007

Inferred Fault (source: U.S.G.S. Open-File Report 2006-1217. Geologic Map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California.)

City Boundary

Freeway / Interstate

State Highway

white line Street

Flood Control Channel

3-Apr-09

Prepared by: DWB

Map Projection: State Plane, Zone 6 (1983)

NORTH

0 750 1,500 Feet

GEOSCIENCE
GEOSCIENCE Support Services, Inc.
P.O. Box 220, Claremont, CA 91711
Tel: (909) 451-6650 Fax: (909) 451-6638
www.gssiwater.com

Figure 3-1

GIS_proj/rbf_irwd_wq_wl_well21-22_3-09/0_Fig_1_wq_wl_Well21-22_4-09.mxd

This page left intentionally blank.

T-PANK (located 4,500 feet southeast of Well 21) has also exhibited a pattern of increase over the same period, that is, from 1987 to 1998, when testing ceased. During that time, the nitrate concentrations increased from 35 mg/L to 52 mg/L at a rate of approximately 1.4 mg/L per year. Currently T-PANK is inactive, thus current measurements of nitrate concentrations are not available.

T-MS3 (located 3,000 feet west of Well 21) has had no significant change in nitrate concentration over the time interval 1987 to 2008, and has had nitrate concentrations that exceed the MCL, generally fluctuating between approximately 40 to 80 mg/L, with the exception of one anomaly of 302 mg/L that was reported in July 1995. T-MS4 (located 750 feet to the east of T-MS3) has also not shown a significant change in nitrate concentration from 1999 to 2008; however, it also has consistently had nitrate concentrations that exceeded the MCL, with fluctuations from approximately 20 mg/L to 53 mg/L.

SA-31 is located approximately 1 ¼ miles down-gradient of Wells 21 and 22 and has also shown no overall positive or negative trend in nitrate concentration, and has ranged from 14 mg/L to 31 mg/L since 1987.

Predicting future nitrate concentrations in Wells 21 and 22 is complicated by the amount of spatial variability exhibited in nitrate trends by the nearby wells. While two wells that are adjacent or up-gradient display increasing trends in nitrate concentration during their periods of record, three other adjacent and down-gradient wells show relatively no change over time. This assumes that the increasing trend in nitrate concentration observed in T-PANK will continue in the same pattern that has been observed from the mid to late 1990s, when data was last available, to the present. Well 21 displays an increase in nitrate of 1.5 mg/L per year based on two data points. The concentration was 43 mg/L in 1992 and 67 mg/L in 2008. Well 22 displays a 3 mg/L increase during the same period of record and two data points. It had a concentration of 47 mg/L in 1992 and 50 mg/L in 2008, for an increase of 0.2 mg/L per year.

The highest nitrate concentration reported in the area is 80 mg/L found at T-MS3. This well had concentrations ranging from 72 mg/L to 76 mg/L reported during 2006 and 2007, and concentrations of 61 mg/L to 73 mg/L reported during 2008. From this data, it is assumed that 80 mg/L will be the maximum level experienced in the future.

Additionally, there is no evidence to suggest that nitrate concentrations are affected by changes in precipitation, nor does there appear to be a correlation between water level and nitrate concentrations in any of the five wells.

3.2.2.2.2 Total Dissolved Solids

The recommended secondary limit for total dissolved solids (TDS) is 500 mg/L. All five wells near Wells 21 and 22 have TDS concentrations that are generally greater than 500 mg/L, with the exception of SA-31. T-WALN ranges from 430 to 919 mg/L; T-MS3 ranges from 532 to 906 mg/L; and T-MS4 ranges from 464 to 791 mg/L. Only T-PANK displays an increasing trend in TDS, spanning from the early 1960s to the mid to late 1990s when testing stopped. During that time, TDS concentration increased by approximately 13 to 17 mg/L per year. The four other wells with a current TDS records each display relatively stable trends in TDS concentration before and after the construction of Wells 21 and 22 in 1992. During the period of record for TDS, there have been multiple changes in precipitation that appear to have had no effect on local TDS concentrations. Additionally, there does not appear to be a correlation between water level and TDS concentrations in any of the five wells.

Therefore, the only evidence that future TDS concentrations in Wells 21 and 22 will rise is from trends ending in the mid to late 1990s in T-PANK that had an increase of up to 17 mg/L per year. However, that the four other wells displayed no increasing trend up to the present date indicates that TDS concentrations will most likely not rise in Wells 21 and 22.

3.2.2.2.3 Perchlorate

The MCL for perchlorate is 6.0 µg/L. The period of record for perchlorate testing in the area is significantly more limited than for other water quality parameters as perchlorate was not widely tested until the late 1990s. The longest periods of perchlorate testing have occurred in T-MS3 and T-WALN from 1998 to 2008. T-WALN values have typically been below detection limit (1.0 µg/L) with a maximum concentration of 4.8 µg/L recorded in 2003. Perchlorate concentrations in T-MS3 have been more consistently above the chemical detection limit to a maximum 6.7 µg/L at one point. In these two wells, perchlorate concentrations frequently fluctuate from below detection limit to more than twice the detection limit often within a span of 6 weeks. This indicates that perchlorate concentration has a large amount of temporal variability in the local ground water. T-PANK and T-MS2 do not have enough data to indicate a temporal trend. The MCL for perchlorate was exceeded only one time in T-MS3; all other reported concentrations have been below the MCL.

From this limited amount of data, there is no indication that perchlorate concentrations in Wells 21 and 22 will increase or decrease, only that there is the potential for highly variable periods. The limited data also does not allow a conclusion to be drawn whether water level has an effect on perchlorate concentrations.

3.2.2.2.4 Hardness

Hardness is a measure of the $CaCO_3$ concentration in water. A concentration of 0-60 mg/L is considered soft; 61-120 mg/L is moderately hard; 121-180 mg/L is hard; 181-250 mg/L is very hard; and any concentration greater than 250 mg/L is extremely hard (Durfar and Becker, 1964 as cited in www.USGS.org). Generally, all of the wells in the vicinity of Wells 21 and 22 have historically had $CaCO_3$ concentrations greater than 200 mg/L and range as high as 620 mg/L in T-MS3 in 2007. This well has exhibited very high concentrations and has not exhibited any steady change in hardness since testing began in 1972. T-PANK displays an increase in hardness from the early 1960s to the mid to late 1990s with ground water classified as extremely hard. Ground water in SA-31 and T-WALN has not experienced significant temporal patterns in hardness and has ranged in classification from hard to extremely hard.

Ground water hardness will most likely not rise in Wells 21 and 22 based on steady concentrations of $CaCO_3$ found in the nearby wells with current records. The only indication that hardness will increase is from trends ending in the mid to late 1990s in T-PANK (now inactive), which increased approximately 7 to 9 mg/L per year during that period.

3.2.2.2.5 Findings

- Nitrate concentrations increased in Well 21 from 43 mg/L in 1992 to 67 mg/L in 2008, yet Well 22 did not see a corresponding increase as it increased only from 47 mg/L in 1992 to 50 mg/L in 2009.

- Nitrate concentrations in the vicinity of Wells 21 and 22 generally approximate or exceed MCL. It is expected that nitrate concentrations in Wells 21 and 22 will remain constant or increase approximately up to 1.4 mg/L per year.

- The highest nitrate concentration in the area is T-MS3 reported at 80 mg/L with concentrations from 72 mg/L to 76 mg/L reported during 2006 and 2007, and levels of 61 mg/L to 73 mg/L reported during 2008.

- It is assumed that a nitrate concentration of 80 mg/L will be the maximum level experienced in Wells 21 and 22 in the future.

- TDS concentrations are not expected to rise in Wells 21 and 22.

- There is not enough perchlorate data to support a prediction of future perchlorate concentrations in Wells 21 and 22.

- Ground water hardness is generally very hard to extremely hard in the area. Hardness in Wells 21 and 22 is expected to remain fairly constant.

- There is no correlation between precipitation and the analyzed ground water quality parameters.

Table 3-2 below summarizes the water quality constituents of concern in Wells 21 and 22 as well as the five nearby wells:

**Table 3-2 Summary of Selected Water Quality Constituents**

| Well | Status | Distance to Well 21 [ft] | Data Range* | TDS [mg/L] | | Hardness, as $CaCO_3$ [mg/L] | | Nitrate, $NO_3^-$ [mg/L] | | Perchlorate [µg/L] | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Min. | Max. | Min. | Max. | Min. | Max. | Min. | Max. |
| Well 21 | Inactive | - | 1992-2008 | 600 | 740 | 475 | 500 | 43 | 67 | 4.3 | 4.3 |
| Well 22 | Inactive | 684 | 1992-2009 | 620 | 650 | 381 | 430 | 47 | 50 | < 4.0 | < 4.0 |
| T-WALN | Active | 2,600 | 1951-2008 | 430 | 919 | 199 | 303 | 13 | 36 | < 2.5 | 4.8 |
| T-MS3 | Active | 3,000 | 1972-2007 | 532 | 906 | 322 | 620 | 40 | 80 | < 2.5 | 6.7 |
| T-MS4 | Active | 3,000 | 1999-2006 | 464 | 791 | 244 | 432 | 20 | 53 | < 2.5 | 5.5 |
| T-PANK | Inactive | 4,500 | 1964-1998 | 472 | 926 | 248 | 343 | 16 | 65 | 4.8 | 4.8 |
| SA-31 | Active | 6,500 | 1987-2009 | 400 | 510 | 198 | 356 | 14 | 31 | < 2.5 | 2.58 |

* The data range for nitrate is 1987 to present while the data range for perchlorate is 1998 to present.

### 3.2.3 Water Quality Conclusions and Recommendations

#### 3.2.3.1 Raw Water Quality

The water quality presented in Table 3-3 is a blend of raw water from Wells 21 and 22 at ratios corresponding to their capacities presented in Section 2. Comprehensive water quality data for each well is presented in Appendix A.

As indicated by the data presented, the blended well water is extremely hard, nitrate levels exceed the California Department of Public Health (CDPH) primary MCL, and TDS levels exceed the CDPH secondary limits.

**Table 3-3 Wells 21 and 22 Average Raw Water Quality**

| Parameter | Units | Blended Water Quality |
|---|---|---|
| Calcium ($Ca^{2+}$) | mg/L | 133 |
| Magnesium ($Mg^{2+}$) | mg/L | 35 |
| Sodium ($Na^{+}$) | mg/L | 69 |
| Potassium ($K^{+}$) | mg/L | 2.3 |
| Barium ($Ba^{2+}$) | mg/L | ND[1] |
| Strontium ($Sr2^{+}$) | mg/L | Not Available |
| Iron ($Fe^{2+}$) | mg/L | ND[1] |
| Manganese ($Mn^{2+}$) | mg/L | ND[1] |
| Ammonium ($NH_4^{+}$) | mg/L | Not Available |
| Aluminum | mg/L | 0 |
| Bicarbonate ($HCO_3^{-}$) | mg/L | 269 |
| Sulfate ($SO_4^{2-}$) | mg/L | 177 |
| Chloride ($Cl^{-}$) | mg/L | 127 |
| Fluoride ($F^{-}$) | mg/L | 0.20 |
| Carbonate ($CO_3^{2-}$)[3] | mg/L | 0.48 |
| **Nitrate ($NO_3^{-}$)** | mg/L | **67-80**[7] |
| Silica | mg/L | 24.7 |
| Color | units | Not Available |
| pH | pH units | 7.4 |
| Alkalinity | mg/L as $CaCO_3$ | 220 |
| **Hardness** | **mg/L as $CaCO_3$** | **477** |
| $CO_2$ [3] | mg/L | 11.5 |
| TOC | mg/L | Not Available |
| *Conductivity* | *μmho/cm* | *1,167* |
| *TOTAL IONS + $SiO_2$* | mg/L | *917* |
| Turbidity | NTU | 0.35 |
| Silt Density Index (SDI) | SDI | Not Available |
| **TDS (by measurement)** | **mg/L** | **710** |
| *TDS by Ion Summation*[2] | mg/L | *780* |
| Temperature | °C | 20[4] |
| pH | mg/L | 7.4 |
| Empirical Factor (TDS/COND) | | 0.67 |
| CCPP[5] | mg/L as $CaCO_3$ | 73.1 |

| Parameter | Units | Blended Water Quality |
|---|---|---|
| LSI[6] | | 1.0 |

Notes:
1. Non Detect
2. Includes total ions (no silica) and 49 percent of the bicarbonate concentration.
3. Concentration is stoichiometrically determined from pH, alkalinity & carbonate speciation.
4. Assumed.
5. CCPP is calcium carbonate precipitation potential and was determined by calculation.
6. LSI is Langlier Saturation Index was calculated by Rothberg, Tamburini and Winsor Model.
7. 67 is average blended nitrate concentration. Maximum anticipated nitrate of 80 mg/l (as NO3), as described in Section 3.2.2.2.1 used for treatment design.

#### 3.2.3.2 Product Water Quality

IRWD currently distributes water from the Deep Aquifer Treatment System (DATS), Dyer Road Well Field, and the Irvine Desalter Project (IDP) that meets all state and federal guidelines. Selected water quality goals, state and federal primary and secondary maximum contaminant levels, and average finished water quality currently distributed to the IRWD service area are presented in Table 3-4. As indicated, IRWD produces high-quality finished water that meets all primary and secondary MCLs. For this project, IRWD has imposed on themselves a more stringent goal for TDS, total hardness, and nitrates than is required by the state and federal MCL.

Total hardness and nitrates goals were set at 150 -180 mg/L and not to exceed 36 mg/L, respectively. The goal for TDS was set to not exceed 420 mg/L, which corresponds to the TDS of the product water from the IRWD's existing Irvine Desalter Project (IDP) treatment plant.

Finished water quality goals for alkalinity, calcium hardness, total hardness and CCPP are presented in Table 3-4 as a means to demonstrate chemical stability (i.e., with respect to pH) and corrosion prevention. Standard practice recommends that finished water alkalinity is greater than 40 mg/L as $CaCO_3$ to provide chemical stability in the distribution system. Total hardness is presented as a goal since hardness values lower than 60 mg/L as $CaCO_3$ may result in excessive scale formation, decreased life of home hot water heaters, and a decreased potential for soap to foam.

The CCPP (calcium carbonate precipitation potential) is used to gauge the precipitation potential of calcium carbonate in the distribution system. CCPP is calculated using the Rothberg, Tamburini and Winsor Model for Water Process and Corrosion Chemistry (available through AWWA), using pH, TDS, alkalinity, and calcium hardness data. A positive CCPP indicates that calcium carbonate exceeds saturation limit and will precipitate to deposit a protective film in the distribution system. However, if CCPP is too high, the beneficial effect is eroded and, excessive scaling occurs in the distribution system. A negative CCPP indicates highly corrosive water, which causes corrosion of the pipeline. Both excesses result in transmission problems. The CCPP goal of 4 to 10 represents a balance between these two extremes, producing water that is corrosion resistant while providing necessary protection to the distribution system.

**Table 3-4 Finished Water Quality Goals**

| Constituent | Unit | Goal | MCL[1] | Current IRWD Domestic Water Quality Range |
|---|---|---|---|---|
| Alkalinity | mg/L as $CaCO_3$ | >40 | NS | 82 - 286 |
| Calcium Hardness | mg/L as $CaCO_3$ | 60 - 180 | NS | 5.25 - 405 |
| **Total Hardness** | **mg/L as $CaCO_3$** | **150-180** | **NS** | **5.2 - 559** |
| CCPP | mg/L as $CaCO_3$ | 4 - 10 | NS | NA |
| **Total Dissolved Solids[2]** | **mg/L** | **420** | **500** | **210 - 1,090** |
| Turbidity | NTU | < 0.3 | NS | ND - 1.2 |
| Color | Color Units | < 5 | 15 | ND - 20 |
| **Nitrate** | **mg/L** | **36** | **45** | **ND - 12** |

Notes:
1. NS = No Standard.
2. MCL for TDS can be as high as 1,000 mg/L as long as no other SMCL is exceeded.

## 3.3 Blending Alternative Evaluation

### 3.3.1 General

Blending with existing IRWD potable water supplies was evaluated for treatment of Wells 21 and 22 raw water in order to meet the stated water quality goals. The two supply sources available for blending in Zone 1 are:

- MWD imported water through the OC-58 turnout via the Orange County Feeder No.2 Pipeline. A total of 10 cubic feet per second (cfs) or 4500 gpm is available for blending
- IRWD's Zone 1 water supplied from the Dyer Road well field (DRWF) and deep aquifer treatment system (DATS). The current maximum capacity of this system is approximately 80 cfs or 36,000 gpm.

#### 3.3.1.1 MWD Imported Water

Imported water to IRWD's Zone 1 system is supplied through the Orange County Feeder No.2 pipeline at various turnout locations, which is supplied from Metropolitan Water District's (MWD) Robert Diemer Filtration Plant (Diemer). This facility receives a blend of water from the Colorado River Aqueduct (CRA) and State Water Project (SWP). The make-up of the blend has significant impacts to the overall water quality, in particular, TDS and hardness.

Due to a recent court ruling limiting pumping from the Bay Delta, which impacts State Water Project supplies to Southern California, a major shift has taken place in the make-up of the water coming from the Diemer plant. According to MWD's annual water quality report, in 2007, the Diemer facility averaged 54 percent of its supply from the SWP. In more recent Diemer treated water quality data, from December 2008, it has an average of just 7 percent SWP. Colorado River Water has increased levels of TDS and hardness; therefore, this new blend ratio greatly impacts the viability of blending for treatment. It is unclear how long the cutback in SWP supplies will persist; therefore, we have assumed the decrease will not change in the near future. Table 3-5 depicts the disparity in water quality originating from Diemer in 2007 and 2008.

**Table 3-5 Diemer Filtration Plant Water Quality Comparison**

| Constituents | Unit | Diemer Plant Water Quality | |
|---|---|---|---|
| | | 2007 (54% SWP)[1] | December 2008 (7% SWP)[2] |
| Total Hardness | mg/L | 201 | 296 |
| Nitrate as $NO_3$ | mg/L | 2.20 | 1.20 |
| TDS | mg/L | 469 | 639 |

[1] Based on MWDSC's 2008 Annual Drinking Water Quality Report for 2007
[2] Based on MWDSC's Table D: Monthly Analyses of the District Water Supplies - December 2008

3.3.1.2 IRWD Zone 1 Water Quality

The main sources of supply to IRWD's Zone 1 system are the DRWF and the DATS. The water quality from these sources is high in quality in all respects. Table 3-6 shows the average IRWD Zone 1 water quality from recent samples taken in January and February 2009.

Table 3-6 IRWD Zone 1 – 2009 Average Water Quality

| Constituents | Unit | Minimum | Average | Maximum |
|---|---|---|---|---|
| Total Hardness | mg/L | 86 | 86 | 86 |
| Calcium | mg/L | - | - | - |
| Magnesium | mg/L | 5.1 | 5.1 | 5.1 |
| Sodium | mg/L | 56.0 | 58.3 | 60.6 |
| Potassium | mg/L | 1.3 | 1.4 | 1.4 |
| Total Alkalinity | mg/L as $CaCO_3$ | 144.0 | 145.5 | 147.0 |
| Hydroxide | mg/L as $CaCO_3$ | 0.0 | 0.0 | 0.0 |
| Carbonate | mg/L as $CaCO_3$ | 0.0 | 0.0 | 0.0 |
| Bicarbonate | mg/L as $CaCO_3$ | 0.0 | 0.0 | 0.0 |
| Chloride | mg/L | 20.40 | 20.40 | 20.40 |
| Sulfate | mg/L | 37.90 | 37.90 | 37.90 |
| Fluoride | mg/L | 0.35 | 0.52 | 0.62 |
| Nitrate as N | mg/L | 0.19 | 0.19 | 0.20 |
| Nitrate as $NO_3$ | mg/L | 0.83 | 0.85 | 0.88 |
| Boron | ug/L | 137.00 | 139.50 | 142.00 |
| Color | units | 2.00 | 3.40 | 5.00 |
| Turbidity | NTU | 0.17 | 0.21 | 0.25 |
| Specific Conductance | umhos/cm | 403 | 422 | 426 |
| TDS | mg/L | 254 | 268 | 282 |
| pH | Standard Units | 8.0 | 8.1 | 8.1 |
| Iron | mg/L | 0.0 | 0.0 | 0.0 |
| Total Silica | | - | - | - |

### 3.3.2 Development of Blending Scenarios

From a review of the potential blending source water quality and the IRWD target water quality goals, it became evident that the TDS and hardness levels would not be reachable by blending with MWD imported water alone. Therefore, for each scenario, the IRWD Zone 1 water would be needed either in conjunction with imported supply or by itself to reach IRWD's desired water quality. With this in mind, three (3) blending scenarios were developed as described below:

- Blending Scenario 1

  Scenario 1 utilizes the full allotment of imported water from the OC-58 turnout and assumes average imported water quality from year 2007 (54 percent SWP). IRWD Zone 1 water is used, in addition to the imported supply, at a flow rate necessary to reach the water quality targets.

- Blending Scenario 2

  Scenario 2 utilizes the full allotment of imported water from the OC-58 turnout and assumes imported water quality from the December 2008 Diemer plant data. IRWD Zone 1 water is used, in addition to the imported supply, at a flow rate necessary to reach the water quality targets.

- Blending Scenario 3

  Scenario 3 uses no imported water for blending. IRWD Zone 1 water is used alone at a flow rate necessary to reach the water quality targets.

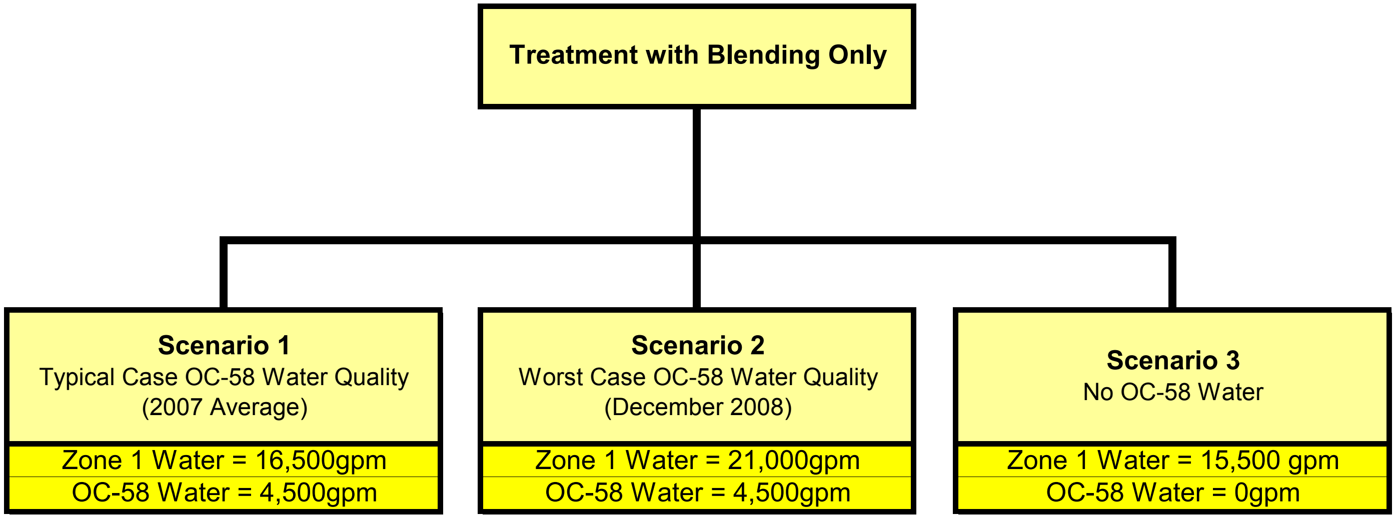
### 3.3.3 Blending Analysis Results and Conclusions

The three scenarios described above were analyzed with the use of a Microsoft Excel-based model. The model performs a mass balance calculation, tracking the constituents of concern based on the source water quality and flow rate inputs. The results of the model runs are summarized in Figure 3-2.

This page left intentionally blank

**IRVINE RANCH WATER DISTRICT**
**Wells 21 and 22**
**Preliminary Design Report**

**Figure 3-2: Blending Analysis Scenarios Results Summary**



Figures 3-3 through 3-5 show the analysis as it was run in the Excel-based model.

The analysis shows that the best scenario for blending as treatment is Scenario 3 using no MWD imported water as this requires a larger contribution from Zone 1 to make up for the high concentration of TDS and Hardness.

In addition to the large amount of source water required to blend down the nitrate, TDS, and hardness in Wells 21 and 22, there are other significant drawbacks to the use of blending for treatment, both logistically and regulatory. These disadvantages are summarized below:

- Variations in the source water quality (i.e., rise in concentrations) could impact the effectiveness of the blend.
- California Department of Public Health (CDPH) may impose lower limits on blended nitrate concentrations.
- CDPH will require continuous monitoring and automatic controls to prevent breakthroughs of nitrate.
- Wells 21 and 22 would be a dependent supply source.
- CDPH will place the burden of proof on IRWD to prove that the source supply is consistent in its water quality.
- Potential for advanced operator requirements increasing District labor costs.
- Blending will increase the overall nitrate, TDS, and hardness levels in the District's domestic water directly and in the recycled water system indirectly.
- Modifications to the Zone 1 system through the construction of a turnout and blending station will have negative impacts on the existing DRWF and DATS pumping systems, potentially limiting production.

Due to the significant drawbacks associated with blending for treatment of Wells 21 and 22 raw water, this method of treatment is not recommended.

This page left intentionally blank

**IRVINE RANCH WATER DISTRICT**
**Wells 21 and 22 Rehabilitation Project**
**Preliminary Design Report**

**Figure 3-3**
**Blending Scenario 1: MWDSC Water = 54% SWP + 46% CRW**

| Well 21 [1] | |
|---|---|
| Q (gpm) | 3,300 |
| TDS (mg/L) | 740 |
| Hardness (mg/L) | 500 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Well 22 [2] | |
|---|---|
| Q (gpm) | 1,600 |
| TDS (mg/L) | 650 |
| Hardness (mg/L) | 430 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Wells 21 and 22 Water | |
|---|---|
| Q (gpm) | 4,900 |
| TDS (mg/L) | 711 |
| Hardness (mg/L) | 477 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| OC 58 (54% SWP Water) [3] | |
|---|---|
| Q (gpm) | 4,500 |
| Q (cfs) | 10 |
| TDS (mg/L) | 469 |
| Hardness (mg/L) | 201 |
| Nitrate as N (mg/L) | 0.5 |
| Nitrate $NO_3$ (mg/L) | 2.2 |

| IRWD Zone 1 [4] | |
|---|---|
| Q (gpm) | 16,500 |
| TDS (mg/L) | 268 |
| Hardness (mg/L) | 86 |
| Nitrate $NO_3$ (mg/L) | 0.85 |

| Blended Water | | IRWD Targets |
|---|---|---|
| Q (gpm) | 25,900 | - |
| TDS (mg/L) | 387 | 420 |
| Hardness (mg/L) | 180 | 150 |
| Nitrate-$NO_3$ (mg/L) | 16 | 36.0 |

[1] Based on water quality results of the Well 21 composite sample, which was collected during the Well 21 constant-rate pumping test (November 2008).
[2] Based on water quality results of the Well 22 composite sample, which was collected during the Well 22 constant-rate pumping test (January 2009).
[3] Based on Diemer Water Treatment Plant Effluent Water Quality per MWDSC's 2008 Annual Drinking Water Quality Report (average 2007 water quality data)
[4] Based on 2009 water quality data (average of January and February 2009)

**IRVINE RANCH WATER DISTRICT**
**Wells 21 and 22 Rehabilitation Project**
**Preliminary Design Report**

**Figure 3-4**
**Blending Scenario 2: MWDSC Water = 7% SWP + 93% CRW**

| Well 21 [1] | |
|---|---|
| Q (gpm) | 3,300 |
| TDS (mg/L) | 740 |
| Hardness (mg/L) | 500 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Well 22 [2] | |
|---|---|
| Q (gpm) | 1,600 |
| TDS (mg/L) | 650 |
| Hardness (mg/L) | 430 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Wells 21 and 22 Water | |
|---|---|
| Q (gpm) | 4,900 |
| TDS (mg/L) | 711 |
| Hardness (mg/L) | 477 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| OC 58 (7% SWP Water) [3] | |
|---|---|
| Q (gpm) | 4,500 |
| Q (cfs) | 10 |
| TDS (mg/L) | 639 |
| Hardness (mg/L) | 296 |
| Nitrate as N (mg/L) | - |
| Nitrate $NO_3$ (mg/L) | 1.2 |

| IRWD Zone 1 [4] | |
|---|---|
| Q (gpm) | 21,000 |
| TDS (mg/L) | 268 |
| Hardness (mg/L) | 86 |
| Nitrate $NO_3$ (mg/L) | 0.85 |

| Blended Water | | IRWD Targets |
|---|---|---|
| Q (gpm) | 30,400 | - |
| TDS (mg/L) | 394 | 420 |
| Hardness (mg/L) | 180 | 180 |
| Nitrate-$NO_3$ (mg/L) | 14 | 36.0 |

[1] Based on water quality results of the Well 21 composite sample, which was collected during the Well 21 constant-rate pumping test (November 2008).
[2] Based on water quality results of the Well 22 composite sample, which was collected during the Well 22 constant-rate pumping test (January 2009).
[3] Based on Diemer Water Treatment Plant Effluent Water Quality, December 2008

**IRVINE RANCH WATER DISTRICT**
**Wells 21 and 22 Rehabilitation Project**
**Preliminary Design Report**

**Figure 3-5**
**Blending Scenario 3: No MWD Imported Water**

| Well 21 [1] | |
|---|---|
| Q (gpm) | 3,300 |
| TDS (mg/L) | 740 |
| Hardness (mg/L) | 500 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Well 22 [2] | |
|---|---|
| Q (gpm) | 1,600 |
| TDS (mg/L) | 650 |
| Hardness (mg/L) | 430 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| Wells 21 and 22 Water | |
|---|---|
| Q (gpm) | 4,900 |
| TDS (mg/L) | 711 |
| Hardness (mg/L) | 477 |
| Nitrate-$NO_3$ (mg/L) | 80.0 |

| OC 58 (7% SWP Water) [3] | |
|---|---|
| Q (gpm) | 0 |
| Q (cfs) | 0 |
| TDS (mg/L) | 639 |
| Hardness (mg/L) | 296 |
| Nitrate as N (mg/L) | - |
| Nitrate $NO_3$ (mg/L) | 1.2 |

| IRWD Zone 1 [4] | |
|---|---|
| Q (gpm) | 15,500 |
| TDS (mg/L) | 268 |
| Hardness (mg/L) | 86 |
| Nitrate $NO_3$ (mg/L) | 0.85 |

| Blended Water | | IRWD Targets |
|---|---|---|
| Q (gpm) | 20,400 | - |
| TDS (mg/L) | 374 | 420 |
| Hardness (mg/L) | 180 | 180 |
| Nitrate-$NO_3$ (mg/L) | 20 | 36.0 |

[1] Based on water quality results of the Well 21 composite sample, which was collected during the Well 21 constant-rate pumping test (November 2008).
[2] Based on water quality results of the Well 22 composite sample, which was collected during the Well 22 constant-rate pumping test (January 2009).
[3] Based on Diemer Water Treatment Plant Effluent Water Quality, December 2008

### 3.3.4 Expanded Blending Evaluation

Upon submission of the Draft PDR in May 2009, the high cost of the project, assuming construction of a membrane treatment facility, brought back into question the viability of the blending option. A detailed technical memorandum was prepared that analyzed the blending option further. The goal of the memorandum was to provide an expanded assessment of the following:

- Feasibility of blending with the current supply available from the Dyer Road Well Field and the IRWD Deep Aquifer Treatment System (DATS).
- Impacts on pumping from the Dyer Road Well field.
- Impacts of varying water quality in the supply and blend source.
- Cost evaluation for the blending facility.

The technical memorandum concluded the following:

- Adequate flow was available for blending from the Dyer Road Well Field and the DATS.
- The cost of the blended water is less than that of the RO treatment facility option, but costs remain high, with a preliminary estimated capital cost to construct the blending station and related facilities at greater than $20 million.
- Based on hydraulic model results, the impact to the Dyer Road Well Field and DATS pumping facilities require additional improvements and operations adjustments.

The complete technical memorandum *Expanded Evaluation for Blending Wells 21 and 22 with Dyer Road Well Field Water* is included in this report in Appendix L.

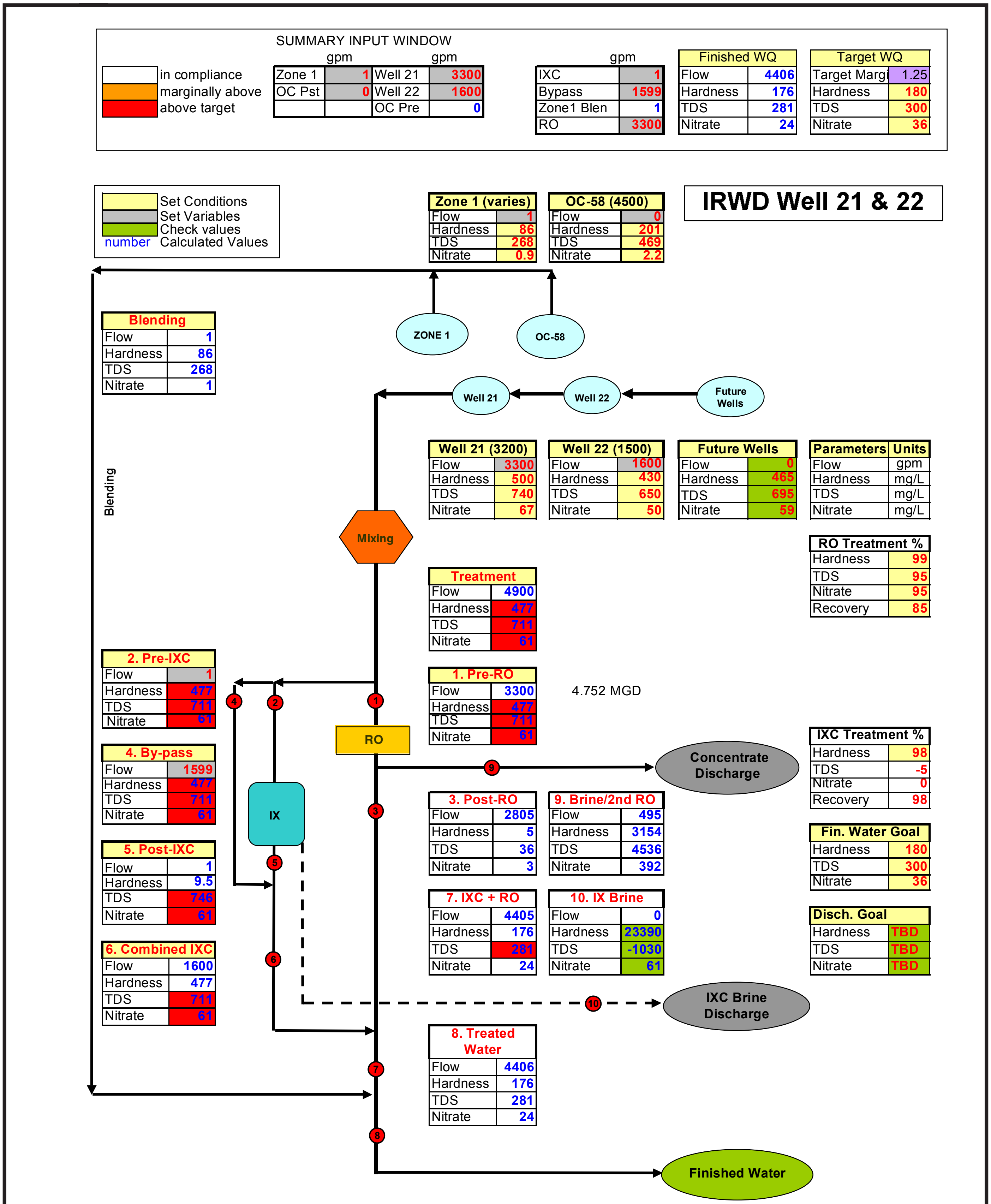
## 3.4 Treatment Technology Evaluation

### 3.4.1 Project Approach

For any drinking water treatment plant, treatment technology is chosen primarily to bridge the gap between actual raw water quality and finished water quality goals. The treatment technology must also meet secondary considerations such as financial feasibility, acceptability and ease of operation. Given the untreated water quality and the product water goals for the Wells 21 and 22 Project, two main technologies were investigated: ion exchange (for hardness or nitrate removal) and membrane separation for hardness, nitrate, and TDS removal. These technologies are briefly described herein.

### 3.4.2 Blending and Treatment Model

Carollo Engineers developed a blending and treatment mass and flow balance model for the IRWD Wells 21 and 22 water. The schematic is presented in Figure 3-5. The model includes four water sources as inputs and tracks the resulting nitrate, TDS and hardness as the water is treated through a combination of reverse osmosis and/or nanofiltration membranes, ion exchange, and bypass streams. In addition to Wells 21 and 22 raw water, two other water source inputs were included to provide options for blending. Zone 1 water quality was based on 2009 data and OC-58 Diemer Water Treatment Plant effluent water quality reported in MWDSC's 2008 Annual Drinking Water Quality Report. These were included to extend the utility of the model to scenarios that might achieve finished water goals through blending alone.




# TREATMENT AND BLENDING MODEL SCHEMATIC

FIGURE 3-6

carollo
Engineers...Working Wonders With Water™

IRVINE RANCH WATER DISTRICT

20-IRWD5-09F3-5.AI

This section presents findings for the treatment scenarios alone. Blending alternatives and results were presented previously. Model results for water quality and flow for the treatment scenarios investigated are presented in Section 3.4.6. Assumptions for recoveries and rejections for the membrane and ion exchange processes are presented in Table 3-7.

**Table 3-7 Design Assumptions Used In Model Predictions**

| | Rejection (%) | | | Recovery (%) |
|---|---|---|---|---|
| **Process** | **Hardness** | **TDS** | **Nitrate** | |
| Reverse Osmosis | 99 | 98 | 91 | 85[1] |
| Nanofiltration (NF) | 90 | 83 | 70 | 85[2] |
| Anion Exchange (IXA) | 0 | -5 | 95 | 98 |
| Cation Exchange (IXC) | 98 | -5 | 0 | 98 |

Notes:

1. NF/RO recoveries are based on model results from Hydranautics IMS Design Software, v. 2009.24, ESPA2 membranes, 5-yr membrane age, balanced flux.
1. NF recoveries are based on model results from Hydranautics IMS Design Software, v. 2009.24, ESNA1-LF membranes, 5-yr membrane age, balanced flux.

### 3.4.3 Ion Exchange

Ion exchange is currently the most demonstrated and implemented technology for treatment of nitrate in drinking water. Most resins are NSF certified, and a number of commercial systems accepted by CDPH have been implemented in several locations throughout California. The common resins used for nitrate removal are strong-base anion exchange resins in the chloride form. As the feed water contacts the resin, the nitrate ions exchange with the chloride ions at the surface of the resin. When all the chloride ions have been replaced by nitrate ions, the resin is considered spent and must be regenerated through a costly process. The end result is a finished water stream with low nitrate concentration but a TDS higher than the feed stream.

While strong-base anion exchange resins enable removal of nitrates, cation-exchange resins facilitate the removal of hardness components (calcium and magnesium). As water passes through this resin bed, calcium and magnesium ions attach to the resin beads, displacing sodium ions that are released by the resin into the water. As with anion-exchange resins, TDS of the finished water is also higher then the feed stream. This increase in TDS (as much as 5 percent above feed water levels) means that ion exchange is not suitable for raw waters with already undesirable levels of TDS. With a Well 21 and 22 raw water blend

TDS concentration of 711 mg/L, finished water goals for Wells 21 and 22 will not be satisfied by ion exchange alone.

### 3.4.4 Membranes

High-pressure membrane processes such as nanofiltration (NF) and reverse osmoses (RO) provide a barrier for rejecting the chemicals of concern for IRWD (nitrates, TDS, and hardness). Further, because they are a non-selective process with high rejections rates, they are of great value in addressing future contaminants of concern. The biggest drawback of membrane processes is that (in comparison to ion exchange processes) they generate a high volume brine stream that must be further treated or discharged as waste.

In general, NF membranes provide lower rejection rates than RO membranes but require lower feed pressures. (Table 3-8). Nitrate rejection of NF membranes is especially low—modeled at 70 percent but reported as low as 36 percent in some literature, depending on the particular membrane chemistry. This means that for the Wells 21 and 22 water, the size of the NF plant required to achieve the water quality objectives may be expected to be much larger than that for a RO plant.

### 3.4.5 Hybrid Processes

#### 3.4.5.1 NF-RO Hybrid

Hybrid arrays are sometimes used to minimize the energy consumption or passively balance flux rates in membrane systems. A hybrid array uses different membrane elements in each stage based on the water quality requirements. For example, high rejection elements can be used in the first stage of a high-pressure membrane system followed by low energy elements in the second stage to facilitate flux balancing. For the low TDS brackish water, such as IRWD feedwater, nanofiltration membranes can be used in the first stage followed by low-pressure brackish RO elements in the second stage. This combination yields higher energy demand requirements and lower TDS than NF alone, but lower energy requirements and higher TDS permeate than RO alone. It should be noted that NF-RO hybrids in which NF and RO elements are alternated within the same vessel are not an industry standard and are, therefore, not considered in this report. In this configuration, the feed pressures for each vessel will be driven by the greater energy demands of the RO membranes. This will erode and likely eliminate the benefits of the NF-RO hybrid.

#### 3.4.5.2 Ion Exchange - Membrane Hybrid

Another hybrid approach that was investigated was the ion exchange-membrane hybrid. This was included to determine if an IX side stream, targeting a single contaminant such as nitrate or hardness, might reduce the amount of membrane treatment required to meet water quality objectives. It was determined that the IX side stream would provide little benefit in regards to water quality and cost savings with increased complexity of operations.

### 3.4.6 Treatment Alternatives

Six (6) scenarios were evaluated using the raw water quality presented in Table 3-3 and the mass balance data resulting from the model. The results are presented in Table 3-8. The treatment alternatives assumed flows of 3,300 gpm and 1,600 gpm from Wells 21 and 22 respectively, as recommended in Section 2. For all scenarios, the goal was to determine the minimum plant size required to achieve finished water quality goals without blending with either Zone 1 or OC-58 waters.

This page left intentionally blank

Controlling Constituent

Table 3-8 Model Results for Treatment Alternatives Investigated

| | Unit Process Flows[1] | | | Finished Water Quality[2] and Flow[1] | | | | Brine[1] |
|---|---|---|---|---|---|---|---|---|
| Description | IX | RO/NF | Bypass | TDS | Hardness | Nitrate | Flow | |
| **Membrane with IX Side Stream** | | | | | | | | |
| 1) RO with IXC | 765 | 3,073 | 1,062 | 345 | 142 | 36 | 4,439 | 461 |
| 2) RO with IXC | 325 | 3,325 | 1,250 | 300 | 150 | 32 | 4,401 | 499 |
| 3) NF with IXA | 325 | 3,750 | 825 | 327 | 150 | 35 | 4,338 | 563 |
| **Membranes Alone** | | | | | | | | |
| 4) RO Alone | 0 | 3,622 | 1,278 | 247 | 150 | 27 | 4,357 | 543 |
| 5) NF Alone | 0 | 4,150 | 750 | 267 | 110 | 36 | 4,278 | 623 |
| **Hybrid Membranes** | | | | | | | | |
| 6) NF-RO | 0 | 3,636 | 1,263 | 278 | 150 | 32 | 4,355 | 545 |

Note: IX = Ion Exchange; IXC= Cation Exchange; IXA = Anion Exchange
1. Process flows presented in gallons per minute.
2. TDS and Nitrates presented as mg/L and hardness as mg/L as $CaCO_3$.

This page left intentionally blank

## 3.5 Recommended Treatment Technology

As shown in Table 3-8, in order to achieve the desired finished water quality goals with ion exchange processes, the size of the ion-exchange plant would range from 200 gpm to 700 gpm. This side stream treatment would be augmented by a much larger membrane plant, ranging in size from 2,500 gpm to 3,800 gpm operating at 85 percent recovery.

For nanofiltration membranes alone, a 4,150 gpm plant would need to be constructed. This large size is dictated by the low nitrate rejection of the NF membranes (modeled at 70 percent). Two membrane scenarios will allow IRWD to achieve finished water quality goals while keeping treatment plant at a manageable size: a 3,622 gpm plant using RO alone or using a NF-RO hybrid. Both will generate very similar water quality at the same treatment capacity.

The NF-RO treatment process has slightly lower pumping needs; however, the RO-RO treatment is recommended as this approach is a standard design for the various membrane suppliers and does not require piloting of the system prior to final design, although this is typically the recommended approach for optimization of the system. The elimination of piloting is critical due to the time constraints of having the plant on-line by September 2011 to take advantage of the awarded BOR Title XVI grant funding through the ARRA. This RO membrane treatment process forms the basis of the design criteria presented in Section 5, Preliminary Design.

This page left intentionally blank

# Section 4

# Evaluation of Alternatives

This page left intentionally blank.

# Section 4 Evaluation of Alternatives

## 4.1 General

Based on the water quality results analysis and treatment alternatives evaluation described in Section 3, the Wells 21 and 22 system will require the following key elements:

- Treatment site with adequate area for construction of the recommended water treatment process.
- Untreated water conveyance pipeline from Wells 21 and 22 to the treatment site.
- Product water pipeline from the treatment facility to an IRWD Zone 1 distribution system tie-in point.
- Brine discharge route and destination for the treatment process waste stream.

Section 4 will discuss how alternatives were developed and evaluated for these key project elements.

## 4.2 Treatment Sites

### 4.2.1 Initial Treatment Site Identification

In preparation of the Draft PDR, ten (10) potential treatment site locations were identified in this study for evaluation to accommodate the treatment facilities recommended in Section 3. The sites were identified through a review of previous reports, a thorough assessment of recent aerial photography, and field reconnaissance. The locations of the sites initially identified are shown on Figure 4-1.

Based on the evaluation criteria and descriptions below, the site initially recommended for the treatment plant was the Marine Corps Air Station (MCAS) site located at the southwest corner of the intersection of Edinger Avenue and Tustin Ranch Road. Subsequent to submission of the Wells 21 and 22 Draft PDR in May 2009, IRWD conducted preliminary discussions with the City of Tustin to confirm feasibility of acquiring the site. Based on these discussions, it became clear that the MCAS site would be unavailable for immediate acquisition.

This page left blank intentionally


RBF CONSULTING GIS

Source: ESRI Online

0 0.5 1 Miles

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure 4-1: Alternative Treatment Sites

This page left intentionally blank.

IRWD directed staff, with technical support from the RBF/Carrolo team, to conduct a focused real estate investigation to determine parcels, in the areas of the two alternate sites (I-5 site and Edinger Avenue site), that are currently available for acquisition, vacant or otherwise.

The initial treatment plant investigation is discussed below in Section 4.2.2. The focused treatment site evaluation is discussed later in Section 4.2.3.

### 4.2.2 Treatment Site Evaluation

The criterion used to evaluate the sites is as follows:

- Adequate area for a treatment plant
- Accessibility
- Land use/zoning
- Required site improvements
- Acquisition cost and difficulty
- Proximity to Wells 21/22 and the IRWD Zone 1 distribution system
- Feasibility of additional on-site wells

Figures 4-2 through 4-11 give a description of location, characteristics, and advantages/disadvantages for each site. Several pictures of each area are shown on the figures highlighting these attributes. Brief descriptions and primary reasoning behind the recommendation for further consideration or elimination of further consideration for each site are given below.

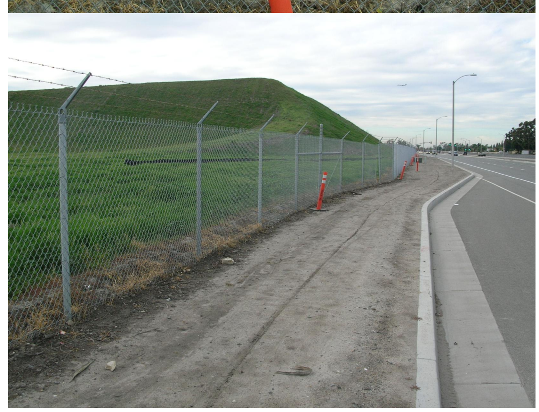
#### 4.2.2.1 Tustin Marine Corps Air Station (MCAS) Site

The MCAS site is an empty parcel located on the former Tustin Marine Corps Air Station at the intersection of Edinger Avenue and the planned Tustin Ranch Road extension. Access to the site is off Edinger Avenue and the planned on-ramp/off ramp of Tustin Ranch Road. The MCAS is slated for mixed use development with large sections of residential

This page left blank intentionally

**Figure 4-2**

**MCAS Tustin Base**



Location
Located at Intersection of Edinger Ave and Tustin Ranch Rd, South of Edinger Ave on former MCAS Tustin Base.

Size
2.87 acres

Description
Open field south of Edinger Ave. Immediately Southeast of planned overpass.

Advantages

- Site of future development, agreement for allocation of land for water supply facilities already in place potentially reducing acquisition costs
- Site adequate size and configuration for large water treatment facilities
- Potential for expansion of treatment facilities with planned on-site wells, reducing $/volume cost of water
- Located off Edinger Ave and proposed Tustin Ranch Road, minimal traffic impacts

Disadvantages

- Visual/Noise impacts must be considered as development site will have residential component
- Potentially high acquisition cost
- Raw Water Supply Pipeline to the site will require railroad crossing

**<u>Figure 4-3</u>**

**I-5 Site**

<u>Location</u>
Southeast corner of the intersection of the I-5 Freeway and Tustin Ranch Rd

<u>Size</u>
2.12 acres

<u>Description</u>
Open lot adjacent to the I-5 Freeway, Tustin Ranch Rd and across from an existing industrial building occupied by Toshiba.

<u>Advantages</u>
- Empty lot has ample size for treatment facilities
- No significant noise or visual impacts
- No residential nearby

<u>Disadvantages</u>
- Extensive access improvements required. Only current access is through Toshiba parking lot off Chamber Rd/Michelle Dr.
- Existing petroleum pipelines running through site
- Zoned agricultural

**<u>Figure 4-4</u>**

**Edinger Site**

<u>Location</u>
North of Edinger Ave between Edinger Ave and railroad, 800 feet east of 55 Fwy.

<u>Size</u>
2.10 acres

<u>Description</u>
Open lot in an existing industrial area. Currently stores scrap steel, aluminum and miscellaneous

<u>Advantages</u>
- Site adequate size and configuration for water treatment facilities
- Access from Edinger Ave, and the 55 Fwy. Minimal traffic impacts
- Located in existing industrial area, minimal noise/visual impacts
- Zoned for industrial

<u>Disadvantages</u>
- One of the farthest sites from supply and tie-in points, extending length of pipeline construction
- Raw Water Conveyance and Finished Water pipeline to and from the site will require railroad crossing.

**<u>Figure 4-5</u>**

**Jamboree III**

<u>Location</u>
Just south of Jamboree Rd/SR-261 and Walnut Ave on the east side of Jamboree Rd, West of Harvard Park and Peters Canyon Channel.

<u>Size</u>
11.15 acres (entire property)

<u>Description</u>
Site of the existing Printronix Inc. building, parking lot and abandoned testing facility.

<u>Advantages</u>
- Empty lot has ample size for treatment facilities
- Existing access road from Myford Rd.
- No significant noise or visual impacts
- Zoned for Industrial
- No residential nearby
- Site already disturbed limiting environmental impacts

<u>Disadvantages</u>
- Requires acquisition of a portion of the existing parking lot. 25-30 spaces would be lost to facilities.
- Requires demolition of abandoned Printronix testing facilities.
- Access improvements required for larger traffic
- Odd shaped site

**<u>Figure 4-6</u>**

**Myford Site**

<u>Location</u>
North of Michell Dr at Intersection of Myford Rd. and Michelle Dr.

<u>Size</u>
0.45 acres

<u>Description</u>
Open lot between an existing industrial building and storm channel, just south of 5 Fwy.


<u>Advantages</u>
- Adequate access from Myford Dr., minimal traffic impacts
- Located in existing industrial area, minimal noise/visual impacts
- Odd shaped site not conducive for typical industrial development, acquisition cost potentially reduced
- Nearby location for alternative tie-in location, minimizing length of treated water piping
- Noise and visual impacts minimal

<u>Disadvantages</u>
- Zoned for agricultural
- Long supply pipeline required from Wells 21/22.
- Inadequate space for treatment facility with Myford Rd extension plans

**Figure 4-7**

**Railroad Site**

Location
South of Walnut Avenue along railroad tracks between Tustin Ranch Rd and Browning Ave.

Size
1.75 acres

Description
Long slender railroad/utility easement between two existing residential neighborhoods

Advantages
- Located off Walnut Ave minimizing traffic impacts

Disadvantages
- Long, skinny site requires access improvements. Difficult turn around for larger trucks
- Resistance from homeowners likely
- Significant noise attenuation required due to proximity of residences and school
- Existing utilities, including 84-inch storm drain and two (2) fuel lines limit available space for treatment plant construction
- Zoned miscellaneous, some re-zoning likely

**Figure 4-8**

**Jamboree II**

Location
Just south of Jamboree Rd/SR-261 and Walnut Ave on the East side of Jamboree Rd, West of Harvard Park and Peters Canyon Channel.

Size
1.17 acres (50,960 sf)

Description
Site of an existing storm water pump station, drainage facilities and detention basin. Proposed site is vacant land on top of existing large concrete drainage channel (El Modena Channel).

Advantages
- Existing access road from Walnut Ave.
- No significant noise or visual impacts
- Previously disturbed site minimizes environmental impact.

Disadvantages
- Site is constrained by existing storm channels. Underground construction will be difficult.
- Large truck access will be difficult, access improvements required.

**Figure 4-9**

**Del Amo Site**

Location
1500 feet south of Edinger Ave. on the East side of Del Amo Ave

Size
1.17 acres

Description
Open lot in an existing industrial park.

Advantages
- Site adequate size and configuration for expanded water treatment facilities
- Access from Edinger Ave, Del Amo Ave and the 55 Fwy. Minimal traffic impacts
- Located in existing industrial area, minimal noise/visual impacts
- Zoned for industrial

Disadvantages
- One of the farthest sites from supply and tie-in points. Expensive pipeline construction
- Major supply and product water pipelines to and from the site will require railroad crossing
- Purchased recently by Real Estate Operating Company and prime location from 5 and 55 freeways. High cost of acquisition likely.

**Figure 4-10**

**Well 21 Site**

Location
14232 Debusk Ln. at the corner of Debusk Ln and Mitchell Ave in the City of Tustin

Size
0.17 ac (7,280 sf) Well Site

Advantages
- Well site owned by IRWD eliminating acquisition difficulties and cost
- Potential for reduced conveyance cost due to proximity of wells to treatment facility

Disadvantages
- Existing well site inadequate in size for proposed water treatment facility
- Opposition by residents to construction of a water treatment facility likely
- Requires re-zoning of acquired residential property
- High property acquisition cost likely
- Small residential streets make difficult access for chemical deliveries/large truck traffic

**Figure 4-11**

**Nisson Site**

Location
Along Nisson Rd midway between Red Hill Ave and Browning Ave.

Size
0.77 acres (33,540 sf)

Description
Vacant land bounded by I-5 freeway, public swimming pool and facilities, mobile home park and single story apartment complex.

Advantages
- Lot is unimproved other than a small portion paved for parking. No existing structures.

Disadvantages
- Treatment capacity limited due to size of site.
- Two owners of property requires multiple negotiations for acquisition
- Opposition by mobile home residents to construction of a water treatment facility likely
- Requires re-zoning of acquired multi family residential zoned property
- Construction of conveyance utilities along 2-lane Nisson Rd will be difficult
- Potential for high property acquisition cost due to MFR zoning
- Small residential streets make difficult access for chemical deliveries/large truck traffic

development. The MCAS site was initially recommended as the preferred site for the following reasons:

- Proposed parcel is available and is large enough to house the proposed treatment plant and all expansion alternatives.
- An additional on-site well for expansion is a viable option as IRWD was previously allotted four sites on the former base for development of wells.
- The site access can be accommodated off major existing thoroughfares and integrated into planned infrastructure improvements.

4.2.2.2 I-5 Site

The I-5 site is located at the southeast corner of the intersection of the I-5 Freeway and Tustin Ranch Road. The site is adjacent to an existing Toshiba industrial building with current access from Michelle Drive. This site has been recommended as an alternate site to the MCAS should acquisition of the MCAS property encounter difficulties warranting reconsideration. The property is owned by the Irvine Company. The site is recommended for alternate consideration for the following reasons:

- The site is large enough for the recommended treatment plant and all of the expansion alternatives discussed in this report.
- The site is located in an existing industrial area eliminating any residential impacts.

4.2.2.3 Edinger Avenue Site

The Edinger Avenue site is located in an industrial area near the 55 Freeway and Edinger Avenue. The site is a fenced open lot currently used as a storage yard for miscellaneous materials. The site is recommended for alternate consideration for the following reasons:

- The site's location is in a primarily industrial area minimizing public impacts including traffic and noise.

- The site is a vacant lot with access off major thoroughfares.

- Site is zoned industrial.

4.2.2.4 Jamboree II

The Jamboree II site is located on the east side of SR-261 at the intersection of Walnut Avenue and Peters Canyon Channel. The site is just south of an existing Caltrans storm water pump station facility and detention basin. This site is not recommended for further consideration due to the following factors:

- Site sits atop a large concrete box drainage structure limiting underground construction and complicating construction of required structures.

- Site access is severely limited to larger traffic and requires major improvements.

4.2.2.5 Jamboree III

The Jamboree III site is located on the west side of SR-261 on the south side of the intersection of Walnut Avenue and Myford Avenue. Printronix, a local printer equipment manufacturer, occupies a large industrial building on the site. The investigation of Jamboree III was limited to the southern portion of the Printronix parking lot, which includes an abandoned testing facility. This site is not recommended for further consideration due to the following:

- Typically, a minimum number of parking stalls is required based on the size of the facility. The treatment plant site would eliminate up to 50 parking spaces requiring potentially costly relocations of these spots.

- There is limited area for addition of any future on-site wells and pump to waste from a well site would require long lengths of pipeline to reach an adequate storm drain or channel.

4.2.2.6 Myford Site

The Myford site is located north of the intersection of Myford Road and Michelle Drive in between an industrial building and existing concrete lined storm drainage channel. The site is an open parcel owned by the Irvine Company. This site is not recommended for further consideration due to the following factors:

- The site has an easement for the future expansion of Myford Road. This extension would leave an area too small for the proposed treatment facility or the expansion alternatives.

4.2.2.7 Railroad Site

The railroad site is located on the south side of Walnut Avenue approximately 700 feet east of Browning Avenue. The site is an old railroad and utility easement that is currently owned by the Irvine Company. The site rests between two existing residential developments. The site is only 75 feet wide and over 2000 feet in length. This site is not recommended for further consideration due to the following factors:

- The site is oddly shaped requiring a very long skinny treatment facility.
- Strong opposition from the residential neighborhoods is likely.
- Existing utilities running the length of the site, including a 78-inch storm drain, a 10-inch abandoned fuel line, and a 6-inch active fuel line make the placement of the treatment plant and construction of underground structures difficult.
- Due to the limited width access and turn-around, area is severely limited.

4.2.2.8 Del Amo Site

The Del Amo site is located in an industrial area near the 55 Freeway and Edinger Avenue. The site is an open lot in between two existing industrial buildings. The site is not recommended for further consideration for the following reason:

- The sites proximity to the wells and the potential Zone 1 tie-in locations requires additional conveyance pipeline, adding cost to the project.

4.2.2.9 Well 21 Site

The existing Well 21 site was evaluated due to the perceived cost savings in regard to conveyance. The Well 21 site is a 0.17 acre site in a residential neighborhood. The Well 21 site is not recommended for further consideration due to the following factors:

- Existing site is not large enough to house the recommended treatment system.
- Strong opposition to facilities likely from existing residential neighborhood surrounding the site.

4.2.2.10 Nisson Site

The Nisson site is an empty lot along Nisson Avenue between Red Hill Avenue and Browning Avenue. The site is a partially graded empty lot with a small parking lot surrounded by a mobile home park and multi-family residential. A community swimming facility sits adjacent to the site. The Nisson site is not recommended for further consideration due to the following factors:

- Existing site is not large enough for expanded treatment facilities. Proposed treatment system would be space and access limited.
- Strong opposition to facilities likely from existing residential neighborhood surrounding the site.

- Potential for additional wells to feed untreated water to the site unlikely due to location.

### 4.2.3 Focused Treatment Site Evaluation

The IRWD focused treatment site evaluation concentrated on sites in two general areas:

1. Area 1—Includes seven (7) sites in the industrial area southwest of the I-5 and Tustin Ranch Road intersection. The Area 1 sites are shown in Figure 4-12.

2. Area 2—Encompasses the Edinger Avenue and Del Amo Sites as identified in the initial evaluation and nine (9) other surrounding sites. The Area 2 sites are shown on Figure 4-13.

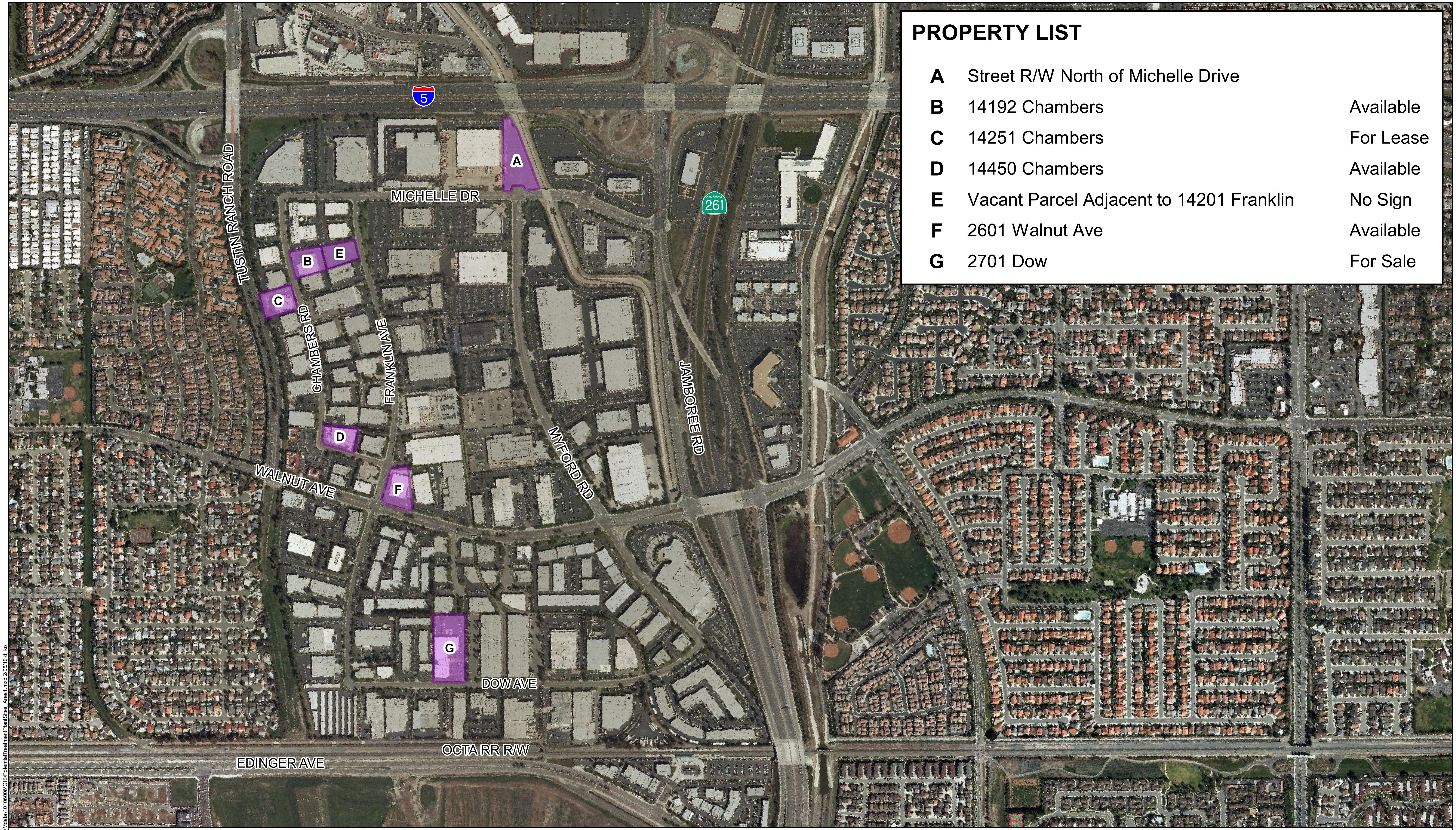
### 4.2.4 Acquired Site

As alternative sites were vetted and subsequently removed from consideration due to either inadequate size or lack of buyout interest from the current owners, two sites, Sites D (1221 Edinger Avenue) and Site I (Del Amo Site), located in Area 2, became the most likely to be acquired expeditiously to meet the overall project timeline. Figure 4-14 depicts the two candidate sites.

Preliminary layouts for the Wells 21 and 22 treatment plant were developed and placed on the sites to ensure available area existed for a facility sized to handle the flow from Wells 21 and 22 and potential expansion. The site layouts indicated that Site I (Del Amo Site) required acquisition of a portion of the property to the north to ensure proper truck access in and around the plant. Site D required no additional property, but houses an existing vacant building that may require demolition prior to construction of the treatment plant. IRWD staff, with support from RBF, is currently investigating the viability of re-using the building; however the cost analysis assumes demolition.

Ultimately, Area 2, Site D (1221 Edinger Avenue) is being acquired by IRWD as escrow opened in February 2010. This site is now the focus of the updated analysis of all facilities in this final PDR; therefore, detailed analysis from the *May 2009 Wells 21 and 22 Draft PDR*, that was specific to the MCAS site, has been placed in Appendix B of this report for preservation.

This page left intentionally blank




RBF CONSULTING GIS

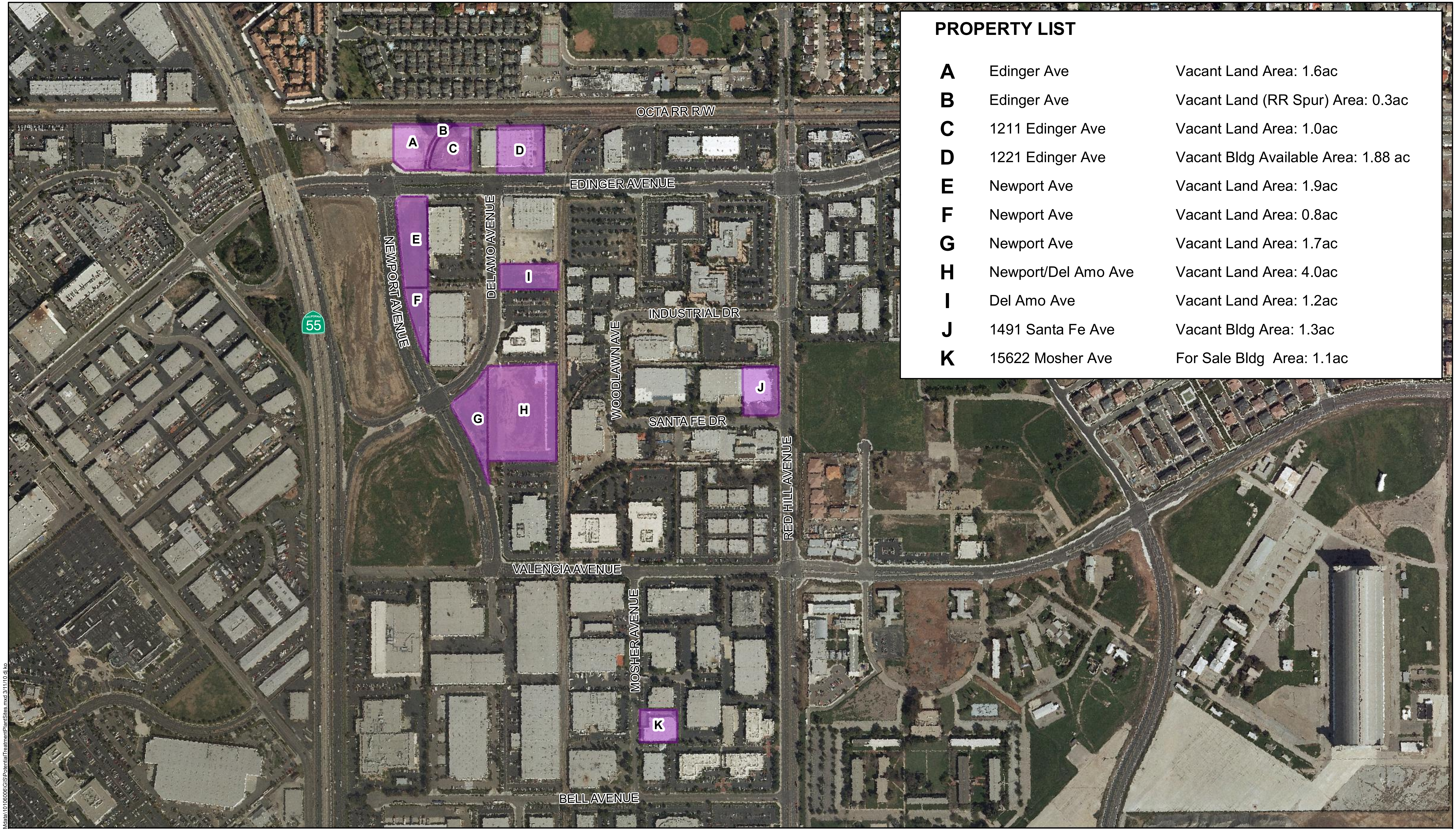
Source: Eagle Aerial, 2009

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure 4-12: Potential Treatment Plant Sites - Area 1

This page left intentionally blank.




**PROPERTY LIST**

| | | |
|---|---|---|
| **A** | Edinger Ave | Vacant Land Area: 1.6ac |
| **B** | Edinger Ave | Vacant Land (RR Spur) Area: 0.3ac |
| **C** | 1211 Edinger Ave | Vacant Land Area: 1.0ac |
| **D** | 1221 Edinger Ave | Vacant Bldg Available Area: 1.88 ac |
| **E** | Newport Ave | Vacant Land Area: 1.9ac |
| **F** | Newport Ave | Vacant Land Area: 0.8ac |
| **G** | Newport Ave | Vacant Land Area: 1.7ac |
| **H** | Newport/Del Amo Ave | Vacant Land Area: 4.0ac |
| **I** | Del Amo Ave | Vacant Land Area: 1.2ac |
| **J** | 1491 Santa Fe Ave | Vacant Bldg Area: 1.3ac |
| **K** | 15622 Mosher Ave | For Sale Bldg Area: 1.1ac |

RBF CONSULTING GIS

0 500 1,000 Feet

Source: Eagle Aerial, 2009

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure 4-13: Potential Treatment Plant Sites - Area 2

This page left intentionally blank.


0 300 600
Feet

Source: Eagle Aerial, 2009

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22

## Figure 4-14: Potential Treatment Plant - Sites D and I

This page left intentionally blank.

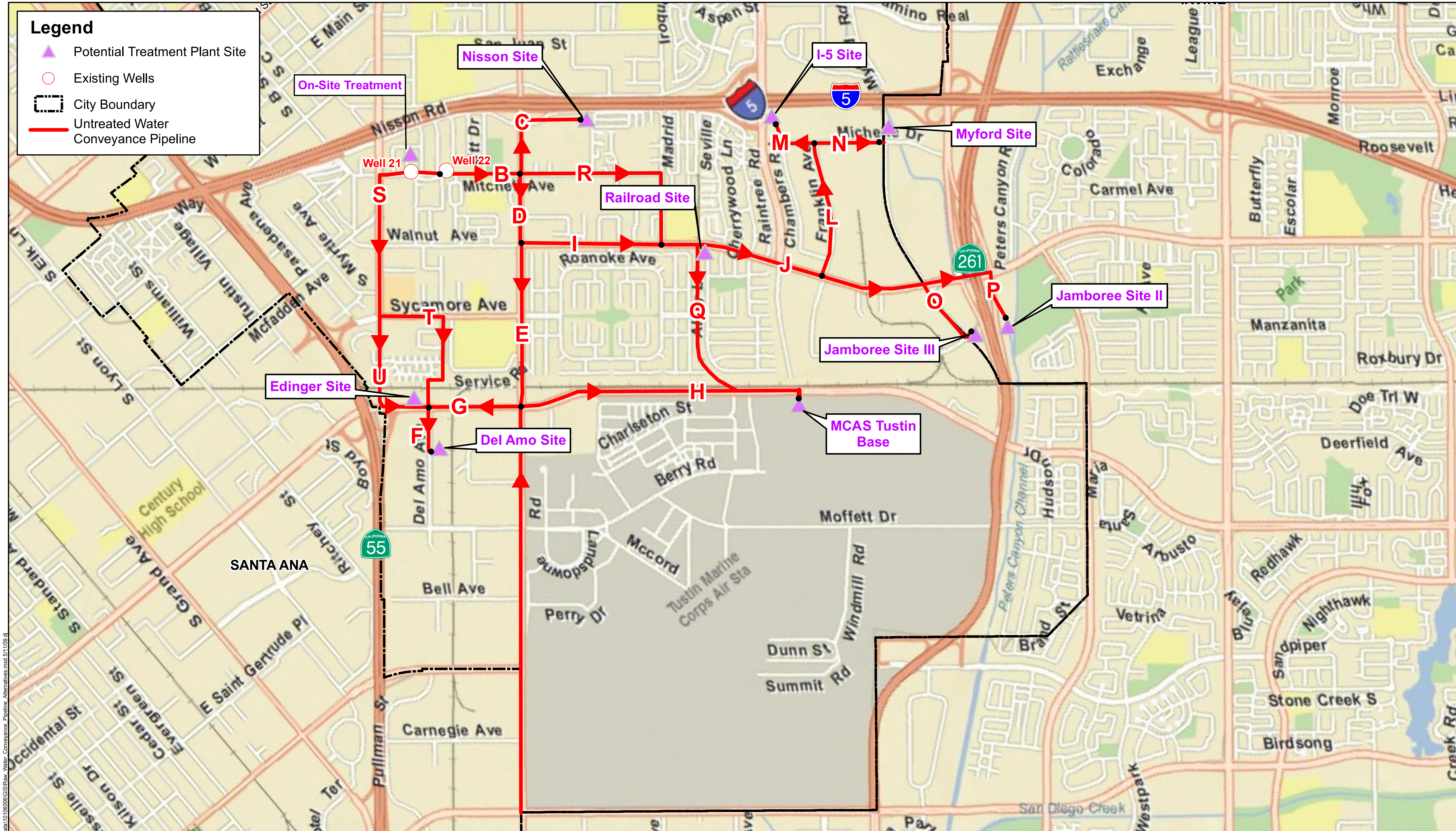
## 4.3 Untreated Water Conveyance Pipeline

### 4.3.1 General

The alignment investigation of the untreated water conveyance pipeline was predicated upon determining a primary route between the wells and proposed alternative treatment site locations. The alternative untreated water conveyance alignments to each of the 10 initial candidate treatment sites identified were used as a preliminary evaluation tool to ascertain those treatment sites that were advantageous in terms of pipe length required and those in more desirable right-of-ways in which to construct. These potential alignments were narrowed down considerably based on the imminent acquisition of Site D (1221 Edinger Avenue) for the Wells 21 and 22 treatment plant. All of the potential alignments that were evaluated for the initial evaluation are shown on Figure 4-15.

Based on the engineer's investigation, which included preliminary utility research, field reconnaissance, coordination with various agencies, and coordination with IRWD, the alignments that were evaluated in greater detail, specific to the 1221 Edinger Avenue treatment site, are shown on Figure 4-16. A summary description of each pipeline segment follows.

This page left intentionally blank



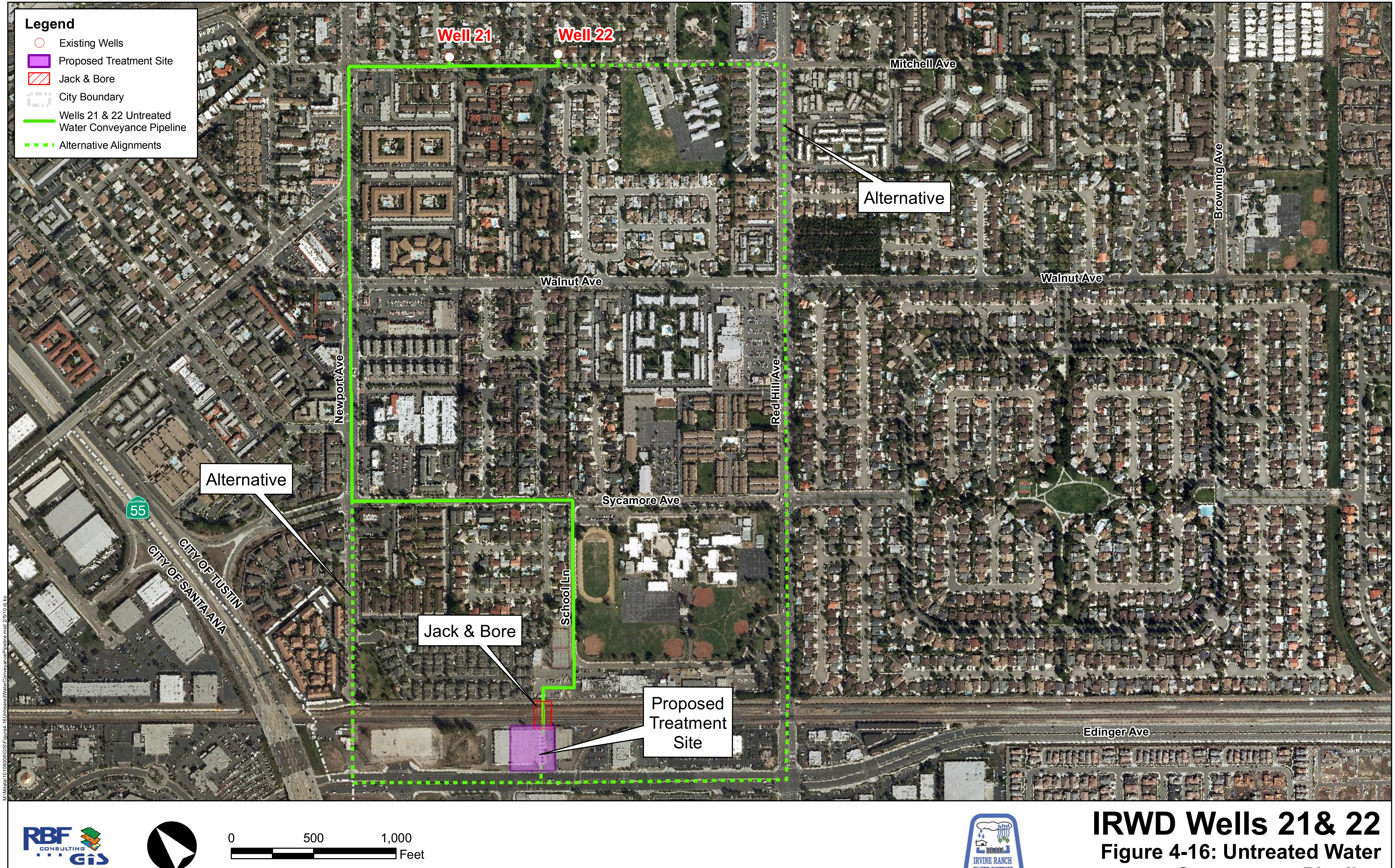

0 0.5 1 Miles

Source: ESRI Online

# IRWD Wells 21 & 22

## Figure 4-15: Untreated Water Conveyance Pipeline Alternatives

This page left intentionally blank.




Source: Eagle Aerial, 2009

**IRWD Wells 21& 22**
**Figure 4-16: Untreated Water Conveyance Pipeline**

This page left intentionally blank.

#### 4.3.1.1 Pipeline Alignment Descriptions

##### 4.3.1.1.1 Mitchell Avenue

Mitchell Avenue is a 2-lane roadway serving a mature residential area. Overall traffic is mild. Impacts to residents will be moderate to severe. Low hanging trees across portions of the right-of-way may also be an issue for larger construction equipment.

Mitchell Avenue was researched in both the east direction towards Red Hill Avenue and west to Newport Avenue. These are the main thoroughfares that will be investigated to continue the untreated water conveyance line south towards the Edinger Avenue treatment plant site.

Mitchell Avenue has several large existing utilities within the right-of-way. The proposed alignment between the Well 22 and Well 21 sites will utilize the space currently occupied by the abandoned IRWD 20-inch Francis Mutual pipeline. The Francis Mutual pipeline was considered for re-use, but preliminary video investigations during well rehabilitation indicated the pipeline is in poor condition. This portion of the new pipeline in Mitchell Avenue provides adequate space for the new pipeline, but will require removal of the abandoned pipeline, increasing cost and the duration of construction.

The alignment investigated west of well 21 extending to Newport Avenue will continue in the southern lane within Mitchell Avenue approximately 7 feet clear, to the south, of an existing OCSD sewer line. Additional design considerations will be required due to the pipeline being within the minimum 10 foot separation mandated by CDPH.

These requirements will be discussed as part of the preliminary design in Section 5.

4.3.1.1.2 Red Hill Avenue

Red Hill Avenue was considered as an alternative alignment for extending the untreated water conveyance pipeline south towards the treatment plant site. Red Hill Avenue is a 6-lane major thoroughfare with heavy traffic throughout the day. Impact to commuters and local traffic will be significant. Traffic control cost will be high and construction hours may be limited.

Red Hill Avenue is congested with existing utilities, in particular at the intersections of Mitchell Avenue and Walnut Avenue. Several large diameter OCSD trunk sewer lines run within the right-of-way as well as the 78-inch Orange County Feeder No.2. Utility relocations, trenchless construction methods and/or constructing the pipeline at greater depths may be necessary, increasing construction costs.

4.3.1.1.3 Newport Avenue

Newport Avenue is an alternative alignment for extending the untreated pipeline south from Mitchell Avenue.

Newport Avenue is a 4-lane major thoroughfare with moderate to high traffic throughout the day. Newport Avenue includes a median/center turn lane from Mitchell Avenue south to Sycamore Avenue. The center lane discontinues south of Sycamore Avenue.

Utilities are moderate within Newport Avenue, making the alignment more desirable as a southern route for the untreated line compared to Red Hill Avenue. There is a large storm drain, two domestic water lines, a gas line, and an 8-inch sewer among other dry utilities.

Along Newport Avenue, south of Sycamore Avenue, there is a proposed City of Tustin grade separation project for the crossing of the existing railroad tracks located just north of Edinger Avenue. The untreated water pipeline south of Sycamore Avenue would require relocation, at IRWD's cost, during the grade separation project.

4.3.1.1.4 Sycamore Avenue

Sycamore Avenue is a 2-lane residential street running east-west between Newport Avenue and Red Hill Avenue. Traffic along Sycamore Avenue is light to moderate. A Tustin Unified School District (TUSD) elementary school and middle school are located adjacent to this alignment at the Sycamore Avenue/School Lane intersection. This will increase traffic volumes during the morning and early afternoon hours. Construction may be limited during these times, should school be in session. Ideally, the pipeline would be constructed during the summer or other extended school break to minimize student/community impacts. School breaks include winter and spring recess as well as the summer break from mid-June to early September.

There are several smaller diameter utilities in Sycamore Avenue including an 8-inch sewer, 2-inch gas line, 8-inch water and a storm drain. Underground dry utilities include a telephone line on the south side of the right-of-way. The pipeline is proposed for the south side of the street, off of the curb approximately 6 feet to maintain adequate separation from the local gravity sewer and avoid the water and storm drain pipes crowding the north lane.

4.3.1.1.5 School Lane

School Lane is a 2-lane minor road extending south from Sycamore Avenue adjacent to an existing TUSD middle school, elementary school and District offices. The school traffic will be high volume during the morning and early afternoon hours while school is in session. Construction may be limited during these times. Similar to Sycamore Avenue, the pipeline would ideally be constructed during the summer or other extended school break to minimize student/community impacts. TUSD has a standard school schedule with extended breaks at the Christmas holiday and spring break in April along with summer break from mid-June to early September.

The pipeline along school lane is preliminarily aligned in the east lane approximately 6 feet off the existing curb. Utilities are light in School Lane with preliminary research indicating a local water and storm drain line within the right-of-way.

At the south end of school lane is the TUSD bus parking lot. The parking lot is north of, and adjacent to, a county flood control channel and railroad tracks bordering the north side of the proposed Wells 21 and 22 treatment plant site. The bus parking lot is proposed to be utilized for a jacking or receiving pit to cross the railroad tracks and channel directly into the treatment plant site. A construction and permanent easement will be required from TUSD for the pipeline.

4.3.1.1.6 Edinger Avenue

Edinger Avenue was investigated as a potential untreated water pipeline alignment either from the east if Red Hill Avenue was utilized or from the west if the Newport Avenue alignment was selected to extend south of Sycamore Avenue to Edinger Avenue.

Edinger Avenue is a 6-lane major thoroughfare with moderate traffic from Newport Avenue to Red Hill Avenue. There is an off ramp from the 55 freeway adjacent to Newport Avenue on the west that funnels traffic from the freeway onto Edinger Avenue.

Utility research indicated extensive existing utilities within the ROW along the potential untreated water conveyance alignments both from the east and west of the treatment facility. Utilities include two existing sewers, (15-inch and 21-inch diameter), two storm drains (16-inch and 36-inch diameter), underground electrical, an 8-inch water, underground telephone and a 30-inch high pressure gas line.

#### 4.3.1.2 Recommended Untreated Water Conveyance Pipeline Alignment

Based on the information gathered and preliminary analysis described above, the project alignment is comprised of the following segments:

- Mitchell Avenue (Well 22 to Newport Ave)
- Newport Avenue (South of Mitchell Ave)
- Sycamore Avenue (East of Newport Ave)
- School Lane (South of Sycamore Ave)

Summary information on each of the pipeline segments is given in Table 4-1. This alignment establishes the basis of the hydraulic and cost analysis provided herein, but will require confirmation in preparation of final design that should include additional utility research and local jurisdiction coordination.

**Table 4-1 Untreated Water Conveyance Pipeline Summary**

| Pipeline Segment Start Point | Pipe Location | Pipe Segment End Point | Approximate Length, ft | Diameter, in* | Special Crossings/Construction Issues |
|---|---|---|---|---|---|
| Mitchell Ave at Well 22 site | Mitchell Ave Right-of-Way | Mitchell Ave at Well 21 site | 700 | 16 | Removal of 20-inch Francis Mutual Pipeline |
| Mitchell Ave at Well 22 site | Mitchell Ave Right-of-Way | Newport Ave at Mitchell Ave | 1,300 | 24 | None |
| Newport Ave at Mitchell Ave | Newport Ave Right-of-Way | Newport Ave at Sycamore Ave | 2,650 | 24 | None |
| Newport Ave at Sycamore Ave | Sycamore Ave Right-of-Way | Sycamore Ave at School Lane | 1,350 | 24 | None |
| Sycamore Ave at School Lane | School Lane Right-of-Way | School Lane at Service Rd | 950 | 24 | None |
| School Lane at Service Rd | Through TUSD Property across RR/Flood Control Channel | Site D - (1221 Edinger Ave) Treatment Plant Site | 650 | 24 | Includes 300 ft jack and bore across railroad tracks and flood control channel south of TUSD school property (Easement Acquisition required) |
| | | **Approximate Total Length** | **7,600** | | |

* Sizes assume no future wells added to untreated water conveyance

## 4.4 Potential Connection Points

The District's existing potable water hydraulic model was utilized to determine potential tie-in (or connection) points to the Zone 1 Central System (HGL = 290 feet). To minimize transmission pipeline costs, Wells 21 and 22 product water is proposed to feed into the nearest Zone 1 facilities capable of accepting and distributing these supplies to IRWD customers, while adhering to the District's hydraulic standards. Connecting to the Zone 2 or Zone 3 systems would require miles of additional pipeline and/or crossing of the Interstate-5 freeway. This report excludes in-depth evaluation of facilities necessary to move water from Zone 1 to upper pressure zones.

Initially, eight (8) alternative connection points were selected for this evaluation based on (1) the proximity of the connection points to the ten alternative treatment sites initially considered, and on (2) the sizes of existing pipe in the vicinity of the connection point (Figure 4-17), while (3) trying to avoid major crossings (i.e., I-5 crossing). Additional factors were considered while evaluating the connection to the 54-inch DRWF transmission pipeline at the intersection of Barranca Parkway and Red Hill Avenue (Connection Point No.8 in Figure 4-17). This 54-inch transmission pipeline is capable of delivering flows up to 35,000 gpm under a velocity of 5 fps. Based on recent SCADA data provided by the District, DRWF supplies to the Zone 1 system currently range from approximately 16,000 gpm during low demand conditions in the winter, to approximately 39,000 gpm during peak hour demand conditions in the summer. Although this 54-inch transmission pipeline would be able to convey additional supplies from Wells 21 and 22 with acceptable velocities under most demand conditions, it is determined that the existing configuration and operation of the Zone 1 system does not favor additional supplies to the system at this (Red Hill-Barranca) connection point. This determination is based on the following considerations:

1. Considering the resources the District has spent over the years to optimize the operation of the DRWF, it is recommended that the impacts to DRWF be minimized while incorporating additional supplies to the Zone 1 system. Since the DRWF wells are expected to operate at slightly higher hydraulic grade lines the closer the connection point is located to these wells, connecting to the 54-inch transmission pipeline in Barranca is estimated to increase impacts on well pumping operations at the DRWF and reduce well field capacity.

2. The total dynamic head (TDH) required for the Wells 21 and 22 product water pumps at the treatment plant will need to be higher for the connection at Red Hill and Barranca, and as a result, energy costs would increase.

3. IRWD's preferred use of future Zone 1 supply sources, including Wells 21 and 22 supplies, is to transfer them to IRWD's upper zones, where most future water demand increases are expected. Currently, three booster pump stations transfer water to IRWD's Zone 3 from Zone 1. Since the Sand Canyon booster pump station (BPS) has the largest pumping capacity, it is the preferred destination of all additional supplies introduced to the Zone 1 system. Based on the geographic location of the proposed treatment plant and the Sand Canyon BPS, Connection Points No. 7 and No.8 were not favorable when compared to other connections located on the north side of the Zone 1 system.

4. Existing utilities and traffic conditions along Red Hill Avenue were also considered unfavorable when compared to other roadways located north of the MCAS Tustin Base (i.e Edinger Avenue).

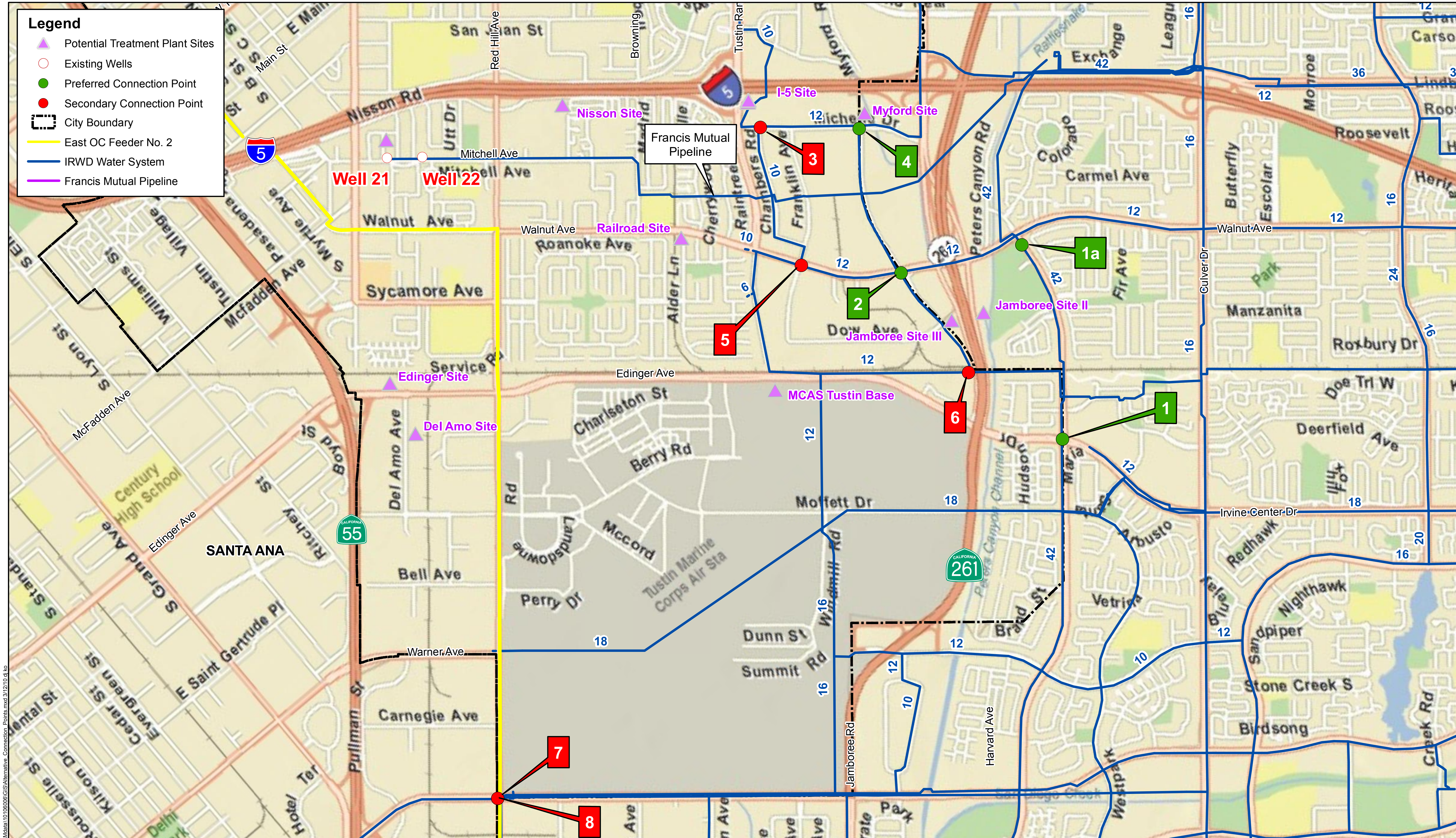
After reviewing the results of the hydraulic model runs, and evaluating the direction of flow, velocity, and capacity of the relative pipelines to each connection point, three of the eight connection points proved to be viable. Due to the capacity of the 42-inch pipeline in Harvard Avenue, and its ability to absorb Wells 21 and 22 supplies (as well as possible additional supplies from future wells that may also be treated at the treatment plant site) with minimal impact on pipeline velocity, treated water supplied by Wells 21 and 22 is recommended to enter the Zone 1 system at Harvard Avenue. A summary of the alternative connection points, hydraulic model runs, and the estimated connection point capacities are shown in Table 4-2.

**Table 4-2 Alternative Connection Points Summary**

| Alternative Connection Points | | | Velocity Criteria (fps) [1] | Estimated Max Q [2] | | Rank | Hydraulic Model Max Q [3] | Rank |
|---|---|---|---|---|---|---|---|---|
| ID | Pipe Size | Street | | (cfs) | (gpm) | | (gpm) | |
| 1 | 42" | Harvard & Edinger | 5 | 48.1 | 21,600 | 1 | 40,000 | 1 |
| | | | 8 | 77.0 | 34,500 | | 58,000 | |
| 1a | 42" | Harvard & Walnut | 5 | 48.1 | 21,600 | | 25,000 | |
| | | | 8 | 77.0 | 34,500 | | 63,000 | |
| 2 | 12",12",12",& 12" | Myford & Walnut | 5 | 15.7 | 7,100 | 2 | 6,000 | 2 |
| | | | 8 | 25.1 | 11,300 | | 9,800 | |
| 3 | 12",12", & 10" | Chambers & Michelle | 5 | 10.6 | 4,700 | 7 | 3,500 | 7 |
| | | | 8 | 16.9 | 7,600 | | 6,700 | |
| 4 | 12",12", & 16" | Myford & Michelle | 5 | 14.8 | 6,700 | 3 | 5,900 | 3 |
| | | | 8 | 23.7 | 10,700 | | 9,700 | |
| 5 | 12",12", & 12" | Walnut & Franklin | 5 | 11.8 | 5,300 | 5 | 4,000 | 6 |
| | | | 8 | 18.8 | 8,500 | | 6,900 | |
| 6 | 12",12", & 16" | Myford & Edinger | 5 | 14.8 | 6,700 | 3 | 5,500 | 4 |
| | | | 8 | 23.7 | 10,700 | | 9,100 | |
| 7 | 12" & 16" | Barranca & Red Hill | 5 | 10.9 | 4,900 | 6 | 4,300 | 5 |
| | | | 8 | 17.5 | 7,800 | | 7,500 | |

[1] Based on IRWD standards, maximum pipeline velocity under average and peak demand conditions is equal to 5 and 8 fps, respectively.
[2] Maximum pipeline capacity calculated based on pipeline size and velocity criteria
[3] Maximum pipeline capacity estimated using IRWD hydraulic model and velocity criteria

This page left intentionally blank




RBF CONSULTING GIS

Source: ESRI Online

0 0.5 1 Miles

IRVINE RANCH WATER DISTRICT

**IRWD Wells 21& 22**

**Figure 4-17: Alternative Connection Points**

This page left intentionally blank.

## 4.5 Product Water Pipeline

### 4.5.1 Alternative Alignments Evaluation

The alternative product water conveyance alignments from each of the 10 initial candidate treatment sites to Harvard Avenue were used as a preliminary evaluation tool to ascertain those treatment sites that were advantageous in terms of pipe length required and those in more desirable right-of-ways in which to construct. These potential alignments were narrowed down considerably based on the selection of Site D (1221 Edinger Avenue) for the treatment plant. All of the potential alignments that were initially evaluated are shown on Figure 4-18.

The product water pipeline is required to extend from the treatment plant site to the Harvard Avenue connection. Several options were explored as potential alignments including the primary east-west route along Edinger Avenue, and alternative SR-241/Peters Canyon Channel crossings along Walnut Avenue and Moffett Drive.

The alternative alignment to reach Walnut Avenue would require a jack and bore section across the railroad tracks north of Edinger Avenue. The Moffett Drive alignment adds a considerable amount of pipe to the alignment and contains a limited right-of-way congested with utilities on the east side of Peters Canyon Channel. Therefore, the recommended alignment, which is also the shortest route, extends from the treatment plant along Edinger Avenue, easterly, to the connection point in Harvard Avenue. The recommended alignment and alternatives is shown on Figure 4-19.

### 4.5.2 Major Crossings

Two major crossings are associated with this alignment: the crossing of SR-261 Highway and the Peters Canyon Channel. A field visit was conducted that indicated SR-261 has adequate height under the existing overpass across Edinger Avenue for conventional construction. The Edinger Avenue bridge is a box girder type crossing Peters Canyon Channel. The bridge was widened in 1997 to allow three lanes of traffic in each direction plus a median.

Five options were considered for crossing of the drainage channel:

- Attach to the side of the bridge with custom designed pipe supports
- Utilize the two (2) 20-inch x 20-inch existing utility cells within the bridge for placement of product water pipelines
- Jack and bore construction with jacking and receiving pits on either side of the channel
- Horizontal Directional Drill (HDD) pipeline with entry and exit pits on either side of the channel
- Construct a separate pedestrian/pipe bridge across the channel to support the product water pipe.

The recommended approach is to attach the pipeline to the south side of the Edinger Avenue bridge. Two small existing conduits are attached to the bridge and would require relocation by attaching the conduits to the new bridge crossing support. A preliminary structural analysis and preliminary design of the product water pipe supports, which show the option as a feasible engineered solution, is included in Section 5. The bridge support would require modification to a cantilever type design if the pipeline were sized greater than 20-inch diameter. A tee should be provided on the upstream side of the bridge crossing to accommodate a potential, smaller diameter (approximately 10-inch) bypass line should expansion of the wells 21 and 22 system be greater than expected, allowing the District to utilize the bridge cells in the Peters Canyon Bridge for additional capacity.

The existing utility cells were eliminated from consideration as the primary route as the size of a pipeline would be limited at 10 to 12-inch diameter maximum. These cells could be utilized in the future to reduce velocities in the main line if expansion of the system is greater than currently anticipated.

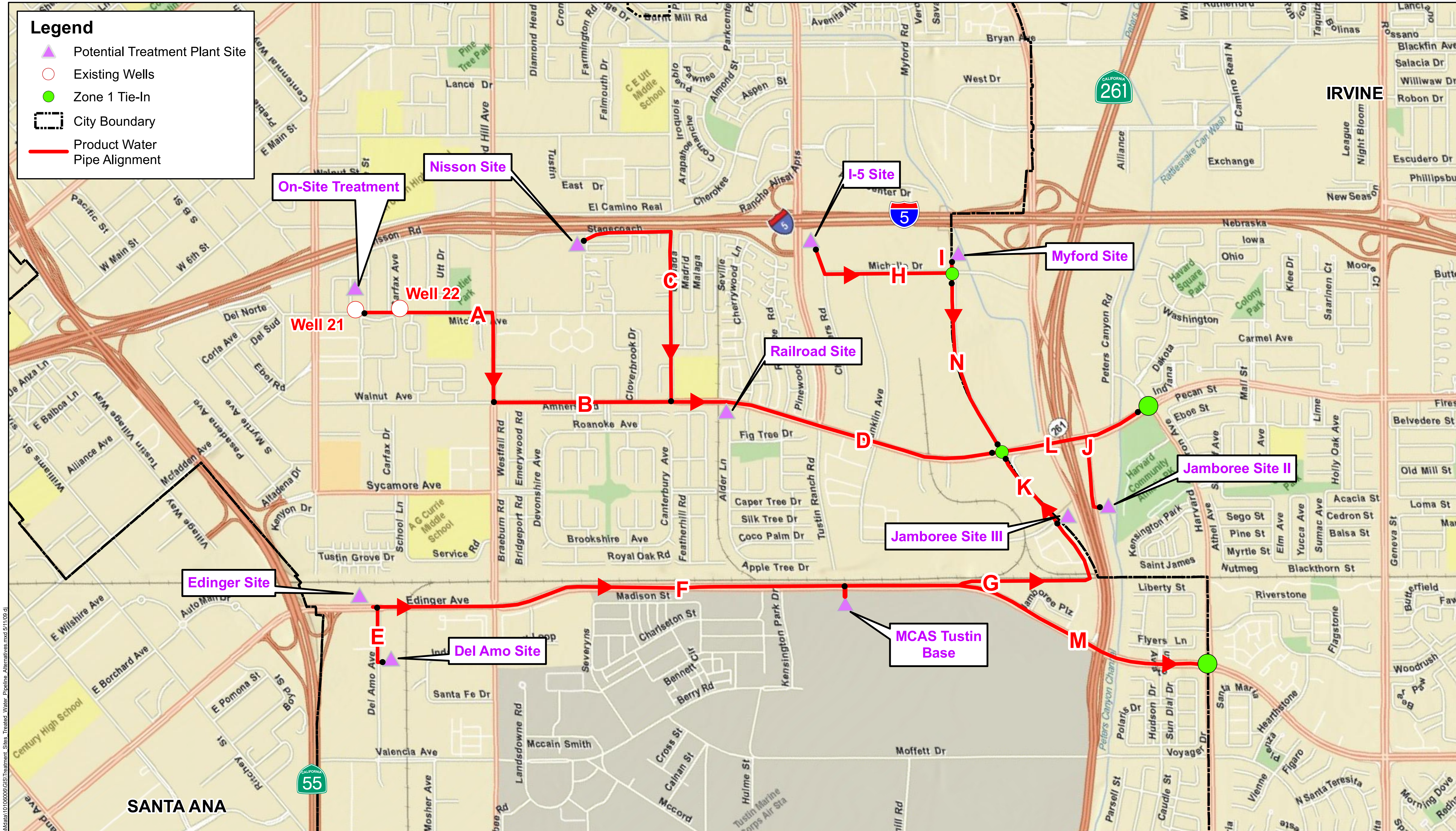
The trenchless techniques, jack and bore and HDD, are considered the most expensive of the alternatives and have limited space on the east side of the channel, outside of the roadway, for a receiving pit or exit point and the associated equipment. The trenchless construction could occur within the roadway on either sides of the channel, however, traffic and public impacts would be great. These trenchless alternatives have not been eliminated from consideration, but are more costly, difficult to construct and have greater public impact. For these reasons, trenchless construction will be considered only as an alternative to the recommended approach.

Construction of a pipe bridge that could potentially double as a pedestrian bridge was considered for the crossing. The Peters Canyon Channel at Edinger Avenue would necessitate a pipe bridge with a span exceeding 200 feet and, based on preliminary structural calculations and discussions with several manufacturers, would require construction of an intermediate support. This would be costly and difficult to permit and construct within the flood control channel. The pipe bridge was eliminated from primary consideration due to these factors.

A photo of the bridge section is shown below:

**Edinger Avenue Bridge over Peters Canyon Channel**

This page left intentionally blank




0 0.25 0.5
Miles

Source: ESRI Online

M:\Mdata\10106006\GIS\Treatment_Sites_Treated_Water_Pipeline_Alternatives.mxd 5/11/09 di

## IRWD Wells 21 & 22

### Figure 4-18: Product Water Pipeline Alternatives

This page left intentionally blank.


RBF CONSULTING GIS

0 1,000 2,000 Feet

Source: Eagle Aerial, 2009

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21& 22

## Figure 4-19: Product Water Transmission Pipeline

This page left intentionally blank.

## 4.6 Brine Disposal Alternatives

### 4.6.1 Brine Waste Destination Evaluation

Brine disposal is a critical element for this project. The recommended treatment process is anticipated to have an overall recovery rate (including raw water bypass) near 89 percent, meaning approximately 11 percent of the treated flow will require disposal. Four alternatives were identified as options for disposal of brine waste. Figure 4-20 is a regional map depicting these four disposal options. The evaluation of alternatives for brine discharge is discussed below:

#### 4.6.1.1 Orange County Sanitation Districts (OCSD) Plant No.1

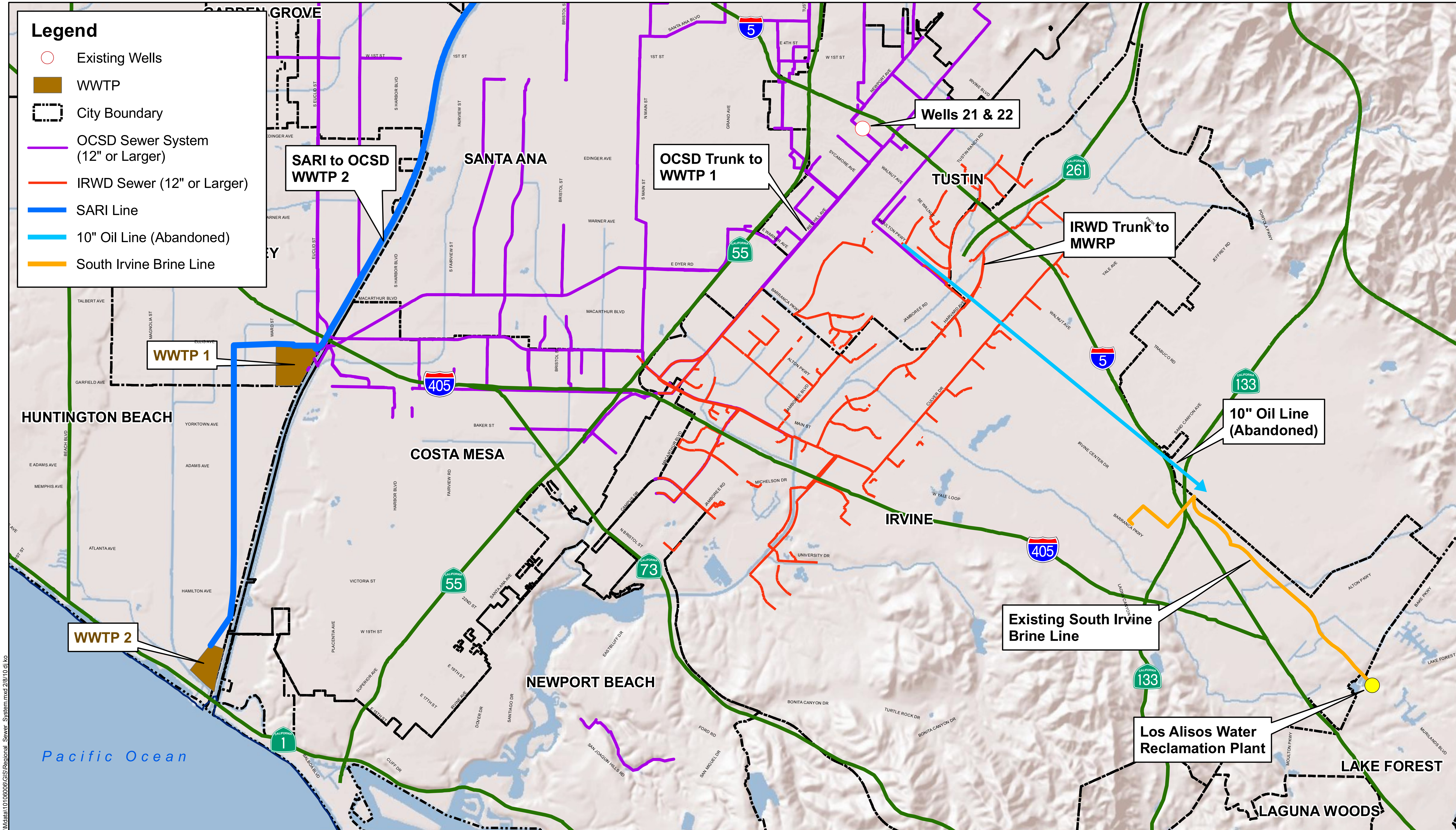
Disposal to OCSD's existing sewer system draining to OCSD Wastewater Treatment Plant No.1 is a viable option considering the relatively close proximity to the project facilities. There are two OCSD trunk lines that run in the Red Hill Avenue right-of-way relatively near the proposed treatment plant site. The issue associated with discharge to OCSD is the increased levels of TDS that will be present in the brine stream. OCSD has undertaken a large scale groundwater replenishment program fed by reclaimed water through OCSD Plant No.1. OCSD indicated that any discharge high in TDS results in cost impacts to the groundwater replenishment project.

OCSD has precedence of allowing brine discharge from a similar water treatment facility in the City of Tustin. OCSD staff has also indicated in informal conversation that they would consider permitting the brine discharge, but that it will need to be reviewed to confirm and could include a fee surcharge, above normal special discharge permit costs, for the additional TDS load at Plant No.1. Since these costs are undefined, they are excluded from the cost estimates at this time.

#### 4.6.1.2 OCSD Santa Ana Regional Interceptor (SARI) Line

The SARI pipeline, which drains to OCSD Plant No.2, is approximately 5 miles from Wells 21 and 22; however, this was evaluated as an option due to the favorable disposition of

This page left intentionally blank




Source: ESRI Online, IRWD, OCSD

# IRWD Wells 21 & 22

## Figure 4-20: Brine Disposal Alternatives

This page left intentionally blank.

OCSD to permit discharge of high salinity waste streams to this line. SARI line is located along the west side of the Santa Ana River Channel, opposite that of Wells 21 and 22, necessitating a crossing of the river to reach the SARI pipeline. A pump station for boosting the brine flow to reach the SARI line would also be necessary. The long distance, pumping requirements, and crossing of the Santa Ana River make the SARI option undesirable due to extremely high construction and on-going operations costs.

4.6.1.3 IRWD's Michelson Plant

Discharge to IRWD's nearby sewer collection system draining to the Michelson Water Reclamation Plant was evaluated. In discussions with IRWD staff, it was indicated that the District prefers not to discharge to the Michelson plant given the sensitivity of their recycled water customers to high saline water. Pipe lengths to reach an adequately sized IRWD sewer would be extensive.

4.6.1.4 10-Inch Jet Fuel Line

Brine disposal via an existing, abandoned 10-inch fuel pipeline owned by Kinder Morgan Partners that would eventually discharge into the South Irvine Brine line was evaluated. This system would ultimately feed to the Los Alisos Water Reclamation Plant. The owner of the 10-inch line has indicated to IRWD and RBF staff that they are not willing to sell the pipeline outright and would not consider releasing operation and maintenance to IRWD resulting in on-going leasing and operations cost from the owner. It has also been speculated that, due to age, the pipeline may be in poor condition or worse in some sections. Pipe lengths from the acquired treatment plant to the fuel line would be extensive.

4.6.1.5 Brine Waste Destination Recommendation

OCSD Plant No.1 is the preferred end location for the brine flow from the treatment plant due to the relative close distance to existing OCSD facilities. The issues associated with the other options, whether cost, customer disruption, or other, deemed the other three options not conducive to handle the waste stream. OCSD had initially indicated a moderate willingness to discuss the option of discharging to Plant No.1, although they have indicated potential concern related to the impacts to their groundwater recharge project, which could result in additional fees or denial of discharge due to the high TDS brine discharge. IRWD is currently coordinating with OCSD in regard to the viability of discharging to Plant No.1 as proposed.

### 4.6.2 Brine Disposal Pipeline Alternatives Evaluation

Four alternative connection points were evaluated for the conveyance of brine from the treatment plant. The alternative connection points evaluated are located at various points along OCSD's trunk sewer system in the City of Tustin, which are tributary to OCSD's Plant No.1. The nearest sewer facilities regarded as potential connection points (Figure 4-21) are described as follows:

#### 4.6.2.1 21-inch Edinger Avenue Sewer

The nearest facility to the treatment site is a 21-inch gravity sewer located in Edinger Avenue. This sewer conveys wastewater flows easterly to a 21-inch trunk sewer in Red Hill Avenue. Wastewater flows are then conveyed southerly to OCSD's Plant No.1. The characteristics of this existing sewer include the following:

- Size: 21-inch
- Average Invert Depth: 10 feet bgs
- Slope: 0.0051
- Estimated Capacity at 75% Full[1]: 4,600 gpm
- Estimated Excess (Available) Capacity: Unknown
- Approx. Distance from Treatment Site: 85 feet

Due to the close proximity of this 21-inch Edinger sewer to the treatment site, this alternative connection point would be most

[1] Estimated sewer capacities shown herein assume a Manning's n of 0.013

cost-effective for this project, provided that excess capacity in is available .

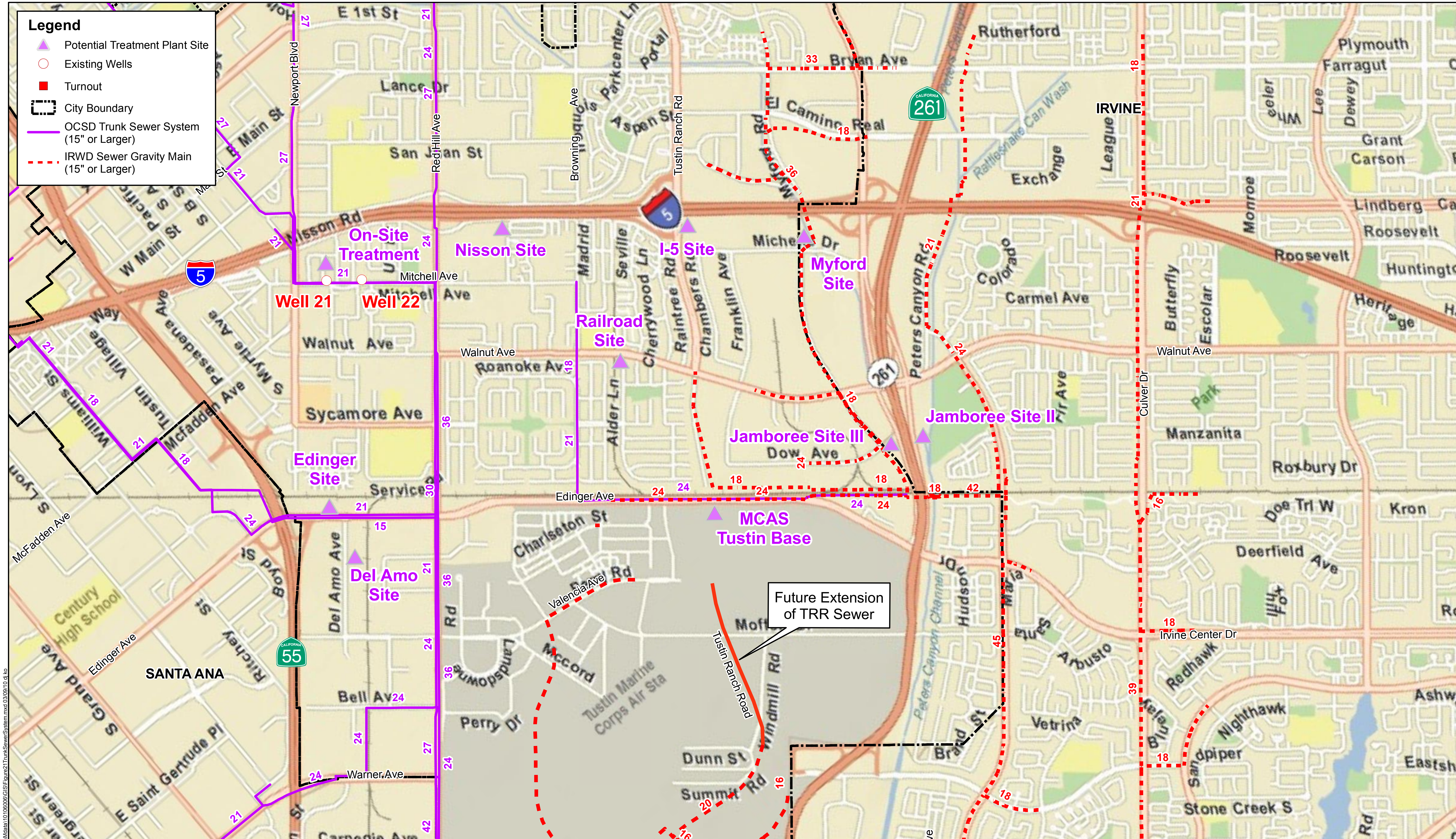
4.6.2.2 15-inch Edinger Avenue Sewer

The second nearest facility to the treatment site is a 15-inch gravity sewer that runs parallel to the existing 21-inch sewer in Edinger Avenue. This trunk sewer also conveys wastewater flows easterly to the 21-inch trunk sewer in Red Hill Avenue. The characteristics of this existing trunk sewer include the following:

- Size: 15-inch
- Average Invert Depth: 15.5 feet bgs
- Slope: 0.004
- Estimated Capacity at 75% Full: 1,650 gpm
- Estimated Excess (Available) Capacity: Unknown
- Approx. Distance from Treatment Site: 140 feet

Due to the relatively low capacity estimated for this trunk sewer, it is assumed that this sewer reach will not be able to convey ultimate brine discharges from the proposed treatment plant. However, further verification of excess capacity will be required to completely discard this alternative.

This page left intentionally blank




Source: ESRI Online

# IRWD Wells 21& 22

## Figure 4-21: Trunk Sewer System

This page left intentionally blank.

#### 4.6.2.3 36-inch/24-inch Red Hill Avenue Trunk Sewer

OCSD's 36-inch trunk sewer in Red Hill Avenue, located approximately 1,800 feet southeast of the proposed treatment plant site at the intersection of Red Hill and Edinger, is also tributary to OCSD's Plant No.1. GIS data obtained form OCSD indicate that this sewer, which conveys wastewater flows southerly, is reduced in size at Bell Avenue to a 24-inch diameter sewer, with a lower slope.

The characteristics of both 36-inch and 24-inch portions in Red Hill Avenue sewer reach are summarized below:

36-inch Red Hill Sewer, North of Bell Avenue:

- Size: 36-inch
- Average Invert Depth: 6 feet bgs
- Slope: 0.0034
- Estimated Capacity at 75% Full: 15,900 gpm
- Estimated Excess (Available) Capacity: Unknown
- Approx. Distance from Treatment Site: 1,800 feet

24-inch Red Hill Sewer, South of Bell Avenue:

- Size: 24-inch
- Average Invert Depth: 20 feet bgs
- Slope: 0.0026
- Estimated Capacity at 75% Full: 4,700 gpm
- Estimated Excess (Available) Capacity: Unknown

Despite the large capacity estimated for the 36-inch portion of this Red Hill sewer reach, the capacity constraint at the downstream 24-inch portion of this reach governs any excess capacity that may be available in this Red Hill sewer system for this project.

4.6.2.4 42-inch Red Hill Sewer

In the absence of hydraulic model or flow monitoring data, the existing 42-inch trunk sewer at the intersection of Red Hill Avenue and Warner Avenue is expected to have the largest excess capacity available for the project as indicated below:

- Size: 42-inch
- Average Invert Depth: 20 feet bgs
- Slope: 0.0026
- Estimated Capacity at 75% Full: 21,000 gpm
- Estimated Excess (Available) Capacity: Unknown
- Approx. Distance from Treatment Site: 7,000 feet

Based on the sewer capacities estimated for all four alternative connection points mentioned herein, and until further information is available to identify existing wastewater flows and determine excess capacities, this PDR assumes the brine disposal force main (or brine line) for the Project will extend from the proposed treatment plant site to the existing 42-inch trunk sewer at Red Hill Avenue and Warner Avenue.

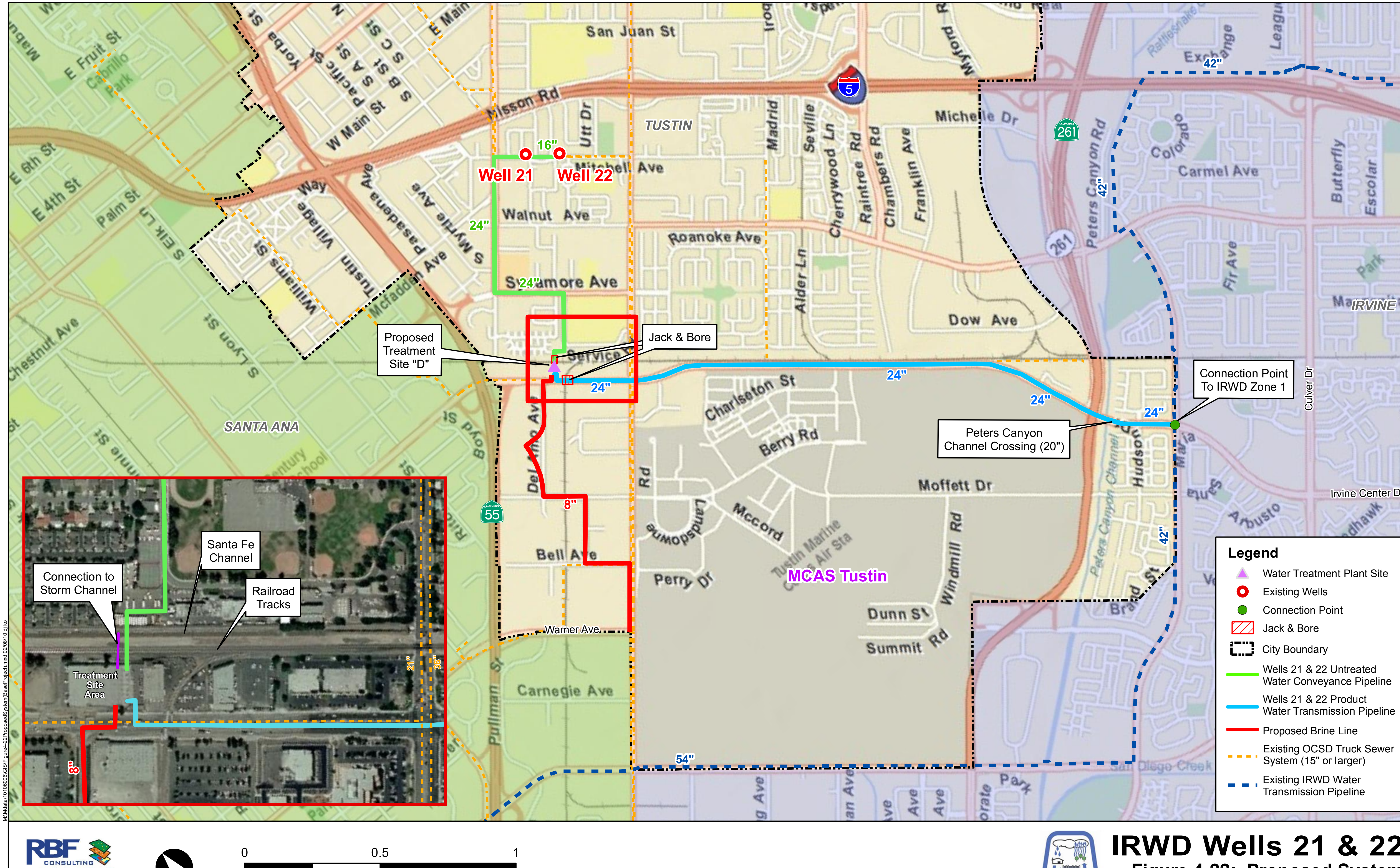
## 4.7 Summary of Recommendations

The alternatives and evaluations described examined various alternatives for integrating Wells 21 and 22 into IRWD's domestic water supply system. Based on these evaluations, the following system has been recommended for the project (Figure 4-22):

- Construct wellhead facilities capable of extracting the recommended flow rates from Wells 21 and 22 as described in Section 2.
- Construct an RO membrane treatment plant on the acquired Site D parcel located at 1221 Edinger Avenue.

- Construct an untreated water conveyance pipeline from Wells 21 and 22 to the treatment plant site utilizing the following right-of-ways:
  - Mitchell Avenue (From Wells 21 and 22, west, to Newport Avenue)
  - Newport Avenue (From Mitchell Avenue, southerly, to Sycamore Avenue)
  - Sycamore Avenue (From Newport Avenue, easterly, to School Lane)
  - School Lane (From Sycamore Avenue, southerly, across RR tracks and flood control channel to the treatment plant site)
- Construct a product water pipeline from the treatment plant, easterly along Edinger Avenue to Harvard Avenue for connection to IRWD's existing 42-inch transmission main located within Harvard Avenue.
- Construct a brine disposal force main from the treatment plant to connect to OCSD's 42-inch trunk sewer at the intersection of Red Hill Avenue and Warner Avenue.

This page left intentionally blank




Source: ESRI Online

# IRWD Wells 21 & 22

## Figure 4-22: Proposed System (Base Project)

This page left intentionally blank.

# Section 5

# Preliminary Design

RBF CONSULTING

This page left intentionally blank.

# Section 5 Preliminary Design

## 5.1 General

This section incorporates the hydrogeology data gathered from Wells 21 and 22 as described in Section 2, the evaluation of untreated water quality and treatment processes from Section 3, the recommendations regarding project facilities from Section 4, and incorporates the design standards described herein to produce the preliminary design for the Wells 21 and 22 Project.

## 5.2 Well Equipping

Existing conditions and construction details for existing Wells 21 and 22 are included in Section 2 of this report. The existing wells will require wellhead equipment, as described herein, for extraction of untreated groundwater at the recommended flow rates.

### 5.2.1 Well Sites

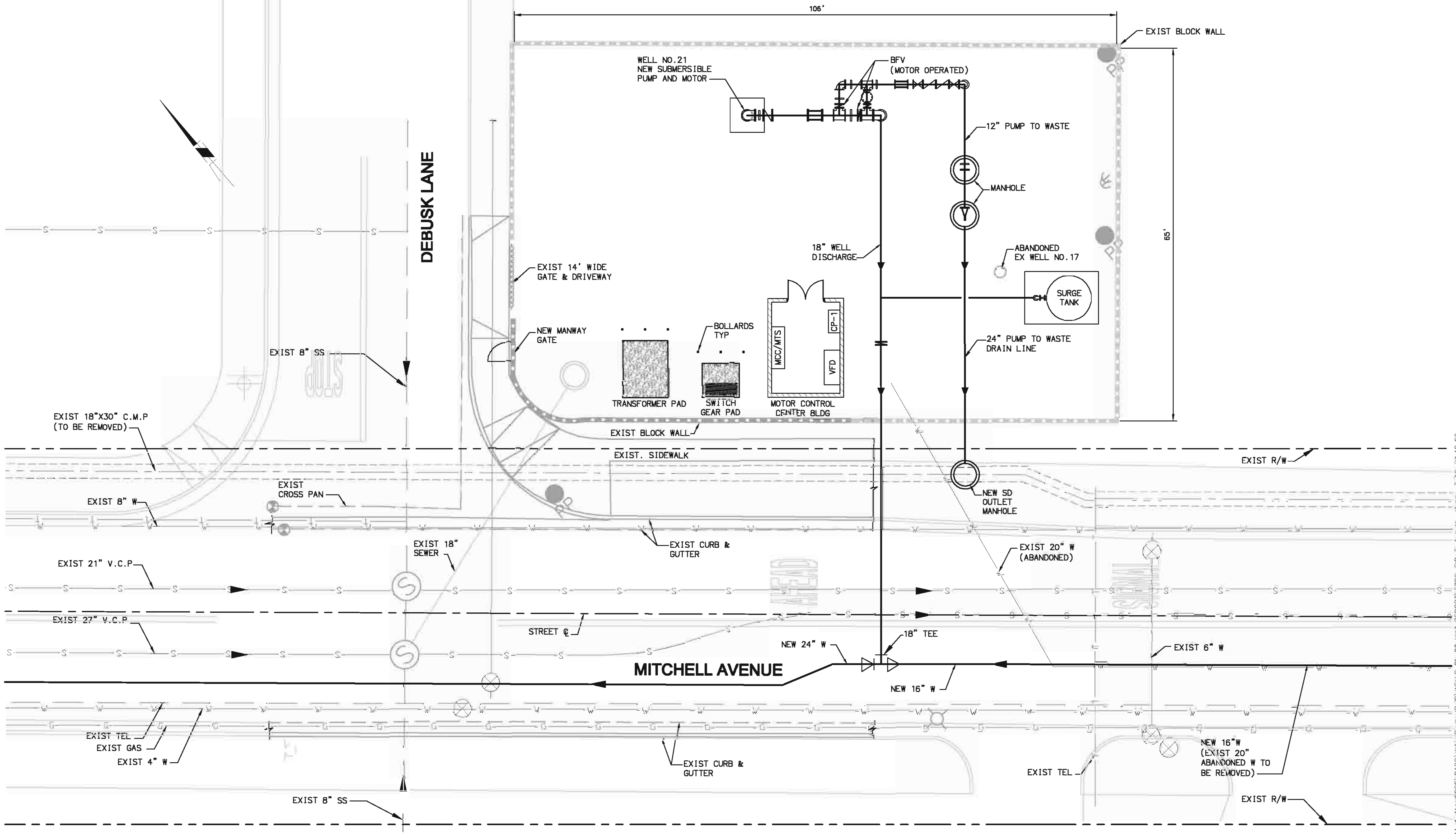
#### 5.2.1.1 Well 21

The Well 21 site is approximately 0.16 acres in size. A 6-foot tall masonry block wall encloses the site on all sides. The entrance to the site is off Debusk Lane, a small residential cul-de-sac off Mitchell Avenue.

The well is situated approximately 11 feet clear of the existing northeast site wall, and very near to an existing residence. Due to the proximity of the well with respect to existing residential, a submersible pump and motor is preferred by the District for noise attenuation as opposed to an above ground motor housed in a building. The advantages and disadvantages of the submersible configuration in this situation are discussed later in this section.

A preliminary site plan has been developed that depicts the existing site with the proposed well head equipment improvements. The Well 21 preliminary equipping plan is shown in Figure 5-1.

This page left blank intentionally




**PLAN**
NO SCALE

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

WELL 21
PRELIMINARY SITE PLAN

FIGURE
**5-1**

This page left intentionally blank.

#### 5.2.1.2 Well 22

The Well 22 site is approximately 0.14 acres in size. A 6-foot tall masonry block wall encloses the site on all sides. The entrance to the site is through an existing driveway and slide gate off Mitchell Avenue. The well is situated approximately 17 feet clear of the existing northeast site wall. The well site is surrounded on three sides by existing single family residential units. Due to the proximity of the well with respect to existing residential, a submersible pump and motor is preferred by the District for noise attenuation as opposed to an above ground motor housed in a sound attenuated building. The advantages and disadvantages to the submersible in this application will be discussed later in this section.

A preliminary site plan has been developed that depicts the existing site with the proposed well head equipment improvements. The Well 22 preliminary plan is shown in Figure 5-2.

### 5.2.2 Well 21 and 22 Water System Hydraulics

The primary hydraulic system curves were developed for the wells using the data and assumptions shown in Table 5-1. Candidate pump curves were superimposed over the proposed system curves. The preliminary horsepower requirement was then determined to estimate electrical loads for each site. Figures 5-3 and 5-4 depict the preliminary hydraulic curves for Wells 21 and 22, respectively.

Based on the potential for variance in water levels as discussed in Section 2, potential changes in long-term specific capacity of the wells and potential well drawdown interference a range of static and dynamic water levels were used to develop low, average, and high head system curves. Based on a review of these curves, the sizeable gap between low, average, and high head operating conditions lends itself to the use of variable frequency drives (VFD) to control well output by varying speed of the pump as opposed to throttling a control valve for flow control. In addition to the well hydraulics the VFD is recommended to provide an added level of control for the untreated water feed to the high pressure RO feed pumps at the treatment facility.

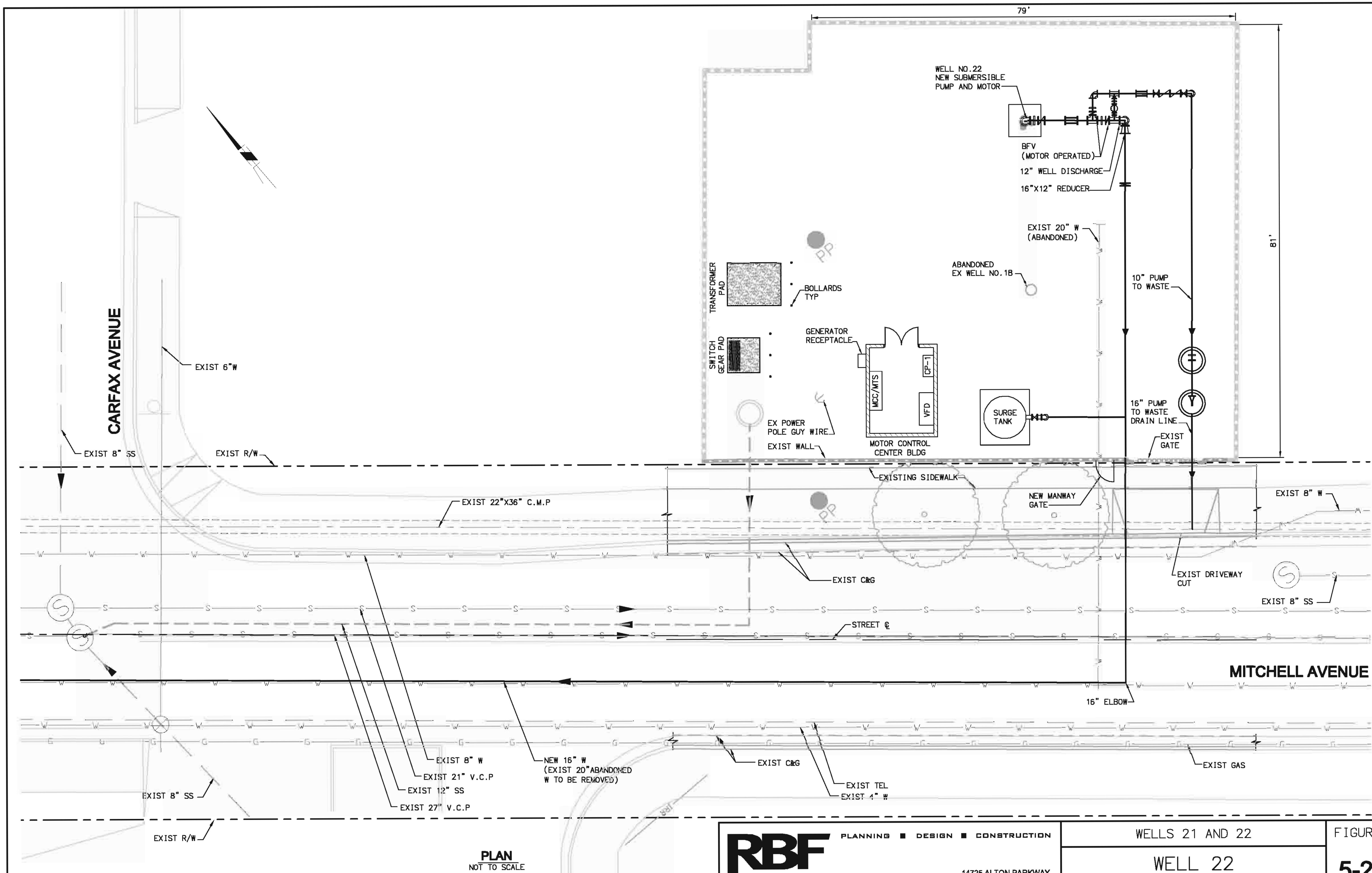
**Table 5-1 Wells 21 and 22 Hydraulic Design Criteria**

| Design Criterion | Value |
|---|---|
| **Well 21** | |
| Flow Rate, gpm | 3300 |
| Wellhead Elevation, ft | 111 |
| Static Water Level (high/avg/low), ft bgs[1] | 45/104/141 |
| Specific Capacity (low/avg/high), gpm/ft | 57/67/77 |
| Approx. Pump Setting Depth, ft[3] | 270 |
| Column Pipe Diameter, in | 12 |
| **Well 22** | |
| Flow Rate, gpm | 1600 |
| Wellhead Elevation, ft | 108 |
| Static Water Level (high/avg/low), ft bgs[1] | 79/154/210 |
| Specific Capacity (low/avg/high), gpm/ft | 14/21/28 |
| Approx. Pump Setting Depth, ft3 | 460 |
| Column Pipe Diameter, in | 10 |
| **Edinger Avenue Treatment Plant** | |
| Treatment Plant Elevation, ft msl | 86 |
| Treatment Plant Delivery Pressure, psi | 40-60 |
| **Discharge Piping** | |
| Hazen Williams "C" | 110-150 (130 Avg) |
| Design Velocity, fps | 3-5 |

1 Includes estimate of well pumping interference described in Section 2

Static Water Level ranges were derived from a thorough review of historical water levels at the well sites. It was noted by the hydrogeologist that, at the time of most recent pump testing, the static levels were at or near their lowest levels on record, therefore, to provide a factor of safety to the pump hydraulics design these low basin water levels were used as “average” or “typical” conditions. An additional 30 feet of static drop was added to the high head hydraulic conditions to account for an even greater regional static decline in water levels. The high head condition also assumed deterioration in well specific capacity, as indicated above, which increases the amount of pumping drawdown considered in the hydraulics. An assumed 50 feet of static water level rise was accounted for in the low head condition along with an assumed increase in the specific capacity, as shown above. The static water level ranges indicated in Table 5-1 also take into account the estimates of additional drawdown from pumping interference provided by the hydrogeolgist in Section 2.

These assumptions provide a conservative range of system curves indicative of the potential water level volatility in the basin.




**PLAN**
NOT TO SCALE

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 • FAX 949.472.8373 • www.RBF.com

WELLS 21 AND 22

WELL 22
PRELIMINARY SITE PLAN

FIGURE
5-2

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\6006-FIG-5-2.DWG JABDI 3/10/10 11:19 am

This page left intentionally blank.

**FIGURE 5-3**
**IRVINE RANCH WATER DISTRICT**
**WELLS 21 AND 22 PRELIMINARY DESIGN REPORT**
**WELL 21 PUMP AND SYSTEM CURVE**

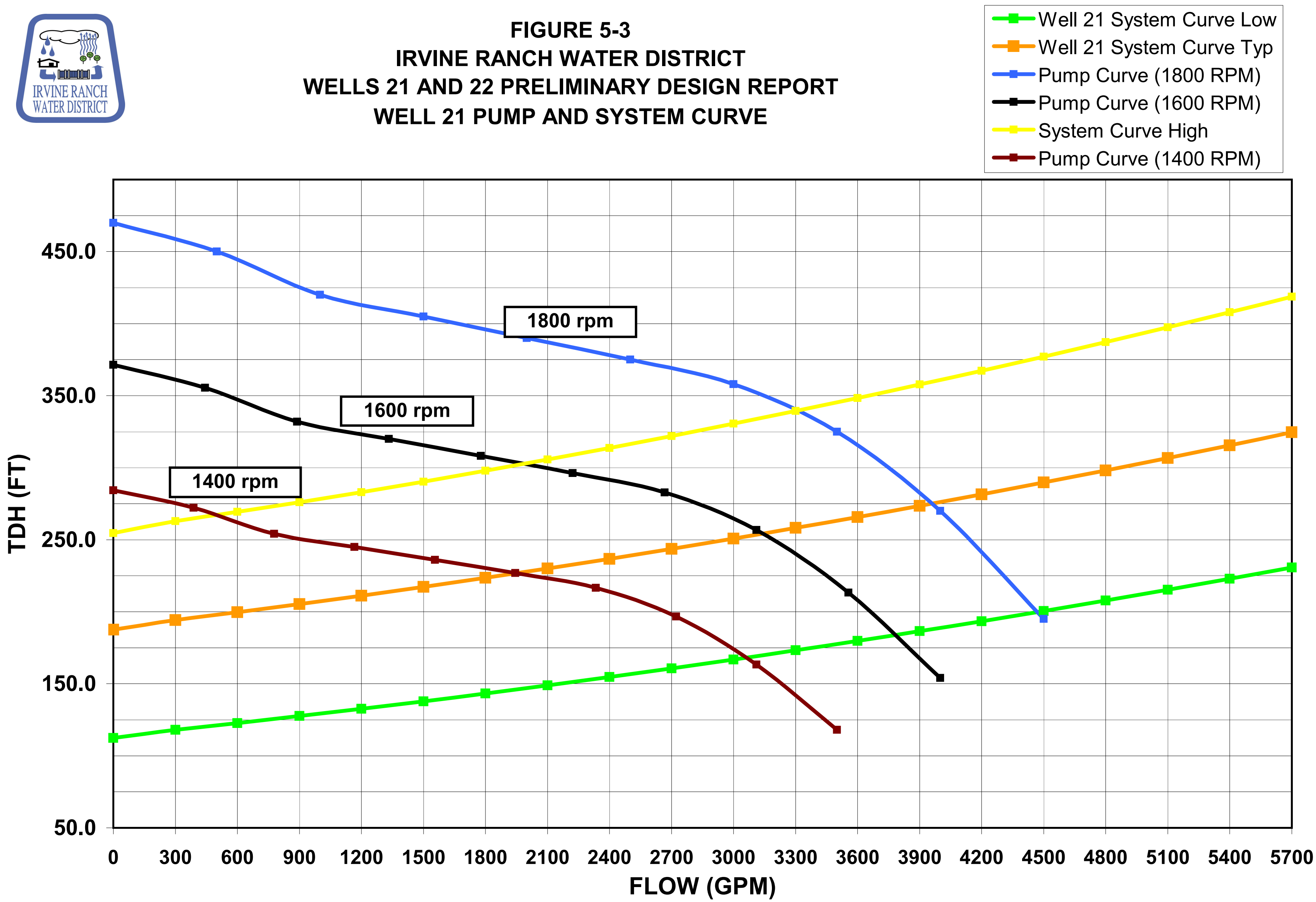
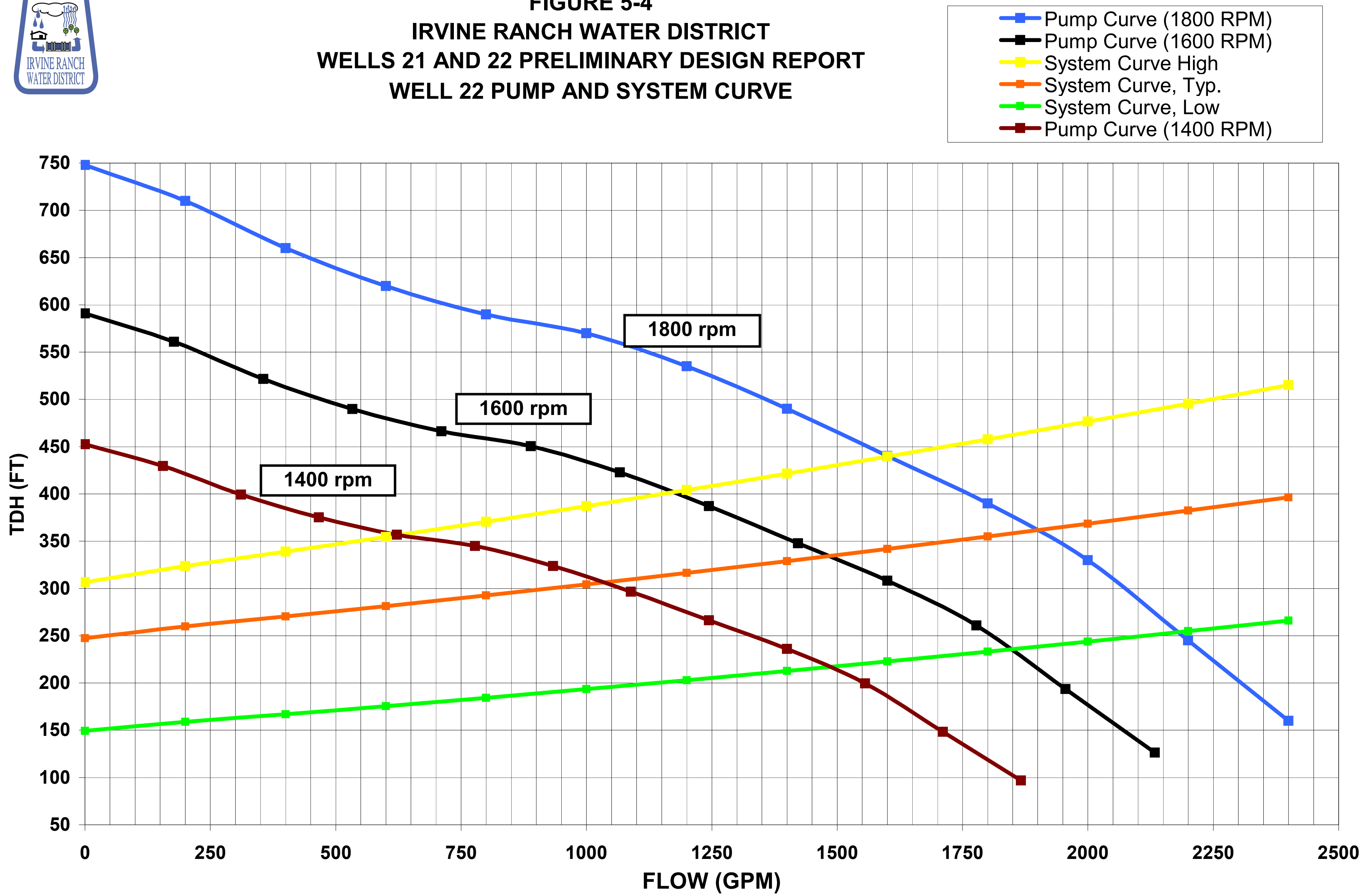


**FIGURE 5-4**
**IRVINE RANCH WATER DISTRICT**
**WELLS 21 AND 22 PRELIMINARY DESIGN REPORT**
**WELL 22 PUMP AND SYSTEM CURVE**



The hydraulic calculations are included in Appendix M of this PDR. Table 5-2 shows the recommended hydraulic design for the well pump systems.

**Table 5-2 Well Pump Information**

| | |
|---|---|
| **Well 21** | |
| Pump/Motor Type | Submersible |
| Rated Flow, gpm | 3300 |
| Maximum Operating TDH @, ft | 340 |
| Minimum Efficiency at Rated Condition | 82% |
| Pump Speed, rpm | 1800 |
| Motor Horsepower | 400 |
| Column Pipe Size, in | 12 |
| Discharge Pipe Size, in | 18 |
| Waste Line Pipe Size, in | 12 |
| **Well 22** | |
| Pump/Motor Type | Submersible |
| Rated Flow, gpm | 1600 |
| Rated TDH, ft | 440 |
| Minimum Efficiency at Rated Condition | 82% |
| Pump Speed, rpm | 1800 |
| Motor Horsepower | 250 |
| Column Pipe Size, in | 10 |
| Discharge Pipe Size, in | 12 |
| Waste Line Pipe Size, in | 10 |

### 5.2.3 Mechanical Equipment Design

The design criteria and hydraulic design parameters were combined with IRWD standards to determine the size, type, and candidate manufacturers for the mechanical equipment associated with the project. Mechanical equipment includes the pump/motors, control/isolation valves, flow meters, piping, backflow prevention, miscellaneous appurtenances, and provisions for surge control.

#### 5.2.3.1 Pump and Motor

A submersible vertical turbine pump and electric motor are proposed for the two wells to attenuate the noise impact to nearby residents. Manufacturers such as Goulds and Flowserve, among others, offer pumps suitable for this project’s conditions. Submersible motors manufacturers that are suitable for these conditions include Hitachi, Plueger (Water Filled), or Byron Jackson (oil filled with double mechanical seal). Table 5-3 shows the proposed pump construction and materials. Certain manufacturers may require an exception per the ARRA funding "American Made" requirements discussed in Section 6.3.

**Table 5-3 Well Pump Construction**

| Component | Material of Construction |
|---|---|
| Bowls | Cast Iron (Lined/Coated) |
| Impellers | Bronze |
| Wear Rings | 316L Stainless Steel |
| Bearings | Rubber/Bronze |
| Discharge Column | ASTM A-120 Steel (Epoxy Lined/Coated) |
| Pump Intake Screen | Stainless Steel |
| Pump Shaft | 17-4PH Stainless Steel |
| Motor | 316 Stainless Steel, Bronze Fitted |

Materials are standard for IRWD submersible well pumps. The column pipe will be epoxy lined and coated steel for added corrosion protection. Vibration detectors, as manufactured by Dytran (Model 3063B), will be provided.

The advantage of the submersible motor is the elimination of the noise associated with an above ground motor utilized for a lineshaft-type vertical turbine well pump. The submersible motor will also save the cost of constructing a sound attenuated well building in a tight area next to a residence.

The disadvantages of the submersible motors are as follows:

- Difficult maintenance as the column pipe, pump, and motor must be pulled from the deep setting for all maintenance.

- The submersible motor cable and cable guard must have room to pass by the outside diameter (OD) of the pump bowls. Limited space in the existing casing creates difficult pump and motor installation.

- Large flow and head requirements for Well 21 necessitate the use of medium voltage to keep motor cable sizes and cost at a reasonable level. This will increase electrical equipment cost.

- The submersible pump and motor in Well 21 will be unable to fit in the existing casing below 278 feet bgs, where the casing is reduced from 20-inch to 16-inch inside diameter. This complies with the hydrogeologist recommended setting of 270 feet bgs; however, with the submersible pump and motor, properly sized, there is no space available to extend the pump and motor further down the well.

- Submersible motors have lower efficiencies (~88-89 percent) than above ground verticals (~92-94 percent) increasing energy cost of pumping.

- The VFD in conjunction with the long cable runs associated with the submersible motor will benefit from the use of a sine wave filter at Well 22, adding to the project costs.

5.2.3.2 Check Valve

The discharge check valve prevents reverse flow to the well discharge head. The check valve recommended is a globe style silent type check as manufactured by Apco. The globe style silent check valve is fast closing, minimizing slam and water hammer. The globe style has a larger flow-through area than the wafer type reducing head loss. These are standard at many of IRWD's existing wells.

5.2.3.3 Isolation Valves

Butterfly valves are typically most cost effective for sizes 12-inches diameter and larger. Butterfly valves also require far fewer turns than gate valves in the larger sizes. Per IRWD standard specifications, candidate manufacturers include Pratt, Kennedy and DeZurik. Butterfly valves are recommended for use as isolation valves 12 inches and larger, gate valves for standard isolation in sections less than 12 inch diameter. The valves will not be used for throttling service.

5.2.3.4 Bypass Valve and Pump Discharge Valve

Motor operated butterfly valves will be used for control of pumped flow from Wells 21 and 22. The bypass valve shall work in conjunction with the pump discharge valve and will initially be open upon start up. After a pre-set and adjustable pump to waste period, the bypass valve will close, while the pump discharge opens simultaneously initiating discharge to the treatment plant.

5.2.3.5 Backflow Prevention

Backflow prevention on the bypass (pump to waste) lines will be provided by a double check valve located upstream of the storm drain connection. A manhole will be constructed with a Tideflex type rubber check valve on the outlet of the pump to waste pipeline. A gravity line from this manhole will be connected to the adjacent storm drain. This system will ensure any storm water backup in the main storm drain cannot backflow to the well discharge line.

5.2.3.6 Pressure Relief Valve/Surge Protection System

A detailed surge analysis was not included in this phase of the project and will need to be conducted during final design. The assumption is the wells will have minor protection from surge and water hammer by using an anti-surge air release and vacuum valve at the discharge head. A check valve should be installed at the pump discharge and at 200 foot intervals in the column pipe to minimize the effects of water hammer due to flow reversal in the column pipe.

Surge protection must also be provided in the event of a power failure. A Cla-Val, Model 52-G-03 pressure relief/surge anticipator valve has been shown on the drawings with discharge to the storm drain via the pump to waste line; however, a surge tank is typically the best mitigation to surge from a power outage. Space has been allocated as shown on the preliminary site plans and costs are included for a surge tank assembly.

5.2.3.7 Flow Meter

Department of Water Resources well design standards require a flow meter on the discharge of each well. Previous IRWD well sites have used the V-cone type flow meter or other type to monitor flow. Current technology favors the magnetic type meter for this application, due to the significantly better accuracy and larger flow range, for the same or reduced cost of the flow meter. Table 5-4 summarizes design criteria for the magnetic flow meter.

**Table 5-4 Flow Meter Design Criteria**

| Parameter | Recommendation |
|---|---|
| Meter Type | Magnetic |
| **Well 21** | |
| Meter Size, inches | 18 |
| Velocity at Rated Flow, fps | 4.2 |
| **Well 22** | |
| Meter Size, inches | 12 |
| Velocity at Rated Flow, fps | 4.5 |

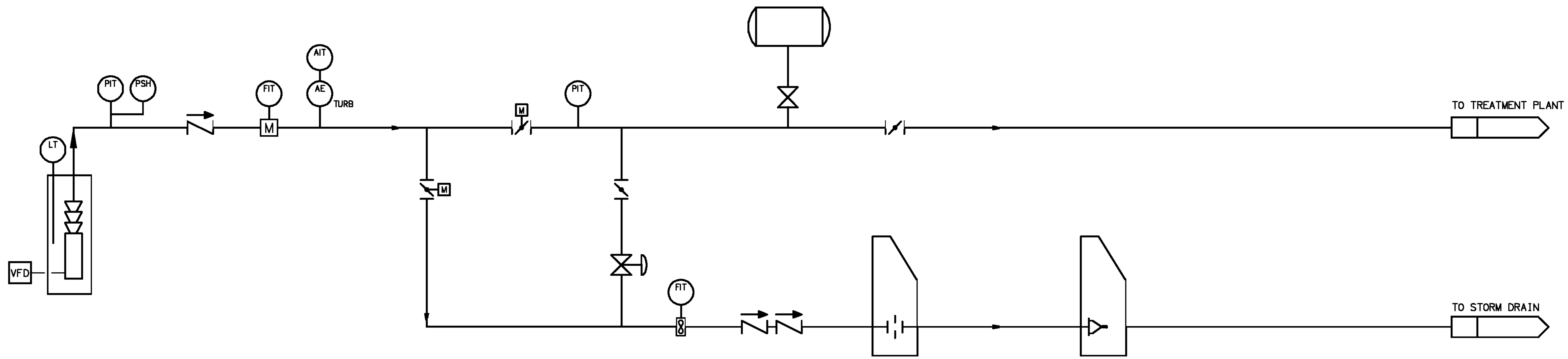
### 5.2.4 Process Flow Diagram

A preliminary flow diagram has been developed for the wells depicting the configuration of mechanical components described herein and based on typical IRWD wellhead equipping standards. The diagram is shown on Figure 5-5.

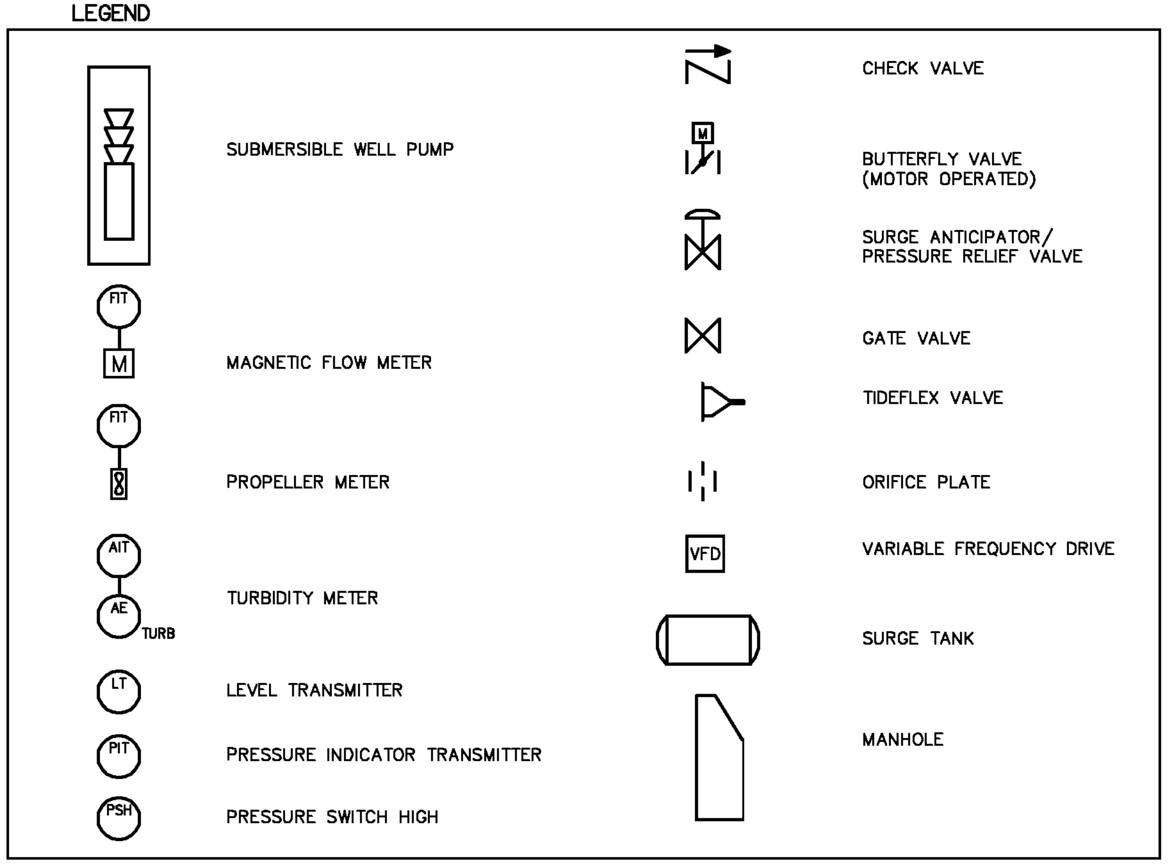
### 5.2.5 Pump to Waste Storm Drain

The storm drain in Mitchell Avenue will be the main pump to waste discharge point. The existing storm drain is planned for replacement by the City of Tustin. Contact has been made with the City indicating our preference to connect to this storm drain with the estimated well flow rates. Further coordination will be required during Wells 21 and 22 final design.

This page left blank intentionally




LEGEND




RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

FLOW DIAGRAM
WELLS 21 / 22

FIGURE
5-5

This page left intentionally blank.

### 5.2.6 Electrical and Instrumentation

#### 5.2.6.1 Well 21

SCE will provide a 4,160 volt, 3-phase electrical service to accommodate the loads of the new well. The SCE transformer pad and metered switchboard are located as shown on Figure 5-1. The metal-clad metered switchgear will be equipped with a main circuit breaker, feeder circuit breaker for the variable frequency drive, and feeder circuit breaker for the 4,160 volt – 208/120 volt, 3-phase transformer.

The VFD, 4160 – 208/120 volt, 3-phase transformer, 120/208 volt distribution panel, and control panel are housed in the electrical building. The 120/208 volt distribution panel serves the motor operated valves and auxiliary loads including air conditioning, lighting and receptacles. No backup power will be provided due to the portable generator requirements associated with the medium voltage power.

The control panel houses the PLC, operator interface terminal, radio, uninterruptible power system (UPS), and associated control components. Radio communication is used to integrate the well with the treatment facility. The antenna is mounted on the roof of the electrical building at a height specified by IRWD.

The major electrical equipment will be specified as indicated Table 5-5:

**Table 5-5 Electrical Equipment Summary**

| Equipment | Manufacturer/Model Number | Sole Source (Yes/No) |
|---|---|---|
| 5 kV Metered Switchgear, Metal Clad | Cutler Hammer, Square D, or equal | No |
| Transformer, Dry Type | Square D electromagnetically shielded type, or equal | No |
| Variable Frequency Drive | Allen Bradley Power Flex 7000 Series, no equal | Yes |
| PLC | Modicon Quantum 140 PCU 434 12U, no equal | Yes |
| Operator Interface Terminal | Maple System HMI 550H-00SE | Yes |
| Radio | MDS iNET 900 Series | Yes |
| UPS | Best Ferrups | Yes |
| Antenna | SCALA TY-900 | Yes |

5.2.6.2 Well 22

SCE will provide a 480 volt, 3-phase electrical service to accommodate the loads of the new well. The SCE transformer pad and metered switchboard are located as shown on Figure 5-2.

The motor control center, variable speed drive and control panel are housed in the electrical building. The motor control center includes a surge protector, power monitor, manual transfer switch, main circuit breaker, feeder circuit breaker for the VFD, feeder circuit breakers for the motor operated valves, 30 kVA, 480-120/208 volt, 3-phase transformer and 120/208 volt distribution panel to accommodate auxiliary loads including lighting and air conditioning.

Emergency power is provided to the facility using a portable emergency generator. The portable generator is connected to the electrical system using the manual transfer switch and generator plug that will be provided as shown previously on Figure 5-2.

The control panel houses the PLC, operator interface terminal, radio, uninterruptible power system (UPS), and associated control components. Radio communication is used to integrate the well with the treatment facility. The antenna is mounted on the roof of the electrical building at a height specified by IRWD.

The major electrical equipment will be specified as indicated Table 5-6:

**Table 5-6 Electrical Equipment Summary**

| Equipment | Manufacturer/Model Number | Sole Source (Yes/No) |
|---|---|---|
| Motor Control Center | Allen-Bradley, no equal | Yes |
| Motor Management Relays | Multilin 369, Model HIRMOE, no equal | Yes |
| Manual Transfer Switch | ASCO, Zenith, or equal | No |
| Switchboards | GE, Square D, or equal | No |
| Power Monitor | Multilin PQM-T20-C-A, no equal | Yes |
| Transformer, Dry Type | Square D electromagnetically shielded type, or equal | No |
| Variable Frequency Drive | Allen Bradley Power Flex 700 Series 18-pulse, no equal | Yes |
| PLC | Modicon Quantum 140 PCU 434 12U, no equal | Yes |
| Operator Interface Terminal | Maple System HMI 550H-00SE | Yes |
| Radio | MDS iNET 900 Series | Yes |
| UPS | Best Ferrups | Yes |
| Antenna | SCALA TY-900 | Yes |

#### 5.2.6.3 Wells 21 and 22 Pump Control

Well pump activation and deactivation will be controlled locally at the individual wells sites or remotely from central SCADA at the treatment plant. Typically the wells would be called to run from the treatment plant. Discharge pressure and flow will be indicated locally, and input to the telemetry system.

High discharge pressure, high motor temperature and low pumping water levels will deactivate the well pump and activate a failure alarm through the telemetry system. Loss of electrical power will also be monitored and activate a failure alarm through the telemetry system.

Control for each well will be through the main (treatment plant) and local (well sites) PLC with an operator interface. The well pumps will vary speed to maintain pressure at the head of the treatment plant, while not exceeding the well discharge capacities. Control functions will include the following at a minimum:

- Well pump start/stop set points
- Opening and closing of control valves
- Pump/motor speed/frequency

PLC indicators will include, at a minimum:

- Flow
- Pump/motor speed/frequency
- Discharge pressure
- Well pump hours
- Well level
- Power failure alarm
- Motor operated valve positions
- Pump Running
- Pump Fail

## 5.3 Pipelines

### 5.3.1 Untreated Water Conveyance Pipeline

An untreated water conveyance pipeline ranging in size from 16-inch to 24-inch diameter will be constructed to deliver 4900 gpm of untreated water from the well sites to the proposed treatment plant. A summary of the criteria utilized for preliminary design of the untreated water conveyance pipeline is given below in Table 5-7. Criteria are based on IRWD standard drawings and specifications and typical industry standards.

**Table 5-7 Untreated Water Conveyance Pipeline Design Criteria Summary**

| Parameter | Recommendation |
|---|---|
| Well 21 Flow, gpm | 3300 |
| Well 22 Flow, gpm | 1600 |
| Total Flow, gpm | 4900 |
| Max Pipeline Velocity, fps | 5 |
| Minimum Velocity, fps | 1.0 |
| Pipe Material | PVC w/DIP Fittings |
| Pipe Joints | Push-On (PVC)/MJ Restraints |
| Minimum Cover, inches | 42 |
| Minimum Pressure Rating, psi | 200* |
| Air Release and Blowoffs | As Required |
| Lining (DIP Fittings) | Cement Mortar |
| Coating (DIP Fittings) | Bitumen/ Epoxy w/polyethylene bag |

* 200 psi rating required to meet sewer separation requirements between 4 to10 feet clear

#### 5.3.1.1 Pipeline Materials

Materials of construction were evaluated for the untreated water conveyance pipeline. Steel, ductile iron pipe (DIP), high density polyethylene (HDPE), and polyvinyl chloride (PVC) were considered. Below is a brief description of the various pipe materials, their advantages, and disadvantages in this application.

##### 5.3.1.1.1 Steel Pipe

IRWD has historically utilized cement mortar lined and coated (CML&C) steel pipe for domestic water lines with diameters greater than 12-inch. Steel has the advantage of a robust design that eliminates the need for expensive thrust restraint with welded joints. Steel pipe has a proven history of retaining its strength over long periods under repetitive surge conditions.

Corrosion resistance is provided by the CML&C. As long as the lining and coating remains intact, corrosion issues are minimal. Corrosion is often seen at the pipe joints that are field lined and coated having the propensity to deteriorate over time.

Current steel pipe pricing was found to be the highest of the alternative pipeline materials discussed herein.

5.3.1.1.2 PVC Pipe

PVC has become a viable alternative for larger untreated and domestic lines and is advantageous due to the light weight, superior corrosion resistance, and, depending on market conditions, pipe material cost savings.

PVC has superior corrosion resistance to steel and DIP. DIP fittings along the PVC line must be lined and coated.

PVC pricing, at the time of this study, appears to be the most cost effective approach, although pricing can be highly volatile with fluctuations in oil prices and general market conditions. PVC pipe requires thrust blocks or mechanical restraints at bends and along straight pipe lengths upstream and downstream as needed. A PVC to steel transition, via a transition coupling, will be required for jack and bore sections.

5.3.1.1.3 High Density Polyethylene (HDPE)

HDPE exhibits similar corrosion resistance to PVC and has become popular for use as intake lines for membrane plants, in particular, those with untreated water that exhibits extreme corrosion potential, such as seawater. HDPE is flexible, making it ideal for trenchless construction methods.

HDPE pipe is joined by the butt fusion method and therefore requires no additional thrust restraint with HDPE fittings.

HDPE has less strength and stiffness than steel, DIP, and PVC making the pipe bedding and overall installation critical to the long term stability of the pipeline. HDPE typically requires a larger wall thickness and, therefore, a larger size nominal pipe diameter to obtain a similar inside diameter as PVC, steel, and DIP.

HDPE pipe material, at this time, is more expensive than PVC, and slightly less but comparable to steel.

5.3.1.1.4 Ductile Iron Pipe

DIP has the advantage of a robust design that can withstand repetitive surge events, however, thrust restraint is required, similar to PVC, adding expense to the pipeline costs without the superior corrosion resistance and ease of handling associated with PVC.

Corrosion resistance is provided by the CML and bitumen coating and would be required to be wrapped with a polyethylene bag for exterior corrosion resistance. Installation of the polyethylene wrap is critical for corrosion resistance. DIP, is estimated to be similar in cost to steel, more expensive to construct than PVC at this time.

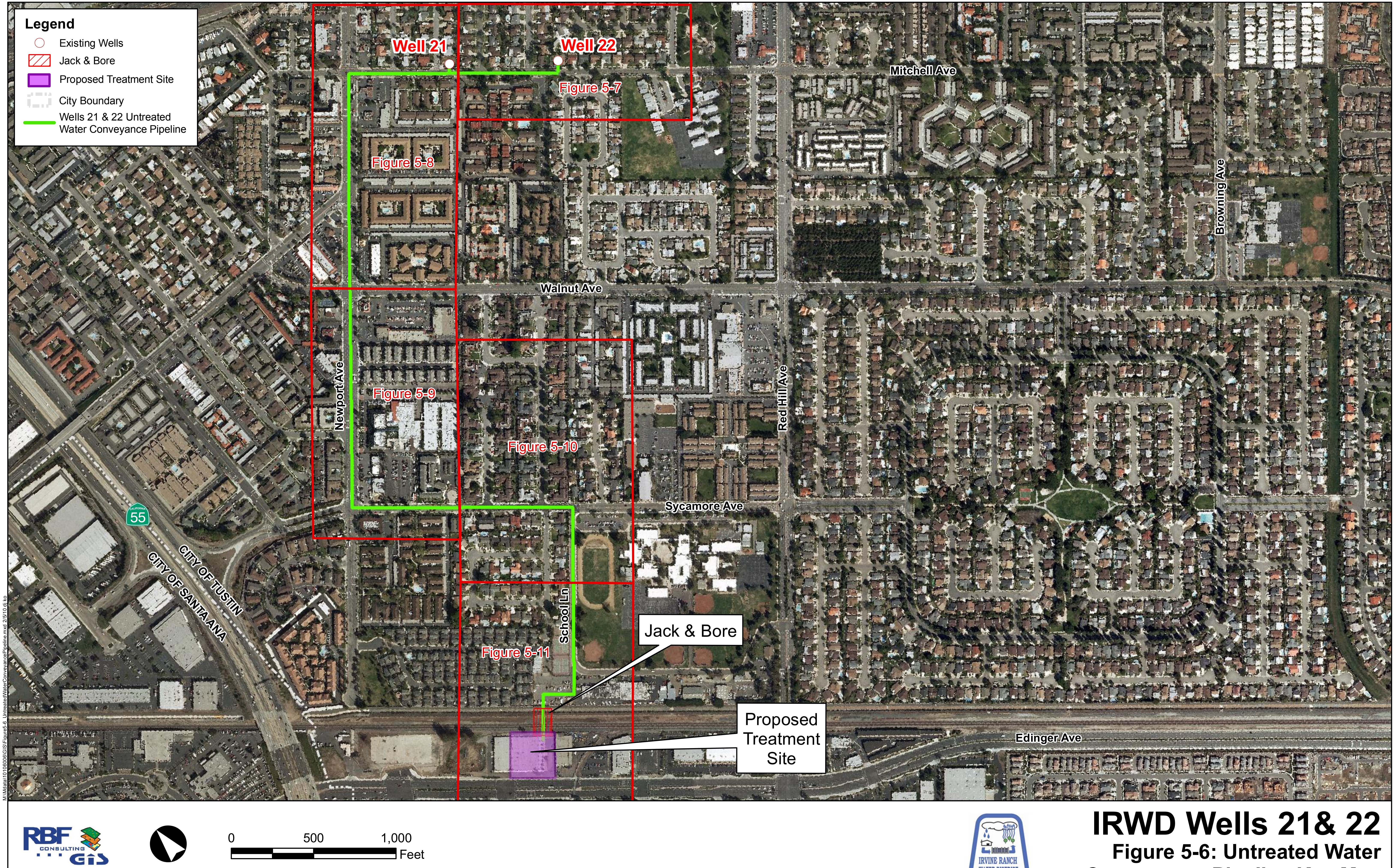
PVC with ductile iron fittings is recommended for the untreated water pipeline. The primary advantages to PVC for this pipeline include:

- Increased corrosion resistance, inside and out, compared to metal pipe.
- Reduced material cost compared to steel.
- Light weight for reduced transportation costs and ease of handling.
- No potential for liner separation
- Higher strength and stiffness compared to HDPE.

- Repetitive surge events less likely in untreated water conveyance line making the steel and DIP strength and rigidity advantage less of a factor.

5.3.1.2 Preliminary Horizontal Alignment

The utilities and right-of-way information gathered were superimposed on an aerial photo background. This served as the basis for placing the pipeline more accurately in its anticipated horizontal alignment. This proposed untreated water conveyance pipeline size and alignment is shown on Figures 5-6 through 5-11. A cross section depicting major utilities and dimensions along each of the major right-of-ways is included.




Source: Eagle Aerial, 2009

IRVINE RANCH WATER DISTRICT

**IRWD Wells 21& 22**

**Figure 5-6: Untreated Water Conveyance Pipeline Key Map**

This page left intentionally blank.


RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

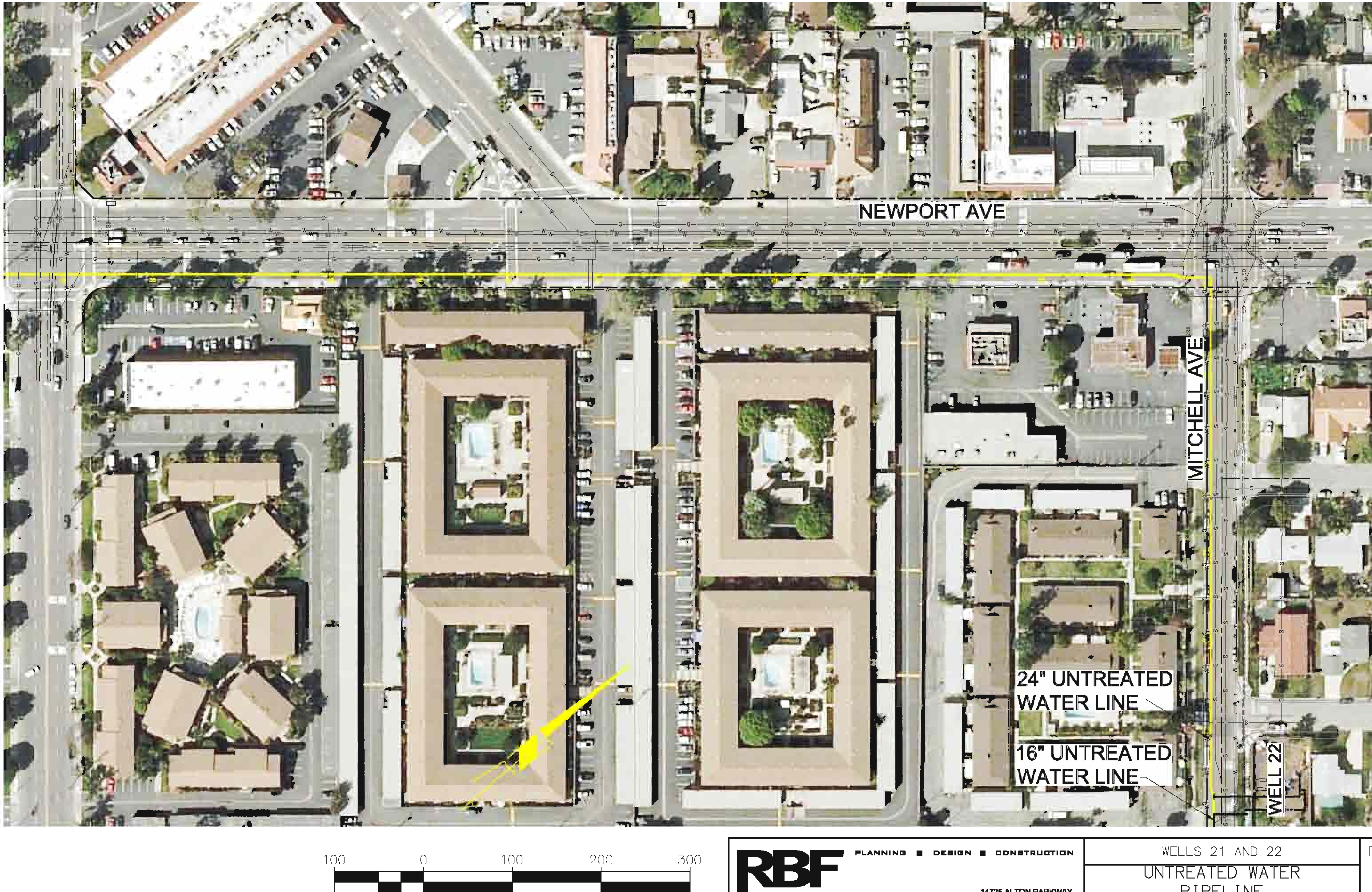
WELLS 21 AND 22

UNTREATED WATER PIPELINE MITCHELL AVENUE

FIGURE 5-7

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-7.DWG DDT 3/12/10 11:53 am

This page left intentionally blank.




RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

UNTREATED WATER PIPELINE
NEWPORT AVE./ MITCHELL AVE

FIGURE 5-8

This page left intentionally blank.


RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

UNTREATED WATER PIPELINE
SYCAMORE AVE./NEWPORT AVE.

FIGURE 5-9

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-9.DWG DDT 3/12/10 12:06 pm

This page left intentionally blank.


100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

UNTREATED WATER PIPELINE
SCHOOL LN./ SYCAMORE AVE.

FIGURE

5-10

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-10.DWG DDT 3/12/10 12:09 pm

This page left intentionally blank.


100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

UNTREATED WATER PIPELINE SCHOOL LANE

FIGURE

5-11

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-11.DWG DDT 3/12/10 12:12 pm

This page left intentionally blank.

### 5.3.2 Product Water Pipeline

A 24-inch diameter product water pipeline, with a 20-inch diameter section where crossing Peters Canyon Channel, will be constructed to deliver water from the treatment plant product water pump station to the proposed IRWD Zone 1 tie-in point. A summary of the criteria utilized for the product water pipeline preliminary design is given below:

**Table 5-8 Product Water Pipeline Design Criteria Summary**

| Parameter | Recommendation |
|---|---|
| Treated Water Flow, gpm | 4,360 |
| Maximum Velocity, fps | 5.0 |
| Minimum Velocity, fps | 1.0 |
| Pipe Material | Steel |
| Pipe Joints | Bell and Spigot – Lap Welded |
| Minimum Cover, inches | 48 |
| Minimum Pressure Rating, psi | 200 |
| Air Release and Blowoffs | As Required |
| Electrolysis Test Station Spacing, ft | 500 |
| Lining | Cement Mortar |
| Coating | Cement Mortar |

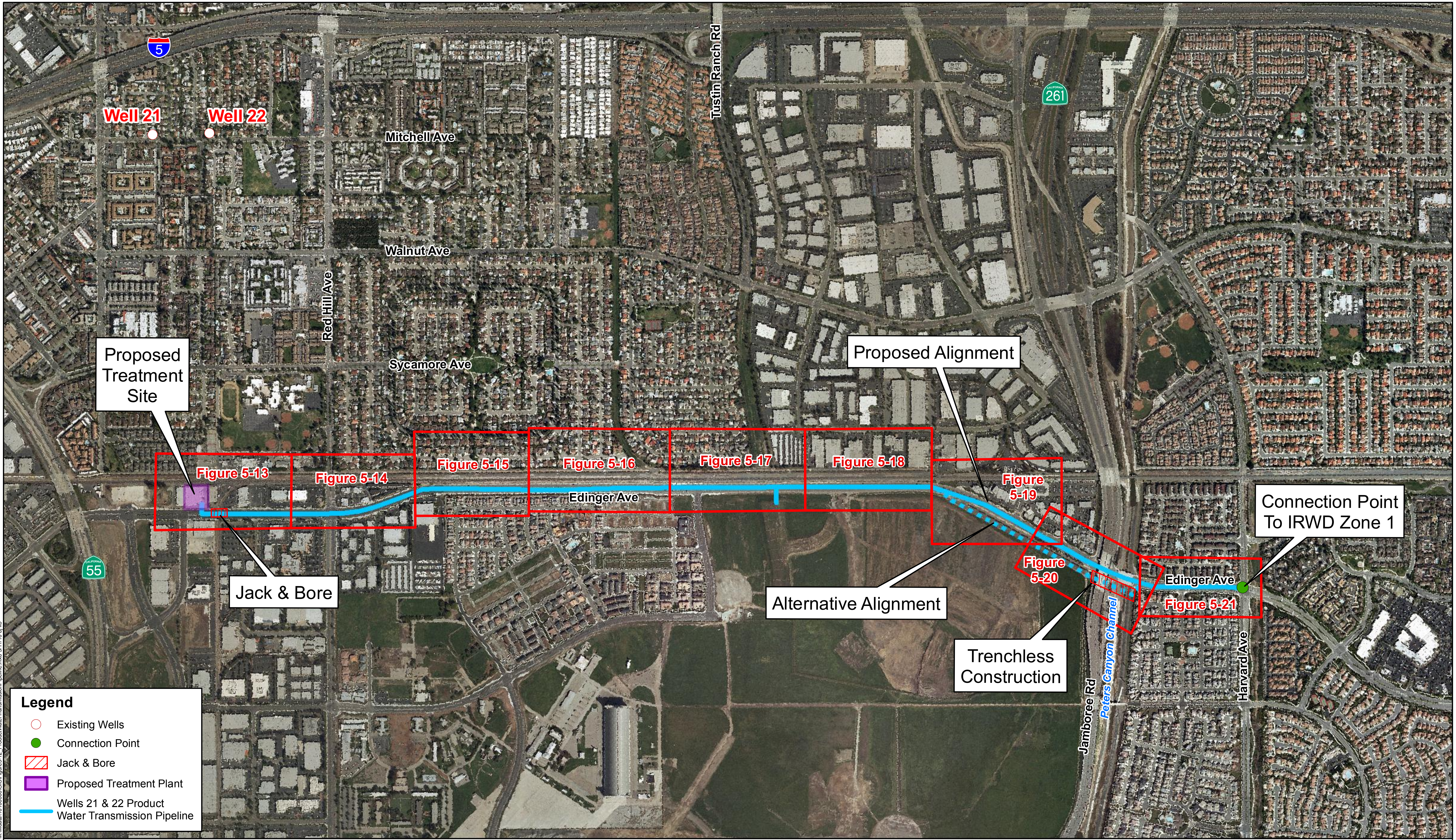
#### 5.3.2.1 Pipeline Materials

IRWD standards call for larger diameter potable water distribution lines to be constructed of cement mortar lined and coated welded steel pipe. This material has a long history of durability and reliability in conveying domestic water in the IRWD system. Welded steel pipe has the proven ability to maintain its strength and rigidity under repetitive surges common to a water distribution network.

#### 5.3.2.2 Preliminary Horizontal Alignment

The utilities and right-of-way information gathered for the recommended product water pipeline alignment were superimposed on an aerial photo background. This served as the basis for placing the pipeline more precisely in its anticipated final horizontal alignment. The proposed pipeline alignment is shown on Figures 5-12 through 5-21. A cross section depicting major utilities and right-of-way dimensions is included.

This page left blank intentionally

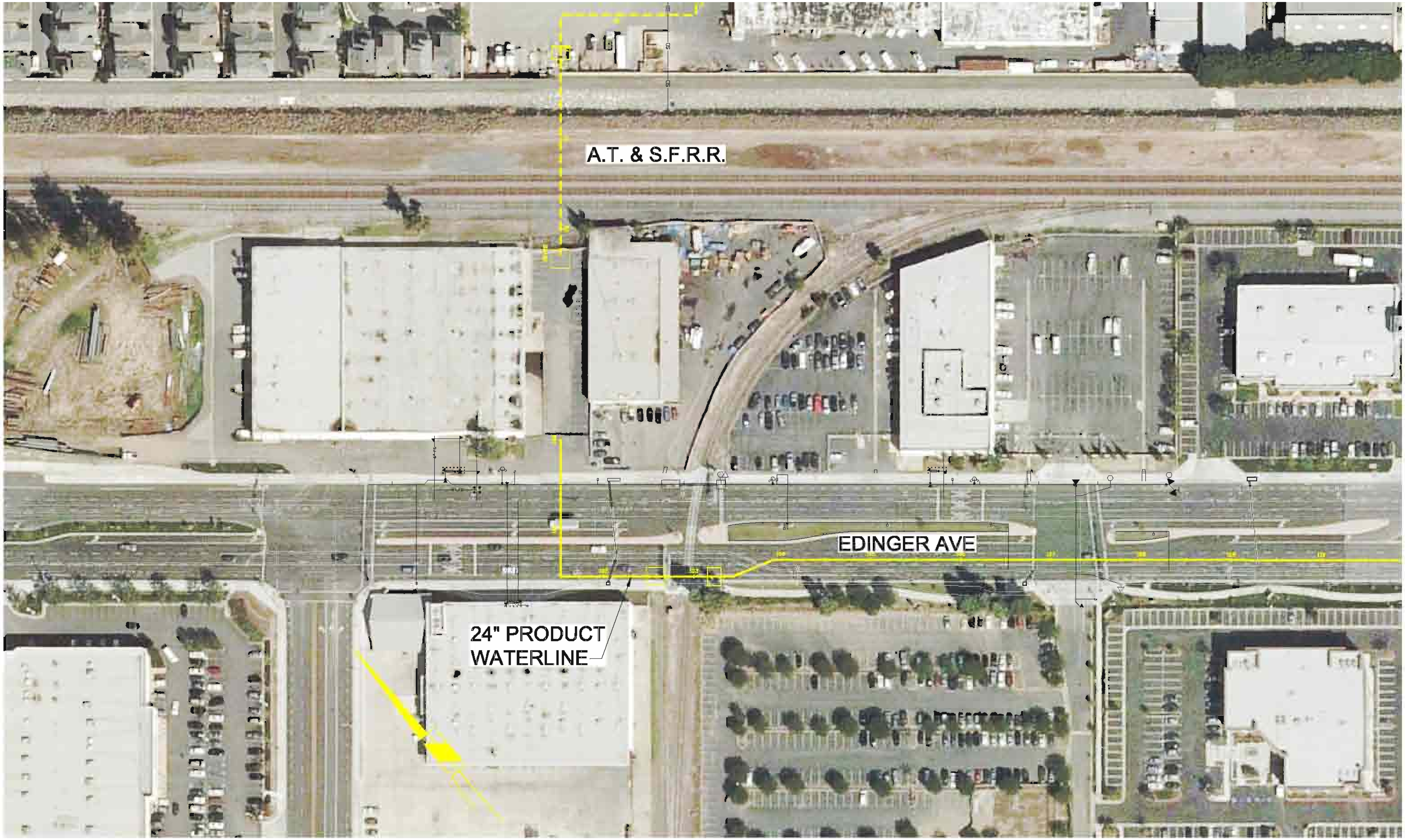



Source: Eagle Aerial, 2009

# IRWD Wells 21& 22

## Figure 5-12: Product Water Transmission Pipeline Key Map

This page left intentionally blank.




100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-13

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-13.DWG DDT 3/12/10 12:16 pm

This page left intentionally blank.


100 0 100 200 300

SCALE: 1"=100'

**RBF CONSULTING**

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

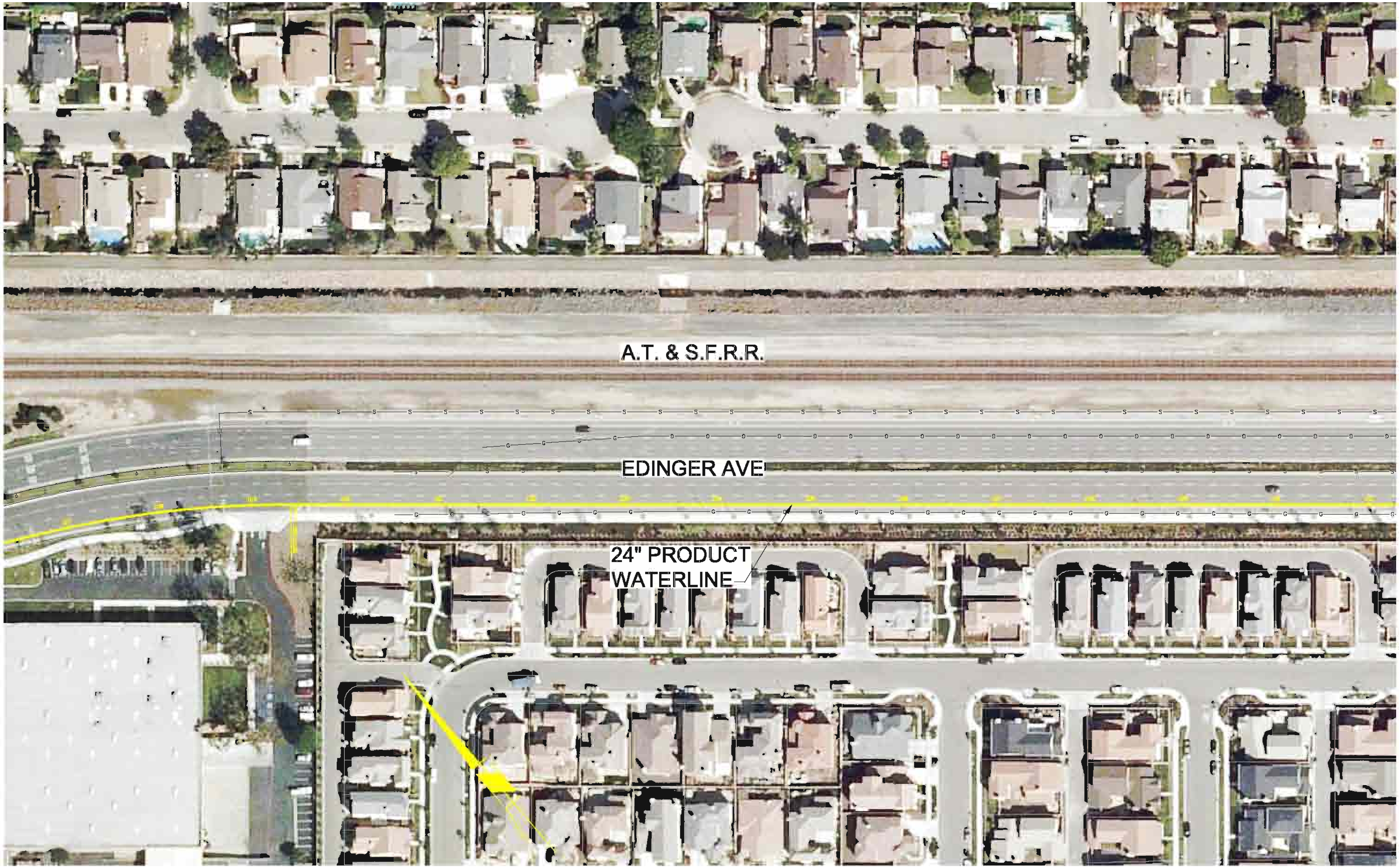
PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE

5-14

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-14.DWG DDT 3/12/10 11:40 am

This page left intentionally blank.




100 0 100 200 300

SCALE: 1"=100'

PLANNING ■ DESIGN ■ CONSTRUCTION

**RBF**

CONSULTING

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

PRODUCT WATER
PIPELINE
EDINGER AVENUE

FIGURE

5-15

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-15.DWG DDT 3/12/10 12:19 pm

This page left intentionally blank.


RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ■ FAX 949.472.8373 ■ www.RBF.com

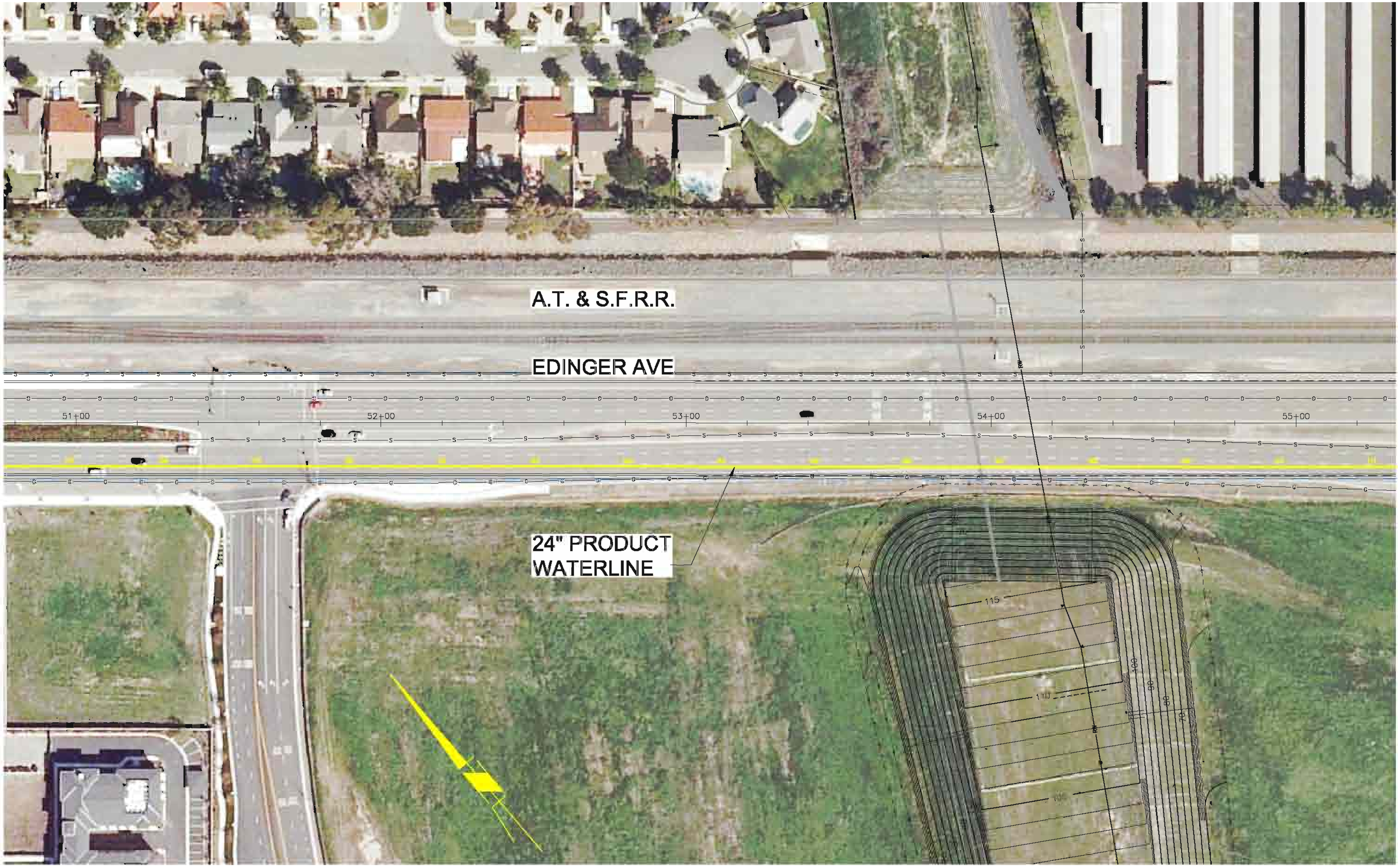
WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-16

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-16.DWG DDT 3/12/10 12:23 pm

This page left intentionally blank.




100 0 100 200 300
SCALE: 1"=100'

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-17

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-17.DWG DDT 3/12/10 12:26 pm

This page left intentionally blank.


100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ■ FAX 949.472.8373 ■ www.RBF.com

WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-18

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-18.DWG DDT 3/12/10 12:53 pm

This page left intentionally blank.


100 0 100 200 300

SCALE: 1"=100'


RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ■ FAX 949.472.8373 ■ www.RBF.com

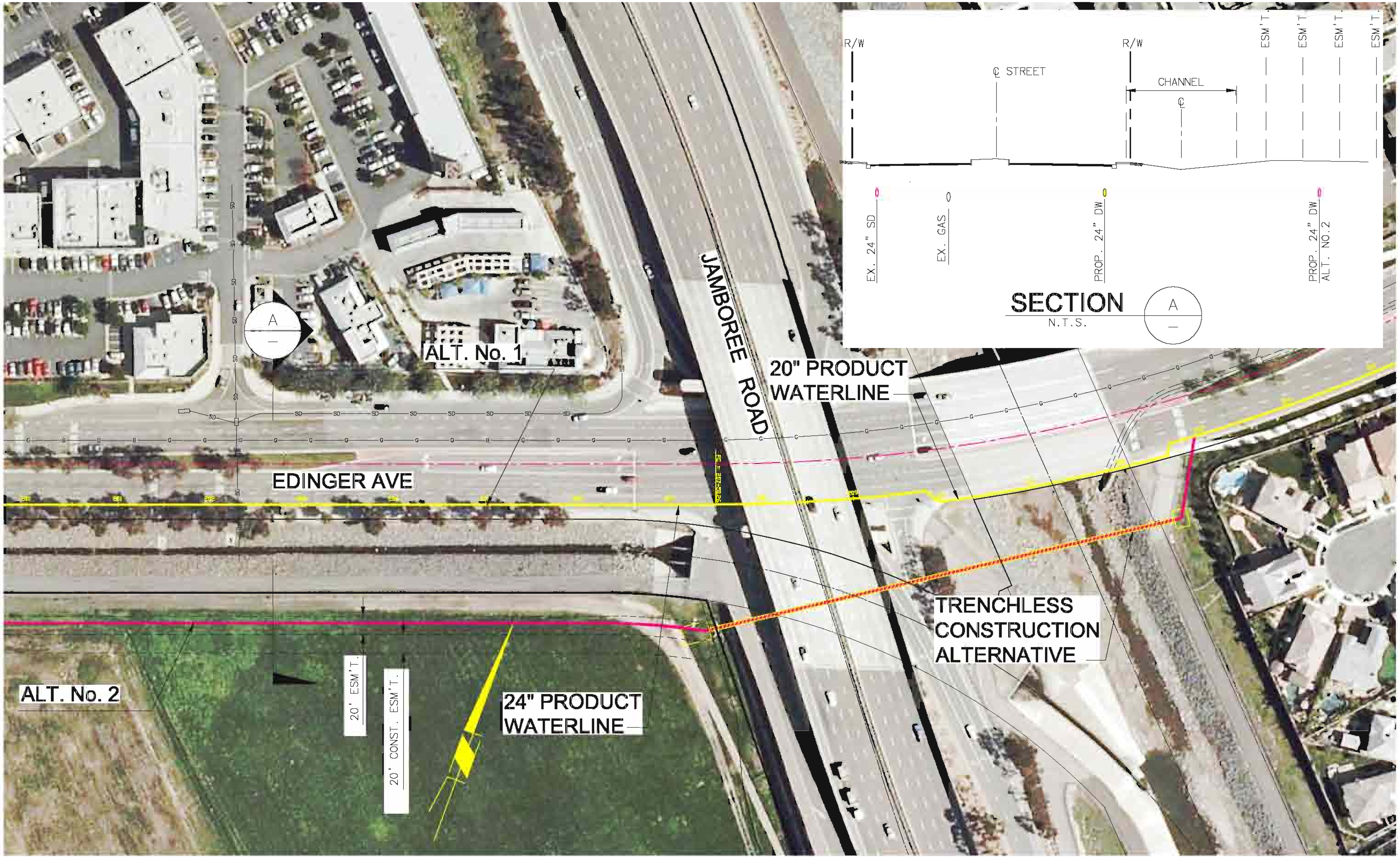

WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-19

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-19.DWG DDT 3/12/10 12:56 pm

This page left intentionally blank.




100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

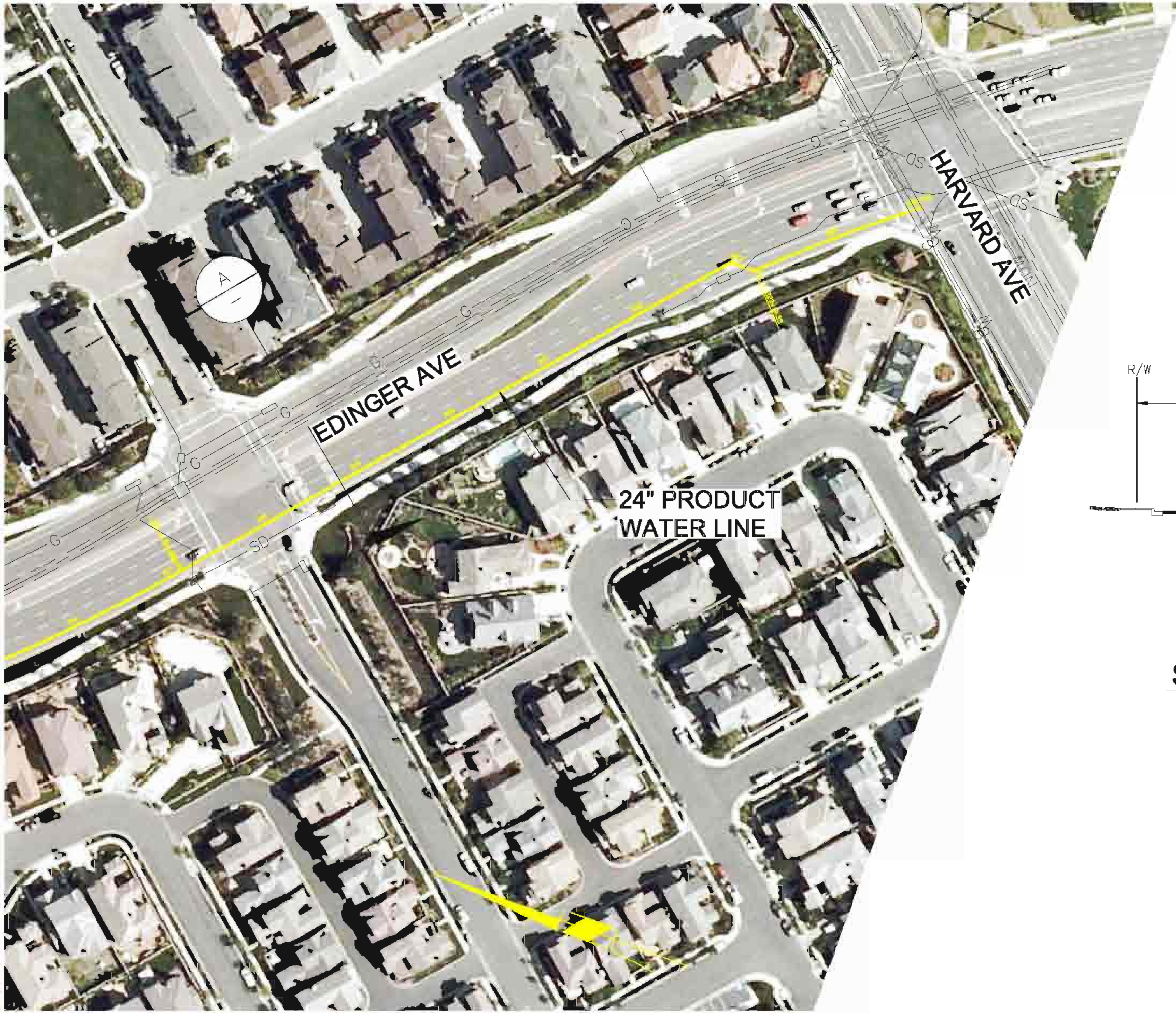
WELLS 21 AND 22

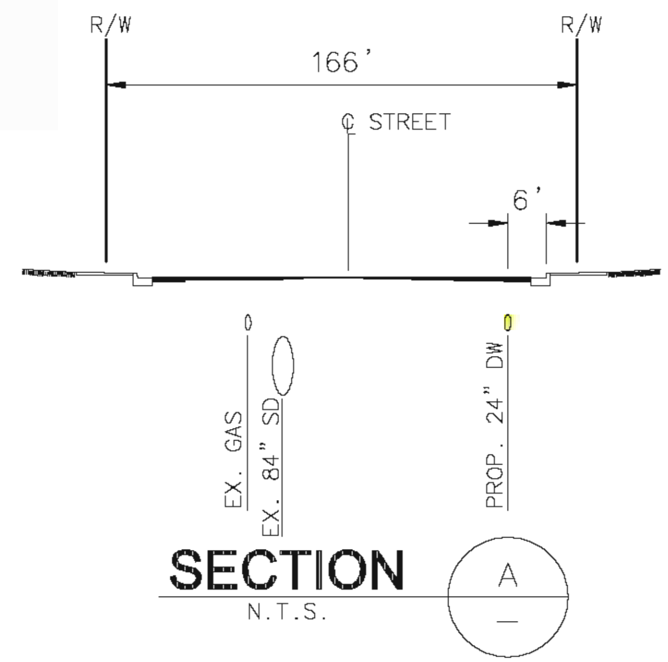
PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-20

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-20.DWG DDT 3/12/10 1:27 pm

This page left intentionally blank.




100 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

PRODUCT WATER PIPELINE EDINGER AVENUE

FIGURE 5-21

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\CONVEYANCE PIPELINES\6006-FIG-5-21.DWG DDT 3/12/10 1:21 pm

This page left intentionally blank.

### 5.3.3 Pipelines Design Details

#### 5.3.3.1 Existing Utilities

Preliminary existing utilities research and right-of-way information was gathered for the recommended alignments. The precise alignment of the pipeline may be affected by detailed analysis of existing utilities required during final design. Major utilities have been identified and were taken into consideration in defining the proposed general alignment of the untreated water conveyance pipeline and more detailed alignment of the product water pipeline. Nevertheless, a more detailed utility search will be undertaken as part of final design in order to locate all known facilities that could potentially hinder pipeline construction work. The proposed pipelines will be installed to comply with Department of Public Health horizontal and vertical separation requirements in conjunction with IRWD standard drawings and specifications.

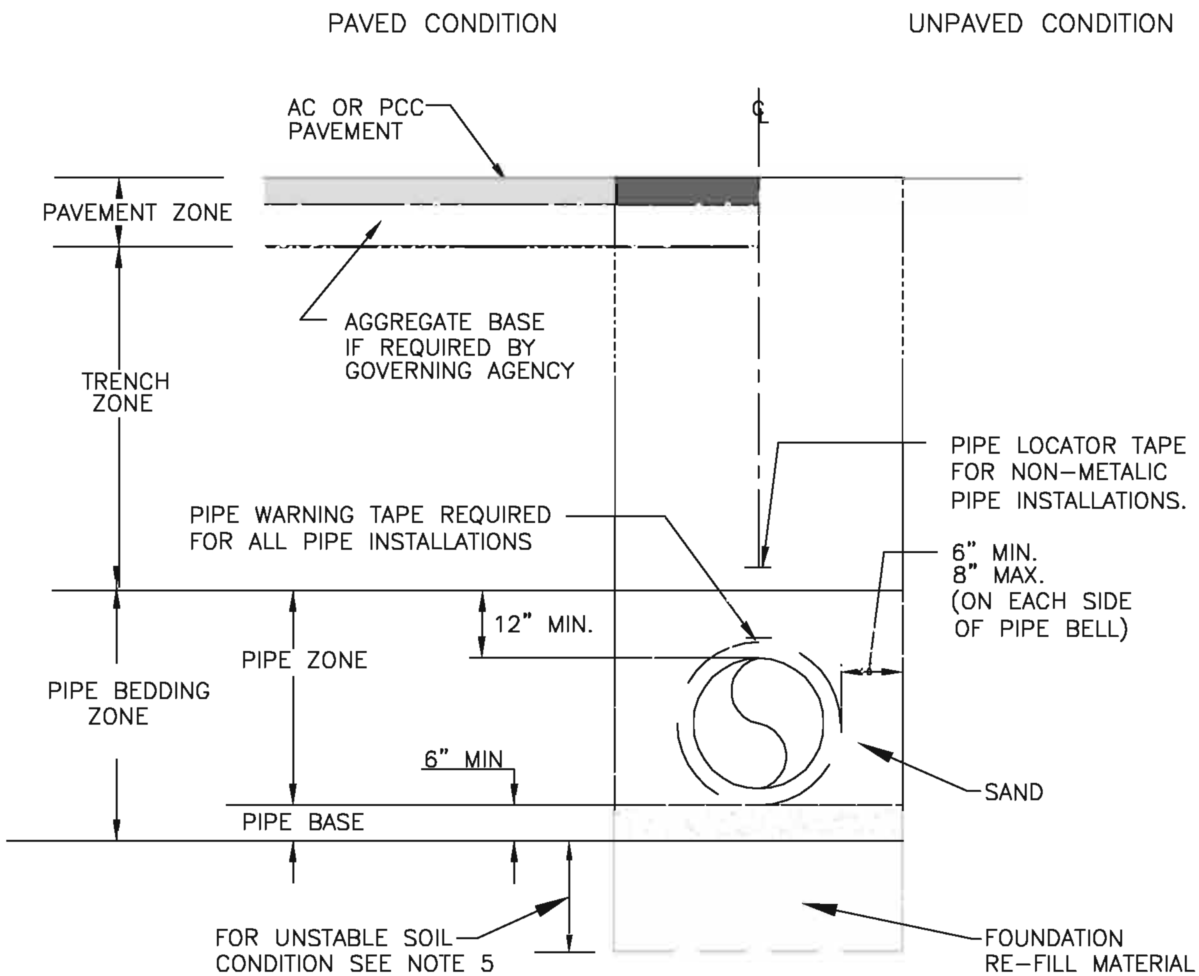
#### 5.3.3.2 Trench Section

IRWD's standard drawings and specifications govern for the typical pipe trench sections. Figure 5-22 shows the typical trench section based on IRWD standards. The standard trench section indicates the following:

- Pipe Bedding Zone—The pipe bedding zone encompasses the area a minimum 6 inches below, 6 to 8 inches on either side, and minimum 12 inches above the pipe. For welded steel pipe, this area will be imported sand per California Standard Specifications for Public Works Construction Section 200-1.5.1 as defined in IRWD's standard specifications.

- Trench Zone—The trench zone is defined as the trenched area above the pipe bedding zone and below the pavement zone (where applicable). The materials can be native or imported, but shall be fine grained, non-organic materials free from peat, roots, debris, and rocks larger than 3 inches that can be compacted to specified relative compaction.

This page left blank intentionally




NOTES:

1. ALL WORK SHALL BE PER IRWD STD. SPEC. SECTION 02223.
2. PIPE BASE SHALL BE A MINIMUM OF 6-INCHES IN DEPTH. PIPE BEDDING MATERIAL SHALL BE PER IRWD STD. SPEC. SECTION 02223-2 AND SHALL INCLUDE BOTH PIPE BASE AND PIPE ZONE.
3. IF THE DEPTH OF THE TRENCH ZONE FOR UNPAVED AREA OR TRENCH ZONE PLUS THE PAVEMENT ZONE FOR PAVED AREA IS UNDER 4' OR OVER 20' A ONE SACK SLURRY WITHIN THE PIPE BEDDING ZONE SHALL BE PLACED UNLESS OTHERWISE APPROVED.
4. CONTRACTOR SHALL PROVIDE HAND EXCAVATED "BELL HOLE" FOR EACH PIPE JOINT SO THAT THE WEIGHT OF PIPE DOES NOT BEAR ON THE BELL. CONTRACTOR SHALL RE-FILL AND HAND-TAMP EACH "BELL HOLE" PRIOR TO COMPLETING THE PLACEMENT OF PIPE BEDDING.
5. IF UNSTABLE SOIL IS ENCOUNTERED, THE DISTRICT REPRESENTATIVE SHALL DETERMINE OVEREXCAVATION DEPTH AND FOUNDATION RE-FILL MATERIAL PER IRWD STD SPEC. SECTION 02223.



RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

IRWD WATER TRENCH DETAIL

FIGURE 5-22

- Pavement Zone—The pavement zone, where pavement removal and replacement is necessary, is the top portion of the trench, above the trench zone to the surface. This section shall consists of an aggregate base course and asphalt paving per the California Standard Specifications for Public Works Construction as defined in IRWD standard specifications Section 02578.

5.3.3.3 Pipe

The following paragraphs contain discussion of various designs, manufacturing, and construction issues related to the untreated and product water pressure pipelines:

5.3.3.3.1 Internal Pressure

The maximum internal pressure loadings will occur at the discharge sides of the well pumping facilities (Wells 21 and 22) and within the product water pipeline due to the higher pressures (>100 psi) in the Zone 1 distribution system. Under normal operating conditions, pressures in the untreated water conveyance pipeline and product water pipeline are not expected to exceed 100 psi and 150 psi, respectively. The PVC untreated water pipeline will have a design pressure rating of 200 psi to conform to the Department of Public Health and IRWD separation standards (to allow sewer separation of 4 to10 feet).

5.3.3.3.2 External Loadings

Both pipeline alignments will be constructed mainly within paved streets. The pipeline will be designed for AASHTO H-20 traffic loading.

The minimum pipeline depth will be 42-inches and 48-inches to top of pipe for PVC and Steel pipe, respectively, per IRWD standard specifications. The pipe depth will vary to avoid other utilities and to cross the railroad tracks and drainage channels.

External loads will be analyzed using site specific soil information. Soil testing will be performed during final design to classify soil and provide parameters to design the wall thickness for external loadings. Minimum wall thicknesses for steel pipe will be 0.25-inch per IRWD standards. A minimum DR-18 will be utilized for the untreated water PVC pipe.

5.3.3.3.3 Thrust Restraint

To adequately restrain the pipe against movement will require welded joints in the steel product water pipeline and mechanically restrained joints for the PVC untreated water pipeline. Thrust blocks for large diameter pipelines are typically too large to construct in public right-of-way. Determining the exact quantity of restrained joints will be done during final design; however, restrained length assumptions along the PVC pipeline have been made to prepare the cost estimates in Section 6.

5.3.3.3.4 Pipe Joints

Welded steel pipe is available with various types of joints. The industry standard and most cost-effective joint type for steel pipe is bell and spigot. Per IRWD standards, buried joints shall be bell and spigot lap welded. PVC joints will be push on type with mechanical restraints as required..

5.3.3.3.5 Corrosion Protection

The engineering design response to the problem of corrosivity will follow from chemical analysis and resistivity measurements of the soils and groundwater over the length of the selected route. A soil corrosivity analysis will be performed during the final design. Electrolysis test stations are planned at 500-foot spacing along the steel product water pipeline reaches. Ductile iron fittings for the PVC untreated water pipeline shall be cement mortar lined with a bitumastic coating and polyethylene bagged.

5.3.3.4 Appurtenances

5.3.3.4.1 Air Release/Vacuum Valves

Valves will be sized and located during final design and after the surge analysis. Valves shall be located adjacent to the pipeline in above grade cans located within the right-of-way.

5.3.3.4.2 Isolation Valves

Isolation valves will be located on the pipeline, at intersections and major crossings. These include the railroad and drainage channel crossings near Edinger Avenue and the bridge crossing at Peters Canyon Channel. Butterfly valves will be used for larger diameter isolation valves.

5.3.3.5 Construction

The means and method of construction will mostly be determined by the contractor. Construction methods will include a combination of open-trench and trenchless construction methods. Trenchless, jack and bore construction, will be required for crossing the railroad and drainage channel along the untreated water conveyance pipeline route just north of Edinger Avenue. This will require a transition to steel material for the carrier pipe. IRWD's standard design for steel casing and carrier water pipe for trenchless construction is shown on Figure 5-23. Open-trenching will require pavement replacement in paved right-of-way.

The Edinger Avenue Bridge crossing has been proposed as a 20-Inch diameter steel pipe that will be hung on the south side of the bridge. The existing bridge is a 2-span cast-in-place box girder structure with a cast-in-place singe box girder widening. The 20" diameter pipe is proposed to span along the widening below the deck overhang. The supports for the pipe will be mounted on the overhang and the exterior web of the box girder. The support frame will be located at 20 feet on centers and will be anchored to the existing concrete with thru-anchors and expansion anchors. All fabricated steel shall be galvanized. A schematic of the proposed pipe support is shown in Figure 5-24.

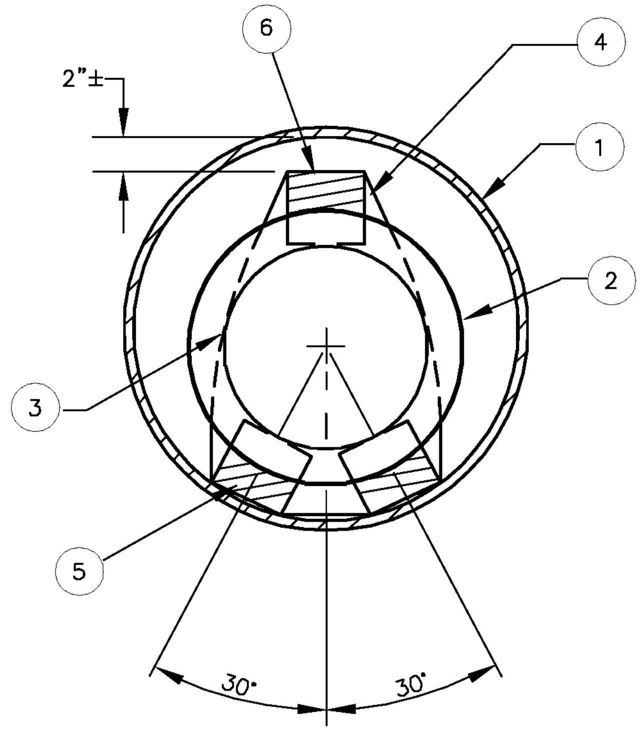
5.3.3.6 Geotechnical

A preliminary geotechnical investigation will be performed prior to preparation of final design documents to identify impediments to the planned alignment of the pipeline. The soil conditions along the proposed pipe alignments are assumed to be sufficient for recompaction as backfill or for subbase. No mitigation for groundwater or liquefaction is anticipated for any of the conveyance facilities.

5.3.3.7 Traffic Control

Traffic control will be required during construction. Traffic control will be in compliance with local agency requirements. Traffic control cost is included as in the pipeline installation cost estimates.

| STEEL CASING SCHEDULE | | |
|---|---|---|
| NOMINAL CARRIER PIPE SIZE | MINIMUM CASING SIZE | MINIMUM WALL THICKNESS |
| 4" | 12" O.D. | 1/4" |
| 6" | 14" O.D. | 1/4" |
| 8" | 16" O.D. | 5/16" |
| 10" | 18" O.D. | 5/16" |
| 12" | 20" O.D. | 3/8" |
| 16" | 24" O.D. | 3/8" |
| 18" | 30" O.D. | 1/2" |
| 24" | 42" O.D. | 1/2" |




| ITEM | MATERIALS |
|---|---|
| 1 | STEEL CASING. |
| 2 | CARRIER PIPE JOINT BELL. |
| 3 | CARRIER PIPE. |
| 4 | STAINLESS STEEL STRAP. |
| 5 | 3' LONG 4"X4" REDWOOD SKIDS FOR EACH LENGTH OF PIPE, BEVELED AT BOTH ENDS, SKIDS TO BE STRAPPED IN PLACE 3' FROM EACH END OF PIPE. NOTCH SKID TO SEAT STRAP. STRAP SHALL BE 316 STAINLESS STEEL. |
| 6 | 3' LONG REDWOOD FLOAT BLOCK WITH 4" MIN. WIDTH. BEVEL AND STRAP FLOAT BLOCK THE SAME AS THE SKIDS. PROVIDE 2"± CLEARANCE FROM TOP OF FLOAT BLOCK TO CASING I.D. |

NOTES:

1. CASING SHALL BE INSTALLED BY THE BORE, JACK AND/OR TUNNEL METHOD.
2. SIZE AND THICKNESS OF CASING SHALL BE AS SHOWN IN SCHEDULE. FOR LONG BORES OR SPECIAL SITUATIONS GREATER WALL THICKNESS THAN SHOWN IN SCHEDULE MAY BE REQUIRED.
3. ALL STEEL CASING PIPE FIELD JOINTS SHALL BE WELDED FULL-CIRCUMFERENCE.
4. CARRIER PIPE SHALL BE PRESSURE TESTED PRIOR TO SEALING ENDS OF CASING.
5. EACH END OF CASING SHALL BE SEALED WITH APPROVED RUBBER CASING END SEALS.
6. BACKFILL FOR CASING IN OPEN CUT SHALL BE PER IRWD STD. DWG. W-11.

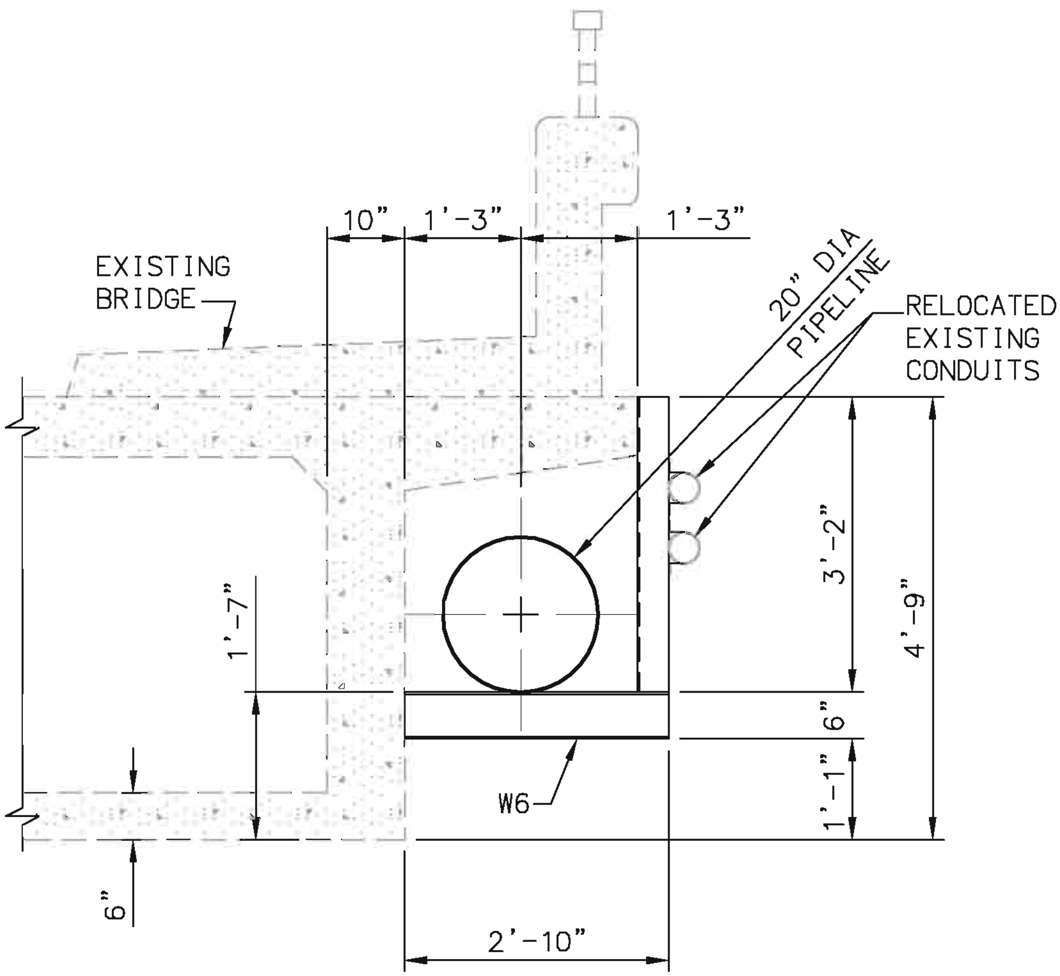


RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

IRWD STEEL CASING FOR WATER PIPE

FIGURE 5-23




**SECTION**
NOT TO SCALE

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\6006-FIG-5-24.DWG JWILEY 3/12/10 3:29 pm

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

EDINGER AVENUE BRIDGE CROSSING
PIPE SUPPORT DETAIL

FIGURE
**5-24**

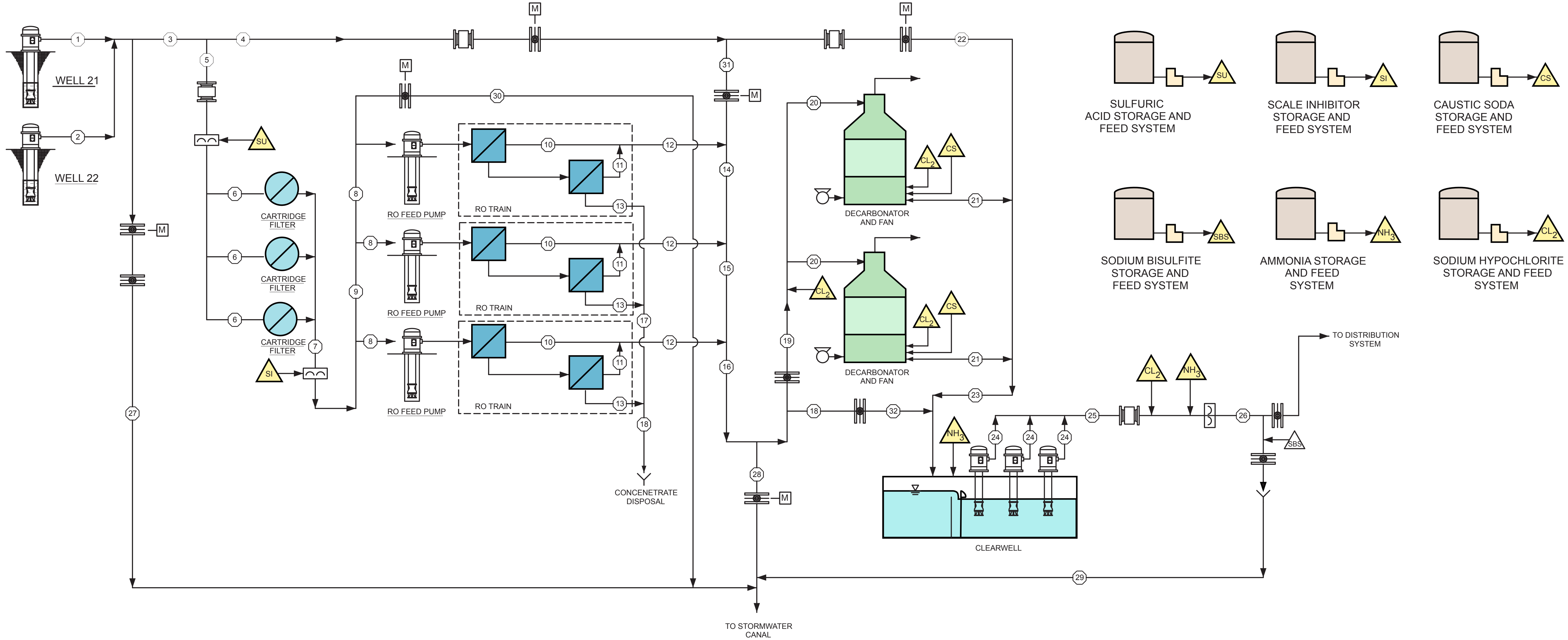
## 5.4 Treatment Facilities

The process flow diagram for the IRWD Wells 21 and 22 Desalter is presented as a single-sheet summary in Figure 5-25. The schematic shows the following major process elements:

- Untreated water supply
  - Two existing wellheads equipped with new submersible pumps
- Reverse osmosis treatment system
  - Cartridge filters to remove residual sand and other particulates
  - Sulfuric acid and scale inhibitor addition for scale control
  - RO membrane feed pumps for boosting the RO feed pressure
  - RO membrane trains for removing dissolved solids, including nitrates
  - Decarbonators for post treatment pH adjustment and stabilization
- Product water facilities:
  - Post treatment pH adjustment (caustic soda) and disinfection (sodium hypochlorite and aqueous ammonia)
  - 0.15 MG clearwell to provide product water storage prior to distribution
  - Product water pumps to transfer treated water to the distribution system

Each of these process and delivery elements is presented herein with detailed design criteria.

This page left blank intentionally




| Parameter | FLOWSTREAM | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 |
| Flowrate (gpm): | 3300 | 1600 | 4900 | 1278 | 3622 | 1207 | 3622 | 1207 | 2415 | 741 | 286 | 1026 | 181 | 1026 | 2053 | 3079 |
| Flowrate (mgd): | 4.75 | 2.30 | 7.06 | 1.84 | 5.22 | 1.74 | 5.22 | 1.74 | 3.48 | 1.07 | 0.41 | 1.48 | 0.26 | 1.48 | 2.96 | 4.43 |
| **Water Quality** | | | | | | | | | | | | | | | | |
| Calcium (mg/L): | 133 | 133 | 133 | 133 | 133 | 133 | 133 | 133 | 133 | 0.49 | 1.83 | 0.87 | 882 | 0.87 | 0.87 | 0.87 |
| Magnesium (mg/L): | 35 | 35 | 35 | 35 | 35 | 35 | 35 | 35 | 35 | 0.13 | 0.50 | 0.23 | 232 | 0.23 | 0.23 | 0.23 |
| Sodium (mg/L): | 69 | 69 | 69 | 69 | 69 | 69 | 69 | 69 | 69 | 1.10 | 4.20 | 1.97 | 449 | 1.97 | 1.97 | 1.97 |
| Chloride (mg/L): | 127 | 127 | 127 | 127 | 127 | 127 | 127 | 127 | 127 | 0.60 | 2.10 | 0.99 | 841 | 0.99 | 0.99 | 0.99 |
| Nitrate (mg/L as NO3): | 80 | 80 | 80 | 80 | 80 | 80 | 80 | 80 | 80 | 2.20 | 7.95 | 3.79 | 512 | 3.79 | 3.79 | 3.79 |
| Alkalinity (mg/L as CaCO3): | 220 | 220 | 220 | 220 | 220 | 187 | 187 | 187 | 187 | 1.61 | 1.61 | 2.35 | 1236 | 2.35 | 2.35 | 2.35 |
| Calcium Hardness (mg/L as CaCO3): | 333 | 333 | 333 | 333 | 333 | 333 | 333 | 333 | 333 | 1.23 | 4.58 | 2.18 | 2204 | 2.18 | 2.18 | 2.18 |
| Magnesium Hardness (mg/L as CaCO3): | 144 | 144 | 144 | 144 | 144 | 144 | 144 | 144 | 144 | 0.53 | 2.06 | 0.95 | 954 | 0.95 | 0.95 | 0.95 |
| Total Hardness (mg/L as CaCO3): | 476 | 476 | 476 | 476 | 476 | 476 | 476 | 476 | 476 | 1.76 | 6.63 | 3.12 | 3158 | 3.12 | 3.12 | 3.12 |
| Total Dissolved Solids (mg/L): | 780 | 780 | 780 | 780 | 780 | 760 | 760 | 760 | 760 | 6.20 | 22.00 | 10.84 | 5008 | 10.84 | 10.84 | 10.84 |
| pH: | 7.4 | 7.4 | 7.4 | 7.4 | 7.4 | 6.9 | 6.9 | 6.9 | 6.9 | 5.0 | 5.4 | 5.2 | 7.5 | 5.2 | 5.2 | 5.2 |
| CCPP (mg/L): | 73.1 | 73.1 | 73.1 | 73.1 | 73.1 | -18.7 | -18.7 | -18.7 | -18.7 | | | -76.5 | 785.3 | -76.5 | -76.5 | -76.5 |
| LSI: | 1.03 | 1.03 | 1.03 | 1.03 | 1.03 | -0.19 | -0.19 | -0.19 | -0.19 | | | -5.76 | 1.95 | -5.76 | -5.76 | -5.76 |

| Parameter | FLOWSTREAM | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 17 | 18 | 19 | 20 | 21 | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 |
| Flowrate (gpm): | 362 | 543 | 3079 | 1539 | 1539 | 1278 | 4357 | 2178 | 4357 | 4357 | 4900 | 3079 | 4357 | 1207 | 1278 | 3079 |
| Flowrate (mgd): | 0.52 | 0.78 | 4.43 | 2.22 | 2.22 | 1.84 | 6.27 | 3.14 | 6.27 | 6.27 | 7.06 | 4.43 | 6.27 | 1.74 | 1.84 | 4.43 |
| **Water Quality** | | | | | | | | | | | | | | | | |
| Calcium (mg/L): | 882 | 882 | 0.87 | 0.87 | 0.87 | 133 | 40 | 40 | 40 | 40 | 133 | 0.87 | 40 | 133 | 133 | 1 |
| Magnesium (mg/L): | 232 | 232 | 0.23 | 0.23 | 0.23 | 35 | 10 | 10 | 10 | 10 | 35 | 0.23 | 10 | 35 | 35 | 0 |
| Sodium (mg/L): | 449 | 449 | 1.97 | 1.97 | 6.57 | 69 | 25 | 25 | 25 | 25 | 69 | 1.97 | 25 | 69 | 69 | 2 |
| Chloride (mg/L): | 841 | 841 | 0.99 | 0.99 | 0.99 | 127 | 38 | 38 | 38 | 38 | 127 | 0.99 | 38 | 127 | 127 | 1 |
| Nitrate (mg/L as NO3): | 512 | 512 | 3.79 | 3.79 | 3.79 | 80 | 26 | 26 | 26 | 26 | 80 | 3.79 | 26 | 80 | 80 | 4 |
| Alkalinity (mg/L as CaCO3): | 1236 | 1236 | 2.35 | 2.35 | 12.36 | 220 | 75 | 75 | 75 | 75 | 220 | 2.35 | 75 | 187 | 220 | 2 |
| Calcium Hardness (mg/L as CaCO3): | 2204 | 2204 | 2.18 | 2.18 | 2.18 | 333 | 99 | 99 | 99 | 99 | 333 | 2.18 | 99 | 333 | 333 | 2 |
| Magnesium Hardness (mg/L as CaCO3): | 954 | 954 | 0.95 | 0.95 | 0.95 | 144 | 43 | 43 | 43 | 43 | 144 | 0.95 | 43 | 144 | 144 | 1 |
| Total Hardness (mg/L as CaCO3): | 3158 | 3158 | 3.12 | 3.12 | 3.12 | 476 | 142 | 142 | 142 | 142 | 476 | 3.12 | 142 | 476 | 476 | 3 |
| Total Dissolved Solids (mg/L): | 5008 | 5008 | 10.84 | 10.84 | 19.62 | 780 | 242 | 245 | 245 | 245 | 780 | 10.84 | 245 | 760 | 780 | 11 |
| pH: | 7.5 | 7.5 | 5.2 | 5.2 | 10.0 | 7.4 | 8.0 | 8.0 | 8.0 | 8.0 | 7.4 | 5.2 | 8.0 | 6.9 | 7.4 | 5.2 |
| CCPP (mg/L): | 785.3 | 785.3 | -76.5 | -76.5 | -4.2 | 73.1 | 0.9 | 0.9 | 0.9 | 0.9 | 73.1 | -76.5 | 0.9 | -18.7 | 73.1 | -76.5 |
| LSI: | 1.95 | 1.95 | -5.76 | -5.76 | -0.28 | 1.03 | 0.11 | 0.11 | 0.11 | 0.11 | 1.03 | -5.76 | 0.11 | -0.19 | 1.03 | -5.76 |

**Notes:**

1. RO Permeate quality based on Filmtec ROSA v6.1.5 projection using BW30-400 RO elements, 5-yr membrane age, 85% recovery.
2. Chemical dosing and resulting water quality calculated using the Rothberg Tamburini & Winsor process chemistry water quality model.
3. Flowstreams 27 through 32 are intermittent, such as during startup, and are not typically online during normal plant operation.

# TREATMENT PROCESS FLOW DIAGRAM

FIGURE 5-25

IRVINE RANCH WATER DISTRICT

Engineers...Working Wonders With Water™

20-IRWD2-10F5-25-8212B00.AI

This page left intentionally blank.

### 5.4.1 Hydraulic Profile

The proposed hydraulic profile for the IRWD Wells 21 and 22 Desalter process is shown in Figure 5-26. The hydraulic profile includes the membrane process, decarbonators, clearwell, and product water pumping. The hydraulic profile is based upon the design water quality from Section 3, RO system recovery of 85 percent, and a 5-year membrane age. The hydraulics were evaluated at the maximum flow condition of 4,900 gpm (7.0 mgd).

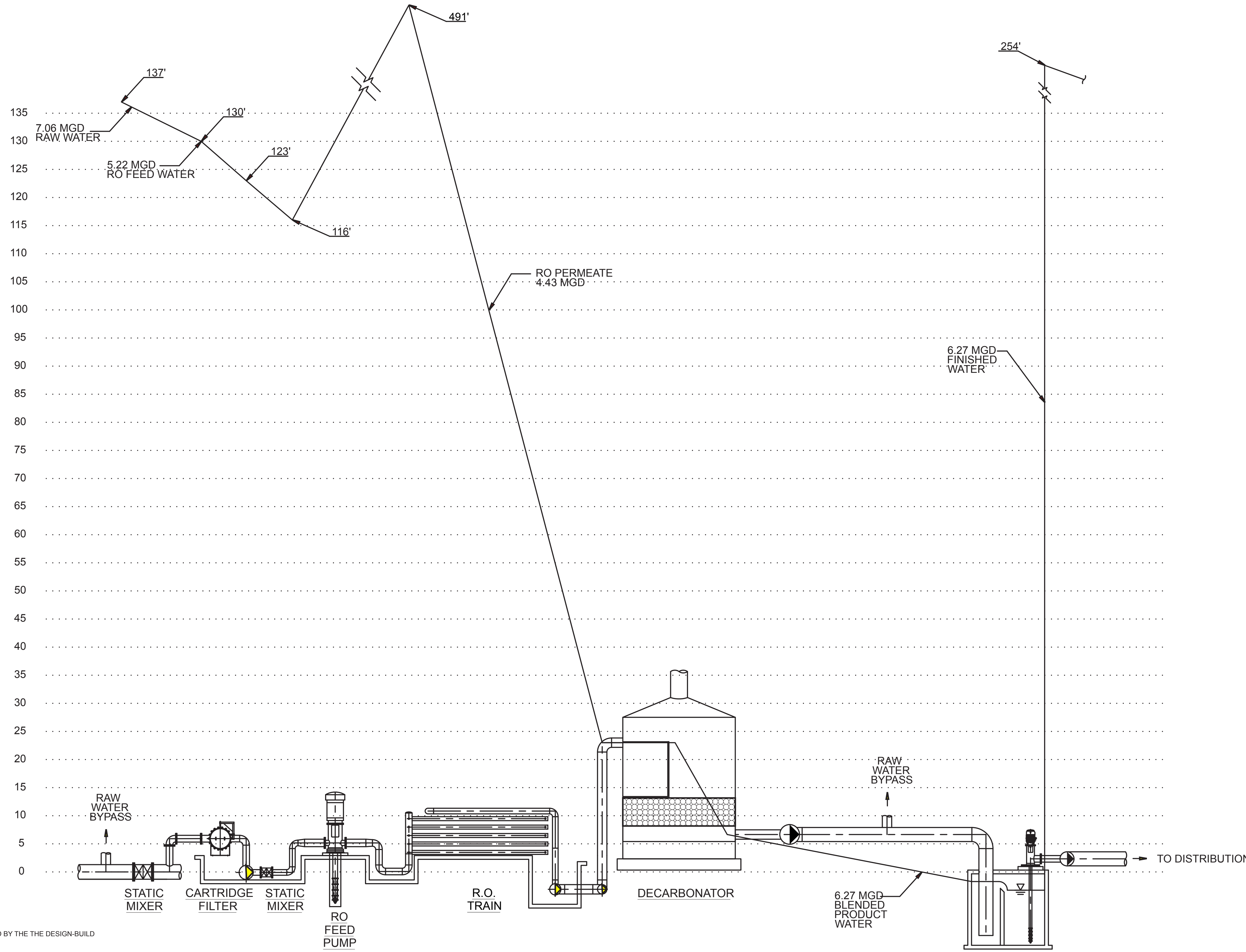
### 5.4.2 Site Plan

A site plan for the IRWD Wells 21 and 22 Desalter is presented in Figure 5-27. Major facilities on the site plan include:

- Cartridge filters
- Chemical storage and feed systems
- RO feed pumps
- RO trains
- Decarbonators
- Product water pump station
- Clearwell
- Storage/electrical building

The site plan has been superimposed onto the treatment site to create a preliminary plan for the facilities. The preliminary plan is shown in Figure 5-28.

This page left blank intentionally




MAIN PROCESS HYDRAULIC PROFILE
VERTICAL SCALE: 1" = 10'

NOTES:

1. HYDRAULIC PROFILE IS GENERAL IN NATURE AND SHALL BE REFINED BY THE THE DESIGN-BUILD CONTRACTOR DURING THE DESIGN PHASE.
2. ASSUMED RO FEED PUMP SUCTION PRESSURE = 50 PSIG.
3. PROFILE GENERATED USING DESIGN WATER QUALITY, 85% RECOVERY, 5-YR MEMBRANE AGE, NO PERMEATE THROTTLING.
4. PROFILE IS BASED ON 0' ELEVATION REFERENCE POINT. ACTUAL PROFILE SHALL BE ADJUSTED TO ACCOUNT FOR ACTUAL SITE ELEVATION.
5. EQUIPMENT AND PIPE SIZES ARE REPRESENTATIVE ONLY AND NOT TO SCALE.

## HYDRAULIC PROFILE

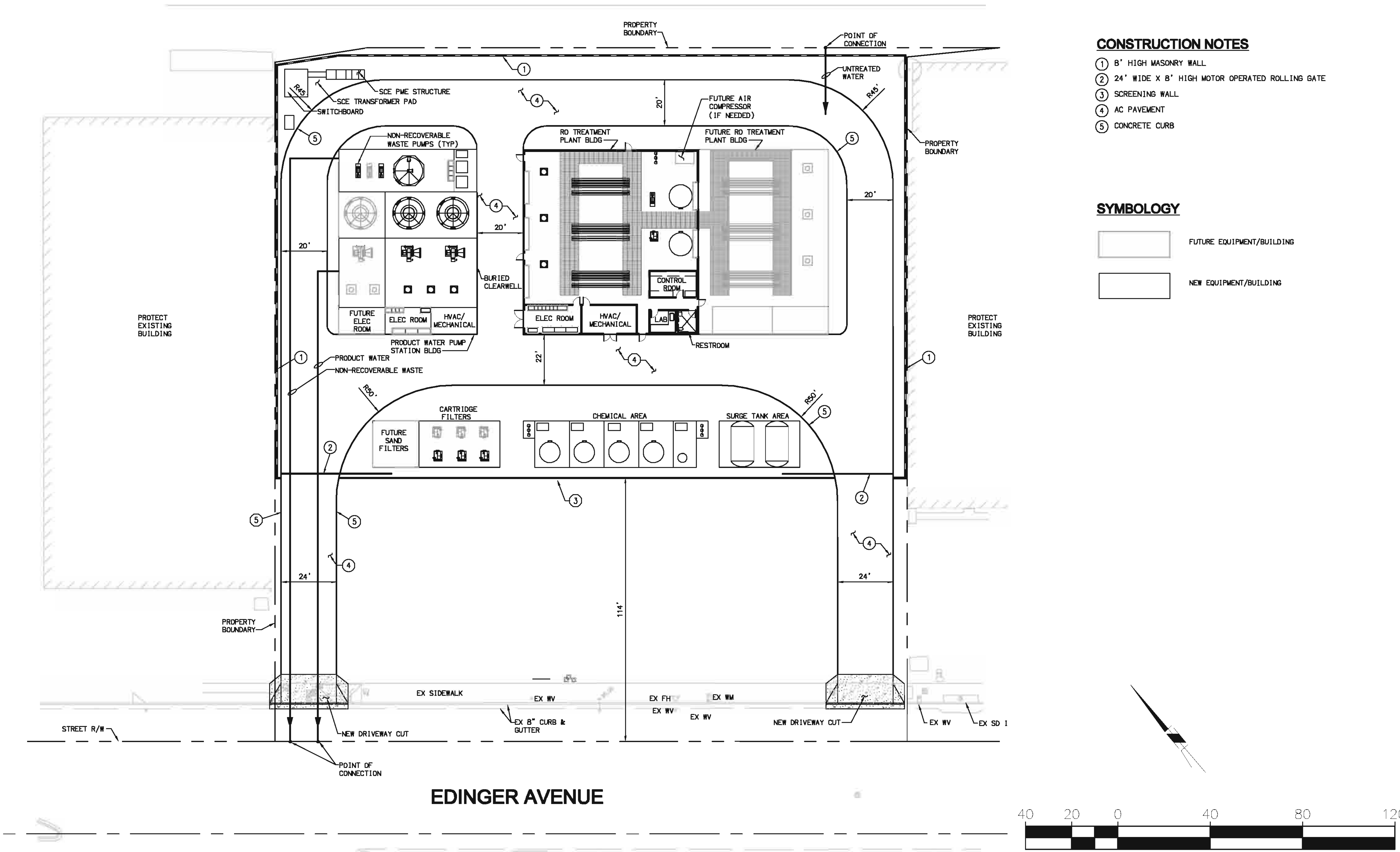
FIGURE 5-26

IRVINE RANCH WATER DISTRICT

Engineers...Working Wonders With Water™

20-IRWD2-10F5-26-8212B00.AI

This page left intentionally blank.




## CONSTRUCTION NOTES

1. 8' HIGH MASONRY WALL
2. 24' WIDE X 8' HIGH MOTOR OPERATED ROLLING GATE
3. SCREENING WALL
4. AC PAVEMENT
5. CONCRETE CURB

## SYMBOLOGY

FUTURE EQUIPMENT/BUILDING

NEW EQUIPMENT/BUILDING

H:\PDATA\10106006\CADD\WATER\EXHIBITS\WELLS 21-22 PDR REPORT\6006-FIG-5-27.DWG JWILEY 3/12/10 3:01 pm

RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

TREATMENT PLANT SITE PLAN

FIGURE 5-27

This page left intentionally blank.


100 50 0 100 200 300

SCALE: 1"=100'

RBF CONSULTING

PLANNING ■ DESIGN ■ CONSTRUCTION

14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

TREATMENT PLANT
SITE PLAN

FIGURE
5-28

This page left intentionally blank.

### 5.4.3 Reverse Osmosis Pretreatment

#### 5.4.3.1 Chemical Conditioning

The untreated water from the well field is treated with scale inhibitor and sulfuric acid. As water is pushed through the RO membranes, sparingly soluble salts of calcium and silica are concentrated and can precipitate on the membrane surface. In typical RO feed waters, ions of concern include calcium, strontium, barium, iron, manganese, silica, and aluminum. However, for the IRWD design water, the ions of concern are limited to calcium and silica. The pretreatment chemicals allow operation at supersaturated conditions, which in turn allows the RO systems to operate at higher system recovery. Higher recovery operation reduces the untreated water requirement and the volume of waste concentrate for disposal.

The chemicals are injected into static mixers (acid before the cartridge filters and scale inhibitor after), which provide the conditions to enable efficient mixing and prevent issues with acid and scale inhibitor being injected too close together. Design criteria for the scale inhibitor and sulfuric acid are presented in Section 5.4.11.

#### 5.4.3.2 Cartridge Filters

Cartridge filters are provided as a protective measure to prevent solids from reaching the RO membrane process. Solids, such as fine sands or silts, will result in RO membrane fouling and may cause serious mechanical damage to the RO membranes. The cartridge filters are provided as the final barrier to protect the valuable RO membranes against fouling or damage from particulates

Design criteria for the cartridge filter vessels are shown in Table 5-9. As indicated in Figure 5-25, there are three cartridge filter vessels, which share a common inlet manifold as well as a common outlet manifold. The cartridge filter vessels are sized so that a single vessel provides redundancy for the entire system if one cartridge filter vessel is out of service for replacement of cartridges.

**Table 5-9 Untreated Water Cartridge Filters Design Criteria**

| Description | Units | Criteria | |
|---|---|---|---|
| Vessel Orientation: Horizontal | | | |
| Cartridge Filter Type: Melt Blown | | | |
| Cartridge Filter Material: Polypropylene | | | |
| Cartridge Filter End Connection: Single Open End, Double O-Ring | | | |
| Cartridge Filter Rating | micron | 5 | |
| Cartridge Filter Length | inches | 40 | |
| Cartridge Filter Loading Rate[1] | gpm/10-inch | 1 to 4.7 | |
| Maximum Pressure Drop | | | |
| Clean Filter | psig | 3 | |
| Dirty Filter | psig | 15 | |
| Vessels (Total) | No. | 3 | |
| Vessels (Reliability) | No. | 1 | |
| Flow per Vessel (Firm Capacity) | | | |
| At 85 Percent Recovery | gpm (MGD) | 1,811 | (2.59) |
| Cartridge Filters per Vessel | No. | 103 | |
| Total Number of Cartridges | No. | 309 | |

Note:
Assumes one unit out of service.

### 5.4.4 RO Feed Pumps

The purpose of the RO feed pumps is to provide the energy to overcome osmotic pressure and dynamic head losses through the RO system. Each feed pump is dedicated to a single RO train. The feed pumps are multistage vertical turbines, mounted in cans with both the suction and discharge flanges on the pump head.

Design criteria for the feed pumps are shown in Table 5-10. As indicated in the table, there is no spare feed pump for any of the trains. Since this plant is being considered to augment existing water supply with local supplies, the additional cost of providing feed pump redundancy is not justifiable.

**Table 5-10 RO Feed Pumps**

| Description | Units | RO | |
|---|---|---|---|
| Type: Vertical Turbines in Pump Cans | | | |
| Number of Membrane Feed Pumps | | | |
| In-Service | No. | 3 | |
| Reliability | No. | 0 | |
| Total | No. | 3 | |
| Capacity (Per Pump) | | | |
| At 85 Percent Recovery | gpm (MGD) | 1,207 | (1.74) |
| Suction Pressure | | | |

| Description | Units | RO | |
|---|---|---|---|
| Minimum | ft $H_2O$ (psig) | 69 | (30) |
| Design | ft $H_2O$ (psig) | 116 | (50) |
| Maximum | ft $H_2O$ (psig) | 139 | (60) |
| Discharge Pressure | | | |
| Clean Membrane (Year 0) | ft H2O (psig) | 420 | (182) |
| Fouled Membrane (Year 5) | ft H2O (psig) | 520 | (225) |
| Total Dynamic Head (TDH) | | | |
| Clean Membrane (Year 0) | ft $H_2O$ (psig) | 282 | (122) |
| Dirty Membrane (Year 5) | ft $H_2O$ (psig) | 427 | (185) |
| Motor Horsepower | | | |
| Per Pump | hp | 200 | |
| Total (In-Service) | hp | 600 | |
| Drive | | VFD | |

### 5.4.5 RO Membrane Trains

The RO membrane trains are the primary process for TDS removal, including nitrates. The proposed plant will consist of three trains each having two stages and each stage containing seven elements per pressure vessel. High rejection RO membranes will be used in both stages. The membrane trains receive pressurized feedwater from the feed pumps. The pressure “pushes” water through the membranes while salt is rejected. The rejected salts are concentrated into a small percentage of the flow and exit the system as waste. The RO trains are designed to operate at a recovery of 85 percent. Two stage systems with seven elements per vessel typically do not operate over 85 percent because the individual element recovery exceeds the manufacturer-defined operational range. The design recovery is based on water quality data from the wells. As the wells are brought online, the parameters controlling recovery may change.

The proposed configuration of the membrane trains, together with pertinent design criteria are shown in Table 5-11. As indicted in Table 5-11, the Wells 21 and 22 Desalter has three RO membrane trains. Each train operates independently to produce 1,026 (1.48 mgd) of permeate plus 181 gpm (0.3 mgd) of concentrate.

**Table 5-11 NF-RO System Design Criteria**

| Description | Units | RO-RO | |
|---|---|---|---|
| Feed Water Flow | gpm (MGD) | 3,622 | (5.22) |
| Permeate Water Flow | gpm (MGD) | 3,079 | (4.43) |
| Concentrate Water Flow | gpm (MGD) | 543 | (0.78) |
| Number of Membrane Trains | | | |
| In-Service | No. | 3 | |
| Reliability | No. | 0 | |
| Total | No. | 3 | |
| Permeate Flow per Train | gpm (MGD) | 1,026 | (1.48) |
| Train Flux Rate | gfd | 14.7 | |
| Recovery (Permeate/Feed Flow) | percent | 85 | |
| Number of Array Stages per Train | No. | 2 | |
| 1st Stage | | | |
| Pressure Vessels per Train | No. | 24 | |
| Elements per Pressure Vessel | No. | 7 | |
| 2nd Stage | | | |
| Pressure Vessels per Train | No. | 12 | |
| Elements per Pressure Vessel | No. | 7 | |
| Number of Elements | | | |
| Per Train | No. | 252 | |
| Total (In-service) | No. | 756 | |
| Manufacturer | | Hydranautics | |
| Model | | | |
| Stage 1 | | ESPA2 | |
| Stage 2 | | ESPA2 | |
| Membrane Area | | | |
| Per Element | sq. ft. | 400 | |
| Per Train | sq. ft. | 100,800 | |
| Total (In-service) | sq. ft. | 302,400 | |

### 5.4.6 Energy Recovery

Increasingly, the design of RO membrane systems are beginning to include energy recovery devices as a means of reducing energy costs. These high-pressure membrane systems produce concentrate streams that leave the system at 20 to 50 psi below the feed pressures. The residual pressure, if not needed to convey the concentrate to a disposal site, is typically dissipated across a control valve. For seawater systems, with extremely high feed pressures, the energy dissipated can be as high as 900 psi. This energy may be harnessed and re-directed to the feed side of the RO system, reducing the size of feed pumps needed. In some cases, these devices approach energy recovery efficiency in excess of 95 percent, dramatically reducing the cost of seawater RO. The decision to employ energy recovery is dependent on the feed pressure, which is related to the TDS. Basically, the feed pressures must be high enough so that the energy available for harnessing on the concentrate stream can reduce the pumping size sufficiently to produce an energy savings-based payback period of less than 5 years.

Historically, reasonable payback periods for brackish water energy recovery devices require TDS levels in excess of 4,000 mg/L. At 780 mg/L feedwater TDS, the IRWD Wells 21 and 22 water does not have the conditions to justify the use of energy recovery devices.

### 5.4.7 Membrane Clean in Place (CIP) system

The CIP system is used to chemically clean and remove foulants (e.g., particles, mineral scale, and biology) from the RO membranes. Foulants result in additional headloss and increased energy requirements to maintain production flow rates. Additionally, foulants may result in a deterioration of permeate water quality.

The CIP system circulates cleaning chemicals to the membrane trains. The CIP system is permanently connected to the membrane skid piping in order to avoid the labor, time, and safety issues involved in connecting and disconnecting hoses or pipe spools. The CIP connections to the permeate side of the RO membrane will have block valves and removable spool pieces to insure that the treated water is isolated from the cleaning solution while in service.

Each stage on the membrane train is cleaned separately to deliver the required cleaning flow velocities to each pressure vessel in the array. The design criteria for the CIP system are presented in Table 5-12. The CIP system design presented in Table 5-12 allows up to 24 membrane pressure vessels to be cleaned at the same time. Cleaning chemical design and costs assume three-stage cleaning with designer chemicals targeting silica. Based on membrane systems performance projections, at the design recovery, silica is expected to be the scalant of concern. Costs include two Step 1, Step 2 and Step 3 cleaning events per year.

The cleaning pump is driven by a variable speed drive and has been sized to provide cleaning flow rates of at least 50 gpm per vessel. The CIP tanks are equipped with immersion heaters capable of heating the CIP solution to temperature of 100ºF. Both of these features are critical to achieve effective cleaning.

Spent CIP solution is required to be neutralized before being discharged to the sanitary sewer system. Spent CIP solution can be neutralized inside the CIP tank by addition appropriate chemicals (as recommended by the CIP chemical manufacturer) to achieve a neutral pH solution. After draining, the tank must be rinsed before being returned to service.

**Table 5-12 Membrane Clean-in-Place System Design Criteria**

| | Units | Criteria |
|---|---|---|
| Pressure Vessel Production Flow Rate | | |
| Stage 1 | gpm/ea. | 29 |
| Stage 2 | gpm/ea. | 28 |
| Pressure Vessel Cleaning Flow Rate | | |
| Stage 1 | gpm./ea | 50 |
| Stage 2 | gpm/ea | 50 |
| Vessels Cleaned Per Cycle | | |
| Stage 1 | No. | 24 |
| Stage 2 | No. | 12 |
| CIP Chemical Tanks | No. (gal) | 2 (4,000) |
| CIP Recirculation Pump | | |
| Type | | FRP End-Suction Centrifugal |
| Number | No. | 1 |
| Flow | gpm | 1,200 |
| Discharge Pressure | Psig | 70 |
| Total Dynamic Head | ft $H_2O$ | 162 |
| Horsepower[1] | Hp | 125 |
| Driver | | VFD |
| CIP Cartridge Filter Micron Rating | Micron | 5 |
| CIP Cartridge Filter Loading Rate | gpm/10-inch | 1.74 - 3.5 |
| Length of Cartridge Filters | In | 40 |
| No. of CIP Cartridge Filters | No. | 86 |
| Maximum CIP Filter Pressure Drop | | |
| Clean Filter | Psig | 3 |
| Dirty Filter | Psig | 20 |
| CIP Tank Heater | | |
| Number per Tank | No. | 2 |
| Size | kW | 100 |
| Total | kW | 200 |
| CIP Mixer | | |
| Number per Tank | No. | 1 |
| Size | Hp | 1 |

Notes:
1. Assuming pump efficiency of 52 percent

## 5.4.8 Post Treatment

### 5.4.8.1 Decarbonators

The decarbonation process is designed to reduce the concentration of carbon dioxide in the membrane permeate water. Removal of carbon dioxide raises the pH, lowers alkalinity, and reduces the amount of post treatment caustic soda required for product water pH adjustment. The decarbonation tower is an air/water mass transfer process where water cascades downward on top of packing media designed to enhance the surface area available for air/water contact. Air flows counter current to the water stream and exits the top of the tower, which is vented to atmosphere. Sodium hypochlorite and caustic soda are added in the sump of the decarbonator to allow for thorough mixing and prevent scaling of the injectors.

The design criteria for the decarbonators are presented in Table 5-13. As indicated in Figure 5-25, the permeate from the RO trains passes through the decarbonators before it is mixed with untreated groundwater. Under normal operating conditions, the entire 1,278 gpm of untreated groundwater will be routed around the decarbonators. Based on modeling results, this flow distribution will allow the plant to operate without the need for corrosion inhibitor. There are two decarbonation towers, each treating half of the combined flow.

**Table 5-13 Decarbonation Tower Design Criteria**

| | Units | Criteria |
|---|---|---|
| Type: Packed Tower Aerators | | |
| Number of Decarbonation Towers | No. | 2 |
| Diameter | ft | 10 |
| Water Flow Rate | gpm/ea | 1,540 |
| Loading Rate | gpm/ft$^2$ | 20 |
| Air Flow Rate per Decarbonator | scfm | 6,000 |
| Air/Water Loading | scfm/gpm | 3.9 |
| Air $\Delta$P(inlet duct to exhaust) | in. $H_2O$ | 7.5 |
| No. of Blowers | No. | 2 |
| Capacity per Blower | cfm | 6,000 |
| Bower horsepower | hp/ea. | 10 |
| Drive | CS | CS |

| | Units | Criteria |
|---|---|---|
| $CO_2$ Removal | | |
| Inlet $CO_2$ | mg/L | 42 |
| Outlet $CO_2$ | mg/L | <5 |
| Removal Efficiency | percent | >85 |
| Inlet pH | pH units | 5.2 |
| Inlet Bicarbonate | mg/L | 4 |
| Inlet Temperature | ºC(ºF) | 20(68) |

### 5.4.9 Clearwell

Blended decarbonator effluent flows to a buried clearwell, which provides sufficient storage to serve as a buffer between production and the ground storage and distribution system. As indicated in Table 5-14, the clearwell is sized to provide approximately 30 minutes of storage at design finish water flow rate. Aqua ammonia is added in the clearwell atop an influent weir to adequately mix and form chloramines while preventing injector scaling.

**Table 5-14 Clearwell Design Criteria**

| | Units | Criteria |
|---|---|---|
| Flow rate | gallon(mgd) | 4,357 (6.28) |
| Volume | gallon | 150,000 |
| Detention Time | min | 34.4 |

### 5.4.10 Product Water Pump Station

Water flows from the clearwell to the product water pump station. From the clearwell, the product water pumps deliver the potable water to IRWD's Zone 1 distribution system.

Design criteria for the product water pump station are presented in Table 5-15.

**Table 5-15 Product Water Pump Design Criteria**

| Description | Units | Criteria |
|---|---|---|
| Type : Vertical Turbines | | |
| Required Capacity (Total) | gpm (mgd) | 4,357 (6.28) |
| No. of Pumps | No | 3 |
| In-Service | No. | 2 |
| Redundant | No. | 1 |
| Total | No. | 3 |
| Pump Flow Rate | gpm (mgd) | 2,179 (3.14) |
| Total Dynamic Head (TDH)[1] | ft $H_2O$ | 330 |
| Discharge Pressure[1] | psig | 135 |
| Motor Horsepower | | |
| Per Pump | hp | 300 |
| Total On-Line | hp | 600 |
| Drive | | VFD |

Note: VFD = Variable Frequency Drive

1. Based on IRWD Zone 1 hydraulic modeling results

### 5.4.11 Chemical Feed Systems

#### 5.4.11.1 Sulfuric Acid

Calcium carbonate scale is controlled by lowering feedwater pH with sulfuric acid, which results in a lower concentrate Langlier Saturation Index (LSI). Historically, an LSI of +1.8 in the concentrate with the addition of scale inhibitors was the baseline for acid dosing. However, advances in scale inhibitors have allowed for successful operation at concentrate LSIs up to 2.5.

Currently, the water chemistry indicates that silica scaling will control the RO system recovery at the proposed membrane plant. Silica solubility is a function of temperature and pH and is not affected in pH ranges from 7 to 8. Based on the design water presented in Section 3 and membrane systems performance projections at 85 percent recovery, the concentrate LSI with no acid addition is approximately 2.58. At this concentrate LSI, the probability of eliminating acid while achieving the design recovery is very low. This justifies the inclusion of a sulfuric acid dosing system for untreated water pH adjustment.

The design criteria for the sulfuric acid system are presented in Table 5-16.

**Table 5-16 Sulfuric Acid Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Sulfuric Acid Characteristics: | | |
| Concentration: 93 percent | % | 93 |
| Specific Gravity: 1.8 | NA | 1.8 |
| Solution Strength: 13.96 lb/gal | Lb/gal | 13.96 |
| Chemical Usage | | |
| Location: Untreated Water | | |
| Process Flow | mgd | 5.22 |
| Chemical Dose | mg/L | 32 |
| Chemical Usage | lb/day | 1,392 |
| Chemical Feed Rate | gpd | 100 |
| Chemical Feed Rate | gph | 4.15 |
| Bulk Storage Tanks | | |
| Number of Tanks | | 1 |
| Tank Capacity, each | Gal | 3,500 |
| Tank Capacity, total | Gal | 3,500 |
| Storage time | Days | 34 |
| Delivery Truck Load full | gal | 3,000 |
| Time Between Delivery | Days | 25 |
| Metering Pumps | | |
| PumpType: Diaphragm | | |
| Motor: AC w/ VFD Drive | | |
| Metering Pump Capacity | gph | 6.9 |
| Turndown (motor speed only) | Ratio | 12:1 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | | 1 |

5.4.11.2 Scale Inhibitor

Scale inhibitor (SI) is added to prevent the precipitation of sparingly soluble salts that may foul the membranes as the feed water becomes a concentrated byproduct. As the RO membrane system feed water becomes more concentrated, the saturation limit of sparingly soluble salts (e.g., $CaCO_3$, $CaSO_4$, $BaSO_4$, $SrSO_4$, $CaF_2$, and $SiO_2$) may be surpassed and precipitation of these salts may occur on the RO membranes. This type of fouling is referred to as scaling. Scale formation will result in increased operating costs (i.e., higher pumping pressures and chemical cleaning to dissolve the scale and restore membrane productivity) and a deteriorated permeate water quality. As indicated in Figure 5-25, scale inhibitor is added to the RO feed water before the cartridge filters.

Design criteria for the scale inhibitor feed system is presented in Table 5-17.

**Table 5-17 Scale Inhibitor Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Scale Inhibitor Characteristics: | | |
| Manufacturer: Nalco | | |
| Specific Gravity: 1.06 | | |
| Solution Strength: 8.84 lb/gal | | |
| Chemical Usage | | |
| Location: Untreated Water | | |
| Process Flow | mgd | 5.22 |
| Chemical Dose | mg/L | 4.0 |
| Chemical Usage | lb/day | 174 |
| Chemical Feed Rate | gpd | 19.7 |
| Chemical Feed Rate | gph | 0.82 |
| Bulk Storage Tanks | | |
| Number of Tanks | | 1 |
| Tank Capacity, each | gal | 2,000 |
| Tank Capacity, total | gal | 2,000 |
| Storage time | Days | 102 |
| Delivery Truck Load full | Gal | 1,500 |
| Time Between Delivery | Days | 76 |
| Metering Pumps | | |
| Pump Type: Diaphragm | | |
| Motor: AC w/ VFD Drive | | |
| Metering Pump Capacity | gph | 1.03 |
| Turndown (motor speed only) | ratio | 8:1 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | | 1 |

5.4.11.3 Caustic Soda

Caustic soda is added to adjust the product water pH and produce a condition, in conjunction with alkalinity and calcium, that is slightly precipitating with respect to calcium carbonate. Caustic soda is added in the decarbonators to ensure adequate mixing and prevent scaling of the injectors. As presented in Section 3 the recommended product water CCPP is between 4 to 10 mg/L as $CaCO_3$ to form a protective film of calcium carbonate on the walls of the piping in the distribution system. This protective film provides a physical barrier between the metallic pipe and water, thereby helping to prevent corrosion. The design criteria for the caustic soda dosing system are presented in Table 5-18.

The caustic soda feed rate is dose controlled based upon operator adjustable dosage set point pH setpoint, which when evaluated with alkalinity and calcium hardness, will result in an acceptable CCPP. To prevent freezing, the caustic soda is stored and delivered at 25 percent solution.

**Table 5-18 Caustic Soda Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Caustic Soda Characteristics: | | |
| Concentration: 25 percent | | |
| Specific Gravity: 1.27 | | |
| Solution Strength: 2.65 lb/gal | | |
| Chemical Usage | | |
| Location: Decarbonator Sump (RO Permeate | | |
| Process Flow | mgd | 4.43 |
| Chemical Dose | mg/L | 8 |
| Chemical Usage | lb/day | 296 |
| Chemical Feed Rate | gpd | 112 |
| Chemical Feed Rate | gph | 4.65 |
| Bulk Storage Tanks | | |
| Number of Tanks | | 1 |
| Tank Capacity, each | gal | 5,000 |
| Tank Capacity, total | gal | 5,000 |
| Storage time | days | 45 |
| Delivery Truck Load full | gal | 3,800 |
| Time Between Delivery | days | 34 |
| Metering Pumps | | |
| Pump Type: Diaphragm | | |
| Motor: AC w/ VFD Drive | | |
| Metering Pump Capacity | gph | 8.23 |
| Turndown (motor speed only) | ratio | 10:1 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | | 1 |

5.4.11.4 Sodium Hypochlorite

As indicated in Figure 5-25, sodium hypochlorite can be added in the following locations depending on the mode of operation:

- In sumps of the decarbonators

- On the discharge side of the product water pumps.

Sodium hypochlorite is added to provide, when combined with ammonia, residual disinfectant in the distribution system. Sodium hypochlorite is stored as 10 to 12.5 percent solution.

Design criteria for the sodium hypochlorite feed system are presented in Table 5-19.

**Table 5-19 Sodium Hypochlorite Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Sodium Hypochlorite Characteristics: | | |
| Concentration: 12.5 percent | | |
| Specific Gravity: 1.15 | | |
| Solution Strength: 1 lb $Cl_2$ /gal | | |
| Chemical Usage | | |
| Process Flow | mgd | 6.27 |
| Chemical Dose (as CL2) | mg/L | 2.5 |
| Chemical Usage | lb/day | 131 |
| Chemical Feed Rate | gpd | 131 |
| Chemical Feed Rate | gph | 5.46 |
| Bulk Storage Tanks | | |
| Number of Tanks | | 1 |
| Tank Capacity, each | gal | 2,000 |
| Tank Capacity, total | gal | 2,000 |
| Storage time | days | 15 |
| Delivery Truck Load full | gal | 1,100 |
| Time Between Delivery | days | 8 |
| Metering Pumps | | |
| Pump Type: Diaphragm | | |
| Metering Pump Capacity | gph | 8.72 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | | 1 |

5.4.11.5 Aqua Ammonia

As indicated in Figure 5-25, aqua ammonia can be added in the following flow locations:

- Above the influent weir in the clearwell

- On the discharge side of the product water pumps.

Aqua ammonia is fed at a chlorine to ammonia (as nitrogen) ratio of approximately 5:1. The ammonia reacts with chlorine to form chloramines, which reduce the formation of potential disinfection byproducts in Zone 1's distribution system. The 5:1 chlorine to ammonia ratio is a guideline used to prevent potential nitrification and biological re-growth in the distribution system. Aqua ammonia is stored and delivered as 30 percent solution.

The design criteria for the aqua ammonia feed system are presented in Table 5-20.

**Table 5-20 Aqua Ammonia Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Aqua Ammonia Characteristics: | | |
| Concentration: 30 percent | | |
| Specific Gravity: 0.896 | | |
| Solution Strength: 2.24 lb/gal | | |
| Chemical Usage | | |
| Process Flow | mgd | 6.27 |
| Chemical Dose | mg/L | 1.25 |
| Chemical Usage | lb/day | 65 |
| Chemical Feed Rate | gpd | 29 |
| Chemical Feed Rate | gph | 1.22 |
| Bulk Storage Tanks | | |
| Number of Tanks | | 1 |
| Tank Capacity, each | gal | 1,000 |
| Tank Capacity, total | gal | 1,000 |
| Storage time | days | 34 |
| Delivery Truck Load full | gal | Variable |
| Time Between Delivery | days | 17 (based on 500 gal delivery) |
| Metering Pumps | | |
| Pump Type: Diaphragm | | |
| Metering Pump Capacity | gph | 2.43 |
| Turndown (motor speed only) | ratio | 17:1 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | No. | 1 |

5.4.11.6 Sodium Bisulfite

As indicated in Figure 5-25, sodium bisulfite can be added to the product water on the discharge of the product water pumps when the flow is being diverted to the storm water system. Sodium bisulfite removes chlorine from the product water to ensure that the water entering the storm water system is chlorine free.

Sodium bisulfite is fed at a bisulfite to chlorine ratio of approximately 1.5:1. Sodium bisulfite is stored and delivered as 30 percent solution. Because of the infrequency of usage, bulk storage is not provided for sodium bisulfite. Instead, the chemical feed system is fed from 275-gallon totes.

Design criteria for the sodium bisulfite feed system are presented in Table 5-21.

**Table 5-21 Sodium Bisulfite Design Criteria**

| Parameters | Units | Criteria |
|---|---|---|
| Sodium Bisulfite Characteristics: | | |
| Concentration: 30 percent | | |
| Specific Gravity: 0.896 | | |
| Solution Strength: 2.24 lb/gal | | |
| Chemical Usage | | |
| Process Flow | mgd | 6.27 |
| Chemical Dose | mg/L | 2.25 |
| Chemical Usage | lb/day | 118 |
| Chemical Feed Rate | gpd | 28 |
| Chemical Feed Rate | gph | 1.15 |
| Tote Storage | | |
| Number of Totes | | 2 |
| Tote Capacity, each | gal | 275 |
| Tote Capacity, total | gal | 550 |
| Storage time | days | Variable |
| Metering Pumps | | |
| Pump Type: Diaphragm | | |
| Metering Pump Capacity | gph | 3.08 |
| Turndown (motor speed only) | ratio | 15:1 |
| No. of Standby Pumps | | 1 |
| No. of Pumps in Service | No. | 1 |

#### 5.4.11.7 Electrical Power Systems

##### 5.4.11.7.1 Electrical Load Summary

Electrical loads for the plant are presented in Table 5-22.

**Table 5-22 Electrical Plant Loads**

| Load Description | Location | Units | Unit Load | No. | Connected Load, HP |
|---|---|---|---|---|---|
| RO Feed Pumps | RO Process Building | hp | 200 | 3 | 600 |
| Decarbonator Blowers | RO Process Building | hp | 10 | 2 | 20 |
| CIP Recirculation Pump | RO Process Building | hp | 125 | 1 | 125 |
| CIP Solution Heaters | RO Process Building | kW | 100 | 4 | 536 |
| CIP Mixers | RO Process Building | hp | 1 | 2 | 2 |
| Metering Pumps | RO Process Building | hp | 0.5 | 10 | 5 |
| Product Water Pumps[2] | TBD | hp | 300 | 3 | 600 |
| | | | **Grand Total** [1] | | **1,888** |

Note:
1. Does not include HVAC, lighting, or receptacles.
2. Two duty pumps with one in standby

## 5.5 Brine Discharge Piping

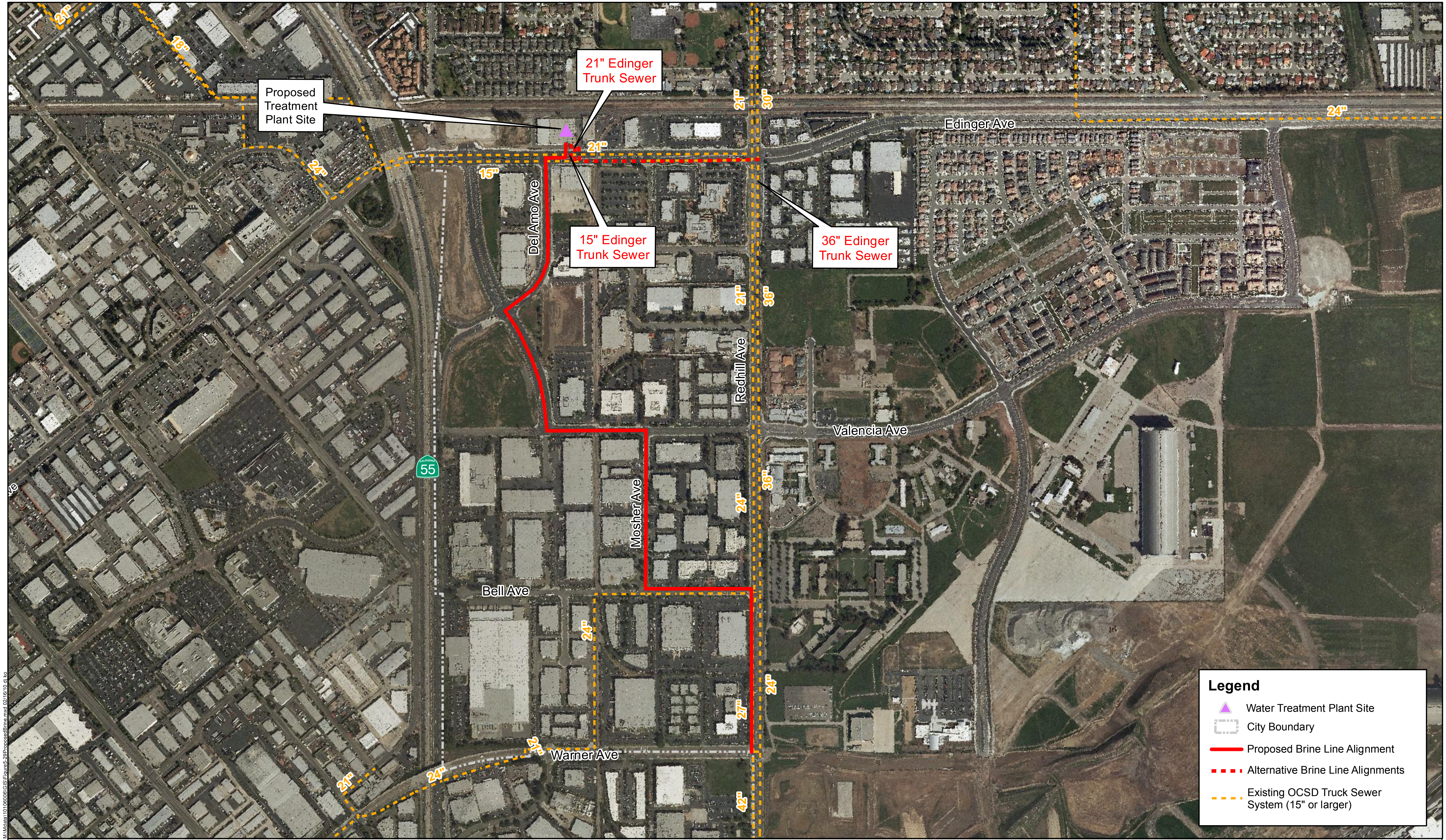
Brine generated from the treatment facility is proposed to be discharged to the existing sewer facilities in Red Hill Avenue owned and maintained by OCSD. The proposed brine pipeline will convey the brine, under pressure, from the treatment site southerly and easterly to the existing 42-inch diameter sewer located at the intersection of Red Hill Avenue and Warner Avenue. The non-recoverable waste pumps from the treatment plant will be used to pump the brine through a force main to this connection. To minimize impacts on Red Hill Avenue traffic during construction, the brine pipeline alignment is proposed to run in Del Amo Avenue, Valencia Avenue, Mosher Avenue, and Bell Avenue to reach Red Hill Avenue and continue south to connect to the 42-inch OCSD sewer. This alignment, as well as the alternative alignments and connection points evaluated for the brine pipeline in Section 4.6.2 of this PDR, are shown in Figure 5-29.

Based on the amount of brine flow anticipated from the treatment facility (approximately 540 gpm), and based on the District's force main design standards, which dictate that the nominal design velocity for a force main should be 3.0 fps, (ranging from a minimum of 2.0 fps to a maximum of 6.0 fps), the force main size to convey the brine is a minimum 8-inch diameter, however, the brine pipeline can be upsized to 10-inch diameter without violating the 2.0 fps low velocity criteria and this size line could accommodate future flows from potential nearby (or adjacent) treatment facilities or additional waste/flush streams from the Wells 21 and 22 plant, therefore, 10-inch diameter (approximate inside diameter) is recommended.

The brine waste, unless chemically conditioned prior to discharge will be corrosive and have scale potential, therefore, pipe material shall be butt fusion welded HDPE or C-900 PVC for maximum corrosion resistance. HDPE (DR 13.5 or Min 160 psi pressure rating) is the preferred choice as there is no thrust restraint required and has limited potential for scale gathering at pipe joints due to the relatively smooth transition from the butt fusion joining method. The pipeline should be designed to flow full so the higher scale potential at the air-water interface is eliminated in the pipeline.

Based on District standards, the minimum depth of cover from finish street grade to the top of sewer force main pipe shall be 7 feet. Sewer trench backfills, as shown in Figure 5-30, shall be compacted to 90 percent relative density as determined by the five-layer test method.

This page left intentionally blank



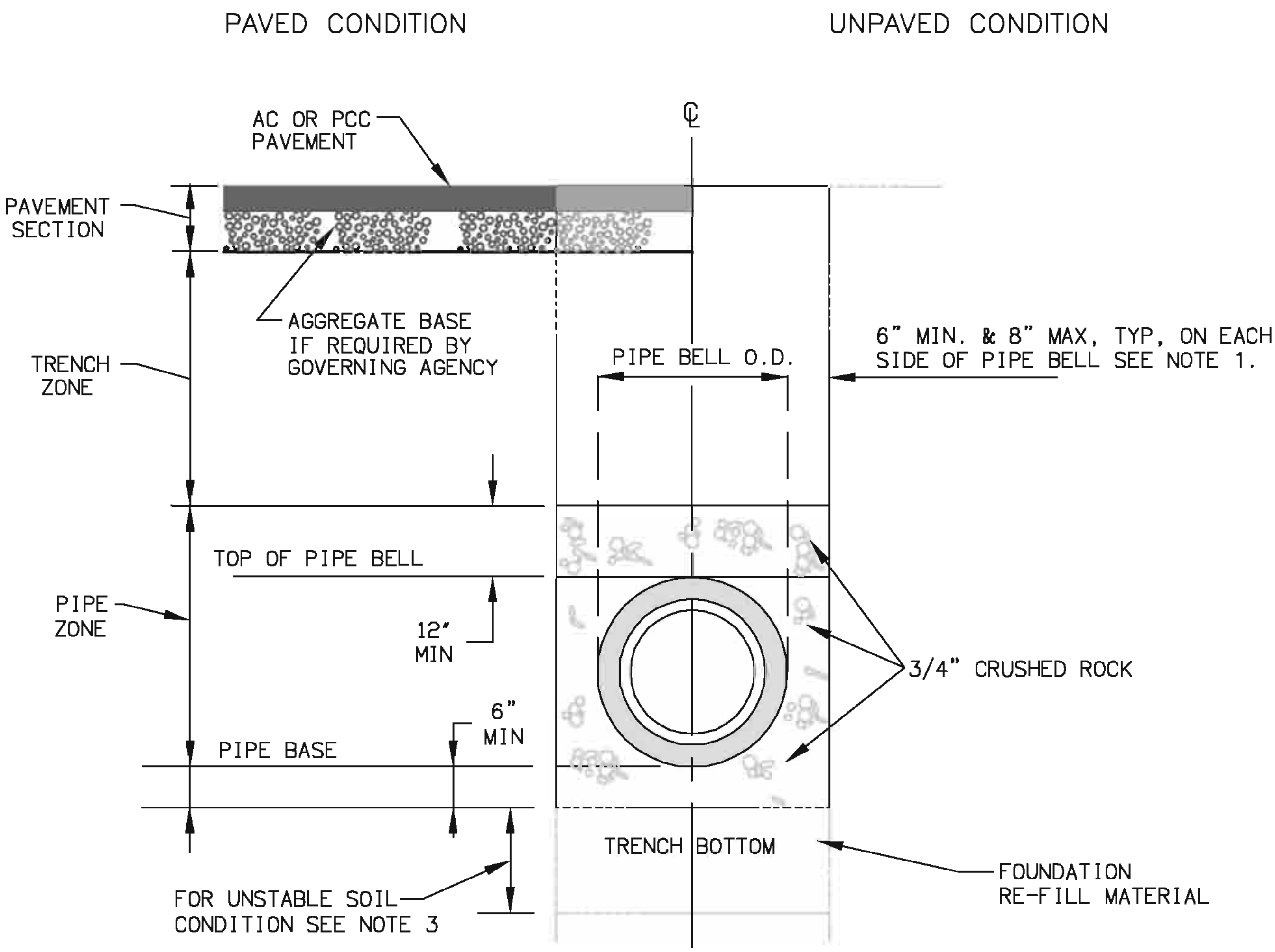

0 700 1,400
Feet

Source: Eagle Aerial, 2009

# IRWD Wells 21 & 22

## Figure 5-29: Proposed Brine Disposal Pipeline

This page left intentionally blank.




NOTES:

1. ALL WORK SHALL BE IN ACCORDANCE WITH IRWD STD. SPEC. SECTION 02223.
2. WHERE CONTRACTOR FAILS TO MAINTAIN PROPER TRENCH WIDTH LIMITS, SPECIAL BACKFILL SUCH AS ONE-SACK SLURRY, AND BEDDING SHALL BE DETERMINED IN THE FIELD BY THE DISTRICT REPRESENTATIVE.
3. IF UNSTABLE SOIL IS ENCOUNTERED, THE DISTRICT REPRESENTATIVE SHALL DETERMINE OVEREXCAVATION DEPTH AND FOUNDATION RE-FILL MATERIAL PER IRWD STD. SPEC. SECTION 02223.
4. CONTRACTOR SHALL PROVIDE HAND EXCAVATED "BELL HOLE" FOR EACH PIPE JOINT SO THAT THE WEIGHT OF PIPE DOES NOT BEAR ON THE BELL. CONTRACTOR SHALL RE-FILL AND HAND-TAMP EACH 'BELL HOLE' PRIOR TO COMPLETING THE PLACEMENT OF PIPE BEDDING.



RBF CONSULTING
PLANNING ■ DESIGN ■ CONSTRUCTION
14725 ALTON PARKWAY
IRVINE, CALIFORNIA 92618-2027
949.472.3505 ▪ FAX 949.472.8373 ▪ www.RBF.com

WELLS 21 AND 22

IRWD SEWER TRENCH DETAIL

FIGURE 5-30

This page left intentionally blank.

## 5.6 Regulatory Requirements

### 5.6.1 Environmental Due Diligence Analysis

A preliminary environmental due diligence (EDD) was conducted in preparation of the *May 2009, Wells 21 and 22 Draft PDR*. The environmental due diligence was prepared based upon available data, additional research through City planning documentation, and an assessment of existing technical data. The environmental due diligence was prepared for the entire subject site to identify key issues, permits, and approvals that may pose significant constraints and opportunities for development with the proposed use. A site reconnaissance was also conducted on September 3, 4, and 9 through 11, 2008, and April 29, 2009, to review the existing physical conditions on- and off-site. The entire EDD analysis along with a critical issues analysis checklist is included in Appendix N.

### 5.6.2 Project Environmental Documents

The (EDD) analysis included a list of required environmental documentation, including the preparation of an Initial Study/Mitigated Negative Declaration (IS/MND), and other appropriate documentation under the California Environmental Quality Act (CEQA) and National Environmental Policy Act (NEPA).

To expedite the project, a mitigated negative declaration (MND) - Initial Study/Environmental Assessment (IS/EA) was completed by IRWD for the proposed project in February 2010. The document is required to meet both California Environmental Quality Act (CEQA) and the National Environmental Policy Act (NEPA). The document details environmental factors potentially affected by the project and incorporates the required mitigation measures. The entire document is available at http://www.irwd.com/AboutIRWD/board_CEQA.php.

The MND IS/EA identified the following environmental factors that could potentially be affected by the project and provided mitigation measures as required:

- Aesthetics
- Biological Resources

- Hazards and Hazardous Materials
- Utilities and Service Systems
- Cultural Resources
- Hydrology and Water Quality
- Noise
- Geology, Soils and Seismicity
- Transportation and Traffic

### 5.6.3 Permits

The following required permits have been identified for each of the project components:

Well Equipping

- Orange County - Flood Control Discharge Encroachment Permit
- City of Tustin Encroachment Permit - (For Connecting Pump to Waste Line to City Storm Drains)
- RWQCB (For Pump to Waste)
- CDPH – Well Operating Permit
- OCSD Special Purpose Discharge Permit

Untreated Water Conveyance Pipeline

- City of Tustin Encroachment Permit
- Orange County - Flood Control Encroachment Permit (For Crossing Santa Fe Channel)
- OCTA Metrolink - right of entry/crossing approval
- OSHA – tunneling permit

Product Water Pipeline

- City of Tustin Encroachment Permit
- City of Irvine Encroachment Permit
- Caltrans encroachment permit for crossing SR-261
- Orange County - Flood Control Encroachment Permit (For crossing Peters Canyon Channel)
- Regulatory Agencies: CA DF&G, US F&W
- OSHA – tunneling permit

<u>Treatment Plant</u>

- Orange County - Flood Control Discharge Encroachment Permit
- Orange County - Flood Control Encroachment Permit (For connection to Santa Fe Channel)
- California Department of Public Health
- City of Tustin Encroachment Permit
- RWQCB (For Connection to Santa Fe Channel)
- CDPH – Operating Permit
- OCFA

<u>Brine Disposal</u>

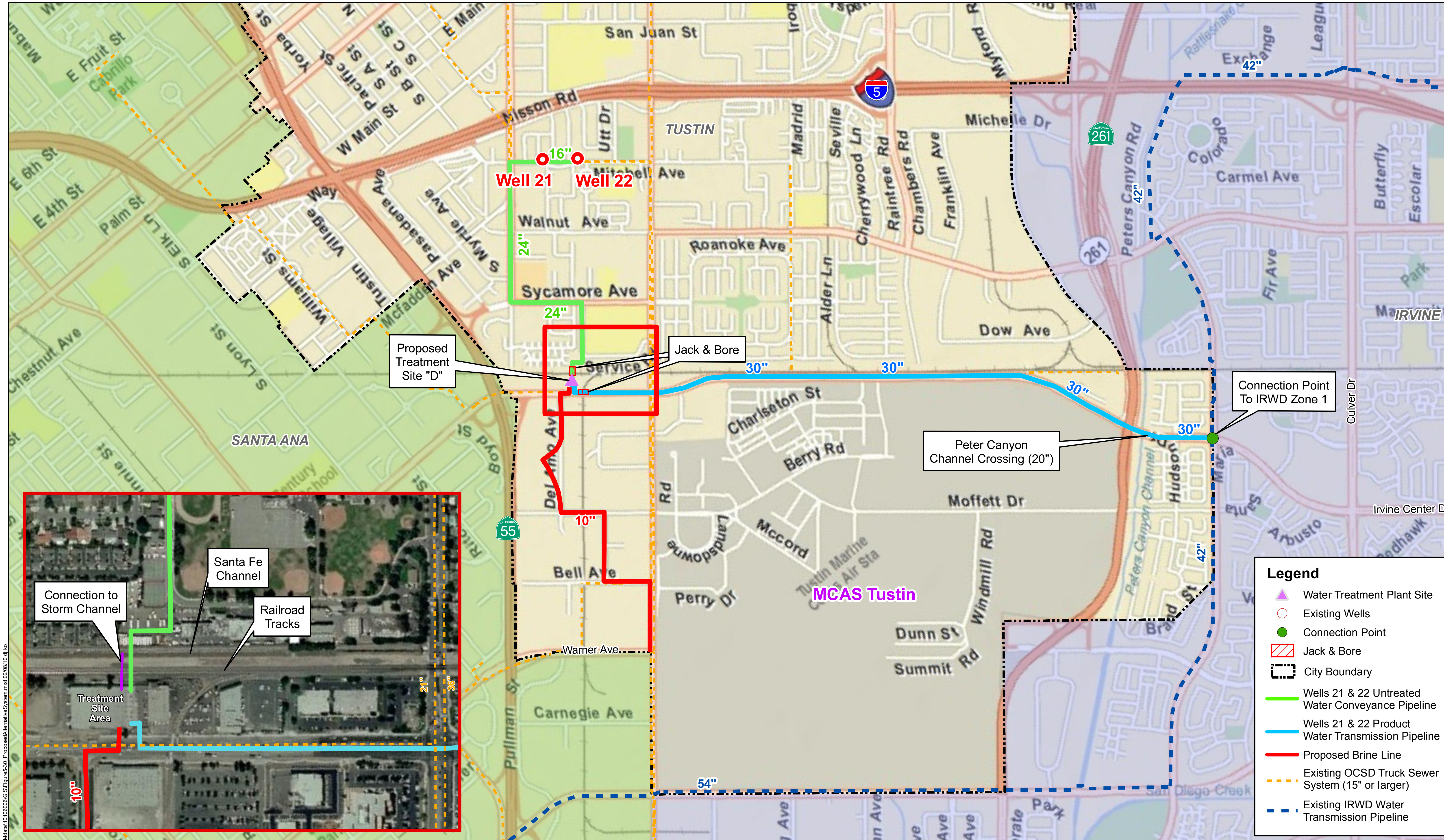
- OCSD Special Purpose Discharge Permit
- OCSD Trunk Sewer Connection Permit
- City of Tustin Encroachment Permit

## 5.7 Alternative Project

An Alternative Project is shown on Figure 5-31. The Alternative Project increases the diameter of the Product Water Pipeline and Brine Disposal Pipeline to allow capacity for Wells 21 and 22 system expansions. The expanded facilities sizing is based on an assumed two additional wells at 2,000 gpm each feeding the treatment plant with an overall treatment recovery rate (including untreated bypass) of 89 percent. This expansion impacts the Base Project facilities as described below:

- Product Water Pipeline—Entire length of product water pipeline upsized from 24-inch to 30-inch diameter, except Peters Canyon Bridge crossing, which remains 20-inch diameter.
- Brine Disposal Pipeline—Increased pipeline diameter from 8-inch to 10-inch.

Much of the cost of a new pipeline is in the construction and installation. An incremental cost adder is associated with upsizing of a pipeline due to a slightly wider and deeper trench and pipe material cost. The larger product water and brine disposal pipelines associated with this alternative project ensure that the Wells 21 and 22 system can be easily expanded without the added cost of additional pipelines required on the treated water side of the plant. Several new IRWD groundwater wells, in the area of the Edinger Avenue treatment plant site, are in the early planning stages and, if treatment is required, could be routed to this facility. Due to the unknown quantity and quality of the groundwater from future wells, ample space will be allocated at the treatment plant site to accommodate additional conveyance, treatment and storage for the assumed additional well flow.




RBF CONSULTING GIS

Source: ESRI Online

0 0.5 1
Miles

IRVINE RANCH WATER DISTRICT

# IRWD Wells 21 & 22
## Figure 5-31: Alternative Project

This page left intentionally blank.

# Section 6

# Cost Evaluation

RBF CONSULTING

This page left intentionally blank.

# Section 6 Cost Evaluation

## 6.1 General

The purpose of this section is to discuss capital and O&M cost factors and assumptions used to develop Wells 21 and 22 project cost estimates. Opinion of costs for both capital and O&M are also included, and were based upon the project components and features generally described in Sections 4 and 5 of this report. All costs presented herein are described in February 2010 dollars (ENR index = 8660).

The capital cost estimates herein approximate Class 4 budget estimates as defined by the Association for the Advancement of Cost Engineering (AACE), with associated accuracy of -15 to +30 percent. Class 4 level estimates are intended for study or feasibility purposes. These estimates are based upon the Engineer's perception of current conditions in the project area and are subject to change as variances in cost of labor, materials, equipment, services provided by others or economic conditions occur.

## 6.2 Opinion of Probable Costs

### 6.2.1 Capital Costs

Table 6-1 provides a summary of the estimated capital costs for the Wells 21 and 22 Base and Alternative Project as described in Sections 4 and 5. A breakdown of the estimated capital costs for the Base and Alternative Projects are shown in Tables 6-2 and 6-3, respectively. Detailed estimates are included in Appendix C. These costs are based on recent project bids, manufacturer quotes, BNI cost estimating references for material and labor, and record data. The estimates provided reflect general design and construction conditions. The derivation of the capital cost for each component of the project including well head equipping, pipelines and treatment plant is discussed below.

**Table 6-1 Summary of Estimated Capital Costs**

| Cost Item | Project Alternative | |
|---|---|---|
| | Wells 21/22 Base Project | Alternative Project |
| Wells 21/22 Wellhead Equipping | $2,770,000 | $2,770,000 |
| Raw Water Conveyance Pipeline | $2,080,000 | $2,080,000 |
| Water Treatment Plant | $20,860,000 | $20,860,000 |
| Finished Water Pipeline | $3,310,000 | $4,400,000 |
| Brine Disposal Pipeline | $793,000 | $793,000 |
| *Subtotal Construction Cost* | $29,810,000 | $30,900,000 |
| *Contingincies @ 20%* | $5,960,000 | $6,180,000 |
| *Preliminary Design* | $1,052,000 | $1,052,000 |
| *Engineering* | $4,313,500 | $4,510,000 |
| District Costs | $1,910,000 | $1,910,000 |
| **Total Estimated Capital Cost** | **$43,050,000** | **$44,550,000** |

**TABLE 6-2**
IRVINE RANCH WATER DISTRICT
WELLS 21 AND 22 PRELIMINARY DESIGN REPORT
ESTIMATE OF CAPITAL COSTS - BASE PROJECT

| Item No. | Item Description | Total Price[1] |
|---|---|---|
| **1** | **Wells 21 and 22 Wellhead Equipping** | |
| | **Well 21** | |
| ***1.1*** | ***General*** [2,3] | ***$81,500*** |
| ***1.2*** | ***Site Work*** | ***$160,950*** |
| ***1.3*** | ***Mechanical*** [4,5] | ***$633,300*** |
| ***1.4*** | ***Electrical*** | ***$750,100*** |
| | **Subtotal Well 21 - Wellhead Equipping** | **$1,625,850** |
| | **Well 22** | |
| ***1.5*** | ***General*** [2] | ***$57,000*** |
| ***1.6*** | ***Site Work*** | ***$152,700*** |
| ***1.7*** | ***Mechanical*** [4,5] | ***$515,200*** |
| ***1.8*** | ***Electrical*** | ***$417,900*** |
| | **Subtotal Well 22 - Wellhead Equipping** | **$1,142,800** |
| | **ITEM NO. 1 TOTAL** | **$2,770,000** |
| **2** | **Untreated Groundwater Conveyance Piping** | |
| ***2.1*** | ***General*** [2,3] | ***$107,500*** |
| ***2.2*** | ***Pipeline Construction*** [7,8,9] | ***$1,724,250*** |
| ***2.3*** | ***Pipeline Appurtenances*** | ***$249,600*** |
| | **ITEM NO 2 TOTAL - UNTREATED GROUNDWATER CONVEYANCE PIPING** | **$2,080,000** |
| **3** | **4.4 MGD Water Treatment Plant** | |
| ***3.1*** | ***General*** [2,3] | ***$1,045,000*** |
| ***3.2*** | ***Land Acquisition*** | ***$4,300,000*** |
| ***3.3*** | ***Treatment Plant*** [6] | ***$11,208,000*** |
| ***3.4*** | ***Electrical, Instrumentation and Control*** [10] | ***$2,802,000*** |
| ***3.5*** | ***Site Work*** [11,12] | ***$1,507,500*** |
| | **ITEM NO 3 TOTAL - MEMBRANE TREATMENT FACILITY** | **$20,860,000** |
| **4** | **Product Water Pipeline** | |
| ***4.1*** | ***General*** [2,3] | ***$160,000*** |
| ***4.2*** | ***Pipeline Construction*** [7,8,9] | ***$2,961,000*** |
| ***4.3*** | ***Pipeline Appurtenances*** | ***$191,750*** |
| | **ITEM NO 4 TOTAL - PRODUCT WATER PIPELINE** | **$3,310,000** |
| **5** | **Brine Disposal Pipeline**[9] | |
| ***5.1*** | ***General*** [2,3] | ***$38,000*** |
| ***5.2*** | ***Construction of New Brine Disposal Facilities*** | ***$755,000*** |
| | **ITEM NO 5 TOTAL - BRINE DISPOSAL PIPELINE** | **$793,000** |
| | ***Subtotal Items 1-5*** | $29,810,000 |
| | ***Contingencies @ 20%*** | $5,960,000 |
| | ***Preliminary Design*** [15] | $1,052,000 |
| | ***Engineering*** [14] | $4,313,500 |
| **6** | **District Costs** [13] | |
| ***6.1*** | ***Pre-Construction Phase*** | ***$682,789*** |
| ***6.2*** | ***Construction Phase*** | ***$1,160,587*** |
| ***6.3*** | ***Other Direct Costs*** | ***$64,575*** |
| | **ITEM NO 6 TOTAL - DISTRICT COSTS** | **$1,910,000** |
| | ***TOTAL ESTIMATE OF CAPITAL COST*** | **$43,050,000** |

**Notes:**

1 Capital cost are Class 4 Estimates as defined by AACEI with estimated -15% to +30 range of accuracy
2 Includes Mobilization and Demobilization estimated at approximately 4% of total construction costs
3 Includes Bonding and insurance assumed at approximately 1% of total construction costs
4 Mechanical equipment based on preliminary design in Section 5 of this report
5 Includes all on-site well equipment and off-site piping to Untreated Water Conveyance Pipeline connection in Mitchell Ave
6 Assumes RO membrane treatment facility
7 Breakdown of Pipeline construction costs elements given in Appendix B
8 Pipeline construction includes traffic control and pavement replacement, where applicable
9 Assumes Sewer Trench section per IRWD standard drawing and specifications
10 Electrical assumed at 25% of total treatment plant
11 Site Work includes general site improvements assumed at 5% of total treatment plant cost
12 Site Mechanical included in site work and assumed at 20% of total treatment plant cost (minus RO System and Building cost)
13 Based on Cost Breakdown Prepared by DDB Engineering for the April 2009 Request for Title XVI Funding
14 Engineering (preliminary design plus engineering) estimated at 15% of total construction cost including contingencies
15 Based on Budget for preparation and completion of Preliminary Design Report

**TABLE 6-3**
IRVINE RANCH WATER DISTRICT
WELLS 21 AND 22 PRELIMINARY DESIGN REPORT
ESTIMATE OF CAPITAL COSTS - ALTERNATIVE PROJECT

| Item No. | Item Description | Total Price[1] |
|---|---|---|
| 1 | **Wells 21 and 22 Wellhead Equipping** | |
| | **Well 21** | |
| 1.1 | General [2,3] | $81,500 |
| 1.2 | Site Work | $160,950 |
| 1.3 | Mechanical [4,5] | $633,300 |
| 1.4 | Electrical | $750,100 |
| | **Subtotal Well 21 - Wellhead Equipping** | **$1,625,850** |
| | **Well 22** | |
| 1.5 | General [2] | $57,000 |
| 1.6 | Site Work | $152,700 |
| 1.7 | Mechanical [4,5] | $515,200 |
| 1.8 | Electrical | $417,900 |
| | **Subtotal Well 22 - Wellhead Equipping** | **$1,142,800** |
| | **ITEM NO. 1 TOTAL** | **$2,770,000** |
| 2 | **Untreated Groundwater Conveyance Piping** | |
| 2.1 | General [2,3] | $107,500 |
| 2.2 | Pipeline Construction [7,8,9] | $1,724,250 |
| 2.3 | Pipeline Appurtenances | $249,600 |
| | **ITEM NO 2 TOTAL - UNTREATED GROUNDWATER CONVEYANCE PIPING** | **$2,080,000** |
| 3 | **4.4 MGD Water Treatment Plant** | |
| 3.1 | General [2,3] | $1,045,000 |
| 3.2 | Land Acquisition | $4,300,000 |
| 3.3 | Treatment Plant [6] | $11,208,000 |
| 3.4 | Electrical, Instrumentation and Control [10] | $2,802,000 |
| 3.5 | Site Work [11,12] | $1,507,500 |
| | **ITEM NO 3 TOTAL - MEMBRANE TREATMENT FACILITY** | **$20,860,000** |
| 4 | **Product Water Pipeline** | |
| 4.1 | General [2,3] | $215,000 |
| 4.2 | Pipeline Construction [7,8,9] | $3,945,000 |
| 4.3 | Pipeline Appurtenances | $236,750 |
| | **ITEM NO 4 TOTAL - PRODUCT WATER PIPELINE** | **$4,400,000** |
| 5 | **Brine Disposal Pipeline[9]** | |
| 5.1 | General [2,3] | $38,000 |
| 5.2 | Construction of New Brine Disposal Facilities | $755,000 |
| | **ITEM NO 5 TOTAL - BRINE DISPOSAL PIPELINE** | **$793,000** |
| | *Subtotal Items 1-5* | $30,900,000 |
| | *Contingencies @ 20%* | $6,180,000 |
| | *Preliminary Design [15]* | $1,052,000 |
| | *Engineering [14]* | $4,510,000 |
| 6 | **District Costs [13]** | |
| 6.1 | Pre-Construction Phase | $682,789 |
| 6.2 | Construction Phase | $1,160,587 |
| 6.3 | Other Direct Costs | $64,575 |
| | **ITEM NO 6 TOTAL - DISTRICT COSTS** | **$1,910,000** |
| | ***TOTAL ESTIMATE OF CAPITAL COST*** | **$44,550,000** |

**Notes:**

1 Capital cost are Class 4 Estimates as defined by AACEI with estimated -15% to +30 range of accuracy
2 Includes Mobilization and Demobilization estimated at approximately 4% of total construction costs
3 Includes Bonding and insurance assumed at approximately 1% of total construction costs
4 Mechanical equipment based on preliminary design in Section 5 of this report
5 Includes all on-site well equipment and off-site piping to Untreated Water Conveyance Pipeline connection in Mitchell Ave
6 Assumes RO membrane treatment facility
7 Breakdown of Pipeline construction costs elements given in Appendix B
8 Pipeline construction includes traffic control and pavement replacement, where applicable
9 Assumes Sewer Trench section per IRWD standard drawing and specifications
10 Electrical assumed at 25% of total treatment plant
11 Site Work includes general site improvements assumed at 5% of total treatment plant cost
12 Site Mechanical included in site work and assumed at 20% of total treatment plant cost (minus RO System and Building cost)
13 Based on Cost Breakdown Prepared by DDB Engineering for the April 2009 Request for Title XVI Funding
14 Engineering (preliminary design plus engineering) estimated at 15% of total construction cost including contingencies
15 Based on budget for preparation and completion of the Preliminary Design Report

#### 6.2.1.1 Well 21 and 22 Wellhead Equipping

Well equipping costs were based upon unit costs applied to preliminary motor and facilities sizing calculated in Section 5. The resulting unit costs were compared with costs obtained through first-hand experience of the engineering team on recent projects. The recent projects either bid or constructed that were used for cost comparison are as follows:

**Table 6-4 Comparison Well Projects**

| Groundwater Well Project ID | Construction Type | Bids Received/ Construction Status or Completion |
|---|---|---|
| Ventura County Well #4 | Well Equipping | March 5, 2009/75% Complete |
| Buena Park Well #2 | Well Drilling and Equipping | January 2006/January 2008 |
| Santa Ana Well #40 | Well Drilling and Equipping | January 2006/November 2007 |
| Santa Ana Well #41 | Well Drilling and Equipping | January 2006/November 2007 |
| Yorba Linda Well #19 | Well Drilling and Equipping | January 2006/October 2007 |
| Westminster Well #75a | Well Drilling and Equipping | January 2006/January 2008 |
| Garden Grove Well #30 | Well Drilling and Equipping | January 2006/November 2007 |
| Chino II Desalter Wells 1-9 | Well Drilling and Equipping | July 2004/April 2005 |

Costs assumed that both wells would be equipped with a submersible pump and motor, requiring no well building. Costs assumed that pump-to-waste will be disposed of in the adjacent storm drain located at the well sites.

#### 6.2.1.2 Pipelines

The unit costs for pipelines were developed based upon a compilation of unit pipe prices obtained from manufacturers and bids received for similar pipeline installation projects in the region. These were used to develop a cost per linear foot for conventional construction of each pipeline. These conventional costs were adjusted to account for areas with difficult construction, traffic control, or other associated variances. Unit costs were also estimated for pavement removal and replacement, and jack and bore. A breakdown of these costs by pipe size is included in Appendix C.

The unit costs were applied to the estimated lengths (pipeline, paving, and jack and bore) determined by the alignment analysis, and the pipeline diameters determined by the hydraulics analysis.

6.2.1.3 Treatment Plant

Treatment plant costs were based on the membrane plant preliminary design described in Section 5. This cost estimate is based upon the Engineer's perception of current conditions in the project area and is subject to change as variances in cost of labor, materials, equipment, services provided by others or economic conditions occur.

Capital costs were developed using engineering judgment and quotes provided for Carollo Engineers recent projects. Reference projects include:

- Chino Basin Desalting Authority Chino Desalter Phase 3, November 2008 (designed and bid). A 10.5 mgd expansion of the Chino Desalter adding new RO trains, decarbonator, onsite storage tanks, and chemical treatment similar to the proposed IRWD NF-RO plant.

- California Department of Corrections and Rehabilitation Deuel Vocational Institution Reverse Osmosis Water Treatment Plant and Brine Disposal System Design, 2005 (constructed). 0.8 mgd RO treatment plant and zero liquid discharge plant in Northern California.

6.2.1.4 Capital Cost

Construction costs have been converted to capital costs by applying the following costs:

- Contingencies—20 percent of construction costs.

- District costs for legal and admin.

- The cost for Engineering at 15 percent of construction costs to include the following items:
  - Preliminary Design
  - Construction Management/Survey/Materials Testing.
  - Preparation of Plans, Specifications, and Estimates.
  - Permit Fees—Estimated based upon a preliminary assessment of permit requirements described in Section 5.
  - Engineering Support during Construction.
- Treatment Plant Site Land Acquisition—The cost to acquire the Edinger Avenue Treatment Plant site is included in the capital cost estimate at a total price of $4.3 million.

### 6.2.2 O&M Costs

Operations and maintenance (O&M) costs have been developed from several data sources, previous projects and engineering calculations. A detailed breakdown of O&M costs have been included in Appendix C. O&M values for each of the project components are described below:

#### 6.2.2.1 Treatment Plant

Operations and maintenance assumptions and costs are presented in Tables 6-5 and 6-6, respectively. Costs were developed based on the operating assumptions outlined in Section 5. Operations cost unit values were based on recent estimates for:

- Power
- Chemicals

- Membranes
- Cartridge filters

Cost assumptions were made for:

- Miscellaneous equipment and building maintenance
- Labor
- Laboratory sample analysis
- Miscellaneous power

Chemical usage and power consumption were developed using data from hydraulic models, the Rothberg, Tamburini & Winsor Model for Corrosion Control and Process Chemistry, and the RO membrane manufacturer's projection software.

The costs used in developing O&M Costs are presented in Table 6-5. These costs are based on price quotes provided by suppliers to Carollo Engineers for projects within the 2008/2009 years. Suppliers include Brenntag Chemicals and Basic Chemical Solutions. The O&M cost summary is presented in Table 6-6.

**Table 6-5 Treatment Plant Operation and Maintenance Cost Assumptions**

| | |
|---|---|
| **Chemicals** | |
| Sulfuric Acid ($/lb): | 0.33 |
| Scale Inhibitor ($/lb) | 0.95 |
| Caustic Soda ($/lb): | 0.20 |
| Sodium Hypochlorite ($/lb) | 0.35 |
| Aqua Ammonia ($/lb) | 0.32 |
| Corrosion Inhibitor ($/lb) | 1.70 |
| **Consumables** | |
| Membranes ($/element): | $600 |
| Filter Cartridges ($/filter): | $12.00 |
| **Chemical Cleanings** | |
| Step 1 Cleaning Chemical Cost ($/lb)[1]: | $2.82 |
| Step 2 Cleaning Chemical Cost ($/lb)[1]: | $3.16 |
| Step 3 Cleaning Chemical Cost ($/lb)[1]: | $2.00 |
| **Other Non- Labor Costs** | |
| Power ($/kWh): | $0.125 |
| Miscellaneous Equipment and Building Maintenance ($/yr): | $45,000 |
| Laboratory Sample Analysis ($/yr): | $135,000 |
| Percentage Adder for Miscellaneous Power (%): | 2% |
| **Labor** | |
| Annual Operator Salaries ($/yr): | $64,351 |
| Fringe Percentage (%): | 40% |
| Administrative Cost Percentage (%): | 50% |

Cleaning chemical costs assume three-stage cleaning with designer chemicals targeting silica. Based on membrane systems performance projections, at the design recovery, silica is expected to be the scalant of concern. Costs include two Step 1 Step 2 and Step 3 cleaning events per year.

**Table 6-6 Treatment Plant/Miscellaneous Operation and Maintenance Cost Summary**

| Description | Base Project Annual Cost ($/yr) | Alt. Project Annual Cost ($/yr) |
|---|---|---|
| Power | $696,428 | $696,428 |
| Chemicals | | |
| Threshold Inhibitor | $33,945 | $33,945 |
| Caustic Soda | $22,868 | $22,868 |
| Sulfuric Acid | $150,920 | $150,920 |
| Sodium Hypochlorite | $12,506 | $12,506 |
| Aqua Ammonia | $8,251 | $8,251 |
| Corrosion Inhibitor | $29,000 | $29,000 |
| Step 1 RO Cleaning | $45,156 | $45,156 |
| Step 2 RO Cleaning | $25,300 | $25,300 |
| Step 3 RO Cleaning | $16,013 | $32,026 |
| Consumables | | |
| Membranes[1]: | $90,720 | $90,720 |
| Filter Cartridges: | $11,124 | $11,124 |
| Miscellaneous Maintenance Costs | $430,500 | $445,500 |
| Laboratory Costs | $150,000 | $150,000 |
| Labor[2] | $122,267 | $122,267 |
| **Annual O&M Cost ($/yr):** | **$1,845,000** | **$1,876,000** |
| **Annual O&M Cost ($/kgal):** | **$0.90** | **$0.91** |
| **Annual O&M Cost ($/AF):** | **$292** | **$297** |

1 Assumes a nominal 1% for general maintenance of the treatment facility, wells and pipelines
2 Assumes one full time operator for the treatment facility

#### 6.2.2.2 Wells

The primary O&M cost for wells is energy usage, which is projected based upon anticipated ground water surface elevation considering the effects of drawdown and system hydraulic losses. Energy costs are estimated at $0.125/kwh. Table 6-7 shows the calculation of annual energy cost for Wells 21 and 22.

**Table 6-7 Wells 21 and 22 Power Cost**

| Item | Well 21 | Well 22 |
|---|---|---|
| Flow, Q (gpm) | 3300 | 1600 |
| Total Dynamic Head, TDH (ft) | 340 | 440 |
| Pump Eff, (%) | 0.80 | 0.80 |
| Motor Eff, (%) | 0.89 | 0.89 |
| Power, HP | 398 | 250 |
| **Power, kw** | **297** | **186** |
| | | |
| Cost of Power ($/kWh) | 0.125 | 0.125 |
| Utilization Rate | 21.6 | 90% |
| Kilowatt Hours per Day (kWh/Day) | 6410 | 4022 |
| Cost per Day ($) | $801 | $503 |
| **Annual Power Cost ($)** | **$292,000** | **$183,000** |
| **Total Annual Power Cost ($)** | **$475,000** | |

6.2.2.3 Pipelines

Pipelines are largely maintenance free when new and usually require significant expenditures for maintenance only when they have reached the end of their useful life

A nominal 1 percent of the total capital costs was included in the overall O&M costs to account for miscellaneous labor and material costs associated with the pipeline, well sites, and treatment plant as shown previously in Table 6-6.

6.2.2.4 Brine Disposal

Brine disposal fees are included in the overall O&M estimate. These costs are based on OCSD Special Discharge rate schedule, which includes a base discharge rate of $768.63 per million gallons discharged plus the Supplemental Capital Facilities Capacity Charge of $0.001392 per gallon per day of discharge above the first 25,000 gallons (rounded-up to nearest $100).

6.2.2.5 OCWD Replenishment Assessment

Replenishment Assessment (RA) Costs are estimated for the project at $249 per acre foot, the current assessment value levied by OCWD.

6.2.2.6 OCWD Basin Equity Assessment

Goundwater production is managed by the OCWD through financial incentives. The framework for the financial incentives is based on OCWD establishing the Basin Production Percentage (BPP) each year. The BPP is the ratio of groundwater production to total water demands. Groundwater production above the BPP is charged a Basin Equity Assessment (BEA), which is set so that the cost of groundwater pumping above the BPP is similar to the cost of imported water. Each year, OCWD sets an allowable amount of pumping (BPP) and assesses a BEA on all water pumped above that limit. Section 38.1 of the Act provides specific criteria for exemption of the BEA, including pumping of impaired groundwater in order to protect water quality in the Basin and to clean up and contain the spread of poor-quality groundwater. IRWD has petitioned the OCWD to exempt the groundwater produced from the proposed Wells 21 and 22 from the BEA, therefore BEA is not included in these costs.

The overall estimated Operation and Maintenance cost for the Wells 21 and 22 Base Project and Alternative Project are summarized in Table 6-8 below.

**Table 6-8 Annual O&M Costs (2010 Dollars)**

| Annual Project O & M Costs | Base Project | Alternative Project |
|---|---|---|
| Well Pump Energy Costs ($/yr) | $475,000 | $475,000 |
| Treatment Plant/Miscellaneous O&M ($/yr) | $1,845,000 | $1,876,000 |
| Brine Disposal Cost ($/yr) | $621,000 | $621,000 |
| Replenishment Assessment Cost ($/yr) | $1,575,000 | $1,575,000 |

6.2.3 Economic Evaluation (Groundwater vs. Imported Water)

6.2.3.1 Methodology

Based upon the capital and O&M costs developed herein, an economic evaluation was conducted for a comparison with current and projected MWDOC imported water costs. Projected MWDOC COSTS were obtained through IRWD. Wells 21 and 22 project water annual costs were developed using the following estimates and assumptions:

- Capital cost debt repayment over 25 years at 4 percent interest.

- O&M costs are assumed to escalate annually as follows:

  - Power and Chemicals – 5.0 percent

  - Brine Disposal Fees – 7.0 percent

  - Labor and Consumables – 3.0 percent

  - OCWD Replenishment Assessment Costs – 5.0 percent (after 2011)

Capital and O&M costs were annualized and converted to $/acre-foot of water unit costs based on anticipated production volume. The wells were assumed at 90 percent utilization. Future unit costs projections to the year 2028 for the Wells 21 and 22 project were developed based on inflation assumptions and anticipated escalation of fees as described above. Two scenarios were developed to compare the unit costs to anticipated imported water cost:

- Scenario 1—No subsidies outside of the awarded BOR Title XVI/ARRA $11.6 million funding.

- Scenario 2—An additional $250 per acre-foot subsidy from MWD through the Local Resources Program (LRP) or other available sources as identified in Section 6.3.

These two scenarios provide a "bookend" high and low cost of water for the project. The wells 21 and 22 project is believed to be a candidate to receive the MWD LRP subsidy, however, the subsidy is discretionary and can range anywhere from $0 to $250 per acre-foot.

#### 6.2.3.2 Scenario 1 Results

For the Base Project and Alternative Project under Scenario 1, water rates in 2010 dollars were calculated at $1,029 and $1,049 per acre-foot, respectively. Table 6-9 summarizes the estimated 2010 water cost. Table 6-10 projects the cost of water, accounting for inflation and other variable costs, for the base project and alternative project through 2028. The projected values are shown graphically, compared to anticipated imported water rates in Figure 6-1.

It was determined, through the long term water cost projection, and as reflected in Figure 6-1, that the unit cost of water for the Wells 21 and 22 Base and Alternative Project, without additional subsidies, would be approximately 32 percent higher than MWDOC Tier 1 imported water cost at the start of the project.

Wells 21 and 22 water, under Scenario 1, is anticipated to become more competitive with imported water rates in the future. The projection shows that in 2028, the cost is estimated at just 2 percent higher than Tier 1 imported water and less expensive than Tier 2 water starting in 2021. This is due to the anticipated increases in import water cost outpacing the assumed escalation of Wells 21 and 22 water costs, as described in Section 6.

6.2.3.3 Scenario 2 Results

For the Base Project and Alternative Project under Scenario 2, water rates in 2010 dollars were calculated at $779 and $799 per acre-foot, respectively. Table 6-11 summarizes the estimated annual water cost in 2010 dollars. Table 6-12 projects the cost of water, accounting for inflation and other variable costs, for the base and alternative project through 2028. The projected values are shown graphically, compared to projected imported water rates in Figure 6-2.

It was determined, through the long term water cost projection, and as reflected in Figure ES-7, with a maximum MWD LRP subsidy of $250 per acre-foot, the cost of Wells 21 and 22 water becomes less expensive than tier 1, tier 2 and the assumed tier 1/tier 2 mix imported water from 2011 through the life of the projection.

The MWD LRP program will not subsidize water that is cheaper than imported water, therefore, the actual MWD LRP subsidy, if granted to IRWD for the Wells 21 and 22 Project, would be subject to yearly audits and adjustment. Based on the long term water cost projections in this report the maximum funding available from would range from $207/AF (in 2011) to $34/AF (in 2028) to bridge the gap between Tier 1 and Wells 21 and 22 water cost.

This page left intentionally blank

**IRVINE RANCH WATER DISTRICT**

**WELLS 21 AND 22**

**PRELIMINARY DESIGN REPORT - ANNUAL WATER COST ANALYSIS**

**Table 6-9 - Summary of Annual Water Cost (2010 Dollars)**

| Annual Costs Items | Annual Water Cost | |
|---|---|---|
| | **Base Project** | **Alternative Project** |
| Total Capital Cost | $43,050,000 | $44,550,000 |
| Title XVI Funding | $11,600,000 | $11,600,000 |
| **IRWD Capital Cost** | **$31,450,000** | **$32,950,000** |
| **1. Annual Capital Cost [2]** | $1,992,000 | $2,087,000 |
| **2. Annual Project O & M Costs** | | |
| 2.1 Well Pumping Energy Costs [3] | $475,000 | $475,000 |
| 2.2 Treatment Plant O&M[4] | $1,845,000 | $1,876,000 |
| 2.3 Brine Disposal Cost [5] | $621,000 | $621,000 |
| **4. Replenishment Assessment Cost ($/YR)** | $1,575,000 | $1,575,000 |
| **5. MWD Local Resource Program ($/AF)[7]** | $0 | $0 |
| **Subtotal** | **$6,508,000** | **$6,634,000** |
| **Water Production, (AFY)[6]** | **6,324** | **6,324** |
| **Unit Cost ($/AF)[1]** | **$1,029** | **$1,049** |

[1] Unit Cost of Water is for 2009 cost assumptions

[2] Based on 25 year loan at 2%

[3] Based on $0.125/kwh and 73% overall pump/motor efficiencies. (Wells 21/22)

[4] Based on an estimated cost of $1.03/kgal

[5] Brine Disposal Cost Estimated from OCSD Special Discharge rate schedule, does not include initial connection fee or surcharges

[6] Based on ~90% well utilization and 89% overall treatment system recovery

[7] Based on ~90% well utilization (7,113 AFY) @ $249/AF

**TABLE 6-10**

**IRVINE RANCH WATER DISTRICT**

**WELLS 21 AND 22**

**PRELIMINARY DESIGN REPORT - LONG TERM COST OF WATER PROJECTIONS**

**MWDOC Import Water Cost vs. Wells 21/22 Water Costs**

| Year | Imported Water Cost - Tier 1 ($/AF)[1] | Imported Water Cost - Tier 2 ($/AF)[1] | Imported Water Cost Tier 1 (90%)/ Tier 2 (10%) Mix, ($/AF)[5] | Base Project Water Cost Scenario 1, $/AF[2,3,4] | Alt. Project Water Cost Scenario 1, $/AF[2,3,4] | Difference from Tier 1 | | Difference from Tier 2 | | Difference from Tier 1/2 Mix | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Base Project | Alt. Project | Base Project | Alt. Project | Base Project | Alt. Project |
| 2010 | $708 | $818 | $719 | $1,029 | $1,044 | $322 | $337 | $212 | $227 | $311 | $326 |
| 2011 | $860 | $934 | $867 | $1,051 | $1,066 | $192 | $207 | $117 | $132 | $184 | $199 |
| 2012 | $903 | $981 | $911 | $1,087 | $1,103 | $185 | $200 | $107 | $122 | $177 | $192 |
| 2013 | $948 | $1,029 | $956 | $1,125 | $1,140 | $178 | $193 | $96 | $111 | $170 | $185 |
| 2014 | $995 | $1,081 | $1,004 | $1,165 | $1,180 | $170 | $185 | $84 | $99 | $162 | $177 |
| 2015 | $1,044 | $1,135 | $1,053 | $1,207 | $1,222 | $163 | $178 | $73 | $88 | $154 | $169 |
| 2016 | $1,097 | $1,191 | $1,106 | $1,251 | $1,266 | $155 | $170 | $60 | $75 | $145 | $160 |
| 2017 | $1,151 | $1,250 | $1,161 | $1,298 | $1,313 | $147 | $162 | $47 | $62 | $137 | $152 |
| 2018 | $1,208 | $1,313 | $1,219 | $1,346 | $1,361 | $138 | $153 | $33 | $48 | $128 | $143 |
| 2019 | $1,268 | $1,378 | $1,279 | $1,398 | $1,413 | $129 | $144 | $20 | $35 | $118 | $133 |
| 2020 | $1,331 | $1,447 | $1,343 | $1,452 | $1,467 | $121 | $136 | $5 | $20 | $109 | $124 |
| 2021 | $1,398 | $1,519 | $1,410 | $1,509 | $1,524 | $111 | $126 | ($10) | $5 | $99 | $114 |
| 2022 | $1,468 | $1,595 | $1,480 | $1,569 | $1,584 | $101 | $116 | ($26) | ($11) | $88 | $103 |
| 2023 | $1,541 | $1,675 | $1,554 | $1,632 | $1,647 | $91 | $106 | ($43) | ($28) | $77 | $92 |
| 2024 | $1,618 | $1,758 | $1,632 | $1,698 | $1,713 | $80 | $95 | ($60) | ($45) | $66 | $81 |
| 2025 | $1,699 | $1,846 | $1,714 | $1,768 | $1,783 | $69 | $84 | ($78) | ($63) | $54 | $69 |
| 2026 | $1,784 | $1,938 | $1,799 | $1,842 | $1,857 | $58 | $73 | ($97) | ($82) | $42 | $57 |
| 2027 | $1,873 | $2,035 | $1,889 | $1,920 | $1,935 | $46 | $61 | ($116) | ($101) | $30 | $45 |
| 2028 | $1,967 | $2,137 | $1,984 | $2,001 | $2,016 | $34 | $49 | ($136) | ($121) | $17 | $32 |

1 Based on IRWD finance department estimates for imported water rates through 2020. Assumes 5% escalation after 2020

2 Calculated based on project cost estimates and assumed well utilization of 90%

3 Based on O&M costs summarized in Section 6

4 Assumes an average annual escalation on Power, Chemicals, Labor, Brine Disposal, OCWD RA as described in Section 6.2.3 of the Wells 21 and 22 PDR

5 Assumes a mix consisting of 90-percent Tier 1 and 10-percent Tier 2 imported water

**FIGURE 6-1**
**Cost of Water Comparison - Scenario 1**
**Imported Water (MWDOC)**
**vs.**
**Wells 21/22 (With Title XVI Funding, Without MWD LRP Funding)**

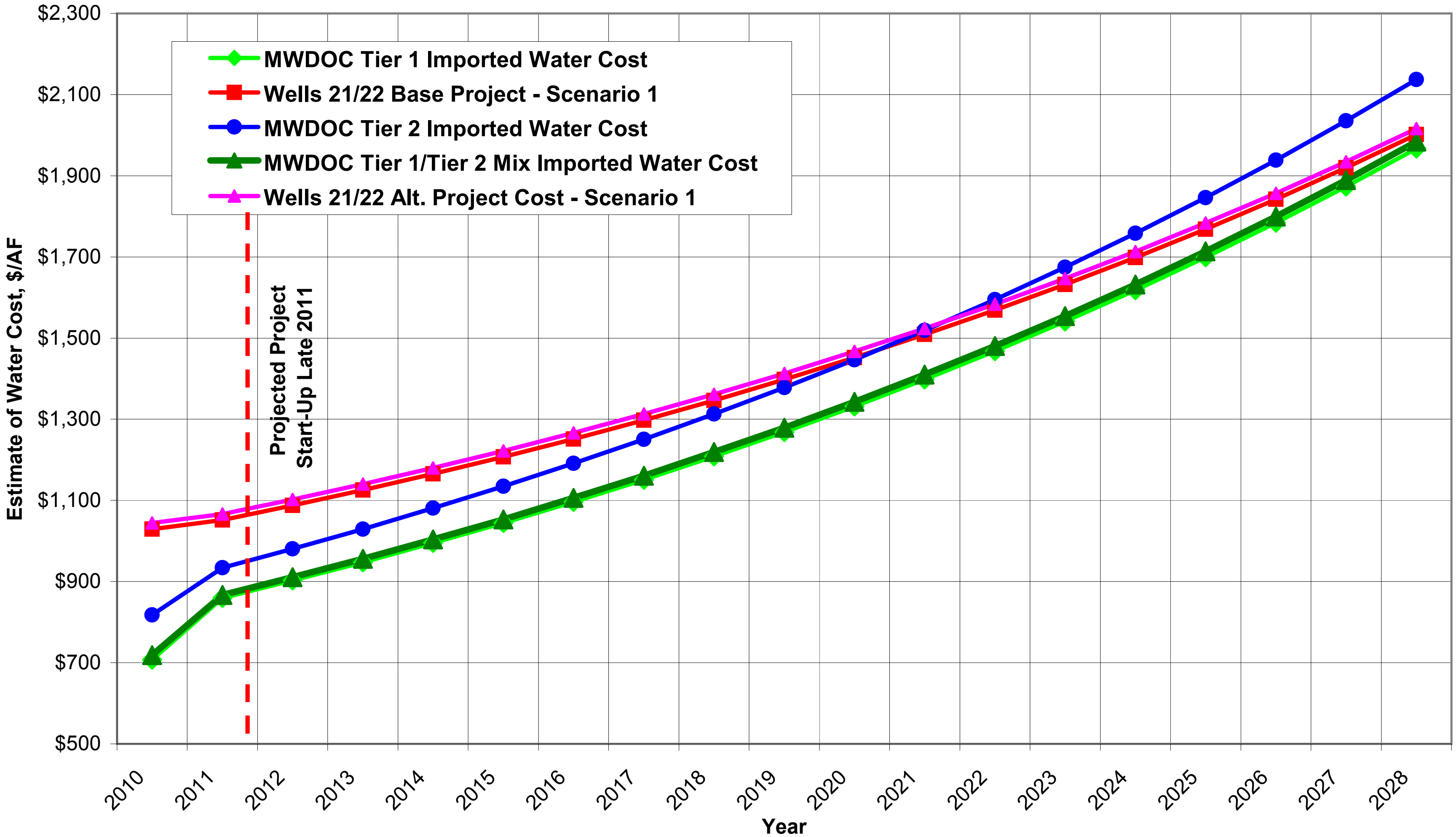


**TABLE 6-11**
**IRVINE RANCH WATER DISTRICT**
**WELLS 21 AND 22**
**PRELIMINARY DESIGN REPORT - ANNUAL WATER COST ANALYSIS**

**Summary of Annual Water Cost (2010 Dollars) with MWD LRP Subsidy**

| Annual Costs Items | Annual Water Cost | |
|---|---|---|
| | Base Project | Alternative Project |
| Total Capital Cost | $43,050,000 | $44,550,000 |
| Title XVI Funding | $11,600,000 | $11,600,000 |
| **IRWD Capital Cost** | **$31,450,000** | **$32,950,000** |
| **1. Annual Capital Cost [2]** | $1,992,056 | $2,087,067 |
| **2. Annual Project O & M Costs** | | |
| 2.1 Well Pumping Energy Costs [3] | $475,000 | $475,000 |
| 2.2 Treatment Plant O&M[4] | $1,845,000 | $1,876,000 |
| 2.3 Brine Disposal Cost [5] | $621,000 | $621,000 |
| **4. Replenishment Assessment Cost ($/YR)** | $1,575,000 | $1,575,000 |
| **5. MWD Local Resource Program ($/AF)** | $250 | $250 |
| **Subtotal** | **$6,508,056** | **$6,634,067** |
| **Water Production, (AFY)[6]** | **6,324** | **6,324** |
| **Unit Cost ($/AF)[1]** | **$779** | **$799** |

[1] Unit Cost of Water is for 2009 cost assumptions

[2] Based on 25 year loan at 2%

[3] Based on $0.125/kwh and 73% overall pump/motor efficiencies. (Wells 21/22)

[4] Based on an estimated cost of $1.03/kgal

[5] Brine Disposal Cost Estimated from OCSD Special Discharge rate schedule, does not include initial connection fee or surcharges

{6] Based on ~90% well utilization, 85% RO Recovery and 26% Untreated Water Bypass

**TABLE 6-12**
**IRVINE RANCH WATER DISTRICT**
**WELLS 21 AND 22**
**PRELIMINARY DESIGN REPORT - LONG TERM COST OF WATER PROJECTIONS**

**MWDOC Import Water Cost vs. Wells 21/22 Water Costs**

| Year | Imported Water Cost - Tier 1 ($/AF)[1] | Imported Water Cost - Tier 2 ($/AF)[1] | Imported Water Cost Tier 1/ Tier 2 Mix ($/AF)[5] | Base Project Water Cost Scenario 2, $/AF[2,3,4] | Alt. Project Water Cost Scenario 2, $/AF[2,3,5] | Difference from Tier 1 | | Difference from Tier 2 | | Difference from Tier 1/2 Mix | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Base Project | Alt. Project | Base Project | Alt. Project | Base Project | Alt. Project |
| 2010 | $708 | $818 | $719 | $779 | $794 | $72 | $87 | ($38) | ($23) | $61 | $76 |
| 2011 | $860 | $934 | $867 | $801 | $816 | ($58) | ($43) | ($133) | ($118) | ($66) | ($51) |
| 2012 | $903 | $981 | $911 | $837 | $853 | ($65) | ($50) | ($143) | ($128) | ($73) | ($58) |
| 2013 | $948 | $1,029 | $956 | $875 | $890 | ($72) | ($57) | ($154) | ($139) | ($80) | ($65) |
| 2014 | $995 | $1,081 | $1,004 | $915 | $930 | ($80) | ($65) | ($166) | ($151) | ($88) | ($73) |
| 2015 | $1,044 | $1,135 | $1,053 | $957 | $972 | ($87) | ($72) | ($177) | ($162) | ($96) | ($81) |
| 2016 | $1,097 | $1,191 | $1,106 | $1,001 | $1,016 | ($95) | ($80) | ($190) | ($175) | ($105) | ($90) |
| 2017 | $1,151 | $1,250 | $1,161 | $1,048 | $1,063 | ($103) | ($88) | ($203) | ($188) | ($113) | ($98) |
| 2018 | $1,208 | $1,313 | $1,219 | $1,096 | $1,111 | ($112) | ($97) | ($217) | ($202) | ($122) | ($107) |
| 2019 | $1,268 | $1,378 | $1,279 | $1,148 | $1,163 | ($121) | ($106) | ($230) | ($215) | ($132) | ($117) |
| 2020 | $1,331 | $1,447 | $1,343 | $1,202 | $1,217 | ($129) | ($114) | ($245) | ($230) | ($141) | ($126) |
| 2021 | $1,398 | $1,519 | $1,410 | $1,259 | $1,274 | ($139) | ($124) | ($260) | ($245) | ($151) | ($136) |
| 2022 | $1,468 | $1,595 | $1,480 | $1,319 | $1,334 | ($149) | ($134) | ($276) | ($261) | ($162) | ($147) |
| 2023 | $1,541 | $1,675 | $1,554 | $1,382 | $1,397 | ($159) | ($144) | ($293) | ($278) | ($173) | ($158) |
| 2024 | $1,618 | $1,758 | $1,632 | $1,448 | $1,463 | ($170) | ($155) | ($310) | ($295) | ($184) | ($169) |
| 2025 | $1,699 | $1,846 | $1,714 | $1,518 | $1,533 | ($181) | ($166) | ($328) | ($313) | ($196) | ($181) |
| 2026 | $1,784 | $1,938 | $1,799 | $1,592 | $1,607 | ($192) | ($177) | ($347) | ($332) | ($208) | ($193) |
| 2027 | $1,873 | $2,035 | $1,889 | $1,670 | $1,685 | ($204) | ($189) | ($366) | ($351) | ($220) | ($205) |
| 2028 | $1,967 | $2,137 | $1,984 | $1,751 | $1,766 | ($216) | ($201) | ($386) | ($371) | ($233) | ($218) |

1 Based on IRWD finance department estimates for imported water. Assumes 5% escalation after 2020

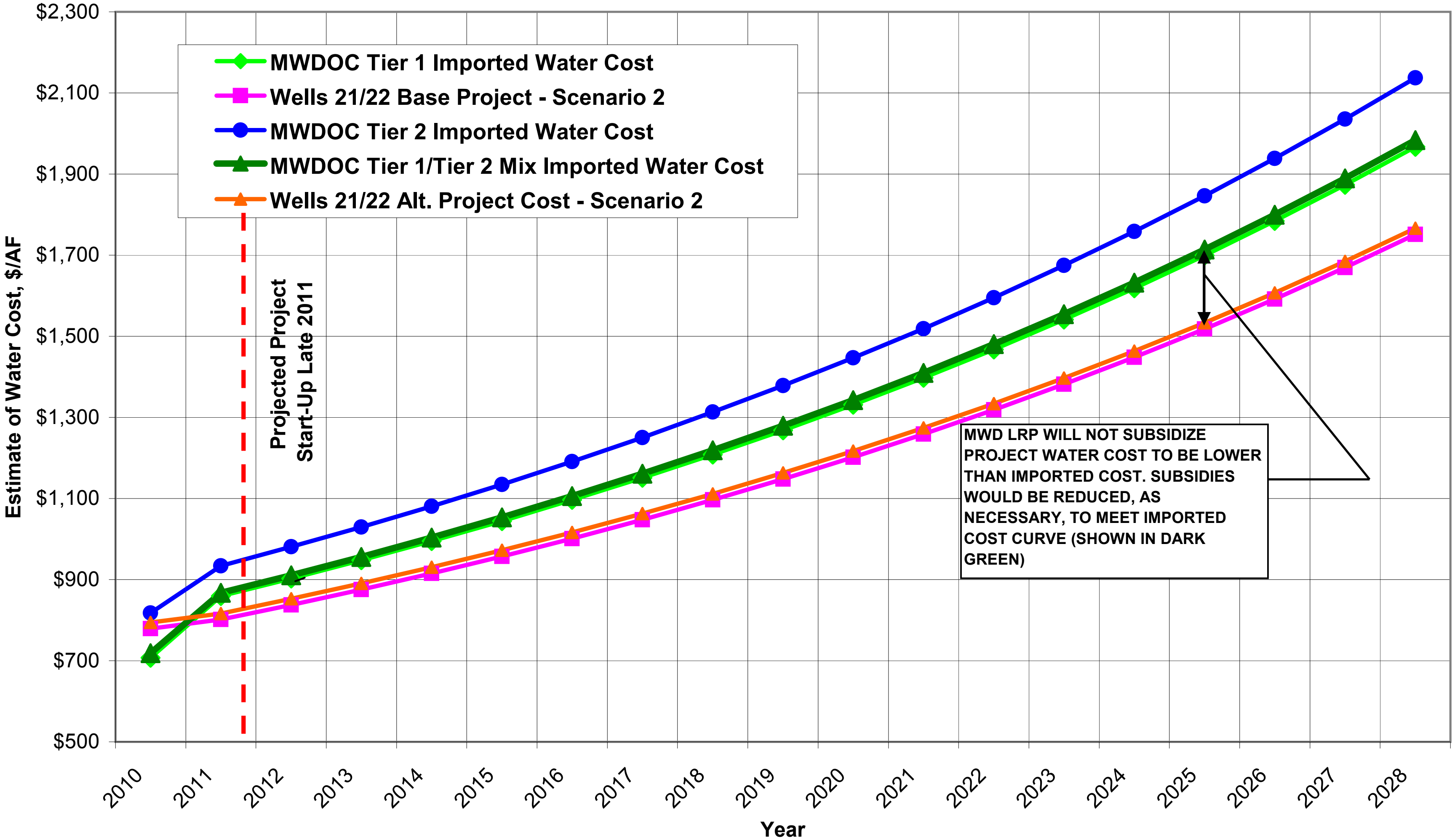
2 Calculated based on project cost estimates and assumed well utilization of 90%

3 Based on O&M costs summarized in Section 6

4 Assumes an average annual escalation on OCWD Replenishment Assessment rates of 5% after 2011

5 Assumes 90-percent of Tier 1 and 10-percent of Tier 2

**FIGURE 6-2**
**Cost of Water Comparison - Scenario 2**
**Imported Water (MWDOC)**
**vs.**
**Wells 21/22 (With Title XVI Funding and $250/AF MWD LRP Subsidy)**



## 6.3 Potential Grant Funding Sources

### 6.3.1 Grant Funding Application Status

During preparation of the Wells 21 and 22 Draft PDR, several funding sources were identified that had deadlines that were set to expire prior to anticipated completion of the report. The following applications were completed and submitted ahead of the *May 2009 Wells 21 and 22 Draft PDR* submittal:

- Safe Drinking Water Revolving Fund (SDWSRF)
- Bureau of Reclamation (BOR) Title XVI.

Due to time-sensitive requirements, the pre-application for the SDWSRF utilizing ARRA funding was submitted in late February 2009 to CDPH. The BOR Title XVI funding request for ARRA funding under the Water Reclamation and Reuse Program was submitted in early April 2009.

The SDWSRF application was denied for Wells 21 and 22.

As discussed previously, the Wells 21 and 22 project was awarded $11.6 million through BOR Title XVI utilizing ARRA funds. The contract program and implementation schedule was modified to comply with the timeline requirements associated with the ARRA funding. In addition to the schedule constraints, this funding comes with several other key requirements as identified below:

- The Contractor shall comply with the required use of American Iron, Steel, and Manufactured Goods-Section 1605 of the American Recovery and Reinvestment Act of 2009 (29 CFR 176.140). Under these terms and conditions, it is required that all iron, steel, and manufactured goods used for the construction, alteration, maintenance, or repair of a public building or public work by the Contractor are produced in the United States, unless one of the specified exemptions applies.
- To properly monitor the expenditure of ARRA funds, the Contractor, as well as the Owner, is required to identify expenditures of the federal funds associated with this project on the Schedule of

Expenditure of Federal Awards (SEFA) and the Data Collection Form (SF-SAC).

- The Contractor shall comply with the wage rate requirements of Section 1606 of the American Recovery and Reinvestment Act of 2009 (29 CFR 5.5), which requires the payment of Davis-Bacon Act (40 U.S.C. 31) wage rates to “laborers and mechanics employed by contractors and subcontractors on projects funded directly by or assisted in whole or in part by and through the Federal Government” pursuant to the Recovery Act.

### 6.3.2 Initial Grant Funding Identification

All identified funding opportunities for the IRWD Wells 21 and 22 Project were identified during the preparation of the May 2009 Draft PDR. These were available from state and federal sources only. No local funding sources are known to be available at this time. Appendix Q summarizes all funding opportunities that were identified as applicable to this project and the necessary steps and contact info to apply.

# Section 7

# Implementation Schedule

Section 7
Implementation Schedule

This page left intentionally blank.

# Section 7 Implementation Schedule

## 7.1 General

Implementation of the Wells 21 and 22 Project is proposed to be divided into two design contracts. This approach will expedite activities, which is one of the primary goals necessary in order to receive maximum Title XVI funding for the project via the Bureau of Reclamation – *Water Reclamation and Reuse Program* (Under the American Recovery and Reinvestment Act of 2009).

Table 7-1 below provides a summary of the anticipated start and completion dates for the design, bidding, and construction milestones for each of the two project programs. The detailed implementation schedule for the Treatment Plant Design-Build Project and the Wells 21 and 22 Equipping and Pipelines Design-Bid-Build Project is provided in Figure 7-1.

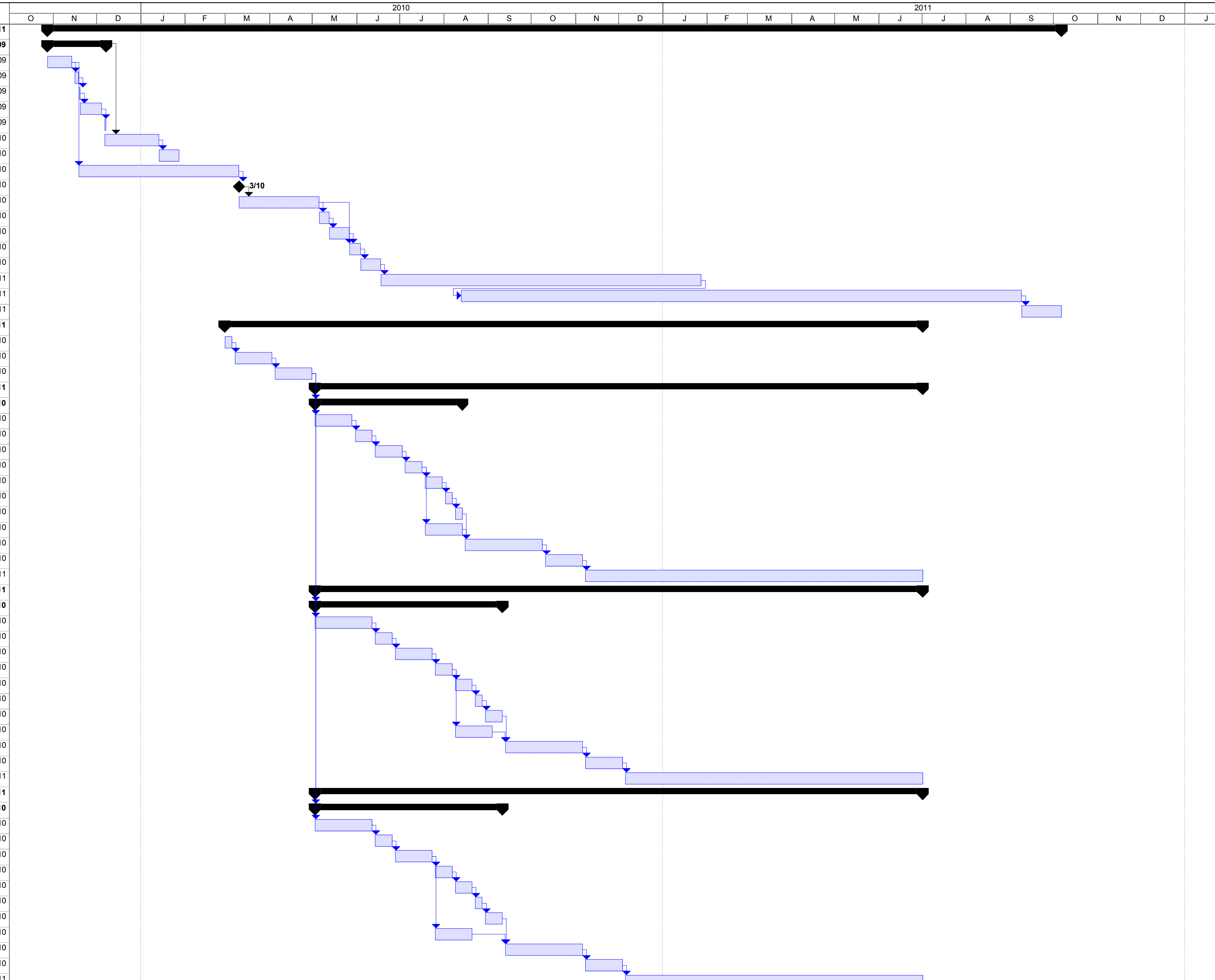
**Table 7-1 Wells 21 and 22 Implementation Schedule Summary**

| **Treatment Plant Design Build (D-B)** | **Start Date** | **Finish Date** |
|---|---|---|
| Prepare and Issue D-B RFP, Plans and Specifications | In Progress | March 10, 2010 |
| Proposal Preparation and Submittal by D-B Firms | March 10, 2010 | May 5, 2010 |
| Owner D-B Proposal Review | May 5, 2010 | May 12, 2010 |
| Award D-B Contract | May 12, 2010 | June 3, 2010 |
| Negotiations/Notice of Award/Issue Notice to Proceed | June 3, 2010 | June 17, 2010 |
| Treatment Plant Design | June 18, 2010 | January 27, 2011 |
| Treatment Plant Construction/Start-Up | August 13, 2010 | September 8, 2011 |
| **Well 21 and 22 Equipping and Pipelines** | **Start Date** | **Finish Date** |
| Prepare Design RFP and Issue | In Progress | March 5, 2010 |
| Receive Design Proposals | March 5, 2010 | April 2, 2010 |
| Review Proposals, Issue Notice to Proceed for Design | April 5, 2010 | April 30, 2010 |
| Well Equipping/Pipelines Design | May 3, 2010 | September 10, 2010 |
| Well Equipping/Pipelines Advertise, Bidding & Award | September 10, 2010 | December 3, 2010 |
| Well Equipping/Pipelines Construction and Start-Up | December 3, 2010 | July 1, 2011 |

This page left intentionally blank

## Figure 7-1
## Wells 21 and 22 - Accelerated Implementation Schedule

| ID | Task Name | Duration | Start | Finish |
|---|---|---|---|---|
| 1 | **Contract 1 - Treatment Facilities (Design-Build)** | **507 days** | **Wed 10/28/09** | **Thu 10/6/11** |
| 2 | **Prepare RFQ** | **29 days** | **Wed 10/28/09** | **Mon 12/7/09** |
| 3 | RBF Submits Draft RFQ to IRWD | 13 days | Wed 10/28/09 | Fri 11/13/09 |
| 4 | IRWD to Review Draft RFQ | 3 days | Mon 11/16/09 | Wed 11/18/09 |
| 5 | RBF Revises RFQ | 1 day | Thu 11/19/09 | Thu 11/19/09 |
| 6 | IRWD Reviews and Accepts Final RFQ | 11 days | Fri 11/20/09 | Fri 12/4/09 |
| 7 | Distribution of RFQ to D-B Firms | 1 day | Mon 12/7/09 | Mon 12/7/09 |
| 8 | SOQ Preparation and Submittal by D-B Firms | 28 days | Mon 12/7/09 | Wed 1/13/10 |
| 9 | Review SOQ's and Short List D-B Firms | 10 days | Thu 1/14/10 | Wed 1/27/10 |
| 10 | Prepare D-B RFP Plans and Specs | 16 wks | Thu 11/19/09 | Wed 3/10/10 |
| 11 | Issue RFP to Short Listed D-B Firms | 0 days | Wed 3/10/10 | Wed 3/10/10 |
| 12 | Proposal Preparation and Submittal by D-B Firms | 2 mons | Thu 3/11/10 | Wed 5/5/10 |
| 13 | Review, Evaluate and Rank D-B Proposals | 1 wk | Thu 5/6/10 | Wed 5/12/10 |
| 14 | Negotiate with Top Ranked D-B Firm | 2 wks | Thu 5/13/10 | Wed 5/26/10 |
| 15 | Award of Contract to D-B Firm | 6 days | Thu 5/27/10 | Thu 6/3/10 |
| 16 | Notice of Award/Notice to Proceed | 2 wks | Fri 6/4/10 | Thu 6/17/10 |
| 17 | Design | 8 mons | Fri 6/18/10 | Thu 1/27/11 |
| 18 | Construction and Start-Up | 14 mons | Fri 8/13/10 | Thu 9/8/11 |
| 19 | Commissioning | 1 mon | Fri 9/9/11 | Thu 10/6/11 |
| 20 | **Contract 2 - Well Equipping and Pipelines (Design-Bid-Build)** | **350 days** | **Mon 3/1/10** | **Fri 7/1/11** |
| 21 | Issue RFP for Well Equipping and Pipelines Design | 1 wk | Mon 3/1/10 | Fri 3/5/10 |
| 22 | Receive Consultant Proposals | 4 wks | Mon 3/8/10 | Fri 4/2/10 |
| 23 | Design Contract Award | 4 wks | Mon 4/5/10 | Fri 4/30/10 |
| 24 | **Wells 21 & 22 Wellhead Equipping** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 25 | **Final Design** | **75 days** | **Mon 5/3/10** | **Fri 8/13/10** |
| 26 | Prepare 30% Design Submittal | 4 wks | Mon 5/3/10 | Fri 5/28/10 |
| 27 | IRWD Review | 2 wks | Mon 5/31/10 | Fri 6/11/10 |
| 28 | Prepare 70% Design Submittal | 3 wks | Mon 6/14/10 | Fri 7/2/10 |
| 29 | IRWD Review | 2 wks | Mon 7/5/10 | Fri 7/16/10 |
| 30 | Prepare 100% Design Submittal | 2 wks | Mon 7/19/10 | Fri 7/30/10 |
| 31 | IRWD Review | 1 wk | Mon 8/2/10 | Fri 8/6/10 |
| 32 | Prepare/Issue Final Bid Documents | 1 wk | Mon 8/9/10 | Fri 8/13/10 |
| 33 | Complete Permits Acquisition | 1 mon | Mon 7/19/10 | Fri 8/13/10 |
| 34 | Advertising & Bidding | 8 wks | Mon 8/16/10 | Fri 10/8/10 |
| 35 | Contract Award | 4 wks | Mon 10/11/10 | Fri 11/5/10 |
| 36 | Construction, Start-Up and Testing | 8.5 mons | Mon 11/8/10 | Fri 7/1/11 |
| 37 | **Raw Water Conveyance Pipeline** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 38 | **Final Design** | **95 days** | **Mon 5/3/10** | **Fri 9/10/10** |
| 39 | Prepare 30% Design Submittal | 6 wks | Mon 5/3/10 | Fri 6/11/10 |
| 40 | IRWD Review | 2 wks | Mon 6/14/10 | Fri 6/25/10 |
| 41 | Prepare 70% Design Submittal | 4 wks | Mon 6/28/10 | Fri 7/23/10 |
| 42 | IRWD Review | 2 wks | Mon 7/26/10 | Fri 8/6/10 |
| 43 | Prepare 100% Design Submittal | 2 wks | Mon 8/9/10 | Fri 8/20/10 |
| 44 | IRWD Review | 1 wk | Mon 8/23/10 | Fri 8/27/10 |
| 45 | Prepare/Issue Final Bid Documents | 2 wks | Mon 8/30/10 | Fri 9/10/10 |
| 46 | Complete Permits Acquisition | 1 mon | Mon 8/9/10 | Fri 9/3/10 |
| 47 | Advertising & Bidding | 8 wks | Mon 9/13/10 | Fri 11/5/10 |
| 48 | Contract Award | 4 wks | Mon 11/8/10 | Fri 12/3/10 |
| 49 | Construction, Start-Up and Testing | 7.5 mons | Mon 12/6/10 | Fri 7/1/11 |
| 50 | **Finished Water Transmission Pipeline** | **305 days** | **Mon 5/3/10** | **Fri 7/1/11** |
| 51 | **Design** | **95 days** | **Mon 5/3/10** | **Fri 9/10/10** |
| 52 | Prepare 30% Design Submittal | 6 wks | Mon 5/3/10 | Fri 6/11/10 |
| 53 | IRWD Review | 2 wks | Mon 6/14/10 | Fri 6/25/10 |
| 54 | Prepare 70% Design Submittal | 4 wks | Mon 6/28/10 | Fri 7/23/10 |
| 55 | IRWD Review | 2 wks | Mon 7/26/10 | Fri 8/6/10 |
| 56 | Prepare 100% Design Submittal | 2 wks | Mon 8/9/10 | Fri 8/20/10 |
| 57 | IRWD Review | 1 wk | Mon 8/23/10 | Fri 8/27/10 |
| 58 | Prepare/Issue Final Bid Documents | 2 wks | Mon 8/30/10 | Fri 9/10/10 |
| 59 | Complete Permits Acquisition | 1 mon | Mon 7/26/10 | Fri 8/20/10 |
| 60 | Advertising & Bidding | 8 wks | Mon 9/13/10 | Fri 11/5/10 |
| 61 | Contract Award | 4 wks | Mon 11/8/10 | Fri 12/3/10 |
| 62 | Construction, Start-Up and Testing | 7.5 mons | Mon 12/6/10 | Fri 7/1/11 |




Project: Schedule 2.17.10
Date: Mon 2/22/10

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

This page left intentionally blank.