Volume II E

Monte Vista Water District

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1.0 INTRODUCTION

1.1 Overview

The Chino Groundwater Basin (Basin) Dry-Year Yield (DYY) Program Expansion (Program Expansion) is a comprehensive water resources management program to maximize conjunctive-use opportunities in the Basin. Program Expansion details are provided in a two-volume Project Development Report (PDR). Volume I traces the development of the original DYY Program, describes the Program Expansion, and presents the technical, financial, and institutional framework within which individual projects will move forward. Volume II consists of 10 lettered sub-volumes (A-J) defining facilities to be developed by the Program Expansion's ten participating appropriators. This Volume II-E describes proposed facilities for Monte Vista Water District (MVWD). Individual chapters provide conceptual development of the ion exchange (IX), groundwater production well, aquifer storage and recovery (ASR) well, and agency interconnection facilities required for MVWD to participate in the Program Expansion. An Opinion of Probable Cost is also presented. This Introduction Chapter provides background information on the DYY Program, the Program Expansion, and the MVWD system.

1.2 Evolution of DYY Program and Program Expansion

The Program Expansion is being developed by the Chino Basin Watermaster (Watermaster) in association with the Inland Empire Utilities Agency (IEUA), Metropolitan Water District of Southern California (Metropolitan), Three Valleys Municipal Water District (TVMWD), and Western Municipal Water District (WMWD). Table 1-1 summarizes the history and evolution of the Expansion Program, which could provide an additional 17,000 acre-feet (acre-ft) of groundwater for dry-year use.

Table 1-1
Evolution of Chino Basin DYY Program Expansion*

Item	Description	Comments
Chino Basin Optimum Basin Management Program (OBMP)	Developed in response to a 1998 court ruling governing water use in the Basin (Chino Judgment). The Judgment was a continuation of a 1978 ruling providing a legal definition for the Basin and establishing a court-appointed Watermaster.	OBMP objectives are to enhance Basin water supplies, protect and enhance water quality, enhance Basin management, and provide equitable financing. Of the OBMP's nine Program Elements, three are applicable to the Expansion Program: Salt Management (7), Groundwater Storage Management (8), and Conjunctive-use (9).
DYY Program	Conjunctive-use program initiated in 2002 among Metropolitan, IEUA, Watermaster, and participating Basin appropriators. IEUA, which manages the distribution of imported water to Basin appropriators, acts as liaison between Watermaster and Metropolitan.	The Program provides for 100,000 acre-ft of water through in-lieu exchange and direct recharge of surplus Metropolitan imported supplies. Water can be "put" into and "taken" out of the Basin at a maximum rate of 25,000 acre-feet per year (afy) and 33,000 afy, respectively.
DYY Program Expansion	Expansion of 2002 DYY Program to produce up to 17,000 afy of additional groundwater for dry-year use, in-lieu of imported water.	Each of the participating appropriators will contribute a portion of the 17,000 acre-ft of additional dry-year yield or necessary "puts" into the Basin.

* Additional details are provided in PDR Volume I.

1.3 Documentation

IEUA assembled the consultant team for both the DYY Program and the Program Expansion. Both Programs have been accomplished through a series of cooperative activities working extensively with Watermaster and the Basin appropriators. From this collaboration, several reports, technical memoranda (TMs), and computer models were produced, which served as the framework of this PDR.

The PDR is organized into four volumes. Volumes I and II, prepared by Black & Veatch (B&V), provide general information on the DYY Program Expansion. Volume I presents background information on the Basin and Program operation, while Volume II presents design criteria specific to each participating agency. Volume III, the Preliminary Modeling Report prepared by Wildermuth Environmental, Inc. (WEI), presents results of a groundwater model used to evaluate the water resources impacts of the DYY Program on the Basin. Volume IV presents the California Environmental Quality Act (CEQA) documentation conducted for this project and was prepared by Tom Dodson & Associates (TDA).

1.4 Summary of Program Participants

Volume II describes the specific site requirements and design criteria for the proposed facilities required to provide the 17,000 acre-ft of additional dry-year yield. Table 1-2 lists the appropriators and the corresponding PDR volume which identifies their project-specific facilities. Construction of these facilities is required for full Program implementation.

Table 1-2
Summary of Program Participants and Facility Requirements

Agency/PDR Volume	Facility Requirements
Chino (II A)	▸ Regenerable IX treatment at existing Well Nos. 3 and 12 ▸ ASR Site at Well No. 14: Regenerable IX treatment at existing Well No. 14 and replacement of existing Chino agriculture well for injection
Chino Hills (II B)	▸ Convert existing Well No. 19 to ASR
Cucamonga Valley Water District (II C)	▸ Four new ASR wells
Jurupa Community Services District (II D)	▸ New Well No. 27 (“Galleano Well”) ▸ New Well No. 28 (“Oda Well”) ▸ New Well No. 29 (“IDI Well”)
Monte Vista Water District (II E)	▸ New ASR well and regenerable IX treatment ▸ Rehabilitate existing Well No. 2 and regenerable IX treatment ▸ Regenerable IX treatment at existing ASR Well No. 4 and Well No. 27 ▸ Conveyance facilities to deliver water from MVWD via Chino Hills to Walnut Valley Water District Service Areas
Ontario (II F)	▸ Conveyance facilities to establish interconnection with CVWD
Pomona (II G)	▸ Regenerable IX treatment at existing Reservoir No. 5 site
Upland (II H)	▸ New well in Six Basins
Three Valleys Municipal Water District (II I)	▸ Treated water pipeline from WFA WTP to Miramar WTP ▸ Turnout along Azusa-Devil Cyn Pipeline
Western Municipal Water District (II J)	▸ Conveyance facilities to establish interconnection between planned Riverside-Corona (RC) Feeder and JCSD service area ▸ Conveyance pipeline to establish interconnection between WMWD service area and Chino II Desalter

1.5 Conceptual Design Assumptions

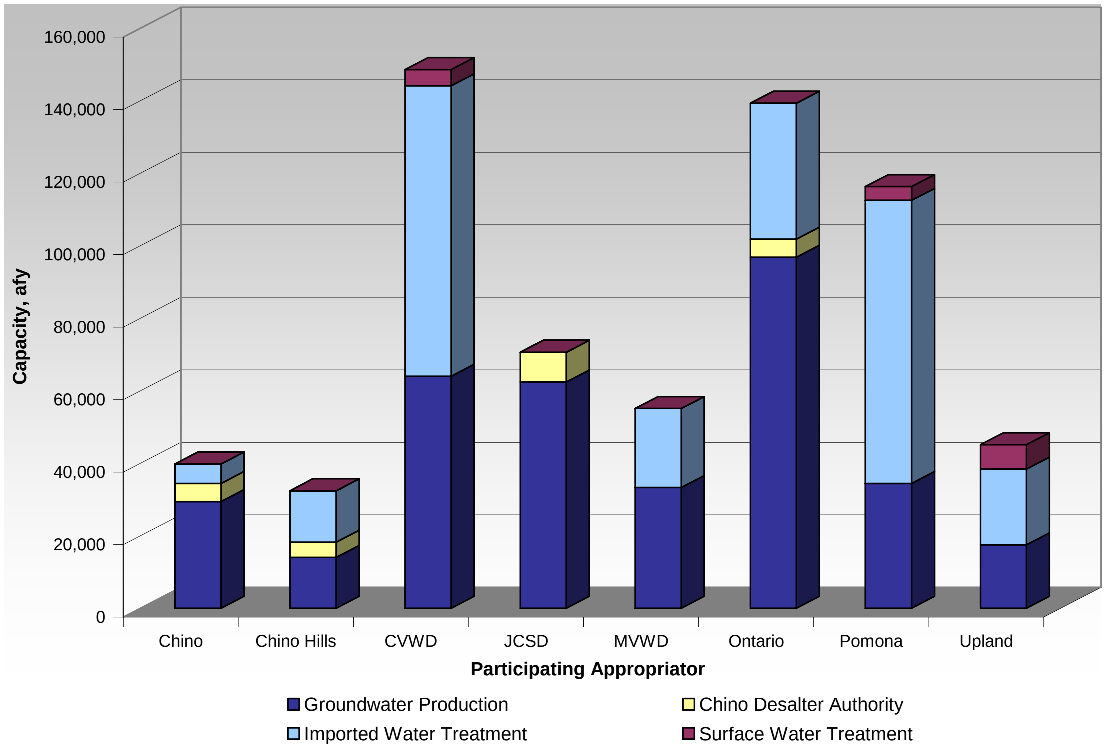
Facilities described in Volume II were designed based upon information available and using the following general design assumptions:

- Elevations were based upon United States Geological Survey (USGS) maps and maps obtained online from Google® Earth and are estimated to be accurate to within 10 percent of the actual elevation. Topographical surveys would be performed as part of the final design.
- Typical engineering calculations and assumptions were used to develop preliminary sizing for equipment and IX facilities. The final designs may vary slightly dependent upon results of the Title 22 water quality testing as well as detailed discussions with IX resin manufacturers.
- Conceptual designs assumed to not have significant permitting restrictions. Investigations of potential permit requirements for each project would be carried out during final design.
- Brine discharge to the Non-reclaimable Wastewater, or Waste, System (NRWS) was assumed to not have a significant impact on NRWS capacity. The available capacity of the NRWS would be evaluated during final design.
- Groundwater levels and flows, anticipated drawdown from well operation and location, and concentration of contaminants was based upon available data provided by WEI based upon their recent modeling efforts.
- Facilities to be constructed on agency or City property were assumed to not require additional land purchase. In addition, pipelines constructed in City or County streets were assumed to be within the right-of-way limits.
- The opinion of probable cost is intended to provide a budgetary estimate of the capital and operational costs. Detailed quantity and unit cost figures for the facilities would depend on specific manufacturer equipment and prices.

1.6 Facility Requirements

An investigation ("Asset Inventory") consisting of several meetings and site visits was conducted to determine the condition of existing facilities and production capacities of each participating appropriator. The Asset Inventory presents a comprehensive list of the facilities available for each appropriator and identifies each participating appropriator's groundwater production capabilities and imported water treatment capacity. The results of the Asset Inventory are discussed in Volume I, Appendix A. Figure 1-1 summarizes Asset Inventory results.

Figure 1-1
Water Resource Capacities for Participating Appropriators(1)(2)



Notes:
(1) Participating Appropriators include current Basin appropriators interested in participating in the DYY Program Expansion. This does not include agencies outside the Basin, such as TVMWD and WMWD.
(2) Does not include recycled water deliveries provided by IEUA.

Table 1-3 lists potential Program participants and each agency's potential "put" and/or "take" contribution. The combined "take" capacity of these agencies ranges from 15,000 to 17,000 afy. The combined "put" capacity of these agencies is approximately 12,300 to 16,800 afy of direct capacity plus Basin-wide in-lieu deliveries and surface spreading contributions.

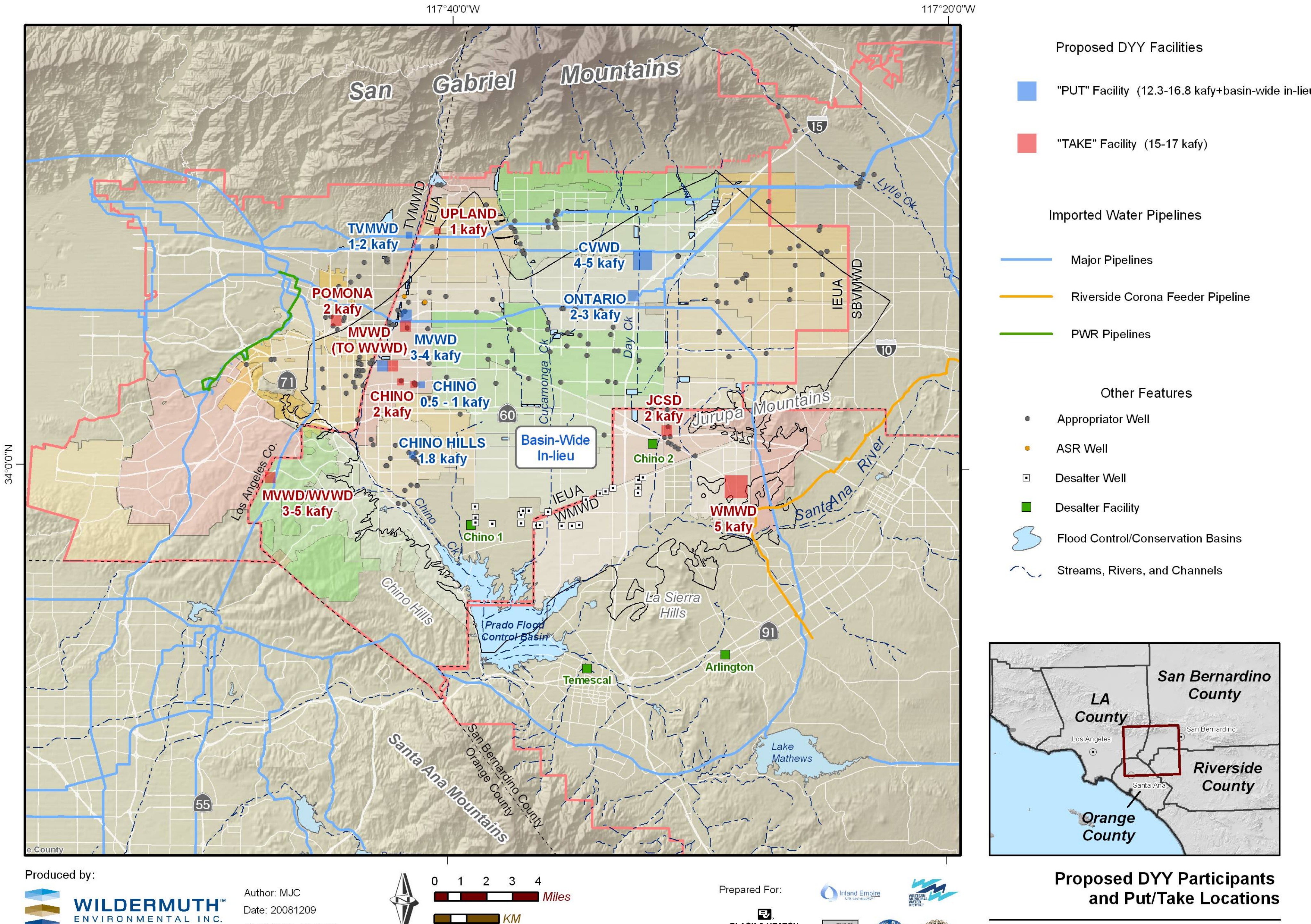
Figure 1-2 shows the locations of each agency's proposed facilities and/or locations where potential "puts" and "takes" could occur within the Basin. As the figure demonstrates, the "puts" and "takes" may be balanced on the east and west sides of the Basin. Through groundwater modeling, Program operations were evaluated to determine the potential for material physical injury to a party of the Chino Judgment or to the Chino Basin as required by the Peace Agreement, (refer to Volume III, Program Modeling Report).

Table 1-3
Summary of Initial and Expanded DYY Program Participants and Proposed Put/Take Capacities

Agency	Initial DYY Program (1)		DYY Program Expansion (2)	
	Put Capacity (afy)	Take Capacity (afy)	Put Capacity (afy) (4)	Take Capacity (afy) (6)
Chino	(3)	1,159	500-1,000	2,000
Chino Hills(5)		1,448	1,800	0
Cucamonga Valley Water District		11,353	4,000-5,000	0
Jurupa Community Services District		2,000	0	2,000
Monte Vista Water District		3,963	3,000-4,000	3,000-5,000
Ontario		8,076	2,000-3,000	0
Pomona		2,000	0	2,000
Upland		3,001	0	1,000
Three Valleys Municipal Water District		0	1,000-2,000	0
Western Municipal Water District		0	0	5,000
Total	25,000	33,000	12,300 – 16,800	15,000 – 17,000

Notes:
(1) Initial 100,000 acre-ft DYY Program includes maximum 25,000 afy "put" over a four-year period of surplus water and a maximum 33,000 afy "take" over a three-year dry period.
(2) DYY Program Expansion includes increases in total storage, "put" capacity, and "take" capacity.
(3) "Puts" for the initial DYY Program are accomplished by a combination of direct recharge and in-lieu deliveries.
(4) Does not include basin-wide in-lieu deliveries and direct recharge.
(5) MVWD assumed Chino Hills' shift obligation of 1,448 afy per an amendment to the agreement between the agencies dated March 5, 2007.
(6) Post modeling, adjusted take capacities. See Volume III for details.

Therefore, while the Basin has adequate storage capacity, any increases in groundwater production during dry years would likely require additional production capacity and/or groundwater treatment. Groundwater treatment during dry years will contribute to the long term sustainable use of the Basin. A further discussion of the Basin Operations Plan is provided in Volume I.



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Proposed DYY Participants and Put/Take Locations

Figure 1-2

1.6.1 Water Resources, Historical Water Use, and Shift Obligation for MVWD

The Asset Inventory data summarizing MVWD's existing water resources capabilities is presented in Table 1-4. The complete Asset Inventory is provided in Appendix A of Volume I. The results of the Asset Inventory indicate that MVWD has an imported water treatment capacity of 19.4 million gallons per day (mgd) (21,773 afy) and groundwater production capacity of 26.5 mgd (29,651 afy). MVWD receives its treated imported water from the Water Facilities Authority (WFA) Agua de Lejos Water Treatment Plant (WTP).

Table 1-4
Existing Water Resource Capacities for MVWD

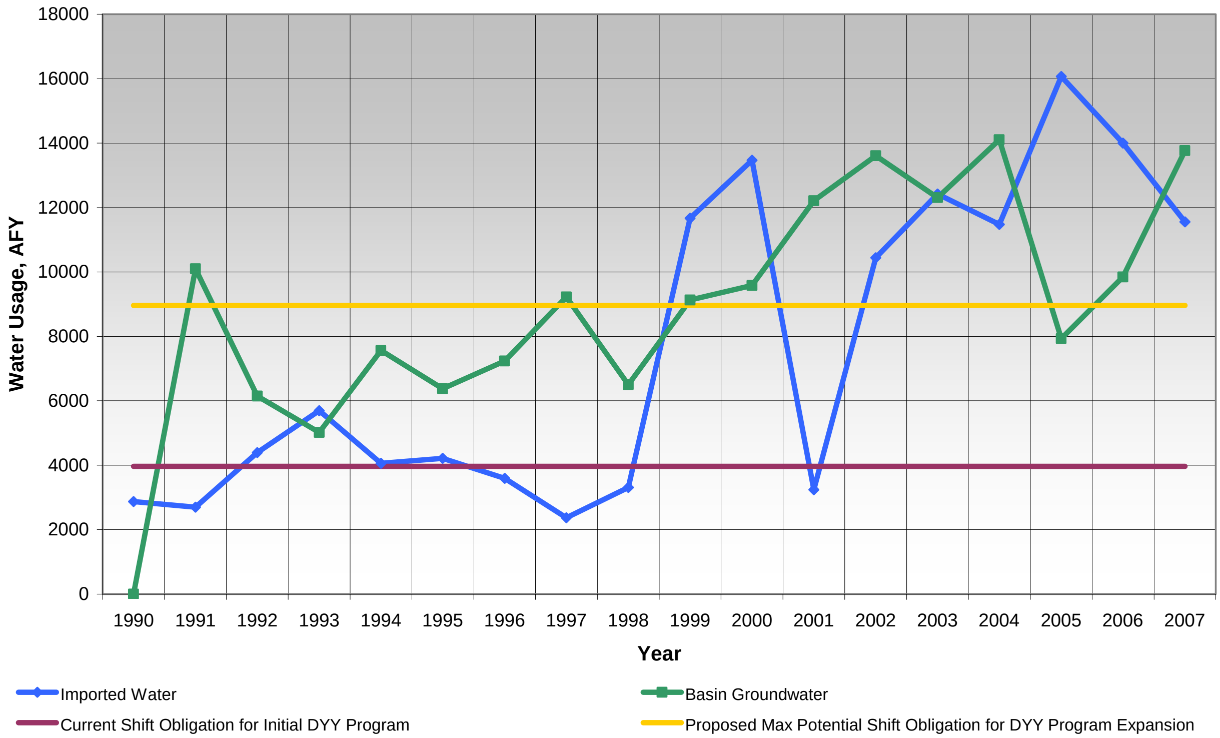
Water Resource	MVWD Capacity, mgd (afy)
Local Surface and Imported Water	
Local Surface Water	
Subtotal	0 (0)
Imported Metropolitan Water	
WFA	19.4 (21,773)
Subtotal	19.4 (21,773)
Total Local Surface and Imported Water	19.4 (21,773)
Groundwater	
Chino Basin Wells(1)	26.5 (29,651)
Non-Chino Basin Wells(1)	--
Total Groundwater	26.5 (29,651)
TOTAL WATER RESOURCES	**45.9 (51,424)**

Notes:
(1) Accounts for all well production capacity, regardless of water quality.

Figure 1-3 presents the historical groundwater production and imported water purchases for MVWD. In 2007, approximately 54 percent of MVWD's 25,326 acre-ft of water usage was Basin groundwater versus approximately 46 percent from imported water supplied by Metropolitan. Based on historical imports and on future growth projections, MVWD has elected to contribute 3,000 – 5,000 afy toward the potential 17,000 afy Program Expansion. To achieve this potential contribution, MVWD has proposed two alternative facility arrangements, options A and B. Facilities associated with these options are discussed in Section 1.5.2

Both options would incorporate "put" and "take" facilities. The "take" facilities would involve the use of new and existing wells and new IX treatment facilities. The "put" facilities would involve the use of existing and new ASR wells. An in-lieu shift would be arranged between MVWD and Walnut Valley Water District (WVWD). Water from the MVWD system would be delivered to the two districts via the City of Chino Hills, where it would be used in-lieu of imported water delivered to WVWD via the Pomona-Walnut-Rowland (PWR) pipeline. The in-lieu shift with WVWD would be provided using existing conveyance facilities and a new section of pipeline.

Figure 1-3
MVWD Historical Imported Water and Groundwater Usage

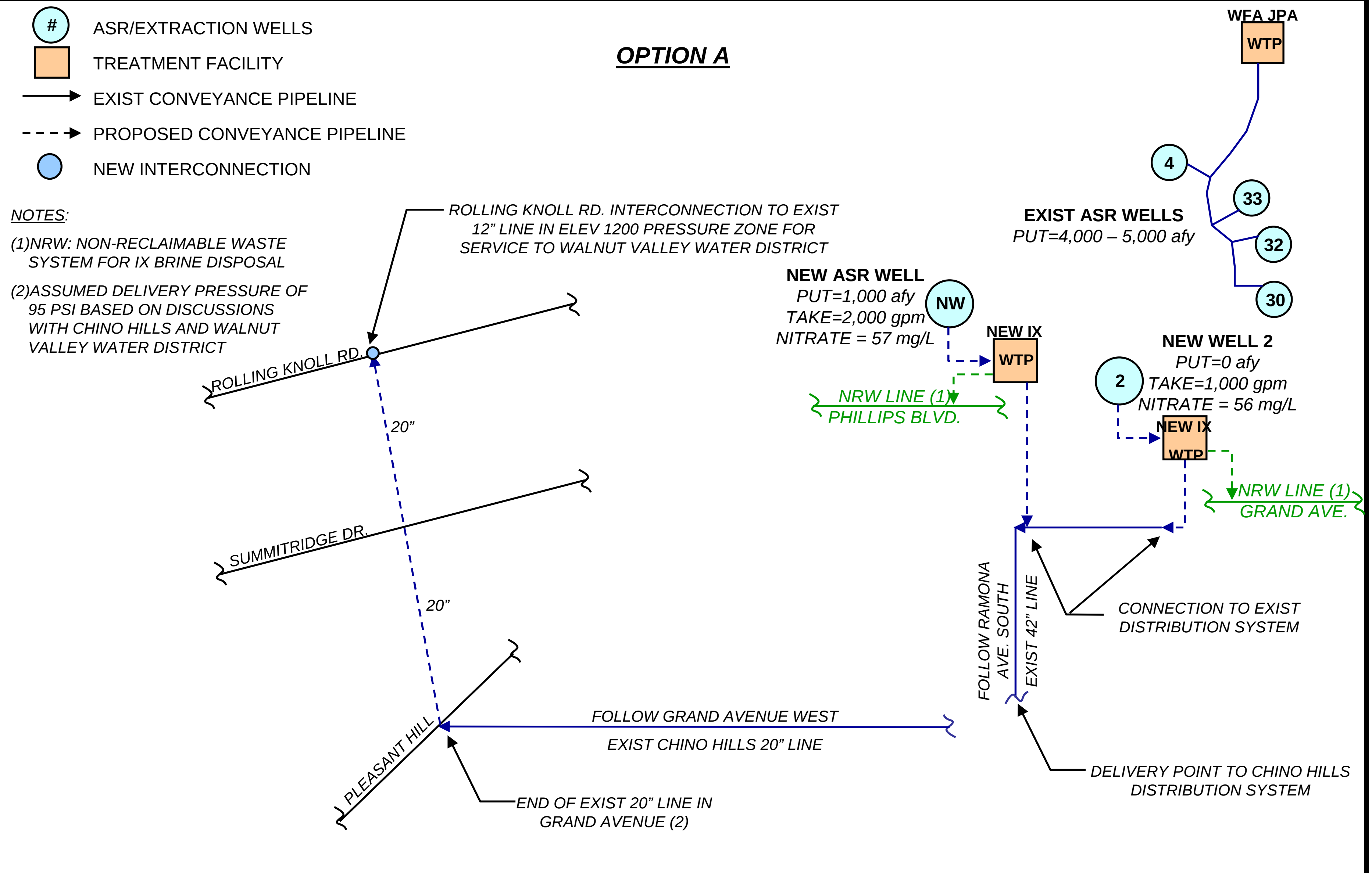


1.6.2 Program Expansion Facility Requirements

1.6.2.1 Option A Facilities

Figure 1-4 shows a conceptual schematic of the Option A facilities. Existing MVWD ASR well Nos. 4, 30, 32, and 33 (jointly owned by the City of Chino) would provide a put capacity of up to 3,000 to 4,000 afy using water primarily from the WFA WTP. A new ASR well and IX facility (New Well IX Facility) would be located in the City of Montclair, on the southeast corner of West State Street and Ramona Avenue. The new ASR well at the New Well IX facility would provide an injection capacity of approximately 1,000 gallons per minute (gpm) and an extraction capacity of 2,000 gpm. The New Well IX facility would thus have a treated water capacity of 2,000 gpm. Option A would also include a new well to replace the existing Well No. 2, which has been out of service for many years, and the addition of a new IX treatment plant (Well No. 2 IX Facility). The Well No. 2 IX Facility would be located in the City of Montclair, south of Grand Avenue and west of West Ramona Place. Well No. 2 IX Facility would treat water from Well No. 2 (used for extraction only) to produce approximately 1,000 gpm of treated water.

A new 20-inch interconnection pipe would be needed to connect the City of Chino Hills and Walnut Valley's service areas. The new pipeline would start at the intersection of Grand Avenue and Pleasant Hill, run along Grand Avenue, and terminate at the intersection of Grand Avenue and Rolling Knoll Road. An interconnection would be provided to deliver water to WVWD's 1200 Zone.



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Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District Project Overview – Option A Facilities

Figure 1-4

WVWD could serve its 1050 and 1350 pressure zones from the 1200 Zone through the use of existing booster pumps and pressure reducing stations. Coordination with TVMWD and WVWD would be required to arrange the in-lieu shift.

1.6.2.2 Option B Facilities

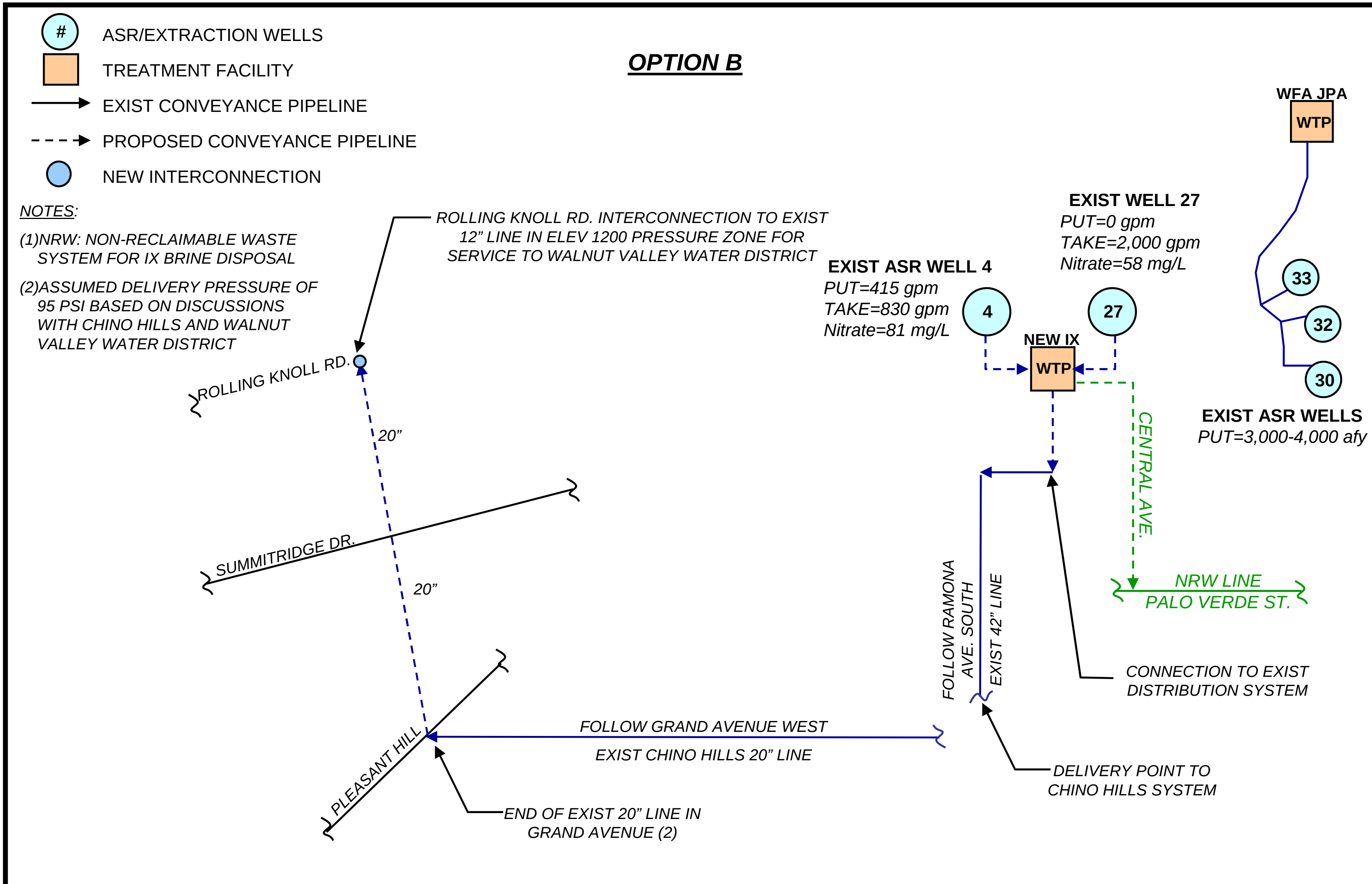
Figure 1-5 shows a conceptual schematic of Option B. Existing MVWD ASR well Nos. 30, 32, and 33 (jointly owned by City of Chino), would provide a put capacity of 3,000 to 4,000 afy using water primarily from the WFA WTP. A new IX facility would provide treatment for existing ASR Well No. 4 and Well No. 27, located in the City of Montclair, south of Arrow Highway/West 8th Street and east of Vernon Avenue. Well No. 4 would provide an injection capacity of 415 gpm and an extraction capacity of 830 gpm. Well No. 27 would provide an extraction capacity of 2,000 gpm. The Well No. 4 and 27 IX Facility would provide a treated water capacity of 2,830 gpm. The in-lieu shift with WVWD would be provided in the same manner as described in Option A facilities outlined in section 1.6.2.1.

The IX facilities for both Options A and B are presented in Chapter 2. The new ASR well, new production well, and conveyance pipe facilities are provided in Chapters 3, 4, and 5, respectively. The preliminary opinion of probable cost is presented in Chapter 6.

1.7 Abbreviations and Acronyms

The following abbreviations/acronyms are used in this report:

acre-ft	acre-feet
AFD	adjustable frequency drive
afy	acre-feet per year
amsl	above mean sea level
ASR	aquifer storage and recovery
ASTM	American Society for Testing Materials
AWWA	American Water Works Association
B&V	Black & Veatch
Basin	Chino Groundwater Basin
bgs	below ground surface
BV	bed volume
ft/day	feet per day
CDPH	California Department of Public Health
CEQA	California Environmental Quality Act
cfs	cubic feet per second
Chino	City of Chino
Chino Hills	City of Chino Hills
ClO_4^-	perchlorate
CML&C	cement mortar lined and coated
CML&W	cement mortar lined and wrapped
CVWD	Cucamonga Valley Water District



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Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District Project Overview – Option B Facilities

Figure 1-5

DYY	Dry-Year Yield
DYY Program	initial Chino Basin Dry-Year Yield Program
DYY Program Expansion	Chino Basin Dry-Year Yield Program Expansion
fps	feet per second
FWC	Fontana Water Company
gal	gallons
gpm	gallons per minute
HDPE	high-density polyethylene
HGL	hydraulic grade line
HMI	human machine interface
HP	horsepower
HVAC	heating, ventilation, and air conditioning
I&C	instrumentation and controls
IEUA	Inland Empire Utilities Agency
IX	Ion Exchange
JCSD	Jurupa Community Services District
Judgment	Chino Basin Municipal Water District vs. the City of Chino et al. (1978)
LACSD	Los Angeles County Sanitation District
lbs	pounds
MCCs	motor control centers
MCL	maximum contaminant level
mg	milligrams
mgd	million gallons per day
Metropolitan	Metropolitan Water District of Southern California
mg/L	milligrams per liter
min	minutes
MVWD	Monte Vista Water District
NO_3^-	nitrate
NRWS	Non-reclaimable Wastewater, or Waste, System
OD	outside diameter
OEM	original equipment manufacturer
Ontario	City of Ontario
O&M	operation and maintenance
OBMP	Optimum Basin Management Program
PDR	project development report
PLC	programmable logic controller
ppb	parts per billion
Pomona	City of Pomona
Program	DYY Program, DYY Program Expansion
Program Expansion	Chino Basin Dry-Year Yield Program Expansion
psi	pounds per square inch
PVC	polyvinyl chloride
PWR	Pomona-Walnut-Rowland
RC	Riverside Corona
ROW	right of way

SARI	Santa Ana Regional Interceptor
TDA	Tom Dodson & Associates
TDH	total dynamic head
TDS	total dissolved solids
TEFC	totally enclosed fan-cooled
TM	technical memorandum
TOC	total organic carbon
TVMWD	Three Valleys Municipal Water District
Upland	City of Upland
USGS	United States Geological Survey
Watermaster	Chino Basin Watermaster
WEI	Wildermuth Environmental, Inc.
WFA	Water Facilities Authority
WTP	water treatment plant
WVWD	Walnut Valley Water District
WMWD	Western Municipal Water District

1.8 References

General references are listed in Volume I, Section 1.9. Agency-specific references for the facilities listed in this Volume II E are shown below.

[MVWD, 1998] *Water Master Plan*, prepared for Monte Vista Water District, Boyle Engineering Corporation, June 1998.

2.0 ION EXCHANGE FACILITIES

2.1 Overview

This chapter presents a detailed description of the proposed MVWD IX treatment facilities, which will provide a portion of MVWD's contribution to the DYY program obligation. This chapter reviews the raw water supply well quality, IX facility components, site requirements, electrical requirements, instrumentation and control (I&C) requirements, and conveyance piping.

MVWD has proposed two options to meet their shift commitment. Option A would include a New ASR Well IX Facility and Well No. 2 IX Facility. Option B would include an IX facility at the Well No. 4 and 27 site. The New ASR Well IX Facility is located in the City of Montclair, on the southeast corner of West State St. and Ramona Avenue. The Well No. 2 IX Facility would be located in the City of Montclair, south of Grand Avenue. and west of West Ramona Place. The Well No. 4 and 27 IX Facility would be located in the City of Montclair, south of Arrow Highway/West 8th St. and east of Vernon Avenue. Figures 2-1, 2-2, and 2-3 present location maps of the new IX facilities, all of which are in residential areas.

2.2 Raw Water Supply

The New ASR Well IX Facility would treat groundwater from a new ASR well. The Well No. 2 IX Facility would treat groundwater from a new production well adjacent to and replacing existing Well No. 2. (In discussions with MVWD operations staff, the existing wellhead equipment and casing were determined to be too old and degraded to rehabilitate.) A discussion of ASR and production wells and typical drawings are provided in Volume I. The Well No. 4 and 27 IX Facility would treat groundwater from existing Wells 4 and 27. In order to approximate the characteristics for the two new wells, data collected by WEI for two nearby, existing Chino wells was used as a basis for the operating conditions of the new wells. Table 2-1 presents the historic groundwater elevations for the out-of-service Well No. 2 used to develop the groundwater elevations for Chino Wells 10 and 12. Chino Wells 10 and 12 are located approximately 3,080 feet from New Well No. 2 and approximately 6,500 feet from the new ASR well location.

Table 2-1
Historical Operating Conditions (1)

Operating Conditions	Chino Well No. 10	Chino Well No. 12
Site Elevation, feet above mean sea level (amsl)	890	890
Production Capacity, gpm	1,087	2,225
Est. Avg. Static Groundwater Elev., ft bgs (2)	286	290
Estimated Average Drawdown, feet (3)	53	66
Approximate Specific Capacity, gpm/ft (4)	20	34

Notes:
(1) Historical operating conditions listed in table are based on actual pump test data conducted in September 2007 and provided by WEI, 2008.
(2) Feet, below ground surface (bgs).
(3) Pump test data provided by City of Chino, 2008.
(4) Gallons per minute per foot of drawdown.

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Monte Vista Water District – New Well IX Facilities Vicinity Map

Figure

2-1

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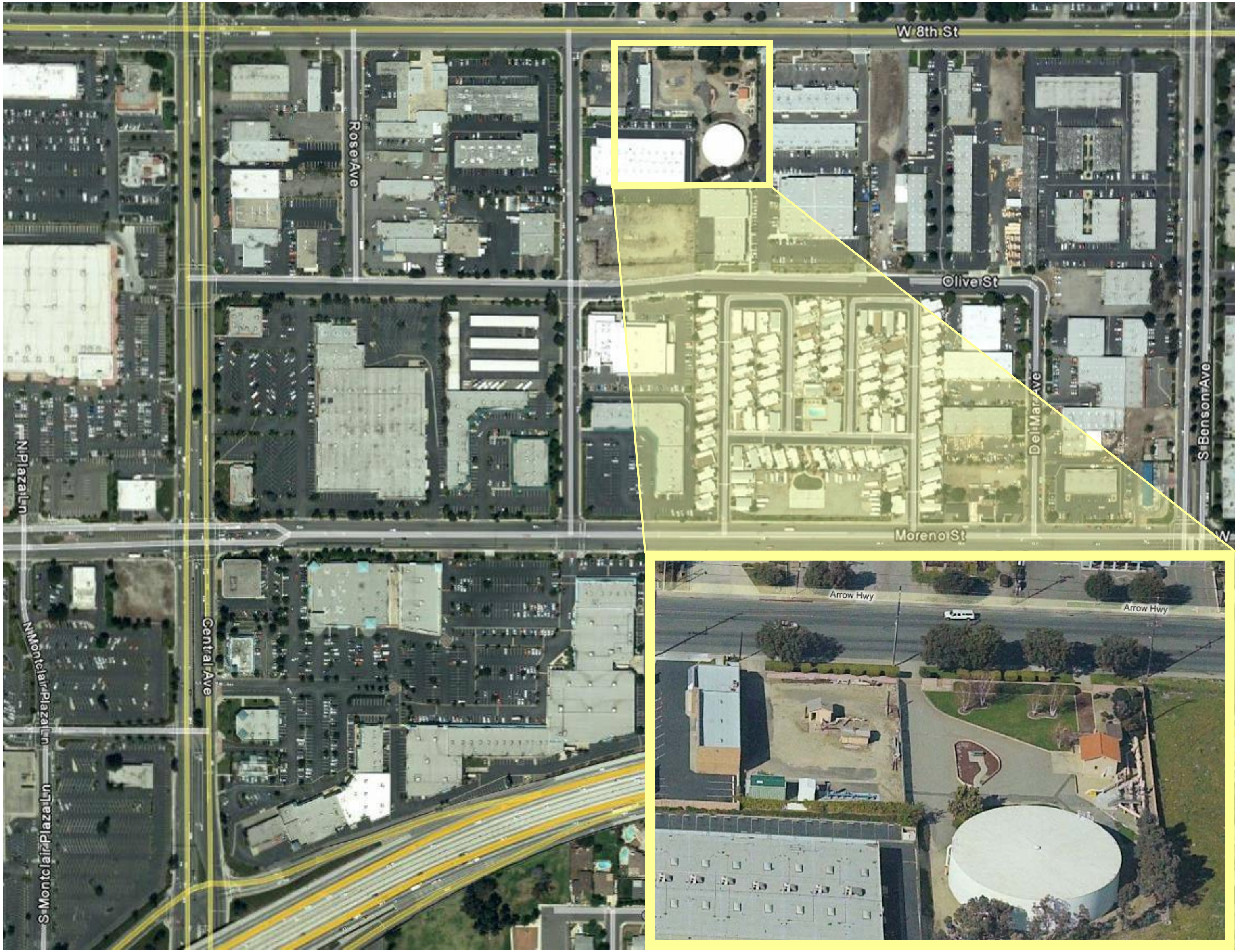
Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District – Well No.2 IX Facility Vicinity Map

Figure

2-2



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Chino Basin Dry Year Yield Program Expansion Project
Monte Vista Water District – Well 4 & 27 IX Facilities Vicinity Map

Figure 2-3

Table 2-2 presents the anticipated operating conditions and performance for both the proposed and existing MVWD wells. This information will be used to develop and confirm hydraulic capabilities of the wells.

Table 2-2
Anticipated Operating Conditions

Conditions	Option A		Option B	
	New ASR Well	New Well No. 2	Well No. 4	Well No. 27
General Conditions				
Basis for Operating Conditions, Well No.	Chino 12	Chino 12	4	27
Distance from Basis Well Above, feet	6,500	3,075	0	0
Location (Intersection)	West State/ Ramona	Grand/ W. Ramona Pl.	Vernon/ Arrow Hwy	Vernon/ Arrow Hwy
Site Elevation, feet amsl (1)	929	910	1,188	1,188
Well HGL/Delivery Zone, feet amsl	1,207/ Zone 2	943/ Zone 3	1,207/ Zone 2	1,207/ Zone 2
Operating Conditions				
Production Capacity, gpm	2,000	1,000	830	2,000
Maximum Injection Capacity, gpm	1,000	N/A	415	N/A
Est. Avg. Static Groundwater Elev., ft bgs	329(3)	310(3)	550	578
Est. Avg. Injection Head, feet (2)	562	N/A	569	N/A
Assumed Specific Capacity, gpm/ft	34	34	30	33
Calculated Estimated Drawdown, feet	59	29	28	61

Notes:
(1) Above mean sea level (amsl).
(2) Addition of static lift and system pressure in delivery zone.
(3) New ASR Well and Well No. 2 sites are at higher elevations than Chino 12, which resulted in a greater depth to static groundwater.

The wellhead pump for the New ASR Well and the New Well No. 2 would be a multistage vertical turbine with an electric motor located above ground. The drive shaft would be water lubricated, and pre-lubrication of the line shaft bearings would be provided during the pump startup. Pump performance design criteria were developed for the expected production rates and are presented in Table 2-3. For the New ASR Well and New Well No. 2, the pumps and motors were sized to deliver water from the wells, through the new IX facilities, and to the required elevation listed in Table 2-2. For existing Wells 4 and 27, the hydraulic capabilities of the existing pumps and motors were verified to ensure that water could be delivered from the wells, through the new IX facilities, and to the required elevation listed in Table 2-2. A preliminary hydraulics investigation showed that the pumps for Wells 4 and 27 would most likely provide enough lift to satisfy the requirements of the new facilities. Further hydraulic studies should be done during detailed design to verify that Well 4 and 27 pumping is sufficient. A more detailed discussion of the hydraulic conditions is presented in Section 2.5.1.4.

Table 2-3
Assumed Pump Performance

Description	Option A		Option B	
	New ASR Well	Well No. 2	Well No. 4	Well No. 27
Pump				
Type	Deep well turbine	Deep well turbine	Deep well turbine	Deep well turbine
Capacity, gpm	2,000	1,000	830	2,000
Total Dynamic Head, feet (1)	724	429	659	720
Pump Efficiency, percent	80	80	80	80
Motor Efficiency, percent	90	90	90	90
Discharge Column Diameter, inches	12	8	8	12
Motor				
Type (2)	TEFC High-Efficiency	TEFC High-Efficiency	TEFC Premium High-Efficiency	TEFC High-Efficiency
Nominal Motor Horsepower, HP	550	200	200	500
Motor Drive (3)	AFD	AFD	AFD	AFD

Notes:
(1) Includes frictional losses and mechanical shaft losses.
(2) Totally Enclosed Fan Cooled (TEFC)
(3) Adjustable Frequency Drive (AFD)

2.3 Raw Water Quality

The water quality data for the MVWD raw water supplies were developed from the WEI database of California Department of Public Health (CDPH) records and cross-referenced with any water quality data received during the development of the Asset Inventory. Table 2-4 presents the raw water quality data for an existing well used to develop the New ASR Well. Because the New ASR well has not yet been drilled and water quality data is not available, the design water quality for this well was based on the water quality data from nearby Pomona Well 10.

Table 2-5 presents the raw water quality data for two existing wells used to develop the New Well No. 2. Since existing Well No. 2 has not been used for several years and water quality data is not available, the design water quality for this well was based on the water quality data from two nearby wells: Chino Wells 10 and 12. The average and maximum values listed were averaged and used as the design water quality for the Well No. 2 IX Facility. The design may change once the actual water quality of the groundwater in Well No. 2 is verified during detailed design.

Table 2-4
MVWD New ASR IX Facility Estimated Raw Water Quality

Constituent	Pomona-10		New ASR Well	
	Avg	Max	Avg	Max
Pumping Capacity	--	800	--	2,000
Cations (mg/L)				
Calcium	75	75	75	75
Magnesium	14	14	14	14
Sodium	11	11	11	11
Potassium	1.8	1.8	1.8	1.8
Anions (mg/L)				
Alkalinity (as $CaCO_3$)	150	150	150	150
Sulfate	45	45	45	45
Chloride	28	28	28	28
Nitrate	43	57	43	57
Other (µg/L)				
DBCP (Dibromo Chloroprone)	0.00	0.00	0.00	0.00
Arsenic	0.7	0.7	0.7	0.7
TCE (Trichloroethylene)	0.7	1.1	0.7	1.1
Perchlorate	5.8	9.6	5.8	9.6
General				
Total Dissolved Solids	330	330	330	330

Table 2-5
MVWD Well No. 2 IX Facility Estimated Raw Water Quality

Constituent	Chino-10		Chino-12		New Well No. 2	
	Avg	Max	Avg	Max	Avg	Max
Pumping Capacity	--	1,500	--	2,250	--	1,000
Cations (mg/L)						
Calcium	75	99	68	79	71	87
Magnesium	17	27	14	14	15	19
Sodium	14	25	16	17	15	20
Potassium	4.1	19	1.9	2.1	2.8	8.9
Anions (mg/L)						
Alkalinity (as $CaCO_3$)	157	170	145	151	150	159
Sulfate	38	87	27	31	32	53
Chloride	17	23	12	16	14	19
Nitrate	18	22	66	78	47	56
Other (µg/L)						
DBCP	0.00	0.00	0.01	0.02	0.00	0.01
Arsenic	0.00	0.00	0.00	0.00	0.00	0.00
TCE	0.00	0.00	0.00	0.00	0.00	0.00
Perchlorate	17.9	23	14.6	18	15.9	20
General						
Total Dissolved Solids	350	440	312	324	327	370

Table 2-6 presents the estimated raw water quality data for the New ASR well, New Well No. 2, and the other two existing wells used as source water for the IX facilities. It should be noted that raw water from Wells 4 and 27 would be blended upstream of the IX facility, which would result in the IX facility being designed to treat the blended water numbers listed in the last two columns of Table 2-6.

Table 2-6
MVWD IX Facilities Estimated Raw Water Quality

Constituent	Option A				Option B					
	MVWD No. 2(1)		New ASR Well(1)		MVWD No. 4		MVWD No. 27		Blend	
	Avg	Max	Avg	Max	Avg	Max	Avg	Max	Avg	Max
Pumping Capacity	--	1,000	--	2,000	--	830	--	2,000	--	2,830
Cations (mg/L)										
Calcium	71	87	75	75	81	81	58	58	64	65
Magnesium	15	19	14	14	21	21	15	15	16	17
Sodium	15	20	11	11	17	17	27	28	24	25
Potassium	2.8	8.9	1.8	1.8	2.1	2.1	32	32	23	23
Anions (mg/L)										
Alkalinity (as $CaCO_3$)	150	159	150	150	180	180	155	160	162	166
Sulfate	32	53	45	45	48	48	32	32	37	37
Chloride	14	19	28	28	13	13	9	11	10	12
Nitrate	47	56	43	57	68	81	55	85	59	84
Other (µg/L)										
DBCP	0.00	0.01	0.00	0.00	0.28	0.43	0.14	0.19	0.18	0.26
Arsenic	0.00	0.00	0.66	0.66	1.20	2.40	0.00	0.00	0.35	0.71
TCE	0.00	0.00	0.65	1.10	0.00	0.00	0.00	0.00	0.00	0.00
Perchlorate	15.9	20.0	5.8	9.6	0.00	0.00	0.00	0.00	0.00	0.00
General										
Total Dissolved Solids	327	370	330	330	0	0	310	320	219	226

Notes:
(1) Water quality developed in Table 2-4 and Table 2-5.

It is also recommended that MVWD conduct a complete Title 22 water quality analysis on the feed water wells to ensure that recent and accurate water quality data is available during the final process design.

2.4 IX Facilities

The IX facilities would be constructed primarily on MVWD property and would treat nitrate-laden groundwater from the New ASR well, New Well No. 2, and existing Wells 4 and 27. A discussion on IX design and a typical process schematic are provided in Volume I. The sections below describe design criteria and components of the IX facilities.

2.4.1 Design Capacity

The New ASR Well IX Facility, Well No. 2 IX facility, and Well No. 4 and 27 IX Facility will treat nitrate-laden groundwater from a new on-site ASR well, a new Well No. 2, and existing on-site wells 4 and 27, respectively. The individual IX facility process flows were developed using the raw water quality data from Table 2-6 and assuming a treated water nitrate (NO_3) concentration of 5 mg/L as NO_3 and a blended (finished) water nitrate goal of 25 mg/L as NO_3. Because the IX feed and treated water flows also serve as the process water required for resin regeneration, the actual IX facility output would be slightly less than the capacities stated above during certain stages of the resin regeneration cycle. Table 2-7 presents the specific design capacity criteria for the three IX facilities based on the assumed water quality data provided in Table 2-6.

Table 2-7
MVWD IX Facilities Design Capacity Criteria

Parameter	Option A		Option B
	New ASR IX Facility	Well No. 2 IX Facility	Well No. 4 & 27 IX Facility
Water Quality [1]			
Raw Water Nitrate, mg/L	57	56	83
Treated Water Nitrate, mg/L	5	5	5
Blended Water Nitrate Goal, mg/L	25	25	25
Average Nitrate Leakage, percent	9	9	6
Process Flows [2]			
Raw Water, gpm	2,000	1,000	2,830
Feed Water, gpm	1,223	605	2,108
Treated Water, gpm	1,223	605	2,108
Bypass Flow, gpm	777	395	723
Blended (Finished) Flow, gpm	2,000	1,000	2,830

Notes:
(1) Values expressed as nitrate as NO_3.
(2) During production mode. Process flows vary slightly during the regeneration mode.

2.4.2 Process Requirements

The IX process would reduce the nitrates present to about 5 mg/L assuming 6 to 9 percent leakage. The IX exchange vessels would have Type I strong base anion exchange resin and would be approximately 6.5 to 10 feet in diameter. The selection of the resin type is discussed in Volume I. Resin depth would be approximately 4.8 feet to provide a range of approximately 162 to 376 cubic feet of resin in each exchange vessel. Sidewall depth would be approximately 11 feet. A viewing port would be provided in the sidewall at the top of the resin.

Three to four exchange vessels would be provided initially, one being a standby vessel. Depending on the facility, the IX vessels would be regenerated once every 18 to 27 hours when operated at full design capacity. A standby vessel is included to allow for continuous operation while one of the vessels is removed from service for regeneration. The IX exchange vessels would be operated in a “staggered exhaustion” mode, such that only one vessel would require regeneration at any given time. IX vessel regeneration would require approximately 3 hours per unit. Figures 2-4, 2-5, and 2-6 present a typical 72-hour operation sequence for each of the three IX facilities. Figures 2-4 and 2-5 show a similar production time because they have similar source water nitrate concentrations. Figure 2-6 shows that because the source water from Wells 4 and 27 have significantly higher nitrate concentrations, shorter and more frequent production cycles would be required.

Figure 2-4
MVWD New ASR Well IX Facility
Typical 72-hour Operation Sequence

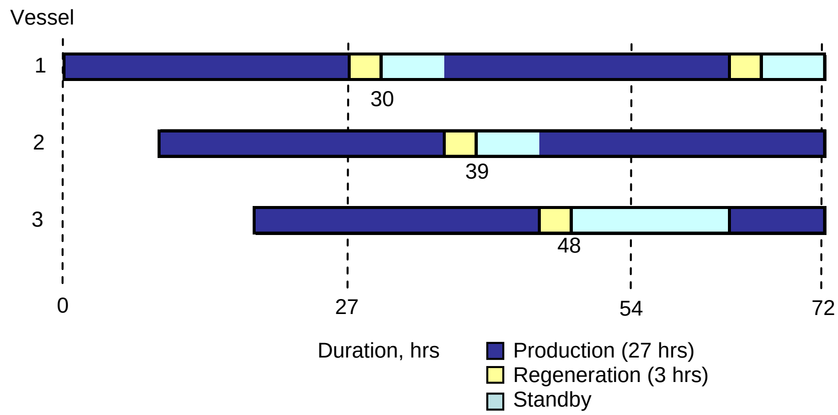


Figure 2-5
MVWD Well No. 2 IX Facility
Typical 72-hour Operation Sequence

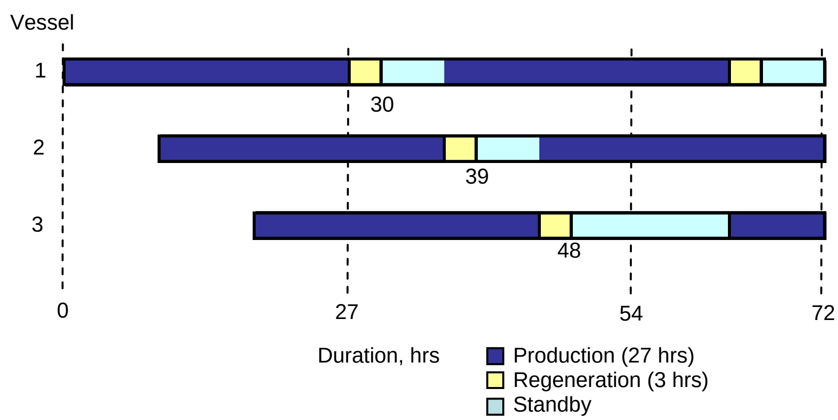


Figure 2-6
MVWD Well No. 4 and 27 IX Facility
Typical 72-hour Operation Sequence

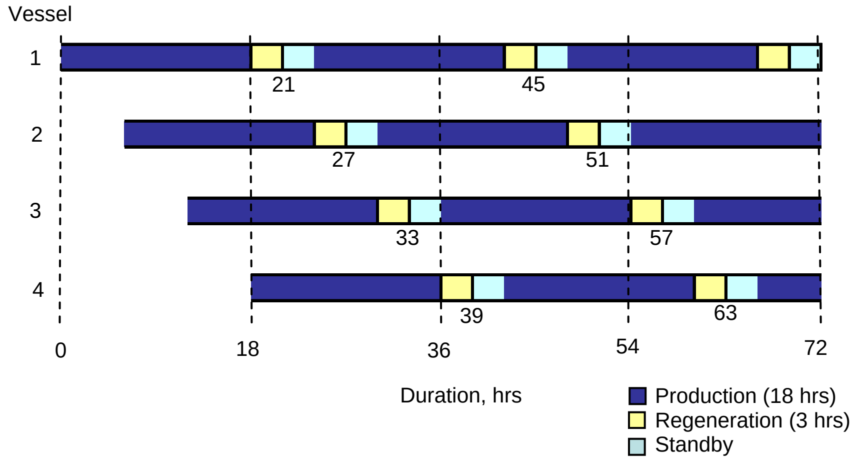


Table 2-8 presents the process requirements for the MVWD IX facilities. The actual vessel dimensions and resin requirements may vary slightly between manufacturers. Most IX system manufacturers furnish all process equipment within the IX treatment system "black box." This would include all IX vessels, process piping, valves and other appurtenances, brine saturators, and waste equalization tanks.

Table 2-8
MVWD IX Facilities Process Requirements

Parameter	Option A		Option B
	New ASR Well IX Facility	Well No. 2 IX Facility	Well No. 4 & 27 IX Facility
No. of IX Vessels (duty/standby)	2/1	2/1	3/1
Hydraulic Loading, gpm/square foot	9	9	9
Bed Volumes Treated per Hour, BV/hour	15	15	15
Production Cycle Length, hours	27	27	18
IX Vessel capacity, each, gpm	612	303	703
IX Vessel Dimensions			
Diameter, feet	10	6.5	10
Sidewall Depth, feet	11	11	11
IX Resin			
Type	Type 1 Strong Base Anionic	Type 1 Strong Base Anionic	Type 1 Strong Base Anionic
Depth, feet	4.8	4.8	4.8
Volume per Vessel, cubic feet	328	162	376

2.4.3 Regeneration System

Countercurrent regeneration is recommended for use in order to minimize nitrate leakage through the IX vessels and to buffer the potential impacts of variations in raw water nitrate concentrations. For countercurrent regeneration, the regenerate solution (brine) is introduced in an upflow mode at the bottom of the IX vessel; the resin at the bottom of the vessel is therefore essentially completely regenerated and free of nitrate. The IX resin would be regenerated using a 7 percent salt solution (0.58 lbs salt per gallon). The brine solution would be applied at a rate of 7.50 pounds of salt per cubic foot of resin. The salt solution would be prepared and stored as concentrated 26 percent brine and diluted to a 7 percent solution prior to entering the IX vessels. Treated water from the IX process would be used as dilution water for the brine generation process. An automated brine production system, which incorporates bulk salt storage and brine preparation/storage facilities within a single tank, would be provided.

The resin regeneration cycle consists of eight steps: 1) a downflow initial backwash is used to remove any remaining suspended solids from the resin; 2) following the backwash, the resin bed is allowed to settle for approximately four minutes, which allows the bed to compact more easily; 3) bed compaction is provided to reduce the bed depth in order to provide closer contact of the media with the salt brine during regeneration; 4) the freeboard above the resin bed is drained to ensure that the brine solution added to the vessel will be applied directly to the resin bed and not diluted with any remaining water in the vessel; 5) the resin is regenerated with a 7 percent salt solution, which would be a blended flow of nitrate-free treated water and concentrated 26 percent salt solution; 6) a slow rinse is performed using softened water to flush out the brine solution; 7) the freeboard is refilled; 8) a fast rinse is performed to ensure that any remaining brine is removed from the resin bed and that the bed is ready for a new production cycle. Tables 2-9, 2-10, and 2-11 summarize the IX vessel regeneration cycle for each facility.

Table 2-9
MVWD New ASR Well IX Facility Regeneration Cycle

Step	Description	Direction of Flow	Time (min)	Flowrate (gpm)	Source	Wastewater (gal)	
						Recycle	Waste
1	Initial Backwash	Down	15	204	IX feed	3,060	--
2	Settle Bed	--	4	--	--	--	--
3	Compact Bed	Down	2	700	IX feed	1,400	--
4	Drain Freeboard	Down	15	136	Water in vessel	2,040	--
5	Resin Regeneration	Up	35	120	7% Salt Brine	--	4,200
6	Slow Rinse	Up	37	125	Softened Water	--	4,625
7	Refill Freeboard	Down	16	127	IX feed		2,032
8	Fast Rinse	Down	5	700	IX feed	--	3,500
Total	--	--	129	--	--	6,500	14,358

Table 2-10
MVWD Well No. 2 IX Facility Regeneration Cycle

Step	Description	Direction of Flow	Time (min)	Flowrate (gpm)	Source	Wastewater (gal)	
						Recycle	Waste
1	Initial Backwash	Down	15	101	IX feed	1,513	--
2	Settle Bed	--	4	--	--	--	--
3	Compact Bed	Down	2	600	IX feed	1,200	--
4	Drain Freeboard	Down	15	68	Water in vessel	1,006	--
5	Resin Regeneration	Up	35	60	7% Salt Brine	--	2,079
6	Slow Rinse	Up	37	125	Softened Water	--	4,625
7	Refill Freeboard	Down	16	63	IX feed	--	1,006
8	Fast Rinse	Down	5	600	IX feed	--	3,000
Total	--	--	129	--	--	3,719	10,680

Table 2-11
MVWD Well No. 4 and 27 IX Facility Regeneration Cycle

Step	Description	Direction of Flow	Time (min)	Flowrate (gpm)	Source	Wastewater (gal)	
						Recycle	Waste
1	Initial Backwash	Down	15	234	IX feed	3,510	--
2	Settle Bed	--	4	--	--	--	--
3	Compact Bed	Down	2	700	IX feed	1,400	--
4	Drain Freeboard	Down	15	156	Water in vessel	2,340	--
5	Resin Regeneration	Up	35	138	7% Salt Brine	--	4,830
6	Slow Rinse	Up	37	125	Softened Water	--	4,625
7	Refill Freeboard	Down	16	146	IX feed	--	2,336
8	Fast Rinse	Down	5	700	IX feed	--	3,500
Total	--	--	129	--	--	7,250	15,291

2.4.4 Waste Disposal

In order to reduce the wastewater discharge to the NRWS lines and conserve raw water supply, the wastewater stream would be divided into two streams. One stream would be intercepted for recycling to the front of the IX facility, and the second stream would be conveyed to a waste equalization tank for ultimate delivery to the NRWS line. Table 2-12 summarizes the duration and volumes of these two streams.

Table 2-12
MVWD IX Facilities Design Capacity Criteria

Parameter	Option A		Option B
	New ASR Well IX Facility	Well No.2 IX Facility	Well No. 4 & 27 IX Facility
Recycle Stream			
Storage Tank Volume, gallons	5,971	3,361	10,142
Storage Tank Drain System			
Type	Pumped	Pumped	Pumped
Average Drain Rate, gpm	50	50	50
Delivery Pressure, psi	70	40	40
Time to Drain 1 Regeneration Cycle, minutes	129	129	129
Waste Regenerate Stream			
No. of Tanks	1	2	2
Equalization Tank Volume, gallons each tank	14,359	5,355	7,644
Equalization Tank Drain System			
Type	Gravity	Gravity	Gravity
Average Drain Rate, gpm	10	10	15
Maximum Drain Rate, gpm	115	85	120
Time to Drain 1 Regeneration Cycle @ Max Drain Rate, hours	2.1	2.1	2.1

The first stream would be intercepted for recycling and would consist of the initial backwash, the bed compaction, and the draining of freeboard. The total volume of wastewater produced from these three steps would occur every 27 hours, 27 hours, and 18 hours for New ASR Well IX Facility, Well No. 2 IX Facility, and Well No. 4 and 27 IX Facility, respectively. The option of discharging this component to the NRWS line would also be provided for operational flexibility.

The second stream would be discharged to the NRWS line and would consist of the regeneration, slow rinse, and fast rinse. The total volume of wastewater produced from these steps would occur at the same frequency as regeneration.

A waste regenerate equalization tank would be provided to reduce the instantaneous flow rate being discharged to the NRWS line. Waste regenerate would enter the equalization tank at the required flow rate for regeneration and would exit the equalization tank at a constant rate by gravity. A metering station would be provided on the wastewater discharge to the NRWS line. Table 2-12 summarizes the waste regenerate equalization and discharge requirements.

2.4.5 Salt Brine Storage and Feed System

Salt storage facilities would be provided on site for the brine generation process. Salt for the preparation of brine would be delivered to the site dry in bulk tanker trucks. The salt delivery trucks would have a maximum capacity of approximately 20 tons (40,000 pounds) and the salt would be unloaded pneumatically into the bulk salt storage tanks on site. The maximum total salt stored on site would range from approximately 36 to 109 tons, which would provide a sufficient salt stock for 14 days of brine production and one entire salt delivery depending on the facility. The salt would be stored in tanks ranging from 6.5 to 12 feet in diameter and 11 to 14 feet in height. Each storage tank would be equipped with a water feed connection to prepare a 26 percent salt brine solution within the tank. Brine production would be limited to approximately 50 gpm from the brinemaker that is part of the storage tank. The IX facilities would require approximately 60 to 138 gpm of salt brine for the resin regeneration cycles. Brine pumps would transfer the salt brine from the storage tank and inject it into the backwash water for resin regeneration. Two salt brine pumps would be provided, one duty and one for standby. The pumps would have variable speed drives. Table 2-13 presents the specific design criteria of the IX facilities' salt brine storage and feed system.

Table 2-13
MVWD IX Facilities Salt Brine Storage and Feed Systems

Parameter	Option A		Option B
	New ASR Well IX Facility	Well No.2 IX Facility	Well No. 4 & 27 IX Facility
Chemical	Salt (NaOCl)	Salt (NaOCl)	Salt (NaOCl)
Product Form	Delivered in bulk	Delivered in bulk	Delivered in bulk
Brine Pumps			
Type	Centrifugal	Centrifugal	Centrifugal
Number	2	2	2
Rated Capacity, gpm	33	16	50
Salt Application Rate, lbs. NaOCl/cu. ft. resin	7.5	7.5	7.5
Salt Required per Regeneration Cycle, lbs.	2,453	1,214	2,818
Salt Brine Solution Concentration, percent by weight	26	26	26
Required 26% Brine Volume per cycle, gallons	4,201	2,079	4826
Dilution (Softened) Water Feed Rate, gpm	88	44	101
Salt Brine Feed Rate for resin regeneration cycle, gpm	121	60	138
Feed Brine Solution Concentration, percent salt by weight	7	7	7
Assumed Regeneration Frequency (1), hours	Once every 27	Once every 27	Once every 18
No. of Regeneration Cycles per day	1	1	4
Salt Usage at Design Flow, tons/day	2.3	1.1	5.9
Bulk Brine Storage Tanks (Saturators)			
Dimensions, diameter x sidewall height	12' x 13'	11' x 11'	12' x 14'
No. of Tanks	1	1	2
Total Salt Storage Capacity, tons	52	36	109
Materials of Construction	Fiberglass Reinforced Plastic	Fiberglass Reinforced Plastic	Fiberglass Reinforced Plastic
Salt Delivery Quantity, tons	20	20	20
Storage Duration, days	14	14	14

Notes:
(1) At design flow.

2.4.6 Disinfection

Disinfection would be required to satisfy chlorine demand and residual. The existing Well No. 4 and No. 27 site contains a sodium hypochlorite feed system that would continue to be utilized. Both the Well No. 2 and the New ASR Well IX Facilities would require new disinfection facilities. It was assumed that MVWD would prefer to install sodium hypochlorite feed systems for both these facilities as well. MVWD has indicated that an aqueous ammonia facility would need to be provided at the New ASR Well IX Facility site to generate chloramines for disinfection. Sodium hypochlorite has minimal chemical handling hazards (i.e. scrubbers are not required). Totes can be easily removed from the site during periods when the well and IX facilities are not in use. For the purposes of this PDR and preparing cost estimates, sodium hypochlorite delivered in totes is the recommended disinfection system for the Well No. 2 and New ASR Well IX Facilities. An aqueous ammonia system would be provided at the New ASR Well IX Facility as well. However, decisions on the disinfection methodology will ultimately require re-examination during the final design stage. Other criteria would also need to be accounted for at this time (i.e. existing conditions, etc.).

2.4.7 Site Requirements

2.4.7.1 Site Descriptions and Existing Facilities

New ASR Well IX Facility

The New ASR Well IX Facility would be located on MVWD property in the City of Montclair, on the southeast corner of West State Street and Ramona Avenue. The portion of the site that can be used for the new facilities has space constraints due to existing metering facility piping located above and below grade. The new ASR well would be drilled on the northwest portion of the site.

Well No. 2 IX Facility

The Well No. 2 IX Facility would be located on MVWD property in the City of Montclair, south of Grand Avenue and west of West Ramona Place. The site contains existing Well No. 2, a well house, and an existing storage reservoir. A new Well No. 2 would be drilled adjacent to existing Well No. 2, equipped, and used as a feed water source for the new IX facility. The existing reservoir would be used as a potential treated water storage facility.

The Well No. 2 site has an existing steel reservoir, well, and electrical building.

Well No. 4 and 27 IX Facility

The Well No. 4 and 27 IX Facility would be located on MVWD property in the City of Montclair, south of Arrow Highway/West 8th Street and east of Vernon Avenue. The site contains Well No. 4 and 27 buildings and an existing storage reservoir. Wells 4 and 27 would be used as feed water sources for the new IX facility. The existing reservoir would be used as a potential treated water storage facility.

2.4.7.2 New Facilities

The MVWD IX Facilities would require site space for the new major components listed in Table 2-14.

Table 2-14
Major IX Components

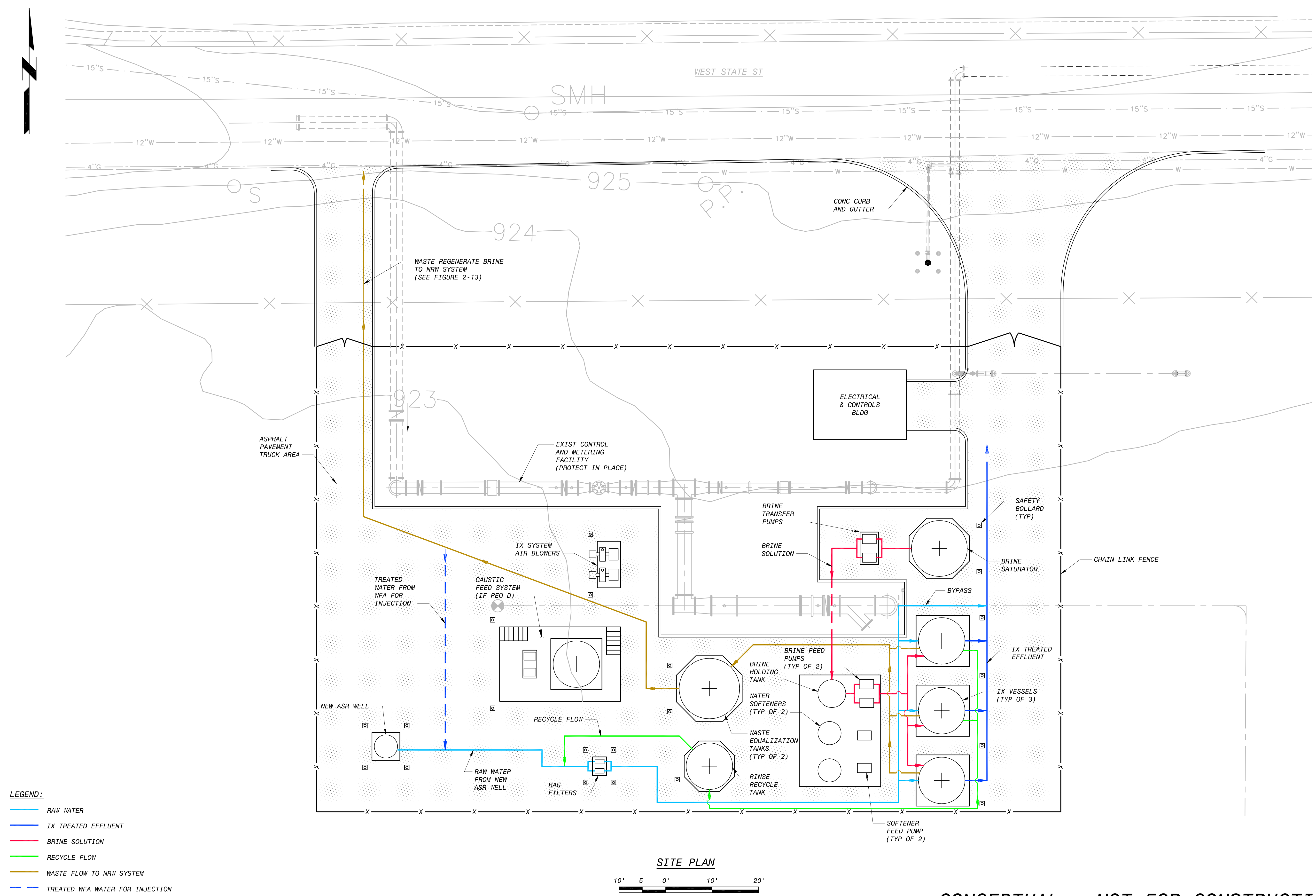
Components	New ASR Well IX Facility	New Well No. 2 IX Facility	Well No. 4 and 27 IX Facility
New Well	1 (ASR)	1 (Production)	
IX Vessels (Standby Included)	3	3	4
Salt Saturator Tank and Transfer Pumps	1	1	2
Water Softener System	1	1	1
Waste Equalization Tank	1	2	2
Recycle Storage Tank and Transfer Pumps	1	1	1

Figures 2-7, 2-8, and 2-9 present conceptual site layouts for the new IX facilities. Raw water from the new wells would be conveyed to the IX vessels, where a portion of the flow would be diverted as a bypass stream. Treated water from the IX vessels would be blended with the bypass stream, sent to disinfection, and delivered to a treated water storage tank and/or the distribution system.

A salt saturator and water softener would be required. Together, these facilities would produce the concentrated brine solution required for resin regeneration. This concentrated brine solution would be conveyed to the IX vessels via transfer pumps where the waste regenerate and rinse flows would be conveyed to either the waste recycle tank or the waste equalization tank. Flows from the waste equalization tank would be delivered to the NRWS line via gravity flow. Water from the downflow backwash, compact bed, and drain freeboard steps would be conveyed to the recycle tank. Recycle flows would be returned to the raw water feed stream via two transfer pumps.

2.4.7.3 Demolition

Some of the facilities at the Well No. 2 IX Facility would be demolished. The existing building would remain in place and the old Well No. 2 motor and electrical equipment would be cleared from the building. It was assumed that the existing booster station would either remain in service or be demolished and salvaged under a separate project. The requirement for a new booster station will be examined during detailed design, when a more detailed hydraulic analysis is performed. The existing 50-foot diameter steel reservoir would remain in place. Portions of the existing paving would need to be demolished and portions of the existing yard piping modified to install underground electrical and piping facilities. The out-of-service Well No. 2 pump would be removed. The well itself would be capped, but would not be filled and abandoned as it has the potential of being used for a monitoring well.



CONCEPTUAL - NOT FOR CONSTRUCTION

CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

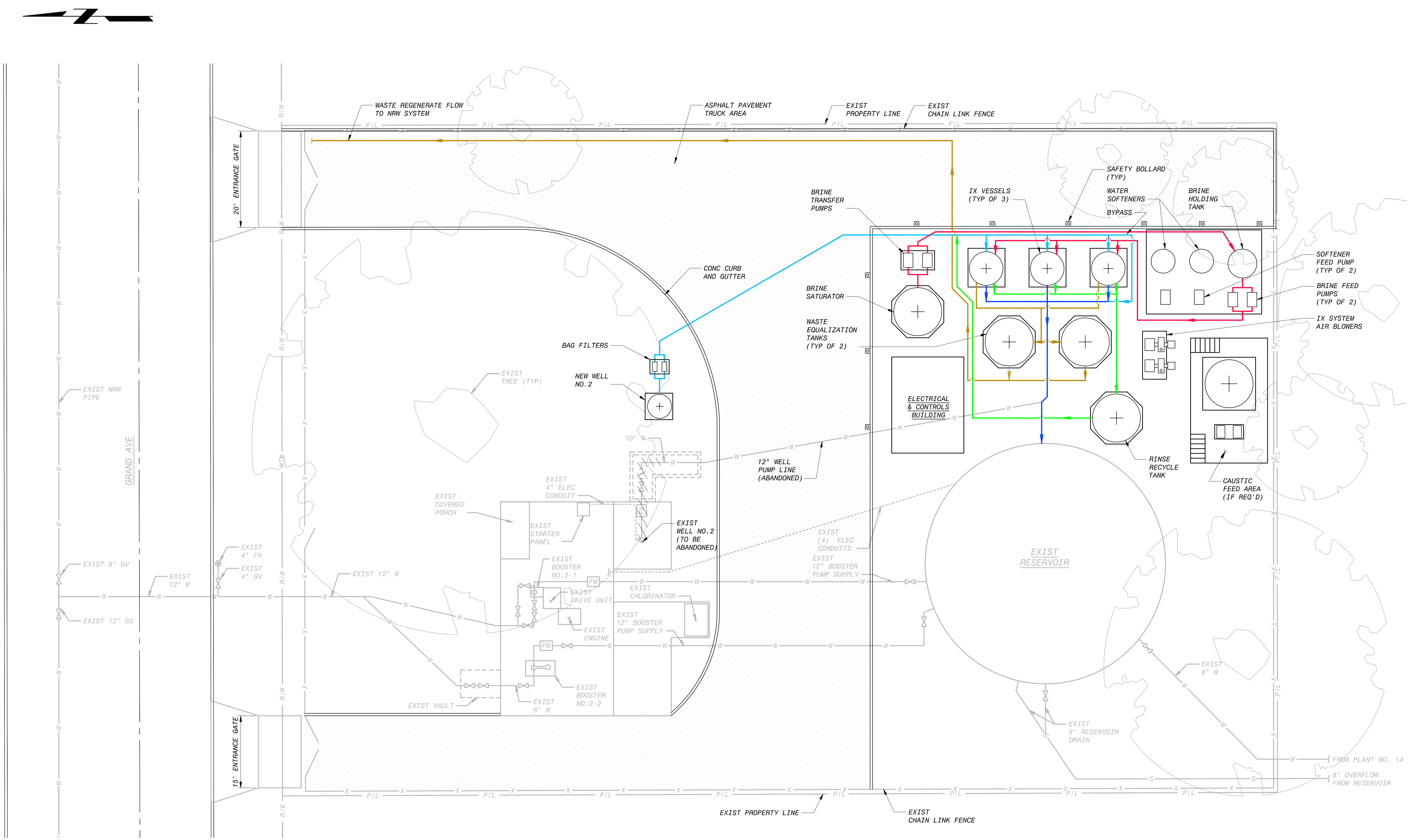
MONTE VISTA WATER DISTRICT
NEW ASR WELL IX FACILITY
SITE AND YARD PIPING PLAN

BLACK & VEATCH
Black & Veatch Corporation
Irvine, California

DESIGNED: JSH
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 2-7



SITE PLAN

10' 5' 0' 10' 20'

1" = 20'-0"

LEGEND:

- RAW WATER
- IX TREATED EFFLUENT
- BRINE SOLUTION
- RECYCLE FLOW
- WASTE FLOW TO NRW SYSTEM

CONCEPTUAL - NOT FOR CONSTRUCTION

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XREF3:
XREF4:
XREF5:

BLACK & VEATCH
Black & Veatch Corporation
Irvine, California

CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

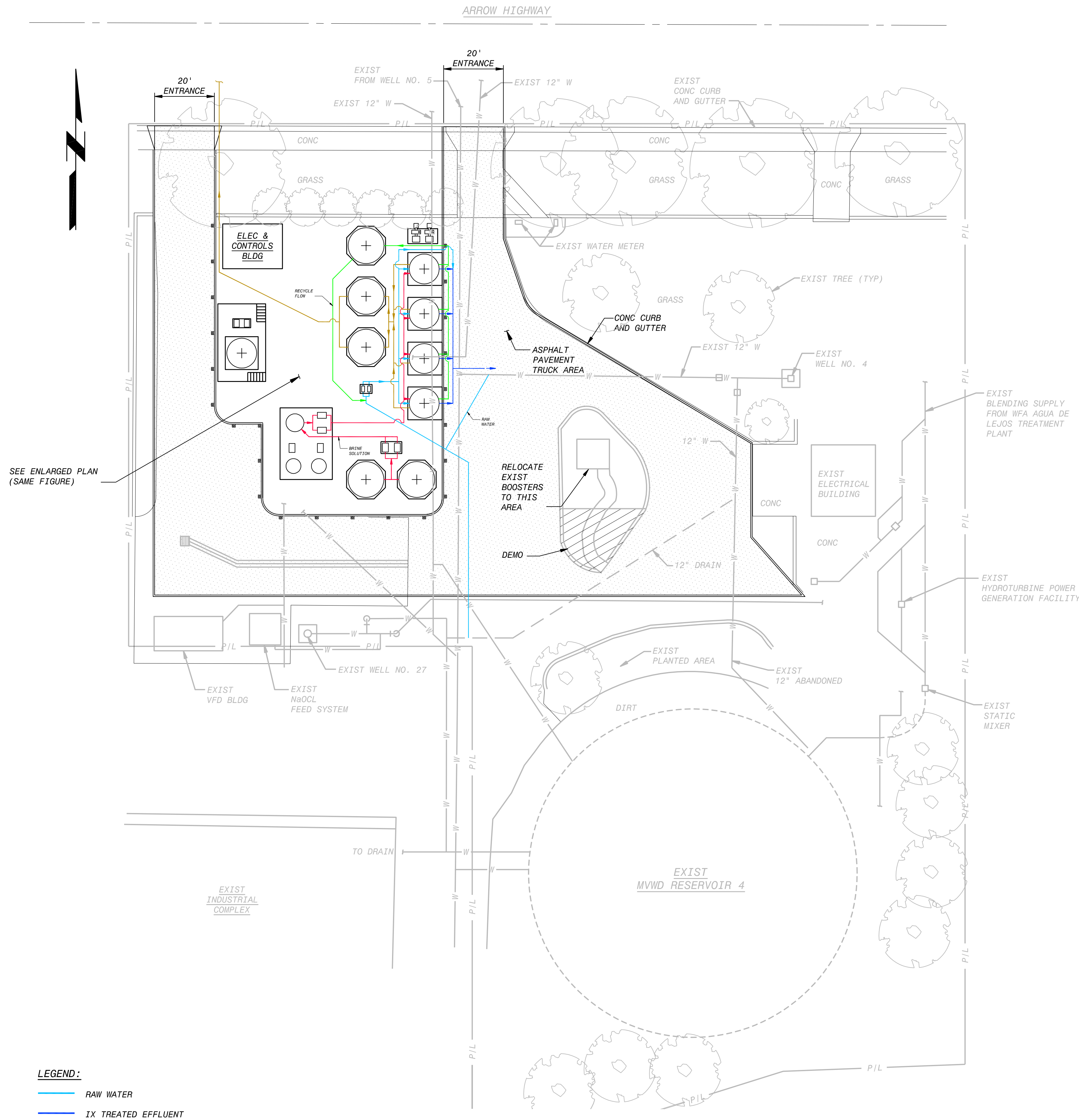
MONTE VISTA WATER DISTRICT
WELL NO. 2 IX FACILITY
SITE PLAN AND YARD PIPING

DESIGNED: JSH
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 2-8

FD160374
D160374



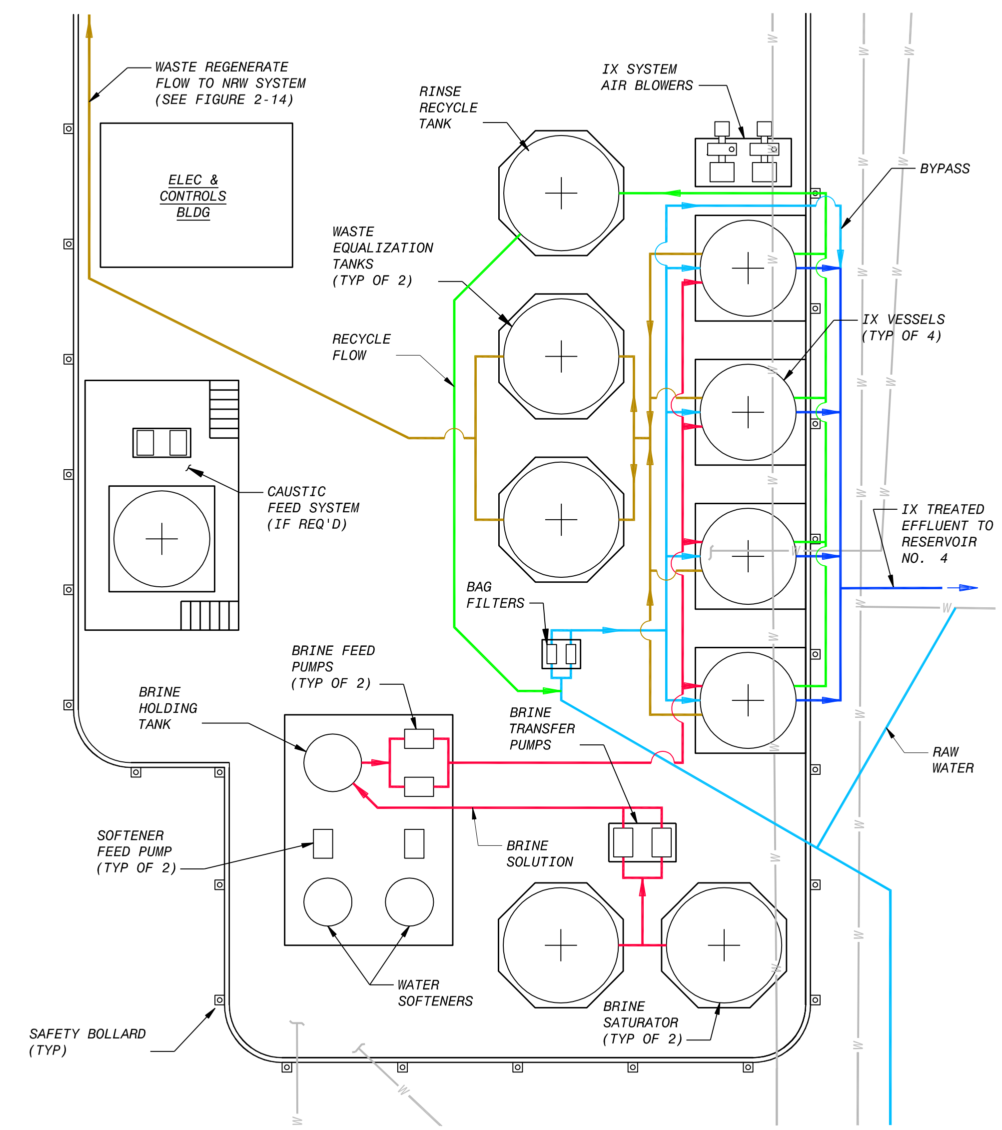
LEGEND:

- RAW WATER
- IX TREATED EFFLUENT
- BRINE SOLUTION
- RECYCLE FLOW
- WASTE FLOW TO NRW SYSTEM

SITE PLAN

20' 10' 0' 20' 40'

1" = 40'-0"



IX FACILITY PLAN

10' 5' 0' 10' 20'

1" = 20'-0"

CONCEPTUAL - NOT FOR CONSTRUCTION

DATE	REVISIONS AND RECORD OF ISSUE	NO.	BY	CK	APP

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XREF5:

BLACK & VEATCH
Black & Veatch Corporation
Irvine, California

CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

MONTE VISTA WATER DISTRICT
WELL NOS. 4 & 27 IX FACILITY
SITE AND YARD PIPING PLAN

DESIGNED: JSH
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 2-9

FD160374
D160374

Demolition at the Well No. 4 and 27 IX Facility would be minimal. The existing well facilities and buildings as well as the chlorination facilities would remain in place. An abandoned well and an abandoned vault on-site may need to be demolished and removed depending on existing site conditions. Portions of the existing paving would need to be demolished and portions of the existing yard piping modified to install underground electrical and piping facilities. The existing booster station located in the northeast corner of the site would need to be relocated to make room for the IX facility.

The New ASR Well IX Facility site contains an existing control and metering facility, which consists of pipe ranging from 8 to 42 inches in diameter, electrical and I&C equipment, pumping and metering equipment, and asphalt paving. The metering facility must remain in service and protected in place during construction.

The New ASR Well site has space constraints due to existing above-grade piping for the new IX and well facilities.

2.4.7.4 Site Improvements

The process areas and ancillary support equipment would likely be constructed on concrete slabs having shallow foundations. This is possible since the system hydraulics of the well production lines on site are assumed to have sufficient head to drive the treatment processes.

The process areas could be exposed, covered, or completely enclosed depending on project economics. The decisions to determine whether a canopy or a building is required would be addressed by MVWD once planning level cost opinions are developed.

Driveway access and site vehicle access would be required for maintenance vehicle access, bulk salt deliveries, and bulk chemical deliveries. Most likely, a paved connection to adjacent streets would allow access to the site. Where access is provided from paved roads, storm drainage would be provided. The surface contours on the sites are relatively flat and would not involve major grading.

2.4.8 Electrical Requirements

The extent of the additional electrical demand for the IX facilities would be largely dependent on the system hydraulics. A feed water pumping system could be eliminated if the well pumps were sized such that enough head is provided to drive the water through the IX process. The electrical loads would include well pumping panels, site lighting, chemical feed equipment, mixing, and heating, ventilation, and air conditioning (HVAC) (if the building enclosure option were selected), etc.

2.4.9 Instrumentation & Control Requirements

Given the size and requirements of the proposed IX systems, the IX system controls packages would likely be furnished by a qualified original equipment manufacturer (OEM) under the general contractor. This will provide the opportunity for the controls packages to be specified using MVWD standard programmable logic controller (PLC) hardware and human/machine interface (HMI) software.

Monitoring equipment (including analyzers) would be provided in the final design in conformance with CDPH requirements. Additional process monitoring equipment (including analyzers, flow meters and pressure transducers) would likely be required for operators to control operations and gauge system performance.

2.5 Conveyance Piping

Conveyance piping would include on-site raw water piping, on-site finished water piping, and the waste regenerate piping from the IX facility to the NRWS pipeline. Figures 2-7, 2-8, and 2-9 also present the general yard piping layouts.

2.5.1 Raw Water Piping

2.5.1.1 New Facilities

New ASR Well IX Facility
Approximately 40 feet of 12-inch diameter raw water piping would convey groundwater from the new ASR well to the IX treatment facility. All new piping would be on-site.

Well No. 2 IX Facility
Approximately 40 feet of 8-inch diameter raw water piping would convey groundwater from new Well No. 2 to the IX treatment facility. All new piping would be on-site.

Well Nos. 4 and 27 IX Facility
Approximately 50 feet of 12-inch diameter raw water piping would convey groundwater from existing Well No. 27 to the IX treatment facility. Approximately 230 feet of 8-inch diameter raw water piping would convey groundwater from existing Well No. 4 discharge piping to the IX treatment facility. All new piping would be on-site.

2.5.1.2 Existing Facilities

New ASR Well IX Facility
Currently no raw water piping exists to convey water from the well to the IX facility.

Well No. 2 IX Facility
Well No. 2 has an existing 12-inch line that currently conveys water from the well discharge to the on-site reservoir. Two (2) 12-inch lines discharge from the reservoir and into the booster station onsite. The two 12-inch booster station discharge lines connect to a common line, which runs to an existing service main along Grand Avenue.

Well Nos. 4 and 27 IX Facility

The 12-inch discharge lines from Well 4 and 27 both split into two 12-inch lines. One line from each well connects to the existing distribution system, and one line connects to the other well line before connecting to the reservoir inlet. An on-site hydro-power generation system takes water from the WFA WTP and converts the excess head into power. After the head is broken, the discharge from the hydro power facility connects to a common header with the Well 4 and 27 common line before discharging into the on-site reservoir or distribution to Chino Hills. Water from the reservoir outlet is boosted into the existing distribution system using an on-site booster station.

Existing Well No. 4 discharge piping will need to be modified to tie into the new plant.

2.5.1.3 Operations

The proposed operations include redirection of the flow from the raw water wells to the new IX facilities. Treated water would be blended with a raw water bypass stream and injected with chlorine on-site for disinfection.

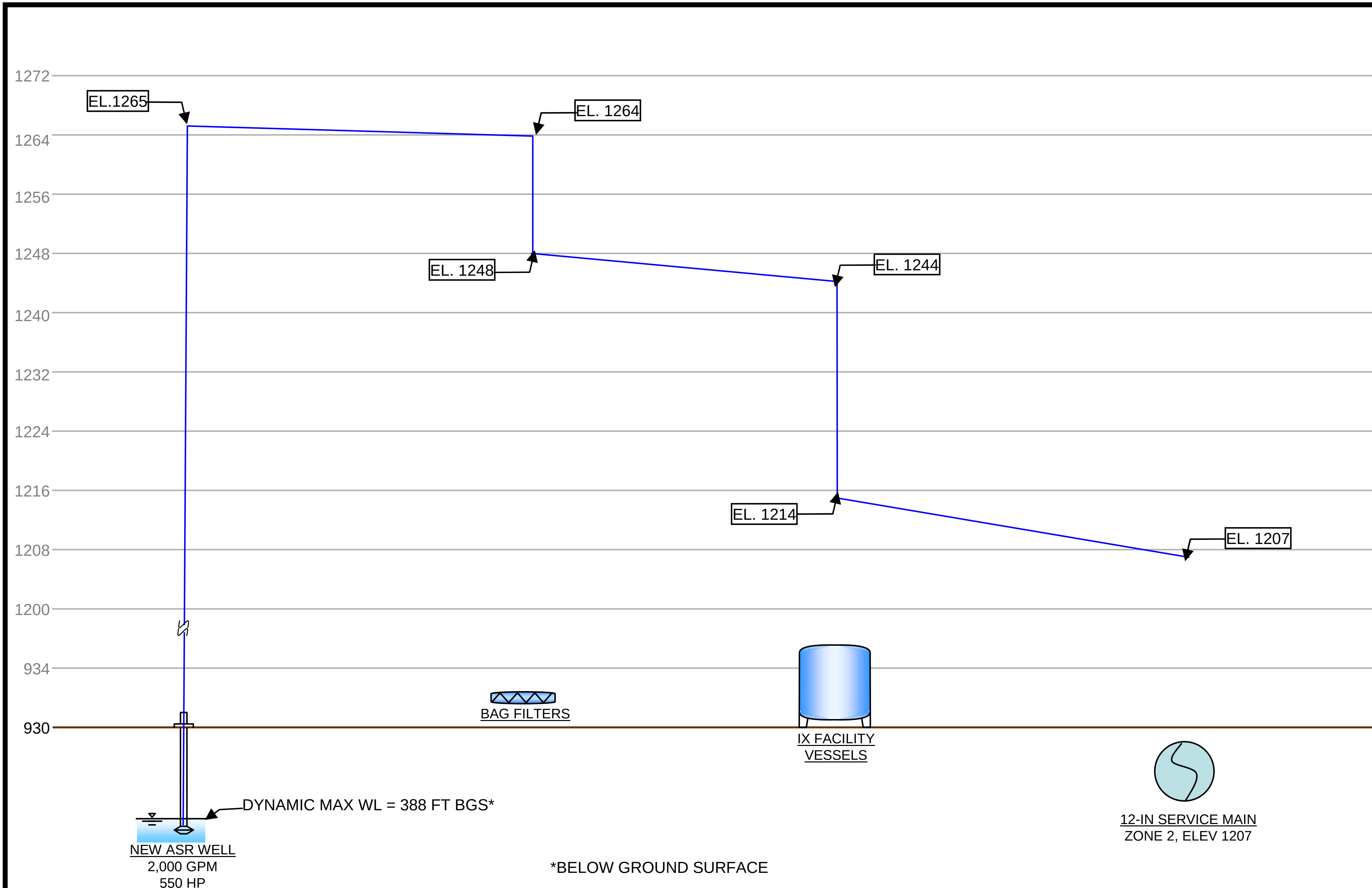
2.5.1.4 Hydraulic Conditions

New ASR Well IX Facility

The proposed ASR well would be sized such that it would pump groundwater through the IX Facilities and into the existing distribution system. The total dynamic head (TDH) required would include the head needed to pump the groundwater to the ground surface and to the maximum pressure in MVWD Pressure Zone 2, which operates at 1207 ft. Losses attributed to pipe friction, specials (bends, valves, flowmeters, etc.), bag filters, and the IX resin bed would be accounted for in the TDH requirement. The TDH required for the New ASR Well IX Facility would be 724 ft. A 550 HP well motor would be required to achieve this TDH. A conceptual hydraulic profile for the new facility is shown on Figure 2-10.

Well No.2 IX Facility

The proposed new Well No. 2 would be sized such that it would pump groundwater through the IX Facilities and into the on-site reservoir. The TDH required would include the head needed to pump the groundwater to the ground surface and to the reservoir maximum water level of 943 ft. Losses attributed to pipe friction, specials (bends, valves, flowmeters, etc.), bag filters, and the IX resin bed would be accounted for in the TDH requirement. The TDH required for the new Well No. 2 IX Facility would be 429 ft. A 200 HP well motor would be required to achieve this TDH. A conceptual hydraulic profile for the new facility is shown on Figure 2-11.



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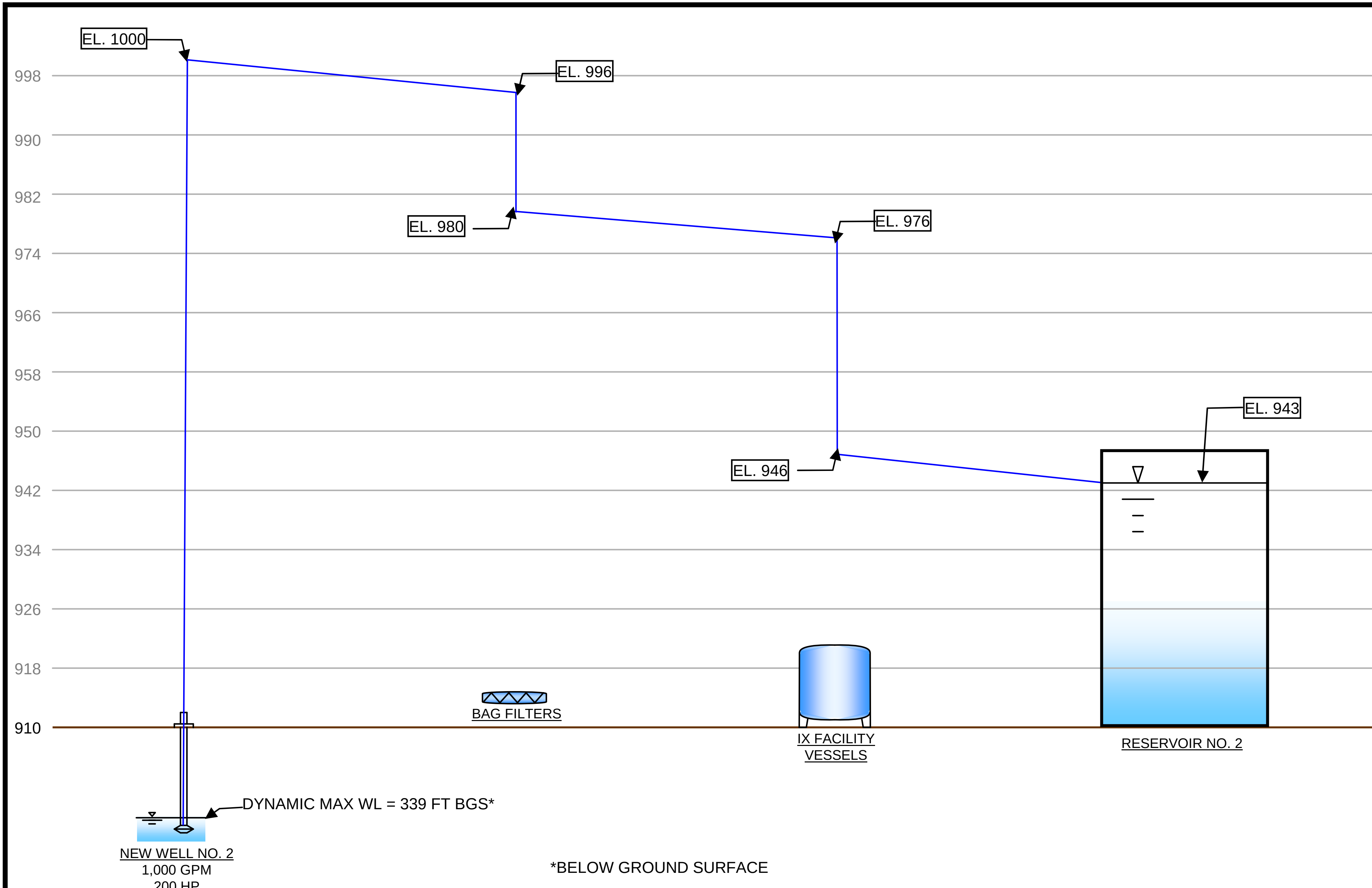
Chino Basin Dry Year Yield Program Expansion Project

MVWD New ASR Well IX Facilities – Hydraulic Profile

Figure 2-10



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MVWD New Well No. 2 IX Facilities – Hydraulic Profile

Figure 2-11

Well No. 4 and 27 IX Facility
A preliminary hydraulics analysis showed that both Wells 4 and 27 will provide enough lift to pump the groundwater through the IX Facilities and into the on-site reservoir. The TDH required included the head needed to pump the groundwater to the ground surface and to the reservoir maximum water level of 1,208 ft. Losses attributed to pipe friction, specials (bends, valves, flowmeters, etc.), bag filters, and the IX resin bed were accounted for in the TDH requirement. The TDH required for the Well No. 4 and 27 would be 659 ft and 720 ft, respectively. The available TDH in Well No. 4 and 27 is 713 ft and 752 ft, respectively. A more detailed hydraulic analysis should be performed during detailed design to verify that Wells 4 and 27 are capable of meeting the pumping requirements of the new facilities. A conceptual hydraulic profile for the new facility is shown on Figure 2-12.

2.5.1.5 Pipe Material

Any new yard piping would either be concrete mortar lined and coated (CML&C) or concrete mortar lined and wrapped (CML&W) steel. Buried piping would have polyethylene wrap and may be concrete encased in specific areas.

2.5.2 Finished Water Piping

New ASR Well IX Facility
Approximately 250 feet of 12-inch diameter finished water piping would convey treated water to an existing water pipeline located in State Street.

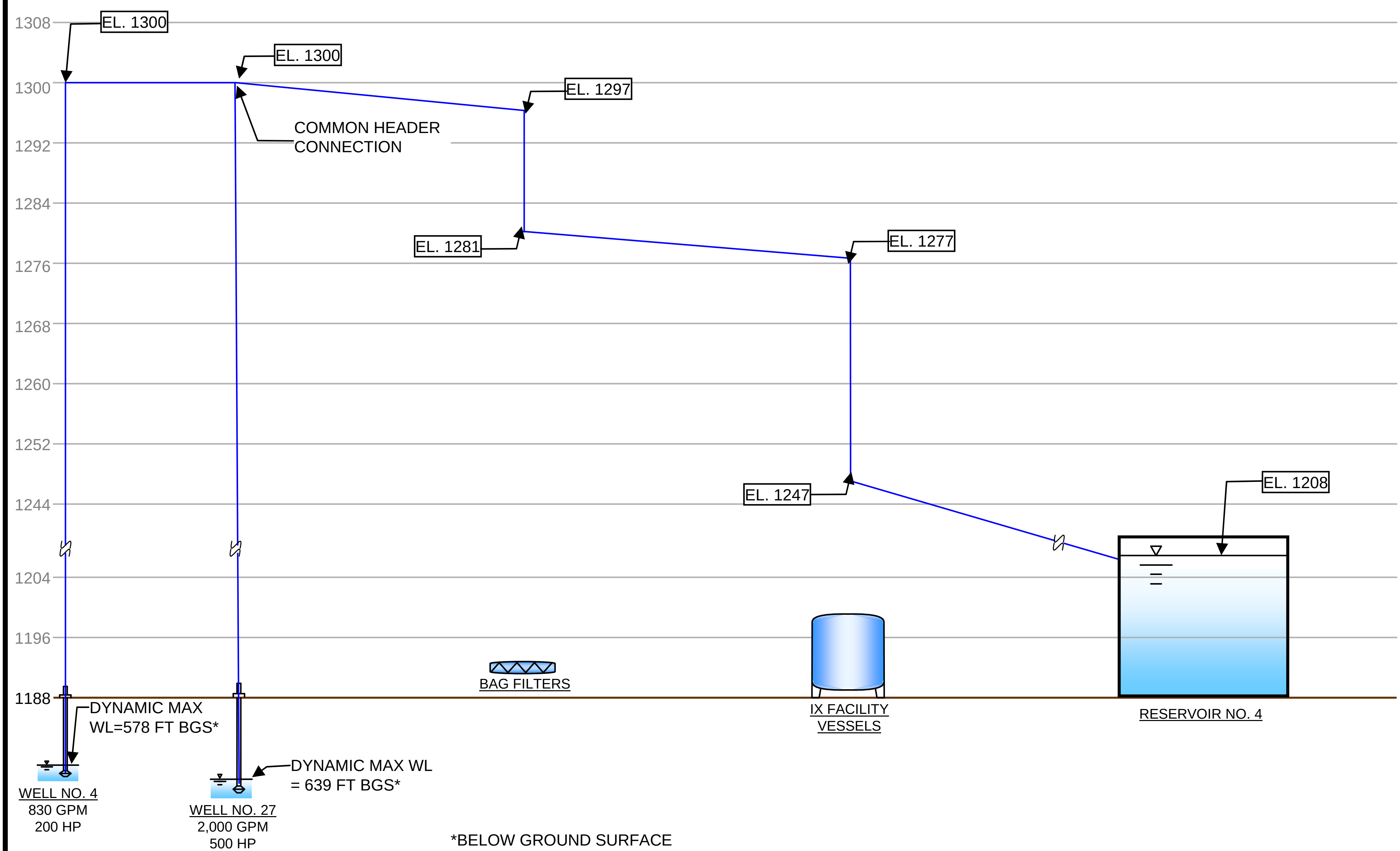
Well No.2 IX Facility
Approximately 100 feet of 8-inch diameter finished water piping would convey treated water to an existing reservoir on-site, where it would be released into the system via existing pipe.

Well No. 4 and 27 IX Facility
Approximately 230 feet of 14-inch diameter finished water piping would convey treated water to an existing reservoir on-site, where it would be released into the system via existing pipe.

2.5.3 Waste Regenerate Piping

Based on information provided by local IX equipment manufacturers, all IX facilities should be provided with dual waste regenerate lines. The waste regenerate lines would be the same diameter and installed in the same trench. The lengths listed below include the total length of the waste regenerate piping. The length of the trench in which the piping would be installed is half of the length listed below.

New ASR Well IX Facility
Approximately 10,300 feet of new 6-inch diameter waste regenerate piping would convey the waste regenerate by gravity from the waste equalization tank of the New ASR Well IX facility south along Ramona Avenue to the NRWS pipeline in Phillips Boulevard. The piping was assumed to be on existing MVWD property or in public right-of-way. Figure 2-13 shows the waste regenerate pipeline plan and profile.



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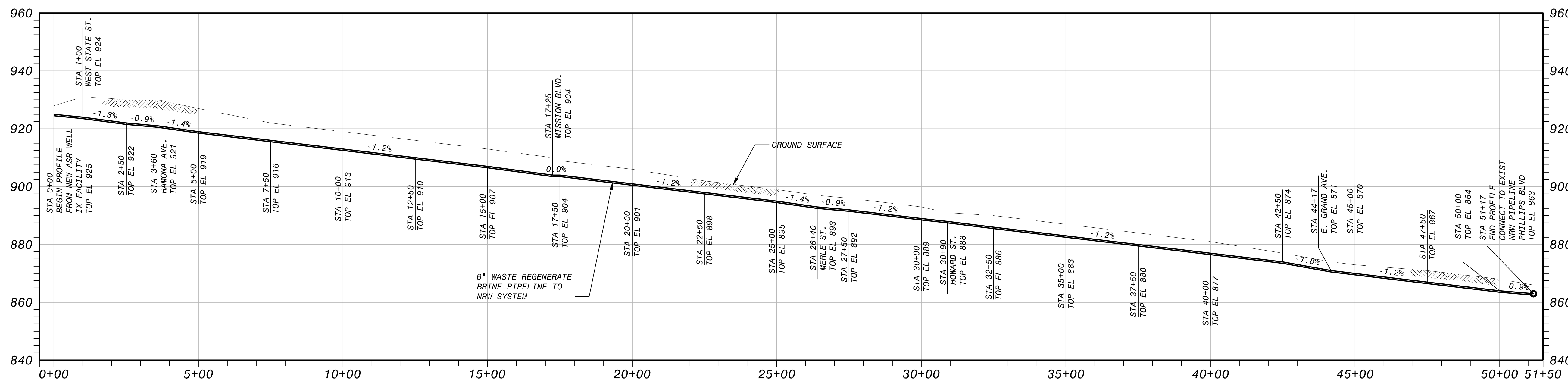
Chino Basin Dry Year Yield Program Expansion Project

MVWD Well Nos. 4 & 27 IX Facilities – Hydraulic Profile

Figure 2-12

PLAN STA 0+00 TO STA 51+17
HORIZ - 1" = 400'



PROFILE STA 0+00 TO STA 51+17
HORIZ - 1" = 400'
VERT - 1" = 40'

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CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

MONTE VISTA WATER DISTRICT
NEW ASR WELL IX FACILITY NRW
PLAN AND PROFILE STA 00+0 TO STA 51+17

DESIGNED: JSH
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 2-13

Well No. 2 IX Facility

Approximately 400 feet of new 6-inch diameter waste regenerate piping would convey the waste regenerate by gravity from the waste equalization tank of the Well No. 2 IX facility to the NRWS pipeline in Grand Avenue adjacent to the IX facility property. The piping was assumed to be on existing MVWD property or public right-of-way. A plan and profile is not shown for this brine pipeline due to its short length and connection to the existing NRWS pipeline is immediately adjacent to the site.

Well No. 4 and 27 IX Facility

Approximately 11,000 feet of new 6-inch diameter waste regenerate piping would convey the waste regenerate by gravity from the waste equalization tank of the Well No. 4 and 27 IX facility west along Arrow Highway to Central Avenue and then south along Central Avenue to the NRWS pipeline in Palo Verde Street. The piping was assumed to be on existing MVWD property or in public right-of-way. Figure 2-14 shows the waste regenerate pipeline plan and profile. An alternative alignment for the waste regenerate piping would be to follow Arrow Highway east and connect to the NRWS pipeline in North Mountain Avenue. The alignment is slightly shorter than the one that follows Central Avenue and pumping would be required since the discharge point is approximately ten feet higher then the IX facility. The alternative alignment could be used if the Central Avenue alignment is eliminated due to congestion of existing facilities in Central Avenue and the required under crossing of Interstate 10.

2.5.3.1 Operation & Hydraulic Conditions

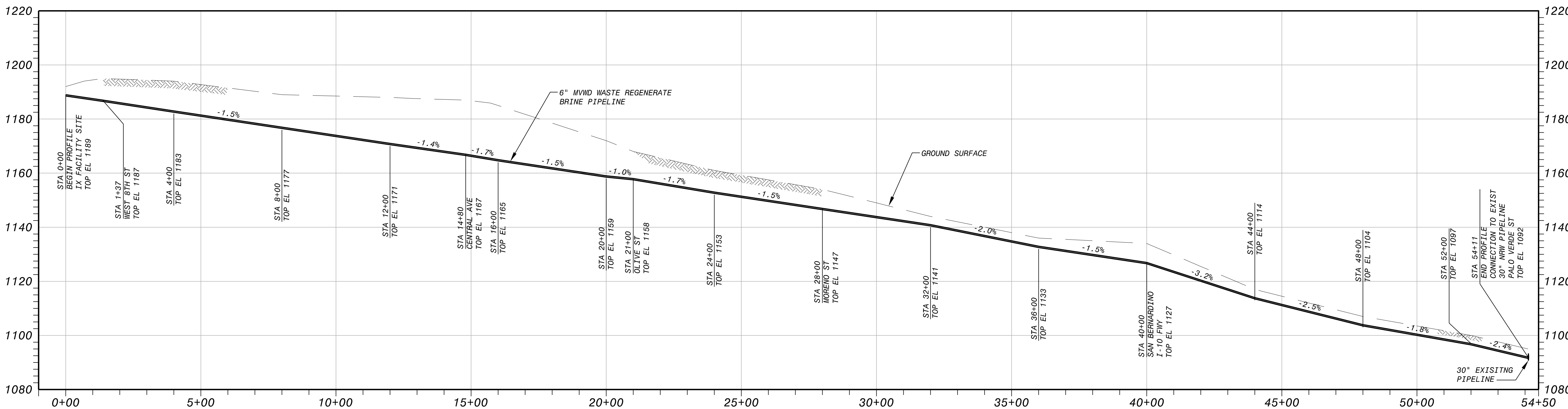
Since there is adequate elevation difference between the IX Facility sites and the NRWS tie-in points, the waste regenerate pipelines would be able to operate under gravity from the IX plants to IEUA's NRWS system. The hydraulic conditions of the pipelines are summarized in Table 2-15.

Table 2-15
Waste Regenerate Pipeline Hydraulic Conditions

Criteria	Option A		Option B
	New ASR Well IX Facility	Well No. 2 IX Facility	Well No. 4 and 27 IX Facility
IX Facility			
Location	West State St. & Ramona Ave.	Grand Ave. & West Ramona Place	Vernon Ave. & Arrow Hwy
Site Elevation, feet(1)	929	882	1191
NRWS Line			
Location	Ramona Ave. & Phillips Blvd.	Grand Ave. adjacent to site	Central Ave. & Palo Verde St.
Connection Invert Elevation, feet	863	880	1092
Hydraulic Conditions			
Elevation Difference, feet	-66	-2	-99
Minimum Slope	0.012	0.009	0.015
Pipeline Diameter, inches	6	6	6
Pipeline Length, feet(2)	9,400	400	11,000

Notes:
(1) Elevation above mean sea level based upon USGS maps.
(2) Total pipeline length of dual waste regenerate lines.

PLAN STA 0+00 TO STA 54+11
HORIZ - 1" = 400'



PROFILE STA 0+00 TO STA 54+11
HORIZ - 1" = 400'
VERT - 1" = 40'

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CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

MONTE VISTA WATER DISTRICT
WELL 4 & 27 IX FACILITY
NRW PLAN AND PROFILE

DESIGNED: JSH
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 2-14

2.5.3.2 Pipe Material

The proposed pipe material for the waste regenerate pipeline is polyvinyl chloride (PVC) sewer pipe. An alternative pipe material is high density polyethylene (HDPE). Pipe materials used at major crossings would be selected based on the type of construction as well as design requirements of the permitting agency.

3.0 GROUNDWATER PRODUCTION WELL

3.1 Overview

This chapter describes the location and facilities for the replacement of an out of service and non-operational MVWD groundwater production well. Replacing MVWD Well No. 2 would enable MVWD to meet its proposed DYY Program Expansion shift obligation. Abandoned Well No. 2 is located in the City of Ontario, south of Grand Avenue and west of West Ramona Place. Figure 3-1 presents a vicinity map for the well location.

3.2 Groundwater Supply and Water Quality

3.2.1 Historical Groundwater and Operating Conditions

Historic groundwater elevations and operating conditions from 2007 were investigated for Chino Wells 10 and 12 to approximate the static and dynamic groundwater elevations for the New Well No. 2. The information presented in the following chapters was derived from the WEI database of annual operating records and from information provided by MVWD and the Watermaster as well as pump test data provided by Chino.

Table 3-1 presents the historic groundwater elevations for Chino Wells 10 and 12. The static groundwater levels for the proposed well would be approximately 280 to 300 feet below ground surface. The dynamic groundwater elevation is approximately 356 feet below ground surface. Available data from historical use of Chino Wells 10 and 12 was reviewed to confirm the production rate, drawdown, specific capacity and screened interval for the proposed replacement well. The data in Table 3-1 was used to develop the operating conditions of the replacement well as shown in Table 3-2.

Table 3-1
Historical Operating Conditions(1)

Operating Conditions	Well No. 10	Well No. 12
Site Elevation, feet above mean sea level (amsl)	890	890
Production Capacity, gpm	1,087	2,225
Est. Avg. Static Groundwater Elev., ft bgs(2)	286	290
Estimated Average Drawdown, feet(3)	53	66
Approximate Specific Capacity, gpm/ft(4)	20	34

Notes:
(1) Historical operating conditions listed in table are based on actual pump test data conducted in September 2007 and provided by WEI, 2008.
(2) Feet, below ground surface (bgs).
(3) Drawdown is the difference between static and dynamic groundwater elevations.
(4) Gallons per minute per foot of drawdown.

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Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District – Well No. 2 Vicinity Map

Figure 3-1

3.2.2 Expected Operating Conditions and Well Performance

Table 3-2 presents the anticipated operating conditions and performance for new Well No. 2 based on the data from Table 3-1. For planning purposes, the new well was assumed to have a production capacity of approximately 1,000 gpm.

Table 3-2
Anticipated Operating Conditions

Conditions	Well No. 2
General Conditions	
Basis for Operating Conditions, Well No.	Chino 12
Distance from Basis Well Above, feet	3,075
Location (Intersection)	Grand/ W. Ramona Pl.
Site Elevation, feet amsl(1)	910
Well HGL/Delivery Zone, feet amsl	943 / Zone 3
Operating Conditions	
Production Capacity, gpm	1,000
Est. Avg. Static Groundwater Elev., ft bgs	310(2)
Assumed Specific Capacity, gpm/ft	34
Calculated Estimated Drawdown, feet	29

Notes:
(1) Above mean sea level (amsl).
(2) Well No. 2 site is at a higher elevation than Chino 12, which resulted in a greater depth to static groundwater.

3.2.3 Anticipated Water Quality

Based on water quality data from nearby wells provided by WEI and discussed in Chapter 2, the expected maximum nitrate concentration as NO_3 is approximately 56 mg/L. Anticipated total dissolved solids (TDS) concentrations in Well No. 2 would range from 327 mg/L – 371 mg/L. Wellhead treatment would be required as discussed in Chapter 2. It is recommended that water quality testing for drinking water, pursuant to the California Code of Regulations Title 22, be conducted as soon as possible to establish the constituents of concern and confirm the wellhead treatment strategy.

3.3 Well Drilling and Development

Before new Well No. 2 can be drilled and developed, abandoned Well No. 2 equipment would be demolished and the well hole itself capped. If left in place unfilled, this well could serve as a monitoring well, if desired. A new pilot bore hole would be drilled within a reasonable distance from the newly filled hole and then reamed to the specified diameter. Selection of screening elevation and seal depths would be determined during final design and the drilling operation.

Casing would be installed the full length of the well and would be copper-bearing steel, with a minimum wall thickness of 5/16-inch. Total length of louvered casing (i.e., screening) and the depth interval where it would be installed would be determined during final design. Gravel pack would be installed along entire length of screening depth interval. A cement grout seal would be installed from ground level to a minimum specified depth.

Requirements for a sounding pipe, permanent gravel feed line, or air vent tube would be evaluated during final design.

3.4 Well Facilities and Wellhead Equipment

New wellhead facilities would be provided including a wellhead pump and motor and electrical and control equipment. The existing discharge and blow-off piping would be utilized to the extent possible depending upon an evaluation of their condition.

3.4.1 Well Pump and Motor

The wellhead pump would be a multistage vertical turbine with an electric motor located above ground. The drive shaft would be water lubricated, and a pre-lubrication of the line shaft bearings would be provided during the pump startup. To proceed with preliminary design, pump performance design criteria were developed for the expected production as presented in Table 3-3.

Table 3-3
Assumed Pump Performance

Description	New MVWD-2
Pump	
Type	Deep Well Turbine
Capacity, gpm	1,000
Total Dynamic Head, feet(1)	429
Pump Efficiency, percent	80
Motor Efficiency, percent	90
Discharge Column Diameter, in	10
Motor	
Type	TEFC High-Efficiency(2)
Nominal Motor Horsepower, HP	200
Motor Drive	AFD(3)

Notes:
(1) Includes frictional losses and mechanical shaft losses.
(2) TEFC - Totally enclosed fan cooled.
(3) AFD - Adjustable frequency drive.

3.4.2 Discharge and Blow-off Piping

The existing wellhead piping and appurtenances would be salvaged to the maximum extent possible. Segments of pipe, valves, and flowmeters should be inspected for possible damage or wear and replaced if necessary. A new 10-inch connection would be needed to connect the well discharge piping to the new IX Facility. The blow-off piping would be utilized for discharge to the site waste pipeline during startup.

3.5 Disinfection Facilities

Disinfection would be required to satisfy chlorine demand and residual. For the purposes of this PDR and preparing cost estimates, it has been assumed that a sodium hypochlorite system similar to what is in use by MVWD at other sites would be the recommended disinfection alternative for the Well No. 2 IX Facilities. However, decisions on the disinfection methodology will ultimately require re-examination during the final design stage.

3.6 Conveyance Piping

Conveyance piping includes on-site raw water piping and on-site finished water piping. The sections below provide a brief summary of the facilities discussed in detail in Chapter 2.

3.6.1 Description of Existing Facilities

Well No. 2 has an existing 12-inch line that currently conveys water from the well discharge to the on-site reservoir. Two (2) 12-inch lines discharge from the reservoir and into the booster station on-site. The two 12-inch booster station discharge lines connect to a common line, which runs to an existing service main in Grand Avenue.

3.6.2 Raw Water Piping

Approximately 40 feet of 8-inch diameter raw water piping would convey groundwater from new Well No. 2 to the IX treatment facility. All new piping would be on-site and would be steel CML&C and steel CML&W. Buried piping will have polyethylene wrap and pipe beneath roads, concrete pads, or other facilities would be concrete encased, as required.

3.6.3 Operations and Hydraulic Conditions

The proposed operations would include redirection of the flow from the raw water wells to the new IX facilities. Treated water would be blended with a raw water bypass stream and chlorinated on-site for disinfection.

The proposed new Well No. 2 would be sized such that it would pump groundwater through the IX facility and into the on-site reservoir and/or the existing distribution system. The TDH required would include the head needed to pump the groundwater to the ground surface and to the reservoir maximum water level at 943 feet. It was assumed that the well pump would be sized to boost the water to the on-site reservoir, and the existing booster station would boost the water into the distribution system. Losses attributed to pipe friction, specials (bends, valves, flowmeters, etc.), bag filters, and the IX resin bed would be accounted for in the TDH requirement. The TDH required for the new Well No. 2 IX Facility is 429 ft. A 200 HP well motor would be required to achieve this TDH.

4.0 AQUIFER STORAGE AND RECOVERY WELL

4.1 Overview


The New ASR Well would be located on the south west corner of the property.

This chapter describes the location and facilities for a new MVWD ASR well. MVWD is planning to construct the well to inject and extract water when needed to meet the additional dry-year shift under this expanded Program. The new ASR well will be constructed on existing MVWD property as shown on Figure 4-1. The site is located in the City of Montclair, on the southeast corner of West State Street and Ramona Avenue and has space constraints due to existing metering facility piping located above and below grade. The site is a candidate to construct an ASR well due to the availability of imported water from the Water Facilities Authority's (WFA) Agua de Lejos Water Treatment Plant (WTP) for injection and the proximity of an existing 12-inch transmission main along State St. on the north side of the property. The property will also have adequate drainage facilities where the waste flows could be conveyed into an existing concrete drain box located on State St.

4.2 Groundwater Supply and Water Quality

4.2.1 Historical Groundwater and Operating Conditions

Historic groundwater elevations and operating conditions were investigated from existing Chino Wells 10 and 12 in the vicinity of the proposed well location. The information presented in the following chapters was derived from the WEI database of annual operating records and from information provided by MVWD and the Watermaster as well as pump test data provided by Chino.

Table 4-1 presents the historic groundwater elevations for existing Chino Wells 10 and 12, which are located in the vicinity of the new ASR well site. Based on the data presented in the table, the static groundwater levels for the proposed ASR well would be approximately 320 to 340 feet below ground surface. Dynamic groundwater levels would be approximately 390 ft below ground surface. Available data from pump tests was reviewed to estimate the production rate and specific capacity for the proposed new well. The data in Table 4-1 was used to develop the anticipated ASR well operating conditions listed in Table 4-2.

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Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District – New ASR Well Vicinity Map

Figure

4-1

Table 4-1
Historical Operating Conditions(1)

Operating Conditions	Well No. 10	Well No. 12
Site Elevation, feet above mean sea level (amsl)	890	890
Production Capacity, gpm	1,087	2,225
Est. Avg. Static Groundwater Elev., ft bgs(2)	286	290
Estimated Average Drawdown, feet(3)	53	66
Approximate Specific Capacity, gpm/ft(4)	20	34

Notes:
(1) Historical operating conditions listed in table are based on actual pump test data conducted in September 2007 and provided by WEI, 2008.
(2) Feet, below ground surface (bgs).
(3) Drawdown is the difference between static and dynamic groundwater elevations.
(4) Gallons per minute per foot of drawdown.

4.3 Expected Operating Conditions and Well Performance

ASR wells are intended to operate as injection wells until the required amount of water is stored in the aquifer. When additional supplies are needed, ASR wells can reverse operations and extract groundwater from the aquifer as a typical production well. A more in-depth discussion of ASR wells and drawings are provided in Volume I. The anticipated production and injection capacities of the well could be 2,000 gpm and 1,000 gpm, respectively. The ASR well also would be intended to inject higher quality water into an aquifer of lesser quality. Imported water, which is low in nitrates, would gradually dilute and displace high nitrate plumes. Additionally, injection would create localized zones of good quality water at the well site and down gradient of the ASR well. Thus, the well could operate to create a zone, or “bubble,” of better quality water to be recovered at a later time. Table 4-2 provides the anticipated operating condition for MVWD’s new ASR well based on the information shown in Table 4-1.

Table 4-2
Anticipated Operating Conditions

Conditions	New ASR Well
General Conditions	
Basis for Operating Conditions, Well No.	Chino 12
Distance from Basis Well Above, feet	6,500
Location (Intersection)	West State/ Ramona
Site Elevation, feet amsl (1)	929
Well HGL/Delivery Zone, feet amsl	1,207/ Zone 2
Operating Conditions	
Production Capacity, gpm	2,000
Maximum Injection Capacity, gpm	1,000
Est. Avg. Static Groundwater Elev., ft bgs	329(3)
Est. Avg. Injection Head, feet (2)	562
Assumed Specific Capacity, gpm/ft	34
Calculated Estimated Drawdown, feet	59

Notes:
(1) Above mean sea level (amsl).
(2) Addition of static lift and system pressure in delivery zone.
(3) New ASR Well site is at a higher elevation than Chino 12, which resulted in a greater depth to static groundwater.

4.3.1 Anticipated Water Quality

Because the new ASR well has not been drilled, water quality data specific to that well site is not available. However, based on the water quality data available from a nearby well, it is anticipated that the nitrate (as NO_3^-) and TDS concentrations in the proposed well would range from 43- 57 mg/L and 320-330 mg/L, respectively, and wellhead treatment would be required as discussed in Section 2.

4.3.2 Injection Cycle

At the beginning of an injection cycle, water would be pumped to waste for five to ten minutes to clear the supply pipeline of any unwanted debris or sediments that may have accumulated in the pipe over time. Following the waste cycle, a motor operated valve would open to allow the casing pipe to fill. During the injection process, flow rate would automatically be monitored, and a flow control valve would be used to adjust and maintain a given flow rate.

An initial amount of low nitrate imported water would be injected in the basin to serve as a buffer zone between the imported supply and the native groundwater. This initial volume of water should be left in place, but could be removed and treated with the new IX Facility.

Under typical operations, treated imported water would be injected when available over the seven month period from October to April using the new ASR well. Treated imported water would be obtained from an existing 12-inch transmission main located in State Street.

4.3.3 Production/Extraction Cycle

The production/extraction cycle for an ASR well will essentially be the same as the production cycle for a typical municipal production well. Typical operation of the well would include starting the well pump and motor, pumping to waste for five to ten minutes, and then pumping to the distribution system.

Under normal operating conditions, extraction of groundwater would take place during the summer months (May through September) when the ASR facility would reverse operations from the winter and be used as a production well. It is anticipated that most of the water previously stored during the winter would be extracted by the ASR well.

4.3.4 Rehabilitation

Periodic rehabilitation is another important aspect in the operations of ASR wells. Rehabilitation typically occurs on a three-to-five year cycle in which the equipment is removed and the casing cleaned. The time between rehabilitations would be extended by backflushing with a pump or by airlifting the well (injecting high pressure air at the bottom of the well to scour the casing). Airlifting is more typical on injection only wells if a pump has not been installed. The frequency of the backflush would be determined on a site-specific basis and may be as often as 20 minutes every two weeks. The need to backflush would be determined by a decline in injection performance, i.e., lower injection flow rate and increased injection pressure readings.

4.4 Well Drilling and Development

The new ASR well would be drilled in the northwest portion of the site. It is anticipated that this well would be 400 to 450 ft deep and would have a production capacity of 2,000 gpm. A general methodology for well construction is mentioned below.

A pilot bore hole would be drilled and then reamed to the specified diameter. Selection of screening elevation and seal depths would be determined during final design and the drilling operation.

A copper-bearing steel casing would be installed the full length of the well, with a minimum wall thickness of 5/16-inch. Total length of louvered casing (i.e., screening) and the depth interval where it would be installed would be determined during final design. Gravel pack would be installed along entire length of screening depth interval. A cement grout seal would be installed from ground level to a minimum specified depth.

Requirements for a sounding pipe, permanent gravel feed line, or air vent tube would be evaluated during final design.

4.5 Well Facilities and Wellhead Equipment

Wellhead facilities would consist of supply pipelines to and from the system, a wellhead pump and motor, and pump to waste and inject to waste pipelines. In addition, a flow control valve would be required to regulate the pressure and amount of water injected.

4.5.1 Flow Control Valve

A flow control valve can be located either on the surface or below the ground in the well. The surface control valve has the advantage of ease of maintenance and removal. The below ground control valve (downhole control valve) has automatic controls located on the surface, but the valve is located in the well. A downhole valve would minimize air fouling, bio-fouling, and calcite formation of the well by eliminating air entrainment.

4.5.2 Well Pump and Motor

The wellhead pump would be a multistage vertical turbine with an electric motor located above ground. The drive shaft would be water lubricated, and pre-lubrication of the line shaft bearings would be provided during the pump startup. To proceed with preliminary design, pump performance design criteria were developed for the expected production as presented in Table 4-3.

The New ASR Well design would be similar to the existing MVWD ASR Well No. 4, as shown.

Table 4-3
Assumed Pump Performance

Description	MVWD-New ASR Well
Pump	
Type	Deep Well Turbine
Capacity, gpm	2,000
Total Dynamic Head, feet (1)	724
Pump Efficiency, percent	80
Motor Efficiency, percent	90
Discharge Column Diameter, in	12
Motor	
Type	TEFC High-Efficiency
Nominal Motor Horsepower, HP	550
Motor Drive	AFD

Notes:
(1) Includes frictional losses and mechanical shaft losses.

4.5.3 Discharge and Blow-Off Piping

The wellhead piping would include a 12-inch diameter discharge pipe, an 8-inch diameter blow-off pipe, two control valves, a check valve, air release valve, flow meter, and other miscellaneous valves and fittings. The blow-off piping would be utilized for discharge to the site waste pipeline during startup.

4.6 Disinfection Facilities

Disinfection would be required to satisfy chlorine demand and residual. The New ASR Well IX Facilities would require new disinfection facilities. It was assumed that MVWD would prefer to install a sodium hypochlorite feed system for this facility since sodium hypochlorite is used at their existing Well No. 4 and 27 site. Sodium hypochlorite has minimal chemical handling hazards (i.e. scrubbers are not required). Totes can be easily removed from the site during periods when the well and IX facilities are not in use. For the purposes of this study and preparing cost estimates, sodium hypochlorite delivered in totes is the recommended disinfection system for the New ASR Well IX Facilities. However, decisions on the disinfection methodology will ultimately require re-examination during the final design stage.

4.7 Conveyance Piping

Conveyance piping would include on-site raw water piping and on-site finished water piping. The sections below provide a brief summary of the facilities discussed in detail in Chapter 2.

The New ASR Well suction and discharge pipe design would connect above grade.

4.7.1 Description of Existing Facilities

The site has space constraints due to existing metering facility piping located above and below grade. The new ASR well would be drilled on the northwest portion of the site.

4.7.2 Raw Water Piping

Approximately 40 feet of 12-inch diameter raw water piping would convey groundwater from the new ASR well to the IX treatment facility. All new piping would be on-site and would be either CML&C steel or CML&W steel. Buried piping would have polyethylene wrap and would be concrete encased.

4.7.3 Operations and Hydraulic Conditions

The proposed operations would include redirection of the flow from the raw water wells to the new IX facilities. Treated water would be blended with a raw water bypass stream and injected with chlorine on-site for disinfection.

The proposed ASR well would be sized such that it would pump groundwater through the IX facility and into the existing distribution system. The TDH required would include the head needed to pump the groundwater to the ground surface and to the maximum pressure in MVWD Pressure Zone 2, which operates at 1207 ft. Losses attributed to pipe friction, specials (bends, valves, flowmeters, etc.), bag filters, and the IX resin bed would be accounted for in the TDH requirement. The TDH required for the New ASR Well IX Facility is 724 ft. A 550 HP well motor would be required to achieve this TDH.

5.0 AGENCY INTERCONNECTION FACILITIES

5.1 Overview

This chapter describes the interconnection facilities required to transfer water from MVWD to Walnut Valley and Rowland Water Districts. The facilities would include approximately 11,800 feet of new 20-inch diameter pipe. Water from the MVWD system would be delivered to Walnut Valley Water District (WVWD) via Chino Hills, where it would be used in-lieu of imported water delivered to WVWD via the Pomona-Walnut-Rowland (PWR) pipeline. This new interconnection will allow delivery of approximately 3,000 to 5,000 afy during dry years. Coordination with TVMWD and WVWD would be required to arrange the in-lieu shift.

5.2 Water Supply

The in-lieu shift with WVWD would be provided using existing conveyance facilities and a new section of pipeline. Water from MVWD would be conveyed south through MVWD's system via the existing 42-inch Ramona Feeder, which has a capacity of 30 mgd. The Ramona Feeder provides a hydraulic connection between MVWD and the City of Chino Hills. Water would then be conveyed west through Chino Hills' service area via an existing 20-inch pipeline along Grand Avenue to its termination at the intersection with Pleasant Hill in the City of Chino Hills. At this location, a new 20-inch pipeline is required to connect the City of Chino Hills and WVWD service areas as shown on Figure 5-1.

The new 20-inch pipeline would terminate with a new 12-inch interconnection at the intersection of Grand Avenue and Rolling Knoll Road. This interconnection would serve the allocated in-lieu shift to WVWD's 1200 pressure zone. From this zone, WVWD would have the ability to serve both their 1050 and 1350 zones through the use of existing pressure reducing stations and booster stations.

5.3 Pipeline Alignment and Conceptual Design

This section presents design criteria for the interconnection pipeline.

The new pipeline would start at the intersection of Grand Avenue and Pleasant Hill, run along Grand Avenue, and terminate at the intersection of Grand Avenue and Rolling Knoll Road. The section of pipe would be approximately 11,800 feet long and 20 inches in diameter.

The design parameters discussed include general design criteria, steel pipe design, coating, and lining materials, load criteria, pipeline plate thickness, joint configurations, trench detail, pipeline crossings, pipeline appurtenances and cathodic protection. At this stage of project development, it has been assumed that steel pipe would be the selected pipe material for the purposes of developing an opinion of probable cost. Alternative pipe materials, such as ductile iron, high-density polyethylene (HDPE), and polyvinyl chloride (PVC), may also be appropriate and should be investigated during the design phase in order to provide a competitive bidding scenario. A summary of the design criteria for the pipeline is presented in Table 5-1.

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Chino Basin Dry Year Yield Program Expansion Project

Monte Vista Water District – New Connection to Walnut/Rowland System

Figure

5-1

Table 5-1
Summary of Pipeline Design Criteria

	Pleasant Hill to Rolling Knoll Road
Pipe	
Pipe Diameter, in	20
Pipe Length, ft	11,800
Design Flows	
Maximum, cfs (1)	13.1
Average (Based on shift), cfs (2)	6.9
Velocities	
Maximum, fps (1)	6.0
Average (Based on shift), fps (2)	3.17
Design Pressure	
Design Hydraulic Gradient, elevation (3)	1441
Approximate Pipe Center Line, elevation (3)	950
Design Pressure, psi	212
Pipe Wall Design	
Diameter/thickness ratio (d/t)	165
Minimum thickness	0.25 (Min. steel thickness)
Pipe and Fittings Materials	Cement Mortar Lined and Coated Welded Steel
Pipe	Steel AWWA C200
Lining	Plant applied cement mortar, AWWA C205
Coating	Cement mortar, AWWA C205
Pipe Trench Criteria	
Minimum Cover, feet	6
Allowable Nominal Deflection	
Percent of Nominal Diameter	2
Modulus of Soil, psi	1400 (B&V design standard)
Pipe Joints	Gasketed, Single or Double welded, or butt strap, as required by District

Notes:
(1) Maximum flow based upon assumed maximum velocity of 6.0 fps.
(2) Flow based upon total shift obligation of 3,000-5,000 afy.
(3) Based upon calculations at the approximate location of proposed interconnection at Rolling Knoll Drive. Depth of pipe assumed to be 6 feet below ground surface.

5.3.1 Applicable Codes and Standards

The following codes and standards are applicable to the design and construction of the pipeline:

- American Society for Testing Materials (ASTM)
- American Water Works Association (AWWA) Codes and Standards
- AWWA Manual M11 (Steel Pipe – A Guide for Design & Installation)

- AWWA Manual M51 (Air Release, Air/Vacuum, and Combination Air Valves)
- B&V Design Procedures
- California Code of Regulations
- State of California Construction Safety Orders (Cal-OSHA)
- CDPH
- MVWD, Chino Hills, and WVWD Standards

5.3.2 Hydraulic Design

Pipeline hydraulic design and requirements are based on information obtained from MVWD. The Chino Hills supply pipeline is assumed to be pressurized to 95 psi at the connection, resulting in a hydraulic elevation of approximately 1505 feet. Table 5-2 summarizes the potential hydraulic losses for the 20-inch diameter, 11,800 foot pipeline at both the maximum flow and average flow determined by the DYY shift. According to MVWD staff, the District has the ability to move water into its upper and lower pressure zones from the 1200 zone. The plan and profile shown on Figure 5-2 also provides the HGLs for both maximum and average flow.

Table 5-2
Summary of Hydraulic Losses

	Flow Rate, cfs	Hydraulic Loss, feet
Maximum flow	13.1	80.3
Average (Based on shift)	6.9	24.6

5.3.3 Pipe Diameter

The pipeline from the Chino Hills service area at Grand Avenue and Pleasant Hill to the WVWD's turnout at Grand Avenue and Rolling Knoll Road would consist of a 20-inch diameter pipeline.

5.3.4 Pipe Materials

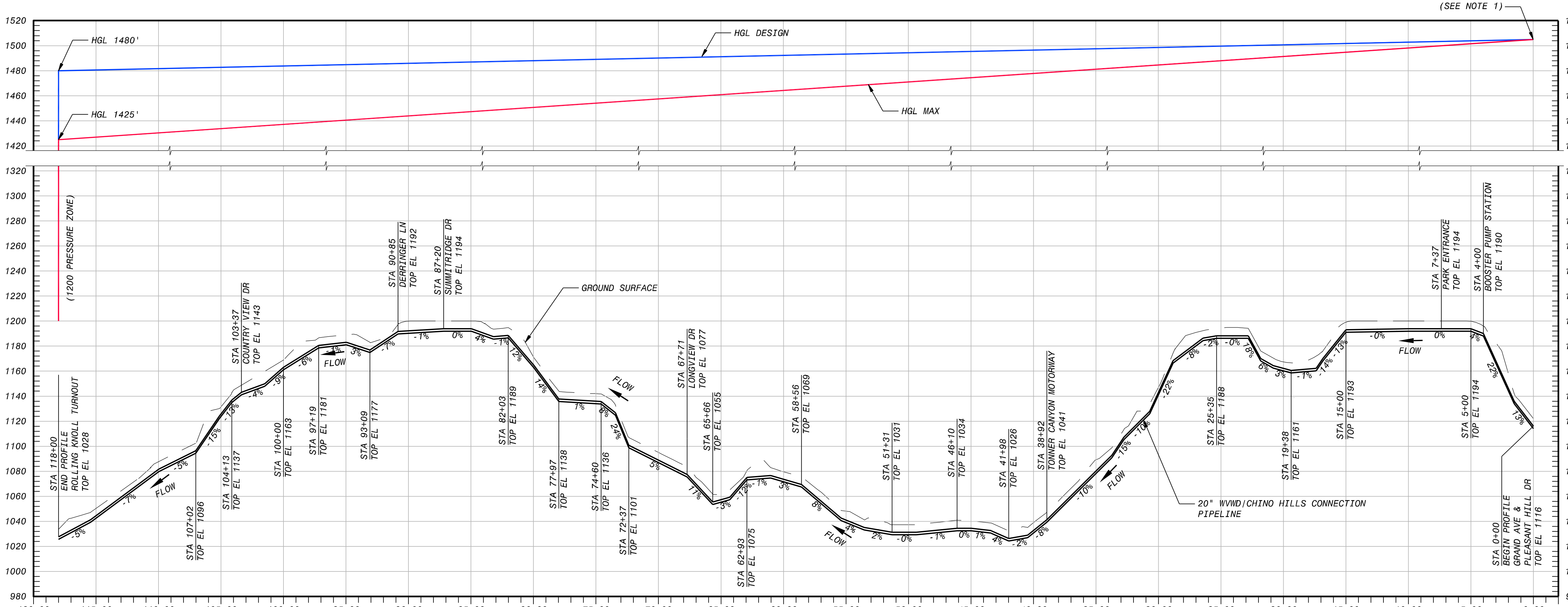
Pipeline materials would be selected to meet ductility and joint design guidelines for superior seismic performance. Steel pipe was selected for the basis of this conceptual project development; however, alternative pipe materials could be evaluated during final design. The pipeline would be cement mortar lined and coated steel pipe conforming to AWWA C200. The pressure class would be allowed to vary along the pipe. The required pipeline wall thickness would be determined for the pipeline and indicated on the plan and profile drawings.

5.3.5 Pipe Sections

Typical pipe sections are available in alternative lengths from 40 to 60 feet, dependent on the pipe manufacturer's mill capabilities. A total of 11,800 feet of pipe would require approximately 295 sticks of 40-foot pipe and 197 sticks of 60-foot pipe.

PLAN STA 0+00 TO STA 118+00

HORIZ - 1" = 1000'



HYDRAULIC PROFILE STA 0+00 TO STA 118+00

HORIZ - 1" = 1000'
VERT - 1" = 100'

HYDRAULIC LEGEND

HGL	FLOW SCENARIO	FLOW RATE	NOTES
(red line)	MAXIMUM CAPACITY	9,500 AFY	BASED ON PIPE FLOW
(blue line)	DESIGN CAPACITY	5,000 AFY	BASED ON SHIFT

NOTE:
1. ASSUMES OPERATING PRESSURE OF 95 PSI WITHIN CHINO HILLS 1286 ZONE.

CONCEPTUAL - NOT FOR CONSTRUCTION

DATE	REVISIONS AND RECORD OF ISSUE	NO.	BY	CK	APP

CYGNET ID: 160374-2000-WTUP-C-N00013PX9 XREF1:WVWD Interconnection.jpg
WF:17.1s (LMS Tech) XREF2:
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PLOTTED:12/12/2008 3:19:25 AM, Batch Plot XREF4:
USER:cygnetadmi DWG VER:1.8 XREF5:

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Black & Veatch Corporation
Irvine, California

CHINO BASIN DYY PROGRAM EXPANSION PROJECT
PROJECT DEVELOPMENT REPORT

MONTE VISTA WATER DISTRICT
WVWD INTERCONNECTION CONVEYANCE PIPELINE
PLAN AND PROFILE

DESIGNED: MEM
DETAILED: JEG
CHECKED:
APPROVED:
DATE:

PROJECT NO.
160374

FIGURE 5-2

FD160374
D160374

5.3.6 Load Criteria

Internal and external loads must be considered to ensure appropriate pipeline design.

5.3.6.1 Internal Load

Design for internal loading would be based on the design hydraulic grade lines (HGLs). Design pressures would be based on the considerations of normal operating conditions, transient surge conditions, hydrostatic test pressures, and other conditions if warranted.

5.3.6.2 External Load

Design of the pipe for external loading would consider the depth of earth cover, live loads, and construction loads. A maximum deflection of 2 percent nominal pipe diameter would be allowed. A maximum allowable design deflection of 2 percent for the 20-inch diameter pipe is 0.40 inches. Based on a modulus of elasticity of 1400 pounds per square inch (psi) for soil, the minimum cover over the pipeline would be 6-feet and the maximum cover 43 feet. Concrete slurry would be required for deeper installation. In areas where utility crossings may occur, pipe cover would range from 6 to 10 feet or be governed by the geotechnical engineer's recommendations.

5.3.7 Pipeline Wall Thickness

Minimum pipe wall thickness is an important consideration for handling and installation and for protection against collapse or buckling due to internal vacuum. Hydraulic requirements often dictate that the pipe wall thickness be increased for internal pressure. The minimum wall thickness and internal pressure were calculated to determine the governing criteria for wall thickness. For this pipeline, the minimum guidelines governed pipe wall thickness design.

The diameter to thickness ratio (d/t) provides the minimum steel thickness for safe transport of the pipe. A d/t of 165 is recommended for this pipeline, resulting in a minimum wall thickness for a 20-inch pipeline of 0.10-inch; however, because the pipeline would be buried in streets with congested underground utilities and/or in areas where future construction may expose the pipe, a wall thickness of 0.25-inch is recommended.

The steel thickness necessary to withstand the internal pressure was also calculated to ensure the minimum thickness is adequate. Based upon preliminary calculations, the internal pressure considered is negligible when considering a thickness of 0.25-inch. The pipe wall thickness would vary along the alignment based on the test HGL and the actual centerline of the installed pipe. These thicknesses would be determined during final design, although the required pipe wall thickness would be 0.25-inch at a minimum.

5.3.8 Pipe Deflection

Steel pipe is a flexible conduit and the maximum cover depth is dependent on the allowable deflection caused by external loads. Maximum allowable deflection resulting from external loading conditions is limited to two percent of the pipe diameter for pipe with shop applied cement mortar coating. The maximum allowable design deflection of two percent, for the 20-inch diameter pipe would be 0.40 inches.

Deflections using the minimum pipe wall thicknesses were calculated assuming a soil unit weight of 120 pcf and assuming Class B bedding as summarized in Table 5-3.

Table 5-3
Pipe Deflection(1)

	20-Inch Pipe Diameter
Deflection (in.)	0.05
Max. Cover Depth, feet	43

Notes:
(1) Assumes w = 120 lbs/ft^3 and Class B bedding.

5.3.9 Joints and Fittings

Pipe installation would use rubber gasket joints, or single or double welded joints to join pipe sections, depending upon District standards.

5.3.10 Trench Design

Excavation for pipe installation would be in accordance with the requirements established by Cal-OSHA and by the applicable agencies. Shoring may be required due to space constraints and possibly soil considerations. Shoring design would be specified to be the responsibility of the contractor. Trench depth should be generally selected based on minimum cover to protect the pipe safely from transient loads. Depth of trench in city streets may be governed by existing utilities or other conditions. If the sides of the trench remain vertical after excavation, and if bedding and backfill were consolidated by hydraulic methods, then the minimum trench width at the top of the pipe would be pipe outside diameter (OD) plus 20 inches on each side of the pipe. If the pipe-zone bedding and backfill require densification by compaction, then the width of the trench at the bottom of the pipe should be determined by the space required for the proper and effective use of tamping equipment, but it should never be less than pipe OD plus 20 inches on each side. Flat bottom trenches should be excavated to a depth of minimum of four inches below the established grade line of the outside bottom of the pipe. Specified building material should be used to fill the excess excavation. Loose subgrade material should be graded uniformly to the established grade line for the full length of the pipe.

5.3.10.1 Open Trench with Flared Sidewalls

This method would require more construction area than any other methods because of the type of equipment used. However, open trenching with flared sidewalls is the least expensive form of excavation for pipelines. This method would generally be used in open terrain and would not likely be used in an installation along Grand Avenue. An open trench would demand the width of two lanes, essentially halting one direction of traffic flow.

5.3.10.2 Open Trench with Shoring

Shored open trench construction would be required for the majority if not all of the pipeline and would be used for confined construction areas and restricted right-of-ways (ROW). Pipe placement along the street would require this method because of space confinement. The entirety of the pipeline would be constructed within the ROW for existing public streets.

5.3.10.3 Jack and Bore Method

The jack and bore method may be utilized if issues exist which would not allow sections of the street to be opened. The contractor shall install a prefabricated pipe through the ground from a jacking pit to a receiving pit. The pipe would be propelled by jacks located in the jacking pit. As the pipe installation progresses, the spoils would be transported out of the pipe either manually or by mechanical methods. The casing pipe material would be steel pipe welded at each joint. The casing pipe would need to accommodate the carrier pipe plus the skids, or pipe spacers, to support the carrier pipe. For a 20-inch pipeline, the casing pipe would be 32-inch. The contractor would need space for the jacking pit (approximately 20 by 40 feet, equipment, (e.g. excavator, crane, generator, small equipment, storage containers), materials, temporary spoils piles, and delivery equipment. The jacking and receiving pits would be supported in a manner similar to open trench excavation with shoring. The contractor would require space around the boring pit for the excavator, crane, and most of the other equipment noted above for jack and bore construction method.

5.3.11 Pipeline Connections

The first connection would be made at the discharge piping of Chino Hills' existing 20-inch service line. The final connection would be at the terminus of the pipeline at Rolling Knoll Road.

5.3.12 Lining and Coatings

All buried steel pipe would be coated and lined. The pipe coating would be a cement mortar coating in accordance with AWWA C205. The lining would also be cement mortar. The lining and coating would be used to protect the pipeline from wear during installation and operation, as well as from corrosion.

5.3.13 Corrosion Control

The water being conveyed is potable water and is not known to be corrosive. Cement mortar lining on the inside of the steel pipe would provide the primary corrosion protection for the steel shell.

If cathodic protection is desired to protect the external pipe surface, cathodic test stations would be included in the pipeline design. Installation of wire jumpers at joints, harness assemblies, and couplings would be provided for continuity along the pipeline. Insulating flanges would be provided to isolate pipeline segments. Where cement mortar coatings are not provided on the pipeline, the pipe would be coated with a high performance protective coating, coated with mastic, and wrapped with polyethylene sheeting.

5.3.14 Construction Requirements

The entire alignment lies within the public ROW. Encroachments through public streets would be handled by the city or county. The contractor would have to work within a restricted construction zone along the road, either on the shoulder or within an identified lane, where the trench would be located using a shored trench. A detailed evaluation of the construction zone requirements versus available width would be required during design.

5.3.14.1 Pipeline Appurtenances

Water conveyance facilities include appurtenant structures for operation and protection against damaging hydraulic transients. Facilities to permit periodic maintenance would also be provided. Specific appurtenances would include couplings, isolation valves, air and vacuum relief, blow-off facilities, access manways, pipe draining and filling, and marker posts.

5.3.14.2 Couplings

Sleeve couplings provide tightness and strength with flexibility. The flexible sleeve coupling would be able to handle acceptable pipe axial movement. If greater displacement were needed, a harness assembly could be installed with each flexible coupling according to AWWA M11.

5.3.14.3 Isolation Valves

The pipeline would be designed to resist damage from earthquakes. In addition, valves may be provided to isolate portions of the pipeline should damage occur. Isolation valves would be the same size as the pipeline and would be manually operated. The location of these valves, if desired, would be determined after the completion of the geotechnical report during final design.

5.3.14.4 Air Release/Vacuum Relief

Air release/vacuum relief valves allow entrained air to vent out of the pipeline during fill, allow air back into the pipeline when it is being drained, and protect the pipeline from collapse due to negative pressures. The air release/vacuum relief valves would be installed at every summit along the pipeline; the valves would prevent accumulation of air pockets at high points, which might impair the pipe’s flow capacity. Air release/vacuum relief valves would be designed to meet all the criteria in AWWA M11 and M51.

5.3.14.5 Blowoff Facilities

Blowoff facilities would be located at the low points and upstream of line valves located on a slope of the pipeline. Blowoff facilities would be used to drain pipe sections and to allow for relief of pipe pressure for inspection and maintenance purposes. The blow off facilities would consist of a short length of pipe connected to the bottom of the main pipe and carried away from the main to a gate valve where the operating nut must be accessible from the surface. The blowoff facility would be designed and set with the stem vertical and just beyond the side of the pipeline.

5.3.14.6 Utility Research

An investigation of existing facilities should be performed to identify approximate locations of crossing or parallel utilities in relation to that of the pipeline. Potholing is also expected in some locations along the pipeline alignment during final design to determine unknown or verify as-built utility locations.

6.0 OPINION OF PROBABLE COST

6.1 Overview

This chapter presents the opinion of probable cost for the facilities described in this Volume IIE of the PDR. General cost assumptions and the opinion of probable capital and annual operations and maintenance (O&M) costs are presented below.

The opinion of probable cost was based on conceptual-level unit cost criteria intended to provide a budgetary estimate of each facility's capital and annual O&M costs. Table 6-1 summarizes the estimated capital and annual O&M costs for the District's proposed facilities. As shown in the table, the total opinion of probable capital and annual O&M costs for Option A facilities would be $17,755,000 and $965,000, respectively. The total opinion of probable capital and annual O&M costs for Option B facilities would be $10,811,000 and $501,000, respectively.

Table 6-1
Summary of Opinion of Probable Capital and Annual O&M Costs

Component	Option A	Option B
Capital Cost		
Construction Cost	$13,451,000	$8,190,000
Contingency (1)	$2,690,000	$1,638,000
Engineering/Administration/CM (2)	$1,614,000	$983,000
Total Capital Cost	**$17,755,000**	**$10,811,000**
Midpoint of Construction Cost (3)	$19,401,000	$11,813,000
Annual Cost		
Annual O&M Cost	$965,000	$501,000
Annualized Capital Cost (4)	$1,518,000	$924,000
Total Annual Cost	**$2,483,000**	**$1,425,000**

Notes:
(1) Based on 20 percent contingency.
(2) Based on 12 percent engineering/administration/construction management (CM).
(3) Assumes midpoint of construction in year 2012 at 3 percent escalation rate.
(4) Assumes amortization period of 25 years and discount rate of 6 percent.
(5) Costs do not include use of MVWD's existing ASR facilities for potential "put" contribution. The total capital value for the use of these facilities may range from $2.0-3.2M/1,000 AFY of "put" capacity. See Appendix D, Volume I, for a preliminary evaluation of these costs.

6.2 General Cost Assumptions

The conceptual-level opinion of probable capital and O&M costs developed in this PDR were derived from quotes received from equipment manufacturers, a survey of bid pricing from participating agency facilities previously or currently under construction, and bid results or construction cost estimates from similar and recent B&V projects. Volume I, Chapter 9, presents a summary of the basis for the unit costs used in this PDR.

Volume I, Chapter 9, also presents the construction, annual O&M, general, and financing unit cost criteria used to develop the cost estimates provided in this chapter.

6.3 Capital Cost

Table 6-2 presents the opinion of probable capital cost for construction of the District's Option A facilities. As shown, the total estimated capital cost for the new Option A facilities would be $17,755,000. Midpoint of construction costs are also provided and indicate the constructions costs in year 2012 using a 3 percent escalation rate.

Table 6-2
Summary of Opinion of Probable Capital Cost--Option A Facilities

Component/Facility Detail	Option A Cost
Well Facilities (1): New ASR Well and Well No. 2 Replacement	
Drilling/Casing/Cap	$2,150,000
Equipping	$2,100,000
Disinfection System	$400,000
Pumphouse/Electrical Building	$250,000
Treatment Facilities	
IX: New ASR Well IX Facility (1,836 gpm installed)	$2,009,000
IX: Well No. 2 IX Facility (909 gpm installed)	$995,000
Pre-engineered Building(s)	$400,000
Conveyance Facilities	
Distribution Pipeline: 11,800 feet @ 20" Diameter	$3,540,000
Brine Pipeline: 5,400 feet @ Dual 6" Diameter	$488,000
Misc. Valves and Flowmeters	$25,000
SARI/NRWS Facilities	
Initial Capacity Charges	$300,000
General Costs	
Mechanical (2)	$90,000
Electrical (2)	$300,000
Site Work (2)	$150,000
General Requirements (3)	$254,000
Total Construction Cost	$13,451,000
Contingency (4)	$2,690,000
Engineering/Administration/CM (5)	$1,614,000
Total Capital Cost	**$17,755,000**
Total Midpoint of Construction Cost (6)	$19,401,000

Notes:
(1) Includes any new production, ASR, and injection wells and well conversion/rehabilitation costs.
(2) Includes general costs for treatment and booster station facilities.
(3) Includes general requirements costs for major facilities (except land and SARI/NRWS).
(4) Based on 20 percent contingency.
(5) Based on 12 percent engineering/administration/CM.
(6) Assumes midpoint of construction in year 2012 at 3 percent escalation rate.
(7) Costs do not include use of MVWD's existing ASR facilities for potential "put" contribution. The total capital value for the use of these facilities may range from $2.0-3.2M/1,000 AFY of "put" capacity. See Appendix D, Volume I, for a preliminary evaluation of these costs.

Table 6-3 presents the opinion of probable capital cost for construction of the District's Option B facilities. As shown, the total estimated capital cost for the new Option B facilities would be $10,811,000.

Table 6-3
Summary of Opinion of Probable Capital Cost--Option B Facilities

Component/Facility Detail	Option B Cost
Treatment Facilities	
IX: Well Nos. 4 and 27 IX Facility	$3,077,000
Pre-engineered Building	$200,000
Conveyance Facilities	
Distribution Pipeline: 11,800 feet @ 20" Diameter	$3,540,000
Brine Pipeline: 5,400 feet @ 6" Diameter	$486,000
Misc. Valves and Flowmeters	$25,000
SARI/NRWS Facilities	
Initial Capacity Charge	$150,000
General Costs	
Mechanical (2)	$92,000
Electrical (2)	$308,000
Site Work (2)	$154,000
General Requirements (3)	$158,000
Total Construction Cost	$8,190,000
Contingency (4)	$1,638,000
Engineering/Administration/CM (5)	$983,000
Total Capital Cost	**$10,811,000**
Total Midpoint of Construction Cost (6)	$11,813,000

Notes:
(1) Includes any new production, ASR, and injection wells and well conversion/rehabilitation costs.
(2) Includes general costs for all treatment and booster station facilities.
(3) Includes general requirements costs for all facilities (except land and SARI/NRWS).
(4) Based on 20 percent contingency.
(5) Based on 12 percent engineering/administration/CM.
(6) Assumes midpoint of construction in year 2012 at 3 percent escalation rate.
(7) Costs do not include use of MVWD's existing ASR facilities for potential "put" contribution. The total capital value for the use of these facilities may range from $2.0-3.2M/1,000 AFY of "put" capacity. See Appendix D, Volume I, for a preliminary evaluation of these costs.

6.4 Annual O&M Cost

Table 6-4 presents the opinion of probable annual O&M cost for the District's Option A facilities. As shown, the total estimated annual O&M cost for the new Option A facilities would be $965,000.

Table 6-4
Summary of Opinion of Probable Annual O&M Cost--Option A Facilities

Component/Facility Detail	Option A Cost
Well Facilities [1]: New ASR Well (550 HP) and Well No. 2 Replacement (200 HP)	
Power	$417,000
Miscellaneous Maintenance	$50,000
Treatment	
New ASR Well IX Facility (1,836 gpm)	
General	$290,000
Resin Replacement	$20,000
Well No. 2 IX Facility (909 gpm)	
General	$143,000
Resin Replacement	$10,000
Conveyance Facilities	
General Pipeline Maintenance: Brine and Distribution	$13,000
SARI/NRWS Facilities	
Capacity Charges	$6,000
Volumetric Charges	$14,000
CIP Charges	$2,000
Total Annual O&M Cost	**$965,000**
Annualized Capital Cost [2]	$1,518,000
Total Annual Cost	$2,483,000

Notes:
(1) Includes any new production, ASR, and injection wells and well conversion/rehabilitation costs.
(2) Assumes amortization period of 25 years and discount rate of 6 percent.

Table 6-5 presents the opinion of probable annual O&M cost for the District's Option B facilities. As shown, the total estimated annual O&M cost for the new Option B facilities would be $501,000.

Table 6-5
Summary of Opinion of Probable Annual O&M Cost--Option B Facilities

Component/Facility Detail	Option B Cost
Treatment Facilities: Well Nos. 4 and 27 IX Facility	
General	$443,000
Resin Replacement	$30,000
Conveyance Facilities	
General Pipeline Maintenance: Brine and Distribution	$13,000
SARI/NRWS Facilities	
Capacity Charge	$3,000
Volumetric Charge	$11,000
CIP Charge	$1,000
Total Annual O&M Cost	**$501,000**
Annualized Capital Cost (1)	$924,000
Total Annual Cost	$1,425,000

Notes:
(1) Assumes amortization period of 25 years and discount rate of 6 percent.