TECHNICAL REPORT

Description of the Refined MRWPCA Project
for the
Monterey Regional Water Supply Program

June 30, 2008

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LIST OF ACRONYMS

180-GWRP	180-foot Aquifer Groundwater Replenishment Project
AF	acre feet
AFY	acre-feet per year
AOP	advanced oxidation process
ASR	aquifer storage and recovery
AWTP	advanced water treatment plant
bgs	below ground surface
Cal Am	California American Water Company
CDPH	California Department of Public Health
cfs	cubic feet per second
CIP	clean-in-place (for a membrane system)
CIWR	Center for Integrated Water Research
CPUC	California Public Utilities Commission
CRSP	Coastal Recharge Supply Pipeline
CSIP	Castroville Seawater Intrusion Project
CWP	Coastal Water Project
DEIR	Draft Environmental Impact Report
DRA	Division of Ratepayer Advocates
EIR	Environmental Impact Report
FLEWR	Filter Loading Evaluation for Water Reuse
FO-7	Monitoring Well Fort Ord 7
FO-9	Monitoring Well Fort Ord 9
gpm	gallons per minute
GWR	Seaside Groundwater Basin Replenishment Project
HDPE	high density polyethylene
hp	horsepower
IEPS	industrial effluent pump station
IRSP	Inland Recharge Supply Pipeline
kW	kilowatt
kW-hr	kilowatt hours
LF	linear feet
MCC	motor control center
MCL	maximum contaminant level
MCWD	Marina Coast Water District

MCWRA	Monterey County Water Resources Agency
MF	microfiltration
MFBW	microfiltration backwash water
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
MPWMD	Monterey Peninsula Water Management District
MRWMD	Monterey Regional Waste Management District
MRWPCA	Monterey Regional Water Pollution Control Agency
MSL	mean sea level
MW	megawatts
PEA	Proponent's Environmental Assessment
PG&E	Pacific Gas and Electric
psi	pounds per square inch
PVC	polyvinyl chloride
PVDF	polyvinyl diflouride
REPOG	Regional Plenary Oversight Group
RO	reverse osmosis
RTP	MRWPCA Regional Treatment Plant
RWQCB	Regional Water Quality Control Board
RUWAP	Regional Urban Water Augmentation Project
SIWTP	Salinas Industrial Water Treatment Plant
SMTIW	Santa Margarita Test Injection Well
SRDF	Salinas River Diversion Facility
SVA	Salinas Valley Aquitard
SVRP	Salinas Valley Reclamation Plant
SWRCB	State Water Resources Control Board
SWTP	surface water treatment plant
SVWP	Salinas Valley Water Project
TDS	total dissolved solids
UCSC	University of California Santa Cruz
UV	ultraviolet light
V	volts
VFD	variable frequency drive
WDR	Waste Discharge Requirements

Chapter 1 – Purpose of Project Description

1.1 Monterey Regional Water Supply Planning Program

The California Public Utilities Commission (CPUC), through its Division of Ratepayers Advocates (DRA), is working with the University of California Santa Cruz (UCSC) Center for Integrated Water Research (CIWR) to evaluate whether an alternative approach to California American Water Company's (Cal Am) proposed Coastal Water Project (CWP) could achieve the water supply goals of the CWP in a less expensive or faster manner. The Monterey Regional Water Pollution Control Agency (MRWPCA) is one of several agencies participating in the development of the regional water supply alternative. Each of the agency sponsors with projects that are included in the regional water supply alternative were requested to provide detailed descriptions of their respective projects for inclusion in the overall regional water supply alternative project description.

For its component of the regional water supply alternative, MRWPCA provided project description information that was included in the June 4, 2008 document titled *Monterey Regional Water Supply Program: EIR Project Description, Proposed Alternative to a Desalination Facility at Moss Landing*, prepared by the consulting firm of RMC Water and Environment for submittal to Cal Am and the CPUC. Since that submittal, MRWPCA has further refined its project, and has focused on developing a project that can assist Cal Am and CPUC achieve their immediate regulatory water supply needs (12,500 acre feet per year [AFY]) to replace water production sources from the Carmel River and pumping reduction requirements of the Seaside Basin Adjudication. The purpose of this document is to present the MRWPCA refined project description for the regional water supply alternative. The target year for implementation of the refined MRWPCA project is 2012.

1.2 MRWPCA Contribution to Regional Water Supply Program

The initial MRWPCA project was comprised of two groundwater replenishment projects, one in the Seaside Groundwater Basin and the other in the 180-foot groundwater aquifer in the North Salinas Valley area, and an advanced water treatment plant (AWTP) to be constructed at the MRWPCA Regional Treatment Plant (RTP). The refined MRWPCA project focuses on the Seaside Groundwater Basin replenishment and AWTP components, and includes additional sources of diluent water (described in Chapter 3). The 180-foot groundwater aquifer replenishment project has been removed for consideration by MRWPCA. The original description of the 180-foot groundwater aquifer replenishment project is included in this report in Appendix A. The original description includes only one source of diluent water for the project, effluent from the Salinas Industrial Water Treatment Plant (SIWTP). As described in Chapter 3, additional diluent water sources are now under consideration for the Seaside Groundwater Replenishment project.

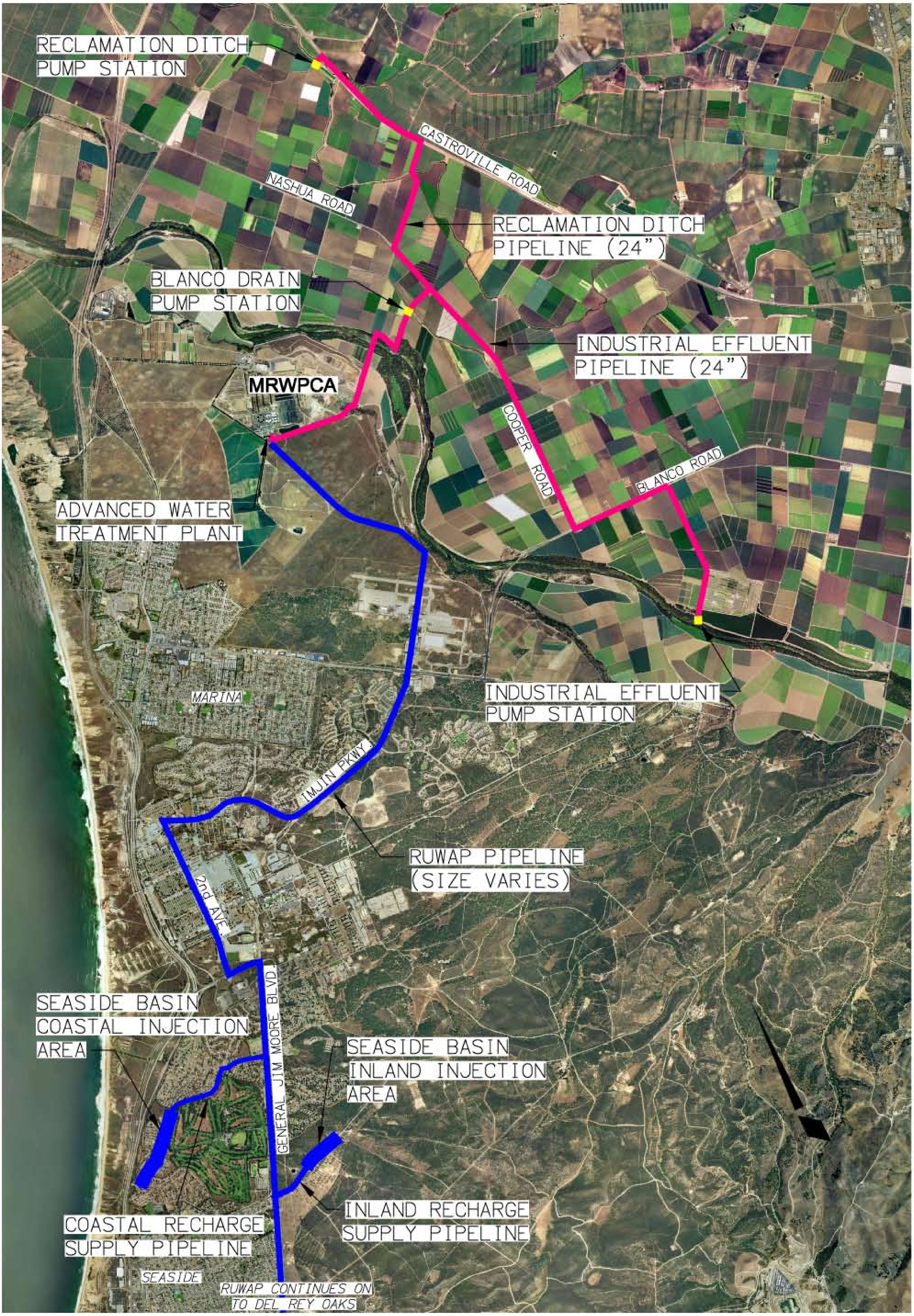
The refined MRWPCA project includes replenishment of the Seaside Groundwater Basin with product water from the AWTP. As indicated above, this product water is anticipated to come from up to three sources: the SIWTP, the Monterey County Water Resources Agency's (MCWRA) Reclamation Ditch system, and the MCWRA Blanco Drain system. MRWPCA would need to enter into an agreement with MCWRA to obtain water from these latter sources. The project includes a new industrial effluent pump station (included in the original MRWPCA project) located near the SIWTP, a new pump station (identical to the industrial effluent pump station) for the Reclamation Ditch water, and an upgraded Blanco Drain pump station (these two pump stations were not included in the original MRWPCA project). The refined project components are shown on Figure 1 and Table 1-1, and are described in more detail in the following sections. The groundwater replenishment project would contribute up to 2,800 acre-feet per year (AFY) to the regional water supply alternative in the winter, plus up to 3,920 AFY during the summer. Therefore, the total recycled water contribution to the regional water supply program would be up to 6,720 AFY. This revised project would provide over one-half (54%) of the 12,500 AF required to meet regulatory needs (SWRCB Order 95-10 and Seaside Basin Adjudication). The balance of the regulatory needs would be made up from conservation, stormwater, and a smaller slant well desalination facility.

The primary objectives of the MRWPCA project are to provide a year round source of supply to the Seaside Groundwater Basin in support of both the Seaside Basin Watermaster and to allow that Basin to be the source of meeting peak demands, which in turn would enable the capacity of the Regional or CWP desalination facility to be reduced.

The Seaside Groundwater Basin Replenishment Project will have wells located at inland and/or coastal locations in the Seaside Basin. MRWPCA anticipates that the purified water from the AWTP will be conveyed to the Seaside Basin through a proposed Marina Coast Water District (MCWD) pipeline to be constructed for the Regional Urban Water Augmentation Project (RUWAP). This refined project would initially supply advanced treated water for some of the larger urban recycled water customers. This system could be expanded in the future to supply other urban customers. If the RUWAP pipeline were not constructed, MRWPCA would explore other approaches to transmit the recycled water to the Seaside Basin. The new AWTP would be designed to produce up to 6 million gallons per day (mgd) of product water utilizing various source waters, including secondary effluent from the RTP, and would provide microfiltration, reverse osmosis, and advanced oxidation/disinfection using ultraviolet light with hydrogen peroxide.

Table 1-1: MRWPCA Project Facilities Summary

Facility	Quantity	Size (Dimensions)
Seaside Groundwater Basin Replenishment (Seaside GWR will use inland and/or coastal recharge locations)		
Conveyance Pipeline (RUWAP transmission pipeline)	1	Approximately 36,000 linear feet of 20-inch pipeline; approximately 6,000 linear feet of 16-inch pipeline to the intersection of General Jim Moore Boulevard and Eucalyptus Road (described in RUWAP EIR)
Inland Recharge Supply Pipeline	1	3,800 linear feet of 20-inch pipeline
Coastal Recharge Supply Pipeline	1	8,800 linear feet of 16-inch pipeline
Turnouts	1 or 2	1 turnout for each recharge area.
Isolation Valves	6 to 13	6 valves (1 per well) for inland recharge area. 7 valves (1 per well) for coastal recharge area.
Recharge Wells (Inland)	6	Four 8-inch PVC vadose zone wells; two 20-inch stainless steel injection wells; backflush pump; backflush pit (new)
Recharge Wells (Coastal)	7	Four 8-inch PVC vadose zone wells; three 20-inch stainless steel injection wells; backflush pump; backflush pit (use existing stormwater detention basin)
Industrial Effluent Pump Station (Diluent water)		
Industrial Effluent Pump Station (IEPS)	1	7.6 mgd, located at Pond #3 of Salinas Industrial Wastewater Treatment Plant
Industrial Effluent Pipeline	1	34,500 linear feet of 24-inch PVC pipeline to AWTP
Reclamation Ditch Water Pump Station (Diluent water)		
Reclamation Ditch Water Pump Station	1	7.6 mgd, location on Reclamation Ditch near Tembladero Slough
Reclamation Ditch Water Pipeline	1	34,500 linear feet of 24-inch PVC pipeline to AWTP
Blanco Drain Water Pump Station (Diluent water)		
Blanco Drain Water Pump Station	1	7.6 mgd located on Reclamation Ditch near Tembladero Slough
Blanco Drain Water Pipeline	1	25 linear feet of 18-inch pipeline to IEPS and Reclamation Ditch pipelines. Water will then be conveyed in either pipeline to AWTP.
Advanced Water Treatment Plant		
Pretreatment System	1	Pre-screening and chemical pretreatment
Membrane Filtration	1	7 mgd capacity, membranes
Reverse Osmosis	1	6 mgd capacity, membranes
Disinfection & Stabilization	1	6 mgd capacity, advanced oxidation process (hydrogen peroxide and UV light treatment)
Brine Disposal	1	12-inch pipeline to convey 1 mgd of brine to existing MRWPCA outfall
Microfiltration Backwash Water	1	600,000 gpd of backwash water returned to plant or further treatment



LEGEND

DILUENT WATER SOURCES

SEASIDE BASIN GWR PROJECT

0 4000

SCALE: 1"=4000'

MRWPCA SEASIDE BASIN GROUNDWATER REPLENISHMENT (GWR) PROJECT

FIGURE 1

Chapter 2 – Seaside Groundwater Basin Replenishment Project

2.1 Introduction, Background and Regulatory Framework

2.1.1 Introduction

The Seaside Groundwater Basin Replenishment Project (GWR) involves recharge of up to 2,400 AFY of highly treated recycled water from the proposed MRWPCA advanced water treatment plant (AWTP) into the Seaside Groundwater Basin during the non-irrigation season. Although the project could potentially provide up to 6,720 SFY on an year round basis, a project of 2,500 AFY of recharge during the non-irrigation season at each of two potential locations is provided herein as the first phase of the project. Additional analyses are planned for the total project, which will be conducted in conjunction with the Seaside Basin Watermaster. The first phase of the project is expected to begin in 2012. The second phase of the project could begin in the 2013 to 2015 timeframe. Targeted zones for recharge include the Paso Robles Aquifer and the Santa Margarita Aquifer, the two primary aquifers providing water supply in the basin. Recharge will be accomplished through a combination of vadose zone wells and groundwater injection wells. Figure 2 shows the Seaside Groundwater Basin boundaries, approximate subarea boundaries, and the two recharge areas identified for the GWR (coastal and inland locations, described later in this section). Recharge would occur at one or both of these locations.

The GWR project will be coordinated with a separate groundwater recharge and recovery project being implemented by the Monterey Peninsula Water Management District (MPWMD). Through an agreement with Cal Am, the main purveyor in the basin, MPWMD has drilled and tested two aquifer storage and recovery (ASR) wells in the Santa Margarita Aquifer. The MPWMD project involves the recharge of excess Carmel River water for seasonal storage in the Seaside Basin. An EIR for this project was prepared and certified in 2006 (Jones & Stokes, 2006). MRWPCA is communicating with Cal Am, MPWMD, the Seaside Basin Watermaster, and other public agencies for coordinated management of the shared groundwater basin resource. The ASR project is planned to be increased as part of Cal Am's Coastal Water Project (CWP).

2.1.2 Background and Regulatory Framework

The Seaside Groundwater Basin has experienced chronic overdraft conditions with declining water levels in both of the aquifers relied on for water supply. Although there is some uncertainty as to the sustainable yield of the basin (estimated at 2,880 AFY by Yates, et al., 2005), it is clear that extractions have exceeded recharge and water levels have declined in most basin wells over the last decade. Although recent evaluations have determined that seawater intrusion is not occurring at present (Feeney, 2007), water levels have declined to below sea level in coastal subareas, increasing the risk. Figure 3 shows groundwater elevation contour maps of the two aquifers and highlights the areas where water levels have fallen below sea level.

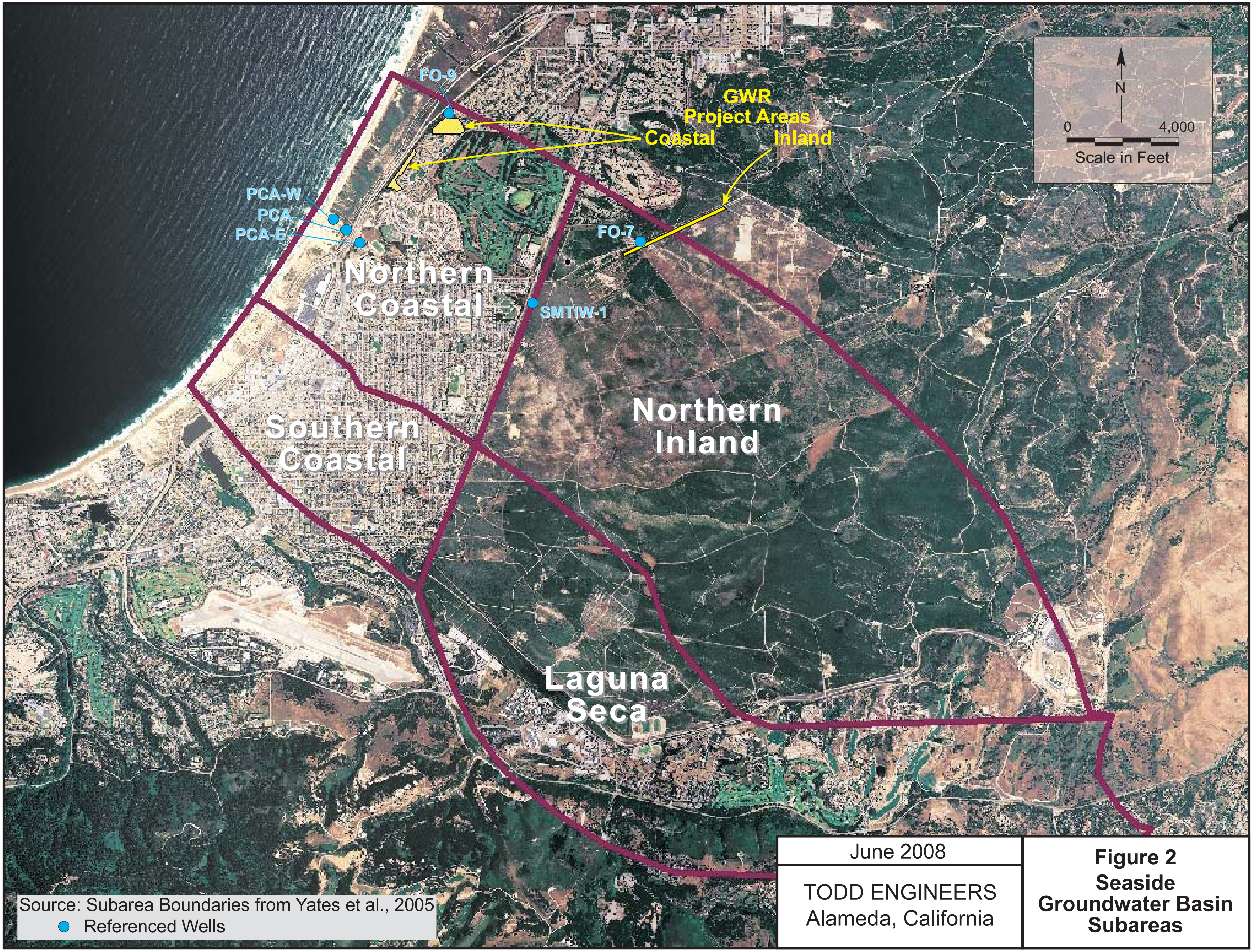


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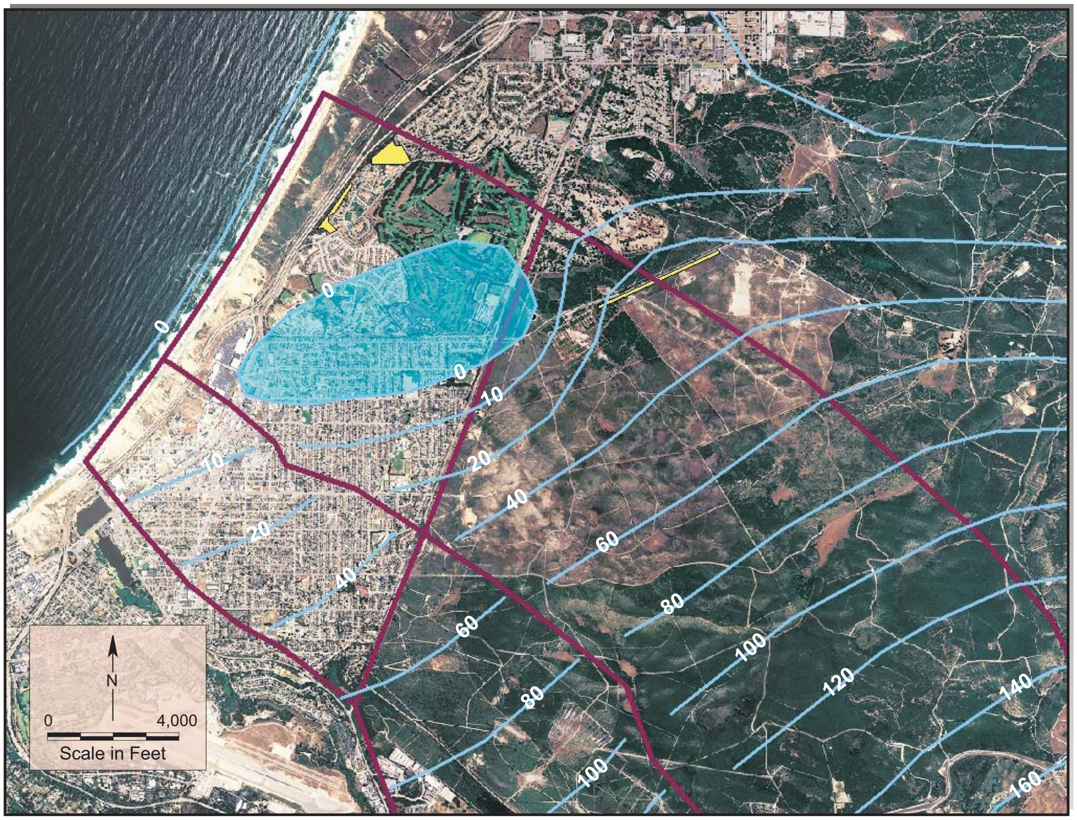
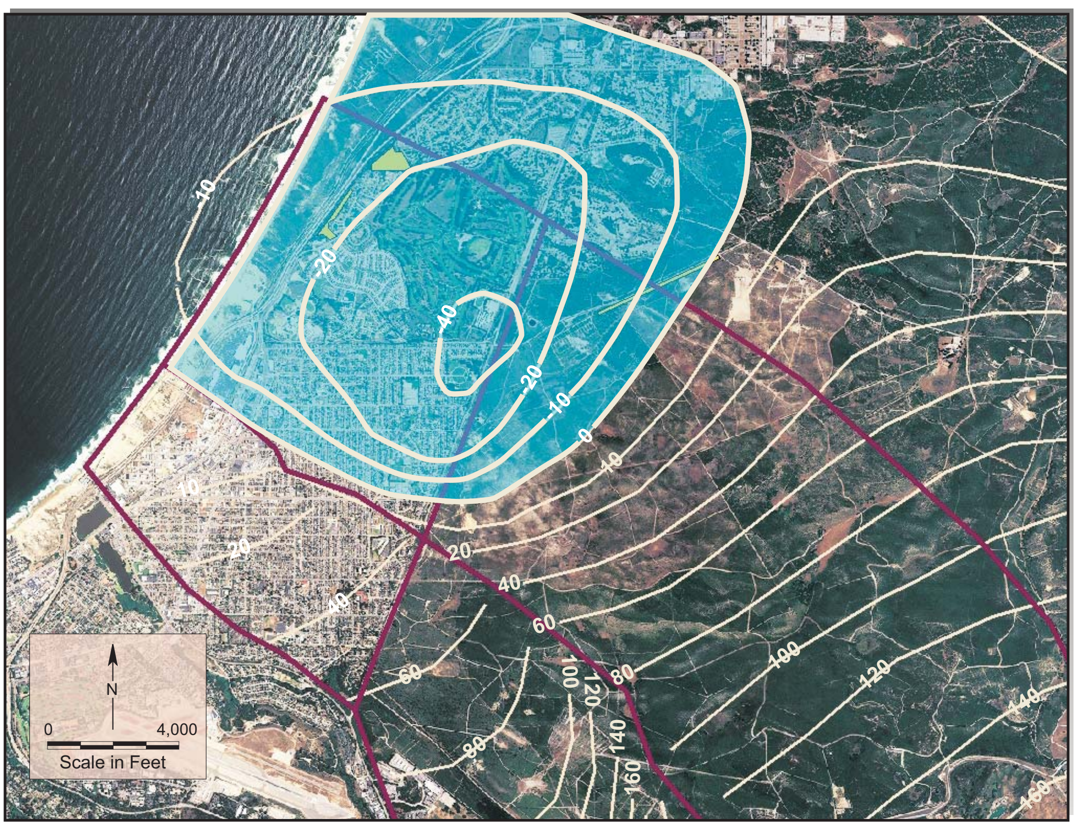
Figure 2
Seaside Groundwater Basin Subareas



Paso Robles Aquifer



Santa Margarita Aquifer

Source: Yates et al., 2005

June 2008	**Figure 3** **Groundwater Elevation Contour Maps, Paso Robles and Santa Margarita Aquifers, Fall 2002**
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In response to overdraft conditions, the basin was adjudicated by the California Superior Court on March 27, 2006, establishing a court-appointed Watermaster to execute the requirements of the adjudication. The court decision found that annual production exceeded the sustainable yield of the basin and noted the risk of seawater intrusion. The adjudication lays out a schedule for mandated decreases in pumping unless replenishment water has been secured or water levels are determined by the Watermaster to be sufficient to ensure a positive offshore gradient to prevent seawater intrusion.

Because the Seaside Basin is used for potable water supply, the recharge of recycled water to the basin is considered an indirect potable reuse. Therefore, MRWPCA is proposing an advanced level of wastewater treatment for the project, similar to other recharge projects in California that inject recycled water for indirect potable reuse. The advanced treatment also provides technical advantages, i.e., the higher quality water reduces the maintenance associated with clogging of recharge wells.

The GWR project is being developed to comply with draft regulations for recycled water recharge currently being developed by the California Department of Public Health (CDPH). The proposed GWR recharge sites allow for appropriate distance from other extraction wells and provide for required residence times within the aquifer. The GWR project will be operated to take advantage of additional replenishment water from the MPWMD ASR project and other sources in the basin to serve as diluent water under the current regulations. Further discussion on how the project incorporates diluent water is provided under Section 2.4.

2.2 Groundwater Basin Setting

The Seaside Groundwater Basin covers approximately 24 square miles between the Monterey Peninsula and the Salinas Valley. The basin has been divided into four subareas based on geologic and hydrogeologic conditions as shown on Figure 2 (Yates, et al., 2005). The northern basin boundary (north boundary of both the Northern Coastal and Northern Inland subareas) represents a groundwater flow divide that has shifted over time in response to changing flow conditions. In general, groundwater flows from the inland subareas toward the coast, but natural flows have been diverted locally due to groundwater pumping patterns.

The hydrostratigraphy of the basin is characterized by a sedimentary sequence up to 1,000 feet thick overlying the Monterey Formation and crystalline bedrock units. The sedimentary units can be subdivided into the following stratigraphic packages listed from shallow to deep:

- Aromas Sand and Older Dune Sands of Pleistocene and Holocene age
- Paso Robles Aquifer and Continental deposits of Pleistocene and Pliocene age
- Purisima Formation of Pleistocene and Pliocene age
- Santa Margarita Aquifer of Miocene age
- Monterey Formation of Miocene age

The Aromas Sand and the Paso Robles Aquifer are considered unconfined and are recharged directly from surface infiltration. The water table occurs in the Aromas Sand near the coast and transitions into the Paso Robles Aquifer inland. The Paso Robles Aquifer occurs within a heterogeneous group of continental deposits that grade downward into the Purisima Formation. The Purisima Formation contains mostly fine-grained sediments and provides confining layers for the underlying Santa Margarita Aquifer. However, sand packages are well developed in some areas and Marina Coast Water District (MCWD) wells produce from the formation north of the basin. The stratigraphy of the Purisima Formation is not well delineated in this basin; therefore, for the purposes of this project description, the Purisima Formation is not defined separately from the Paso Robles Formation.

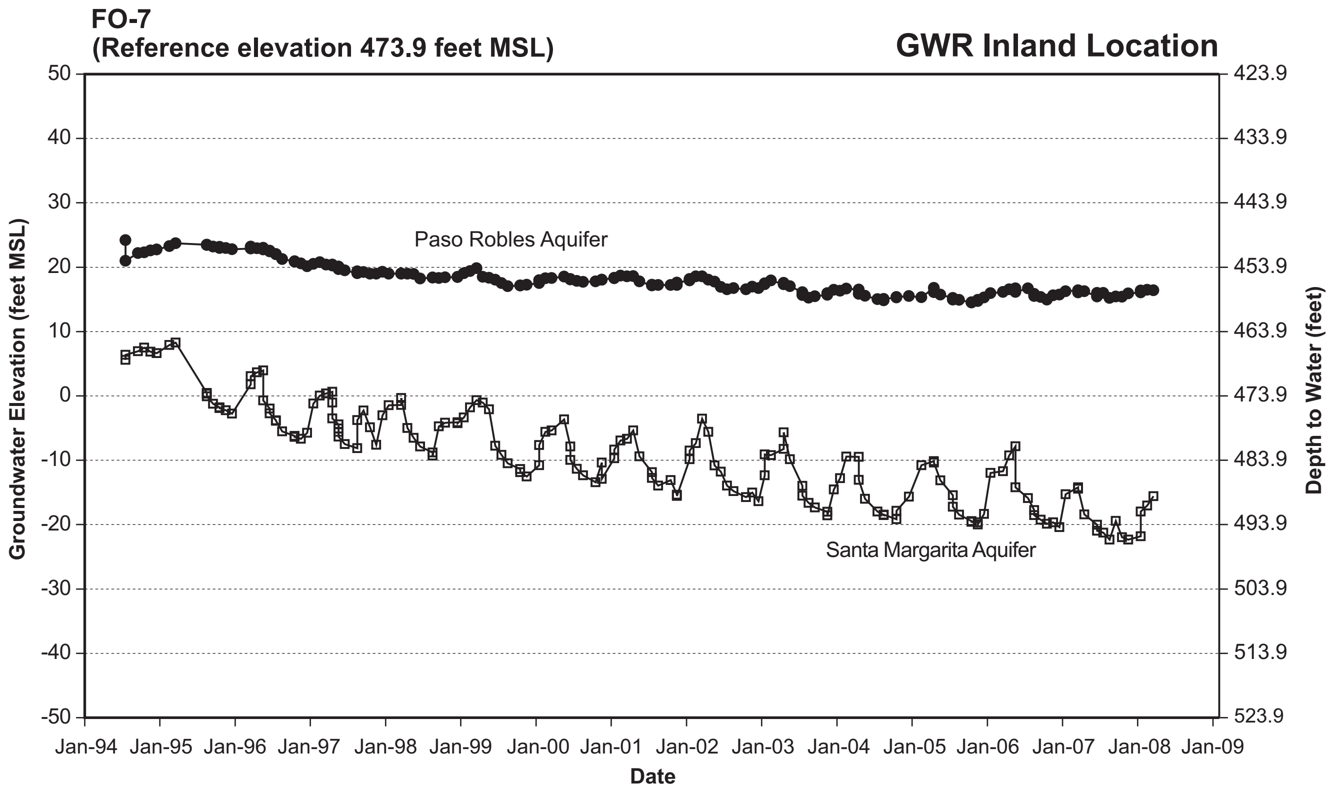
The Santa Margarita Aquifer is an approximate 200-foot thick sandstone that varies in thickness near bedrock highs and basin faults. Recent re-interpretations of basin stratigraphy indicate that the Santa Margarita Aquifer is absent in coastal wells north of the GWR project area but has been interpreted to occur in Monitoring Well Fort Ord 9 (FO-9), adjacent to the GWR coastal recharge location (see Figure 2).

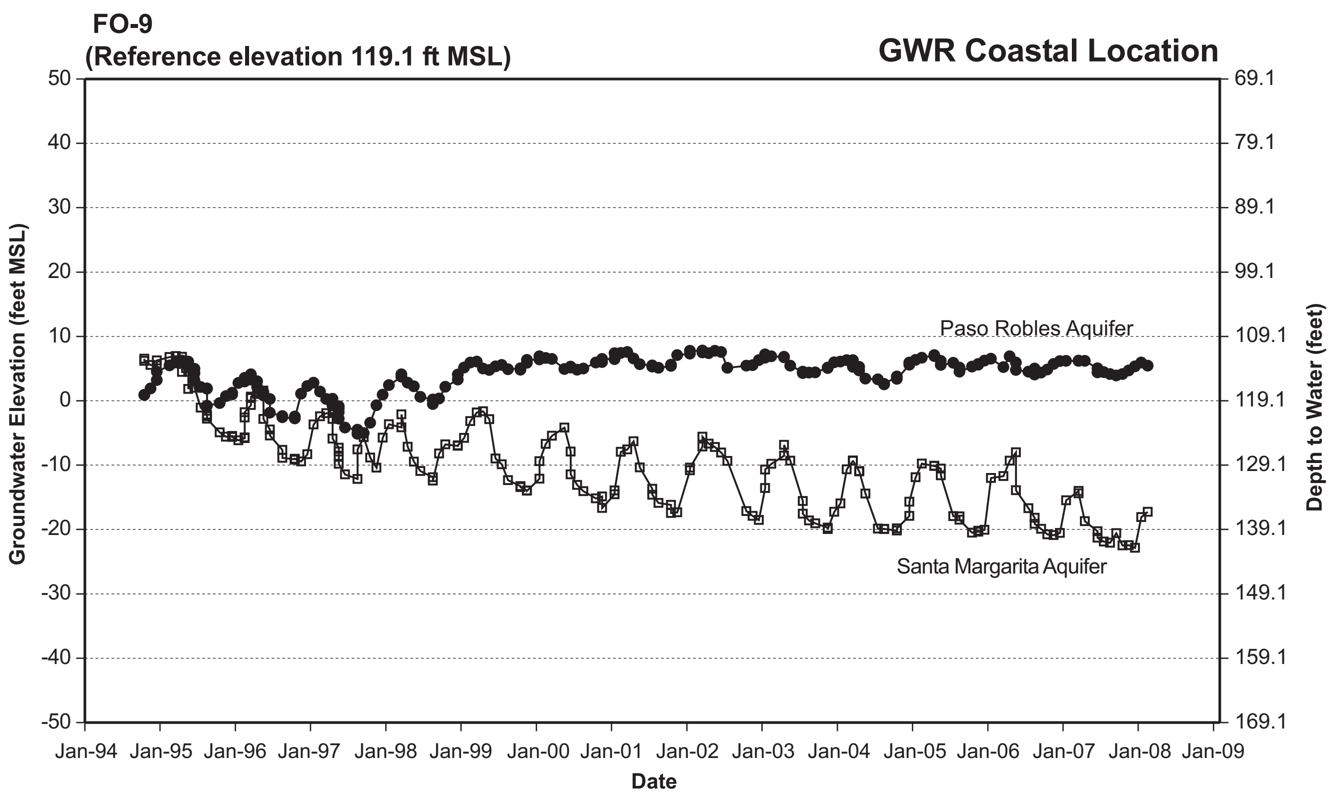
Groundwater in the Santa Margarita Aquifer occurs under confined conditions and recharge likely occurs through vertical leakage from the overlying Purisima/Paso Robles units. The underlying Monterey Formation has relatively low permeability and is considered the base of the water-bearing units in the basin.

The Paso Robles and Santa Margarita Aquifers provide almost all of the water pumped from the basin (an amount totaling 5,043 AFY in 2006) with about 60 percent from the Santa Margarita Aquifer. With an estimated sustainable yield of about 2,880 AFY, production exceeds the basin yield by almost a factor of two. Figure 3 shows the results of over-pumping, illustrated by two groundwater elevation contour maps in each aquifer for water levels measured in the Fall of 2002 (Yates, et al., 2005). Areas below sea level are shaded for emphasis. As shown on Figure 3, most of the Northern Coastal Subarea has a water table below mean sea level (MSL) in the Paso Robles Aquifer (top map of Figure 3). For the Santa Margarita Aquifer, the entire Northern Coastal Subarea and beyond has a potentiometric surface below sea level (bottom map of Figure 3). Importantly, the area of decline has expanded beyond the previous subarea boundary of the basin, reflecting the connection with aquifers to the north.

Since the time of the water level contour maps in 2002, water levels have continued to decline in both aquifers. Hydrographs on Figure 4 show water levels for two monitoring wells, FO-7 and FO-9, each located adjacent to one of the GWR project areas (well locations on Figure 2). Well FO-7 is located at the GWR inland recharge location and FO-9 is located adjacent to the GWR coastal recharge location. Both wells are designed as well clusters with a shallow well screened in the Paso Robles Aquifer and a deeper well screened in the Santa Margarita Aquifer at each location.

As shown on Figure 4, water levels in both aquifers have declined at the inland location (FO-7) since 1994 with the larger declines in the Santa Margarita Aquifer. The small storativity of the confined aquifer and greater extraction amounts have resulted in water level declines of about 24 feet over 19 years. Although levels in the Paso Robles Aquifer have only declined about five feet over the same period, the decline represents a larger loss of groundwater in storage due to the unconfined nature of the aquifer. At the coastal location, represented by FO-9, water level declines in the Santa Margarita Aquifer are similar to those recorded at the inland location. Water levels have risen slightly in the Paso Robles Aquifer at this location, presumably due to decreased extractions (Yates, et al., 2005). However, the water table occurs only a few feet above MSL.





June 2008	**Figure 4**
TODD ENGINEERS Alameda, California	**Water Levels Monitoring Wells FO-7 (Inland) and FO-9 (Coastal)**

2.3 Project Facilities

2.3.1 Conveyance Pipelines

As indicated previously, recycled water from MRWPCA's proposed advanced water treatment plant is anticipated to be conveyed to the Seaside Basin area through use of a MCWD pipeline originally proposed as part of the Regional Urban Water Augmentation Project (RUWAP). An EIR was prepared and certified for that project in 2004, and an Addendum was issued in 2006. This pipeline is projected to extend along General Jim Moore Boulevard, located less than one mile from each of the potential GWR injection sites.

For the inland location, an additional pipeline is needed to extend from the RUWAP up Eucalyptus Road approximately 2,500 feet (see Figure 5). This pipeline, called the Inland Recharge Supply Pipeline (IRSP), would be made of either ductile iron, cement mortar lined steel, or heavy-wall polyethylene pipe and would be constructed within the Eucalyptus Road right-of-way. Just prior to the first (westernmost) well, the IRSP would move into a 100-foot wide easement, contiguous to the southern boundary of the Eucalyptus Road right-of-way. Separate turnouts with isolation valves would be provided to each individual well site. Figure 6 presents a schematic diagram of the IRSP and inland recharge area plan. Figure 7 shows the IRSP in profile and the preliminary well spacing and locations. Proceeding northeasterly, the pipeline would step down in size after the third well. Given the preliminary well spacing, the length of the IRSP in the easement would be approximately 1,300 feet, for a total IRSP length of about 3,800 feet. As shown on Figure 6, each well would have an isolation valve, flow meter and an air release valve at the well head to prevent air from entering the well.

Each inland injection well would be equipped with a well pump to backflush the well (as described elsewhere herein). The backflushing rate would be approximately 2,000 gallons per minute (gpm) and would require a 400 horsepower (hp) motor for a lift of about 500 feet. Pump speed would be variable by inclusion of a variable frequency drive (VFD), so that backflushing can be ramped up (manually) from initial lower flow to full flow, so as not to impact the formation in the vicinity of the well.

The backflush water would be discharged into a storage pond from which it would percolate into the vadose zone. The pond would have an approximate capacity of 200,000 gallons and would serve the backflushing needs of both injection wells. The size of the pond would be approximately 30-feet wide by 100-feet long by 8-feet deep. The pond would be located in the easement, likely between the two injection wells. The final location would be determined by a more detailed topographic survey.

For the coastal location, an additional pipeline is needed to extend from the RUWAP to the GWR injection site, following existing streets (Normandy Road to Monterey Road) as shown on Figure 8. This pipeline, called the Coastal Recharge Supply Pipeline (CRSP), would be made of similar materials as the IRSP. It would be constructed within the rights-of-way of Normandy Road and Monterey Road, as shown in Figure 8. Approximately 5,200 feet from the intersection of General Jim Moore Boulevard, the CRSP would depart public rights-of-way. A portion of the area in yellow on Figure 8 would be acquired for construction of the first set of recharge wells.

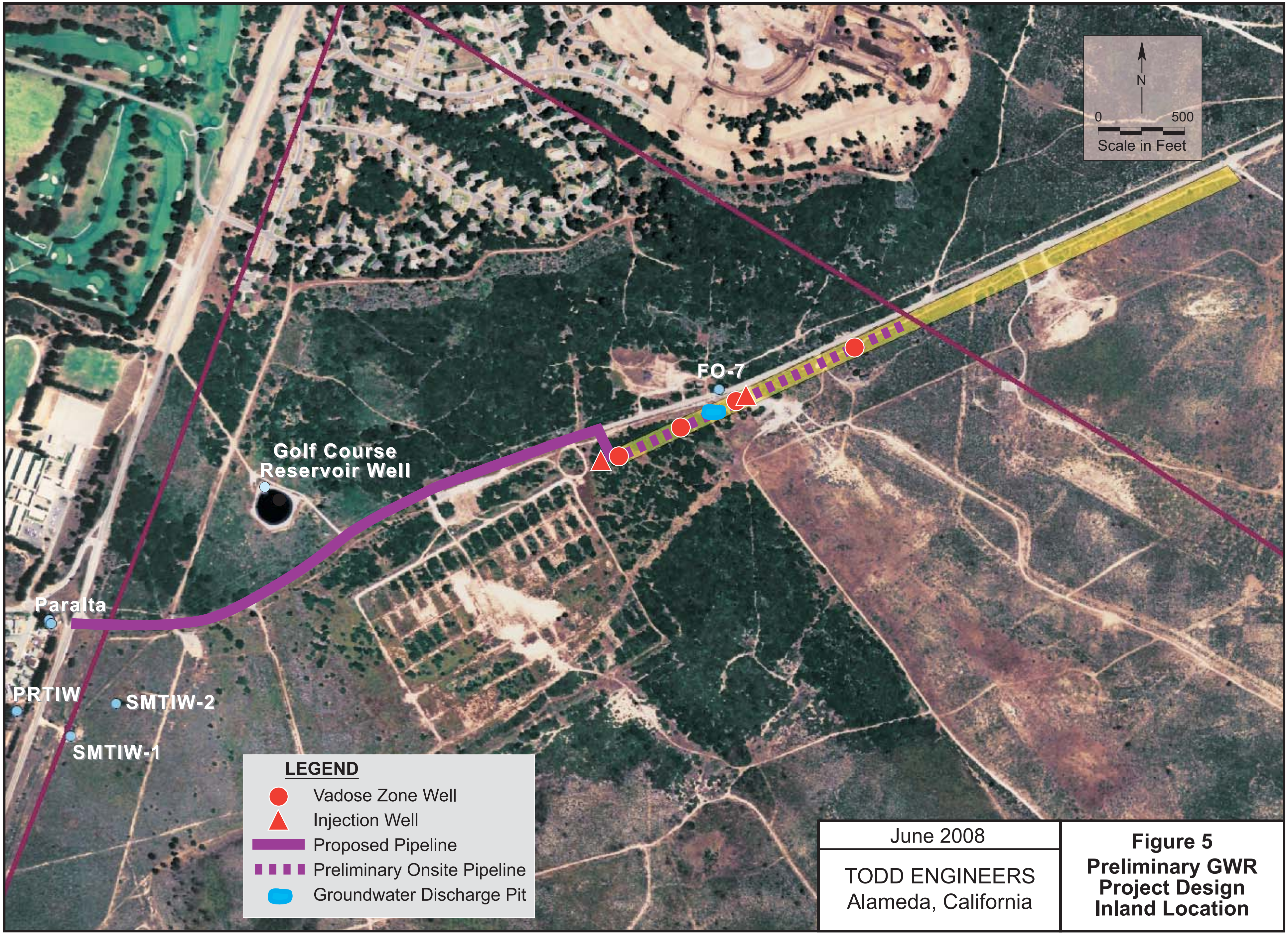

N
0
500
Scale in Feet
FO-7
Golf Course Reservoir Well
Paralta
PRTIW
SMTIW-2
SMTIW-1
LEGEND
Vadose Zone Well
Injection Well
Proposed Pipeline
Preliminary Onsite Pipeline
Groundwater Discharge Pit
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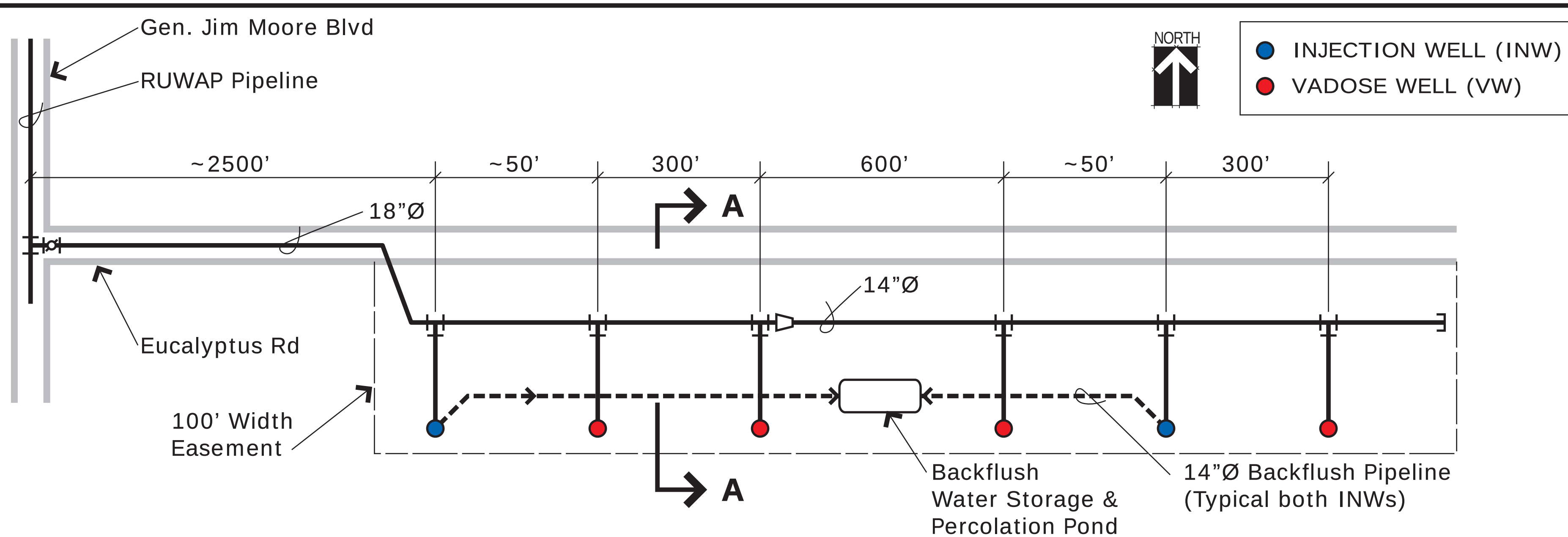
Figure 5
Preliminary GWR Project Design
Inland Location



Conceptual Schematic Inland Recharge Area Plan

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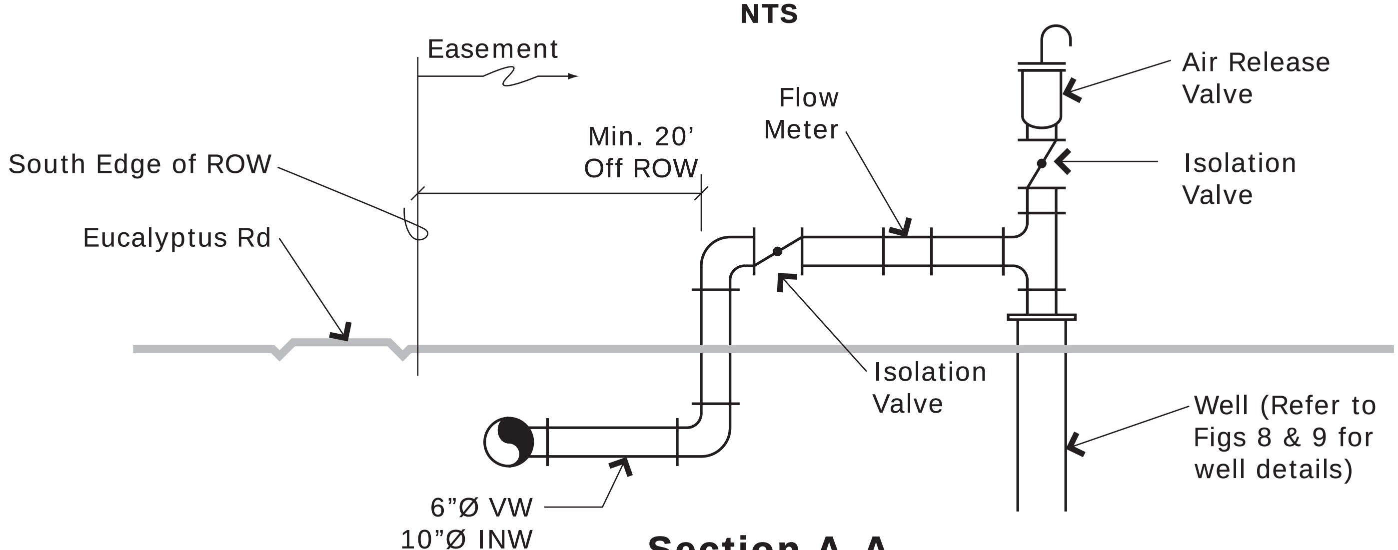

Section A-A

NTS

CDM

Figure 6
Conceptual Schematic - Inland Recharge Area

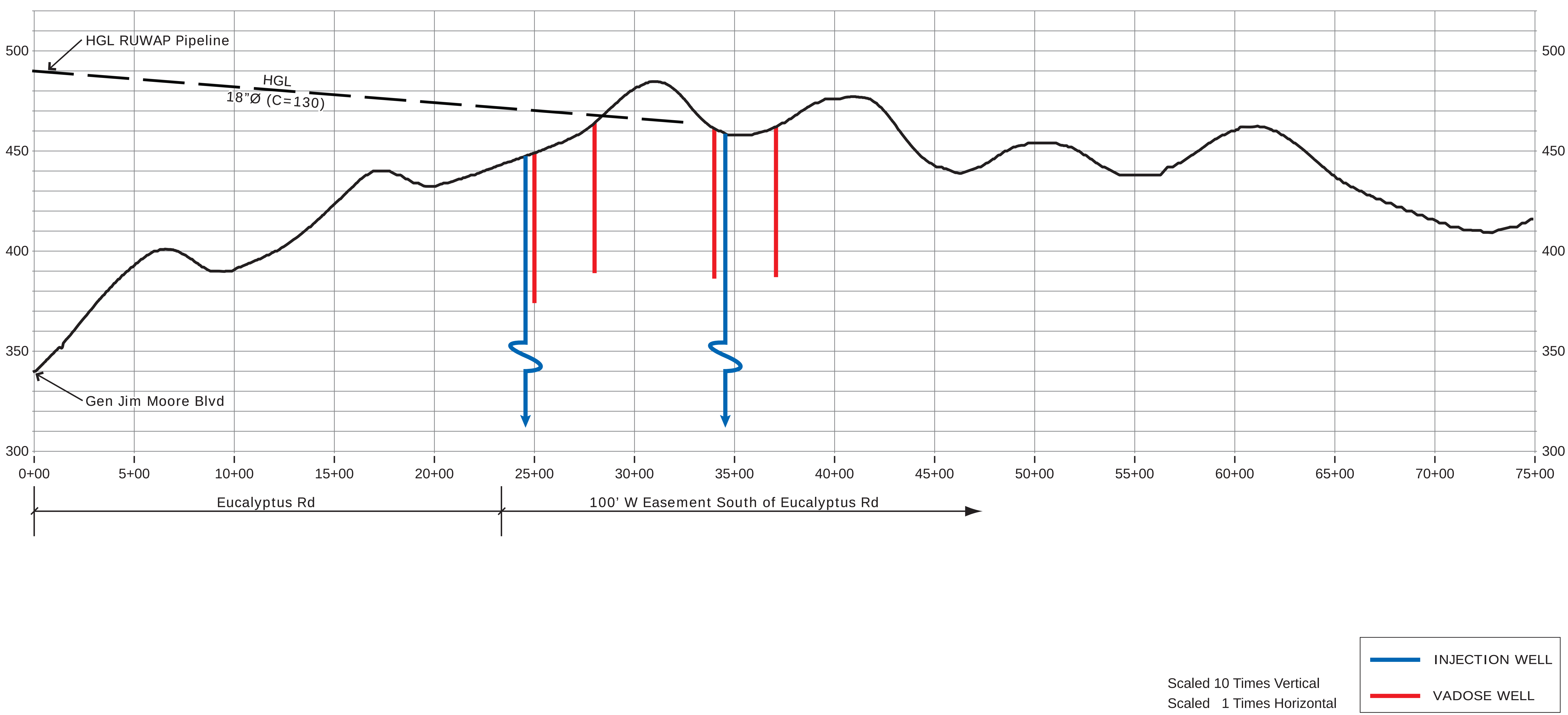

Figure 7
Pipeline Profile and Approximate Well Locations – Inland Rechargre Location

CDM

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Figure 8
Preliminary GWR Project Design Coastal Location

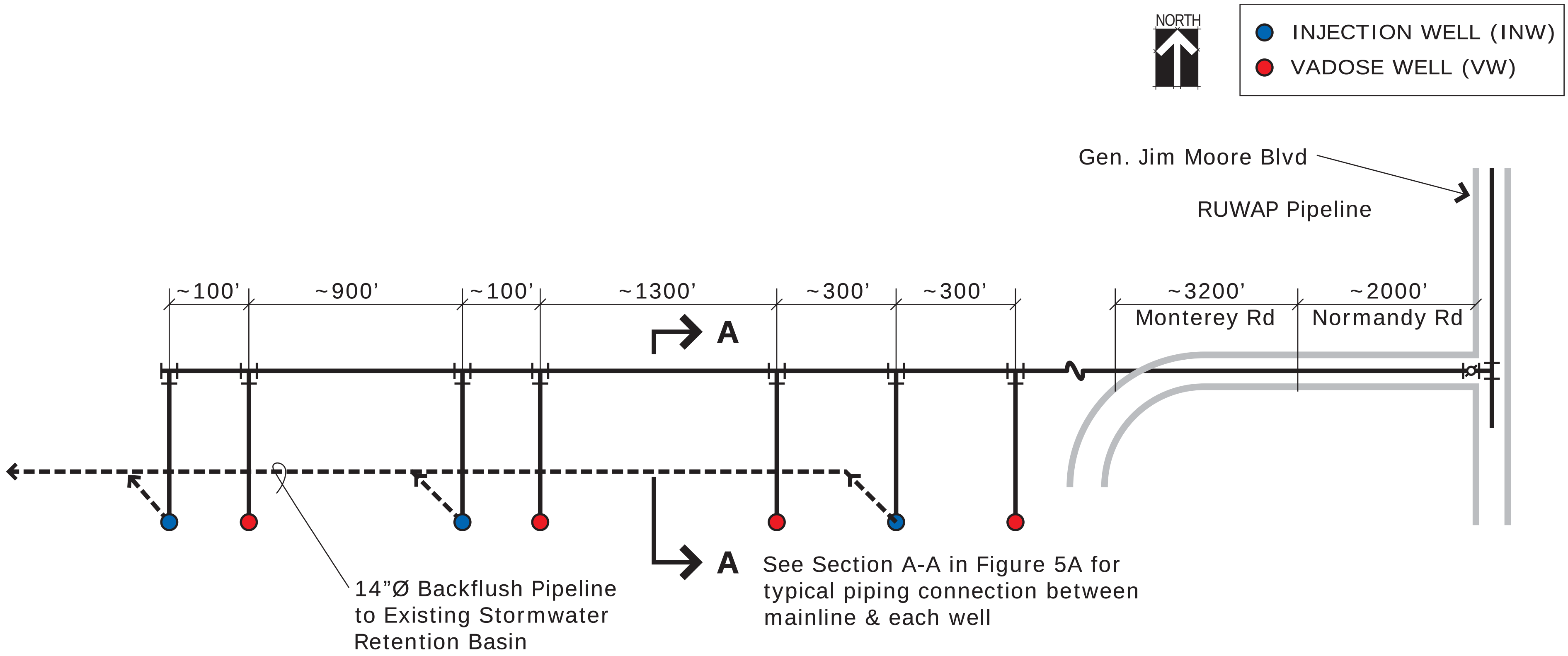
Separate turnouts with isolation valves would be provided to each individual well site. Figure 9 presents a schematic diagram of the CRSP. Figure 10 shows the CRSP in profile and the well spacing. Proceeding south-southwesterly, the pipeline would step down in size after the second well in the second well group. Given the preliminary well spacing, the length of the CRSP off roadway easements would be approximately 3,100 feet, for a total CRSP length of about 8,300 feet. As shown on Figure 9, each well would have an isolation valve, flow meter and an air release valve at the well head to prevent air from entering the well. Owing to the pressure in the RUWAP pipeline in General Jim Moore Boulevard, a pressure reducing valve may be necessary somewhere along the CRSP prior to the first recharge well. This will be determined by further hydraulic analysis.

Similar to the inland location injection wells, each coastal injection well would be equipped with a well pump to backflush the well. The backflushing rate would be 2,000 gpm, similar to the inland injection wells, but would require a smaller motor of 150 hp for a lift of about 200 feet. Similar to the inland injection well pumps, the electrical power to the motors would be controlled by VFDs.

Backflush water from the three injection wells would be conveyed in a separate pipeline down to a stormwater retention basin, located just west of Monterey Road, about 400 feet south of the intersection with Buna Road. The pipeline would be approximately 16-inch in diameter, made of polyethylene (or other synthetic material) and would be constructed (buried) in the easement. Given the preliminary topographic information, it appears that the backflush water could flow by gravity from each of the three injection wells to the retention basin. Backflushing would be scheduled to avoid coincidence with storm events.

2.3.2 Aquifer Characteristics

Aquifer characteristics for both the Paso Robles and Santa Margarita Aquifers were evaluated to develop the conceptual design for the recharge project and analyze the effects of recharge on basin aquifers. Data for aquifer characteristics at the inland location are provided by an adjacent MPWMD monitoring well (FO-7) and an MPWMD ASR well (Santa Margarita Test Injection Well-1 or SMTIW-1) located about 4,500 feet southeast of the site (see Figure 2). Figure 11 presents a portion of the geophysical and lithologic logs from FO-7 (note: lithologic unit thicknesses have been modified slightly from the original log to more closely correlate to the geophysical log). As shown on the logs, the Aromas Sand and undifferentiated older dune sands occur from the ground surface to a depth of about 420 feet below the site, providing a relatively permeable, although thick, vadose zone. The water table occurs at a depth of approximately 460 feet, providing a large amount of available storage for aquifer recharge. Previous interpretations of the top of the Paso Robles Aquifer in FO-7 placed the formation at about 280 feet based on lithologic color changes (MPWMD, 1994), but the top has been re-interpreted as further down in section to more closely correlate to the interpretations in SMTIW-1 (Padre, 2002). With either interpretation, the water table occurs in the upper portion of Paso Robles Aquifer.



Conceptual Schematic Coastal Recharge Area

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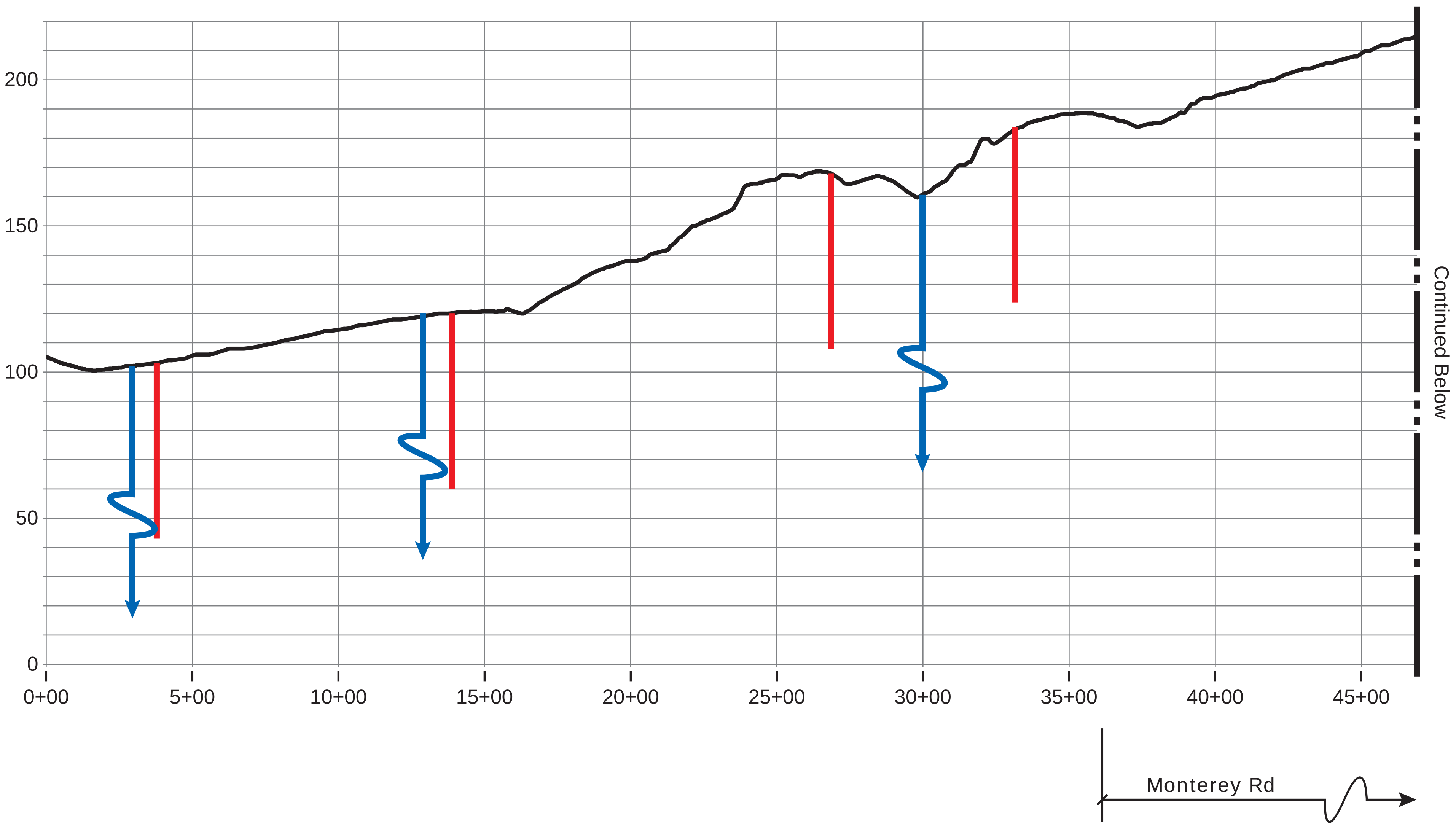


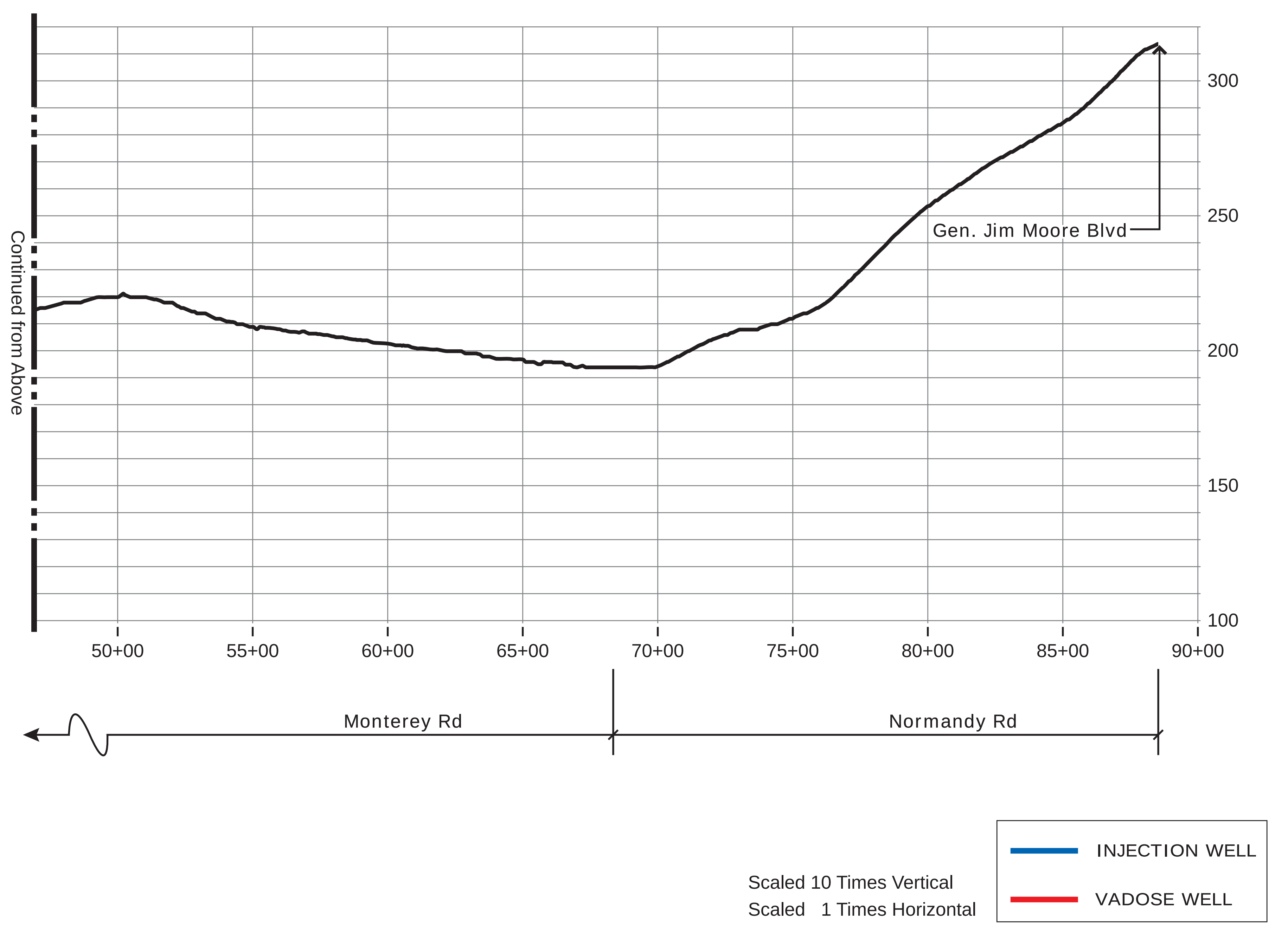
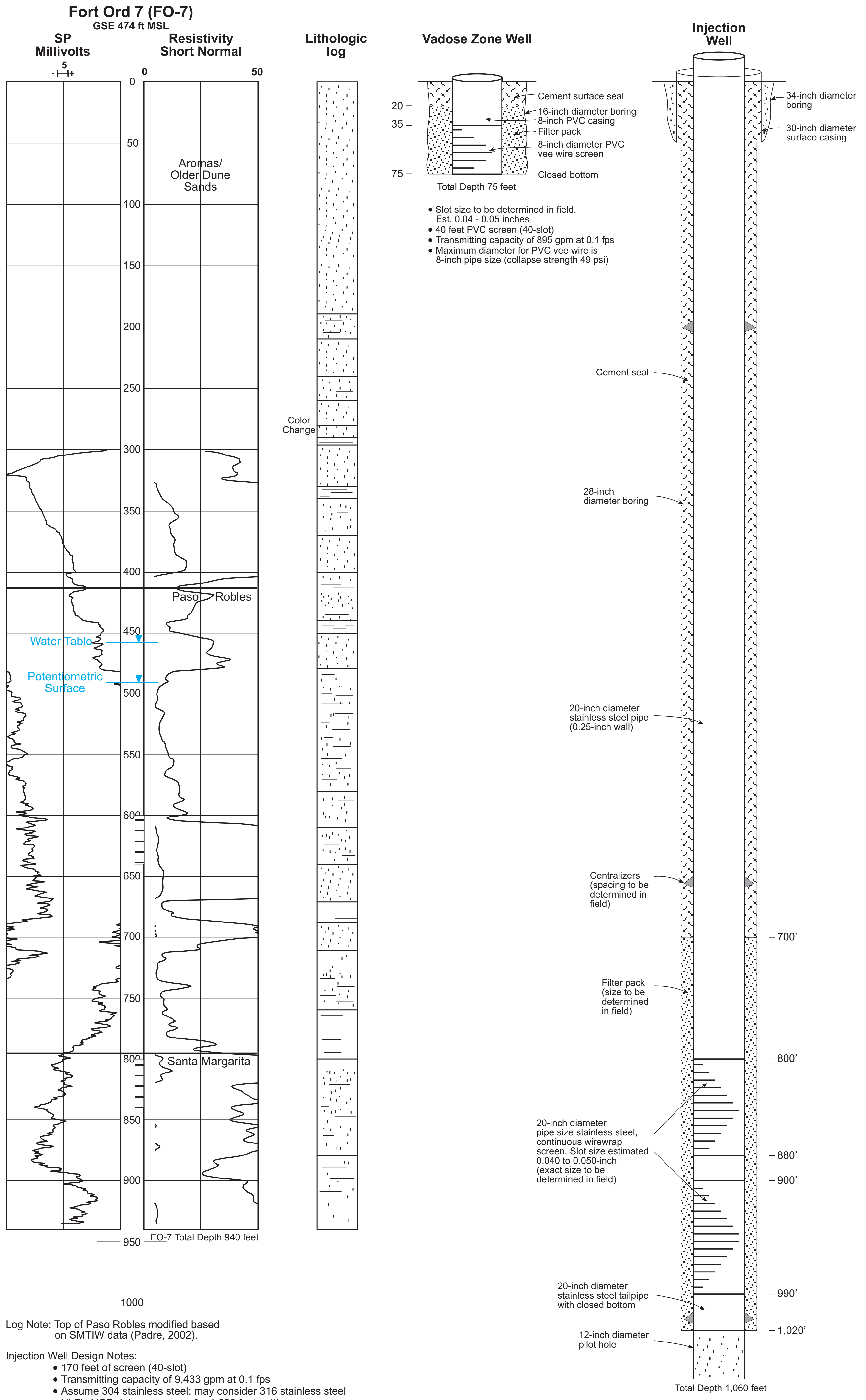


Figure 10
Pipeline Profile and Approximate Well Locations – Coastal Recharge Location



Log Note: Top of Paso Robles modified based on SMTIW data (Padre, 2002).

Injection Well Design Notes:
- 170 feet of screen (40-slot)
- Transmitting capacity of 9,433 gpm at 0.1 fps
- Assume 304 stainless steel: may consider 316 stainless steel
- Hi Flo VOD Johnson screen for 1,000 foot settings
- 14-inch nominal size pump bowls

June 2008	**Figure 11**
TODD ENGINEERS Alameda, California	**GWR Well Design Inland Recharge Location**

The Paso Robles Aquifer is interpreted to be approximately 350 feet thick and is characterized by heterogeneous interbeds of sand and clay mixtures. A relatively thick sand layer encountered from 600 to 670 feet is screened in FO-7, as indicated on Figure 11. Although no aquifer parameters are available from FO-7, data from a nearby MPWMD Paso Robles test well indicate a transmissivity (T) between 4,930 gallons per day per foot (gpd/ft) and 11,400 gpd/ft (659 ft^2/day and 1,524 ft^2/day) (Fugro, 1998). An average hydraulic conductivity (K) of about 20 feet per day (ft/day) has been estimated by others (Yates, et al., 2005) and is consistent with the T values. The vertical hydraulic conductivity (K_v) is estimated at about one-tenth of the horizontal K value. A storativity value of 0.12 is thought to be representative of the unconfined aquifer (Fugro, 1997). Aquifer characteristics are summarized in Table 2-1.

Table 2-1 also contains aquifer characteristics for the Santa Margarita Aquifer at the inland location. These data are primarily derived from detailed aquifer injection testing at SMTIW-1 and lithologic and water level data from adjacent monitoring well FO-7. Portions of the FO-7 geophysical and lithologic logs covering the Santa Margarita Aquifer are shown on Figure 11. In general, the Santa Margarita Aquifer is more permeable and homogeneous than the Paso Robles Aquifer. According to SMTIW-1 data, the unit is approximately 280 feet thick in this area, 150 feet of which were penetrated in FO-7. The potentiometric surface is approximately 15 feet below sea level (490 feet deep) as measured in FO-7 and shown on the top of Figure 4. Hydrographs indicate a downward vertical gradient from the Paso Robles Aquifer to the Santa Margarita Aquifer.

A number of evaluations of Santa Margarita Aquifer parameters, injection capacity, and water level impacts from injection have been conducted at the SMTIW wellfield and basin-wide in support of the MPWMD ASR project (Padre, 2002; MPMWD, 2002; ASR Systems, 2005, Jones & Stokes, 2006). Collectively, these evaluations have produced relevant data for predicting the potential performance of the GWR project. T values have been estimated basin-wide at about 85,100 gpd/ft (11,377 ft^2/day) (Fugro, 1997; MPWMD, 2002). A T value of 104,325 gpd/ft (13,947 ft^2/day) was estimated at SMTIW-1 (Padre, 2002). Even higher T values were thought to be reasonable for basin-wide groundwater model calibration (ASR Systems, 2005). Basin-wide K values average about 63 ft/day. Although T and K values vary among the numerous aquifer tests that have been conducted, data consistently indicate higher values than measured in the Paso Robles Aquifer. Data also indicate sufficient permeability to sustain relatively high long-term injection rates. Injection testing at the SMTIW wellfield indicates an injection capacity of about 1,500 gallons per minute (gpm) per well (Padre, 2002; ASR Systems, 2005).

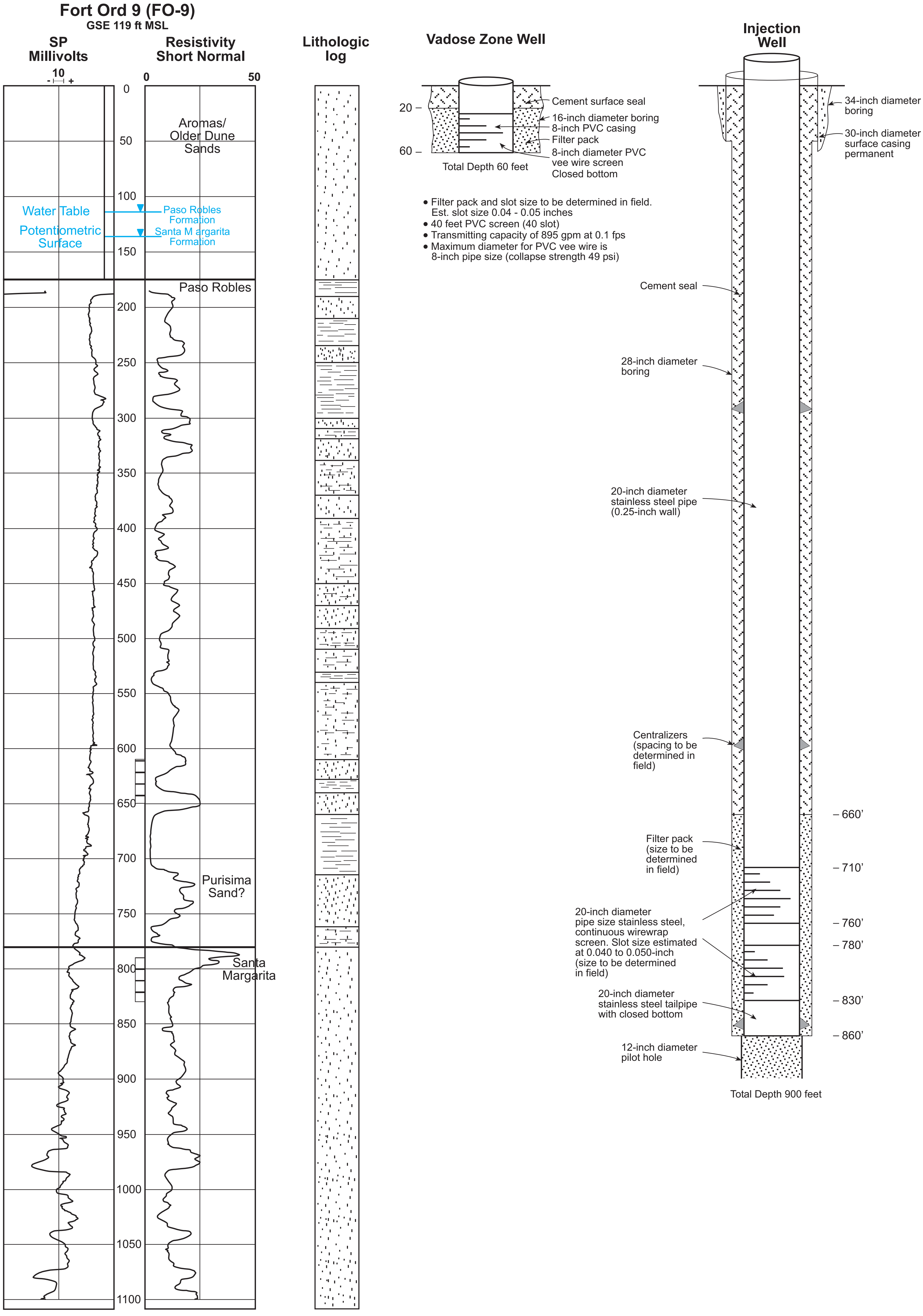
Table 2-2 summarizes aquifer characteristics at the coastal location. Relevant data for this location were provided by MPWMD monitoring wells FO-9, PCA-E and PCA-W, along with a nearby production well (PCA). A portion of the geophysical and lithologic logs from FO-9 are provided on Figure 12. Information from a recent coastal investigation by the Seaside Basin Watermaster was also incorporated (Feeney, 2007). As shown in Table 2-2, estimated T values for the Paso Robles Aquifer are 1,337 ft^2/day and 2,275 ft^2/day, similar to the inland location. An average T value of 21,924 ft^2/day has been used for Santa Margarita Aquifer evaluations, also similar to values listed for the inland location.

Table 2-1: Aquifer Characteristics – Inland Recharge Location
Seaside Groundwater Basin

Target Aquifer:	Paso Robles Aquifer	Santa Margarita Aquifer	Source
Lithology	Heterogeneous interbeds of sand, silt, and clay mixtures.	Fine- to medium-grained well sorted sand to silty sand; sandy silt in lower portions of formation; minor clay.	FO-7 Well (MPWMD, 1994) SMTIW 1 (Padre, 2002)
Interval Thickness	350 feet	280 feet	MPWMD, 1994; Padre, 2002
Aquifer Depth	420	790	MPWMD, 1994; Paso Robles re-correlated to Padre, 2002
Groundwater Conditions	unconfined	semi-confined	ASR Systems, 2005; Padre, 2002
Depth to Water	450 feet	500 feet	FO-7 Well (Watermaster Database, 2008)
Groundwater Elevation	16 feet, msl	-15 feet, msl	FO-7 Well (Watermaster Database, 2008)
Aquifer Parameters			
Transmissivity (T)	659 to 1,524 feet^2/day	11,377 to 13,947 feet^2/day	Fugro, 1997; MPWMD, 2002
Horizontal Hydraulic Conductivity (Kh)	20 feet/day	63 feet/day	Yates et al., 2005 (from Fugro, 1997); Padre, 2002
Vertical Hydraulic Conductivity (Kv)	0.2 feet/day;	0.63 feet/day	1:10 K_h/K_v estimated
Storativity (S)	0.12	0.0018 0.00142	MPWMD, 2002; ASR Systems, 2005

Table 2-2: Aquifer Conditions – Coastal Recharge Location
Seaside Groundwater Basin

Target Aquifer:	Paso Robles Aquifer	Santa Margarita Aquifer	Source
Lithology	Heterogeneous package of interbeds of sand, silt, and clay mixtures. Average bed thickness of 25 feet.	Fine- to coarse-grained sand; predominantly quartz with minor clay and chert fragments	FO-9 Well (MPWMD, 1994); PCA-E Well (SGD, 1990)
Interval Thickness	605 feet 522 to 667 feet	>330 feet 173 to 185 feet	FO-9 Well (SGD, 1990) PCA East/West (MPWMD, 2002)
Aquifer Depth	175 feet 100 to 125 feet	780 feet 622 to 792 feet	FO-9 Well (MPWMD, 1994) PCA East/West (SGD, 1990)
Groundwater Conditions	Upper unconfined; Lower confined?	confined	PCA Aquifer Test (SGD, 1990) MPWMD, 2002
Depth to Water	114	136	FO-9 Well (Watermaster Database, 2008)
Groundwater Elevation	5	-17	FO-9 Well (Watermaster Database, 2008)
Aquifer Parameters			
Transmissivity (T)	1,337 feet^2/day 2,275 ft^2/day	11,377 feet^2/day	PCA Aquifer Test (SGD 1990); ASR Systems, 2005; Fugro, 1997
Horizontal Hydraulic Conductivity (K_h)	3.5 feet/day (35 feet/day for vadose zone)	104	1:10 K_h/K_v estimated (Paso Robles and vadose zone); Santa Margarita (ASR Systems, 2005)
Vertical Hydraulic Conductivity (K_v)	0.35 feet/day (3.5 feet/day for vadose zone)	N/A	Paso Robles (Hydrometrics, 2006) Aromas Sand (Hydrometrics, 2006)
Storativity (S)	$5x10^{-5}$ - $8x10^{-4}$ Paso Robles; (0.25 vadose zone)	0.00081	PCA Aquifer Test (SGD, 1990); Estimated for vadose zone; ASR Systems, 2005



Injection Well Design Notes:

- 100 feet of screen (40-slot)
- Transmitting capacity of 5,549 gpm at 0.1 fps
- Assume 304 stainless steel: may consider 316 stainless steel
- 14-inch nominal size pump bowls

June 2008	Figure 12
TODD ENGINEERS Alameda, California	**GWR Well Design Coastal Recharge Location**

Although T and K values are thought to be similar to those at the inland location, geologic and groundwater conditions are significantly different. Due in part to dipping strata, both aquifers at the coastal location are thicker than encountered at the inland location. Although not differentiated separately in this well, the lower portion of the Paso Robles Aquifer and the Santa Margarita Aquifer shown on Figure 12 may be correlative to the Purisima Sands encountered in recently-drilled coastal wells (Feeney, 2007). Nonetheless, sufficiently thick and permeable sand units hydraulically connected to the Santa Margarita Aquifer appear to exist at this location.

At the coastal location, the water table is encountered about 60 feet above the Paso Robles Aquifer in the surficial Aromas/Older Dune Sands. An aquifer test in the PCA Well, located one mile south of FO-9, indicated a range of low S values for the lower Paso Robles Aquifer. These values are indicative of confined conditions and may suggest local hydraulic separation for lower Paso Robles Sand layers. However, the sandy nature of the upper Paso Robles Aquifer indicates hydraulic communication with the water level encountered in the Aromas Sand and provides assurance that water levels in the Paso Robles can be raised with recharge in this area.

The water table at the coastal location is about 114 feet deep with a groundwater elevation of about five feet above sea level. The potentiometric surface of the confined Santa Margarita Aquifer is encountered at a depth of about 136 feet with an equivalent elevation of about -17 feet below MSL. As shown on Figure 3, the area where water levels are below MSL in the Santa Margarita Aquifer extends past the subarea boundary to the north and likely extends offshore.

2.3.3 Recharge Locations and Wells

Groundwater conditions in the basin suggest that both aquifers would benefit from recharge. Although declines in groundwater levels have been more pronounced in the Santa Margarita Aquifer, groundwater storage has also been lost from the Paso Robles Aquifer. In addition, a preliminary evaluation indicates that replenishment could occur at the inland location, the coastal location, or both locations with slightly different objectives. The inland location provides upgradient recharge for downgradient extraction at production wells in an area of large available storage in the vadose zone. The coastal location provides the more direct protection from future seawater intrusion and could allow for increased inland extraction while maintaining coastal water levels above sea level. The selection of which wellfield to utilize or whether both should be utilized will be based, in part, on site investigations, environmental review, ongoing operation of the MPMWD ASR project, and other factors. For the purposes of this project description, both locations are included for a potential GWR site and both sites include vadose zone wells and injection wells. Vadose zone wells are used for recharge of the unconfined Paso Robles Aquifer and injection wells would directly replenish the confined Santa Margarita Aquifer.

The GWR project is anticipated to provide approximately 2,400 AFY during the five-month non-irrigation period for basin replenishment (continuous flow rate of about 3,600 gpm). Multi-year injection testing at nearby SMTIW-1 indicates that a long-term injection rate of about 1,500 gpm could be sustained. Because injection testing has not yet been conducted at either GWR project location, a reduced rate of 1,000 gpm is used for planning purposes. In addition, pilot testing has not yet been accomplished for vadose zone wells and recharge rates are unknown. However, injection rates into a MPWMD

Paso Robles injection well were estimated at about 350 gpm. For vadose zone wells, the recharge rate is anticipated to be much higher given the higher permeability estimates for the overlying Aromas Sand, which accounts for almost all of the vadose zone thickness. For planning purposes, a recharge rate between 150 and 400 gpm is anticipated for each well. For the inland location, four vadose zone wells (400 gpm each) are planned along with two injection wells. For the coastal location, four vadose zone wells (150 gpm each) and three injection wells are anticipated. Figures 5 and 8 show preliminary locations of vadose zone and injection wells for the inland and coastal locations, respectively.

Based on the aquifer characteristics described above, design criteria have been established for both vadose zone and injection wells for the GWR project. In addition, preliminary well designs have been developed. The following text summarizes the design criteria for the wells. Preliminary well designs for both aquifers at both locations are shown on Figures 11 and 12.

Paso Robles vadose zone well designs for both the inland and coastal locations are similar, although groundwater conditions and eventual operation may differ considerably. At the inland location, the water table is 450 feet deep and there is significant available storage in the vadose zone. In addition, there are no significant clay layers to bypass above about 400 feet (based on FO-7 data; see Figure 11). Given these conditions, the inland vadose zone wells extend to approximately 75 feet, providing for a cost efficient well design that can be inexpensively replaced if clogging (or air entrainment) becomes an issue. Spacing is about 600 feet between wells to minimize interference. The 9-acre site is sufficiently large to accommodate additional vadose zone wells if necessary. Figure 11 shows the preliminary vadose zone well design. Table 2-3 summarizes the vadose zone well design criteria.

Table 2-3: Summary of Vadose Zone Well Design Criteria

Component/Parameter	Criteria	
	Inland Location	Coastal Location
Number of wells	4	4
Depth to water table	450 feet	114 feet
Borehole Diameter	16 inches	16 inches
Borehole Depth	75 feet	60 feet
Casing/Screen Diameter	8 inch OD PVC with 40 feet vee wire screen (40 slot)	8 inch OD PVC with 35 feet vee wire screen (40 slot)
Injection	4-inch OD PVC eductor line	4-inch OD PVC eductor line
Injection Capacity	400 gpm	150 gpm
Annular Material	Artificial filter pack	Artificial filter pack
Additional Monitoring Equipment	Transducer and high water level alarm	Transducer and high water level alarm

For the coastal location, the depth to water is much shallower (114 feet). Vadose zone wells are projected to extend to about 60 feet; this is not only cost effective but will keep water levels from rising into the lower portion of the wells and causing rejection of recharge. To further manage water levels, the well spacing is greater than the inland location and recharge per well is less. Even though the water table is shallow, the vadose zone consists of the more permeable Aromas Sand, allowing for considerable groundwater storage per foot of vadose zone. However, as a conservative assumption for planning purposes, flow rates into the vadose zone wells are reduced to 150 gpm/well to ensure that water levels are controlled in the project area. Although there will be significant flow in the direction of the coast for protection against seawater intrusion, most of the recharged water is expected to flow inland toward the water level depression. The preliminary vadose zone well designs for the inland location and the coastal location are shown on Figures 11 and 12, respectively.

The design criteria for the Santa Margarita injection wells are similar for both the inland and coastal locations. For both locations, a 20-inch outer diameter casing/screen assembly is planned to accommodate a sufficiently large pump to provide for a discharge rate higher than the injection rate for well maintenance. For the inland location, approximately 170 feet of screen will be used based on the occurrence of the Santa Margarita Aquifer in nearby FO-7 and SMTIW-1 (Figures 5 and 11).

For the coastal location, the screen length is anticipated to be approximately 100 feet. This well also takes advantage of a lower Paso Robles (Purisima?) Sand layer immediately above the Santa Margarita Aquifer. Since recharge of the Paso Robles Aquifer is lower in the vadose zone wells, sufficient replenishment water for three injection wells would be available for the coastal location. Table 2-4 summarizes these design criteria for injection wells.

Table 2- 4: Summary of Design Criteria for Injection Wells

Component/Parameter	Criteria	
	Inland Location	Coastal Location
Number of Santa Margarita wells	2	3
Depth to potentiometric surface	500 feet	136 feet
Injection rate per well	1,000 gpm	1,000 gpm
Discharge rate per well	2,000 gpm	2,000 gpm
Well depth	1,060 feet	900 feet
Casing size and materials	20-inch OD stainless steel	20-inch OD stainless steel
Screen assembly	170 feet stainless steel	100 feet stainless steel
Pump	400 hp	150 hp

2.3.4 Backflushing

Decreasing injection capacity in injection wells over time can occur from numerous factors including air entrainment, filtration of suspended or organic material, bacterial growth, precipitates due to geochemical reactions, swelling of clay colloids, dispersal of clay particles due to ion exchange, and/or mechanical compaction of aquifer materials due to high injection rates. To regain lost capacity, wells are typically pumped periodically, a process referred to as backflushing. For backflushing, wells are usually pumped at an extraction rate that is twice the injection rate.

MPWMD found that periodic backflushing helped to regain lost capacity at the nearby SMTIW wells. At that site, a 240,000-gallon pit was dug and bermed for discharge during backflushing. Although optimal backflushing schedules are being evaluated, MPWMD reports that weekly backflushing may be most efficient. The backflushing pit was sized for 80 minutes of pumping at 3,000 gpm. Discharge water is allowed to infiltrate the bottom of the pit and percolate to groundwater. MPWMD reports that the SMTIW pit discharge water infiltrates within one day (J. Oliver, personal communication, April 2008).

For the injection wells, each one would be equipped with a well pump to backflush the well. The backflushing rate would be approximately 2,000 gallons per minute (gpm) and would be controlled by VFD pumps, so that backflushing can be ramped up (manually) from initial lower flow to full flow and so as not to impact the formation in the vicinity of the well. A backflushing pit is planned for both the inland and coastal locations. For the inland location, the pit will be constructed in between the two injection wells. For the coastal location, a stormwater detention basin already exists at the southern edge of the potential GWR site and appears sufficiently large to handle the required backflushing volumes. In lieu of constructing a separate pipeline for conveyance of backflush water from the northern portion of the recharge area, an additional pit may be constructed on the northern parcel of the coastal location. Backflushing operations can be scheduled between storm events to minimize interference with the stormwater basin's original purpose.

2.4 Project Operations

As previously described, the Seaside Basin GWR project will be operated during at least the five-month non-irrigation season with up to about 2,400 available for recharge. During the irrigation months, well rehabilitation and other activities for project optimization can occur, or additional water can be injected into project wells. A project monitoring program will be implemented and coordinated with other monitoring programs to allocate water between vadose zone wells and injection wells. Water from the AWTP will be piped to the GWR site and injected into project wells. The GWR project will be set up for operational flexibility, allowing amounts of recharge per well to vary over time. Injection well capacity is being conservatively estimated and additional vadose zone wells can be added inexpensively at either site.

The GWR project will be designed to comply with the most recent draft regulations for recycled water recharge as published by the CDPH. The project has been designed to comply with required distances from the recharge sites to other extraction wells residence times within the aquifer, as described in Chapter 5, Groundwater Analyses.

The GWR project will be operated to take advantage of additional replenishment water from the MPWMD ASR project, treated Salinas Industrial Water Treatment Plant (SIWTP) water, and potentially Blanco Drain and Reclamation Ditch water (see Chapter 3). These water sources could serve as diluent water (i.e., water used to dilute recycled municipal wastewater in a groundwater recharge project). The SIWTP (and Blanco Drain and Reclamation Ditch, if utilized) water will be treated in a separate stream of the AWTP and sampled prior to mixing so that it will qualify as diluent water. All source water coming to the AWTP will receive pre-treatment (e.g., pre-screening) to remove any large particles in the waste stream, thus protecting the MF membranes from damage and premature fouling. The size of the winter GWR project is the approximate size of the MPWMD/CWP ASR projects, which plan to recharge up to 2,426 AFY of excess Carmel River or CWP water when available. All four additional water sources can be considered to be diluent water for the purposes of complying with the CDPH draft recycled water recharge regulations. As required by the draft regulations, not more than 50 percent of the water extracted at any potable water supply well can be recycled water during the beginning years of a recycled water recharge project involving injection. Since two of the ASR wells are already in place and injection testing is underway, the ASR project will likely begin recharging prior to the implementation of the GWR project. Although the ASR project plans to extract an amount equal to recharge, the actual recharge water will migrate downgradient away from the well as recharge occurs. Groundwater analyses being prepared for the GWR are anticipated to show that this process, along with natural mixing in the aquifers, will ensure that not more than 50 percent of the water extracted at any basin well will be recycled water. If full year recharge of the GWR is planned, then it would probably be desirable to not build Phase 2 of the CWP ASR project.

Project performance will be monitored by a series of groundwater monitoring wells. The complete GWR monitoring program will be developed in consultation with regulators and in conjunction with existing basin-wide monitoring programs. For planning purposes, three groundwater monitoring wells are proposed for either the inland or coastal location. Two wells would be located downgradient from the project and at least one well would be placed upgradient. Wells will be constructed as nested wells or well clusters to allow for monitoring of both the Paso Robles and Santa Margarita Aquifers. The shallow completion would screen permeable intervals within the upper portion of the Paso Robles Aquifer (allowing for appropriate well development drawdown) to monitor vadose zone well performance. The deep completion would be screened across the Santa Margarita Sandstone injection interval. It is assumed that each monitoring well will contain two nests within one 8-inch steel casing, properly sealed to allow depth-specific sampling. All three wells (at either location) would be drilled to an approximate total depth of 900 feet. Final well design will be based on final project layout, well testing, and the location of other available monitoring points.

Energy Source and Requirements. For both the inland and coastal locations, power supply for the injection well pumps would be from a trailer-mounted portable generator, since the flushing operation is intermittent and is required only a few hours per month. Low voltage (240-V) power would be required for each motor control center (MCC), instrumentation, telemetry, emergency lighting and other uses. For the inland location, PG&E has a major power line in General Jim Moore Boulevard. A “drop” from PG&E would be required and the voltage transformed down to 480-V (volts) for conveyance to the recharge area and then stepped down again at each pump MCC. Whether the power would be conveyed overhead or underground is to be determined. An above-ground, all-weather electrical cabinet would be provided at each of the two injection wells to house breaker, starter, VFD and other small electrical equipment components. The cabinet would be equipped with a quick coupling for connection of the portable generator. Backflushing of the wells would be performed on only one well at a time, so the portable generator would be sized to meet the demand of one pump at a time.

A portable generator will also be used for the coastal injection wells. There are power lines in the vicinity of all three of the coastal injection wells since the area has been developed for residential land uses. It is likely that there will be two separate drops, one for the upper area and one for the lower area. The PG&E line voltage will also require stepping down to 240-V at each well. Whether the power would be conveyed overhead or underground is to be determined. Also to be determined is whether the voltage would be stepped down at the drop or at each injection well site. An aboveground, all-weather electrical cabinet would be provided at each of the two injection wells, similar to the cabinets for the inland injection wells. Backflushing of the wells would be performed only one at a time. The portable generator for the inland injection wells will have adequate capacity to meet the demand of one coastal injection well pump at a time.

The annual power demand is estimated at 25,000 kW-hr and 15,000 kW-hr for the inland recharge and coastal recharge locations, respectively.

2.5 Permits and Approvals Required

This section provides a preliminary identification of the permits and agreements required for the Seaside GWR project, and a time estimate for completing them.

- Seaside Basin Watermaster review and approval: 2 – 4 months
- Monterey County Division of Environmental Health Discharge Conditions Permit: 6 – 9 months
- RWQCB WDR: 4 – 6 months
- Monterey County Health Department Division of Environmental Health drilling permits: 1 – 2 months
- Monterey County Water Resources Agency well/extraction permit: 3 – 6 months
- Monterey County Health Department Division of Environmental Health (for WDR modification): 4 – 6 months
- RWQCB permit for drilling (if needed): 3 – 6 months
- CDPH approval for recharge: 12 – 18 months
- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months
- City of Seaside agreement to use water from golf course wells for dilution water. Requires Watermaster permission that Seaside would not be charged with extracting water since it would not be returned to the aquifer: 12 – 18 months. This agreement is needed only if there is a delay in acquiring SIWTP water for dilution.
- Agreement with MCWD for RUWAP and possibly for dilution water: 6 – 12 months. An agreement for dilution water is only needed if there are delays in the SIWTP water for dilution and with obtaining alternate water from the City of Seaside.
- Fort Ord Reuse Authority agreement to install pipeline in Eucalyptus Road. Possible easement for future City of Seaside property if not transferred by then: 9 – 12 months
- Army – Agreement for Coastal or Encroachment and/or right-of-way permits from City of Seaside for pipelines and wells: 12 – 15 months
- City of Seaside encroachment agreement for future well access: 6 – 12 months
- Coastal Commission Permit (if required): 9 – 12 months
- PG&E power: 12 – 15 months

2.6 Construction Methods

Pipeline Construction Methods and Types of Equipment. Pipelines will be constructed by open trench construction methods using a combined backhoe for excavation and front-end loader for moving dirt around the construction site. Maximum trench width will be 36 inches for 20-inch pipe and narrower for smaller pipe. Pipe will be installed with a minimum of four feet of cover in areas usually or potentially subjected to traffic. Production rates assumed are 500 feet per day for the pipeline in the Eucalyptus Road right-of-way and 250 feet per day for all other locations. Dump trucks will be required to haul off excess excavation. The time to construct the pipelines and well connections for the inland and the coastal recharge area is estimated at 10 months each. Final testing and commissioning would require up to another two months, and potable recharge or diluent water will need to be available. These projects would be constructed simultaneously by one contractor with two separate pipeline crews, or by two separate construction contracts.

Actual field construction of pipeline work is estimated to take about 6 to 8 weeks for the inland recharge area and about 12 to 14 weeks for the coastal area, including repaving the areas in roadways. The main portion of the construction period is for the preparation, review/approval of shop drawing and fabrication of the pipe, valves, meters and electrical starter cabinets for the injection wells.

Well Construction Methods and Types of Equipment. Vadose zone wells are relatively quick and inexpensive to construct. Total depths are shallow (60 to 75 feet) and no well development is anticipated. The drilling method will be based on initial site investigations but will consider methods to minimize the use of fluids and potential borehole damage. Cable-tool drilling will be considered as well as other shallow-borehole methods such as bucket auger drilling. The four vadose zone wells are expected to take about two weeks to drill and install. Injection wells will be drilled with the reverse rotary method. Well development (if required) and injection testing will also occur at each well. Given the depth, deep seal, and development requirements, each well is expected to take approximately one month to install and develop. Drilling fluids will be containerized onsite.

Construction Staging. For the inland recharge location, it is assumed that there is adequate space along the Eucalyptus Road right-of-way to provide for staging and storage, particularly in the approximate 100-foot by 1,200-foot easement to be acquired for the well sites. Staging and storage will be required for pipe, temporary storage for excavated materials, small construction trailer, equipment parking, worker parking and the like. For the coastal recharge location, there is adequate open space for construction staging along the bluff just off Monterey Road near the first (northernmost) vadose well and the first injection well. An area of approximately ½-acre will need to be acquired for providing the same staging and storage requirements identified for the inland recharge location. Another small staging area should be considered in the lower area parallel to Highway 1 in the 1,300-foot long strip between the second injection well and fourth vadose well. This area could be used for pipe storage and excavation equipment parking, as well as temporary storage for excavated material.

Daily Construction Truck Trips. During pipeline construction activities, approximately five 10-cubic yard truck trips per day will be required for about two weeks for the inland pipeline installation. For the coastal pipeline installation, approximately three 10-cubic yard truck trips per day will be required for about four weeks during construction in the city roadway areas, and about 5 to 6 truck trips for construction in the off-road area, where both the recycled water supply pipeline and the backflush disposal pipeline are constructed in the same trench.

Number of Construction Workers. For the well construction, a crew of two to five construction workers will be onsite for the construction of each well, along with a project geologist to log the cuttings and oversee the work. For the pipeline construction, a crew of six construction workers will be required for pipeline installation, including two laborers, one truck driver, one backhoe operator, one small crane operator (may not be necessary for PVC or HDPE pipe), and a foreman. For alignments in city streets, two flaggers will be required for traffic control and safety. It is assumed that only one pipeline heading will be under construction in each of the two recharge areas. Also, it is assumed that the backflush water conveyance pipeline in the coastal area will be constructed in the same trench as the supply pipeline.

Quantity of Excavation and Backfill. For the wells and appurtenances, vadose zone well drilling will generate only about 3 to 5 cubic yards of cuttings and material per well. For four wells at each location, the amount of material anticipated is about 12 to 20 cubic yards. Each injection well will generate approximately 150 to 175 cubic yards of cuttings. This translates to about 350 cubic yards for the inland location and up to about 525 cubic yards for the coastal location. These materials are non-hazardous and can be stockpiled on location or spread onsite as was done at the MPWMD Santa Margarita Aquifer test injection wellfield. For the conveyance pipelines, the basic assumptions are that each pipeline will have a minimum four feet of cover, trench widths will be 12 inches wider that the pipe outside diameter, and native material will be suitable for backfill in the pipe zone, owing to mostly sandy conditions along the proposed pipeline routes. Using these assumptions, approximately 460 cubic yards of excess excavated material will be required to be hauled off for disposal for the inland recharge location, and approximately 950 cubic yards of excavated materials will be hauled off for the coastal recharge location. These volumes include the connector pipelines from main line to each well and the backflush water disposal pipeline at each recharge location.

Chapter 3 – DILUENT WATER PROJECT

3.1 Introduction

State regulations require that, for projects involving injection of reclaimed water into groundwater basins used for potable supply, the reclaimed water be diluted by "diluent water." For the MRWPCA project, there are three potential sources of water that, after treatment, can be used as diluent: Salinas Industrial Water Treatment Plant (SIWTP) effluent, Blanco Drain water, and Reclamation Ditch water. Up to three pump stations and pipelines, one for each water source, will be needed to collect and deliver these dilution waters to the RTP for treatment, sampling, and distribution for groundwater replenishment. The diluent water project facilities and locations are shown on Figure 1.

3.2 Salinas Industrial Water Treatment Plant Effluent

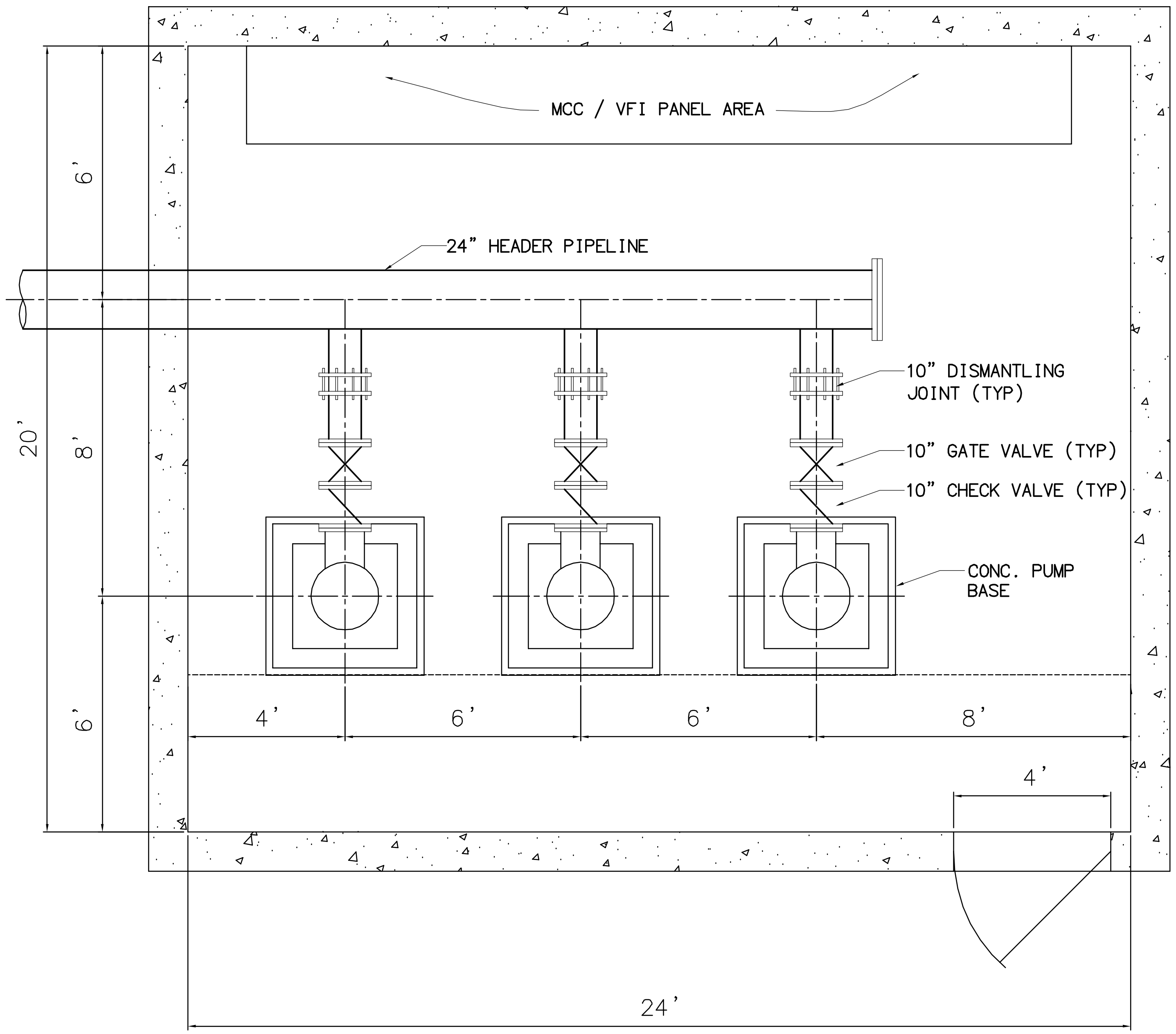
One source of injection water will be commercial effluent obtained from the SIWTP, which is located north of Davis Road adjacent to the Salinas River. The SIWTP treats commercial wastewater from vegetable packers and processors, seafood processors, preserve manufacturers, etc. The influent to the SIWTP is treated through aeration ponds and is currently discharged to disposal beds and percolation ponds. MRWPCA operates the system for the City of Salinas.

3.2.1 Avaliability of Effluent

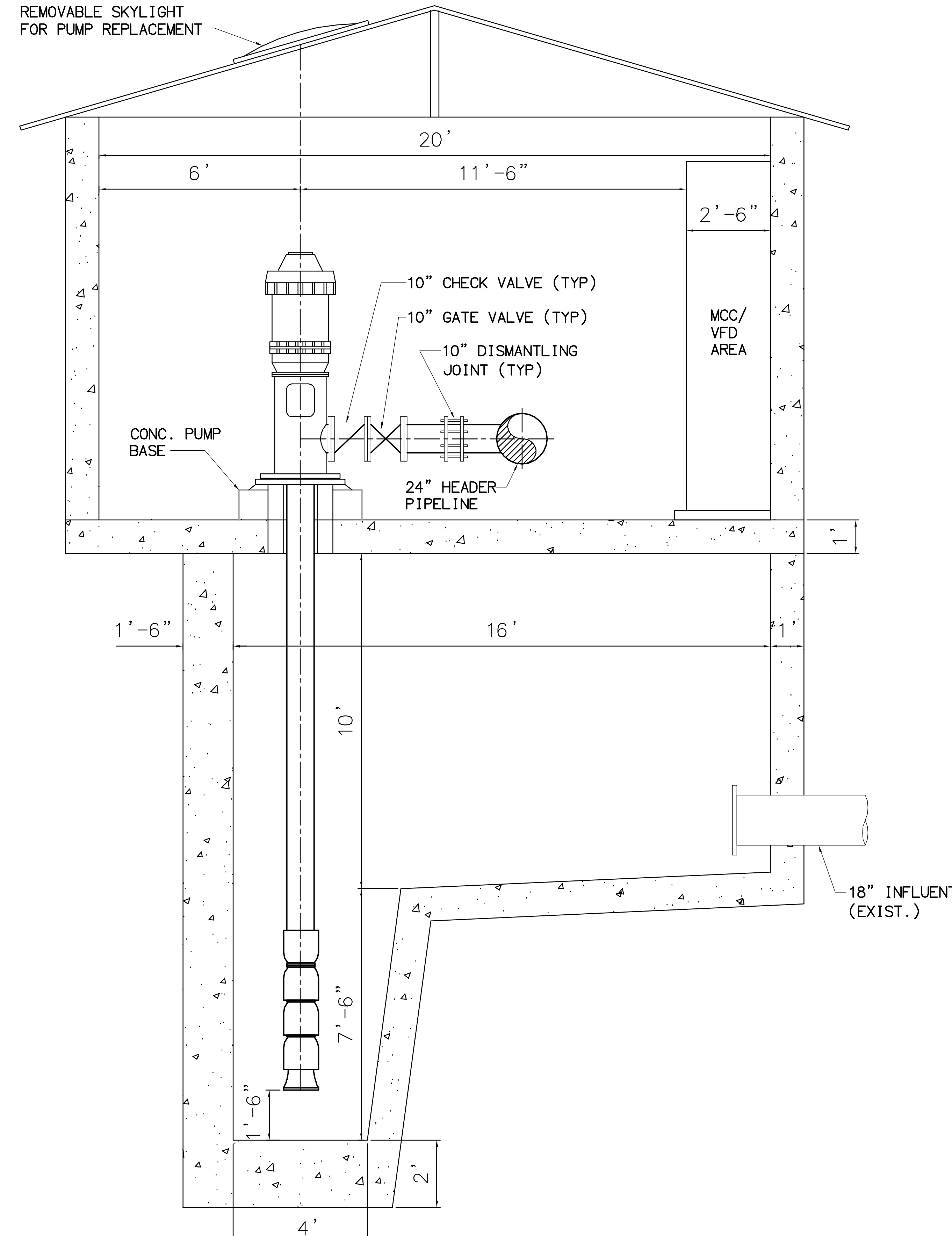
Commercial effluent will be pumped from Pond #3 at the SIWTP and conveyed through a proposed 24-inch PVC pipeline to the AWTP. After treatment through a separate flow stream of the AWTP, the commercial effluent will be injected into the Seaside groundwater replenishment sites. The average monthly flow rates into the SIWTP ponds for 2007 are shown in Table 3-1. According to representatives of the City of Salinas (C. Niizawa, personal communication, May 2008), the average flow rate from the SIWTP ponds during the irrigation season (approximately April to October) is projected to increase from the current flow of approximately 3 mgd to 10 mgd in the year 2030. The average flow rate during the non-irrigation season (approximately November to March) is projected to increase from the current flow of 1.3 mgd to 4.4 mgd.

3.2.2 Industrial Effluent Pump Station

A new 7.6 mgd industrial effluent pump station (IEPS) would be required to convey water from the SIWTP ponds (approximate elevation of 32 feet MSL) to the proposed AWTP (approximate elevation of 145 feet MSL) located at the RTP. Commercial effluent water from Pond #3 would be discharged to the IEPS through the existing 18-inch diameter outlet pipeline near the southwest end of Pond #3. Table 3-2 provides the IEPS design criteria. Figure 13 illustrates the IEPS layout.



PLAN
SCALE: 1/2" = 1'-0"



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INDUSTRIAL EFFLUENT PUMP STATION

FIGURE NO. 13

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Table 3-1: Average Monthly Flow Rates Into SIWTP Ponds (2007)

Month	Average Flow Rate (mgd)
January	1.0
February	1.1
March	1.1
April	2.2
May	2.6
June	2.8
July	2.8
August	2.8
September	2.8
October	2.6
November	2.2
December	1.2
Average (irrigation season)	***2.7***
Average (non-irrigation season)	***1.3***

Table 3-2: Industrial Effluent Pump Station Design Criteria

Pump Station Description	Q_{max} (gpm)	Static Head (ft)	TDH (ft)	# of Pumps	Pump Description	Pump Configuration
Commercial Effluent	5,210	113	185	3	VFD - 150 HP (3.75 MGD)	2 active, 1 stand-by

3.2.3 Industrial Effluent Pipeline

The SIWTP effluent will be conveyed from the proposed IEPS to the AWTP through a proposed 24-inch diameter pipeline. Table 3-3 provides the industrial effluent pipeline design criteria. The industrial effluent pipeline alignment would begin at the IEPS and continue north through farmland to W. Blanco Road. The alignment will continue west on Blanco Road and then turn north on Cooper Road to Nashua Road. Where the alignment parallels public roads, the pipeline would be constructed within the public right-of-way but out of the paved road (in the shoulder) when possible. Once at Blanco Drain, the pipeline will continue west to the Salinas River Diversion Facility (SRDF), currently under construction. The pipeline crossing at the Salinas River is proposed to be incorporated into the SRDF project.

Table 3-3: Industrial Effluent Pipeline Design Criteria

Pipeline Description	Pipe Material	Pipe Class (psi)	Length (ft)	Diameter (in)	Qmax (gpm)	Vmax (ft/s)
Industrial Effluent Pipeline	PVC (C905)	150	34,500	24	6,950	4.9

3.3 Reclamation Ditch Water

The Reclamation Ditch Watershed includes 157 square miles mostly within Monterey County and is operated by the Monterey County Water Resources Agency (MCWRA, 2006). The watershed drains the northwestern slopes of the Gabilan Range as well as much of the City of Salinas and its surrounding lands. The Ditch was created between 1917 and 1920 and is a network of excavated earthen channels used to drain surface runoff generated in the watershed. Urban runoff from the City of Salinas also drains into various channels of the Reclamation Ditch system via numerous stormwater outfalls. The system drains into Tembladero Slough, then the Old Salinas River Channel, and ultimately into Moss Landing Harbor through the Potrero Tide Gates. The ditch system provides for stormwater and flood control during the winter, but consists mostly of agricultural tail water during the summer months.

3.3.1 Availability of Water

Reclamation Ditch water will be collected in a sump along the Ditch before entering Tembladero Slough. Water will be pumped from the sump and conveyed through a proposed 24-inch PVC pipeline to the AWTP. After treatment through a separate flow stream of the AWTP, the water will be injected into the groundwater replenishment sites. The average monthly flow rates from the Reclamation Ditch are shown in Table 3-4. Those flows are based on a gage located upstream of the proposed Reclamation Ditch pump station site. The gage location measures flow for a drainage area of approximately 53 square miles or about one third of the Ditch's full drainage area (157 square miles). It is expected that the flow near Tembladero Slough could be three times as large, and the proposed pump station design reflects that expected flow.

Table 3-4: Average Monthly Flow Rates In Reclamation Ditch (2007/08)

Month	Average Flow Rate (mgd)[1]
January	23.2
February	19.5
March	1.7
April	2.1
May	1.7
June	1.6
July	2.3
August	2.4
September	2.2
October	2.4
November	1.7
December	4.0
Average (irrigation season)	***2.1***
Average (non-irrigation season)	***9.9***

[1] Data from USGS for site 1152650, which includes 53.2 square miles of drainage area of the total 157 square miles

3.3.2 Reclamation Ditch Pump Station

A new 7.6 mgd Reclamation Ditch Pump Station (RDPS) would be required to convey water from the Reclamation Ditch sump (approximate elevation of 32 feet MSL) to the proposed AWTP (approximate elevation of 145 feet MSL) located at the RTP. Ditch water would enter the RDPS through a screened 24-inch opening from the Ditch. Table 3-5 provides the RDPS design criteria. The RDPS layout would be the same layout as the IEPS (shown in Figure 13).

Table 3-5: Reclamation Ditch Pump Station Design Criteria

Pump Station Description	Q_{max} (gpm)	Static Head (ft)	TDH (ft)	# of Pumps	Pump Description	Pump Configuration
Reclamation Ditch	5,210	113	185	3	VFD - 150 HP (3.75 MGD)	2 active, 1 stand-by

3.3.3 Reclamation Ditch Water Pipeline

The Reclamation Ditch water will be conveyed from the proposed RDPS to the AWTP through a proposed 24-inch diameter pipeline. Table 3-6 provides the Reclamation Ditch pipeline design criteria. The Reclamation Ditch pipeline alignment would begin at the RDPS and continue south and east through farmland to the point where the proposed industrial effluent pipeline would cross the Blanco Drain. Where the alignment parallels public roads, the pipeline would be constructed within the public right-of-way but out of the paved road (in the shoulder) when possible. Once at the Blanco Drain, the pipeline will continue west to the proposed SRDF. The pipeline crossing at the Salinas River is proposed to be incorporated into the SRDF project. The Reclamation Ditch water pipeline alignment upstream of Blanco Drain is still under consideration.

Table 3-6: Reclamation Ditch Pipeline Design Criteria

Pipeline Description	Pipe Material	Pipe Class (psi)	Length (ft)	Diameter (in)	Qmax (gpm)	Vmax (ft/s)
Reclamation Ditch Pipeline	PVC (C905)	150	34,500	24	6,950	4.9

3.4 Blanco Drain Water

The Blanco Drain Watershed includes about 6,000 acres of farmland within the Castroville Seawater Intrusion Project (CSIP) area. The water is predominantly agricultural runoff but includes natural storm runoff. It drains into the Salinas River just upstream of the new SRDF. The Blanco Drain water has been documented to have high concentrations of salt and coliform (The Watershed Institute, 2002). It also has been documented to contain the insecticides Chlorpyrifos and Diazinon. The MCWRA owns and is responsible for the Blanco Drain system and its water. Similar to the Reclamation Ditch system, the Blanco Drain system provides for stormwater flows during the winter but consists mostly of agricultural tail water during the summer months.

3.4.1 Availability of Water

Blanco Drain water will be collected at the current pump location before it enters the Salinas River. Water will be pumped from the pump station and conveyed through either the industrial effluent pipeline or Reclamation Ditch pipeline from that location to the AWTP. After treatment through a separate flow stream of the AWTP, the Blanco Drain water will be injected into the groundwater replenishment sites. The average monthly flow rates from Blanco Drain are shown in Table 3-7. The flows shown are based on pumped flows. Some flow bypasses the existing pump station and flows directly to the Salinas River.

Table 3-7: Average Monthly Flow Rates In Blanco Drain (2007)

Month	Average Flow Rate (mgd) [1]
January	NA
February	NA
March	NA
April	0.5
May	1.7
June	2.7
July	3.0
August	2.7
September	1.8
October	0.6
November	NA
December	NA
Average (irrigation season)	***1.9***
Average (non-irrigation season)	***NA***

[1] Data from MCWRA did not include winter flows.

3.4.2 Blanco Drain Water Pump Station

The existing Blanco Drain Pump Station (BDPS) would be upgraded to provide 3 mgd of flow at a higher pressure. The BDPS would convey the water from an approximate elevation of 5 feet MSL to the proposed AWTP at approximate elevation of 145 feet MSL located at the RTP. The upgraded BDPS design criteria will be developed at a later time.

3.4.3 Blanco Drain Water Pipeline

The Blanco Drain water will be pumped 25 feet and connect into the industrial effluent pipeline or to the Reclamation Ditch pipeline, whichever has excess capacity, to be conveyed from the existing BDPS to the AWTP.

3.5 Project Operations

The destinations for distribution of the advanced treated water are described in Table 3-8 for the initial year of operation (2013). Table 3-9 illustrates how the water source changes over the years. As more diluent water is available, less RTP water will probably be used.

Table 3-8: Destination for AWTP Water

Season	AWTP[1] Product	GWR[2]	RUWAP[3]
Seaside GWR Injection	2,428[4] to 6,216[5]	2,386[4] to 5,533[5]	42[4] to 683[5]

[1]AWTP = Advanced Water Treatment Plant.
[2]GWR = Seaside Groundwater Replenishment Project Injection.
[3]RUWAP = Regional Urban Water Augmentation Project, Initial phase with some of urban users connected.
[4] Phase 1 with just winter injection in Seaside Aquifer.
[5] Phase 2 with full year injection in Seaside Aquifer.

Table 3-9: Source of AWTP Water (in AF)

Year	Source Water (Maximum in AF)					
	RTP	Diluent[1]	Total[2]	MFBW[3]	Brine	Product[4]
2013 Winter[5]	0 to 1,807[6]	1,269 to 3,076	3,076	243	405	2,428
2013 Summer	0 to 709[6]	3,619 to 4,328	4,328	342	569	3,788
2030 Winter	0 to 1,507[6]	2,040 to 3,547	3,547	280	467	2,800
2030 Summer	0	4,965	4,965	392	653	3,920

[1]Diluent=Salinas Industrial Effluent, Reclamation Ditch, and/or Blanco Drain Water.
[2]Total= Influent flow to AWTP. Includes MF backwash water.
[3]MFBW=MF backwash water which adds to MRWPCA's allocation as a recycled water flow.
[4]Product water=AWTP water for injection, RUWAP, or other use.
[5]First full year of operation.
[6]RTP water limited by allocation of 2,400 AFY plus MFBW.

The IEPS, RDPS, and/or BDPS would operate throughout the year to deliver diluent water to the AWTP via the industrial effluent pipeline and Reclamation Ditch water pipeline. Initial operation would include instantaneous pumping of at least 4 mgd, with an average of 3 mgd pumped during the irrigation season and an average of 1 mgd pumped during the non-irrigation season. Ultimate operations would pump up to 7.6 mgd to the AWTP. Up to 6 mgd of advanced treated water would be produced.

Energy Source and Requirements. The source of energy for the IEPS will be obtained at the existing industrial wastewater facility. Table 3-10 shows the power, energy and time estimates. The source of energy for the RDPS will be from a new PG&E drop at the pump station site. The same (Table 3-10) power, energy, and time estimates for the IEPS apply to the RDPS. Further study is needed to determine it an upgraded power drop will be required for the new Blanco Drain Pump Station.

Table 3-10: Power, Energy and Time Estimates for IEPS

	Power (Hp)	Time (hr)	Energy (kW-hr)
IEPS*	450 (installed)	8,760 (12 months)	1,955,000

*Two pumps will run during the irrigation season (7 months), while one pump will run during the non-irrigation season (5 months).

3.6 Permits and Approvals Required

This section provides a preliminary identification of the permits and agreements required for the three diluent waters and a time estimate for completing them.

Industrial Effluent Conveyance System

- City of Salinas (to take wastewater from the Salinas Industrial Water Treatment Plant): 5 – 6 months
- Monterey County Health Department Division of Environmental Health (for WDR modification for SIWTP and AWTP): 4 – 6 months
- Monterey County Health Department Division of Environmental Health (discharge permit for wastewater treatment for SIWTP): 4 – 6 months
- Monterey County Resource Management Agency Planning Services Department for Use Permit modification for Industrial Wastewater Facility: 6 – 9 months (up to 18 months if EIR required)
- RWQCB (for WDR modification): 4 – 6 months
- Right-of-way (private farms, etc.): 6 – 12 months
- Monterey County Public Works Department for encroachment permits: 1 month
- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months
- Building Permit from Monterey Resource Management Agency Planning Services Department (if required): 12 – 18 months
- PG&E power: 12 – 15 months

Reclamation Ditch Conveyance System

- Monterey County Water Resources Agency (for water rights for water from the Reclamation Ditch): 9 – 12 months
- Monterey County Health Department Division of Environmental Health (for WDR modification for AWTP): 4 – 6 months
- Monterey County Health Department Division of Environmental Health (discharge permit for wastewater treatment): 4 – 6 months

- Monterey County Resource Management Agency Planning Services Department for Use Permit modification for Industrial Wastewater Facility: 6 – 9 months (up to 18 months if EIR required)
- RWQCB (for WDR modification): 4 – 6 months
- Right-of-way (private farms, etc.): 6 – 12 months
- Monterey County Public Works Department for encroachment permits: 1 month
- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months
- Building Permit from Monterey Resource Management Agency Planning Services Department (if required): 12 – 18 months
- PG&E power: 12 – 15 months

Blanco Drain Conveyance System

- Monterey County Water Resources Agency (for water rights for water from Blanco Drain): 9 – 12 months
- Monterey County Health Department Division of Environmental Health (for WDR modification for AWTP): 4 – 6 months
- Monterey County Health Department Division of Environmental Health (discharge permit for wastewater treatment): 4 – 6 months
- Monterey County Resource Management Agency Planning Services Department for Use Permit modification for Industrial Wastewater Facility: 6 – 9 months (up to 18 months if EIR required)
- RWQCB (for WDR modification): 4 – 6 months
- Right-of-way (private farms, etc.): 6 – 12 months
- Monterey County Public Works Department for encroachment permits: 1 month
- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months
- Building Permit from Monterey Resource Management Agency Planning Services Department (if required): 12

3.7 Construction Methods

This section discusses the anticipated construction methods for the facilities associated with the diluent waters. Tables 3-11A, 3-11B, and 3-11C provide traffic data for peak day construction-related trips by project facility (materials delivery, haul trips, and worker commute). Table 3-12 provides a list of the types and number of typical construction equipment that may be used for the construction of the diluent facilities.

Industrial Effluent Pipeline. The industrial effluent water pipeline would follow the existing CSIP pipeline corridor from the AWTP to Nashua Road. The pipeline would be installed using conventional open-trench construction techniques. The SRDF project will incorporate the Salinas River crossing of the pipelines; therefore, no new construction activities will occur. The crossings of the Blanco Drain would be accomplished either by sheeting and dewatering the drain, or by jack-and-bore, depending on drain flows and the irrigation season.

Table 3-11 A
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Materials Delivery							
Pipelines	Description	Max # of Truck Loads per max day	Average Miles per Trip	Hrs of Daily Delivery	Duration (hrs) of Daily Delivery	Max Trips Per Day	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Industrial Effluent Pipeline	Import Soil	12	10	6am-3pm	9	24	3	1
	Pipe Delivery	5	80	6am-3pm	9	10	1	1
Total Pipelines		*17*				*34*	*4*	*2*
Pump Stations								
Industrial Effluent Pump Station	Concrete Delivery	10	20	6am-12pm	6	20	3	-
	Miscellaneous Materials	5	100	6am-3pm	9	10	1	1
Total Pump Stations		*15*				*30*	*4*	*1*
GRAND TOTAL		**32**				**64**	**8**	**3**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table 3-11 B
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Haul Trips							
	Description	Max # Truck Loads per Day	Average Miles per Trip	Hrs of Daily Delivery	Duration (hrs) of Daily Delivery	Max Trips Per Day	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Pipelines								
Industrial Effluent Pipeline	Export Native Soil	15	15	6am-3pm	9	30	3	1
					9	-	-	1
Total Pipelines		*15*				*30*	*3*	*2*
Pump Stations								
Industrial Effluent Pump Station	Earthwork Balanced	-				-		
						-		
Total Pump Stations		-				-	-	-
GRAND TOTAL		**15**				**30**	**3**	**2**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table 3-11 C
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Worker Commute						
	# Workers	Average Miles per Trip	Daily Work Hours	Trips Per Worker Per Day related to Construction	# of Daily Trips	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Pipelines							
Industrial Effluent Pipeline	72						
Total Pipelines	*72*	12	*7am-4pm*	*2.2*	*157*	*72*	*72*
Pump Stations							
Industrial Effluent Pump Station	38						
Total Pump Stations	*38*	12	*7am-4pm*	*2.2*	*83*	*38*	*38*
GRAND TOTAL					**240**	**109**	**109**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table 3-12
Equipment Inventory During Peak Construction, by Project Area

Facility	Activity / Equipment																
	ROW/ Site Work						Building/ Installation						Asphalt Paving/ Finishing Work				
	Tractors	Graders	On-Site Trucks	Off-Site Trucks	Excavator[1]	Traffic Equipment[1]	Trenchers[1]	Crane[1]	On-Site Trucks	Off-Site Trucks	Bore/Drill Rig	Other	Trucks	Pavers[1]	Paving Equipment[1]	Rollers	Other
Pipelines																	
Industrial Effluent Pipeline	4	-	-	-	3	4	2	2	4	6	1	2	6	1	2	2	2
Total Pipelines	*4*	*-*	*-*	*-*	*3*	*4*	*2*	*2*	*4*	*6*	*1*	*2*	*6*	*1*	*2*	*2*	*2*
Pump Stations																	
Industrial Effluent Pump Station	2	-	2	4	1	-	-	2	4	2	1	1	2	1	1	1	-
Total Pump Stations	*2*	*-*	*2*	*4*	*1*	*-*	*-*	*2*	*4*	*2*	*1*	*1*	*2*	*1*	*1*	*1*	*-*
Grand Total	***6***	***-***	***2***	***4***	***4***	***4***	***2***	***4***	***8***	***8***	***2***	***3***	***8***	***2***	***3***	***3***	***2***

1. Pipelines installed in paved roadways, dirt and gravel in roadway shoulders, and in agricultural services roads. Roadway paving/ resurfacing required for Industrial Effluent Pipeline
2. Salinas River Crossings will be constructed by SRDF Contractor during SRDF construction. Therefore no new or additional equipment will be utilized
3. The Grand Total is applicable if assuming each facility is constructed independent of one another; and, the peak construction day for each facility occurs on the same day during the construction period

The industrial effluent pipeline would be then be open-cut installed within the right of way along the shoulder of Nashua Road and Cooper Road, would cross Blanco Road, and would be installed along the shoulder of Blanco Road until turning into dirt and gravel agricultural service roads. This approach minimizes disruption to traffic as well as property owners, although traffic control would be required during the construction period. Temporary construction rights-of-way for pipeline installation would be up to 60 to 100 feet in width.

As stated, most of the construction would be open-cut trenching. Pipe sections would be placed in a trench of varying depth, and covered using conventional equipment such as backhoes, side-boom cranes, wheeled loaders, sheep's-foot excavators, and compactors. The pipelines would be constructed of PVC pressure pipe and would be typically delivered and installed in 40-foot-long sections. The pace of work for pipe installation is estimated to be about 250 linear feet per day.

Construction activities may involve trenching, spoil handling, pipeline installation, backfilling and restoration, and vehicle ingress and egress. All roadways and surfaces disturbed during pipeline installation would be restored. If possible, trench spoils would be temporarily stockpiled within the construction easement, then backfilled into the trench after pipeline installation. However, for purposes of this report, it was assumed that pipe zone material would be imported from offsite, and spoils would be exported for disposal. Earth cover over the pipe was assumed to be four feet. Variations in this depth would be required to accommodate local topography, hydraulic grade, and utility congestion, among other factors. Assuming a "neat" trench line, the trench width would be up to six feet. Therefore, each pipeline would result in up to 10 to 15 truckloads of import soil per day and 10 to 15 truckloads each of export soil. The total volume of non-native soil that would be imported to the trenches would be approximately 15,000-20,000 cubic yards and 5,000-10,000 cubic yards for the industrial effluent and injection water pipelines, respectively. In addition, the total volume of native soil that would be exported from the trenches would be approximately 20,000-25,000 cubic yards and 5,000-10,000 cubic yards for the industrial effluent and injection water pipelines, respectively.

During the construction period, a maximum of approximately 110 workers of various skilled trades may work along the pipeline routes, which would result in temporary increases in traffic due to worker commutes and general construction activity.

Industrial Effluent Pump Station. The IEPS is proposed to be located on previously disturbed land adjacent to the SIWTP percolation ponds, near Pond #3 north of the Salinas River. Heavy equipment must be brought in and stored onsite. A temporary increase in traffic volumes on public roadways and agricultural service roads will occur as a result of project-generated traffic. During the construction period, up to approximately 35 to 40 workers of various skilled trades may be working during the peak of construction.

Site preparation would entail clearing of the area. Excavation and leveling of the site would be required for uneven gradient. Ground clearing and excavation of the site would be performed using available heavy equipment normally found at most construction sites. It is assumed that earthwork at the site would be balanced and, therefore, import or export of soil materials would not be required.

Construction operations would involve excavation and installation for pumping and piping systems, concrete foundation and surface pouring, pump-house building construction, and installing pipeline and pumping connections and other support equipment (control panels), finishing of the site, and perimeter fencing.
The construction of the IEPS will occur over an approximate 12-month period, with the peak of construction occurring during the non-growing season in order to minimize project-induced disruptions to agricultural activities and to take advantage of the lower groundwater levels near the vicinity of the Salinas River and the SIWTP ponds.

Reclamation Ditch Water Pipeline. The Reclamation Ditch water pipeline will be about the same length as the industrial effluent pipeline and will be constructed in an equal manner.

Reclamation Ditch Pump Station. The Reclamation Ditch Pump Station will be constructed the same as the IEPS.

Blanco Drain Water Pipeline. The Blanco Drain water pipeline will be within the disturbed area of the Blanco Drain Pump Station expansion.

Blanco Drain Pump Station. The Blanco Drain Pump Station upgrades have not been determined at this time.

Chapter 4 – ADVANCED WATER TREATMENT PLANT

4.1 Introduction

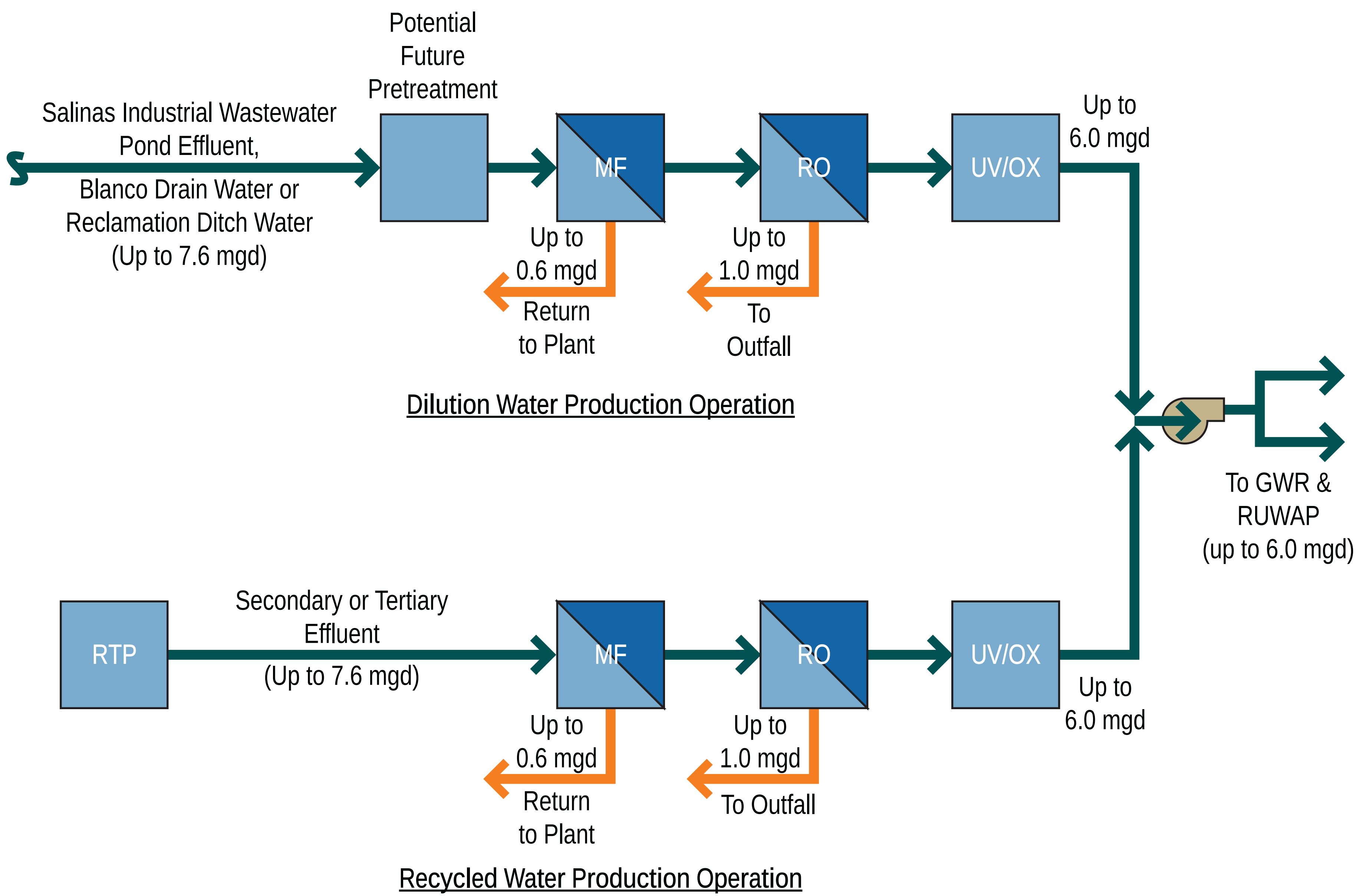
For the Seaside groundwater replenishment project, MRWPCA proposes to eventually produce approximately 6,720 AFY of AWTP product water, of which up to 6,037 AFY could be used for replenishment of the Seaside Groundwater Basin. This would equate to approximately 6 mgd of product water over the full year. The source water from MRWPCA will be either secondary or tertiary effluent from the SVRP. This water will undergo advanced treatment to meet regulatory treatment and water quality requirements for indirect potable reuse. The AWTP facilities will consist of pre-treatment (e.g., pre-screening), membrane filtration, reverse osmosis, and advanced oxidation in order to produce high quality effluent meeting State, local, and federal drinking water quality standards.

MRWPCA is proposing three active treatment trains, each sized to produce 2 mgd, for a total treatment facility size of 6 mgd. The multiple trains could be used to treat different water sources. This allows for optimization of anti-scalants, other chemicals, and other treatment parameters for a particular water. This also allows for sampling of the product water from the SIWTP (or possible future Blanco Drain or Reclamation Ditch) water before it mixes with the other train of AWTP product water. By providing separate treatment and sampling, either SIWTP, Blanco Drain, or Reclamation Ditch water would be considered diluent water by CDPH. Excess SIWTP water may be used for brine dilution in the ocean outfall. Figure 14 is a schematic diagram of the seasonal operating scenarios for the AWTP.

4.2 Regulatory Requirements and Water Quality Objectives

The AWTP will need to produce water suitable for subsurface application via vadose zone wells or injection into the Seaside Groundwater Basin. State requirements for treatment and quality of recycled water for recharge of a potable supply aquifer are currently being developed by the CDPH and, upon adoption, will be included in Title 22 of the California Code of Regulations. The Regional Water Quality Control Board (RWQCB) establishes project-specific quality requirements based, in part, upon the CDPH regulations and recommendations. Tables 4-1 through 4-4 summarize the existing feed water quality from the anticipated four sources of water to the AWTP, the existing primary and secondary Maximum Contaminant Levels (MCLs) established by CDPH, and projected finished water quality after advanced treatment. The actual limits will be set as part of the advanced treatment facility operating permit, to be established at a later date.

In establishing water quality goals, CDPH will consider the current water quality of the Seaside Basin to confirm that the quality of the water in the basin would not be degraded by the recycled water, and that the recycled water meets all requirements of the California drinking water primary and secondary MCLs. In addition, certain levels of treatment may be specifically required by CDPH before recycled water can be injected into a drinking water supply aquifer. For direct subsurface injection of water, this includes oxidation, membrane filtration, reverse osmosis and advanced oxidation (including disinfection). In addition to these requirements, brine disposal will be regulated by the RWQCB, with the water quality limits subject to the provisions of the MRWPCA's existing regional outfall permit (NPDES).



CDM

Figure 14
AWTP - Conceptual Operating Conditions

Table 4-1: RTP Feed Water Quality and Estimated Finished Water Quality

Parameter (unit of measurement)	RTP Effluent (1)	Regulated limit	Estimated Conc. From RO permeate (4)	Estimated Conc. From RO concentrate (4)	Estimated Conc. of Finished Water (5)
Ca (mg/L)	58	NA	1.5	378	36
Mg (mg/L)	21	NA	0.5	137	0.5
Na (mg/L)	174	NA	20	1044	20
K, mg/L	21	NA	3.0	123	3.0
HCO_3, mg/L	344	NA	51	1,610	85
SO_4, mg/L	101	250 (2)	2.4	972	2.4
Cl, mg/L	226	250 (2)	10.4	1,469	10.4
NO_3, mg/L	20	45 (2)	9.8	78	9.8
Total Nitrogen (mg/L)	31	5(3)			
TDS, mg/L	815	500 (2)	110	6,207	178
pH	7.3	6.5-8.5	6.1	7.5	7.6
Turbidity, NTU			0.02		<0.2
TOC, mg/L		16(3)	0		
Hardness, mg/L as $CaCO_3$	232		6.5	1,548	92
Alkalinity, mg/L	282		53	1,660	140
Total phosphorous, mg/L	2.2		0		
NDMA (ng/L)	100			NA	
Temperature, °F			70	70	70

Notes:

(1) Concentrations based on average plant effluent data 2005-2007.
(2) MCL = Drinking Water Maximum Contaminant Levels specified by the CDPH.
(3) Specific criteria identified in Draft CDPH Groundwater Recharge Reuse regulations.
(4) Concentrations were estimated using IMS Design software, assuming use of Hydranautics ESPA1 membranes, 85% recovery
(5) Assumes 68 mg/L lime addition for stable product water with LSI>1 and CCPP = 5.0 mg/L.

Table 4-2: SIWTP Feed Water Quality and Estimated Finished Water Quality

Parameter (unit of measurement)	Salinas Industrial Pond Effluent[(1)]	Regulated limit	Estimated Conc. From RO permeate [(4)]	Estimated Conc. From RO concentrate [(4)]	Estimated Conc. of Finished Water
Ca (mg/L)	100	NA	2.3	577	12
Mg (mg/L)	30	NA	0.7	173	0.7
Na (mg/L)	276	NA	30	1,477	30
K, mg/L		NA	2.8	110	2.8
HCO_3, mg/L		NA	54	1,171	68
SO_4, mg/L	259	250 [(2)]	5.2	1,498	5.2
Cl, mg/L	362	250 [(2)]	22.4	2,222	22.4
NO_3, mg/L	ND	45 [(2)]	ND		
Total Nitrogen (mg/L)	11	5[(3)]			
TDS, mg/L	1358	500 [(2)]	126	7,495	146
pH	7.5	6.5-8.5	6.9	8.1	8.3
Turbidity, NTU			0.02		<0.2
TOC, mg/L		16[(3)]	0		
Hardness, mg/L as $CaCO_3$			7		30
Alkalinity, mg/L	200		88		111
Total phosphorous, mg/L					
NDMA (ng/L)					
Temperature, °F			70	70	70

Notes:

(1) Based on grab sample from October 2007.
(2) MCL = Drinking Water Maximum Contaminant Levels specified by the CDPH.
(3) Specific criteria identified in Draft CDPH Groundwater Recharge Reuse regulations.
(4) Concentrations were estimated using IMS Design software, assuming use of Hydranautics ESPA1 membranes, 83% recovery.
(5) Assumes 20 mg/L lime addition for stable product water with LSI>1 and CCPP = 4.0 mg/L.

Table 4-3: Blanco Drain Feed Water Quality and Estimated Finished Water Quality

Parameter (unit of measurement)	Blanco Drain[1]	Regulated limit	Estimated Conc. From RO permeate [4]	Estimated Conc. From RO concentrate [4]	Estimated Conc. of Finished Water[5]
Ca (mg/L)	119	NA			
Mg (mg/L)	135	NA			
Na (mg/L)	263	NA			
K, mg/L	2.2	NA			
HCO_3, mg/L	430	NA			
SO_4, mg/L	522	250 [2]			
Cl, mg/L	185	250 [2]			
NO_3, mg/L	209	45 [2]			
Total Nitrogen (mg/L)	49	5[3]			
TDS, mg/L	1,875	500 [2]			
pH	8.2	6.5-8.5			
Turbidity, NTU					
TOC, mg/L		16[3]			
Hardness, mg/L as $CaCO_3$	852				
Alkalinity, mg/L	353				
Total phosphorous, mg/L	<1				
NDMA (ng/L)					
Temperature, °F					

Notes:
(1) Based on 23 grab samples from November 2006 to July 2007.
(2) MCL = Drinking Water Maximum Contaminant Levels specified by the CDPH.
(3) Specific criteria identified in Draft CDPH Groundwater Recharge Reuse regulations.
(4) Concentrations will be estimated using IMS Design software, assuming use of Hydranautics ESPA1 membranes, 87% recovery.
(5) Lime addition and/or RO bypass water may be used for stable product water.

Table 4-4: Reclamation Ditch Feed Water Quality and Estimated Finished Water Quality

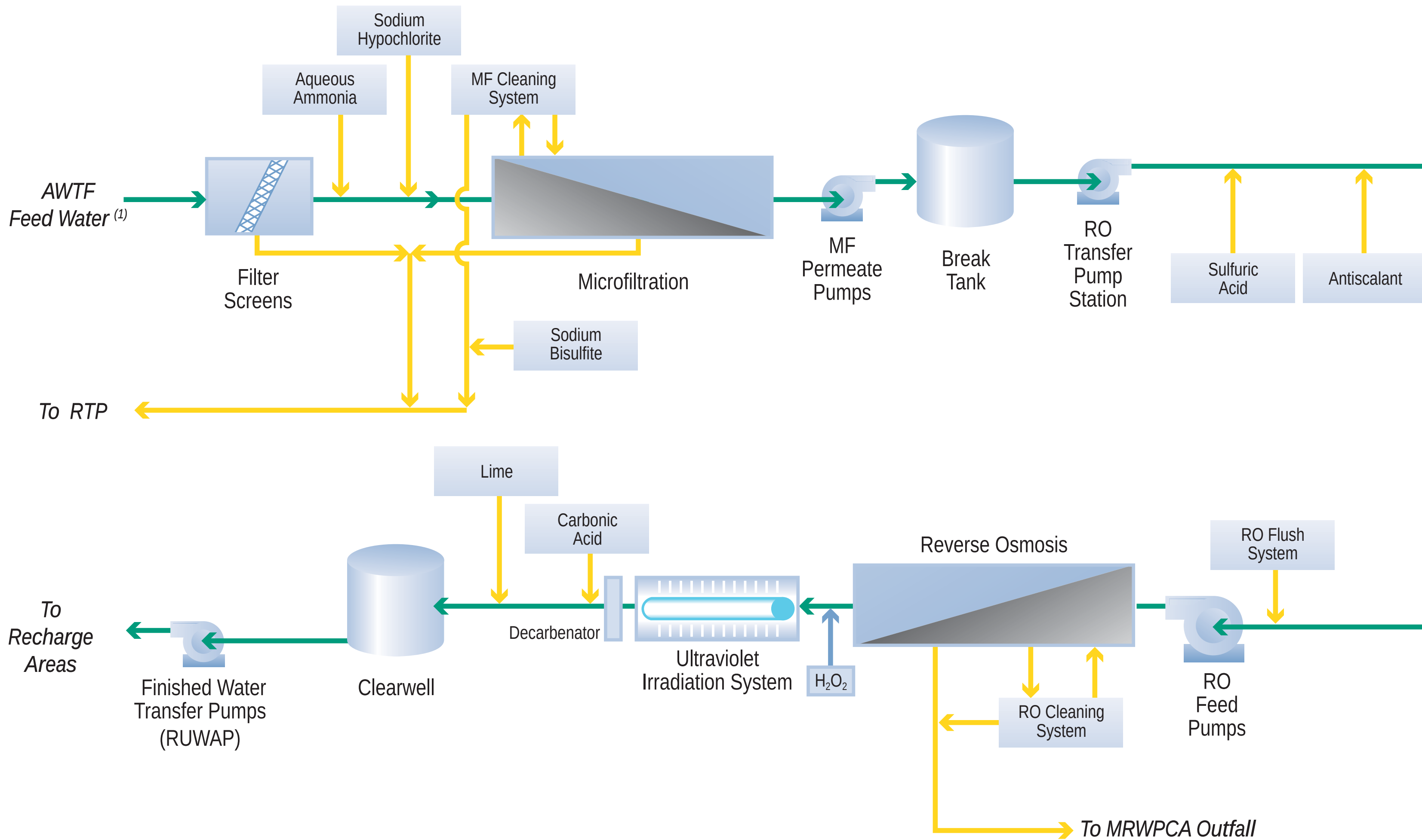
Parameter (unit of measurement)	Reclamation Ditch(1)	Regulated limit	Estimated Conc. From RO permeate (4)	Estimated Conc. From RO concentrate (4)	Estimated Conc. of Finished Water(5)
Ca (mg/L)	106	NA			
Mg (mg/L)	87	NA			
Na (mg/L)	246	NA			
K, mg/L	4.8	NA			
HCO_3, mg/L	371	NA			
SO_4, mg/L	253	250 (2)			
Cl, mg/L	185	250 (2)			
NO_3, mg/L	209	45 (2)			
Total Nitrogen (mg/L)	49	5(3)			
TDS, mg/L	1,555	500 (2)			
pH	8.7	6.5-8.5			
Turbidity, NTU					
TOC, mg/L		16(3)			
Hardness, mg/L as $CaCO_3$	623				
Alkalinity, mg/L	304				
Total phosphorous, mg/L	<1				
NDMA (ng/L)					
Temperature, °F					

Notes:

(1) Based on 4 grab samples of Tembladero Slough from June and July 2007.
(2) MCL = Drinking Water Maximum Contaminant Levels specified by the CDPH.
(3) Specific criteria identified in Draft CDPH Groundwater Recharge Reuse regulations.
(4) Concentrations will be estimated using IMS Design software, assuming use of Hydranautics ESPA1 membranes, 87% recovery.
(5) Lime addition and/or RO bypass water may be used for stable product water.

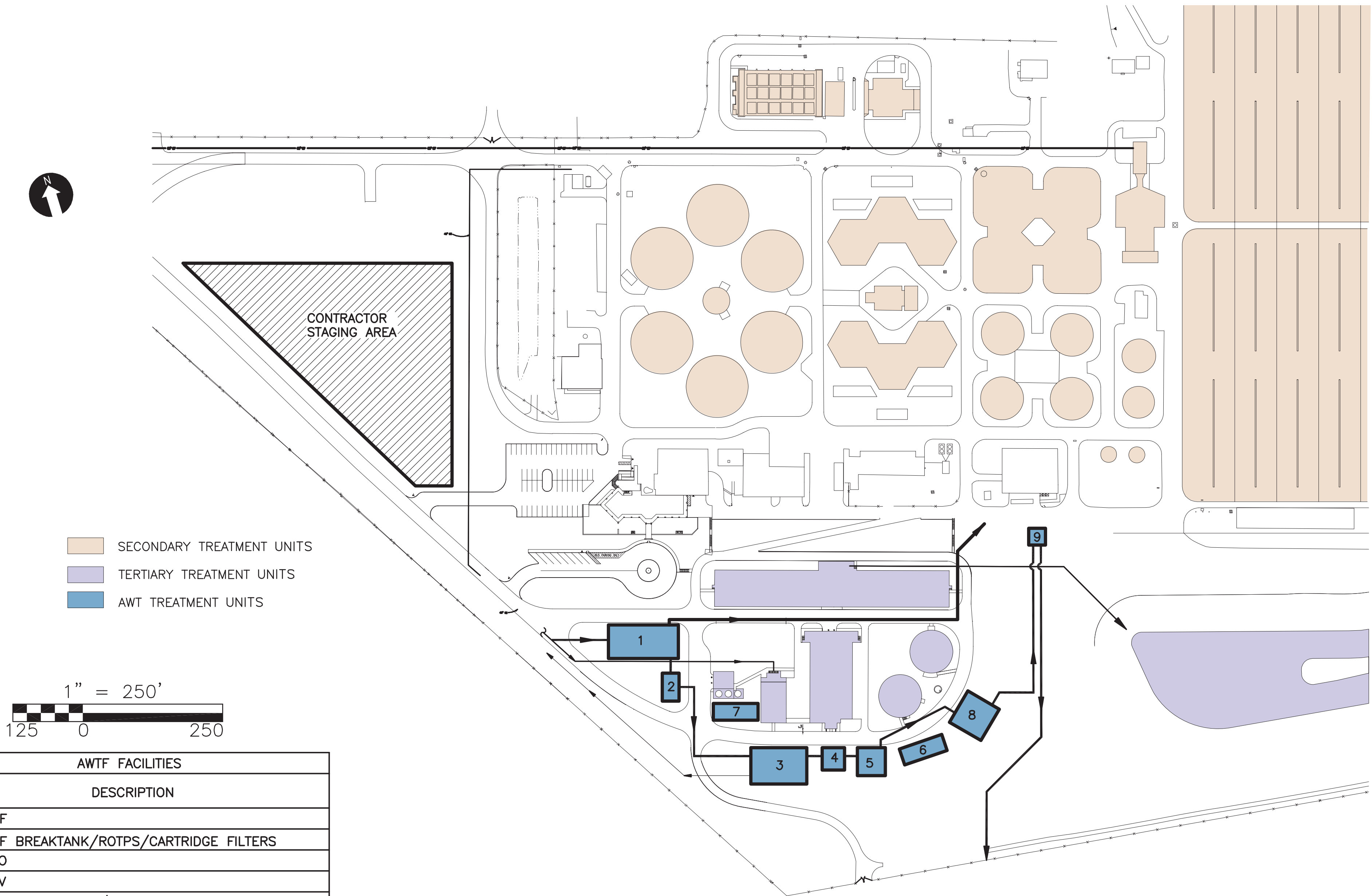
4.3 Project Facilities and Operations

The AWTP process, facilities, and operations are described in this section. A preliminary layout for the proposed AWTP facilities has been developed for the MWRPCA RTP site and is shown in Figure 15. Figure 16 illustrates the overall AWTP site plan, while Figure 17 illustrates the conceptual space requirements for each of the processes and the major interconnecting piping.



Note:
1. For AWTP Feed Water Sources, See Figure 17.

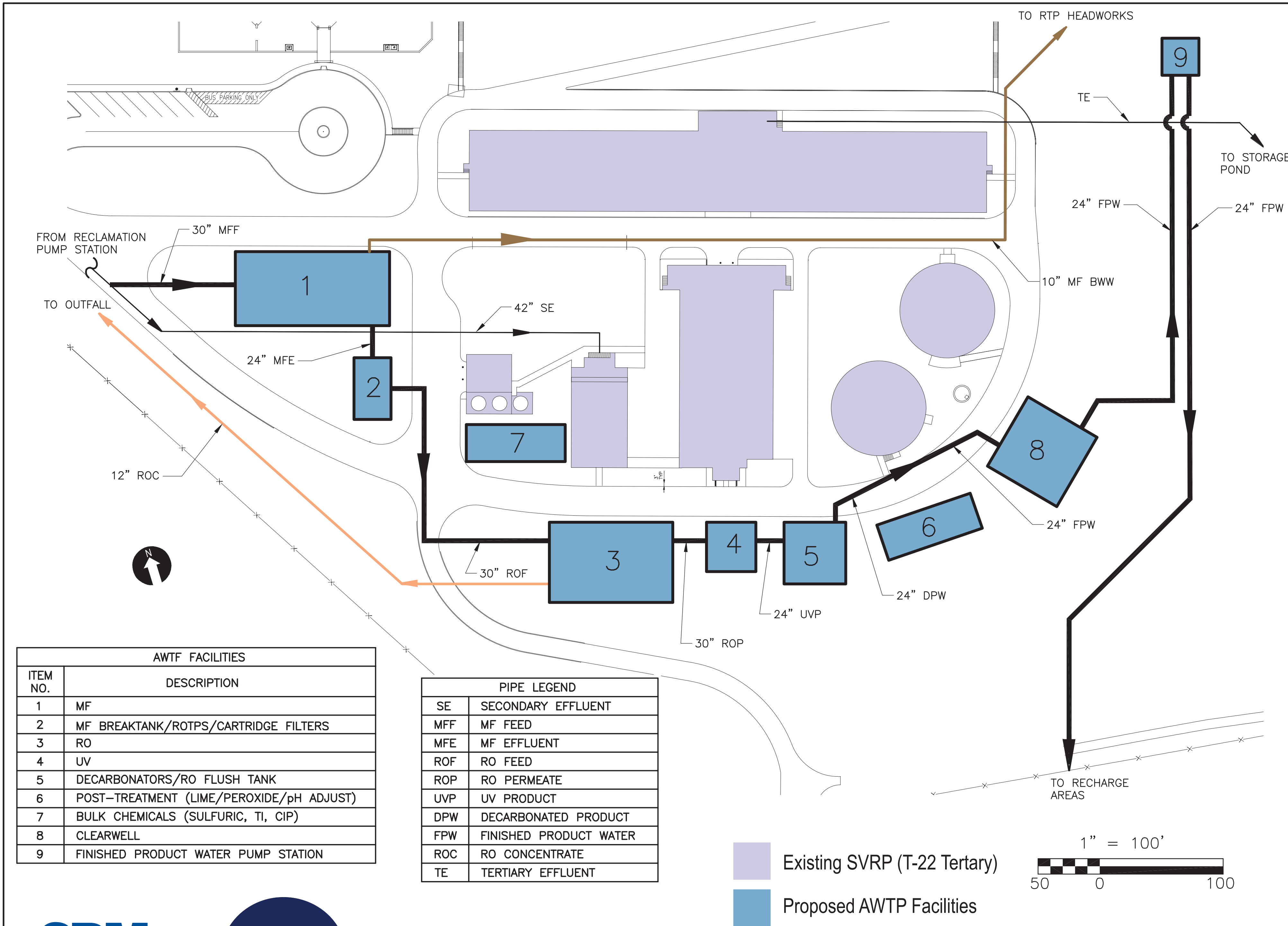
Figure 15
AWTP - Conceptual Process Flow Diagram



AWTF FACILITIES	
ITEM NO.	DESCRIPTION
1	MF
2	MF BREAKTANK/ROTPS/CARTRIDGE FILTERS
3	RO
4	UV
5	DECARBONATORS/RO FLUSH TANK
6	POST–TREATMENT (LIME/PEROXIDE/pH ADJUST)
7	BULK CHEMICALS (SULFURIC, TI, CIP)
8	CLEARWELL
9	FINISHED PRODUCT WATER PUMP STATION

CDM MRWPCA

Figure 16
AWTP - Overall Site Plan



AWTF FACILITIES	
ITEM NO.	DESCRIPTION
1	MF
2	MF BREAKTANK/ROTPS/CARTRIDGE FILTERS
3	RO
4	UV
5	DECARBONATORS/RO FLUSH TANK
6	POST–TREATMENT (LIME/PEROXIDE/pH ADJUST)
7	BULK CHEMICALS (SULFURIC, TI, CIP)
8	CLEARWELL
9	FINISHED PRODUCT WATER PUMP STATION

PIPE LEGEND	
SE	SECONDARY EFFLUENT
MFF	MF FEED
MFE	MF EFFLUENT
ROF	RO FEED
ROP	RO PERMEATE
UVP	UV PRODUCT
DPW	DECARBONATED PRODUCT
FPW	FINISHED PRODUCT WATER
ROC	RO CONCENTRATE
TE	TERTIARY EFFLUENT

CDM

MRWPCA

Figure 17
AWTP - Site and Piping Plan

4.3.1 Pretreatment

Pre-screening of the SVRP secondary or tertiary effluent or other water sources will remove any large particles in the waste stream, protecting the MF membranes from damage and from premature fouling. For this project, rotating screens will be utilized for pre-filtration. The screens must be capable of removing particles greater than two millimeters (mm) in size. Following the pre-screening, aqueous ammonia (not necessary for wastewater sources) and sodium hypochlorite would be added to produce chloramines, a disinfectant used to prevent bio-fouling or growth of microorganisms on the microfiltration (MF) and reverse osmosis (RO) membranes. As an alternative, chloramines could be added downstream of the MF, reducing the risk of NDMA formation, but requiring the addition of hypochlorite into the MF backwash water to prevent biogrowth on the membrane surface. Some pretreatment may be required for nutrient removal in some of the non-wastewater sources. The need for pretreatment will be identified during pilot testing.

4.3.2 Membrane Filtration

The membrane filtration stage can be either microfiltration or ultrafiltration (both referred to as MF in this report). This process is used to reduce, and essentially eliminate the turbidity, biological constituents, and silt density for increased RO system reliability and reduced RO membrane fouling. The filtration units are periodically backwashed to clean the membranes, with the backwash waste discharged back to bioflocculation basins, the trickling filter pump station, the headworks of the RTP, or to other treatment. While the MF system can effectively remove parasites such as *Giardia* and *Cryptosporidium* and reduce suspended solids and some bacteria to low levels, it is not intended to provide effective reductions in dissolved organics, pesticides, oil and grease, total dissolved solids, silica, or viruses that will otherwise be removed by the RO system. A clean-in-place (CIP) system will provide regenerative cleaning of the MF membranes using chemical addition and air scour approximately every 30 days. The MF system would also be capable of operating in stand-by mode using a small residual of hypochlorite circulated periodically to prevent fouling and biological growth during extended shut-down periods. Such a system would be necessary if the AWTP were to be operated only during winter months. Design criteria for the MF equipment are summarized in Table 4-5.

For the purpose of the concept design for this project description, a Siemens Memcor CMF-S microfiltration system is assumed. These are immersed membranes utilizing “outside-in” flow through a hollow-fiber membrane that has nominal pore sizes of approximately 0.04 microns. These membranes are made from a polyvinyl difluoride (PVDF) material, which has a high resistance to both oxidants and biological foulants. The membranes operate under vacuum, drawing treated water through the membrane pores into the inside of the hollow fibers. Periodically, air is introduced at the bottom of the membrane modules to create turbulence along the membrane surface to scour and clean the outside of the membrane fibers. Pressure type MF membrane systems are also available; they will be evaluated during preliminary design.

Table 4-5: Typical AWTP Unit Process Design Criteria	
Fine Screening	
Screen size (mm)	2
Membrane Filtration	
Treatment capacity (mgd)	7.0
Membrane system recovery (percent)	92
Required feed flow (mgd)	7.6
Backwash waste flow (mgd)	0.6
Number of active process units	4
No. Modules per unit	292
Instantaneous flux rate (gfd)	20
Clean in place frequency (days)	30
Membrane area per module (sq.ft.)	300
Break Tank	
Tank capacity (gal)	150,000
Hydraulic retention time (min)	30
Reverse Osmosis	
Treatment capacity (mgd)	6.0
Membrane system recovery (percent)	85
Required feed flow (mgd)	7.0
Concentrate/brine flow (mgd)	1.0
Number of active skids	3
Treatment capacity, per skid (mgd)	2
Instantaneous flux rate (gfd)	11
Number of stages per skid	2
Diameter of membrane element, each (in)	8
Membrane element area, each (sq.ft.)	400
Number of membrane elements per pressure vessel	7
Number of pressure vessels per skid (1^{st} stage x 2^{nd} stage)	44x20
Number of chemical cleanings per year	1
UV Disinfection – Advanced Oxidation Process	
Treatment capacity (mgd)	6.0
UV transmittance – 254 wavelength (%/cm)	95
Number of active reactors	3
Number of standby reactors	1
Reactor type	Low pressure/ high output
Lamp replacement frequency (hrs)	12,000

MF backwash water will be returned as described above. Approximately 600,000 gpd of backwash wash water can be expected. Depending on the source water, CIP cleaning for the MF will take place every 21-60 days. The amount of CIP wastewater would be small, with approximately 2,500 gallons produced each cleaning. Since its flow rate is relatively small compared to total RTP influent flow, neutralization may not be required and it may be directed through the plant drain system to the headworks. Otherwise, neutralization will be effected within the AWTP.

4.3.3 Reverse Osmosis

MF filtrate would be pumped into an MF filtrate tank which provides a 30-minute buffer between MF and RO. The tank would also be used to store backwash wash water for the membrane filtration system, if the MF system requires a volume of water larger than what is stored in the permeate piping. Water would be pumped from the filtrate tank to the RO treatment trains. The RO system removes dissolved solids and other priority constituents from the MF filtrate, making it acceptable for groundwater recharge. Although RO membranes are often thought of as filters, they operate by means of osmotic principles rather than strict filtration and, therefore, exhibit considerably different characteristics than lower pressure membrane filters. The RO trains use two-stage, spiral wound, thin film composite membranes to remove salts, minerals, metal ions, and organic compounds, as well as providing a barrier to microorganisms.

The RO feed pump operating pressures range from about 150 psig to a maximum of 300 psig. The pressures will be lower when the membranes are new and gradually increase as the membranes become dirty. The RO recovery rate is typically around 85 percent for similar facilities. When RO membrane transmembrane pressure increases towards the upper limit of the design range, the membranes will be cleaned. A permanent CIP system is included. RO membrane cleaning is estimated to be required about once per year. A membrane flush system flushes the membranes with RO permeate each time that the RO system is turned off for more than about 20 minutes. Similar to the MF system, the RO system would be capable of operating in stand-by mode using an initial permeate flush and a bisulfite or biocide solution soak to prevent fouling and biological growth during extended shut-down periods.

The specific membrane elements used in preparing the RO projections were modeled using ESPA1 from Hydranautics. Scale inhibitor and sulfuric acid are injected prior to RO to reduce scaling of the RO membranes. Major RO system design criteria are shown in Table 4-4.

4.3.4 Disinfection and Stabilization

Disinfection is performed through an advanced oxidation process (AOP) consisting of two steps: hydrogen peroxide and ultraviolet (UV) light treatment. Hydrogen peroxide (H_2O_2) is added to the RO permeate upstream of the UV light. The UV light breaks the H_2O_2 bond by photolysis, creating a hydroxyl radical, a powerful oxidant. In addition, UV irradiation also breaks down light sensitive contaminants. The assumed system uses either two or three active in-line high intensity Trojan UVSwift reactors, assuming an estimated 95 percent UV transmittance. One redundant reactor is included as a standby. Hydrated lime (calcium hydroxide, $Ca(OH)_2$) and carbon dioxide (CO_2) are added to the product water from the UV system to adjust pH, alkalinity, and hardness. The approximate target values are a pH of 7.0, alkalinity of 60 mg/L as $CaCO_3$, and hardness of 100 mg/L $CaCO_3$ to match the water quality conditions in the Seaside Basin and to produce a water that will not corrode the transmission pipeline.

4.3.5 Brine Disposal

A 12-inch brine disposal pipeline will convey approximately 1 mgd of brine to the existing MRWPCA outfall for disposal.

4.3.6 Energy Source and Requirements

Electrical power supply will be from a separate service and transformer from PG&E. The estimated connected load of the AWTP is 1,750 kW and the demand load is estimated at 1,300 kW. Incoming power is at 21 kV and will be stepped down to 480-V for distribution to motors and other electrical demands. For the plant operating at near full capacity (95 percent of 6 mgd output), the annual power consumption is estimated at approximately 11 million kW-hr. Operating at a steady output capacity year round, the average monthly demand is the same as the estimated demand load of about 1,300 kW. The estimated monthly power consumption is estimated at about 920,000 kWhr. Using these values, the estimated power demand per AF of recycled water produced is 0.20 kW per AF. As shown in Table 4-6, this energy demand compares favorably other water sources.

Table 4-6: Power Requirements for Regional Project Alternatives

Alternative	kW/AF
AWTP and Seaside GWR (6,720 AFY)	0.20
Brackish Water Desalination Facility (12,430 AFY)	0.402
CWP North Marina Desalination Facility (10,430 AFY)	0.517

4.4 Permits and Approvals Required

This section provides a preliminary identification of the permits and agreements required for the AWTP, and a time estimate for completing them.

- NEPA process for Water Augmentation Pumping Plant pump station (if required): 18 – 24 months
- New WDR from RWQCB: 6 – 12 months
- CDPH recommendation to RWQCB for approval of the project: 1 – 3 months
- Independent Advisory Panel recommendation for approval of project to CDPH: 24 – 36 months
- NEPA process for reclamation pump station and pipeline from SVRP (if required): 18 – 24 months
- Monterey County Resource Management Agency Planning Services Department for Use Permit update (if required): 12 – 18 months
- PG&E power: 12 – 15 months
- Potential agreements with Waste Management District regarding power and food waste digestion: 9 – 15 months
- Monterey County Health Department Division of Environmental Health hazardous materials permit amendment: 3 – 6 months

- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months

4.5 Construction Methods

The AWTP is proposed to be located at the southern edge of the existing RTP adjacent to the SVRP tertiary filter facilities. Treatment processes at the AWTP will include membrane filtration, reverse osmosis and advanced oxidation using UV and hydrogen peroxide. These processes are typically constructed aboveground using concrete foundation slabs, building enclosures, canopies, sheds, and tanks. Details for the construction methods, construction duration, traffic, and excavation are described below. The AWTP site will be constructed primarily on a slope south of the existing SVRP and west of the storage pond. The required area is approximately 700 feet long by 100 feet wide. The MF system is located between the SVRP and the entrance parking area for the RTP. The other major processes are located south of the SVRP. To construct the process facilities, a flat pad will be required. Therefore, sloped excavation or retaining walls will be required to construct the AWTP foundations. For the AWTP, long term erosion control using berms, landscaping, and drainage facilities will be provided.

The AWTP processes can be constructed outside, under canopies, or in building enclosures. Processes that have sensitive instruments or that could be damaged by the weather (cold, rain) are typically constructed in buildings. Buildings can also be used to house the major pump stations to mitigate pump motor noise. These systems may include the microfiltration and reverse osmosis systems, the RO transfer and finished water pump stations.

Building layouts will allow for easy access to process equipment for operation and maintenance. Buildings will include concrete slab foundations and footings. The style of the building will conform to other existing structures at the RTP and SVRP. Buildings will typically be one-story structures with pipe trenches to maintain access around the process units. Process equipment canopies will be provided where buildings are not required. These canopies will be constructed of steel frames with a metal deck and sheet metal roofing.

Major pipelines include the influent to the plant (MF feed), the pipeline between the MF and RO systems (MF effluent and RO feed), the RO product pipeline carrying RO treated water, a pipe to carry UV product from the UV system to the RO flush tank, and the finished product water. Outside piping will be buried with a minimum of four feet of cover. Utility trenches will be constructed to provide protection, containment, and access to the process piping at and around the treatment processes. Trenches will be cast-in-place and sized as appropriate to accommodate the process piping.

The construction duration for the AWTP should be approximately 18-24 months. The construction schedule is significantly influenced by the operational requirements of the existing RTP and SVRP, labor availability, equipment delivery, and the weather. Because undeveloped space directly adjacent to the AWTP site is small and sloped, construction staging will be located elsewhere on the RTP. Construction of the AWTP will take place at the RTP using existing roads to access the site. Existing roads in the immediate vicinity of the AWTP site will be closed to facilitate construction, with on-site detours and temporary construction roads constructed to maintain plant operation. Daily construction traffic will include contractor, engineer and operator employee vehicles, equipment/material delivery, chemical delivery and waste disposal. Construction traffic is dependent on the schedule, the number of construction labor. No off-site grading or waste earth disposal is anticipated.

Chapter 5 – GROUNDWATER ANALYSES

5.1 Introduction

This chapter of the report presents the results of groundwater analyses prepared by Todd Engineers for the Seaside Groundwater Basin replenishment project. As previously discussed, the analysis focuses on recharge of about 2,400 AF during the non-irrigation season at each of two potential project locations. The total project may provide up to 6,720 AFY on a year round basis, including additional sources of diluent water. Additional hydrogeologic analyses for the total project are being coordinated through the Seaside Basin Watermaster.

5.2 Seaside Groundwater Basin

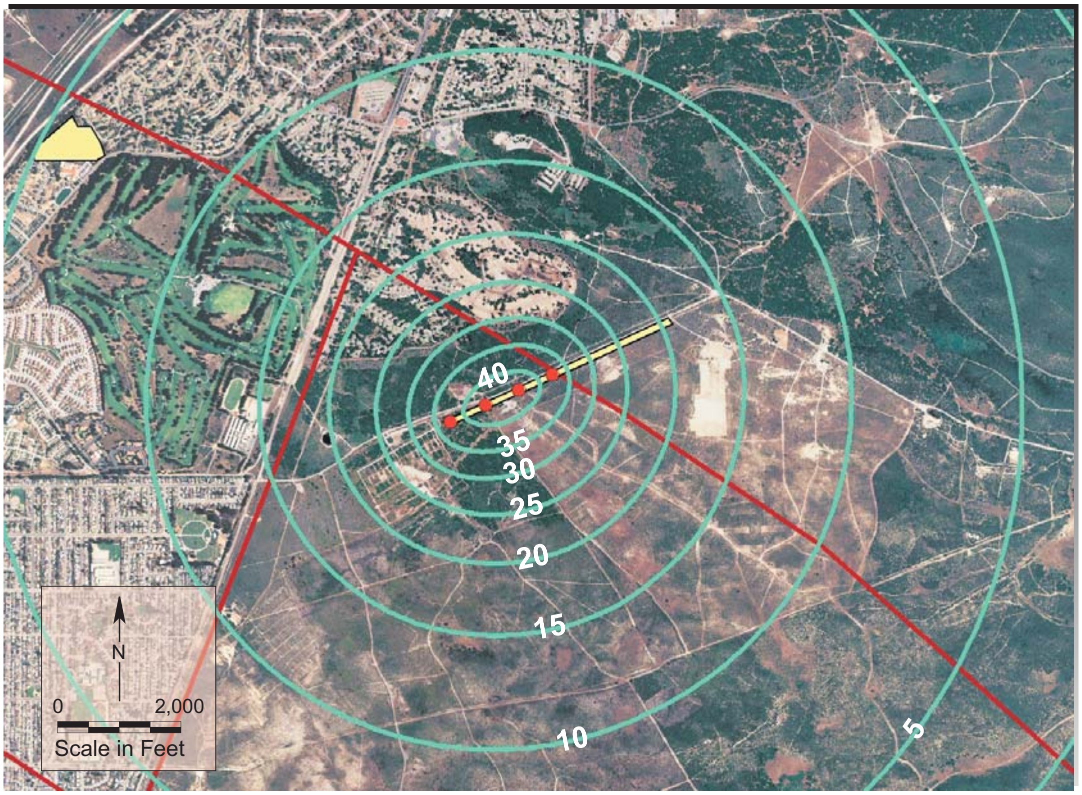
This section describes the methods used to evaluate potential impacts from the Seaside Basin GWR project on groundwater levels, quality, and existing wells, considering regulatory requirements. Potential impacts from operation of vadose zone wells and injection wells at the inland and coastal locations are analyzed.

5.2.1 Impacts to Groundwater Levels

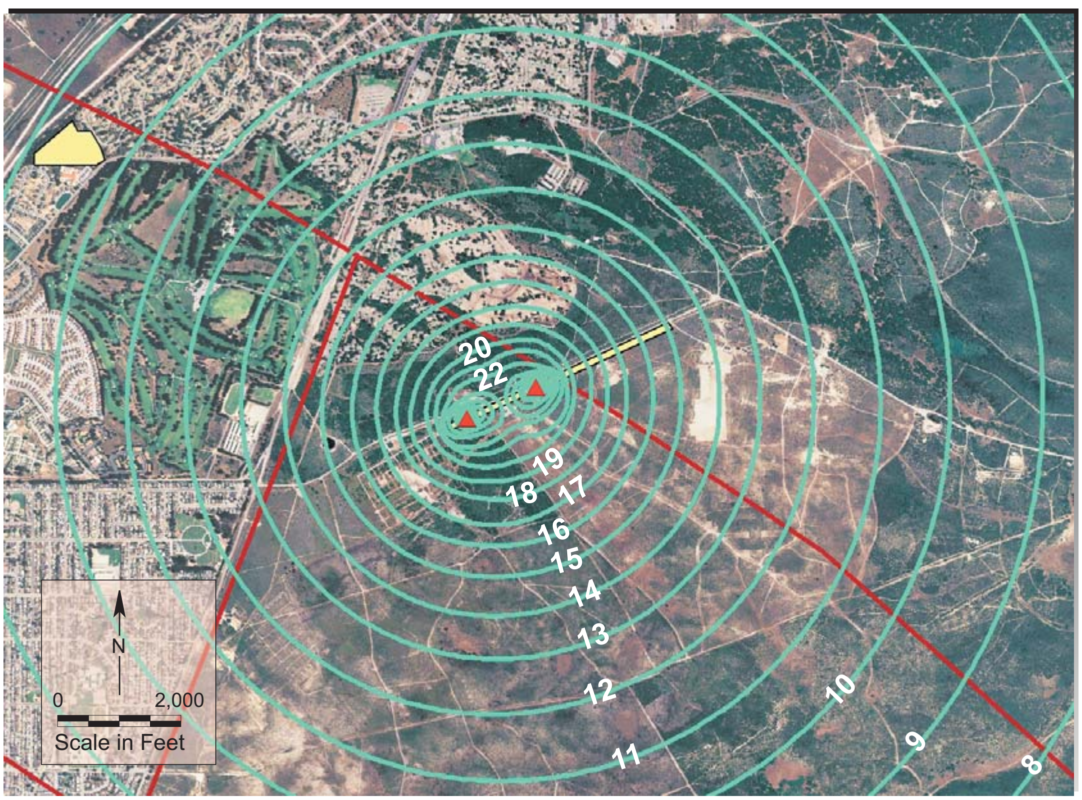
Estimated impacts to water levels are presented on Figures 18 and 19 and discussed below. Operation of the Seaside Basin GWR project will impact water levels in the basin during the recharge season, allowing for basin replenishment and increased yield. Water level rises are expected to occur over most of the Northern Coastal Subarea for either the inland or coastal locations. Rising water levels will increase offshore flow in the aquifers, reducing the potential for seawater intrusion.

Water level impacts for injection into the Santa Margarita Aquifer have been evaluated in an EIR associated with the MPWMD ASR project (Jones & Stokes, 2006). For that project, 2,426 AFY of Carmel River water was to be injected into three ASR wells along General Jim Moore Boulevard in the vicinity of (and including) SMTIW-1. Although the recharge amount of 2,426 AFY is similar to the initial amount of water available for the GWR project (2,400 AFY), impacts to water levels are expected to be less than for the ASR project because recharge is divided among two aquifer systems. Further, the project will be operated so as to mitigate any adverse impacts associated with the water level rise. For example, the allocation among the two aquifers can be modified over time.

Vadose Zone Wells. Effects on groundwater levels from operation of vadose zone wells were evaluated using an analytical equation developed by Hantush (1967) for surface recharge basins. For each vadose zone well, the radius of effective recharge was estimated based on the planned recharge rate and estimated vertical infiltration rate at each location. The estimated effective radius of recharge is about 160 feet for vadose zone wells at the inland location and about 50 feet for vadose zone wells at the coastal location. These two differ because of the larger amount of water being recharged at the inland location and the lower assumed vertical infiltration rate. The infiltration rate at the inland location is affected by the lower permeability of the Paso Robles Formation. At the coastal location, the water table is contained within the more permeable Aromas Sand.



Vadose Zone Wells - Aromas/Paso Robles Aquifer
Predicted rise in water levels (ft) after 5 months of recharge at 400 gpm/well



Injection Wells - Santa Margarita Aquifer
Predicted rise in water levels (ft) after 5 months of recharge at 1,000 gpm/well

June 2008	**Figure 18**
TODD ENGINEERS Alameda, California	**Potential Water Level Impacts Inland Recharge Location**

Vadose Zone Wells - Aromas/Paso Robles Aquifer
Predicted rise in water levels (ft) after 5 months of recharge at 150 gpm/well

Injection Wells - Santa Margarita Aquifer
Predicted rise in water levels (ft) after 5 months of recharge at 1,000 gpm/well

June 2008	**Figure 19**
TODD ENGINEERS Alameda, California	**Potential Water Level Impacts Coastal Recharge Location**

The Hantush equation estimates the height of the groundwater recharge mound as a function of time and distance from the recharge area. The Hantush equation assumes that the underlying aquifer is unconfined, homogeneous, isotropic, and effectively infinite in areal extent. The analysis does not account for travel time and lateral flow of recharge water through the unsaturated zone, a sloping groundwater table, or aquifer stresses, such as pumping. Notwithstanding these limitations, the application of the equation to both the coastal and inland locations provides a reasonable preliminary estimate of groundwater level impacts from various volumes of recharge.

The Hantush equation was solved using the mounding function for a circular recharge area in Aqtesolv Pro 4.0 (Hydrosolve, Inc., 2006), the equation for which follows:

$$Z\,(r,t) = h_m^2 - h_i^2 = (V/2\pi K)[w(u_0) + (1-\exp(-u_0))/u_0]$$

where,

$Z\,(r,t)$ = Height of the mound above initial height of water table with respect to distance from center of recharge area over time

h_m = Height of mound above aquifer base

h_i = Initial height of water table above aquifer base

V = Volume of recharge water expressed as $w\pi R^2$, where w is the vertical infiltration rate from a circular recharge area of radius, R

K = Horizontal hydraulic conductivity of the aquifer

S = Storativity, or specific yield of the unsaturated zone

$w(u)$ = Theis well function for nonleaky aquifers

u_0 = $R^2/4nt$, where $n = Kb/S$ and $b = 0.5[h_i(0) + h(t)]$, where t = time since start of recharge, and b = constant of linearization

As shown in the equation, the development of the recharge mound is largely dependent on the vertical infiltration rate (w), storativity (S) of the unsaturated zone, and the horizontal hydraulic conductivity (K_h) and thickness of the saturated zone (b). Based on laboratory analyses of formation samples collected from the Aromas Sand during investigations at nearby Fort Ord, a vertical infiltration rate (K_v) of 3.5 feet/day, and K_h of 35 feet/day, and S of 0.25 were applied in the Hantush equation for the coastal location (Hydrometrics, 2007). Similarly, based on laboratory analyses of formation samples collected from the Paso Robles Formation, a K_v of 1.0 feet/day, K_h of 20 feet/day, and S of 0.12 were applied in the Hantush equation for the inland location. In order to select an initial saturated thickness for application in the equation, this value was limited to the calculated effective radius of recharge, as recommended by published data (Bouwer, 2002). Recharge mound contours generated by Aqtesolv Pro were exported as shape files and projected in the project GIS.

Results from the mounding calculations are shown on the upper portions of Figures 18 and 19 for the inland and coastal locations, respectively. For the inland location, four vadose zone recharge wells recharging at 400 gpm over five months are predicted to raise water levels about 40 feet near the wellfield (upper map on Figure 18). Although impacts are shown in a simplistic radial manner, most of the water will flow toward the pumping depression controlled by hydraulic gradients. Impacts will not occur at exactly the same time as recharge, given the thick vadose zone at the site. Vadose zone travel time was not evaluated in the analysis, but may provide water level benefits that overlap the pumping season.

Injection Wells. Effects on groundwater levels from operation of injection wells were evaluated using the Cooper and Jacob (1946) modification to the non-equilibrium Theis equation developed for confined aquifers. For purposes of this analysis, a flat surface is assumed for static water levels. As a result, groundwater level impacts are slightly overestimated in areas upgradient of each injection site and underestimated in the direction of the existing groundwater depression in the Santa Margarita Aquifer. The principle of superposition was used to evaluate the effects from multiple wells in a uniform regional flow field.

Aquifer parameters for the Santa Margarita Aquifer were selected based on existing findings from aquifer testing in the Seaside Basin. An aquifer transmissivity value (T value) of 85,100 gpd/ft was assumed for the Santa Margarita Aquifer for both the inland and coastal locations. For the inland location, a storativity value (S value) of 0.0018 was assumed for the Santa Margarita Aquifer, based on water level response measured in the FO-7 well during aquifer testing of the Paralta Well (Fugro, 1997). The T and S values are the same as those used to evaluate groundwater level impacts from the ASR project for MPWMD (Jones & Stokes, 2006). For the coastal location, an S value of 0.00081 was assumed for the Santa Margarita Aquifer, based on water level response measured in the PCA well during aquifer testing of the SMTIW-1 (ASR Systems, 2005).

The maps on the lower portion of Figures 18 and 19 show the analysis results for the inland and coastal locations, respectively. Impacts to water levels are larger than estimated for the vadose zone wells due to the confined nature of the Santa Margarita Aquifer. For the inland location, water levels are predicted to rise above 25 feet within the injection wellfield at the end of the recharge season (bottom Figure 18) and decrease radially with distance from the project site. Maximum drawup is estimated to be about 50 feet in the immediate vicinity of each injection well. Water levels are predicted to be impacted within the entire Northern Coastal Subarea and north of the basin.

For the coastal location, water levels are predicted to rise to higher levels over the subarea, given the larger allocation of GWR project water to injection and the planning of three injection wells. As shown by the contour map on the lower portion of Figure 19, water levels are expected to rise more than 32 feet in the vicinity of the injection wellfield. Maximum drawup is estimated to be about 40 feet in the immediate vicinity of each injection well. Increases in water levels are predicted over the entire Northern Coastal Subarea and cover the existing pumping depression in the aquifer.

Actual impacts will be configured differently than shown, based on the location of the pumping depression, heterogeneity in subsurface geology, and other factors. Nonetheless, the simplistic analysis provides a reasonable estimate for water level impacts. Both injection wellfields are assumed to be minimally operated during times of available Carmel River water injection at the ASR wells until well interference impacts can be determined.

5.2.2 Residence Time and Distance to Extraction Wells

Section 60320.010 of CDPH's Draft Regulations for Groundwater Recharge Reuse states: "For a subsurface injection project, all the recycled water shall be retained underground for a minimum of twelve months prior to extraction for use as a drinking water supply, and shall not be extracted within 2,000 feet of any GRRP [Groundwater Recharge Reuse Project] subsurface injection well" (CDPH, 2007).

Evaluation of existing production wells in the Seaside Groundwater Basin shows that no existing production wells are located within 2,000 feet of proposed recharge wells at either the coastal or inland locations as described below.

Inland Location. The City of Seaside's (former Ford Ord) Golf Course Reservoir well is the nearest existing production well to the inland location. The Reservoir well is an irrigation well tapping the Paso Robles Aquifer and is located 2,100 feet from the nearest proposed inland recharge well. The Paralta well is the nearest existing drinking water production well to the inland location. The Paralta well taps both the Paso Robles Aquifer and Santa Margarita Aquifer and is located 3,400 feet from the nearest proposed inland recharge well.

Coastal Location. The nearest existing production well to the coastal location is the City of Seaside's (former Ford Ord) Golf Course Coe Avenue well. The Coe Avenue well is an irrigation well tapping the Paso Robles Aquifer and is located 2,200 feet from the nearest proposed coastal recharge well. The Cal Am Military well is the nearest existing drinking water production well to the coastal location. The Military well taps the Paso Robles Formation and is located 3,700 feet from the nearest proposed coastal recharge well. Other nearby drinking water production wells include the Cal Am Luzerne and Luzerne Replacement wells. Both of these wells are located approximately 4,600 feet from the nearest proposed coastal recharge well. The Luzerne well taps the Paso Robles Aquifer and the Luzerne Replacement well taps the Santa Margarita Aquifer.

To determine whether the Seaside GWR meets the 12-month aquifer residence time, the time interval for injected recycled water to travel to the nearest existing production well was estimated for both the coastal and inland locations. The distance to the nearest production well at each location was divided by the average linear groundwater velocity of recycled water flowing through the subsurface, which was estimated using the following equation:

$$v = Ki/n_e$$

where, v = average linear groundwater velocity
K = hydraulic conductivity
i = hydraulic gradient (dh/dl)
n_e = effective porosity

Average aquifer hydraulic conductivities of 20 feet/day for the Paso Robles Aquifer and 63 feet/day for the Santa Margarita Aquifer were calculated from aquifer pumping tests and were applied for this analysis. An aquifer porosity of 0.12 and 0.10 for the Paso Robles Aquifer and Santa Margarita Aquifer, respectively, were assumed. Hydraulic gradients were measured were water level contour maps as presented in Yates et al. (2005) and shown on Figure 3 in Chapter 2.

The results of groundwater velocity analysis and the estimated residence time of recycled water for the inland and coastal locations are summarized in Table 5-1.

Table 5-1: Projected Groundwater Velocity and Residence Time of Recycled Water

Site	Nearest Production Wells	Groundwater Velocity (ft/day)	Residence Time (years)
Inland	Fort Ord Golf Course Reservoir - Q_{Tc}	1.43	4.0
	Paralta - Q_{Tc}	0.98	9.5
	Paralta - Q_{Sm}	5.56	1.7
Coastal	Fort Ord Golf Course Coe Ave. - Q_{Tc}	0.45	13.5
	Cal Am Military - Q_{Tc}	0.44	23.2
	Luzerne - Q_{Tc}	0.36	34.8
	Luzerne Replacement - Q_{Sm}	1.1	11.5

Q_{TC} = Residence time in Paso Robles Formation
Q_{SM} = Residence time in Santa Margarita Formation

Table 5-1 shows that average linear groundwater velocities range from about 0.36 feet/day to 5.56 feet/day. Based on these velocities, estimated residence times of recycled water at both sites range from 1.7 to 34.8 years. Residence times for the inland location are shorter due primarily to the steeper groundwater gradient in both the Paso Robles Formation and Santa Margarita Formation at the inland location. Also, residence times are shorter for recycled water traveling through the Santa Margarita Aquifer due to its relatively high hydraulic conductivity. The shortest residence time (1.7 years) was estimated at the inland location for recycled water flowing to the Paralta well through the Santa Margarita Aquifer.

5.2.3 Potential Water Quality Impacts

Both the Paso Robles and Santa Margarita Aquifers provide potable drinking water supply for Basin users that generally meets water quality standards. The Paso Robles Aquifer is characterized by a sodium chloride/calcium bicarbonate water type (Jones & Stokes, 2006). Mineral content expressed as total dissolved solids (TDS) averages 264 milligrams per liter (mg/L) in recent samples from FO-9 at the coastal location (Watermaster Database, 2008). The TDS concentration in the Paso Robles Test Injection Well (PRTIW) near SMTIW-1 was 371 mg/L in a 2008 sample (Watermaster Database, 2008). Water quality data are generally unavailable in the upper sands of the Paso Robles Aquifer or in the shallower Aromas Sand where it is saturated near the coast. Shallow water quality data will be collected and evaluated in site investigations associated with the GWR project.

The inorganic water chemistry and mineral content of the Santa Margarita Aquifer is thought to be similar to the Paso Robles Aquifer (Jones & Stokes, 2006), but mineral content can range up to much higher concentrations in the Santa Margarita Aquifer (Yates, et al., 2005). Recent samples from the aquifer in FO-9 indicate an average TDS of 264 mg/L at the coastal location (Watermaster Database, 2008). However, the TDS value at SMTIW-1 is 618 mg/L, indicating a higher mineral content inland.

The advanced wastewater treatment proposed for the MRWPCA groundwater replenishment projects will produce a high quality recharge water that is considered superior in many aspects to the ambient groundwater. The advanced wastewater treatment facility is described in Chapter 4 of this report. An additional evaluation will be conducted prior to project implementation to ensure the compatibility of recharge water with the Basin ambient groundwater and diluent water. Geochemical modeling will be conducted as soon as additional data are generated from the advanced wastewater treatment facility design process.

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APPENDIX A

REMOVED FROM CONSIDERATION

DESCRIPTION OF 180-FOOT AQUIFER GROUNDWATER REPLENISHMENT PROJECT

Introduction

The 180-foot Aquifer Groundwater Replenishment Project (180-GWRP) intends to work in conjunction with the regional brackish water extraction component of the regional water supply alternative. The 180-GWRP would deliver Salinas Industrial Water Treatment Plant (SIWTP) effluent to the proposed MRWPCA AWTP and inject the advanced treated water into the 180-foot aquifer north of the City of Marina along the Salinas River, thereby replenishing the groundwater in the Salinas Groundwater Basin. The proposed injection wells would be located on both sides of the Salinas River in close vicinity to the proposed Salinas River rubber dam that would be constructed as part of the Salinas River Diversion Facility (SRDF) Project (see Figure A-1). An EIR was prepared and certified for the SRDF in 2001.

The source of injection water would include industrial effluent obtained from the SIWTP, which is located north of Davis Road adjacent to the Salinas River. The SIWTP treats industrial wastewater from vegetable packers and processors, seafood processors, preserve manufacturers, etc. The influent to the SIWTP is treated through aeration ponds and is currently discharged to disposal beds and percolation ponds, which are operated by MRWPCA.

Groundwater Basin Setting

The Salinas Valley Groundwater Basin extends approximately 100 miles from river headwaters in the southeast to Monterey Bay in the northwest. Water-bearing materials within the aquifer consist of Holocene Valley Fill materials, Aromas Red Sands and Plio-Pleistocene Paso Robles Formation deposits, which are found from 0 to 1,000 feet below ground surface (bgs). The Salinas Valley Groundwater Basin has two main aquifers located at approximately 180 feet and 400 feet deep. The 180-foot aquifer is located between 50 and 250 feet bgs and is between 50 and 150 feet thick. The 180-foot aquifer is currently intruded with seawater due to over-extraction, which extends approximately three to five miles inland.

Groundwater extraction wells currently pump brackish water along the western and eastern borders of the Salinas Valley Aquifer. The injection of fresh water into the aquifer will occur between these two extraction areas, creating a hydraulic pressure ridge with a peak at the injection area. The injection of treated water into the aquifer will initially create a mixing of fresh and brackish water. However, the combination of saline water extraction and the injection of fresh water will eventually convert the aquifer to a more viable source of fresh groundwater.

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SCALE: 1"=1000'

MRWPCA 180-FOOT GROUNDWATER
REPLENISHMENT PROJECT

OVERALL 180-FT GWR PROJECT LAYOUT

FIGURE NO.
A-1

Project Facilities

Industrial Effluent

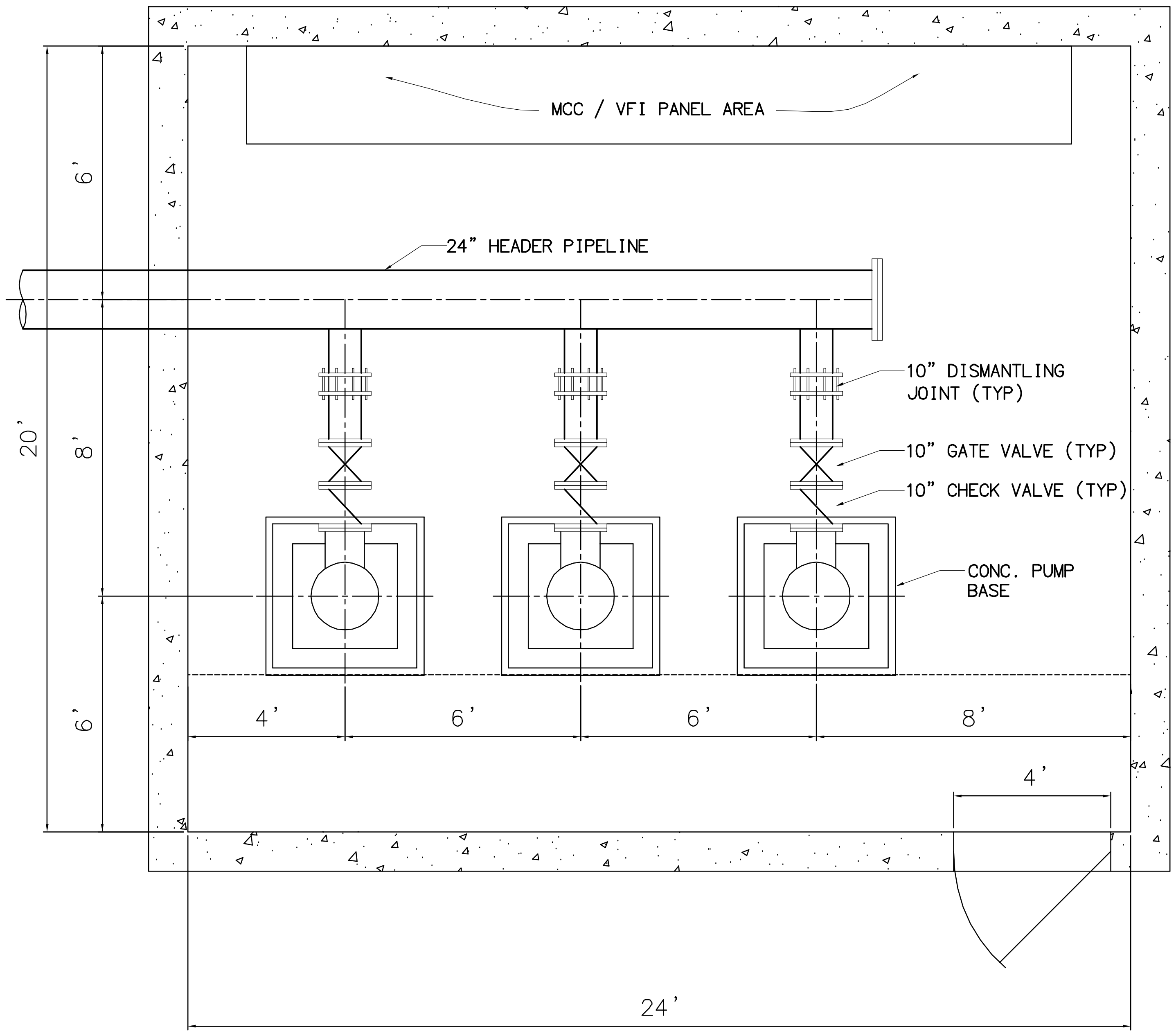
Industrial effluent would be pumped from Pond #3 at the SIWTP and conveyed through a proposed 24-inch PVC pipeline to the AWTP. After treatment through a separate flow stream of the AWTP, the industrial effluent would be injected into the 180-foot aquifer. The average monthly flow rates into the SIWTP ponds for 2007 are shown in Table A-1. According to representatives of the City of Salinas (C. Nizawa, personal communication, May 2008), the average flow rate from the SIWTP ponds during the irrigation season (approximately April to October) is projected to increase from the current flow of approximately 3 mgd to 10 mgd in the year 2030. The average flow rate during the non-irrigation season (approximately November to March) is projected to increase from the current flow of 1.3 mgd to 4.4 mgd.

Table A-1: Average Monthly Flow Rates Into SIWTP Ponds (2007)

Month	Average Flow Rate (mgd)
January	1.0
February	1.1
March	1.1
April	2.2
May	2.6
June	2.8
July	2.8
August	2.8
September	2.8
October	2.6
November	2.2
December	1.2
Average (irrigation season)	***2.7***
Average (non-irrigation season)	***1.3***

Industrial Effluent Pump Station

A new 7.6 mgd Industrial Effluent Pump Station (IEPS) would be required to convey water from the SIWTP ponds (approximate elevation of 32 feet MSL) to the proposed AWTP (approximate elevation of 145 feet MSL) located at the RTP. Industrial effluent water from Pond #3 would be discharged to the IEPS through the existing 18-inch diameter outlet pipeline near the southwest end of Pond #3. Table A-2 provides the IEPS design criteria. Figure A-2 illustrates the IEPS layout.



PLAN
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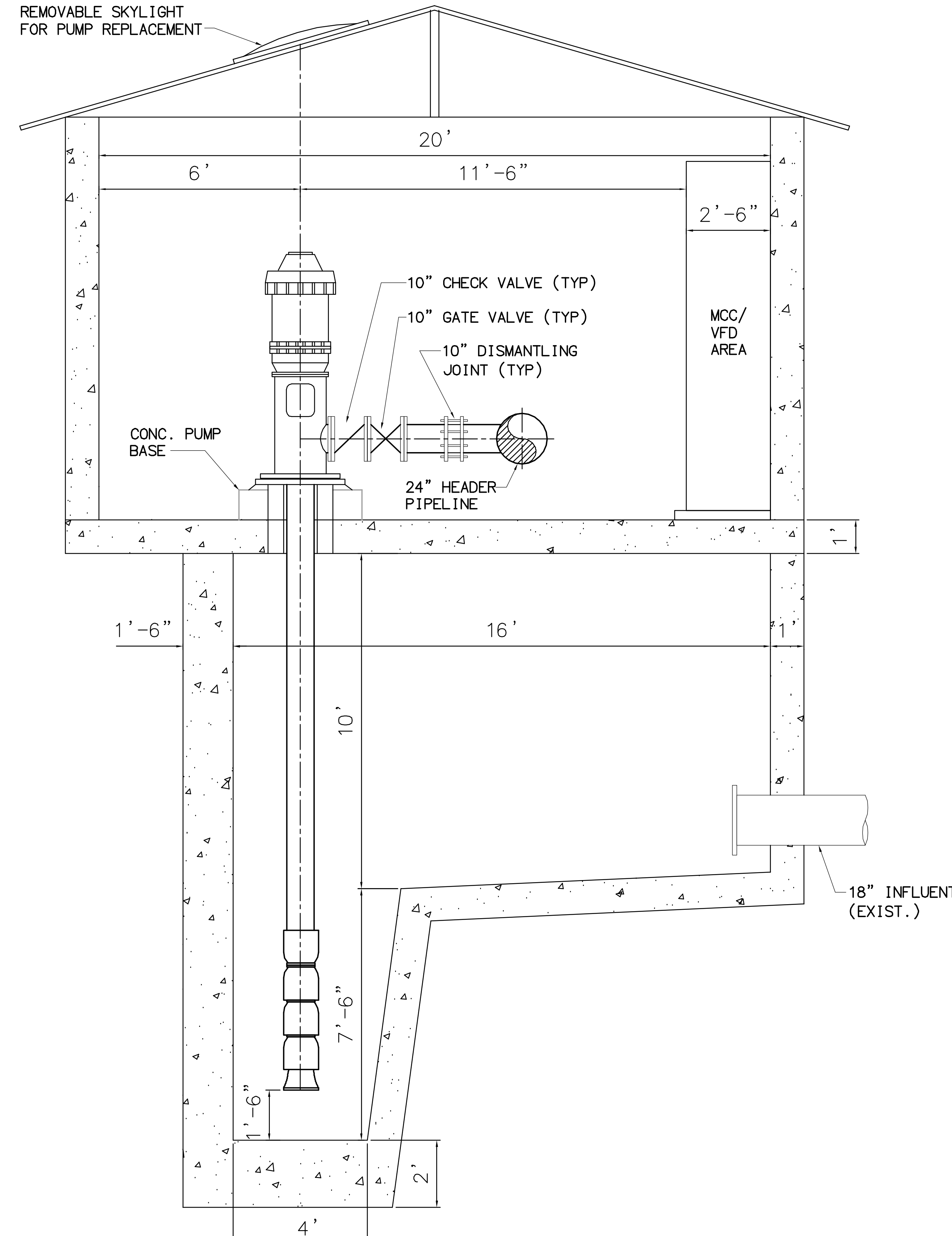

SECTION A/–
SCALE: 1/2" = 1'-0"

MRWPCA 180-FOOT GROUNDWATER REPLENISHMENT PROJECT

INDUSTRIAL EFFLUENT PUMP STATION

FIGURE NO. **A-2**

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Table A-2: Industrial Effluent Pump Station Design Criteria						
Pump Station Description	**Q_{max} (gpm)**	**Static Head (ft)**	**TDH (ft)**	**# of Pumps**	**Pump Description**	**Pump Configuration**
Industrial Effluent	5,210	113	185	3	VFD - 150 HP (3.75 MGD)	2 active, 1 stand-by

Industrial Effluent Pipeline

The SIWTP effluent would be conveyed from the proposed IEPS to the AWTP through a proposed 24-inch diameter pipeline. Table A-3 provides the industrial effluent pipeline design criteria. The industrial effluent pipeline alignment would begin at the IEPS and continue north through farmland to W. Blanco Road. The alignment would continue west on Blanco Road and then turn north on Cooper Road to Nashua Road. Where the alignment parallels public roads, the pipeline would be constructed within the public right-of-way but out of the paved road (in the shoulder) when possible. Once at the Blanco Drain, the pipeline would continue west to the proposed SRDF. The pipeline crossing at the Salinas River is proposed to be incorporated into the SRDF project. Figure A-1 shows the industrial effluent pipeline alignment.

Table A-3: Industrial Effluent Pipeline Design Criteria						
Pipeline Description	**Pipe Material**	**Pipe Class (psi)**	**Length (ft)**	**Diameter (in)**	**Qmax (gpm)**	**Vmax (ft/s)**
Industrial Effluent Pipeline	PVC (C905)	150	34,500	24	6,950	4.9

Injection Water Pipeline

Advanced treated water would flow by gravity through a proposed 24-inch PVC injection water pipeline to the injection well locations. Table A-4 provides the injection water pipeline design criteria. The alignment for the injection water pipeline would begin at the AWTP and follow the MRWPCA RTP and Monterey Peninsula Waste Management District (MPWMD) property lines east towards the proposed SRDF. The pipeline crossing at the Salinas River is assumed to be incorporated into the SRDF project. Figure A-1 illustrates the injection water pipeline alignment.

Table A-4: Injection Water Pipeline Design Criteria						
Pipeline Description	**Pipe Material**	**Pressure Class (psi)**	**Length (ft)**	**Diameter (in)**	**Qmax (gpm)**	**Vmax (ft/s)**
Injection Water Pipeline	PVC (C905)	150	10,000	24	4,200	3.0

Groundwater Injection Wells

Five 24-inch vertical groundwater injection wells are proposed for the 180-GWRP. They would be located perpendicular of the Salinas River. Three of the wells would lie on the western side of the Salinas River, while the other two would lie on the eastern side of the river. Table A-5 provides the injection wells design criteria. The injection wells would be laterally spaced approximately 1,000 feet from one another. Each proposed injection well would have an associated 6-inch diameter monitoring well installed approximately 50 feet down-gradient (west). Each well would be drilled an approximate length of 150 to 250 feet bgs to the bottom of the 180-foot aquifer, and is assumed to have 100 feet of 12-inch to 24-inch well screen installed. Each well would inject an average of approximately 900 gpm of advanced treated water into the aquifer. Based on currently proposed regulatory requirements, each injection well should be located a minimum distance of 1,000 feet from any existing groundwater extraction well. Figure A-1 illustrates the approximate injection well locations, and Figure A-3 illustrates a typical injection well site layout. Figure A-4 shows a side view profile of a typical injection well within the 180-foot aquifer.

Table A-5: Injection Well Design Criteria

# of Wells	Spacing (ft)	Depth (ft)	Injection Head (ft)	Injection Qtotal (gpm)	Injection Qavg/well (gpm)	Injection Qmax*/well (gpm)	Backflush Qmax/well (gpm)	TDH (ft)
5	1,000	230	50 (> 20 psi)	4,200	900	1,050	1,800	320

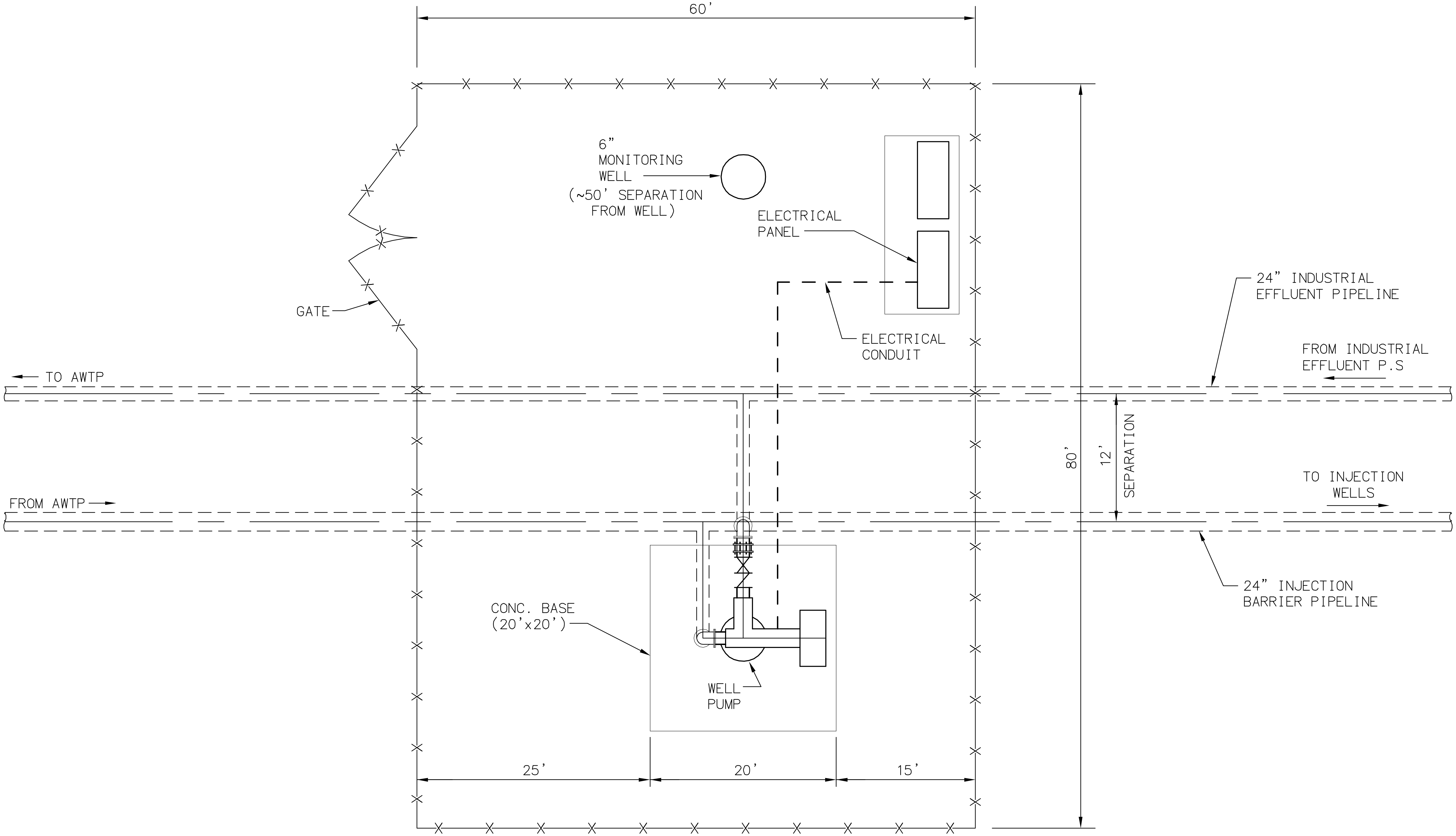
* Assuming one well out of service.

Project Operations

Four scenarios for distribution of the advanced treated water are described in Table A-6 for the initial year of operation (2013).

- Scenario 1: 100 percent distribution to the Seaside Groundwater Basin Replenishment Project (GWR) during the winter and 100 percent distribution to the 180-GWRP during the summer. This is the normal expected operation.
- Scenario 2: 100 percent distribution to the 180-GWRP all year. This scenario could occur due to issues with the RUWAP or, in the case of needing more dilution water, to issues with the 180-GWRP or due to Seaside Groundwater Basin needs.
- Scenario 3: 100 percent distribution to the GWR all year. This scenario could occur due to issues with the 180-GWRP or due to Seaside Groundwater Basin needs.
- Scenario 4: Shared distribution between the 180-GWRP and the Seaside GWR.

Table A-7 illustrates how the water source changes over the years. The SIWTP effluent could eventually be replaced or augmented by Blanco Drain water. Also, excess RTP secondary effluent or SVRP tertiary effluent not used for the CSIP Expansion could be processed through the AWTP and utilized.



MRWPCA 180-FOOT GROUNDWATER
REPLINISHMENT PROJECT

INJECTION WELL LAYOUT
(TYP)

FIGURE NO.
A-3

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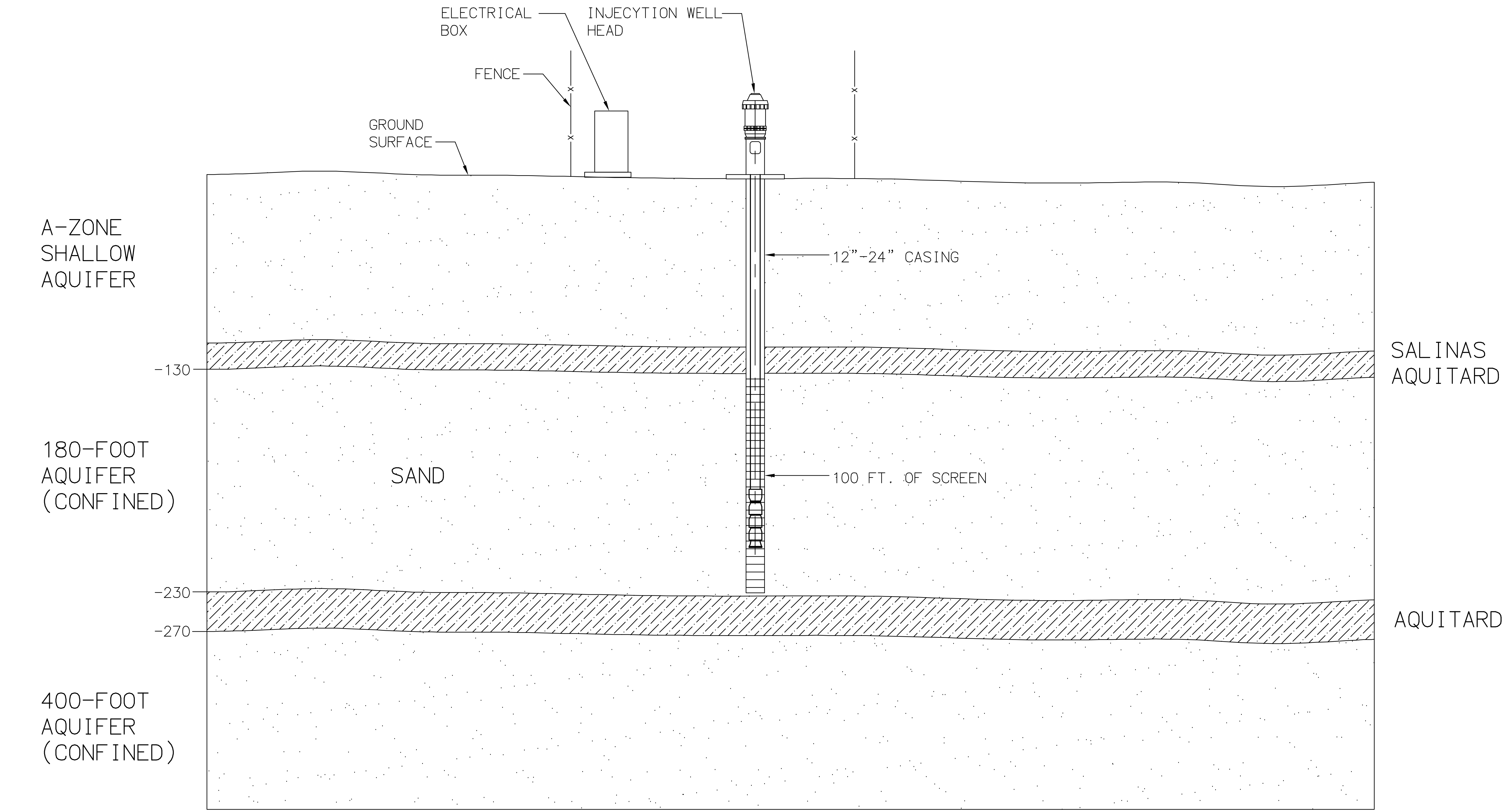

MRWPCA 180-FOOT GROUNDWATER REPLINISHMENT PROJECT

INJECTION WELL PROFILE (TYP)

FIGURE NO.
A-4

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Table A-6: Destination for AWTP Water (in AF) in 2013

Season	Destination			
	AWTP Product	GWR	RUWAP	180-GWRP
Scenario 1: Winter GWR and Summer 180-GWRP	5,845	2,400	28	3,417
Scenario 2: Maximize 180-GWRP Injection	5,845	0	0[1]	5,845
Scenario 3: Maximize Seaside GWR Injection	5,845	5,390	455[2]	0
Scenario 4: Shared 180-GWRP and Seaside GWR	5,845	0 to 5,300	0 to 455[2]	0 to 5,845

[1]RUWAP water is not treated with AWTP.

[2]Only Seaside golf courses connected to RUWAP initially, and they will receive only AWTP water.

Table A-7: Source of AWTP Water (in AF)

Year	Source Water (Maximum in AF)					
	RTP	SIWTP	Total[1]	MFBW[2]	Brine	Product[3]
2013 Winter[4]	1,807	1,269	3,076	243	405	2,428
2013 Summer	709[5]	3,619	4,328	342	569	3,417
2030 Winter	1,386[5]	2,040	3,426	271	451	2,705
2030 Summer	0	5,107	5,107	403	672	4,032

Blanco Drain Water is assumed to be 0 AF.

[1]Total= Influent flow to AWTP. Includes MF backwash water.

[2]MFBW=MF backwash water which adds to MRWPCA's allocation as a recycle flow.

[3]Product water=AWTP water for injection, RUWAP, or other use.

[4]First full year of operation.

[5]RTP water limited by allocation of 2,400 AF plus MFBW.

The IEPS would operate throughout the year to deliver industrial effluent to the AWTP via the industrial effluent pipeline. Initial operation would include instantaneous pumping of about 4 mgd, with an average of 3 mgd pumped during the irrigation season and an average of 1 mgd pumped during the non-irrigation season. Final operation would pump up to 7.6 mgd to the AWTP. Up to 6 mgd of advanced treated recycled and industrial water would flow by gravity, and be delivered to the 180-GWRP wells via the injection water pipeline. Advance treated water would gravity flow to the injection wells. Each well would receive approximately 900 gpm. Each injection well would be equipped with a pump and motor for periodic backflush of the well screens, which may occur once or twice a year. Water pumped from the periodic backflush would be discharged into the 24-inch industrial effluent pipeline and transported back to the AWTP for treatment and subsequent re-injection.

Energy Source and Requirements. The source of energy for the IEPS will be obtained at the existing industrial wastewater facility. Since the injection wells will only need to be backflushed approximately twice a year, the backflush pumps at the five injection wells will be powered by a portable engine generator. An aboveground, all weather electrical cabinet would be provided at each injection well. Backflushing of the wells would be performed one at a time; therefore, the portable generator for the injection wells will have adequate capacity to meet the demand of one injection well pump at a time. Table A-8 shows the power, energy and time estimates.

Table A-8: Power, Energy and Time Estimates for 180-GWRP

	Power (Hp)	Time (hr)	Energy (kW-hr)
Injection Well Backflush Pump	200 (installed per well)	240	40,800
IEPS*	450 (installed)	8,760 (12 months)	1,955,232

*Two pumps will run during the irrigation season (7 months), while one pump will run during the non-irrigation season (5 months).

Permits and Approvals Required

This section provides a preliminary identification of the permits and agreements required for the 180-GWRP, and a time estimate for completing them.

Industrial Effluent Conveyance System

- City of Salinas (to take wastewater from the Salinas Industrial Water Treatment Plant): 5 – 6 months
- Monterey County Health Department Division of Environmental Health (for WDR modification): 4 – 6 months
- Monterey County Health Department Division of Environmental Health (discharge permit for wastewater treatment): 4 – 6 months
- Monterey County Resource Management Agency Planning Services Department for Use Permit modification for Industrial Wastewater Facility: 6 – 9 months (up to 18 months if EIR required)
- RWQCB (for WDR modification): 4 – 6 months
- Right-of-way (private farms, etc.): 6 – 12 months
- Monterey County Public Works Department for encroachment permits: 1 month
- Monterey Bay Unified Air Pollution Control District (Authority to Construct Permit): 2 months

- Building Permit from Monterey Resource Management Agency Planning Services Department (if required): 12 – 18 months
- PG&E power: 12 – 15 months

Injection Pipeline and Wells

- Right-of-way from land owners for injection wells: 6 – 12 months
- Monterey County Health Department Division of Environmental Health drilling permits: 1 – 2 months
- Monterey County Water Resources Agency well/extraction permit: 3 – 6 months
- Monterey County Water Resources Agency agreement for river crossings at Salinas River Diversion Facility: 2 – 3 months
- RWQCB permit for drilling (if needed): 3 – 6 months
- CDPH approval for recharge: 12 – 18 months
- PG&E easements: 3 months

Construction Methods

This section discusses the anticipated construction methods for the facilities associated with the 180-GWRP. Tables A-9, A-10, and A-11 provide traffic data for peak day construction-related trips by project facility (materials delivery, haul trips, and worker commute). Table A-12 provides a list of the types and number of typical construction equipment that may be used for the construction of the 180-GWRP facilities.

Industrial Effluent and Injection Water Pipelines. The industrial effluent and injection water pipelines would follow the existing CSIP pipeline corridor from the AWTP to Nashua Road. The pipelines would be installed using conventional open-trench construction techniques. The SRDF project will incorporate the Salinas River crossing of the pipelines; therefore, no new construction activities or impacts will occur. The crossings of the Blanco Drain would be accomplished either by sheeting and dewatering the drain, or by jack-and-bore, depending on drain flows and the irrigation season.

The industrial effluent pipeline would then be open-cut installed within the public right of way along the shoulder of Nashua Road and Cooper Road, would cross Blanco Road, and would be installed along the shoulder of Blanco Road until turning into dirt and gravel agricultural service roads. This approach minimizes disruption to traffic as well as property owners, although traffic control would be required during the construction period. A temporary construction easement for pipeline installation would be up to 60 to 100 feet in width.

As stated, most of the construction would be open-cut trenching. Pipe sections would be placed in a trench of varying depth, and covered using conventional equipment such as backhoes, side-boom cranes, wheeled loaders, sheep's-foot excavators, and compactors. The pipelines would be constructed of PVC pressure pipe and would be typically delivered and installed in 40-foot-long sections. The pace of work for pipe installation is estimated to be about 250 linear feet per day.

Table A-9
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Materials Delivery							
Pipelines	Description	Max # of Truck Loads per max day	Average Miles per Trip	Hrs of Daily Delivery	Duration (hrs) of Daily Delivery	Max Trips Per Day	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Industrial Effluent Pipeline	Import Soil	12	10	6am-3pm	9	24	3	1
	Pipe Delivery	5	80	6am-3pm	9	10	1	1
Injection Water Pipeline	Import Soil	12	5	6am-3pm	9	24	3	1
	Pipe Delivery	5	80	6am-3pm	9	10	1	1
Total Pipelines		*34*				*68*	*8*	*4*
Pump Stations								
Industrial Effluent Pump Station	Concrete Delivery	10	20	6am-12pm	6	20	3	-
	Miscellaneous Materials	5	100	6am-3pm	9	10	1	1
Total Pump Stations		*15*				*30*	*4*	*1*
Injection Barrier Wells								
Wells and Backflush Facilities	Miscellaneous Materials	5	80	6am-3pm	9	10	1	1
Total Injection Barrier Wells		*5*				*10*	*1*	*1*
GRAND TOTAL		**54**				**108**	**13**	**6**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table A-10
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Haul Trips							
	Description	Max # Truck Loads per Day	Average Miles per Trip	Hrs of Daily Delivery	Duration (hrs) of Daily Delivery	Max Trips Per Day	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Pipelines								
Industrial Effluent Pipeline	Export Native Soil	15	15	6am-3pm	9	30	3	1
					9	-	-	1
Injection Water Pipeline	Export Native Soil	15	5	6am-3pm	9	30	3	1
					9	-	-	1
Total Pipelines		*30*				*60*	*7*	*4*
Pump Stations								
Industrial Effluent Pump Station	Earthwork Balanced	-				-		
						-		
Total Pump Stations		-				-	-	-
Injection Barrier Wells								
Wells and Backflush Facilities	Export drilling spoils	1		6am-3pm		2		
Total Injection Barrier Wells		*1*				*2*	-	-
GRAND TOTAL		**31**				**62**	**7**	**4**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table A-11
Traffic Data for Peak Day Construction Trips and Worker Commutes by Project Facility

Facility	Worker Commute						
	# Workers	Average Miles per Trip	Daily Work Hours	Trips Per Worker Per Day related to Construction	# of Daily Trips	# of Trips Per AM Peak Hour	# of Trips Per PM Peak Hour
Pipelines							
Industrial Effluent Pipeline	72						
Injection Water Pipeline	38						
Total Pipelines	*109*	12	*7am-4pm*	*2.2*	*240*	*109*	*109*
Pump Stations							
Industrial Effluent Pump Station	38						
Total Pump Stations	*38*	12	*7am-4pm*	*2.2*	*83*	*38*	*38*
Injection Barrier Wells							
Wells and Backflush Facilities	26						
Total Injection Barrier Wells	*26*	12	*7am-4pm*	*2.2*	*57*	*26*	*26*
GRAND TOTAL					**380**	**173**	**173**

1. Number of workers based on peak construction equipment operations estimated for Figure 16
2. Minimum material delivery is assumed to be 5 trips
3. Average load for hauling is 10 cubic yards per truck.
4. Data presents a combined worst case scenario for each and between the individual projects.
5. Materials and pipeline deliveries assumed from South Bay area.
6. Central Monterey Bay location assumed for concrete batch plant.
7. Central Monterey Bay location assumed for average mileage of worker commutes.
8. Average worker commute computed using AMBAG base year travel demand model - trip matrices by trip purpose, congested travel time skim matrices
9. AM peak is typically between 6:30am and 8:30 am. PM peak is typically between 3:30pm and 6:00pm.
10. Construction worker trips assumptions: one inbound trip during the AM, one outbound trip during PM, per worker. No carpooling is assumed during AM and PM and thus presents a worst case scenario.
11. Construction worker trips assumptions: Lunch hour/other trips: one trip per 5 workers for a 4 mile trip.
12. Grand Total assumes that all peak of truck trips and peak of worker commutes for each of the project facilities occurs on the same day.

Table A-12
Equipment Inventory During Peak Construction, by Project Area

Facility	Activity / Equipment																
	ROW/ Site Work						Building/ Installation						Asphalt Paving/ Finishing Work				
	Tractors	Graders	On-Site Trucks	Off-Site Trucks	Excavator[1]	Traffic Equipment[1]	Trenchers[1]	Crane[1]	On-Site Trucks	Off-Site Trucks	Bore/Drill Rig	Other	Trucks	Pavers[1]	Paving Equipment[1]	Rollers	Other
Pipelines																	
Industrial Effluent Pipeline	4	-	-	-	3	4	2	2	4	6	1	2	6	1	2	2	2
Injection Pipeline	4	-	-	-	3	-	1	1	2	4	1	2	1	1	1	1	-
Total Pipelines	*8*	-	-	-	*6*	*4*	*3*	*3*	*6*	*10*	*2*	*4*	*7*	*2*	*3*	*3*	*2*
Pump Stations																	
Industrial Effluent Pump Station	2	-	2	4	1	-	-	2	4	2	1	1	2	1	1	1	-
Total Pump Stations	*2*	-	*2*	*4*	*1*	-	-	*2*	*4*	*2*	*1*	*1*	*2*	*1*	*1*	*1*	-
Injection Barrier Wells																	
Wells and Backflush Facilities	1	-	-	-	1	-	1	1	2	2	1	2	1	1	1	1	
Total Injection Barrier Wells	*1*	-	-	-	*1*	-	*1*	*1*	*2*	*2*	*1*	*2*	*1*	*1*	*1*	*1*	-
Grand Total	*11*	-	*2*	*4*	*8*	*4*	*4*	*6*	*12*	*14*	*4*	*7*	*10*	*4*	*5*	*5*	*2*

1. Pipelines installed in paved roadways, dirt and gravel in roadway shoulders, and in agricultural services roads. Roadway paving/ resurfacing required for Industrial Effluent Pipeline
2. Salinas River Crossings will be constructed by SRDF Contractor during SRDF construction. Therefore no new or additional equipment will be utilized
3. The Grand Total is applicable if assuming each facility is constructed independent of one another; and, the peak construction day for each facility occurs on the same day during the construction period

Construction activities may involve trenching, spoil handling, pipeline installation, backfilling and restoration, and vehicle ingress and egress. All roadways and surfaces disturbed during pipeline installation would be restored. If possible, trench spoils would be temporarily stockpiled within the construction easement, then backfilled into the trench after pipeline installation. However, for purposes of this report, it was assumed that pipe zone material would be imported from offsite, and spoils would be exported for disposal. Earth cover over the pipe was assumed to be four feet. Variations in this depth would be required to accommodate local topography, hydraulic grade, and utility congestion, among other factors. Assuming a “neat” trench line, the trench width would be up to six feet. Therefore, each pipeline would result in up to 10 to 15 truckloads of import soil per day and 10 to 15 truckloads each of export soil. The total volume of non-native soil that would be imported to the trenches would be approximately 15,000-20,000 cubic yards and 5,000-10,000 cubic yards for the industrial effluent and injection water pipelines, respectively. In addition, the total volume of native soil that would be exported from the trenches would be approximately 20,000-25,000 cubic yards and 5,000-10,000 cubic yards for the industrial effluent and injection water pipelines, respectively. During the construction period, a maximum of approximately 110 workers of various skilled trades may work along the pipeline routes, which would result in temporary increases in traffic due to worker commutes and general construction activity.

Industrial Effluent Pump Station. The IEPS is proposed to be located on previously disturbed land adjacent to the SIWTP percolation ponds, near Pond #3 north of the Salinas River. Heavy equipment must be brought in and stored onsite. A temporary increase in traffic volumes on public roadways and agricultural service roads will occur as a result of project-generated traffic. During the construction period, up to approximately 35 to 40 workers of various skilled trades may be working during the peak of construction.

Site preparation would entail clearing of the area. Excavation and leveling of the site would be required for uneven gradient. Ground clearing and excavation of the site would be performed using available heavy equipment normally found at most construction sites. It is assumed that earthwork at the site would be balanced and, therefore, import or export of soil materials would not be required.

Construction operations would involve excavation and installation for pumping and piping systems, concrete foundation and surface pouring, pump-house building construction, and installing pipeline and pumping connections and other support equipment (control panels), finishing of the site, and perimeter fencing.

The construction of the IEPS will occur over an approximate 12-month period, with the peak of construction occurring during the non-growing season in order to minimize project-induced disruptions to agricultural activities and to take advantage of the lower groundwater levels near the vicinity of the Salinas River and the SIWTP ponds.

Injection Wells. Vertical injection wells are anticipated to be a maximum 24 inches in diameter, completed with a stainless steel, wire wrapped well screen to a depth up to 250 feet bgs. Each injection well would be accompanied with a 6-inch diameter, PVC-cased monitoring well screened to the same depth as the injection wells. Construction of the wells will be completed utilizing typical vertical well construction methods and will require the use of large drilling machinery. The equipment that will be used onsite consists of a 50-foot-long drill rig with a 40-foot mast, two 40-foot pipe trailers with drill pipe, two portable drilling fluid tanks, cutting spoil pile area for debris, gravel pack storage area for 100 cubic yards, casing storage area for 24-inch pipe, a 40-foot-long 20,000 gallon frac-tank for development water de-silting, and area for miscellaneous equipment required including support trucks, pumps, air compressors, light plants, out-house, driller's trailer, and engineer trailer. Typically an area of 200 feet by 200 feet is necessary for the well construction operation, although with off-site staging, such as at the AWTP site, the area can be reduced to 100 feet by 200 feet.

The injection wells may be drilled with up to 36-inch diameter drill bits to the proposed well screen depth (varies per well), and 24-32 inch diameter well casing would be installed at that depth. The drill bit would then be reduced to 20-24 inches in diameter and the well drilled to a depth of up to 250 feet bgs. The well screen may be 16-24 inches in diameter and will be confirmed during the detailed hydrogeological investigation and testing phase of the project.

For each injection and monitoring well couplet, approximately 50 cubic yards of drilling spoils would be generated that would require offsite disposal. Materials would be delivered to the site as required. Construction, development, and equipping of the injection wells and installation of the backflush facilities would occur over an approximate 14 to 18 month period, if it is assumed that each well is drilled and developed sequentially. Drilling and development of each well and its accompanying monitoring well would occur over a 2-month period. Equipping and installation of the backflush facilities, electrical connections and system controls would occur over a four to eight month period and may overlap the well-drilling period. During the construction period, approximately 25 to 30 workers of various skilled trades may be working during the peak construction day.

Potential Water Level Impacts of 180-GWRP

The Salinas Valley Integrated Groundwater and Surface Water Model (SVIGSM) was used to evaluate potential groundwater and surface water impacts from the 180-GWRP. Impacts to the 180-foot aquifer were analyzed through comparison to a future conditions baseline simulation (called "Baseline" in this section). The Baseline includes land use and water use indicative of 2030 conditions, and is a refined version of the future conditions Baseline utilized in the EIR/EIS for the Salinas Valley Water Project (SVWP). Two scenarios were evaluated: 1) Regional Project Scenario 3a, which includes the facilities associated with Cal Am's CWP (i.e., desalination facility, additional recycled water deliveries, changes to surface water diversions, etc.), and 2) Regional Project With MRWPCA Injection Wells, which is defined as Regional Project Scenario 3a plus the 180-GWRP project.

Regional Project Scenario 3a

The results of the Regional Project 3a Scenario analysis show that the proposed ten seawater and inland wells pumping continuously in the 180-foot aquifer would create an extraction barrier or trough parallel to the coast. This feature is formed by causing seawater to flow inland towards the seawater wells (the five wells closest to the ocean, see Figure A-5), while brackish water from seawater-intruded groundwater would flow seaward towards the five inland wells. Operating the wells continuously in this manner will maintain a barrier that would prevent future seawater intrusion of the 180-foot aquifer.

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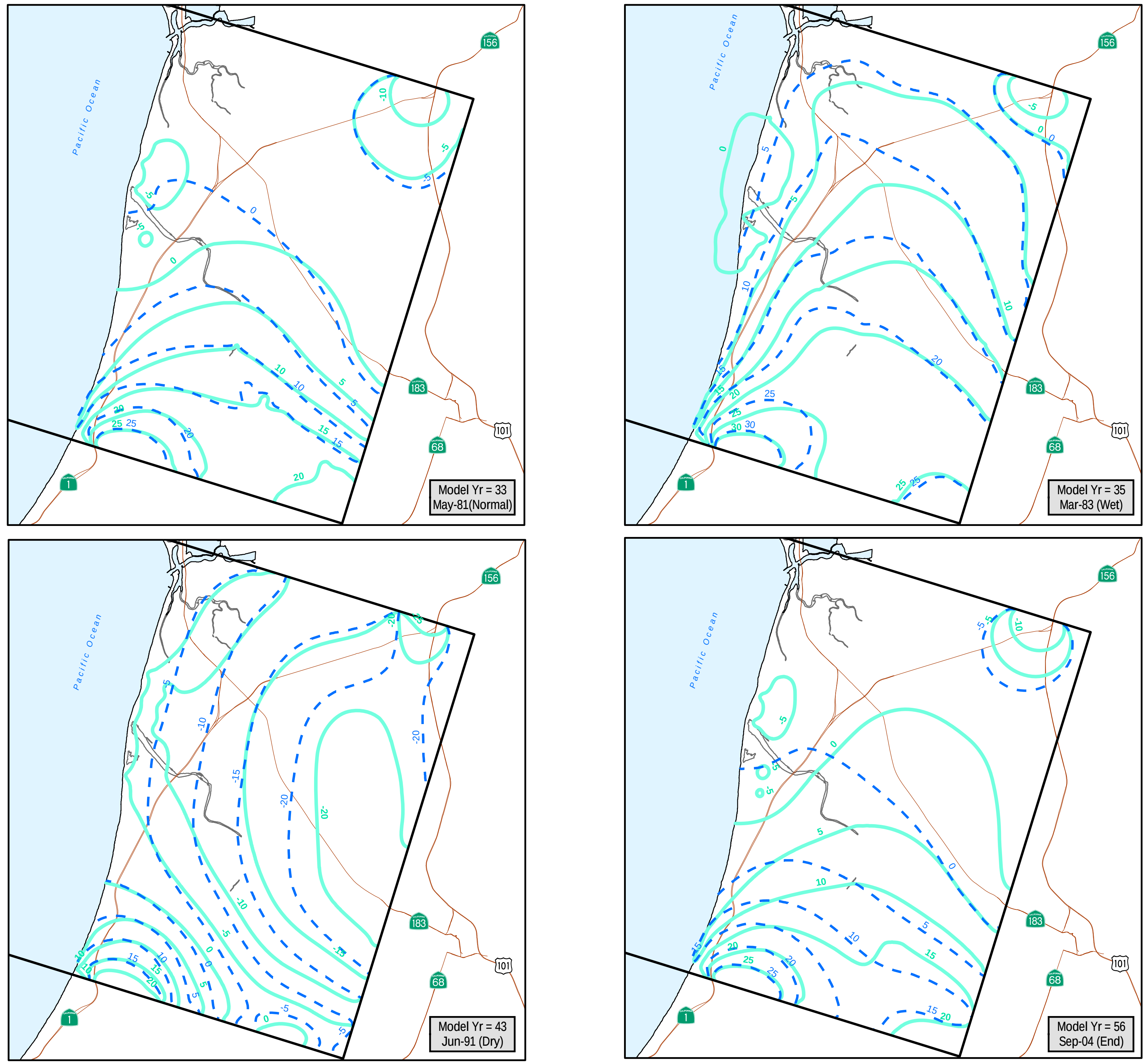
NORTH MARINA GROUND WATER MODEL
EVALUATION OF POTENTIAL PROJECTS



180FT AQUIFER BASELINE vs. REGIONAL PROJECT SCENARIO GROUND WATER ELEVATIONS

EXPLANATION

GEOSCIENCE Model Boundary

Baseline Ground Water Elevation, ft amsl

Regional Project Scenario Ground Water Elevation, ft amsl

Highway

Model Yr = 13
Jun-1961 (Dry)

Predictive Model Year*
Hydrologic Year

* Years Since Start of Model Scenario

13-Jun-08

Prepared by: DWB

Map Projection:
State Plane 1983, California Zone IV

NORTH

0 2 4 Miles

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Figure A-5

Other changes in groundwater levels between the Baseline and the Regional Project 3a Scenario within the focused model area are shown on Figure A-6 and summarized below:

- In normal hydrologic years (precipitation is close to the long-term average), groundwater flow caused by the Regional Project (southwest to northeast) remains similar to the Baseline condition, with the exception of the pumping trough developed around the Regional Project desalination wells. This locally alters the groundwater flow by drawing down ground water by 10 feet more than would have occurred under Baseline conditions near the coast.

- Under wet hydrologic conditions (precipitation is well above average), the effects of the Regional Project are less than under normal hydrologic conditions. In general, groundwater flow direction for Baseline and Regional Project conditions are quite similar (flowing southwest to northeast) with a component also flowing towards the ocean. Although the pumping trough is still present, it has less of an effect south and east of the desalination wells compared to No Project conditions. Increased recharge to the 180-foot aquifer from infiltration of precipitation and streamflow percolation during wet years allows for more groundwater outflow to the ocean.

- In dry years (precipitation well below average), the groundwater elevations east of the Regional Project wells are higher than under Baseline conditions. There is a strong component of groundwater flow from west to east (i.e., inland flow), which is reversed from flow in wet conditions (i.e., towards the ocean). The pumping trough developed by the Regional Project in dry years will reduce the hydraulic gradient towards the east compared to the Baseline conditions. In effect, the Regional Project would reduce the rate of seawater intrusion, which would normally be more prevalent during dry years under Baseline conditions.

- After 56 years of operating the Regional Project, the inland groundwater elevations in the 180-foot aquifer would be higher than under Baseline conditions. The area around the Regional Project wells would have lower groundwater elevations due to the trough developed by continuous pumping. Groundwater flow directions would be similar to normal hydrologic year flow directions.

Selected hydrographs showing the Baseline and Regional Project groundwater elevations over the 56 years of the predictive model are provided on Figure A-7. In general, the desalination wells of the Regional Project show a decline in groundwater levels just less than 20 feet inland of the Project wells; differences in groundwater levels between the Baseline and Regional Project conditions are minimal (less than 5 feet). This includes wells completed in the 400-foot aquifer and deep aquifer underlying the 180-foot aquifer. These deeper aquifers show almost no impacts from the Regional Project pumping in the 180-foot aquifer.

Figure A-8 shows the 500 mg/L chloride limit of the seawater intrusion in the 180-foot aquifer at selected times over the 56-year model period. In general, the intrusion is reduced at a faster rate when the Regional Project is operating compared to No Project conditions. Only the area just south of the Salinas River mouth remains intruded longer than if there was no project. This is due to the trough that is designed to extract mostly seawater from the seawater wells of the Regional Project.

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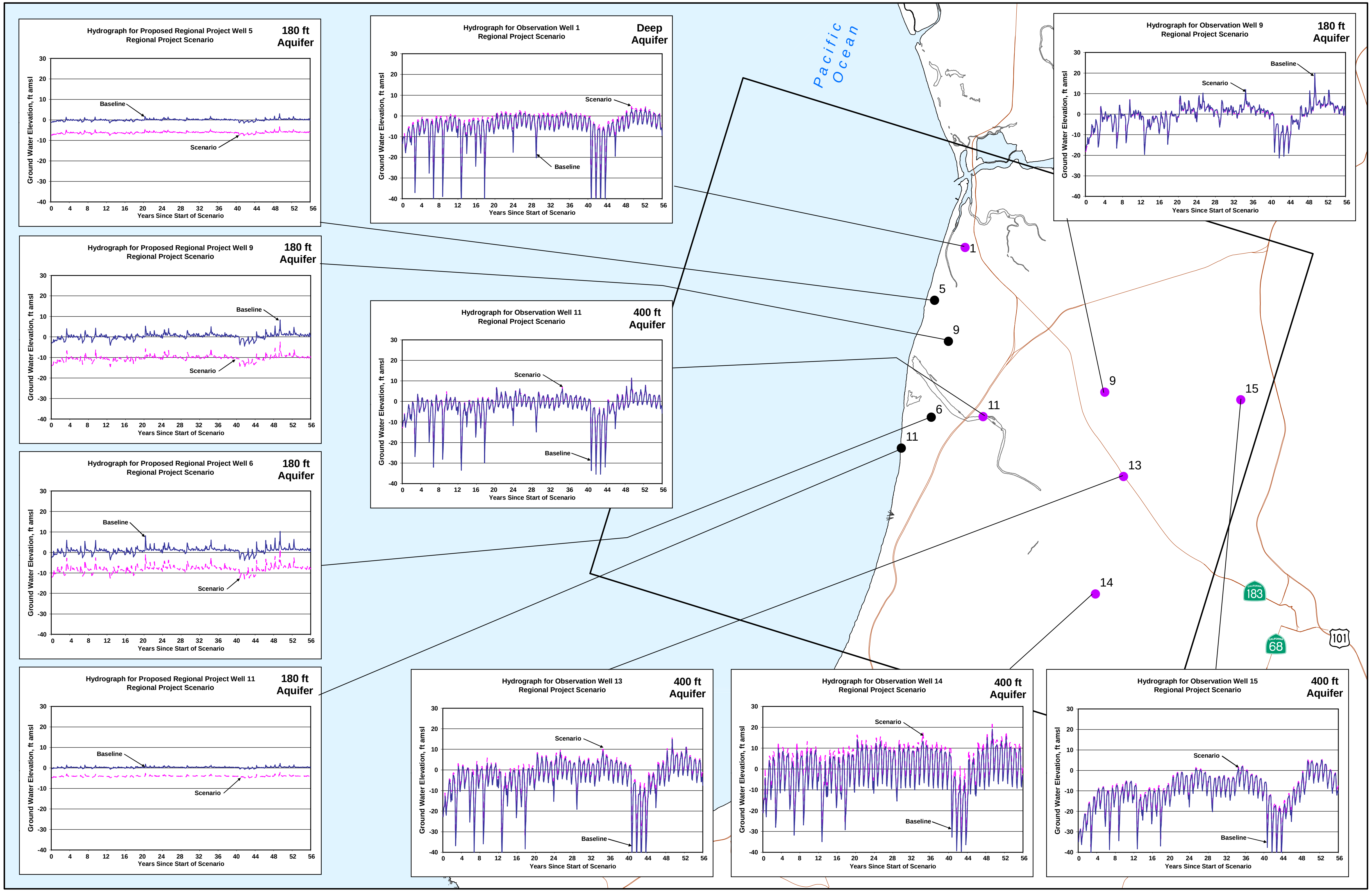
EVALUATION OF POTENTIAL PROJECTS

REGIONAL PROJECT SCENARIO GROUND WATER HYDROGRAPHS



EXPLANATION

Well Hydrograph Locations

- Monterey Regional Project Well
- Observation Well
- Slant Wells
- GEOSCIENCE Model Boundary
- Highway

13-Jun-08

Prepared by: DWB

Map Projection:
State Plane 1983, California Zone IV

NORTH

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Figure A-6

GIS_proj/cal_am_north_marina_modeling_5-08/0_Fig_A-6_3a_reg_proj_Hydrographs.mxd

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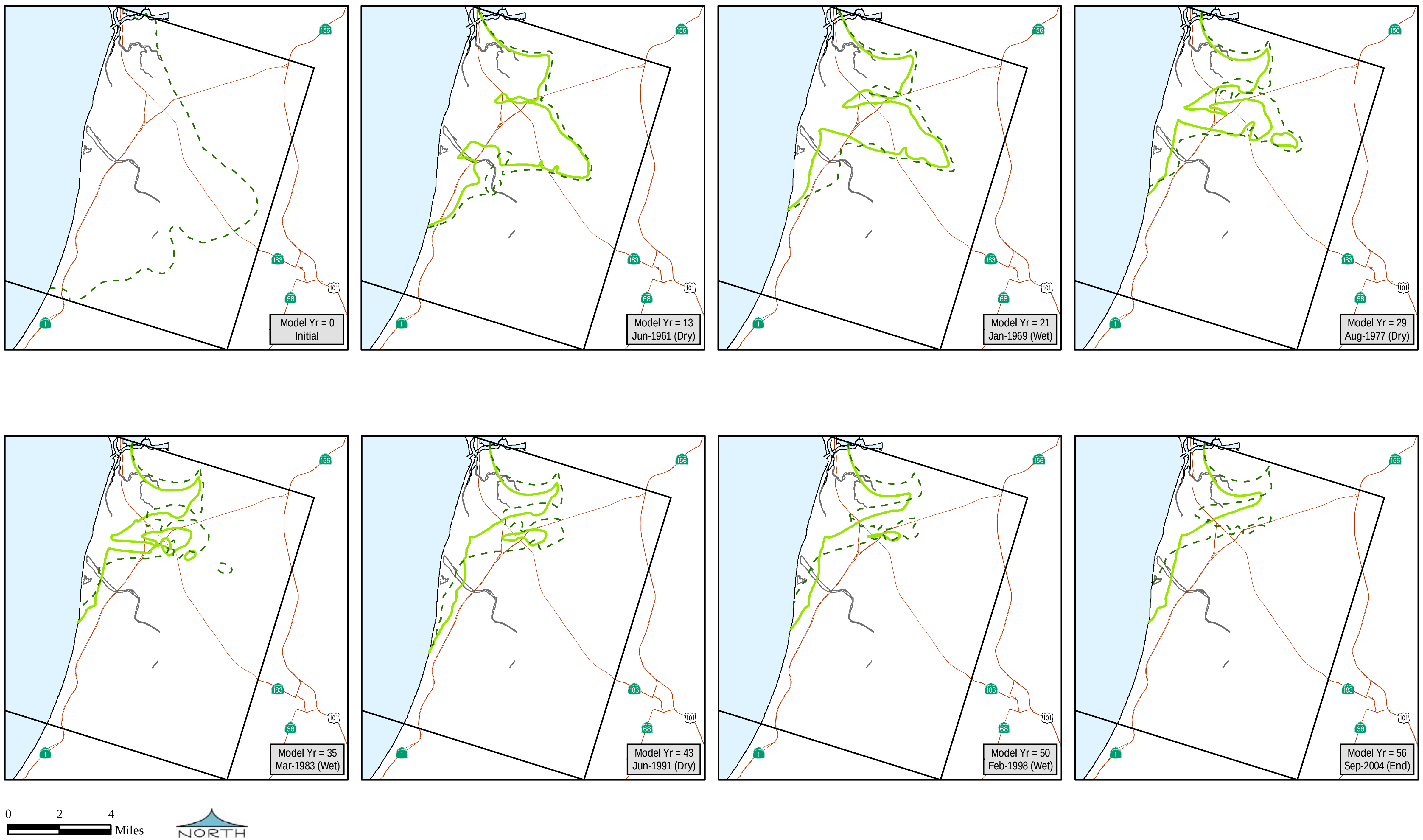
EVALUATION OF POTENTIAL PROJECTS

180FT AQUIFER BASELINE vs. REGIONAL PROJECT SCENARIO SEAWATER INTRUSION



EXPLANATION

GEOSCIENCE Model Boundary

Baseline Seawater Intrusion Chloride = 500 mg/L

Regional Project Scenario Seawater Intrusion, Chloride = 500 mg/L

Highway

Model Yr = 13 Jun-1961 (Dry) — Predictive Model Year* Hydrologic Year

* Years Since Start of Model Scenario

13-Jun-08

Prepared by: DWB

Map Projection:
State Plane 1983, California Zone IV

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Figure A-7

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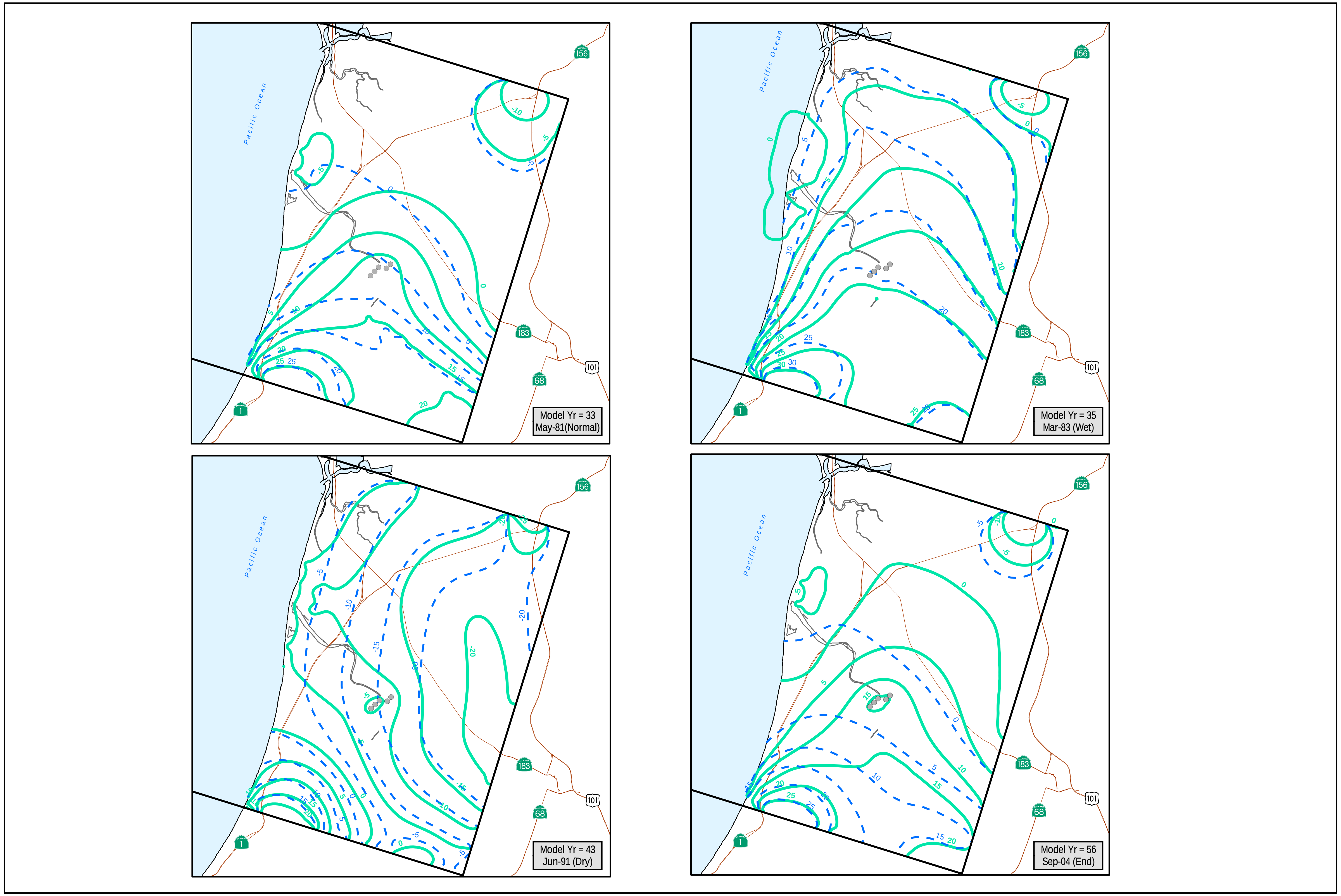
NORTH MARINA GROUND WATER MODEL
EVALUATION OF POTENTIAL PROJECTS



180FT AQUIFER BASELINE vs. REGIONAL PROJECT AND INJECTION SCENARIO GROUND WATER ELEVATIONS

EXPLANATION

GEOSCIENCE Model Boundary

Injection Well

Baseline Ground Water Elevation, ft amsl

Regional Project and Injection Scenario Ground Water Elevation, ft amsl

Highway

Model Yr = 13
Jun-1961 (Dry)
Predictive Model Year*
Hydrologic Year

* Years Since Start of Model Scenario

13-Jun-08

Prepared by: DWB

Map Projection:
State Plane 1983, California Zone IV

NORTH

0 2 4 Miles

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Figure A-8

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Regional Project With MRWPCA Injection Wells

The Regional Project Scenario 3a operating together with the proposed five MRWPCA 180-GWRP injection wells results in very similar impacts to the 180-foot aquifer as the Regional Project on its own. Treated water injection does cause a mound that slightly changes flow directions and gradients in the near vicinity (up to 2 miles away) of the injection wells (see Figure A-9).

Differences between the Regional Project Scenario 3a only and the Regional Project With MRWPCA Injection Wells are summarized below.

- In normal hydrologic years (precipitation is close to the long-term average), groundwater will mound by up to 10 feet in the area around the injection wells. This will result in higher groundwater elevations around the injection wells for up to a distance of 2 miles. The trough developed by the Regional Project will be slightly smaller than if the MRWPCA injection wells were not operating. The effectiveness of the trough to act as a barrier to seawater intrusion will not be compromised as a result of MRWPCA injection. In fact, it is anticipated that due to the increase in hydraulic gradient between the MRWPCA injection wells and Regional Project, the seawater intrusion will be reduced faster.

- Under wet hydrologic conditions (precipitation is well above average), the effects of MRWPCA injection will hardly be noticeable.

- In dry years (precipitation well below average), the MRWPCA injection operation will cause up to a 10-foot increase in groundwater elevations around the injection wells. Overall, groundwater flow will not be altered much from Regional Project only conditions.

- After 56 years of operating the Regional Project, the stabilized impact of MRWPCA injection will be a mound of approximately 10 feet in the vicinity of the injection wells, with a slightly smaller pumping trough developed by the Regional Project.

The hydrographs on Figure A-8 show the 10-foot increase in groundwater elevation in the injection wells, with the Regional Project wells having just under 20 feet of drawdown. Inland wells completed in the deeper aquifers (400-foot aquifer and Deep Aquifer) and 180-foot aquifer show minimal changes.

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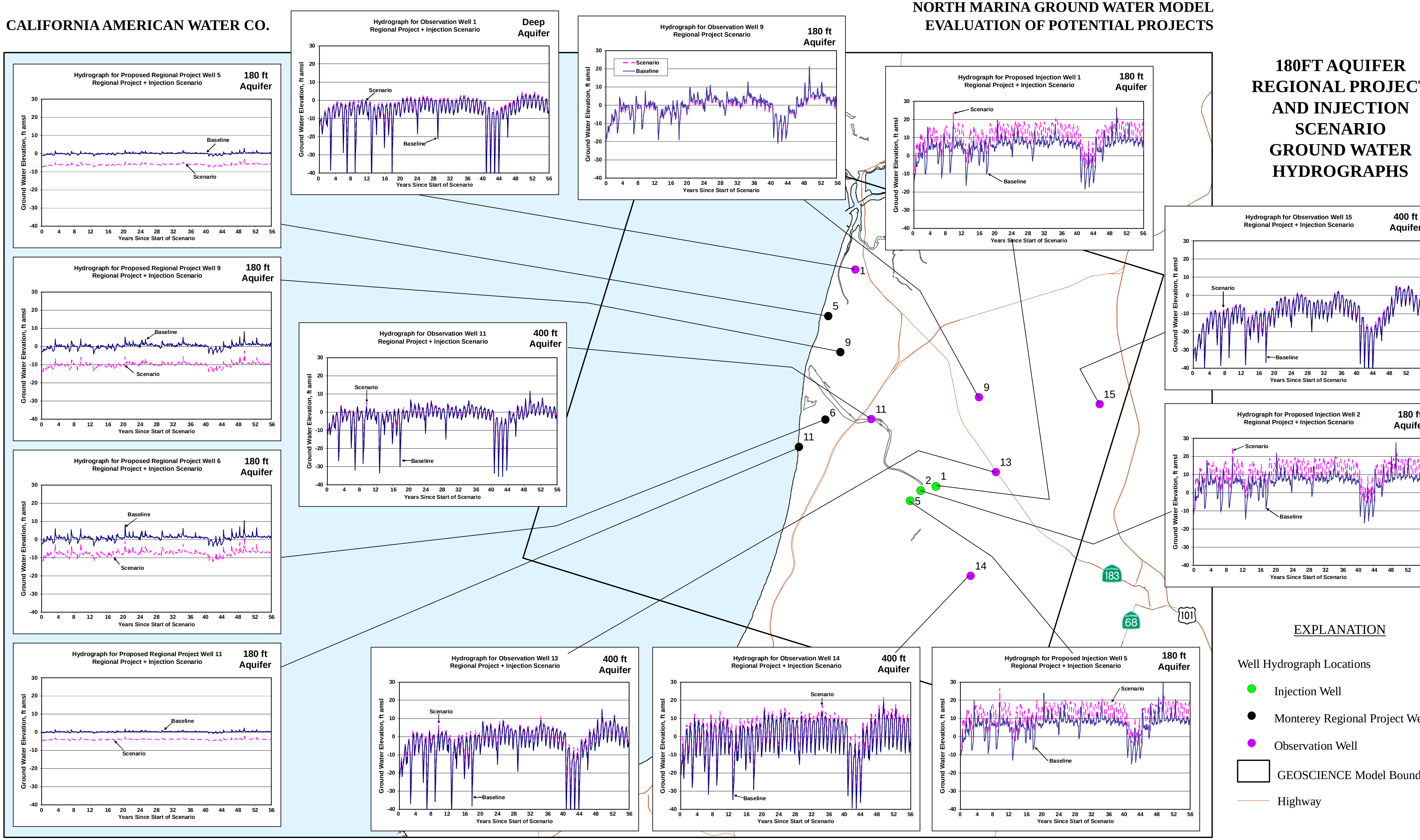
NORTH MARINA GROUND WATER MODEL
EVALUATION OF POTENTIAL PROJECTS

180FT AQUIFER REGIONAL PROJECT AND INJECTION SCENARIO GROUND WATER HYDROGRAPHS



EXPLANATION

Well Hydrograph Locations

- Injection Well
- Monterey Regional Project Well
- Observation Well
- GEOSCIENCE Model Boundary
- Highway

13-Jun-08

Prepared by: DWB

Map Projection:
State Plane 1983, California Zone IV

NORTH

0 2 4 Miles

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Figure A-9

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