Regional Conveyance System Study - Phase A

Final Report

August 2020

Prepared for:

Our Region's Trusted Water Leader
San Diego County Water Authority

Prepared by:

BLACK & VEATCH

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BLACK & VEATCH
Building a world of difference.

August 7, 2020

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Acronyms and Abbreviations

%	percent
AAC	All-American Canal
ABDSP	Anza-Borrego Desert State Park
ACEC	Area of Critical Environmental Concern
ACSR	aluminum conductor steel reinforced
AF	acre feet
AFY	acre feet per year
APCD	Air Pollution Control District
Avg	average
AWPF	advanced water purification facility
B	billion
Basin Plan	The Water Quality Control Plan for Region 7 of the Colorado River Basin
BLM	Bureau of Land Management
BMED	Bipolar Membrane Electrodialysis
BMP	Best Management Practice
BOR	Bureau of Reclamation
BW	backwash system
BWD	Borrego Valley Water District
Caltrans	California Department of Transportation
CCPP	Calcium Carbonate Precipitation Potential
CCRO	Closed Circuit Reverse Osmosis
CCVT	Capacitor Voltage Transformers
CDFW	California Department of Fish and Wildlife
CEQA	California Environmental Quality Act
CESA	California Endangered Species Act
CERRO	concentrate enhanced recovery reverse osmosis
cfs	cubic feet per second
CIP	clean in place
City	City of San Diego

Cl_2	chlorine
ClO_2	chlorite
ClO_3	chlorate
CMBRC	California-Mexico Border Relations Council
CMLC	Cement Mortar Lined and Coated
CNRA	California Natural Resources Agency
CRC	Colorado River Conveyance
CSP	Carryover Storage Project
CT/PT	Current Transformer/Potential Transformer
CVWD	Coachella Valley Water District
CWA	Clean Water Act
Dia	diameter
DAF	dissolved air flotation
DBPs	disinfection byproducts
DDVV	Del Dios Valve Vault
DEIR	Draft Environmental Impact Report
DEIS	Draft Environmental Impact Study
DO	Dissolved Oxygen
DSOD	Division of Safety of Dams
EA	Environmental Assessment
ECA	Energy Cost Adjustment
ECA-R	Energy Cost Adjustment Renewables
EDM	Electrodialysis Metathesis
EDR	electrodialysis reversal
EHS	extra high strength
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
ENR	Engineering News Record
EPA	Environmental Protection Agency
ESA	Endangered Species Act
FEIR	Federal Environmental Impact Report
FERC	Federal Energy Regulatory Commissions
FO	Forward Osmosis

fps	feet per second
FRP	fiberglass reinforced plastic
FR-RO	Flow Reversal Reverse Osmosis
FRS	Flow Regulatory Structure
ft	foot/feet
GDR	Geotechnical Desktop Report
gfd	gallons per square foot per day
GHG	greenhouse gas
GIS	Geographic Information System
Gmp	gallons per minute
GSP	Groundwater Sustainability Plan
GWH/yr	Gigawatt hours per year
H	height
HDPE	high-density polyethylene
HERO	High Efficiency Reverse Osmosis
HGL	hydraulic grade line
hp	horsepower
HPG	Hunter Pacific Group
HRRO	High Recovery Reverse Osmosis
HVAC	Heating, ventilation, and air conditioning
I	Interstate
ID	Inside diameter
IID	Imperial Irrigation District
IX	struvite recovery
kV	kilovolt
kW	kilowatt
kWh	kilowatt hour
LSI	Langelier Saturation Index
MA	Member Agency
max	maximum
MF	membrane filtration
MF/RO	microfiltration/reverses osmosis
MG	million gallons

mg/L	milligrams per liter
mgd	million gallons per day
min	minimum
MMF	multimedia filtration
MMRP	Mitigation Monitoring and Reporting Program
MNP	Maximum Normal Pool
MOU	Memorandum of Understanding
MSL	mean sea level
MW	Megawatt
MWD	Metropolitan Water District of Southern California
NA	Not Applicable
ND	Non Detect
NEPA	National Environmental Policy Act
NGO's	Non-Government Organizations
NHPA	National Historic Preservation Act
NM	not measured
No.	number
NOA	Notice of Availability
NOC	Notice of Completion
NOD	Notice of Determination
NOP	Notice of Preparation
NPDES	National Pollutant Discharge Elimination System
NPS	National Park Service
NPV	net present value
NTU	Nephelometric Turbidity Unit
O&M	Operation and Maintenance
OD	outside diameter
OMR	operations, maintenance, and replacement
OOO	Oceanside Ocean Outfall
PCF	Pressure Control Facilities
PES	Polyethersulfone
PGF	power generating facilities
pH	Potential of Hydrogen

ppb	parts per billion
ppd	pounds per day
ppm	parts per million
P3	Public Private Partnerships
PS	Pump Station
psi	per square inch
psig	pounds per square inch gauge
PSUs	plate settler treatment units
PVDF	Polyvinylidene
QSA	Quantification Settlement Agreement
RCS	Regional Conveyance System
RO	reverse osmosis
ROFs	Requests for Offers
ROW	rights-of-way
RP PC&HF	Rancho Penasquitos Pressure Control and Hydroelectric Facility's
RWQCB	Regional Water Quality Control Board
SANDAG	San Diego Association of Governments
SCADA	supervisory control and data acquisition
SCH	Species Conservation Habitat
SDCWA	San Diego County Water Authority
SDG&E	San Diego Gas & Electric
SDI	silt density index
SHC	Salton Sea Conservation Habitat
SHPO	State Historic Preservation Office
SMCL	secondary contaminant level
SR	State Route
sq ft	square foot
SSMP	Salton Sea Management Plan
STF	Salinity Treatment Facility
Study	Regional Conveyance System Study
SS	Stainless Steel
SVP	San Vicente Pipeline
SVPS	San Vicente Pump Station

SVR	San Vicente Reservoir
SVSCF	San Vicente Surge Control Facility
SWPPP	Storm Water Pollution Prevention Plan
SWQMP	Storm Water Quality Management Plan
SWRCB	State Water Resources Control Board
T	tunnel
TBM	tunnel boring machine
TBD	to be determined
TDS	total dissolved solids
TF	Treatment Facility
THM	trihalomethane
TODS	Twin Oaks Diversion Structure
TOU	time-of-use
TOVWTP	Twin Oaks Valley Water Treatment Plant
TSS	total suspended solids
TWS	Traveling Water Screens
UF	Ultrafiltration
μg/L	micrograms per liter
U.S.	United States
USEPA	United States Environmental Protection Agency
USGS	U.S. Geological Survey
USFWS	United State Fish and Wildlife Service
UWMP	Urban Water Management Plan
VCMWD	Valley Center Municipal Water District
VFD	Variable Frequency Drives
VSEP	Vibratory Shear Enhanced Process
W	width
Water Authority	San Diego County Water Authority
WAIV	Wind Aided Intensified Evaporation
WRO	water rights order
WSM	Westside Main Canal
WTF	Water Treatment Facility
WTP	Water Treatment Plant

Yr	year
YID	Yuma Irrigation District
ZDD	zero discharge desalination
μg	microgram
μg/L	microgram/Liter

Member Agencies of San Diego County Water Authority

Abbreviations	Agency
CMWD	Carlsbad Municipal Water District
CP	Camp Pendleton Marine Corps Base
Del Mar	City of Del Mar
Escondido	City of Escondido
FPUD	Fallbrook Public Utility District
HWD	Helix Water District
LWD	Lakeside Water District
National City	City of National City
Oceanside	City of Oceanside
OMWD	Olivenhain Municipal Water District
OWD	Otay Water District
PDMWD	Padre Dam Municipal Water District
Poway	City of Poway
Ramona MWD	Ramona Municipal Water District
Rincon del Diablo MWD	Rincon del Diablo Municipal Water District
RMWD	Ramona Municipal Water District
San Diego	City of San Diego
SBID	South Bay Irrigation District
SDWD	San Dieguito Water District
SFID	Santa Fe Irrigation Water District
VCMWD	Valley Center Municipal Water District
VID	Vista Irrigation District
VWD	Vallecitos Water District
Yuima	Yuima Municipal Water District

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INTRODUCTION

Chapter 1.0 Introduction

1.1 Background

The San Diego County Water Authority (Water Authority) sustains a population of 3.3 million and a $245 billion economy by providing safe and reliable water supplies to its retail Member Agencies. Since the early 1990s, the Water Authority's Board of Directors has led the effort to diversify its water supply portfolio and improve regional water reliability during droughts and natural disasters. That approach is a national model that aligns with state mandates to reduce reliance on the Sacramento-San Joaquin Bay-Delta, increase water-use efficiency, and develop sustainable new supplies.

Thirty years ago, nearly all of San Diego County's water was supplied by the Metropolitan Water District of Southern California (MWD), placing the region at risk of severe curtailments. Today, the Water Authority's portfolio comprises new local and imported supplies from a variety sources, including the landmark Colorado River Quantification Settlement Agreement (QSA). As it ramps up to full deliveries by 2021, the QSA gives the Water Authority access to approximately 280,000 acre-feet annually from a conservation-and-transfer agreement with the Imperial Irrigation District (IID) and the lining of the All-American and Coachella Canals. These supplies are cost effective and highly reliable due to their high-priority status of Colorado River allocations. Terms of the IID Transfer Agreement and Canal Lining Agreement would secure these supplies for 75 years and 110 years, respectively.

Since the Water Authority does not own pipelines or have other means to directly convey QSA water to the San Diego region, an Exchange Agreement was entered into by the Water Authority to pay MWD to deliver these supplies through its Colorado River Aqueduct system. This agreement expires in 2047 for the transfer water but covers the entire term of the canal lining water to 2112. The initial term of the IID Water Transfer also expires in 2047 with an option to extend through 2077. The decision to extend this supply option is dependent on many factors including cost, reliability, and risk. Since it takes many years to plan, design, permit, and construct large infrastructure projects to provide water, a decision is on the horizon that is equal parts supply and conveyance.

As part of its due diligence to ensure cost effective delivery of these highly reliable supplies, the Water Authority has periodically and incrementally studied new conveyance systems that would be operated independent of MWD's Colorado River Aqueduct. Viable cost savings maybe realized due to MWD's large and growing annual transportation fees that are beyond the Water Authority's control.

1.2 Study Objectives

The primary objective for this study is to assess the technical and economic feasibility of constructing the Regional Conveyance System (RCS) to provide an independent Water Authority-owned and operated system of pipelines, tunnels, canals, pump stations, storage reservoirs, and treatment facilities needed for direct conveyance of the Water Authority's QSA supplies to San Diego County. The scope of this study is intended to build and expand upon past studies to identify and evaluate options for direct conveyance of QSA supplies. In general, feasibility will be determined by comparing the long-term economic viability of this new conveyance system to other options for delivering equivalent volumes of untreated water supplies to San Diego County into the next century. Namely, this entails a comparison to MWD's current distribution pipelines and cost of service projections for wheeling non-MWD supplies through its system. This study is helping inform the decision on supply and conveyance that is coming in 2047.

As an added layer to past studies, potential partnership opportunities compatible with the RCS objectives that yield regional benefits including a reduction in cost or risk to the Water Authority and its member agencies, were identified. However, as part of this report, the baseline cost estimate assumes no partnership participation or associated benefits and funding opportunities. As such, the baseline assumption in Phase A of the study provides that the Water Authority would fund all capital costs and would own and operate the facilities. This assumption would be revisited in Phase B, if authorized. This phasing is further discussed in Section 1.2.2.

1.2.1 Key Determinations

In addition to the primary project objectives, this study would provide more detailed analyses of new facility requirements, including impacts related to integration of RCS flow into the Water Authority aqueduct system. Several of the key determinations to be made from this study regarding RCS supplies and new facility requirements are listed in Table 1-1 below.

TABLE 1-1
Key Determinations

Study Objective	Key Determination
Confirm QSA Supply Projections	Evaluate projections for local supply development and determine if recent trends for regional untreated water demands would continue to support the need for full QSA supply allotments through 2112.
Confirm Conveyance System Sizing Parameters	Evaluate options for sizing of new RCS facilities to meet annual QSA delivery requirements consistent with historical monthly demand variations.
Identify IID Canal System Capacity Constraints	Evaluate both capacity and operational constraints of existing IID-owned and operated canal systems that would be used to convey RCS supplies from the Colorado River to the beginning point of new construction. Confirm the need for new conveyance facilities where insufficient capacity exists.
Identify Aqueduct System Operational Impacts	Evaluate new requirements and/or reoperation of existing aqueduct system components required for the integration of RCS supplies at new delivery points, including connections to Water Authority facilities in Twin Oaks Valley area and in the San Vicente Reservoir area. This includes identifying new aqueduct and in-region storage required to provide reliable levels of service equivalent to existing connections to the MWD system.
Confirm Salinity Treatment Options	Evaluate viability of blending options or new salinity treatment requirements to meet a finished untreated water TDS objective of 500 mg/l. Identify technical and regulatory constraints for brine disposal options.

TABLE 1-1
Key Determinations

Study Objective	Key Determination
Update Power Needs and Power Supply Options	Evaluate power needs based on updated facilities and operations, and identify power supply options for 3A and the treatment plants considering current IID and SDG&E rate schedules, transmission capability, and renewable energy opportunities.
Identify Potential Partnership Opportunities	Evaluate potential partnership and funding opportunities that could enhance the regional benefits of viable alternatives, provide water resiliency, support energy efficiencies, and provide regional environmental benefits.
Identify Environmental and Regulatory Agency Constraints	Provide an updated assessment of the content and steps required to complete the California Environmental Quality Act (CEQA) and National Environmental Policy Act (NEPA) processes, including related consultation and permitting actions. List anticipated environmental issues and potential environmental benefits resulting from the project. Provide a budget estimate and schedule for the future environmental process.
Identify Potential Implementation Risks	Develop a list of potential factors related to project feasibility, cost uncertainty, and operational reliability to assess the relative risk of implementing each alternative.
Update Capital and Operating Cost Impacts	Provide an estimate of capital and operating costs for all facilities associated with Alignment 3A; update previous capital and operating costs for 5A and 5C; and determine if viable RCS options are economically competitive with other water supply options at least through 2112.

1.2.2 Study Approach - Phases A and B

This study provides a more detailed evaluation of three previously identified RCS alternatives, label as Alternative 3A – Northern Alignment, Alternative 5A – Southern Tunnel Alignment, and Alternative 5C – Southern Pipeline Alignment. The approach for completing the evaluations encompasses two phases and assumes that if the alternatives evaluated as part of the initial step are deemed viable, then the two highest-ranking alternatives may move forward to the second phase of the study if authorized. The Water Authority's Board of Directors, Colorado River Work Group, and Member Agency Managers Group were integral in helping to develop this approach and final study scope of work.

- **Phase A –** This initial step consists of evaluations and data collection necessary to describe and compare the three alternatives to the status quo of wheeling QSA water through the MWD system. This phase largely focuses on re-examining Alternative 3A, which was previously evaluated under a 1996 study. Alternatives 5A and 5C, on the other hand, were evaluated under multiple studies, most recently via a *2017 Supplemental Colorado River Conveyance Alternative Report Update (2017 Cost Update)*. Under Phase A, the costs and salinity treatment options for Alternatives 5A and 5C are also updated, allowing for a common basis of assessment and comparison of all alternatives (3A, 5A, and 5C). There is an offramp at the end of Phase A for the Water Authority Board of Directors to determine whether to proceed to the next step, or Phase B. As such, throughout the report, Phase B is labeled as "potential" or "to be authorized".

- **Phase B -** The second step is intended to provide additional project descriptions and information necessary to initiate environmental studies in support of the California Environmental Quality Act (CEQA)/National Environmental Policy Act (NEPA) review process. Under this phase, pipeline alignment drawings would be prepared, and site layout drawings would be developed for the pump stations, hydroelectric facilities and treatment plants. This phase also includes performing constructability reviews, gathering additional geotechnical information, and defining property acquisition requirements. Phase B would also refine, update, and quantify potential partnerships, multi-use projects and funding opportunities based on additional analyses and dialogue with identified potential partners. Phase B cost estimates would be updated to account for costs and benefits associated with such partnerships. There would also be an offramp at the end of Phase B for the Water Authority Board of Directors to determine whether to proceed to the next phase of project implementation, with several planned offramps thereafter leading up to construction.

1.2.3 Planning Horizon

An online date of year 2045 has been assumed for the RCS. This date considers the arduous and complex undertaking to advance the project through the initial studies to determine project feasibility, complete a CEQA/NEPA review process, obtain regulatory approvals, prepare preliminary and final designs, and complete all construction and perform operational testing and acceptance. As the project moves through each phase of development, further challenges may occur from new regulations, consultations with resource agencies, as well as possible ligation delays to resolve potential stakeholder disputes. Implementation schedules would be developed in subsequent project phases, but the main drivers to meet the on-line date are summarized below:

- Preparation of CEQA/NEPA reviews, preliminary designs, and agency consultations — 5 years
- Permitting coordination and approvals, property acquisitions — 2 years
- Technical report preparation and design of all facilities, including pipelines, tunnels, pump stations, and treatment plants — 3 years
- Construction, testing, and project acceptance — 15 years

As can be seen by this long pre-construction (10 years) and construction (15 years) period to achieve an online date of 2045, as compared to the supply and conveyance decisions coming in 2047, it is appropriate to study options now. Detailed schedules for each alternative are provided in the appendices.

1.3 Overview of RCS Alternatives

As stated above, three alternatives are considered under this study phase. The alternatives include similar components, such as: 1) each begins at a common connection to the All-American Canal (AAC); 2) each requires the same level of treatment to reduce finished water salinity; and 3) each provides the same volume of new operational storage in Imperial

County. There are also significant differences between the alternatives, including a unique system of pump stations, canals, pipelines, and tunnels that ultimately connect to the Water Authority's aqueduct system.

The three alternatives can be seen on Figure 1-1 (at the end of this chapter) and are briefly described below, with the primary design and operational characteristics of each alternative summarized in Table 1-2.

TABLE 1-2
Primary Design and Operational Characteristics of RCS Alternatives

Facility Description	Alternative 3A	Alternative 5A	Alternative 5C
Storage Reservoir in Imperial Valley	Size: 900 AF Type: Open, Lined	Size: 900 AF Type: Open, Lined	Size: 900 AF Type: Open, Lined
Canals	Length: 46.7 mi Width (top): 17.75 ft Water Depth: 4.5 ft	Length: 8.8 mi Width (top): 17.75 ft Water Depth: 4.5 ft	Length: 2 mi Width (top): 17.75 ft Water Depth: 4.5 ft
Pipelines	Length: 38.8 mi Diameter: 102 in	Length: 34.8 mi Diameter: 102 in	Length: 81.2 mi Diameter: 102 in
Tunnels	Length: 46.5 miles Diameter: 14 ft	Length: 41.4 miles Diameter: 14 – 15 ft	Length: 11.0 miles Diameter: 12-15 ft
Pump Stations	Number: 3 Flowrate: 396 cfs, 423.5 cfs Size: 12,500 hp, per pump	Number: 2 Flowrate: 396 cfs Size: 14,100 hp, per pump	Number: 5 Flowrate: 396 cfs Size: 14,100 hp, per pump
Hydroelectric Facilities	NA	NA	Number: 3 Size: 20 MW
Treatment Plant	Flowrate: 134 mgd Influent TDS: 600 - 879 mg/l Effluent TDS: 500 mg/l	Flowrate: 134 mgd Influent TDS: 600 - 879 mg/l Effluent TDS: 500 mg/l	Flowrate: 134 mgd Influent TDS: 600 - 879 mg/l Effluent TDS: 500 mg/l
Brine Management	Length: 2.4 mi Diameter: 30 in	Length: 27.5 mi Diameter: 30 in	Length: 31.7 mi Diameter: 30 in
Regulatory Storage in San Diego County	Capacity: 40 MG Type: Covered Tank	Capacity: 40 MG Type: Covered Tank	Capacity: 40 MG Type: Covered Tank
Storage Reservoir in San Diego	Size: 3,500-4,000 AF Type: Open	NA	NA
Aqueduct System Pump Station	NA	Flowrate: 220 cfs Size: 5,000 hp, per pump	Flowrate: 220 cfs Size: 5,000 hp, per pump
Aqueduct System Pipelines	NA	Length: 12.5 mi Diameter: 78 in	Length: 12.5 mi Diameter: 78 in

Alternative 3A – Northern Alignment

Alternative 3A is the northernmost alignment and begins at the westerly terminus of AAC and generally runs parallel to existing canals along the western edge of agricultural lands in Imperial Valley. At the State Route 78/86 junction, the alignment turns west towards Borrego Valley before entering 46-miles of tunnel that exit west of Interstate-15 near Twin Oaks Valley. The alignment terminates at a new regulatory control structure near the Twin Oaks Valley Water Treatment Plant (TOVWTP). In addition to the 46-miles of tunnel, the alignment includes 39 miles of pipeline, three pump stations, and a 3,500 AF open reservoir in north San Diego County. By terminating near the north end of the Aqueduct System at TOVWTP, integration of RCS deliveries allows for gravity flow to existing Member Agency turnouts, in the same manner as deliveries from MWD.

Alternative 5A – Southern Tunnel Alignment

Alternative 5A also runs north beginning at the westerly terminus of the AAC and parallels existing canals on the western edge of agricultural lands to just north of Interstate-8. The alignment then follows County Routes S80 and S2 to the start of 41-miles of tunnel which exit near the San Vicente Reservoir (SVR). This alignment also includes 35-miles of pipelines and two pump stations. The alignment terminates at a connection to the San Vicente Pipeline, allowing use of the SVR for daily and seasonal operations while also having the ability for gravity service to central and south county Member Agencies. Service to the north requires a new pump station and pipeline to convey a portion of the RCS supplies to TOVWTP.

Alternative 5C – Southern Pipeline Alignment

Alternative 5C is the southernmost alternative and begins at the westerly terminus of the AAC where it heads northwest towards Ocotillo and a series of pump stations that lift the water to Jacumba Hot Springs. From here, the alignment runs in a west and northwest direction joining up with Alternative 5A south of the El Capitan Reservoir. Alternatives 5A and 5C share a common alignment from El Capitan to the SVR. Alternative 5C includes 11-miles of tunnel, 81-miles of pipelines, five pump stations and three hydroelectric energy recovery facilities to move the water over the mountains into San Diego County. Like Alternative 5A, this alignment includes a new pump station and pipeline to provide service for north county Member Agencies.

1.4 Report Organization

The information presented in this report is organized into the following report chapters as shown in Table 1-3.

TABLE 1-3
Report Chapters

Chapter	Description
1.0 Introduction	Provides background information on Colorado River supply options, including descriptions of primary project objectives and an overview of the three alternatives considered in the study.
2.0 Regional Conveyance System Operations and Sizing	This chapter provides detailed descriptions of the canals, pipelines, tunnels, pump stations, and storage reservoirs that comprise Alternatives 3A, 5A and 5C. Facility sizing criteria is also detailed.
3.0 Aqueduct Operations and Integration of the RCS	This chapter evaluates potential impacts of RCS deliveries on the existing aqueduct system, including an evaluation of monthly supply imbalances and an assessment of new facility requirements based on RCS delivery points in the Twin Oaks Valley and the San Vicente Reservoir areas.

TABLE 1-3
Report Chapters

Chapter	Description
4.0 Treatment, Blending and Brine Management Options	This chapter builds on previous strategies to evaluate treatment and brine management alternatives to meet a target TDS goal of 500 mg/L. Project updates include a blending analysis, comparison of treatment options, plant layouts, and an evaluation of brine management options and permitting considerations associated with conveying effluent to the Salton Sea.
5.0 Power Supply Alternatives	This chapter describes the power needs of each RCS facility, identifies existing and proposed utility power supply alternatives, reviews the utility rate schedules, and provides an overview of renewable energy alternatives for the project.
6.0 Risk, Cost Opinions and Economic Comparison	This chapter provides a qualitative risk analysis and describes the risks factors that were determined to impact overall project feasibility, the uncertainty associated with cost development, and operational risks. Capital and annual operations, maintenance, and replacement cost development and a summary of the final cost opinions for each of the major facility components are presented. An economic model developed by the Water Authority is used to evaluate economic returns for various options including the status quo, RCS implementation, reliance on MWD, and local supply development.
7.0 Environmental Review and Permitting	This chapter provides an updated assessment of the content and steps required to perform a CEQA and NEPA review process, including a description of related consultation and permitting actions. Lists of anticipated environmental issues and potential environmental benefits are presented along with a budget estimate and schedule of the environmental process.
8.0 Partnership Funding and Opportunities	This chapter presents potential partnership and funding opportunities that could enhance the regional benefits of viable alternatives. Partnerships that could attract Federal, State and local funding addressing environmental, disadvantaged communities, and water resiliency are highlighted.
9.0 Screening Criteria and Evaluation	This chapter presents the initial or coarse screening technical analysis used to compare and rank the three alternatives, considering the information and evaluations performed in Phase A. Documentation is provided regarding evaluation goals and coarse screening results. A sensitivity analysis is used to confirm screening results.
10.0 Conclusions	This chapter summarizes key findings and results from the Phase A evaluations, and provides recommendations for potential next steps.

1.5 Previous Studies

Several Water Authority reports have been prepared analyzing various aspects of a direct conveyance system from the Colorado River to San Diego and Mexico over the last 24 years. A summary of these reports and their focus is provided as background information in Table 1-4.

TABLE 1-4
Prior Studies of Colorado River Conveyance Options

Year	Study Title	Description
1996	Water Transfer Study - Feasibility Level Engineering for Facilities to Transfer Water from The Imperial Irrigation District (1996 Water	This study analyzed a new direct conveyance system for Colorado River supplies to meet projected 2100-year demands. It included five alternative corridors and three annual transfer volumes. It assessed land use, geology, energy management, water quality, corridor engineering, electric power markets, natural gas markets,

TABLE 1-4
Prior Studies of Colorado River Conveyance Options

Year	Study Title	Description
	Transfer Study)	environmental, construction costs, and decision analysis. It provided 300,000 AF and 500,000 transfer volumes.
2001	Feasibility Study Cost Refinement	This study purpose was to refine costs for Alignments 5A and 5C using new information from 2001 Geotechnical Data Reports. Other changes included refining tunnel and pipeline details and updating the implementation schedule. It considered 300,000 AF and 500,000 AF transfer volumes.
2002	Regional Colorado River Conveyance Feasibility Study Final Report	This report acknowledged, evaluated and recorded jointly beneficial opportunities for binational cooperation between San Diego and Tijuana metropolitan areas on the development of new water conveyance facilities. These facilities would deliver agricultural water from Imperial and Mexicali valleys to the San Diego and Tijuana metropolitan areas.
2013	Regional Water Facilities Optimization and Master Plan Update (2013 Master Plan Update)	Appendix G of this study added changes for Alignments 5A and 5C, including updates to land use, easements, and jurisdictional boundaries. System hydraulics were revised to include the San Vicente Dam Raise and AAC Relining. Flows were revised to 280,200 AFY. Costs were escalated to current dollars for the evaluation.
2017	2017 Supplemental Colorado River Conveyance Alternative Report Update (2017 Cost Update)	This study re-evaluated Alignments 5A and 5C to provide an annual delivery of 400,000 AF and was built upon the 2013 Regional Water Facilities Master Plan, updating facility sizes, alignments, capital costs, and operation and maintenance costs.

1.6 Independent Cost Estimate Review

An independent party, Hunter Pacific Group (HPG), was retained by the Water Authority to conduct a third-party review of Black & Veatch's initial cost information. HPG reviewed the cost assumptions and provided independent cost estimates for each of the alternatives for the RCS. HPG also reviewed operations, maintenance, and replacement costs for all alternatives, as well as the project schedule and risk register.

The HPG cost estimates were all higher than Black & Veatch's draft estimates. Based on the independent review and as part of the further project development, the draft estimates were updated, with Black & Veatch agreeing to some of the findings by independent reviewer, but not all of the findings. For the revised estimates included in this report, HPG agreed with Black & Veatch's operations, maintenance, and replacement costs. HPG also agreed with Black & Veatch's estimated facility life expectancies and construction schedules. HPG's agreed with most aspects of Black & Veatch's Risk Register and suggested additional risk assessments be performed if the program advances to new phases.

1.7 Report Conclusions and Next Steps

As can be seen in the following chapters, Phase A of this study provides additional information necessary to further refine facility requirements and answer several key questions regarding the viability of an independent Water Authority owned and operated system to convey its QSA supplies directly to San Diego County. In addition to developing a more refined description of the RCS, this study provides a comparison of other water supply options including the status quo of paying MWD for wheeling the Water Authority's QSA supplies through its system, reliance on MWD for imported water supplies, and development of new local supplies to become less reliant on MWD. The results of these comparisons would support both near- and long-terms critical decision-making by the Water Authority's Board of Directors related to extending QSA-related agreements.

A key component of that decision-making is recognition that the RCS could improve the region's water resiliency by assuring continuation of the highly-reliable QSA supplies for the foreseeable future. Should the results and comparisons presented in this report demonstrate project viability, and the Board of Directors chooses to advance the program, the next step would be to proceed with the Phase B study effort that would advance the project for initiation of CEQA/NEPA reviews. Phase B would also provide the following information:

- **Updated Water Supply Demands** – The 2020 Urban Water Management Plan would be used to update future water demands and compare that to the current QSA supply projections. This will inform long-term negotiations related to the QSA.
- **Further Refinement of Facility Layouts** – Further definition of pipelines, tunnels and other facility layouts would refine cost estimating and lower the risk of cost escalation.
- **Refined Cost Comparisons** – Based on further facility refinements, the cost estimates would be updated and used to update the economic considerations of RCS supply options.
- **Partnership Discussions** – During Phase A, most identified partnership opportunities were not discussed with the potential partners. During Phase B, many of the identified potential partnerships would be explored with the respective parties.
- **Quantitative Risk Assessment** – Furthering the Risk Register and applying quantitative risk mitigation would be applied, resulting in risk cost impacts addressed in the economic analysis.
- **Economic Sensitivity Analysis** – Only limited sensitivity analyses were applied to the economic analysis as part of Phase A. Additional sensitives would be assessed as part of Phase B to develop an envelope of potential economic outcomes for the RCS alternatives, MWD alternatives, and local supply development options. These refinements would include incorporating funding opportunities and benefits resulting from the partnership discussions.

As part of Phase B, the Water Authority would also engage other consultants to assist with providing financial advice and legal review of QSA agreements and any required new agreements, including those pertaining to partnerships.

It is also important to recognize the important role Water Authority staff played in the development of this report. While Black & Veatch led the development of the report and performed the engineering, technical, and cost development, Water Authority staff provided guidance related to aqueduct operations, demand forecasting, partnership possibilities, agency coordination, and other key items that would result in a thorough and comprehensive assessment of project viability.

Focused workshops were led by Black & Veatch for various topics such as partnerships, aqueduct operations and integration, treatment, power supply, and operations and maintenance needs. This critical collaboration provided the framework for informed decisions that led to project efficiencies and set the stage for the study development. Capital, O&M, and replacement cost estimates were developed independently by Black & Veatch and then reviewed by the independent reviewer. Water Authority staff developed an economic model and performed analyses utilizing the cost information provided by Black & Veatch.

The following Figure 1-1 provides an overview of the RCS alternatives that will be referenced throughout this study.

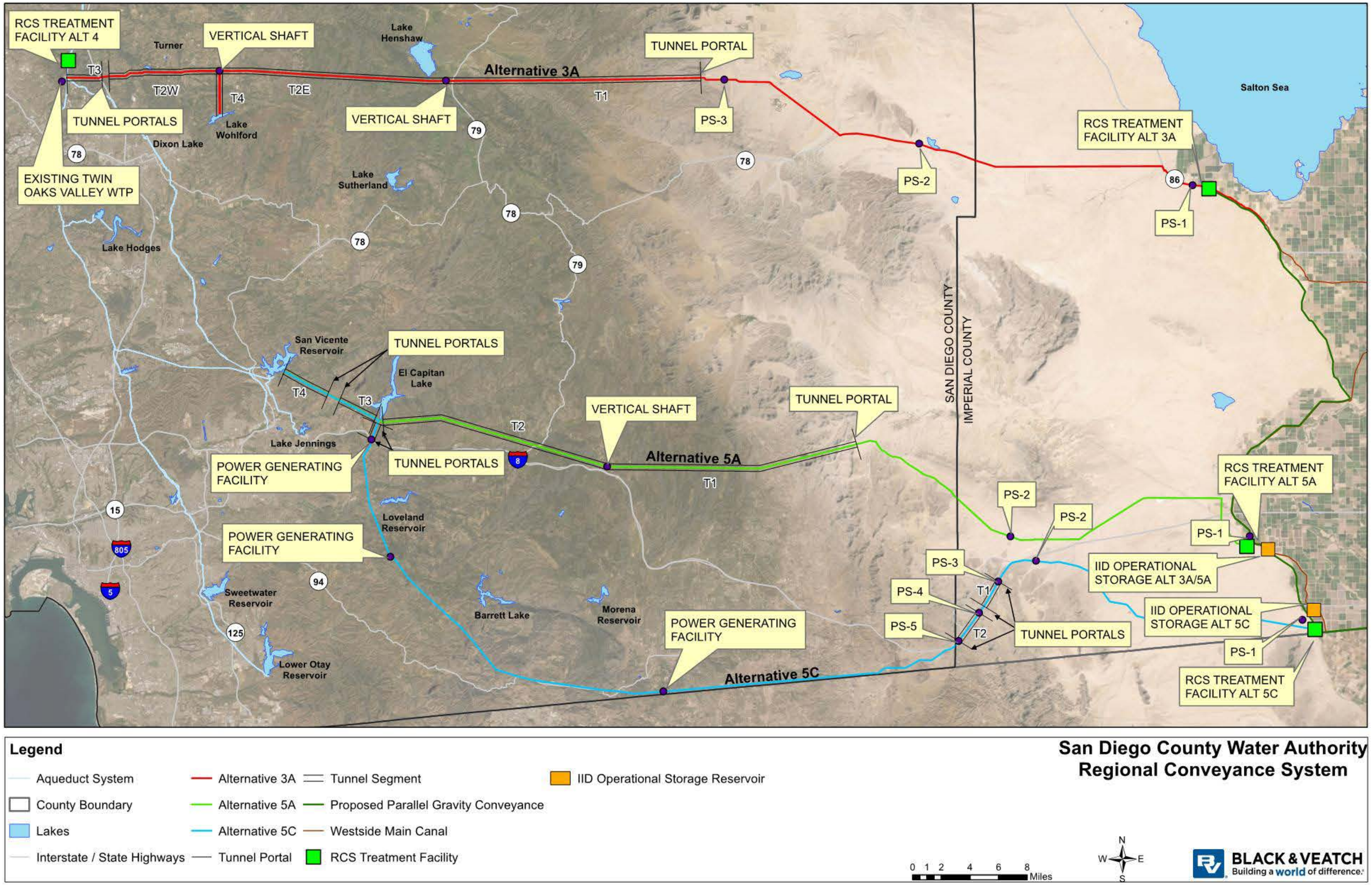

FIGURE 1-1
RCS Alternatives Overview Map

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2

REGIONAL CONVEYANCE SYSTEM OPERATIONS AND SIZING

Chapter 2.0 Regional Conveyance System Operations and Sizing

2.1 Introduction

2.1.1 Overview

This chapter describes the key facility components required for the conveyance of Water Authority's QSA water supplies to San Diego. It also describes some of the potential partnership opportunities for joint-use of certain facilities that could provide mutual benefits. These joint-use facilities either (1) would already be required to make the RCS system work and therefore the costs are accounted for in the baseline RCS costs estimates or (2) would not be needed to make the RCS work and costs are not included in the Phase A baseline cost estimates. The benefits resulting from these or any other potential partnerships discussed in this report have not been quantified or included in the baseline RCS cost estimates due to the preliminary nature of the evaluation and limited dialogue with potential partners.

Each of the alternatives considered in this assessment – Alternatives 3A, 5A, and 5C – were previously studied to varying levels, beginning with the 1996 Water Transfer Study. Alternatives 5A and 5C were evaluated in greater detail during a series of follow up studies, which most recently included the *2013 Master Plan Update* and the *2017 Cost Update*. These studies further defined facility requirements and overall project costs. As such, the initial focus of this study was to define facility requirements for Alternative 3A, allowing for a common basis of assessment and comparison of all alternatives (3A, 5A, and 5C).

Table 2-1 below summarizes key project components and characteristics associated with the three alternatives, which includes application of updated hydraulic parameters, as described later in this chapter.

TABLE 2-1
Alternative Characteristics

Characteristic	Alternative 3A	Alternative 5A	Alternative 5C
Minimum Elevation, feet above mean sea level (MSL)	-218	-5	-35
Maximum Elevation, feet (MSL)	1,140	1,150	4,050
Total Pumping Head, feet	1,980	1,555(1)(2)	4,225(1)(2)
Total Hydroelectric Head, feet(3)	0	0	2,350
Gravity Flow Canals, miles	46.7	8.8	1.5
Pressure Pipelines, miles	38.8	34.8	81.2
Pressure Tunnels, miles	46.5	41.4	10.6
Overall Alignment Length, miles	132.0	85.0	93.3
Pump Stations, each	3	2(2)	5(2)
Power Generating Facilities, each	0(3)	0	3

TABLE 2-1
Alternative Characteristics

Characteristic	Alternative 3A	Alternative 5A	Alternative 5C
Pressure Control Facilities, each	1	1	1
Forebays, capacity	3 (40 acre feet [AF] each)	2 (40 AF each)	5 (40 AF each)
Afterbays, capacity	0	0	3 (40 AF each)
RCS Terminal Storage, capacity	3,500 – 4,000 AF	-	-
AAC Operational Storage, capacity(4)	900 AF(4)	900 AF(4)	900 AF(4)

Notes:

1. The total pumping head and number of pumping stations do not include the improvements required to convey water from the San Vicente Reservoir (SVR) north to the Twin Oaks Valley Water Treatment Plant (TOVWTP), as defined in *Chapter 3.0 – Aqueduct Operations and Integration the RCS*, which would add an additional pump station with 490 feet of total pumping head to Alternatives 5A and 5C.
2. The *2013 Master Plan Update* assumed a common pump station lift of 800 feet for each station. This study has carried forward this assumption for this initial phase. Phase B, should it be authorized, would include a more detailed evaluation of Alternatives 5A and 5C, including their pump station assumptions.
3. Opportunities for micro-hydro could exist at canal drop structures and pipeline turnout facilities. These opportunities should be further evaluated during subsequent phases of design.
4. Potential joint-use storage facility with IID to allow for QSA deliveries through the existing AAC.

2.1.2 Chapter Organization

Key operating parameters and project components affecting alignment decisions for the Regional Conveyance System (RCS) are summarized below and discussed in the following chapter sections.

RCS Deliveries – This section discusses the volumes of untreated water that would be conveyed based on QSA water supplies; initial assumptions to convey additional flows that would address potential partnership opportunities; and net conveyance from post-salinity treatment options.

Gravity Flow Conveyance (Canal) Facilities – This section reviews capacity constraints of the IID-operated canal system and identifies parallel gravity flow facility options to convey RCS flows from the AAC to a location for salinity treatment prior to the start of pressurized flow.

Pipelines and Tunnels - This section describes the system of pressurized pipelines and tunnels for Alternative 3A, including design criteria applicable to pipeline sizing. Similar descriptions for Alternatives 5A and 5C would be addressed in Phase B of this study, if those alternatives move forward and if Phase B is authorized. In-region storage options related to reliable operation of the RCS are also discussed.

Pump Stations – This section provides a summary description of the pump stations associated with Alternative 3A.

2.1.3 Summary of RCS Operations

The purpose of the RCS is to provide direct conveyance of QSA water supplies to the Water Authority's service area. RCS flows would be delivered using a combination of existing canals and newly constructed conveyance facilities. The new conveyance system would consist of gravity flow canals and a pressurized system of pipelines and tunnels extending from the IID's existing canal system and ending at new connections to the Water Authority's aqueduct facilities at either the Twin Oaks Diversion Structure (TOVDS) (Alternative 3A) or the SVR (Alternatives 5A, 5C). At this time, it is assumed that the Water Authority would own and operate the new gravity flow canals or that they would be jointly owned and operated with IID. However, further coordination is required with IID during potential subsequent phases of work to confirm. Most of the remaining newly constructed RCS facilities are expected to be owned, operated, and maintained under Water Authority jurisdiction, unless noted otherwise.

To ascertain the extent of new RCS facilities needed for the safe and reliable delivery of QSA water supplies, as well as to gain an understanding of the impacts that the RCS would have on operation of existing conveyance facilities within Imperial and San Diego Counties, a series of meetings were held with staff from both IID and the Water Authority. Key takeaways from these meetings related to RCS facility requirements are summarized in Table 2-2 below.

TABLE 2-2
IID and SDCWA Operational Considerations

Agency	Operational Considerations
IID	There is sufficient capacity in the AAC upstream of the New River Siphon for RCS flows.
	Between the New River Siphon and its westerly terminus with the Westside Main Canal (WSM), the AAC capacity is limited to 1,800 cubic feet per second (cfs). This capacity is largely used to meet current peak irrigation demands; however, capacity for RCS flows appears to be available during lower demands months and off-peak irrigation hours. IID provided flow data for the existing canal system for the period between 2010 and 2019, which is included as Appendix A.
	WSM maximum daily flows run near rated capacity (1,200 cfs) for several months each year. Capacity north of Fillaree Check is insufficient for RCS flows.
	12-hour irrigation schedules for AAC and WSM require a 100 cfs surcharge over actual irrigation orders, further affecting available canal capacity. Peak vs. non-peak 12-hour orders typically vary between 100 and 200 cfs.
	Operational reservoir storage is planned near the AAC/East Highland Canal (2,900 AF) to rebalance system-wide 12-hour irrigation.
	A smaller operational reservoir is planned along the WSM at Foxglove Check (600 AF) and/or at Trifolium 8 (300 AF) to rebalance westside 12-hour irrigation.
	Reservoir operations to balance 12-hour irrigation schedules and return flows should be compatible with storage needs required for daily operation of the RCS.
Water Authority	Terminal storage downstream of the RCS pump stations would be needed to meet Aqueduct System reliability requirements. Terminal storage should be sized to provide approximately 5 days of untreated water demand at maximum flow conditions.
	The delivery gradient for Alternative 3A at Twin Oaks should be 1,140 feet to meet Crossover Pipeline operations.
	The delivery gradient for Alternatives 5A, 5C at San Vicente should be 922 feet to minimize operations of the San Vicente Pump Station.
	RCS sizing should be based on conveying design flows plus a 10% surcharge for planned and unplanned system downtimes.

All alternatives are proposed to begin with a new connection to IID's existing canal system. QSA water supplies would be ordered daily via IID's Water Dispatching Unit, combined with daily agricultural orders, and conveyed through the AAC from Imperial Dam. As reported by IID, the AAC upstream of the New River Siphon, covering an approximate length of 72.2 miles, has sufficient capacity for the additional RCS flows.

From the New River Siphon to the westerly terminus of the AAC at its junction with the WSM, available canal capacity typically drops below the level required for a constant daily delivery of RCS flows. This study evaluated two options to address the capacity constraint in this lower reach of the AAC: 1) a new parallel gravity flow system or 2) coordinated operations during non-peak hours coupled with a new storage reservoir. Details of this assessment are documented in Section 2.3.

Downstream of the AAC, IID has indicated that significant portions of the WSM are currently operated at or near capacity for several months each year. Reoperation of WSM deliveries would not free significant capacity and as a result a new concrete lined canal running parallel to the WSM, Thistle, and Trifolium Extension Canals is proposed from the westerly terminus of the AAC to the beginning point of pressurized pipeline flow for each alternative.

Within this reach of new parallel canal, a new storage reservoir would be constructed to assure consistent deliveries of QSA supplies via the RCS. The new storage reservoir could be jointly operated with IID and sized to provide upstream storage for the proposed treatment facility while supporting IID's dispatching requirements for westside irrigators. The shared reservoir could provide operational advantages to the Water Authority and IID as discussed in *Chapter 8.0 - Partnership and Funding Opportunities*.

Discussions on alternative treatment options are defined in *Chapter 4.0 - Treatment, Blending and Brine Management Options*. For purposes of this chapter, treatment discussions are based around a membrane filtration (MF), which consists of either ultra-filtration or micro-filtration, followed by reverse osmosis (RO). The treatment is also assumed to include a brine minimization step, such as High Recovery Reverse Osmosis (HRRO).

Downstream of the proposed MF/RO treatment plant, QSA flows would be delivered through a combination of pressurized pipelines, tunnels, booster pump stations, in-line hydroelectric energy recovery facilities, and pressure control facilities. Minimum sizing requirements for the pipelines, tunnels and booster pump stations are to deliver QSA flows, flows for potential partners that could us the RCS for transportation of their own supplies, plus a ten percent surcharge to account for system downtimes and planned maintenance. Sizing the facilities with a ten percent surcharge would also allow for seasonal variations in demand, thereby reducing in-region storage needs during low demand winter months. Pump station and treatment forebays are sized to provide 60 minutes of operational storage.

To assure reliability requirements are achieved for RCS operations, terminal storage downstream of the RCS pump stations would be needed. Terminal storage would be sized to provide a minimum of 5 days at peak flow delivery. The terminal storage affords a buffer for unplanned outages, allowing system operators to either start-up/operate in-region emergency storage facilities or schedule and ramp-up supplemental untreated water orders from Metropolitan Water District of Southern California (MWD). For Alternatives 5A and 5C, terminal storage would be provided by the SVR. For Alternative 3A, terminal storage is assumed to be provided at Lake Wohlford, but other new surface water reservoir options are considered as described in Section 2.4.8.

At the westerly terminus of the RCS, the delivery gradient for Alternative 3A at Twin Oaks would be 1,140 feet. This elevation matches current Rejection Tower operating mode at the TOVDS and allows for operation of the Crossover Pipeline at its maximum delivery gradient. For Alternatives 5A and 5C the delivery gradient at San Vicente would be 922 feet, which matches the spill elevation for the San Vicente Surge Control Facility and allows bypassing of the San Vicente Pump Station for deliveries to the Second Aqueduct.

Figure 2-1 presents a general configuration of the RCS alternatives being considered. Refer to Figure 2-4 for additional detail on the existing canals and proposed gravity systems.

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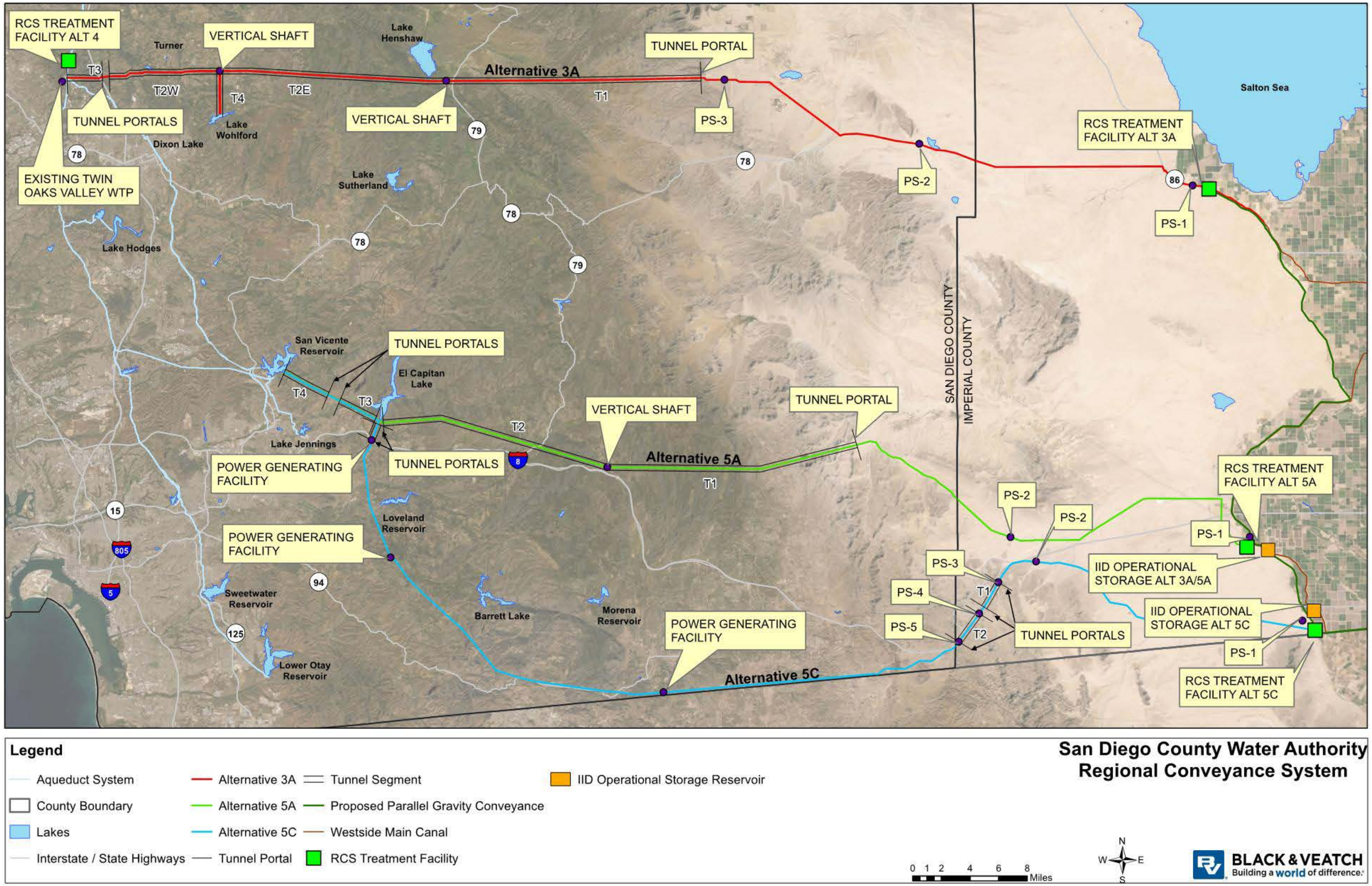

FIGURE 2-1
RCS Alternatives Overview Map

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2.2 RCS Deliveries

This section documents the various supplies that the RCS would need to convey. These supplies form the basis for the sizing of the facilities and include both the Water Authority's QSA supplies and flows required for potential partnership opportunities. The RCS facilities would be sized to convey these supplies less losses associated with salinity treatment.

2.2.1 QSA Water Supplies

The Water Authority's QSA water supplies are comprised of the conserved water transfer with IID and the All-American and Coachella Canal Lining projects, of 200,000 AF/Year and 79,500/AF/YR, respectively.

2.2.2 Partnership Supplies

The RCS would be sized to accommodate conveyance of additional supplies from potential partners to a point of delivery near the potential partner. For purposes of sizing RCS facilities, this study has assumed a partner provided supply of 20,000 AF/yr (post-treatment) would be delivered to the Borrego Springs area for use by a potential partner.

Table 2-3 summarizes RCS supply development as it relates to facility sizing. Additional partnership opportunities and conveyance impacts are addressed in *Chapter 8.0 – Partnership and Funding Opportunities*.

TABLE 2-3
RCS Deliveries Summary

Supply Source	AF/yr	MGD (cfs)	Comments
QSA Supplies	279,500	250 (386)	---
Partner-Provided Supply	22,000	20 (30)	Allowance for transportation of supplies provided by potential partners to achieve 20,000 AF/yr post-treatment
Total	**301,500**	**269 (416)**	Feed Water Supply for MF/RO Treatment

2.2.3 Salinity Treatment

Salinity treatment refers to the application of treatment processes to achieve a finished water salinity concentration not exceeding 500 mg/l. This study includes an evaluation on treatment technologies and recovery rates based on anticipated water quality which results in a finished water supply of 278,700 AF/yr without a brine minimization treatment step. Details on this evaluation are presented in *Chapter 4.0 - Treatment, Blending and Brine Management Options* and are based on a process flow involving membrane filtration (MF), reverse osmosis (RO), treatment bypass stream, and a brine minimization system, such as HRRO.

The treatment facility has been designed to produce 278,700 AF/yr based on a conservative influent total dissolved solids (TDS) content. However, during periods of the year it is anticipated that the influent TDS would be lower and that more flow could bypass the treatment processes. During these periods it would be possible to exceed 278,700 AF/yr of finished water at the target concentration of 500 mg/l.

2.2.4 RCS Conveyance Flows

Each conveyance component of the RCS would be sized to deliver QSA supplies and, as appropriate, additional supplies resulting from partnership opportunities. A 10 percent flow surcharge would be added for conveyance sizing to account for system downtimes. The 10 percent surcharge better assures annual QSA deliveries could be achieved should a planned or unplanned outage occur during any given year. The increased conveyance capacity would also improve operational flexibility to meet seasonal variations in demand, thereby reducing in-region storage needs during low demand winter months.

As summarized in Table 2-4, flow requirements for sizing of the RCS is divided into three separate reaches.

TABLE 2-4
Summary of RCS Conveyance Flow Requirements

RCS Conveyance Limits	Supply Capacity (AF/yr)	Design Capacity (AF/yr)	Impacted Facilities
AAC to the Treatment Facility (Q_1)	301,500	331,700$^{(1)}$	Design Flow Q_1: Gravity Flow Systems Parallel to AAC and WSM Canal
Treatment Facility Finish Water to Borrego Springs (Q_2)$^{(2)}$	278,700	306,570$^{(1)}$	Design Flow Q_2: Pressurized Pipelines, PS-1, PS-2
Borrego Springs to Twin Oaks (Q_3)	258,700	286,570$^{(1)}$	Design Flow Q_3: Pressurized Pipelines, Tunnels, PS-3

Notes:

1. Includes 10% allowance for system shutdowns.
2. See Chapter 4 for details on salinity treatment and recovery rates. Values shown do not include a brine minimization treatment step.

2.3 Gravity Flow Facilities

As discussed previously, discussions with IID have indicated that sections of their existing canal system are not anticipated to have available capacity to meet the constant daily deliveries required for RCS flows.

This section documents the completion of the following tasks:

- Evaluation of the capacity of the existing canal system (the AAC and the WSM) based on physical characteristics and current operating practices to identify locations without capacity for constant daily deliveries of RCS flows
- Identification of solutions for locations where the capacity within the existing canal system would not be available, such as new gravity conveyance facilities or storage reservoirs

IID provided the average maximum daily flow rates for each month for its existing system from 2010 to 2019 for use in this evaluation. 2010, 2011, and 2012 were the peak use years in the last decade. Since then, IID's water allotment has dropped due, in part, to farmland near the terminus of the AAC having been converted to solar use. As such, IID indicated that the 2013-2019 flow data was better indicative of future flows and, therefore, serves as the basis of this evaluation. The historical flow data provided by IID is included in Appendix A.

2.3.1 Existing Canal Facilities

Water from the Colorado River is diverted at the Imperial Dam through desilting headworks and into the AAC for delivery to IID, the Coachella Valley Water District (CVWD) and the Yuma Irrigation District (YID). Approximately 14 miles downstream from the headworks, YID's water is diverted from the AAC at the Siphon Drop. Approximately 36 miles downstream from the headworks, CVWD's water is diverted at Drop No. 1 into the Coachella Branch of the AAC. All water conveyed past Drop No.1 is designated for use by IID, with the AAC being the sole conveyance facility of Colorado River water into the IID service area. Following a series of drop structures and hydroelectric power plants, water is conveyed from the AAC throughout the IID service area utilizing three main canals: East Highline, Central Main, and WSM. For purposes of this evaluation, the AAC and WSM were considered for gravity conveyance to the downstream delivery point.

2.3.2 Use of the All-American Canal

An evaluation of available capacity within the AAC was performed by reviewing the latest design capacity and flow data provided by IID.

Existing Canal Capacity. First, the capacity of the existing canals was checked based on the latest data provided by IID, which confirmed that there is enough available capacity in the AAC upstream of the New River Siphon. However, downstream of the New River Siphon, the latest information from IID indicates that the capacity is limited to 1,800 cfs, representing a significant reduction in canal capacity from previous reports. Conversations with IID confirmed that the limited capacity of 1,800 cfs has been consistent over 40 plus years of operation and is not a result of silt accumulation or the erosion of the canal cross-sectional area.

Table 2-5 summarizes the available capacities for each reach of the AAC based on existing capacities and the maximum daily flows and, as shown, the maximum single day flow downstream of the New River Siphon is equal to the canal's capacity.

TABLE 2-5
Capacity of All-American Canal

Canal Reach	Canal Capacity (cfs)	Maximum Daily Flow (cfs)	Available Capacity (cfs)
Passing Drop No.1	7,600	6,300	1,300
Passing Drop No. 4 to East Highline Canal	6,800	6,000	800
East Highline Canal to Drop No. 5	5,060	3,700	1,360
Passing Drop No. 5 to Central Main Canal	3,700	3,000	700

TABLE 2-5
Capacity of All-American Canal

Canal Reach	Canal Capacity (cfs)	Maximum Daily Flow (cfs)	Available Capacity (cfs)
Passing Central Main Check to New River Siphon	2,800	2,000	800
Passing New River Siphon to Westside Main Canal	1,800	1,800	0

Review of Current Operational Flow Data. To further assess the restriction starting at the New River Siphon, additional flow data was requested from and provided by IID such that the average monthly mean and maximum daily flows could be evaluated to determine if operational modifications, including avoidance of peak 12-hour irrigation periods, could result in sufficient capacity to meet annual RCS delivery requirements. According to IID, deliveries to the WSM operate under two 12-hour water irrigation cycles with flow adjustments made at 8 AM and 8 PM. The “on-peak” 12-hour irrigation cycle flow rate can be up to 200 cfs greater than the “off-peak” 12-hour flow rate. Table 2-6 presents the average monthly on- and off-peak daily flows for the AAC downstream of the New River Siphon from 2013-2019, which, according to IID, is mostly reserved for irrigation deliveries to the WSM. The on-peak average daily flow for each month was provided by IID, while the off-peak average daily flow was assumed to be 200 cfs less than the on-peak 12-hour flow rate.

TABLE 2-6
Average On- and Off-Peak Daily Flow per Month Downstream of the New River Siphon (2013-2019)

Month	Off-Peak Average Daily Flow (cfs)	On-Peak Average Daily Flow (cfs)
January	556	756
February	951	1,051
March	1,148	1,348
April	1,115	1,315
May	1,191	1,391
June	1,128	1,328
July	1,130	1,330
August	1,056	1,256
September	907	1,107
October	846	1,046
November	714	914
December	605	805

IID noted that the WSM requires an additional 100 cfs of operational surplus (extra water) over the actual demand shown on Table 2-6. This extra water is used to balance daily variations in irrigation orders, with the extra water eventually being moved to available storage and subsequently returned for irrigation flows later.

Check of Available Capacity for RCS Flows. Based on operational requirements, and a required RCS delivery rate of approximately 450 cfs (which is equivalent to 331,700 AF/yr per line 1 of Table 2-4), conveyance of RCS flows would be restricted when the maximum daily flows in the AAC exceed a certain threshold. In order to ensure that the constant delivery of 450 cfs is available to meet RCS demands, the maximum daily flow for irrigation must be less than 1,250 cfs. As shown on Table 2-7, this leaves 450 cfs available for RCS deliveries.

TABLE 2-7
Available AAC Capacity

Calculation of Available AAC Capacity	cfs	AF/day
AAC capacity	1,800	3,570
AAC capacity less 100 cfs operational buffer	1,700	3,372
Total daily RCS deliveries	450	905
Remaining AAC capacity for normal deliveries	1,250	2,480

Per Table 2-6, sufficient capacity to convey RCS flows at all times would occur in six months. To meet demands in the remaining six months, two potential solutions were identified: 1) build a parallel canal for this section of the AAC to meet the daily RCS demands, or 2) build a new storage reservoir downstream of the westerly terminus of the AAC to store water when excess capacity is available (during non-peak hours or off peak months) and use it when capacity is unavailable to meet the full 450 cfs demand. This section documents the analysis completed to size a reservoir that would provide for constant deliveries of RCS flows. The evaluation conducted to size a parallel gravity conveyance system to provide the additional capacity required for constant deliveries of RCS flows is documented in Section 2.3.4.

AAC Operational Storage Reservoir Sizing. The new operational storage reservoir was sized for two purposes, 1) to balance on- and off-peak daily flow periods and 2) to provide sufficient storage to meet the constant RCS demands when capacity is unavailable in the AAC. Table 2-8 compares the average on- and off-peak flows in the AAC to the overall capacity to determine the surplus and shortfall each month.

TABLE 2-8
Summary of Available AAC Canal Capacity and Storage Requirements

Month	Average Off-Peak Flow (AF)	Average On-Peak Flow (AF)	RCS Flows (AF/day)	AAC Daily Capacity (AF/day)	Daily Delta (AF)	Monthly Delta (AF)
Jan	551	750	905	3,372	1,166	36,147
Feb	844	1,042	905	3,372	581	16,265
Mar	1,139	1,337	905	3,372	-8	(254)
Apr	1,106	1,304	905	3,372	57	1,718
May	1,181	1,380	905	3,372	-93	(2,898)
Jun	1,119	1,317	905	3,372	31	945

TABLE 2-8
Summary of Available AAC Canal Capacity and Storage Requirements

Month	Average Off-Peak Flow (AF)	Average On-Peak Flow (AF)	RCS Flows (AF/day)	AAC Daily Capacity (AF/day)	Daily Delta (AF)	Monthly Delta (AF)
Jul	1,121	1,319	905	3,372	28	853
Aug	1,047	1,246	905	3,372	174	5,403
Sep	900	1,098	905	3,372	470	14,095
Oct	839	1,037	905	3,372	591	18,316
Nov	708	906	905	3,372	853	25,579
Dec	600	798	905	3,372	1,069	33,134

Figure 2-2 presents the results of Table 2-8 in chart form for the daily capacity by month.

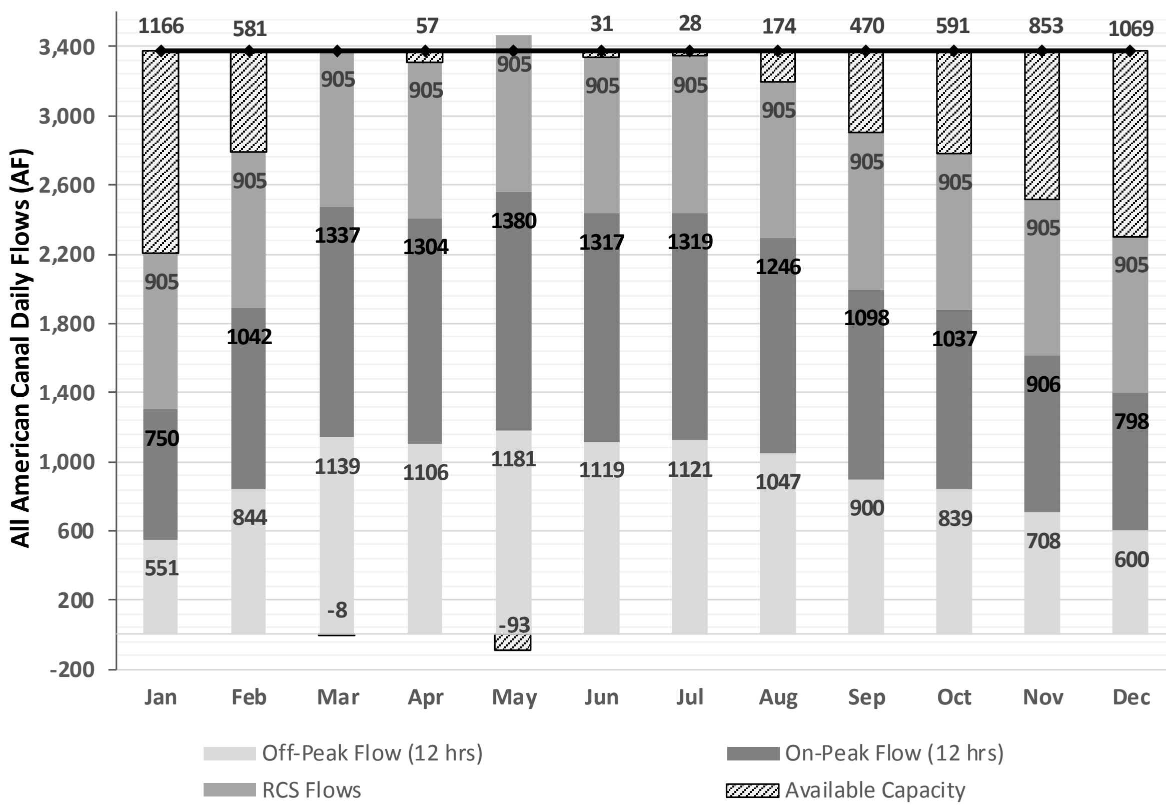

FIGURE 2-2
Summary of Available Daily Capacity in the AAC for RCS Flows by Month

As can be seen, a storage volume of approximately 2,900 AF would be needed to balance deliveries for the peak month of May, while a lesser storage volume would be needed for March. In the remaining ten months, the AAC has a surplus of capacity.

While this storage volume is required to be located downstream of the terminus of the AAC, it does not need to be located in a single location or even entirely within Imperial County. In the peak month of May, there is a shortfall of 93 AF per day of capacity within the AAC. A 900 AF storage reservoir near the terminus of the AAC would provide for approximately ten days of shortfall. The remaining 2,000 AF of storage could then be provided in San Diego County, such as at the SVR. During months when the Water Authority's demands are less than could be supplied by the RCS (i.e., December through March), the excess water could be stored in San Diego County to meet 2,000 AF shortfall in May. As the system is being designed to convey ten percent more than the QSA supply, there is excess capacity to deliver the water during non-peak months. *Chapter 3.0 – Aqueduct Operations and Integration of the RCS* provides greater detail on the Water Authority's demands and storage opportunities in San Diego County.

The 900 AF storage reservoir in Imperial County would have approximately a 200 acre footprint.

While this evaluation only considers RCS deliveries, as noted in Table 2-2, IID is considering a 600 AF storage reservoir near Foxglove Check to rebalance 12-hour irrigation flows. This storage reservoir could provide opportunities for shared operational benefits with IID. Further evaluation would be required to determine if additional storage beyond 900 AF would be required to meet those operational requirements.

It should be noted that this evaluation assumed the off-peak 12-hour flows in the AAC were 200 cfs less than the on-peak 12-hour flow data provided by IID (Table 2-6 and Table 2-8). These assumptions were reviewed with IID. More detailed evaluations using additional flow information should be completed in subsequent phases of work to validate the operational reservoir sizing.

The operational storage reservoir is anticipated to have a water depth of approximately five feet to match the elevation change in the existing canal system at the Foxglove Check, or at another check, with an additional two feet of freeboard above the water surface elevation. The reservoir would be earthen with a plastic liner, such as a high-density polyethylene (HDPE).

In summary, it appears feasible to convey RCS flows through the existing AAC, provided that 2,900 AF of storage is provided downstream of the westerly terminus by storing water during non-peak hours and using it during peak-hours. The storage could be split such that 900 AF is provided in Imperial County (near the terminus of the AAC) and the remaining 2,000 AF is provided in San Diego County, such as at the SVR. Use of this storage would require a revised operation plan of the AAC and would need input from IID to confirm both current and long-term projections of irrigation demand downstream of the Central Main Check.

The storage reservoir for Alternatives 5A and 5C is anticipated to be jointly located with the proposed treatment facility adjacent to the WSM. IID has indicated a preference to site the storage reservoir no further north than the Fox Glove Check. As such, the storage reservoir for Alternative 3A would be sited in the same location as Alternative 5A, which is near the existing Fox Glove Check. Siting of storage reservoir is discussed in greater detail in *Chapter 4.0 - Treatment, Blending and Brine Management Options*.

2.3.3 Use of the Westside Main Canal

A similar evaluation of existing canal capacity was performed for the WSM, Thistle and Trifolium canals, which are the canals required to get to the beginnings of Alternatives 3A, 5A, and 5C from the AAC. Table 2-9 provides a summary of mean flows and available capacities.

TABLE 2-9
Existing Canal Capacities

Canal	Mean Daily Flow Rate, cfs	Maximum Daily Flow, cfs	Maximum Capacity, cfs
Westside Main Canal	900	1,200	1,200
Trifolium Extension	---	---	150
Thistle	---	---	200

Additional monthly flow data provided by IID revealed that some capacity could exist in the lower reaches of the WSM, but that during most of the year, maximum daily flows approach canal capacity of 1,200 cfs. The capacity of the WSM drops significantly at Foxglove Check (10.5 miles north of the WSM heading) and at the Thistle and Suma Canal Check (25.6 miles north of WSM heading). North of this latter check structure, canal capacity is 600 cfs and less.

Based on Table 2-9 and the additional information provided by IID, the entire length of the WSM, along with the Thistle and Trifolium Extension canals, would have insufficient capacity for RCS flows.

2.3.4 New Gravity Flow Conveyance Options

As described above, the AAC and the WSM both have insufficient capacity for constant deliveries of RCS flows. For the AAC, this evaluation considered either a new gravity conveyance system, as described herein, or a new storage reservoir coupled with coordinated operations during non-peak hours, as described in Section 2.3.2. For the WSM, only a gravity conveyance system was considered as it would be the most economical. This section presents the new gravity flow conveyance systems that would allow for constant deliveries of RCS flows for both the AAC and the WSM.

AAC. To provide capacity in this section, a new parallel gravity conveyance system would be required from upstream of the New River Siphon to the terminus of the AAC at the WSM. As shown in Figure 2-3, the parallel gravity conveyance system for the AAC would begin at the New River Siphon and is anticipated to be a buried culvert or dual barreled pipe until it is past the existing residential areas and storage yards located south of the New River Siphon. Once past the developed areas, there appears to be enough right-of-way for a concrete lined canal parallel to and north of the existing AAC for the remainder, except where it crosses roads, irrigation laterals, and drains.

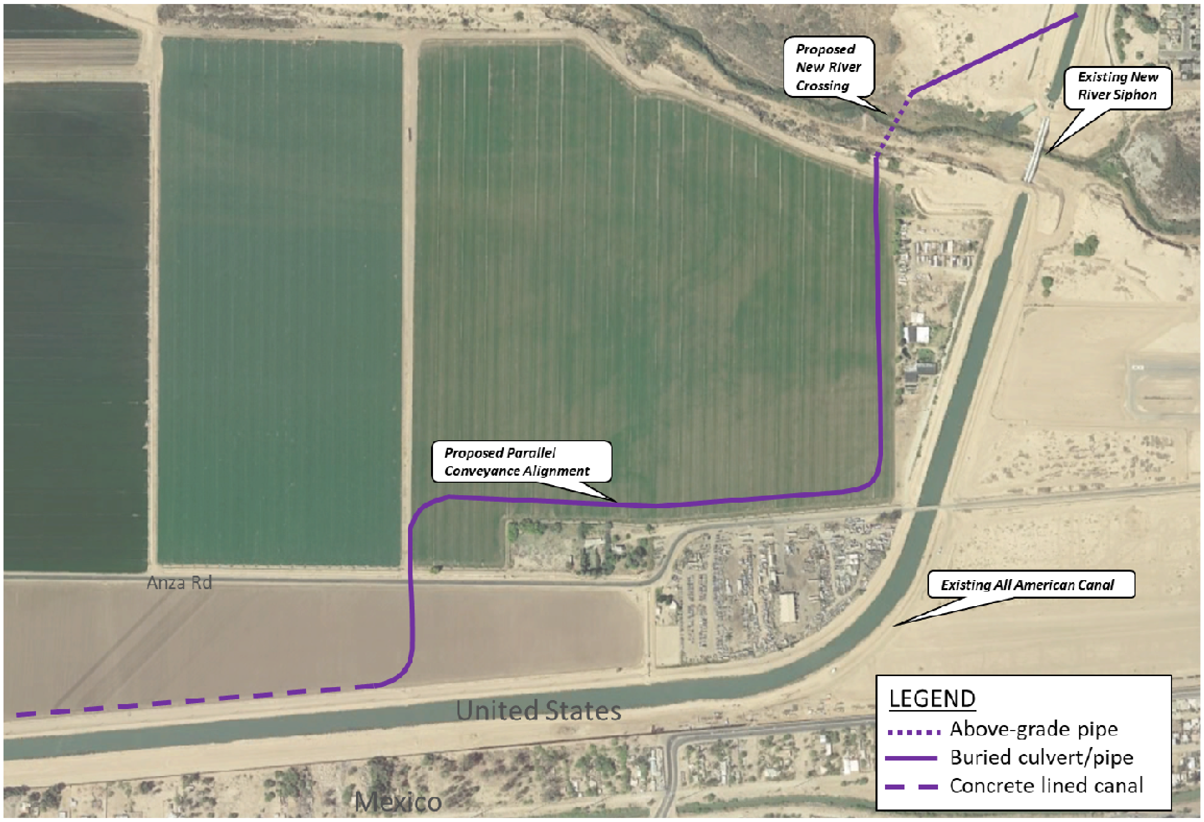

FIGURE 2-3
AAC Parallel Conveyance Alternative

While a gravity flow system would provide the capacity required for RCS deliveries, this study recommends and incorporates the storage reservoir described in Section 2.3.2 in lieu of a parallel gravity flow system for the AAC. The storage reservoir would provide many potential benefits to the RCS program, primarily due to the ability to balance the on- and off-peak flow periods, thereby limiting the storage volume required. Additionally, the storage reservoir could be sized to provide dual benefits with IID allowing them to rebalance their irrigation deliveries. Partnership opportunities are discussed in greater detail in *Chapter 8.0 – Partnership and Funding Opportunities*.

WSM. As described previously, the flows provided by IID show that the maximum daily flows in the WSM approach its capacity during much of the year. As such, a parallel gravity conveyance system sized to provide capacity for constant deliveries of RCS flows was the only solution considered.

The parallel gravity conveyance system would begin at the terminus of the AAC and would continue roughly parallel to the WSM, Thistle, and Trifolium Extension canals to the treatment facility locations for each of the alternatives.

Figure 2-4 presents the parallel gravity conveyance system for the WSM and the AAC. As discussed previously, the gravity conveyance system parallel to the AAC is not assumed to be part of the base project description but is shown as optional.

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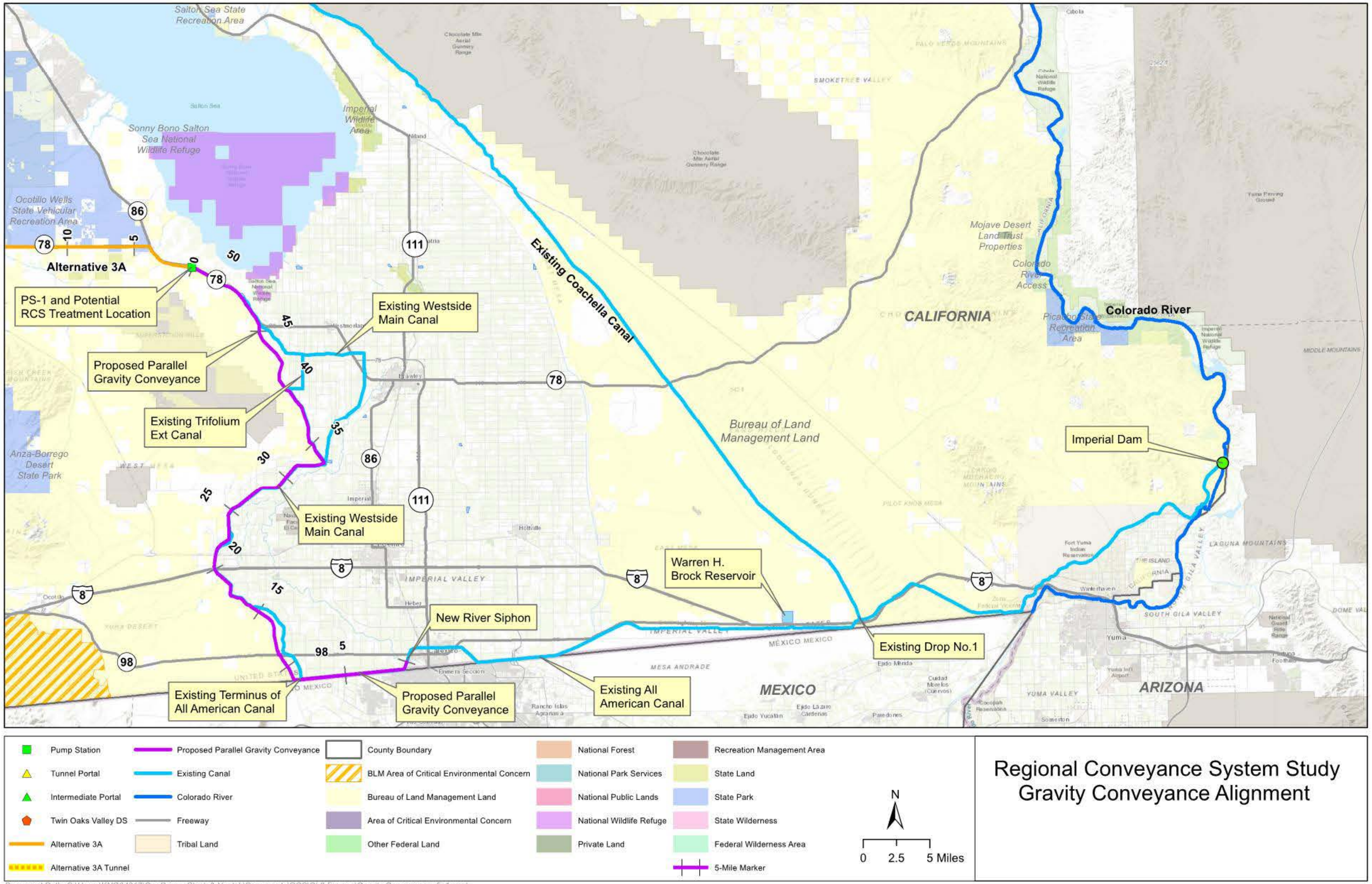

FIGURE 2-4
Gravity Conveyance Alignment

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2.3.5 Comparison of Gravity Conveyance Alternatives

Multiple designs are possible for the gravity conveyance systems that would deliver RCS flows, including box culverts, concrete pipes, or open channels (canals). This section presents the analysis completed to select the preferred gravity conveyance system design and included the completion of the following tasks:

- Identification of gravity conveyance system alternatives
- Determination of design assumptions to size facilities
- Comparison of advantages and disadvantages to each alternative

The intent of this section is to provide enough detail in order to; 1) determine a preferred gravity flow conveyance system type; 2) establish a conservative cost for budgeting purposes and; 3) describe the system in enough detail to support future environmental studies and form the basis for subsequent design phases.

Three types of gravity conveyance were identified for delivering water to the downstream delivery point at the pump station forebay or treatment facility:

- A concrete box culvert
- Two parallel reinforced concrete pipes
- Concrete-lined canal

Typical cross sections for these three alternatives are shown on Figure 2-5. Design assumptions for each alternative are listed in Table 2-10.

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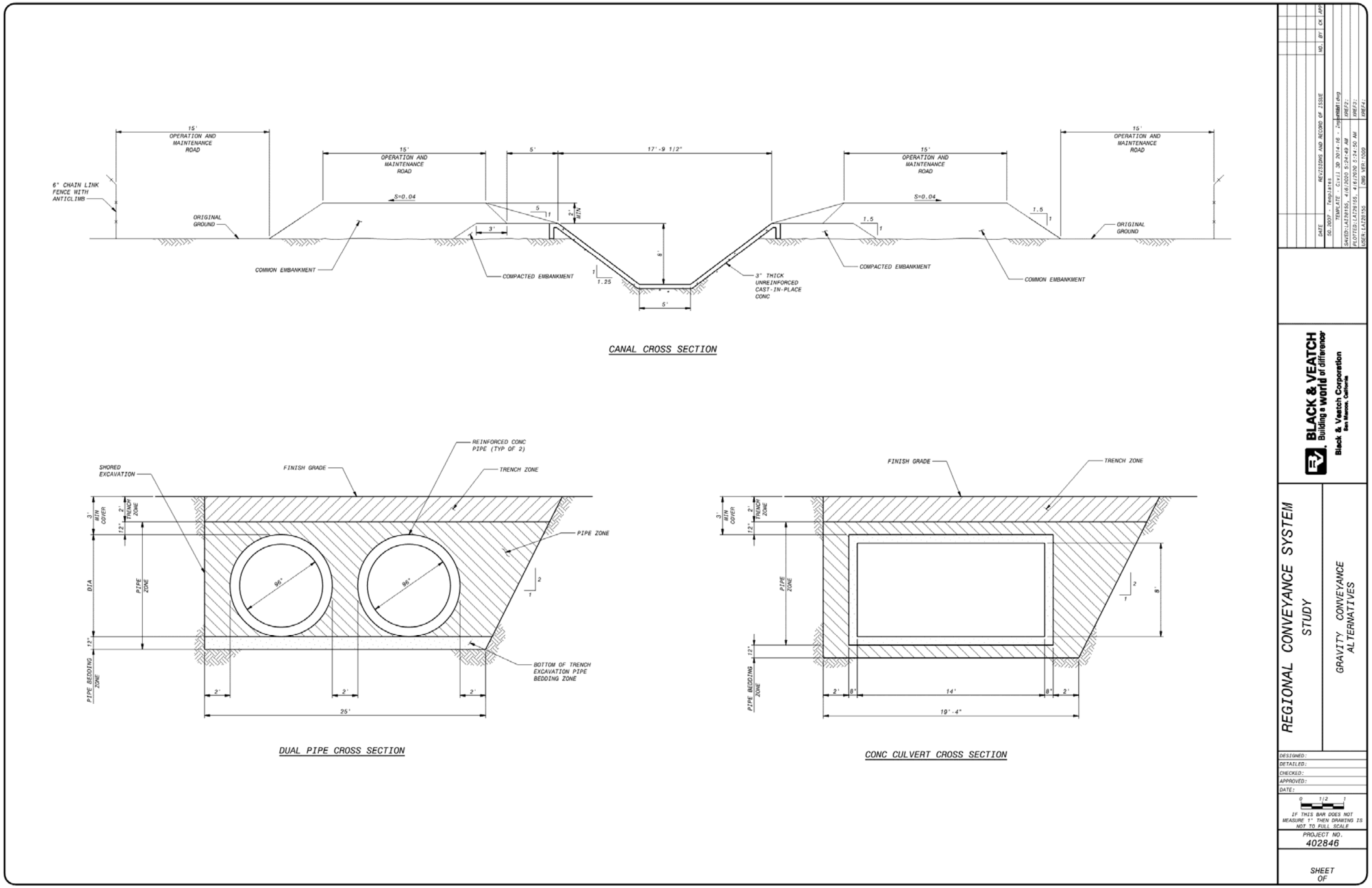

FIGURE 2-5
Gravity Conveyance Alternatives

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TABLE 2-10
Gravity Conveyance Design Assumptions

Design Criteria	Concrete Box Culvert	Two Parallel Reinforced Pipe	Concrete Lined Canal
Design Capacity, AF/yr	331,700	331,700	331,700
Slope, %	0.0006	0.0006	0.0006
Internal Dimensions	14 ft x 8 ft W x H	8 ft Dia.	5 ft x 8 ft Bottom W x H Side Slope 1.25:1
Depth of Cover, ft	3	3	N/A
Water Depth, ft	7	8	7.25
Easement Width, ft	60	60	105
Other Design Considerations	Gasketed joints	Gasketed joints	Unreinforced, 3” thick concrete

Table 2-11 provides a summary of the advantages and disadvantages for each gravity conveyance alternative.

TABLE 2-11
Gravity Conveyance Advantages/Disadvantages

Alternative	Advantages	Disadvantages
Concrete Box Culvert	• Little seepage • Closed conduit • No evaporation loss • Less silt build-up	• High cost • Confined space • Access manholes required • More difficult silt removal
Two Parallel Reinforced Pipe	• Little seepage • Closed conduit • No evaporation loss • Less silt build-up • O&M skills and requirements known to Water Authority	• High cost • Confined space • Access manholes required • More difficult silt removal
Concrete Lined Canal	• Lowest cost • Easier silt removal	• Higher evaporation loss • Open channel safety risk and liability • Siphons required at road, canal, and creek crossings • O&M skills and requirements new to Water Authority • Relatively higher cost for permanent easement • Higher silt build-up • Higher maintenance cost, relative to others

At the given design criteria, the concrete lined canal is anticipated to have a construction cost of seven to ten times less than either a concrete culvert or dual reinforced concrete pipes. With the relatively low annual operating and maintenance costs associated with any of the alternatives, it is anticipated that the canal would also have the lowest life cycle cost. For this reason and the other advantages listed in Table 2-11, the concrete lined canal alternative was carried forward for additional analysis as the basis of the gravity conveyance system. A concrete lined canal system requires siphons to cross highways, roads and irrigation canals. Table 2-12 provides a list of required siphon locations identified for the concrete lined canal gravity conveyance alternative. This list was developed based on Google Earth and extends from the terminus of the AAC to the downstream delivery point at the pump station forebay. The locations are shown on Figure 2-6.

TABLE 2-12
Canal Siphon Locations[1]

Location (Mile Marker)	Crossing Type	Length of Siphon (ft)	Siphon Description
0.6	Wash	75	132" Pipe Crossing
1.2	Wash	75	132" Pipe Crossing
3	Wash	75	132" Pipe Crossing
3.5	Wash	75	132" Pipe Crossing
4.4	Wash	75	132" Pipe Crossing
5.8	W. Bannister Rd	40	132" Pipe Crossing
14.3	Farm Road	40	132" Pipe Crossing
14.6	Farm Road	40	132" Pipe Crossing
15.9	Imler Rd	40	132" Pipe Crossing
18.8	Edgar Rd	40	132" Pipe Crossing
23.3	Erskine Rd	40	132" Pipe Crossing
24.9	Huff Road	40	132" Pipe Crossing
26.5	Farm Road	40	132" Pipe Crossing
28.8	County Road	40	132" Pipe Crossing
31	County Route S80	150	132" Pipe Bored Crossing
31.2	Train Tracks	150	132" Pipe Bored Crossing
32.3	Interstate 8	300	132" Pipe Bored Crossing
36.8	Imperial Valley Substation Access Road	40	132" Pipe Crossing
41.1	CA State Route 98	150	132" Pipe Bored Crossing

Note:

1. An additional siphon would be required for the AAC parallel canal to cross the New River Siphon should it be selected.

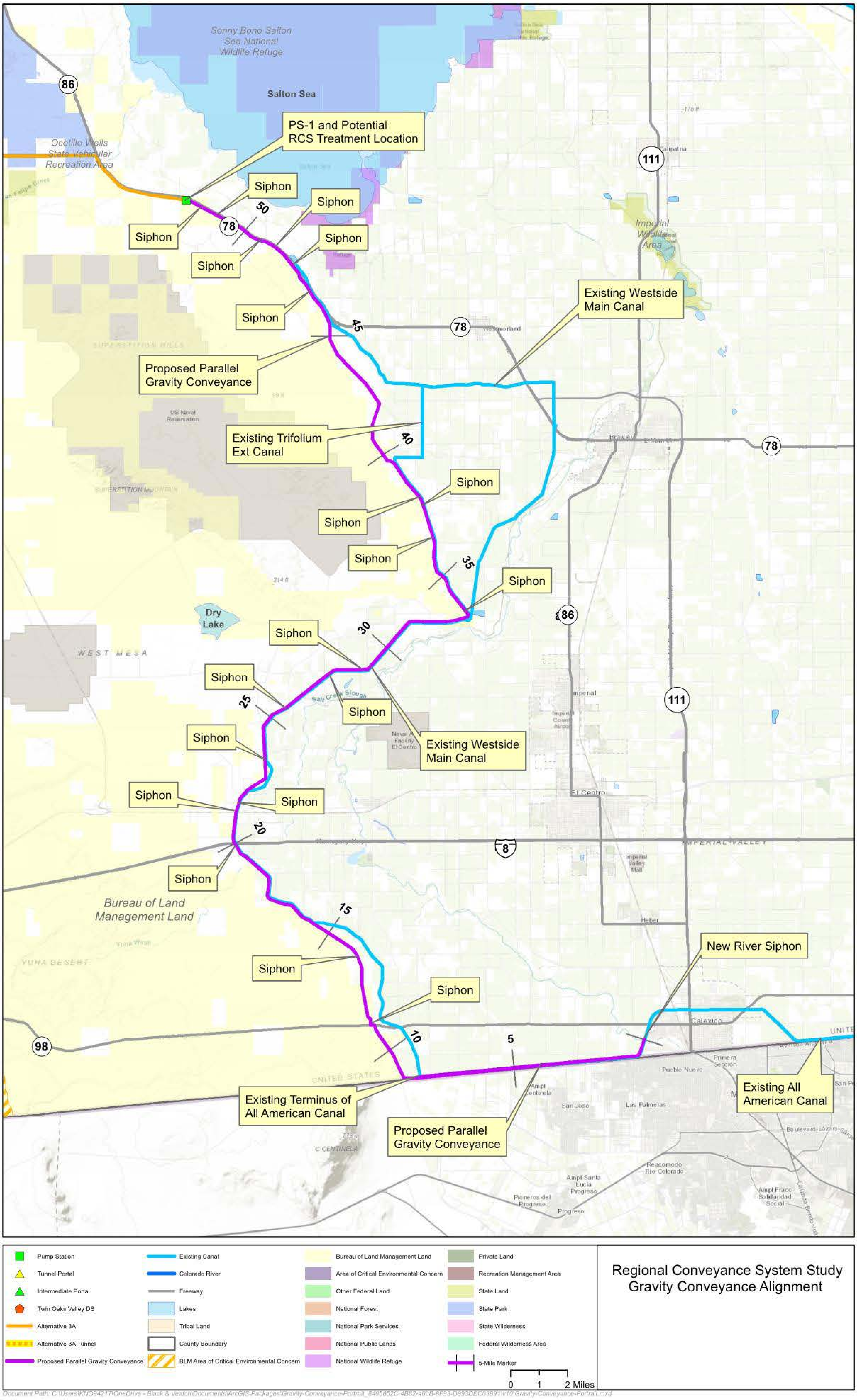

FIGURE 2-6
Proposed Gravity Conveyance Alignment with Siphon Locations

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2.4 Pipelines and Tunnels (Alternative 3A)

This section describes the physical characteristics of Alternative 3A's pressurized components as currently envisioned, including the locations of new pump stations and options for new operational storage. This chapter also documents the hydraulic and operating criteria used to size the facilities and presents a resulting general horizontal and vertical alignment of the pipeline and tunnels.

2.4.1 Alternative Overview

The pressurized portion of Alternative 3A is comprised of a series of pipelines and tunnels that begin at Pump Station No. 1 (PS-1), located at the north terminus of the new gravity flow canal that extends approximately 46.7 miles from the end of the AAC, and ends at a connection to existing aqueduct facilities at the TOVDS. The total length of this portion of Alternative 3A is 85.3 miles, which includes 38.8 miles of pipeline and 46.5 miles of tunnel. Table 2-13 provides a summary of the key characteristics of the Alternative 3A pressurized components.

TABLE 2-13
Overview of Alternative 3A Pressurized Components

Characteristic	Alternative 3A
Minimum Invert Elevation, Feet (MSL)	-218
Maximum Invert Elevation, Feet (MSL)	1,140
Total Pumping Head, Feet	1,980
Pipeline, Miles	38.8
Tunnel, Miles	46.5
Total Length of Pipeline and Tunnels, Miles	85.3
Pump Stations	3
Pressure Control Facilities (PCF)	1
Forebays/Storage, Capacity	3 (40 AF each)
Afterbays/Storage, Capacity	0
Terminal Storage, Capacity	3,500 – 4,000 AF

2.4.2 Alternative 3A Description

As shown on Figures 2-7 and 2-8 (following page 2-25), the pressurized portion of Alternative 3A would begin at PS-1, which is currently proposed to be located along the south side of State Route (SR) 86 at its intersection with Allen Road. The approximate ground elevation at PS-1 is negative 180 feet MSL.

Reach 1 consists of 21.4 miles of pipeline beginning at the collecting header of PS-1. From this location Reach 1 runs parallel and adjacent to the south side of SR-86 for 3.8 miles, crossing the alignment low elevation point at San Felipe Creek, to the intersection of SR-78.

Reach 1 continues west parallel to and south of SR-78, past the community of Ocotillo Wells, and ending at Pump Station No. 2 (PS-2).

Reach 2 consists of 15.2 miles of pipeline beginning at PS-2 running west generally adjacent and parallel to the following roads: SR 78 (south side), Borrego Springs Road (north side), and Tubb Canyon Road (south side). When Reach 2 nears the residential areas (community of Desert Lodge) bordering Borrego Springs Road, the alignment moves approximately 800 feet north of Borrego Springs Road to minimize the impact on the community. At Tubb Canyon Road, Reach 2 again runs parallel to and adjacent to this unpaved road to end at Pump Station No. 3.

Reach 3 is comprised of a short segment of pipeline extending 1.3 miles west from PS-3 following Tubb Canyon Road (south side) and nearby foothills to the location of the Tubb Canyon Portal.

Reach 4 consists of a 41.4-mile hard rock tunnel excavated with a tunnel boring machine (TBM). The 41.4-mile tunnel would extend between the Tubb Canyon Portal in the east and the Moss Tree Portal in the west. The tunnel would have two deep intermediate shafts that would serve as points of access to shorten the individual tunnel drives. The intermediate shaft near Lake Henshaw would be located at the tunnel high point and would provide passive venting during filling and draining of the tunnel. The other intermediate shaft serves as the heading for the connecting tunnel to Lake Wohlford. The vertical shafts are assumed to be constructed with raised-bore methods.

The eastern heading of the 41.4-mile hard rock tunnel would be 17.4 miles long and is identified as T1, while the western heading would be 24 miles long and is broken into two subsegments, known as T2E and T2W. T2E refers to the eastern section of tunnel T2 between the Lake Henshaw vertical shaft at mile 55.3 and the Lake Wohlford vertical shaft at mile 72.0, while T2W refers to the western section of tunnel T2 between mile 72.0 and the Moss Tree tunnel portal at mile 79.3. The two headings would meet at the intermediate shaft near Lake Henshaw at mile 55.3. The Tubb Canyon Portal would have an invert elevation of approximately 1,023 feet MSL and would be excavated upgradient at 0.1 percent slope. The western portal, known as the Moss Tree portal, would have an assumed nominal invert of elevation 955 feet MSL and would be excavated upgradient at 0.13 percent slope. The two headings would meet at mile 55.3 with a common invert of elevation 1,115 feet MSL. Between mile 79.3 and 79.9 the 3A alignment would utilize open cut construction with buried steel pipe and would end at the I-15 portal.

As expected for a linear project with a long hard rock tunnel, the tunnel drives set the critical path for the overall construction duration. To reduce the schedule to the extent possible, and to break the overall tunnel into multiple contracts, multiple TBMs would be used within tunnels T1, T2E, and T2W. For T1, two TBMs would excavate the tunnel from separate headings on each end; one would begin at the Lake Henshaw vertical shaft and the other would begin at the Tubb Canyon Portal. T2W would be constructed with one TBM beginning at the Moss Tree Portal and heading east. T2E would be constructed with two TBMs. The first would begin at the Lake Wohlford vertical shaft and head east and the second would begin at the Lake Henshaw vertical shaft and head west.

Reach 5 consists of 2.4 miles of tunnel (known as T3) and open cut buried steel pipe. Reach 5 would begin at the I-15 portal with a nominal invert of elevation 721 feet MSL and would be excavated upgradient at a 2.4-percent slope. The western portal, known as the Twin Oaks Valley Portal, would have an assumed nominal invert of elevation 986 feet MSL. Between miles 82.0 and 82.3 the pipeline would be constructed with open cut buried steel pipe and would continue to the new day tank sited near the existing at TOVDS.

Reach 6 consists of a 3.0-mile tunnel lateral to Lake Wohlford, known as T4. Reach 6 would begin at the Lake Wohlford vertical shaft and end at the Lake Wohlford portal. Reach 6 is anticipated to operate with flows in either direction, such that it could fill or drain the terminal storage water at Lake Wohlford.

Table 2-14 summarizes key information about each reach of Alternative 3A, including its beginning and ending location and station, the lift provided by the pump station (if the reach begins at a pump station), and the construction type.

TABLE 2-14
Key Characteristics of Alternative 3A Reaches

Reach	Beginning/Ending Location	Stationing (Miles)	Lift (Ft)	Construction Type
1	PS-1 to Pump Station 2 (PS-2)	0.0 – 21.4	650	Buried Pipe
2	PS-2 to Pump Station 3 (PS-3)	21.4 – 36.6	635	Buried Pipe
3	PS-3 to Tubb Canyon Portal	36.6 – 37.9	695	Buried Pipe
4	Tubb Canyon Portal to I-15 Portal	37.9 – 79.9	(Note 1)	(Note 2)
5	I-15 Portal to TOVWTP	79.9 – 82.3	(Note 1)	(Note 3)
6	Lake Wohlford Vertical Shaft to Lake Wohlford Portal	72.0 – Lake Wohlford	(Note 4)	Tunnel

Notes:
1. PS-3 provides the lift for Reaches 4 and 5.
2. Reach 4 would consist of a hard rock tunnel and a short section of buried steel pipe. Between mile 37.9 and 79.3 is assumed to be a hard rock tunnel (T1 and T2E/W) and between mile 79.3 and 79.9 would be open cut construction with buried steel pipe.
3. Reach 5 would consist of a hard rock tunnel and a short section of buried steel pipe. Between mile 79.9 and 82.0 is assumed to be a hard rock tunnel (T3) and between mile 82.0 and 82.3 would be open cut construction with buried steel pipe.
4. Further evaluation is required to determine the pumping requirements to provide operational storage at Lake Wohlford. Section 2.4.8 presents details on potential pumping scenarios.

Alternative 3A offers a few opportunities to partner with other agencies and jurisdictions to jointly use the facilities being considered. Two of the partnership opportunities being considered could impact the description of the facilities described above. These two partnership opportunities are described in Table 2-15, along with the potential impacts to Alternative 3A. The Phase A baseline cost estimates assume no partnership participation or associated benefits/funding opportunities.

Chapter 8 – Partnership and Funding Opportunities discusses all the potential partnership opportunities in more detail.

TABLE 2-15
Summary of Partnership Opportunities

Partnership Opportunity	Impact
A potential partnership to convey up to 20,000 AF/yr of water to Borrego Springs is being considered.	A turnout sized for 20,000 AF/yr would be located near mile 33.1 on the conveyance facilities. Downstream of this turnout the RCS conveyance system would be sized to convey 286,570 AF/yr.
A potential partnership opportunity was identified to incorporate pumped storage at Lake Henshaw.	To incorporate this opportunity, a pumped storage cavern would be required with a new pump turbine to pump up approximately 1,500 ft to Lake Henshaw. This opportunity would take advantage of the vertical shaft already planned near Lake Henshaw and could be further evaluated in future phases to determine its feasibility and benefits.
A potential partnership for the joint use of a 900-AF storage reservoir with IID is being considered (Note 1).	A 900 AF storage reservoir would be required downstream of the westerly terminus of the AAC to in order to address the lack of capacity in the canal to meet RCS demands. The reservoir would store water during non-peak hours to use it during peak-hours. Since IID is already considering a 600 AF storage reservoir near Foxglove Check to rebalance 12-hour irrigation flows, this storage reservoir also provides opportunities for shared operational benefits with IID. Further coordination with IID would be required in future phases to determine the size and potential benefits.
A potential partnership with the City of Escondido to repurpose Lake Wohlford for the terminal storage of the RCS (Note 1).	A terminal storage reservoir would be required for operation of the RCS. This study has identified multiple potential locations for terminal storage, one of which involves the repurposing of Lake Wohlford. In order to repurpose Lake Wohlford, the existing dam would have to be repaired to restore the reservoir to its historic volumes and alleviate safety issues. These repairs are currently cost prohibitive for the City of Escondido. If an agreement between the Water Authority and the City of Escondido could be reached, then it's possible the improvements could be cost viable for both parties. The agreement would be similar to the joint-use agreement between the City of San Diego and the Water Authority for improvements at SVR. It should be noted that this study has assumed the repurposing of Lake Wohlford as the basis for terminal storage of the RCS, including all infrastructure and improvements necessary for its implementation.

Note:

1. The cost opinions prepared for this study include the infrastructure necessary to implement a 900-AF storage reservoir in Imperial County and the use of Lake Wohlford for terminal storage of the RCS, as these facilities, or facilities like them, would be required in order for the project operate as desired. However, no cost recovery, revenue, or other benefits resulting from these or any other potential partnerships discussed in this report have been assumed at this conceptual-level of study.

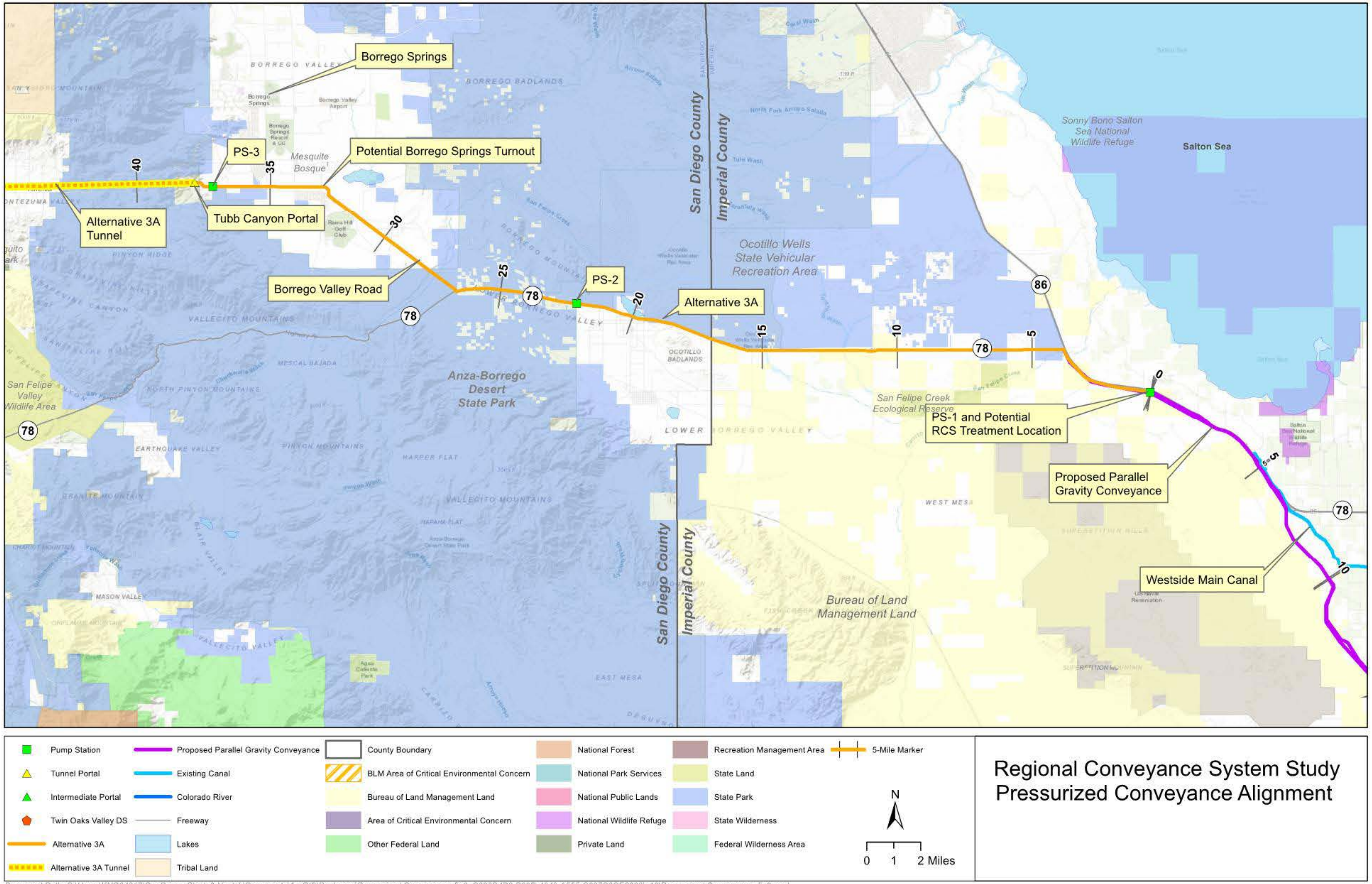

FIGURE 2-7
Alternative 3A Overview and Land Use Map (1 of 2)

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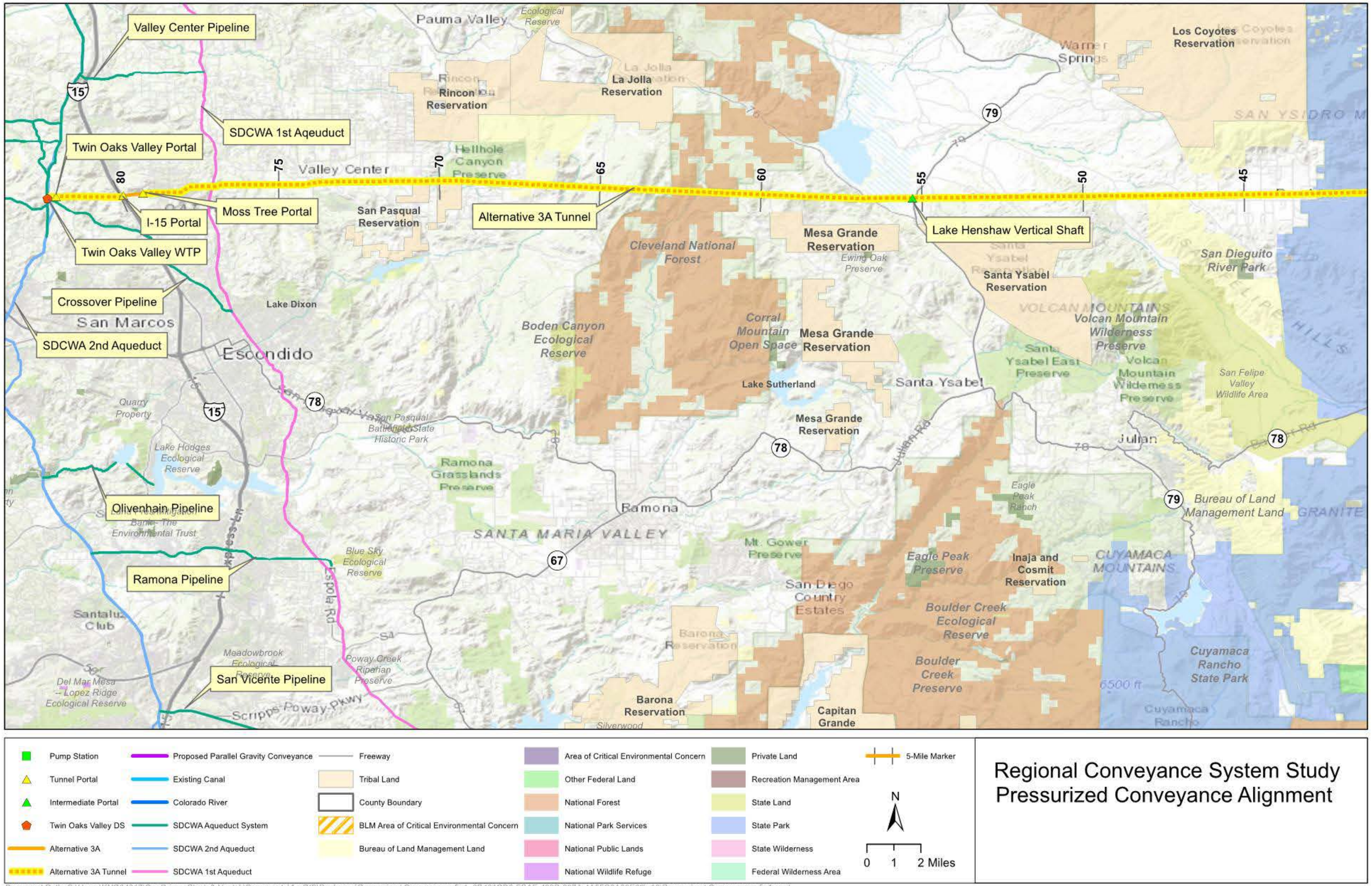

FIGURE 2-8
Alternative 3A Overview and Land Use Map (2 of 2)

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2.4.3 Land Use

This section identifies the land use types that would be encountered along Alternative 3A. Coordination and permits required to construct within these land use types are described in greater detail in *Chapter 7 – Environmental Review and Permitting*. To identify the various land use types along the alignment, records were reviewed from the Bureau of Land Management (BLM) and San Diego and Imperial Counties. Furthermore, a field visit was conducted on September 17, 2019 to verify the land use types previously observed from a desktop level and to identify any land use concerns or opportunities along the corridor. The land use types encountered by Alternative 3A are illustrated on Figures 2-7 and 2-8 above.

Alternative 3A would closely follow the California Department of Transportation (Caltrans) road right-of-way (ROW) for SR-78 from PS-1 to Borrego Valley Road. While the alignment would be closely adjacent to Caltrans ROW for SR-78, as currently envisioned the pipeline would be constructed outside of the ROW in a dedicated easement. Along this corridor the alignment would cross privately owned agricultural and non-agricultural land, open space land, and BLM lands.

Upon entering San Diego County, the north side of SR-78, and a small stretch on the south side, is designated as the Ocotillo Wells State Vehicular Recreation Area. At a minimum, the alignment would cross the short stretch of Ocotillo Wells State Vehicular Recreation Area on the south side of SR 78. While the alignment would primarily parallel SR-78 outside of the Caltrans ROW, when passing through Ocotillo Wells the alignment is assumed to be constructed longitudinally within the Caltrans ROW to avoid the existing development. Should Caltrans not allow the construction of a parallel utility within their ROW, the alignment could be rerouted around the existing development in Ocotillo Wells. Additional details on Caltrans ROW requirements are further discussed below.

West of Ocotillo Wells, the corridor enters the Anza-Borrego Desert State Park (ABDSP) for approximately 6 miles while traveling adjacent to SR-78 and the Borrego Valley Road. At this point the alignment enters Borrego Springs and leaves the ABDSP. Upon reaching Borrego, the alignment would diverge from paralleling Borrego Valley Road and travel around existing residential developments on the east and north sides. Once past the development, the alignment turns west using a combination of private property and Tubb Canyon Road ROW to the end of Tubb Canyon Road. Leaving the road ROW, the alignment veers northwest across more privately-owned property before entering the tunnel at a portal site within private property.

The tunneled alignment would cross beneath the ABDSP again, as well as privately owned land, BLM land, the Cleveland National Forest (for approximately 2 miles), and Caltrans ROW. Along the way, the corridor would cross between the BLM San Ysidro Wilderness Study Area and the San Felipe Hills Wilderness Study Area. It would travel north of the Santa Ysabel and San Pasqual Reservations.

The tunnel lateral to Lake Wohlford as currently envisioned would cross beneath privately-owned land as well as the San Pasqual Reservation.

A description of the general land use types crossed by Alternative 3A is as follows.

BLM Land. BLM overseas 15 million acres of public lands in California. They strive to ensure the best balance of uses for the land it overseas, as well as protecting natural resources. To do this, BLM has developed extensive land use plans to provide the framework to guide decisions for approved uses on BLM-managed lands.

Alternative 3A crosses through BLM land, mostly in Imperial County in Reach 1 where the pipeline would be constructed alongside SR-78. Coordination with the BLM would be required to ensure the pipeline met the intentions of the applicable land use plans.

Anza-Borrego Desert State Park. The ABDSP is a National Landmark and is a significant conservation area. Alternative 3A would cross the State Park with approximately 6 miles of trench pipeline construction, in addition to the approximately 4-mile tunneled portion. While in the State Park, the alignment would closely parallel SR-78 but would be located outside of Caltrans ROW and would, thus, cross through State Park land. The 6 miles of open trench construction was assumed for the alignment due to the favorable conditions for open trench construction, including open, mostly undeveloped land with minimal vegetation, excavatable soils (soft alluvial and lakebed sediments), and convenient access. For these reasons, among others, open trench construction was assumed to minimize the duration of the disruption to the park. Tunneling could also be considered for the entire section beneath the park as long as the portals and spoil sites were outside of the park's jurisdiction. Construction through the ABDSP would require coordination with the park, encroachment permits, and mitigation of adverse effects resulting from construction.

Cleveland National Forest. The Cleveland National Forest is administered by the United States Forest Service. Alternative 3A would cross beneath the forest with a tunnel for approximately 2 miles. Similar to Alternatives 5A and 5C from the 2002 Feasibility Study, it is anticipated that National Environmental Policy Act (NEPA) documentation, such as an Environmental Assessment (EA) or Environmental Impact Statement (EIS) would be required, as well as special use permits for drilling and tunneling.

Caltrans. Caltrans has jurisdictional authority over the state highways and interstate freeways, such as SR-78 and I-15. Caltrans has documented criteria for crossing expressways, freeways, and highways. These requirements are discussed in detail in *Chapter 7 – Environmental Review and Permitting*. Alternative 3A would cross beneath SR-78, SR-79, SR-76 and I-15.

Longitudinal encroachment permits within Caltrans ROW are generally very difficult to obtain. However, based on a conversation with Caltrans' District 11 Division of Planning and Local Assistance, Caltrans does make exceptions for public utilities depending on the situation and other factors, such as ROW ownership, other parallel utilities and powerlines, and environmental factors.

Private Land. Private land has the risk of development occurring on it between the completion of this study and the beginning of construction that would impact the ability to acquire land to construct the pipeline. Should the project proceed beyond this study, it will be imperative for the Water Authority to keep abreast of development plans along the selected alignment to ensure the corridor remains viable.

San Pasqual Reservation. The San Pasqual Reservation consists of lands held in trust by the United States government for the federally recognized Indian tribe. The land is federally managed by the United States Bureau of Indian Affairs rather than the state government in which it is physically located. Construction of a tunnel within an Indian Reservation is anticipated to be extremely difficult, as the tribe retains the rights to the water and minerals underlying their land. Further, there are increased concerns with subsurface facilities impacting a reservations groundwater resources due to a recent instance where an MWD tunnel is purported to have impacted the groundwater of a nearby reservation in San Bernardino County. Early coordination with the tribe is critical to understand the jurisdictional requirements and studies that would be required.

2.4.4 Pipe Sizing

This section documents the evaluations completed to size the pipelines for Alternative 3A. The information provides the basis for the pipeline cost estimate which is further described in *Chapter 6 – Risk, Cost Opinions, and Economic Analysis*. See Section 2.4.7 for details on tunnel sizing.

Several criteria were used for the preliminary hydraulic analysis and pipe sizing, including: flow rate, hydraulic losses, permissible velocities, and maximum allowable pumping head.

Design Flow Rate. As discussed previously, from mile 0.0 to 33.1 the system is being designed to convey an annual volume of 306,570 AF. From mile 33.1 to the TOVDS, the system is being designed to convey a uniform annual volume of 286,570 AF.

Hydraulic Losses. Manning's equation was used to calculate the hydraulic grade line (HGL) and friction losses throughout the system.

Manning's Equation: $H_{ft} = 4.66\ n^2\ L_{ft}\ Q_{cfs}^2\ /\ D_{ft}^{5.33}$

Where:

- H = Friction loss, feet
- n = Manning's n-value
- D = Pipeline internal diameter, feet
- L = Pipeline length, feet
- Q = Flow rate, cubic feet per second

Manning's n-values varies between the pipes and tunnel sections. For the cement-mortar lined steel pipes, a Manning's n-value of 0.012 was used. The lining and support configurations for the tunnels are discussed in greater detail in Section 2.5.7, but at this time there is insufficient information available to determine final lining system.

For the purposes of establishing n-values for hydraulic losses, three configurations were assumed for the tunnels: bare or shotcrete lined tunnels, steel-ribbed tunnels covered with shotcrete, or steel pipe lined tunnels. A Manning's n-value of 0.012 was selected for the steel pipe lined tunnels, 0.020 was selected for the bare and shotcrete lined tunnels, and 0.028 was selected for steel-ribbed tunnels covered with shotcrete. As most of the tunnel is assumed to be bare and shotcrete lined, this evaluation used a Manning's n-value of 0.020 for the entire tunnel. An additional 10-percent frictional assessment was made to account for minor losses. This is a conservative assumption.

Permissible Velocities. Transmission pipelines are typically designed to have velocities of seven to eight feet per second (fps) at the design flow rate to minimize surge potential and cavitation damage to the pipeline but could have velocities as high as 10 fps. For short periods of time under emergency operating conditions, a maximum pipeline velocity of up to 15 fps could also be considered. This evaluation targeted approximately seven to eight fps at the design flow rate.

Maximum Allowable Pumping Head. A maximum allowable pumping lift of 1,000-feet per pump station was selected to limit the internal pressure of the pipeline to 433 pounds per square inch (psi). This was done to limit the wall thickness of the pipelines to less than 1.25-inch. Thicker plates require special fabrication processes and transverse welds. By limiting the maximum pressure, it ensures the pipe could be fabricated by standard methods with longitudinal welds.

Pipe Size Optimization. Higher design velocities translate to higher hydraulic losses in the pipeline and, subsequently, higher pumping costs. Higher velocities can also require more expensive lining methods and lead to higher maintenance costs. Conversely, lower design velocities require larger pipe diameters which correlates to higher capital costs to construct. These factors all need to be balanced to determine an optimal pipe size.

At this stage of the study, a comparison of pipeline velocities and average head loss due to friction was completed for the range of pipe sizes under consideration.

Figures 2-9 and 2-10 present the velocities and friction loss per mile for each pipe diameter for the flows before and after the potential Borrego Springs turnout.

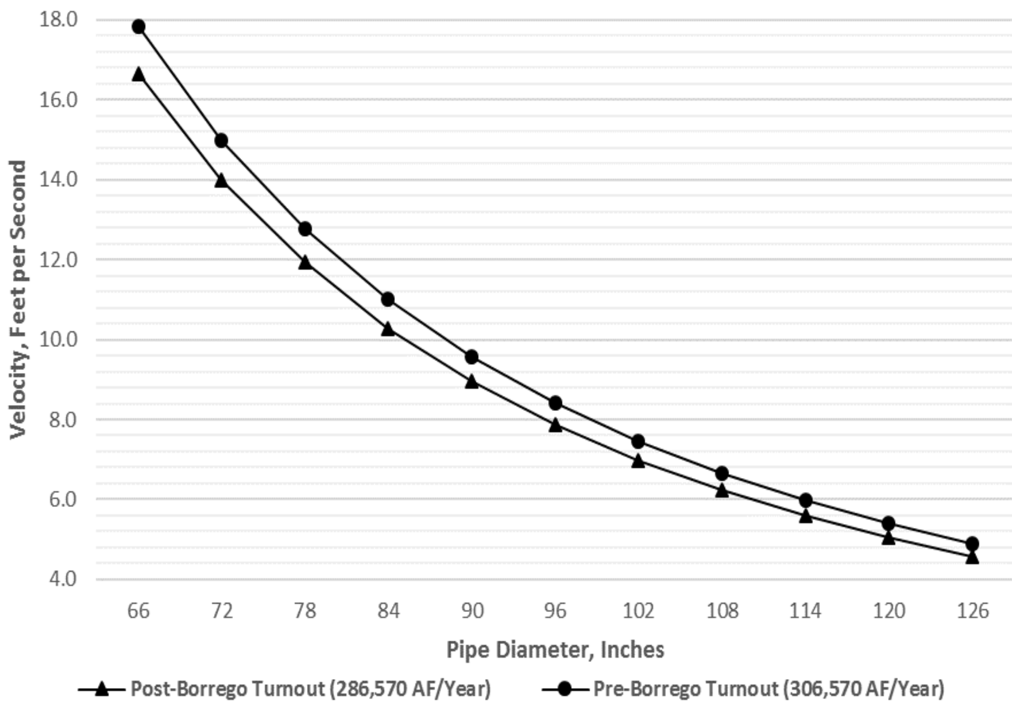

FIGURE 2-9
Plot of Pipe Diameters and Associated Velocities

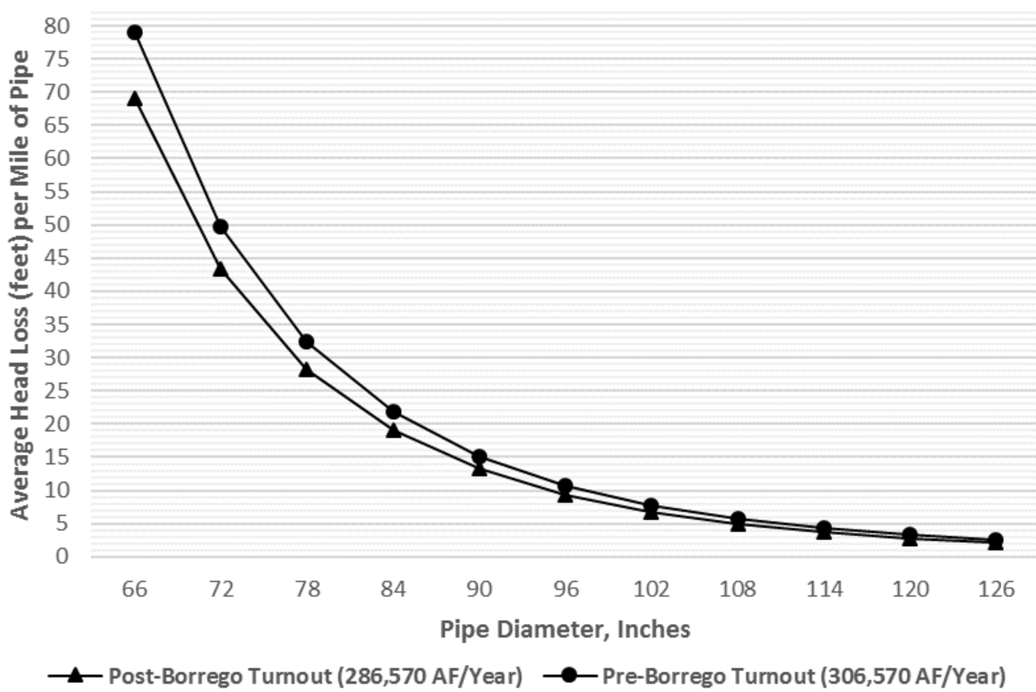

FIGURE 2-10
Plot of Pipe Size and Corresponding Head Loss Per Mile Due to Friction Velocity

The selected pipe size was 102-inches which results in a velocity of 7.5 fps. The stated diameter refers to the clear inside diameter of the pipe. Table 2-16 presents the corresponding velocities and average head loss per mile for both before and after the potential Borrego Springs turnout.

TABLE 2-16
Pipeline Velocity and Average Losses

Item	Pre-Borrego Springs Turnout	Post-Borrego Springs Turnout
Velocity, Feet per Second	7.5	7.0
Average Head Loss Per Mile, Feet(1)	7.8	6.8

Note:
1. Includes an additional 10% for minor losses.

The recommended pipe sizes for Alternative 3A should be further evaluated to balance the pumping power costs with construction capital costs. The analysis should compare the amortized capital costs and the annual energy consumption required to pump the water for each pipe size to determine a cost-effective diameter.

Figure 2-11 presents the hydraulic profile for Alternative 3A as described.

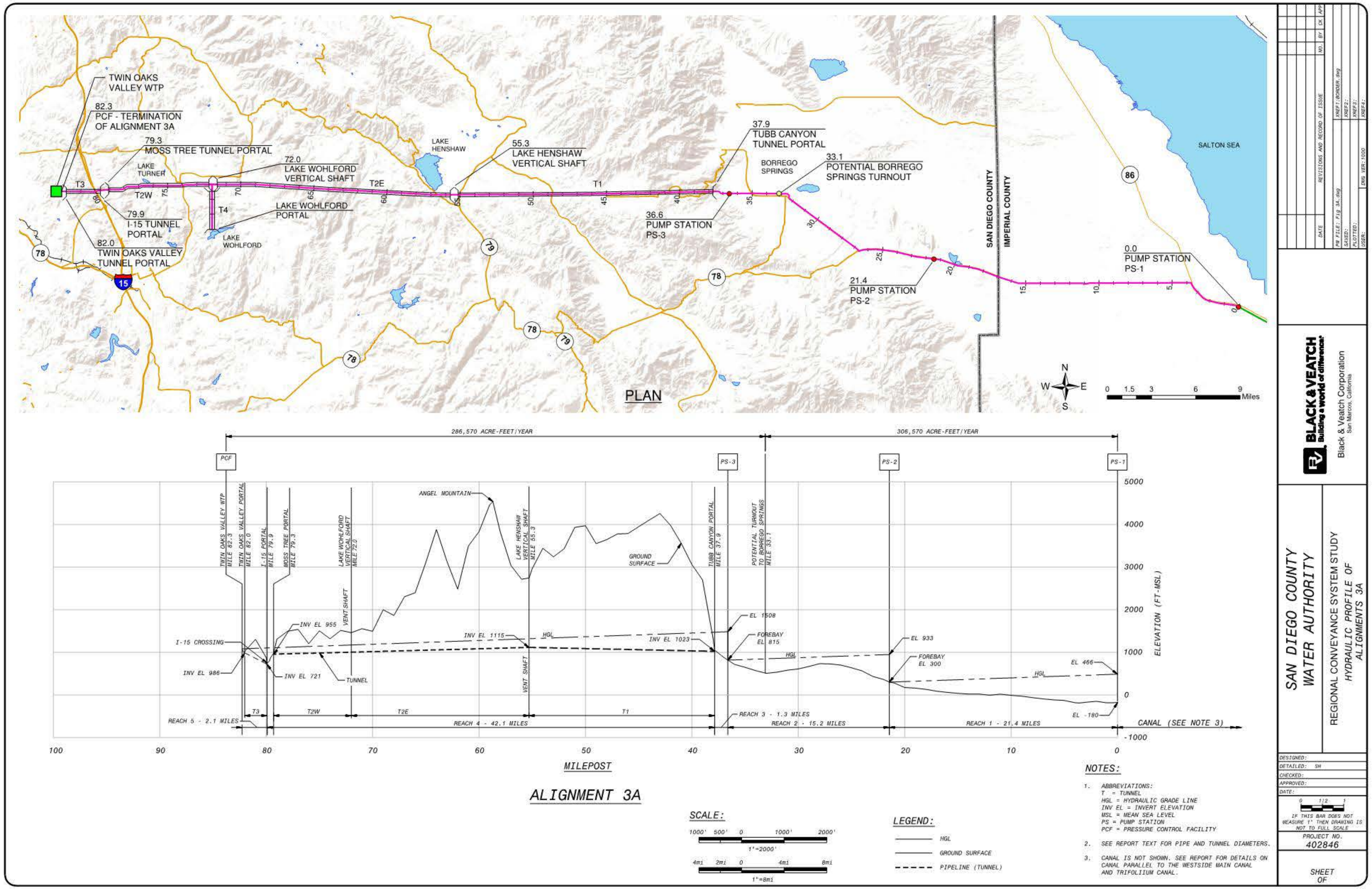

FIGURE 2-11
Alternative 3A Plan and Hydraulic Profile

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2.4.5 Alignment Geology

A Geotechnical Desktop Report (GDR) was completed as part of this study, titled "Geotechnical Desktop Study – Regional Conveyance System Study Alternative 3A." The report was prepared by Kleinfelder and is presented in its entirety in Appendix C.

The purpose of the GDR was to assess the general geotechnical and geological conditions along the proposed Alternative 3A. The information used in the preparation of this report included published literature, government agency websites, prior studies, and field geologic reconnaissance to document exposed geologic conditions. No subsurface field explorations or laboratory testing occurred during this phase of the project.

This section presents key information on the geology of the alignment summarized from the GDR.

Alternative 3A General Characteristics. Similar to Alternatives 5A and 5C, Alternative 3A is positioned within two geomorphic provinces. The eastern 17.5 miles of the alignment and the parallel gravity conveyance alignment are located within the Salton Trough Geomorphic Province. The remainder of the alignment is within the Peninsular Ranges Geomorphic Province.

Within the Salton Trough and the open-cut portion of the Peninsular Ranges, the alignment is anticipated to encounter soft alluvial and lakebed sediments that are easily excavated. Lake deposits occur most frequently on the eastern side of the alignment and consist of a variety of unconsolidated clay, silt, and sand. Alluvial soils are thinner in the east near the Salton Sea and generally grow thicker as the alignment continues west. Alluvial materials are typically comprised of coarse angular to subangular sands, gravels and conglomerate.

Alluvial and lake deposits overlie Pliocene age sedimentary rock (Pc) at depth. These units are visible at the surface near the San Jacinto fault.

At the southern end of the gravity conveyance system, the depth to groundwater is reported by the U.S. Geological Survey (USGS) to be from 31 to 100 feet below the ground surface (bgs). As the alignment nears the Salton Sea, the depth to groundwater is anticipated to shallow to 6 to 8 feet bgs.

To the west of the Salton Sea, the depth to groundwater increases to approximately 205 feet bgs in the Ocotillo Valley Groundwater Basin and up to 241 to 312 feet bgs in Borrego Valley.

The cut and cover section of the alignment is crossed by an active segment of the San Jacinto Fault Zone near the airport at Ocotillo Wells, while the gravity conveyance system is crossed by six active faults. Four of these faults are short strands of the Elmore Ranch Fault and cross the alignment in a ~three-mile span between Kane Springs and Elmore Ranch. The next is the Superstition Hills fault which crosses near Salt Creek Slough 2.5 miles west of Forrester Road. The last is a short active fault segment near the International Border.

The tunnel section of the alignment occurs within Mesozoic granitic and metamorphic rocks (bi, gr, gr^1, gr^9, JTrv, Ju, ms, and gr-m). The rock quality is anticipated to be weathered near the surface of the tunnels at the portal locations but would improve away from the openings to a slightly weathered or fresh condition. The tunnels are anticipated to be below the groundwater table and would cross several fractured zones, including an active segment of the Elsinore Fault Zone just west of Lake Henshaw near mile marker 56.6. The GDR contains tunnel plan and profiles that depict the anticipated location of various rock types and fault lineaments.

The bedrock comprising the majority of the 3A tunnels contains little to no porosity within the rock moss in which groundwater could reside. However, where the bedrock has been fractured from faulting and at joints, groundwater is able to reside. Further, the fractured rock could act as a conduit for flow parallel to the fault. The current geologic maps suggest that the void spaces formed from discontinuities in the tunnel portion of the alignment could act in this manner. At the discontinuities, high volume and high-pressure groundwater inflows could occur, and aggressive groundwater control measures would likely be required.

Geologic Issues. Key issues that impact the cost and construction schedule of these long and deep tunnels include:

- Potentially high rock temperatures due to the earth's natural geothermal gradient
- High in situ stresses requiring additional ground support
- Several significant lineaments (fault zones) which must be crossed, requiring probe drilling, pre-excavation grouting, and support
- Potentially high pressure/high volume groundwater inflows, requiring pre-excavation grouting and/or an impermeable liner to mitigate impacts to the groundwater resources during construction and to ensure worker safety
- Potential for squeezing ground conditions which affect tunneling methods and advancement rates

Groundwater within the tunnel section of the alignment serves commercial and residential purposes, as well as the U.S. Forest Service, BLM, ABDSP, and the San Pasqual Reservation. The design of the tunnels would need to include a water-tight lining and/or an extensive pre-excavation grouting program through the fault zones and other areas of discontinuities to prevent impacts to the groundwater table. Other aggressive measures to ensure groundwater control that could be required include maintaining probe holes in advance of the TBM and performing formation grouting.

Geotechnical Considerations. Key geotechnical considerations include:

- The amount of blasting required in hard rock areas to excavate the trench in open-cut areas
- The need to perform dewatering near the Salton Sea, along the WSM, and across major drainages
- Special fault crossing designs for crossing active fault zones (such as the Elsinore fault zone)

- The degree and length of ground improvement or special foundation design in areas subject to liquefaction or scour

Geologic Formations. The various geologic formations that underlie portions of the project alignment are as follows:

Qal – Unconsolidated alluvial stream, river and fan deposits (sand gravel and conglomerate)

Ql – Lake deposits of ancient Lake Cahuilla (clay, silt, and sand)

Qt – Nonmarine terrace deposits (silt, sand, and gravel)

Qc – Nonmarine sedimentary deposits (sand, gravel, and fanglomerate)

Pc – Undivided nonmarine sedimentary rocks (claystone, siltstone, and sandstone)

bi – Basic intrusive rock (gabbro)

gr – Undivided intrusive granitic rocks (tonalite, granodiorite, and diorite)

gr^t – Intrusive granitic rock (tonalite)

gr^g – Intrusive granitic rock (granodiorite)

JTrv – Metasedimentary rock (tuff, breccia, agglomerate, and volcanics)

Ju – Marine sedimentary and metasedimentary rock (argillite, slate, graywacke, and limestone)

ms – Metasedimentary rock (schist, quartzite, and gneiss)

gr-m – Granitic and metamorphic rock (schist, diorite, gneiss, phyllite, gabbro & monzonite)

2.4.6 Pipeline Construction Methods

The trenched construction recommendations remain unchanged from previous reports. Most of the trenched portion of the pipeline would be installed in open cut trenches to limit construction costs, but shored trenches would be utilized at stream crossings to minimize environmental impacts. Trenchless construction methods would be required for all interstate crossings, heavily traveled roads, and railroad crossings. A summary of key criteria for pipeline construction is provided in Table 2-17.

TABLE 2-17
Pipeline Trench Conditions

Item	Criteria
Depth of Cover (unrestricted areas within Right of Way)	7 ft
Un-shored slopes	1:1, 1/2:1
Minimum trench widths (shored)	Pipe outside diameter (OD) plus 4 ft
Minimum trench width (un-shored)	35 ft
Minimum ROW Required	91 ft
Right of Way Width	100 ft

Pipe Material. Water Authority standards require Cement Mortar Lined and Coated (CMLC) steel pipe with tape wrap.

Trench Backfill. Trench backfill shall be per Water Authority design guidelines. It is anticipated that a portion of the excavated trench material could be utilized as backfill within the trench zone, and any excess would be hauled offsite or used elsewhere. Excess spoils would not be allowed to be disposed in the ABDSP.

Temporary and Permanent Easements and ROW. Construction access for the trenched pipeline would primarily be via SR 78 and Borrego Valley Road and the right-of-way secured for the pipeline. No additional temporary construction easements are anticipated for access to the trenched pipeline construction. However, temporary construction easements would be required for the contractor's staging areas.

As presented in Table 2-17 above, the permanent ROW recommended for the pipeline would be 100-feet.

Construction Contracts. Due to the length of the pipeline and the varying construction techniques required for the corridor construction, it is anticipated that multiple construction contracts would be required.

Special Construction Areas. River and stream crossings would require special design and construction techniques, specifically shored trench construction or trenchless construction methods, such as jack and bore. Construction would be limited within these areas to minimize environmental impacts. When shored trench construction is utilized, the excavation would be deep enough to prevent scour and flotation of the pipe and the pipe could need to be encased. If sandy soils are encountered and deeper excavation is required, trenchless construction methods would be utilized.

Interstate crossings would require Caltrans encroachment permits. Additional coordination could be required with Borrego Springs to cross streets, such as Yaqui Pass Road and Borrego Springs Road. Trenchless construction methods would be required for interstate and railroad crossings, in addition to the associated encroachment permits.

2.4.7 Tunnels

Alternative 3A has four defined tunnel segments (T1, T2 (E and W), T3, and T4), as shown on Figure 2-11. Segments T1, T2E, and T2W total 41.4 miles, Segment T3 totals 2.1 miles, and Segment T4 totals 3.0 miles. The following sections provide a general perspective of the influence ground conditions have on tunnel design, construction, and construction costs and schedules. While insufficient information is known to determine the excavation, stabilization, and lining methods, assumptions were made based on the best available information to establish a conservative budget and construction schedule for planning purposes. These assumptions are summarized at the end of this section.

Ground Excavation. The method of tunnel excavation depends on many factors, including rock mass or soil strength and rock abrasivity. The method of excavation would also be governed by the construction schedule, the tunnel diameter, and the tunnel length.

All four tunnels along Alternative 3A are currently assumed to be located entirely within hard rock. Rocks of these types and strengths would require excavation using hard rock TBMs with disk and/or drilling and blasting.

Drilling and blasting is commonly used to mine hard rock but is generally slower than excavating with a TBM. Given the limited access along this tunnel alignment, drilling and blasting could be used for hard rock starter and exit tunnels to accommodate assembling the TBM below ground, to create additional space inside the tunnel for the contractor's needs, and at other localized locations. In addition, drilling and blasting could be considered for tunnel section T3 as a possible alternative to TBM tunnel excavation given its shorter length, provided blasting is allowed by local authorities having jurisdiction. All other locations are assumed to be excavated with a TBM such that the tunnels could be constructed within a reasonable construction schedule.

Faults. Special considerations would be required by the tunneling contractor when tunneling through faults and fault zones, especially those crossing the alignment at a high angle from perpendicular. These considerations could include slowing the tunnel advance rate, monitoring of groundwater inflow, and/or modifying the initial and final tunnel ground support and/or final lining. This is because the weakened state of the rock mass in the fault zone could lead to increased ground support requirements, which slows the overall tunnel advance rate, in addition to the increased potential for groundwater inflow. Probe drilling and associated pre-excavation grouting in front of the tunnel heading could be required for areas with known faults or fault zones, especially when high groundwater inflows would be anticipated, to reduce project risks and the potential for delays.

Alternative 3A crosses the Elsinore fault, which is a significant fault zone. At this crossing, it could be more cost-effective to change the method of construction to accommodate the faulted ground. For example, it could be more cost effective to "park" the TBM and use drill-and-blast techniques to cross the fault zone, versus to proceed through the fault zone with the hard rock TBM. Specialized designs would be developed for fault crossings. These designs could include, but are not limited to: 1) over-excavation or enlargement of the tunnel to provide for future movement of the fault where the tunnel crosses the fault; 2) filling of the annular space between the initial tunnel excavation and the exterior of the tunnel final lining with low strength material such as cellular concrete; 3) grouting the faulted ground to increase the strength and ductility of the faulted ground; and/or 4) using flexible joints to increase the longitudinal flexibility of the tunnel final lining.

Ground Stabilization. The initial and final ground stabilization and support requirements for the different tunnels would ultimately depend on the method of tunnel excavation selected, the groundwater inflow rates, and actual geologic and hydrogeologic conditions. The support system must: 1) provide adequate temporary support of the tunnel excavation until the final lining is placed; 2) be compatible with the elected method of excavation and final lining of the tunnel; and 3) permit transfer of the internal and external hydrostatic and rock loads to occur between the final lining and the surrounding rock mass without excessive deformation.

For portions of tunnels in intact rock, the initial ground support could consist of localized methods, such as rock bolts or dowels, to stabilize the appropriate rock blocks. The frequency and pattern of rock bolts required depends on the number and angle of rock block discontinuities at each location. The rock bolts could also serve as the final ground support, assuming no lining is required. Additionally, the tunnel could also be lined with shotcrete.

For portions of tunnels in areas of with severe spalling, numerous faults or fault zones, and/or high groundwater inflow conditions, various types of rock support, shotcrete, and/or precast or cast in place concrete or steel lining or a combination thereof would be required to serve as initial and final ground support. Shotcrete, cast-in-place concrete, precast concrete segments, and/or steel lining could serve as the final lining for tunnels. For the purposes of this study, steel-ribbing with an interior steel pipe lining was assumed at the fracture/shear zones.

Near the tunnel portals, there is a higher potential for poorer rock mass quality as a result of surface weathering. It is likely that full-perimeter initial supports would be required for the tunnel, such as steel-ribbing and lagging, rock bolts or dowels, steel mesh, and/or shotcrete, or a combination thereof. Cast in place concrete could serve as the final ground support and final lining at the completion of tunneling. For the purposes of this study, steel-ribbing with a steel pipe lining was assumed near the tunnel portals.

Tunnel Portals/Shafts. As previously discussed, there are seven tunnel portals, or shafts, currently being considered. The Tubb Canyon Portal would be located on the eastern end of tunnel T1, the Lake Henshaw Vertical Shaft would be located at the junction of tunnels T1 and T2E, the Moss Tree Portal would be located on the western end of tunnel T2W, the I-5 Portal would be located at the east end of tunnel T3, the Twin Oaks Valley Portal would be located at the western end of tunnel T3, the Lake Wohlford Vertical Shaft would be located between T2E and T2W at mile 72.0 and marks the northern end of tunnel T4, and the Lake Wohlford Portal would be located at the southern end of tunnel T4. The Lake Henshaw Vertical Shaft would be approximately 1,630 feet deep and the Lake Wohlford Vertical Shaft would be approximately 550 feet deep. The remaining portals would only be as deep as required based on site conditions.

The initial and final ground support for the shafts, especially for the deep shaft proposed at the Lake Henshaw Portal, would be selected based on the method of shaft excavation, groundwater inflow requirements, and actual geologic and hydrogeologic conditions. For portions of shafts in intact rock, the initial ground support could consist at a minimum of a few spot rock bolts or dowels, which could also serve as the final ground support, assuming no final lining is required. However, for portions of shafts in areas with faults or fault zones, and or high groundwater inflows, various types of rock support, shotcrete, or cast in place concrete, or a combination thereof would be required to be installed and would serve as the initial and final ground support and final lining.

Shafts used for launching and assembling TBMs (launch shafts), such as the Tubb Canyon Portal, the Moss Tree Portal, the Lake Henshaw Vertical Shaft, and the I-5 Portal, would need to be at least 2.5 times the diameter of the TBM. Shafts used for retrieval of TBMs (retrieval shafts), such as the Twin Oaks Valley Portal, would need to be at least 2.0 times the diameter of the TBM. For these tunnels, the anticipated diameter of launch and retrieval shafts would be in the range of 25 to 35 feet and could exceed 40 feet to account for additional utilities and ventilation lines required for longer tunnel drives, such as tunnels T1, T2E, and T2W.

TBMs require a substantial amount of electrical power to run the large motors and hydraulic pumps onboard the TBM and water to cool the components of the TBM. Therefore, a source of electrical power and water would need to be provided at the launching portals.

Temporary easements, approximately five (5) acres in size, would be required at each portal and shaft location associated with launching of a TBM for the duration of construction. For portals and shafts where TBMs would only be retrieved, the temporary easements would be on the order of two (2) acres in size. Permanent easements would be required for access to such portals and shafts for operations and maintenance in perpetuity by the Owner.

Due to the length of drives being considered, reviews were made of prior case histories to ensure the distances proposed are safe, efficient, and cost-effective for the mountainous terrain. TBM manufacturers were contacted during the 2001 Geotechnical Interpretive Report completed for Alternatives 5A and 5C to discuss the similar drive lengths proposed on Alternative 5A. Feedback from the TBM manufacturers verified that there was no reason why the drives would not be feasible as long as the ventilation requirements could be maintained, the tunnel muck could be efficiently removed from the heading, and groundwater could be removed from the tunnel. Utilities would be supplied to the tunnels from the portals and other equipment, such as booster fans, water pumps, and transformers, would be installed inside the tunnels as needed. Prior tunnel projects with similar drive distances are shown in Table 2-18.

TABLE 2-18
Prior Tunnels with Similar RCS Drive Distances

Tunnel	Location	Total Length (miles)	Diameter (feet)	Distance Between Portals (miles)
Gotthard Base Tunnel (Rail)	Switzerland	35.2	~30	9.9
Eurotunnel (Rail)	France / Great Britain	30.0	~25	24
Pahang Selangor Raw Water Tunnel (Water)	Malysia	27.7	17.2	14.6
Alimineti Madhava Reddy Tunnel (Water)	India	27	32.8	27

Groundwater Concerns. Nearly all the proposed tunnel reaches are expected to be constructed below regional or local water tables. Groundwater inflow volumes are dependent on the permeability of the rock or soil through which the tunnel is constructed, the pressure head of groundwater above the tunnel, and the length of time during which the tunnel is open and unlined.

The greatest potential for high volume and high-pressure groundwater inflows to the tunnel excavations is anticipated in the fractures associated with faults along the tunnel through the Elsinore fault zone and through the jointed zone from mile 66.1 to 66.6. Relatively minor inflows would be expected from general fractures and joints outside the influence of faults.

As the tunnel crosses beneath the Cleveland National Forest and near to Indian reservations, there would be increased sensitivity to the proposed tunnel affecting the groundwater. Proper planning to mitigate affects during construction and operation would be essential during the design of the tunnels, as they could be exposed to extremely high groundwater pressures, high temperatures, high groundwater inflows, and unavoidable impacts to water resources. Subsurface investigations coupled with hydrogeologic modeling is recommended during subsequent phases of work to better understand the impact groundwater could have on the project.

There are many feasible methods for controlling groundwater inflows in long hard rock tunnel excavations with high inflows, such as installing precast concrete liner segments with gaskets to assist in sealing the lining against groundwater leakage through the segment joints. These segments would be installed at the end of the TBM and grouted in place immediately after the TBM advances. For this study, it was assumed that probe drilling and associated pre-excavation grouting would occur in front of the tunnel heading. For the entire length beneath the Cleveland National Forest and when in proximity to Indian reservations, a more extensive pre-excavation grouting program coupled with the installation of an impermeable steel pipe for the final lining was also assumed. This area includes the Elsinore fault zone. The same construction approach was also assumed at the jointed zones from mile 66.1 to 66.6 and from mile 73.3 to 74.2.

Construction Water Disposal. As discussed previously, water used for cooling of the TBM, dust control, and groundwater inflows, would be present in the tunnel excavation. In general, tunnels are excavated upgradient such that the water would flow naturally to the portal of the tunnel. However, if the gradient is shallow then the water would need to be pumped to the portal or shaft. In either case, the water would be released at the portals or shafts.

Coordination with the San Diego County and Imperial County Regional Water Quality Control Board would be required during design and construction for the release of the water back into the groundwater basins. Discharge requirements for permits are based on the specific water quality objectives for the basins into which the water would be released, and the quality of the water being generated. It is anticipated that the water produced during the tunneling would require treatment for constituents introduced by construction, as well as for some naturally occurring constituents that could exceed current basin water quality standards.

Spoil Sites. Location(s) would need to be identified adjacent to portal and shaft sites for efficient disposal of the anticipated volume of excavated materials that would be generated during tunneling and shaft excavation. Spoil disposal sites would not be allowed in the Anza-Borrego Desert State Park near the Tubb Canyon Portal.

Easements and Right-of-Way. Subterranean, non-surface rights, easements would be required to be obtained for the entire length of the tunnel, except for portions within existing ROW as allowed by owner(s) of the ROW. For the tunnels being considered, the easement width would need to be on the order of 30-feet.

Excavated Diameter. The excavated diameter of the tunnel is governed by many factors, including: 1) the finished diameter of the tunnel required by hydraulics, 2) the initial and final ground support, 3) the required lining system required, and 4) the length of the tunnel due to the utilities necessary to support mining long tunnels, such as the ventilation needed to complete the required air exchange.

The cover on tunnels T1, T2E, and T2W, the longer individual tunnels, generally ranges between approximately 500-feet to 3,000-feet. Due to this depth, it becomes uneconomical to install additional portals to shorten the drive lengths. Therefore, the minimum excavated diameter would range from approximately 12 feet to 14 feet to support the size of the TBMs required, as well as the utilities, ventilation lines, conveyor belt muck removal system, large piping for groundwater removal, and equipment necessary for the longer tunnel drives. The actual excavated diameter of the tunnel would be determined by the tunneling contractor selected based on the equipment available. For the purposes of this study, an excavated diameter of 14 feet has been assumed.

Summary of Design Assumptions. As described earlier, assumptions were made for the design parameters of the Alternative 3A tunnels based on the best available information in order to establish a conservative budget and construction schedule for planning purposes, and to serve as the basis for future design phases and environmental studies. This section summarizes the assumed design parameters for the tunnels on Alternative 3A.

- Controlling groundwater inflows to minimize the impact on water resources would be required at all areas of discontinuities (fault and joint zones) and areas underlying National Forests or Indian Reservations. The following design conditions were assumed for mitigation in these areas:
    - Tunnel sections under National Forests or Indian reservations or crossing jointed / faulted zones would have an impermeable steel pipe as the final liner system.
    - A more extensive pre-excavation grouting program to control groundwater inflow would be required at all jointed zones equal to 1.5 times the assumed length of the crossing and at all fault zones equal to 3.0 times the assumed length of the crossing.
- Steel lining would be required at portals and extend into the tunnel until the overburden pressure is enough to resist the internal hydraulic pressure. Due to the slopes of the mountainous region surrounding the tunnel portals, the steel liner required is assumed to be 500 feet at each portal.
- The final lining system for the remaining portion of the tunnel is assumed to be shotcrete with rock dowels where needed.
- The tunnel is assumed to be excavated with an open-face, hard rock TBM.

Table 2-19 summarizes the assumed design parameters for the tunnels on Alternative 3A.

TABLE 2-19
Design Parameters for Alternative 3A Tunnels

Item	T1	T2 (E and W)	T3	T4
Start	Tubb Canyon Portal	Lake Henshaw Vertical Shaft	I-15 Portal	Lake Wohlford Vertical Shaft
End	Lake Henshaw Vertical Shaft	Moss Tree Portal	Twin Oaks Valley Portal	Lake Wohlford Portal
Excavated Diameter	14 feet	14 feet	14 feet	14 feet
Excavation Method	TBM	TBM	TBM	TBM
Length	17.4 miles	24.0 miles	2.1 miles	3.0 miles
Fault Zone Crossings	280 feet	300 feet	200 feet	140 feet
Joint Systems	---	3,000 feet	4,000 feet	---
Steel Liner at Portals	500 feet	500 feet	1,000 feet	500 feet
Pre-Excavation Grouting	840 feet	5,400 feet	6,600 feet	420 feet
Reservation and National Forests	---	36,960 feet	---	---
Shotcrete Lined	91,305 feet	85,960 feet	5,925 feet	15,060 feet

2.4.8 Terminal Storage

As previously discussed, a terminal storage reservoir would be required to provide for daily operation, to balance variations in monthly supply and demand, and to provide for periods of scheduled and unscheduled pumping outages. Based on conversations with the Water Authority's operations staff, approximately five days of active storage capacity (between 3,500 and 4,000 AF) is desired.

While the repurposing of Lake Wohlford has been assumed as the site of terminal storage for Alternative 3A, an alternative has also been identified by building a new reservoir located at or near Lake Turner.

This section provides a brief description of the use of Lake Wohlford for terminal storage, as well as the alternative of building a new reservoir at Lake Turner. Additional evaluations and coordination with the governing jurisdictions would be required to confirm the final site of the terminal storage.

Lake Wohlford. Lake Wohlford is a man-made reservoir formed by the Lake Wohlford Dam and owned and operated by the City of Escondido. The dam has a storage capacity of 6,800 AF with a water surface elevation of approximately 1,480 MSL. However, currently, due to dam safety issues, the reservoir is restricted to operating 20-feet lower than the dam's spillway reducing its available capacity. The City of Escondido has been studying dam retrofits to restore the dam to its historic capacity of 6,800 AF and alleviate the existing safety issues. The City of Escondido designed and permitted for a full dam replacement project. However, increased construction costs have made the complete dam replacement cost prohibitive for the city and the project has been delayed indefinitely while they evaluate more cost-effective rehabilitation options. If an agreement between the Water

Authority and the City of Escondido could be reached for the terminal storage of the conveyance system water at Lake Wohlford, then the improvements could be cost viable for both parties. This agreement would be similar to the joint-use agreement between the City of San Diego and the Water Authority for improvements made at SVR as part of the Emergency and Carryover Storage Project.

Lake Wohlford is approximately three miles south of Alternative 3A. As currently envisioned, tunnel T4 would connect Lake Wohlford to the main alignment at mile 72.0. Tunnel T4 would be designed for flow to occur in both directions such that the lake could be both filled and drained from it.

Historically the water surface elevation at Lake Wohlford is around 1,480, or about 350 feet above the HGL of the current system. To fill the lake, several options have been identified. Since the storage would only be used when required, there is no need to pump all the QSA water up to elevation 1,480. Instead, a new pump station or set of pumps at PS-3 would be installed that would be able to fill the lake when required. Another option, and the one that has been assumed for this study, would be to design PS-3 such that at least one of the pumps would be able to pump up to elevation 1,480 at a point higher up on its pump curve. Under normal operation, Lake Wohlford would be isolated from the rest of Alternative 3A and only drained when necessary.

Lake Wohlford is currently filled by runoff from its 7.3 square-mile drainage area, as well as water released from Lake Henshaw reservoir, via the San Luis Rey River and the Escondido Canal, as other options of filling.

New Reservoir at/near Lake Turner. *The Feasibility Level Engineering for Facilities to Transfer Water from The Imperial Irrigation District Study (September 1996*) proposed constructing a new reservoir, known as Moosa Canyon Reservoir, at or near Lake Turner which is located on the upper reaches of the Moosa Creek. This location was previously considered for the Water Authority's Emergency Storage Project. Lake Turner was built in the early 1970s to store raw Colorado River Water that was then disinfected and served to Valley Center customers. Due to changes in water quality, it has been cost prohibitive to serve potable water from Lake Turner since the 1980s. Since then, Valley Center Municipal Water District (VCMWD), the owner and operator of the lake and dam, has maintained the lake and studied various beneficial uses.

Lake Turner has a water surface elevation of approximately elevation 1,065 MSL. However, the storage reservoir for Alternative 3A needs a water surface elevation of at least elevation 1,190 MSL in order to gravity flow to the TOVDS at the appropriate hydraulic grade line. Furthermore, Lake Turner only has a storage capacity of about 2,000 AF. As such, significant modifications or reconstruction of Lake Turner would be needed to consider this location for required RCS operational storage.

Another possible reservoir site located southwest of Lake Turner would provide similar storage quantities but at elevation 1,215 MSL. To provide active storage, the canyon would likely need to be dammed off at elevation 1,300 MSL requiring about 110 feet of additional lift from the current pumping system.

Alternative 3A would keep the same general upstream alignment to reach either Moosa Canyon Reservoir location but the tunneling elevations would be revised such that it would end at or near Lake Turner. A new tunnel launching portal would be required at this location.

Upon discharging from the new reservoir, Alternative 3A could begin a new tunnel following the same alignment previously described to the I-15 Portal and the TOVDS. As an alternate to tunneling the final 6 miles, an alignment that would primarily allow for open-trench construction to deliver water from the Moosa Canyon Reservoir to the TOVDS could be considered. The alternate route is approximately 36,600-feet of new pipe and would generally follow Moosa Canyon to I-15. After crossing I-15, the alignment would follow El Paseo and Silverleaf Lane until it reached the Water Authority's second aqueduct pipelines near the City of Oceanside Water Treatment Plant. From here, the alignment would make use of the existing aqueducts for the remaining two miles to the TOVDS. This alignment has high points of elevations around 1,200 MSL and would require a water surface elevation at the storage reservoir approximately 1,245 MSL.

Figure 2-12 presents the open-trench alignment from the Moosa Canyon Reservoir to the TOVDS.

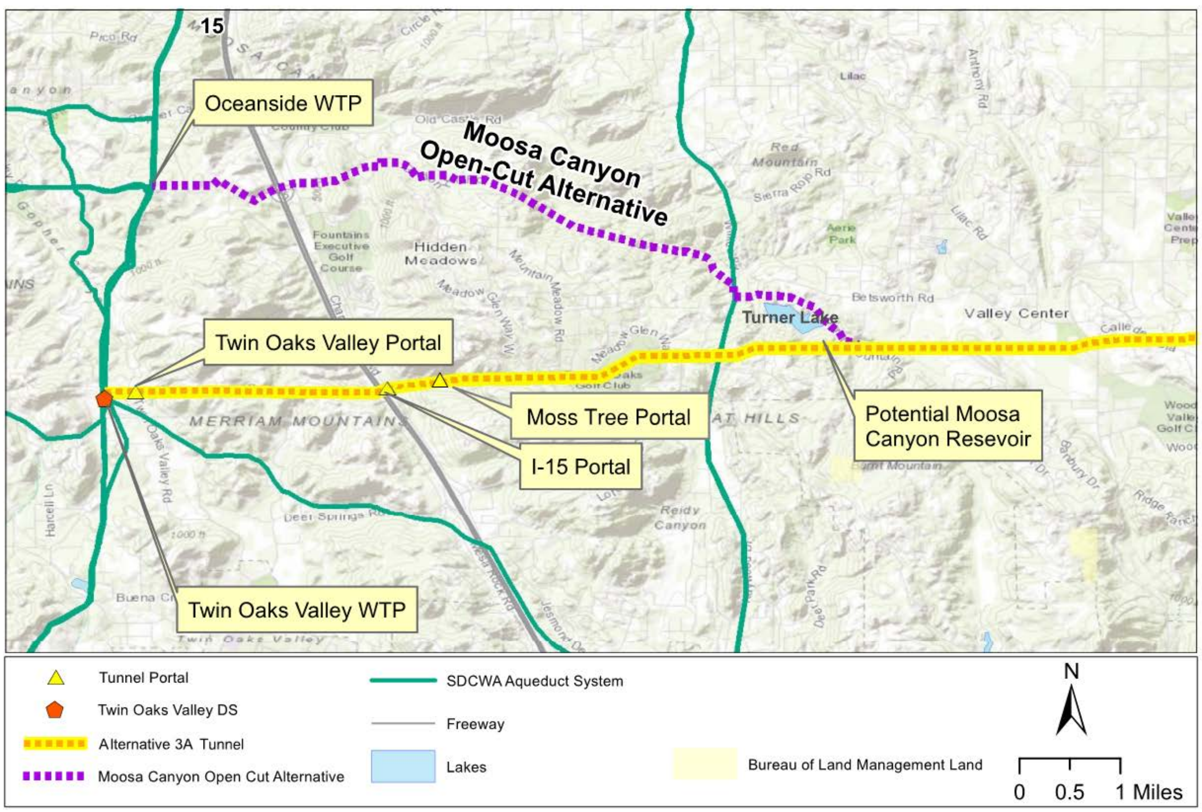

FIGURE 2-12
Moosa Canyon Open Cut Alternative Alignment Map

2.4.9 Filling and Draining

The proposed profile for Alternative 3A was reviewed at a high level to ensure that draining of the pipeline would be feasible and would not require extensive infrastructure to accomplish. Much of the conveyance system from the tunnel high point near Lake Henshaw could drain by gravity back to the Westside Main Canal or to the TOVWTP. The sag in the alignment between mile 28 and 37 would need additional blow off points for discharge. This area represents approximately 60+ AF of water. Drainage routes would need to be reviewed during subsequent phases of work to determine the final locations of the blow off/drainage points. At this stage of project development, detailed draining plans are not warranted but it does not appear that draining the water away from developed areas would be a significant concern.

2.5 Pump Stations (Alternative 3A)

This section presents the operating criteria and facility descriptions for the pump stations and forebays proposed for use for Alternative 3A. The information serves as the basis for the cost estimates. Cost are presented in *Chapter 6.0 – Risk, Cost Opinions and Economic Comparison.*

As discussed previously, it was determined that Alternative 3A would require three pump stations located in series to provide the energy required to overcome the changes in elevation and system head losses along the alignment and limit the maximum lift required to less than 1,000-feet, per pump station. Pumping is assumed to occur at a constate rate throughout the year. An additional 10-percent pumping capacity is assumed to account for unplanned or planned outages within the RCS and is aligned with capacities needed for aqueduct integration.

This study has developed a single, typical facility layout that was applied to each pump station along the alignment to provide a common basis for pumping facility cost estimates. Table 2-20 presents the key design criteria for each of the pump stations being considered for Alternative 3A.

TABLE 2-20
Pump Station Design Criteria

Item	Criteria
Number of Pumps	3 duty, 1 standby
Pump Type	Vertical turbine, single stage, constant speed
Rated Discharge, each pump	PS-1 and PS-2: 142 cfs PS-3: 132 cfs
Rated Total Head	657 ft (average)
Pump Efficiency (assumed)	85%
Rated Horsepower (hp), each pump	12,500 hp
Motor Type	Vertical, synchronous
Maximum Pipeline Wall Thickness	1¼ inch
Property Acquisition	10 acres/Pump Station & Forebay combination

The pump station attributes described above were based on the recommendations from previous studies. These recommendations would be further evaluated as part of Phase B of this study, should Phase B move forward. Examples of the evaluations to be considered would be:

- Number of Pumps: The number of duty pumps should be further evaluated. By increasing the number of pumps, it would reduce the rated hp of each pump which could increase the pool of manufacturers capable of supplying pumps of that size.
- Pump Type: Other pump types besides vertical turbine pumps should be considered. For instance, the Water Authority's San Vicente Pump Station uses horizontal split case pumps which could also provide benefits at these pump stations.
- Motor Controller Type: Various combinations of motor controller types will be considered for each of the pumps at each pump station, including full voltage starters, soft starters, and variable frequency drives.

As recommended in prior studies, the pump stations would include the following features: 1) a reinforced concrete structure with a steel framed superstructure and metal wall panels, 2) trash racks and gates slots on each pumping units intake, 3) a 35-ton traveling bridge crane and a 10-ton gantry crane designed to include the ability to pick up and move equipment directly out of the building and onto a truck, and 4) typical auxiliary electrical and mechanical systems meeting the Water Authority's design standards. Each pumping unit would be provided with double isolation such that there is not a need for a safety siphon or additional draindown of the conveyance system.

Each pump station would require a new substation with main step-up transformers, circuit breakers, and a takeoff tower. Backup systems to ensure reliable, uninterrupted power to the pumps would be provided. Details on the power supply and infrastructure requirements are presented in *Chapter 5.0 – Power Supply Alternatives*.

Each pump station would be provided with a forebay that would be sized for normal startup and shutdown of the pump station and for unscheduled outages of individual pumps or the entire pump station. The forebays would also be sized to balance the differences in discharge between the three pump stations, which would be in series.

Table 2-21 presents the forebay design criteria, which are based on the recommendations from previous studies. These recommendations would be further evaluated as part of Phase B of this study, should Phase B move forward. A single forebay design was used for the pump stations based on the larger 319,011 AF/yr flowrate.

TABLE 2-21
Forebay Design Criteria

Item	Criteria
Operational Storage	60 minutes at 100 percent pumped discharge
Operational Storage required for 306,570 AF/yr	40 AF
Reservoir Surface Area	4.0 acres
Type	Earthen with plastic liner (if topography allows)

2.6 Conclusion

This chapter summarizes the key facility components required for the RCS with a focus on bringing the project description of Alternative 3A to the same level of detail as that of Alternatives 5A and 5C. It is recognized that other components, such as blow-offs, air-release and vacuum valves, access manways, isolation valves, flow meters, pumping wells, and other miscellaneous appurtenances, would be required for the proper operation and maintenance of the RCS. The intent of this chapter was to provide enough detail to support the potential subsequent preliminary design and environmental study phases. As such, the focus was on key facility components, as is typical of a planning level study.

Additional coordination and evaluation would be required to verify several of the key facilities, including:

AAC Operational Storage Reservoir. The flow information provided by IID and the assumptions used to size the storage reservoir need to be further validated with IID as part of Phase B should it move forward. Further, as this facility could provide shared operational benefits with IID, additional evaluations would be required that consider any potential use that IID would desire, such as balancing their 12-hour irrigation cycles, simultaneous with the operation of the RCS.

Lake Wohlford Terminal Storage. While the Water Authority has met with the City of Escondido regarding the potential use of Lake Wohlford as terminal storage for the RCS, no formal agreements have been discussed or reached.

Connection to Lake Wohlford. As currently described, Alternative 3A connects to Lake Wohlford via a tunnel that crosses beneath the San Pasqual Reservation. Coordination would be required with the reservation to verify the feasibility of this route. Should the alignment shown prove infeasible, alternative alignments/connection strategies should be considered.

Again, the benefits associated with the potential aforementioned potential joint uses facilities are not included in the baseline RCS cost estimates. Additional facilities beyond those described in this chapter are required for the implementation of the RCS. *Chapter 3.0 – Aqueduct Operations and Integration of the RCS* discusses the details regarding the integration of the RCS with the Water Authority's existing aqueduct system, while *Chapter 4.0 - Treatment, Blending and Brine Management Options* describes the proposed treatment facility.

3

AQUEDUCT OPERATIONS AND INTEGRATION OF THE RCS

Chapter 3.0 Aqueduct Operations and Integration of the RCS

3.1 Introduction

3.1.1 Overview

The existing Water Authority aqueduct system was largely built to provide north-to-south gravity flow conveyance that is reliant on imported water supplies from MWD. The introduction of the RCS would alter current aqueduct operating strategies and result in new facility requirements to assure the reliable delivery of untreated water supplies to local water treatment plants and storage reservoirs.

The purpose of this chapter is to evaluate new strategies for operation of the aqueduct system specifically needed for the distribution of the Water Authority's QSA supplies. New facility requirements, along with any required modification to the operation of existing facilities, are identified herein. High-level descriptions of current aqueduct operating procedures are provided to show how the integration of the RCS would commence at the new connections to the existing aqueduct facilities at two general locations: (1) Pipeline No. 5 near the TOVDS for Alternative 3A or (2) the SVR area for Alternatives 5A and 5C.

A summary of new facility requirements and key operational characteristics associated with RCS deliveries are provided below in Table 3-1.

TABLE 3-1
Key Characteristics for Revised Aqueduct Operations

Characteristic	Alternative 3A (TOVDS)	Alternatives 5A, 5C (SVR)
Annual QSA Supplies, AF	258,364	258,364
Delivery Gradient, feet	1,140	920[1]
Pump Stations[2], each	0	1
Forebays[3], capacity	0	1 (6 MG)
Total Pump Head[4], feet	0	490
Peak Month Design Flow[5], cfs	n/a	220
Pressure Pipelines, miles	0	12.5
Flow Regulatory Storage[6], MG	40	40
Seasonal Storage (at SVR), AF	19,610	19,610
Untreated Water Demand Scenarios		
High Projection (year 2045), AF	308,000	308,000
Low Projection[7], AF	273,000	273,000

Notes:

1. Two options for delivery gradients are considered; San Vicente Surge Control Facility (SVSCF) at 920 ft and SVR at 764 ft.

TABLE 3-1
Key Characteristics for Revised Aqueduct Operations

Characteristic	Alternative 3A (TOVDS)	Alternatives 5A, 5C (SVR)

2. The pump station would be required to serve untreated water demands north of the Del Dios Valve Vault (DDVV).
3. The forebays are sized to provide 60 minutes of storage at peak flows.
4. Total pump head is based on a pump station elevation of 800 ft and a delivery gradient of 1,140 ft at Twin Oaks.
5. The design flow includes 10 percent surcharge to account for system downtimes.
6. The size and location of the flow regulatory storage structure would be determined during subsequent phases of work to provide hydraulic gradient control and storage volume to balance daily flow changes. To meet only the needs of the RCS, 60 minutes of storage at peak flow rates is anticipated to be enough. 40 MG was assumed for planning purposes.
7. A low projection for untreated water demands was used to further evaluate seasonal storage requirements. The low demand projection is based on year 2035 projections, representing a 11.4% lower demand response compared to 2045 projections.

As noted in Table 3-1, a number of untreated water demand scenarios are evaluated in this chapter to gauge system response to varied monthly demands and relatively constant deliveries by the RCS. Supply imbalances, which would occur when deliveries by the RCS exceed untreated water demands, would entail the delivery of QSA supplies to seasonal storage. Potential reservoir operating impacts related to providing seasonal storage at SVR are further discussed in this chapter. Lastly, supply imbalances and storage implications related to an extended shutdown period for planned maintenance activities are also considered.

3.1.2 Chapter Organization

As noted above, this chapter evaluates new strategies for operation of the aqueduct system for proper distribution of the QSA supplies and is broken up into the following sections.

Summary of RCS Delivery Points – This section discussed the connection points for Alternatives 3A, 5A, and 5C into the existing Water Authority aqueduct system.

Aqueduct System Operating Considerations – This section evaluates the impact the RCS would have on the operation of the existing aqueduct system and what new facilities would be required for proper operation.

Evaluation of Untreated Water Supply Imbalances – This section describes the comparison of QSA supplies and untreated water demands to identify supply imbalances. Imbalances would be addressed through storage, reoperations of existing facilities or construction of new facilities.

New Facility Requirements – This section describes the improvements that would be required to the Water Authority's existing aqueduct system for integration of the RCS.

Impacts to SVR – This section describes how the existing reservoir operation would be impacted by the new RCS.

Impacts to Other Facility Operations – This section describes the potential impacts the RCS might have on the Water Authority's existing facilities.

Conclusion – This section provides a summary of the analysis performed and recommended improvements required for proper integration of the RCS into the existing Water Authority Aqueduct system.

3.2 Summary of RCS Delivery Points

As noted above, two locations were considered in this study for the RCS's westerly terminus and delivery point for QSA supplies into the Water Authority's Aqueduct system. New facility requirements for each location are briefly described below, with further details included in Section 3.5.

- **TOVDS Delivery Point** – Alternative 3A is proposed to terminate at a new flow regulatory structure (FRS)/day tank, positioned near the TOVWTP, as shown on Figure 3-1 below. The new FRS/day tank would be used for providing hydraulic elevation (1,140 feet) for gravity flow control of untreated water deliveries throughout the Water Authority service area and for blending with Metropolitan supplies. Except for the new FRS/day tank, and associated pipelines, no other new facilities are needed for the distribution of untreated water supplies to local treatment plants and storage reservoirs.

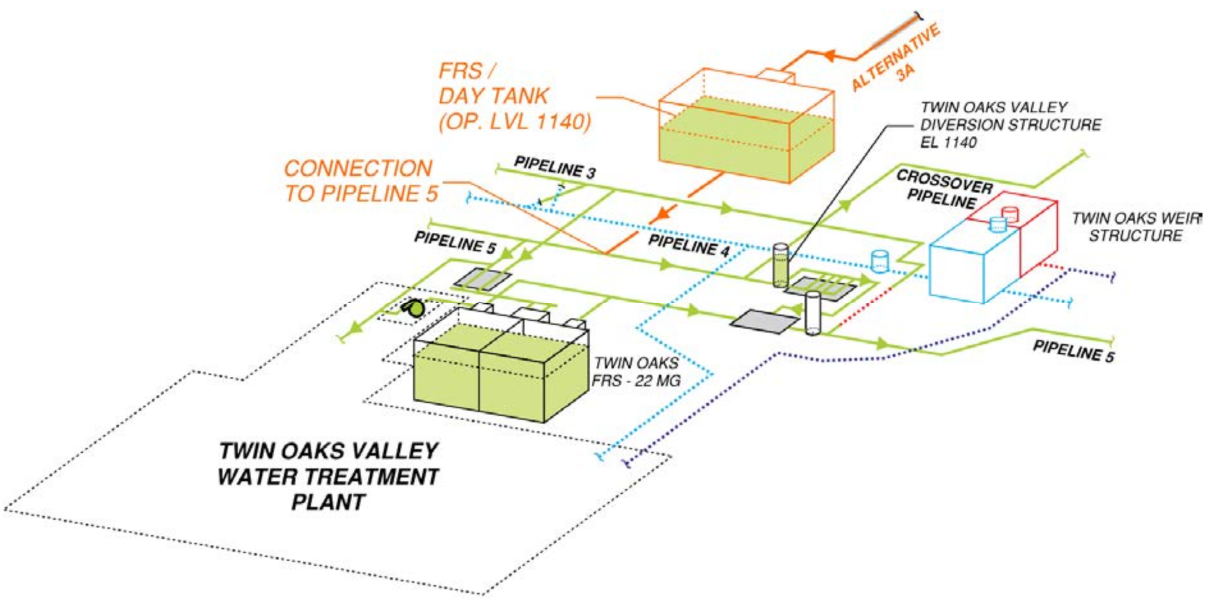

FIGURE 3-1
Alternative 3A Delivery Point Near the TOVDS

- **SVR Delivery Point** – Two connection points in the SVR area were considered in this study for Alternatives 5A and 5C, as shown in Figure 3-2. The first connection point would deliver QSA supplies at the Maximum Normal Pool (MNP) elevation of 764 feet via a new flow control structure and fill chute. This new structure would be located at the proposed tunnel portal above the SVR shoreline approximately 4,300 feet east of the San Vicente Dam. The benefit of this option is that delivering all QSA flows at the MNP elevation would result in lower pumping requirements for the large pump stations that lift supplies from Imperial Valley as compared to the second connection point discussed below. However, the water would then need to be pumped out of the SVR by the San

Vicente Pump Station (SVPS). The SVPS would require power supply upgrades, including a new Variable Frequency Drives (VFD) and backup power, to be able to pump all the QSA supplies out of the SVR.

The second connection point considered would extend the RCS pipeline from the proposed tunnel portal to a point of connection on the San Vicente Pipeline (SVP) at, or near, the east Access Vault Structure; providing a delivery gradient of 920 feet to match the spill elevation of the San Vicente Surge Control Facility (SVSCF). 920 feet was selected as the maximum delivery gradient for preliminary sizing of the RCS pump stations for Alternatives 5A and5C as it is the most conservative delivery gradient. Further, this delivery gradient allows for operation of the SVPS and full or partial RCS deliveries simultaneously, for up to a total flow of 444 cfs in the SVPS (once new power supplies are constructed). During normal operations when the SVPS is not running simultaneously, a lower delivery gradient would be anticipated. At a minimum, the delivery gradient would need to be enough to balance flows through the Rancho Penasquitos Pressure Control and Hydroelectric Facility's (RP PC&HF) control valves and be able to deliver water over the Miramar Vent (El 813).

This connection point allows the bulk of deliveries by the RCS to bypass SVR, thereby eliminating use of the SVPS for delivery of QSA water. Water intended for seasonal storage could be discharged to the SVR via the SVR fill chute by throttling the control valves at the RP PC&HF. This allows filling of the MNP without operating the SVPS.

For both connection options, a new booster pump station located near the Del Dios Valve Vault (DDVV) and a transmission line extending from this pump station to the Twin Oaks would be required to serve untreated water to North County agencies. The RP PC&HF would apply pressure control to attenuate pressure variations using the SVSCF for either RCS connection point.

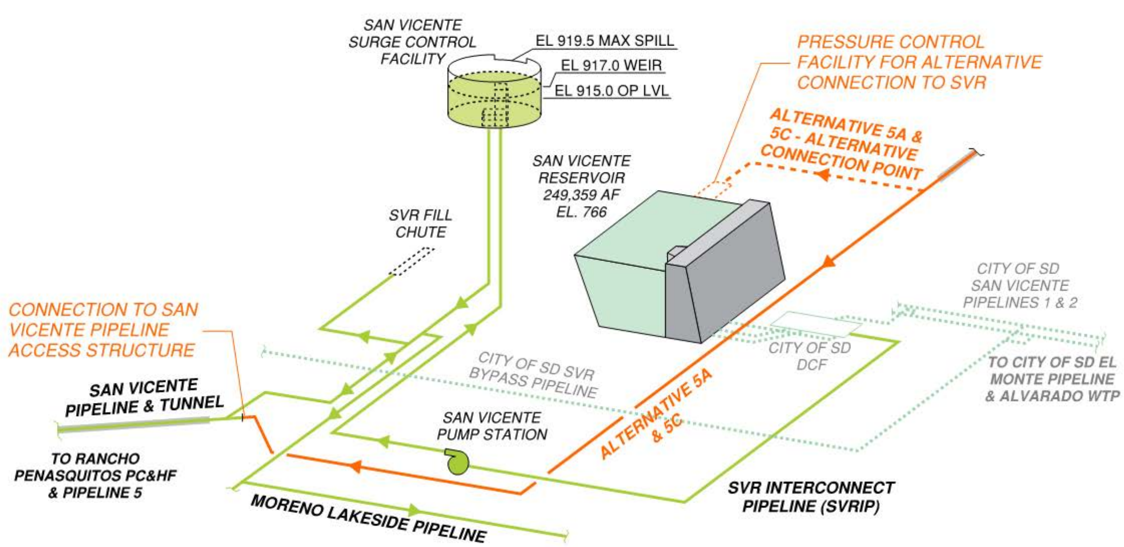

FIGURE 3-2
Delivery Point at SVR for Alternatives 5A,5C

3.3 Aqueduct System Operating Considerations

To obtain a better understanding of the impacts that the RCS would have on operation of existing aqueduct facilities, operations-focused meetings were held with Water Authority operations staff to review concerns associated with integrating the RCS. Developing an operating plan for the RCS that provides an acceptable level of supply reliability to maintain continuous service and allow for safe operations was the overriding consideration expressed by operations staff.

Key takeaways from the operation-focused meetings related to aqueduct operations and new facility requirements are summarized in Table 3-2 below.

TABLE 3-2
Aqueduct System Operating Considerations

■ A new FRS/day tank would be required to provide gravity flow hydraulic control for untreated water supplies that are sent to facilities in the Twin Oaks area. At a minimum, the tank would be sized for approximately 10.5 – 21.0 million gallons for Alternative 3A and approximately 6.0 – 12.0 million gallons for Alternative 5A, 5C. The new FRS/day tank would be used to blend RCS and Metropolitan supplies and provide an operating elevation of 1,140 feet to serve the TOVWTP and Crossover Pipeline.
■ Flows from the RCS delivered at the existing FRS gradient (1,078 feet) would require additional pumping to serve the TOVWTP. The Twin Oaks Valley Raw Water Pump Station is currently powered using on-site diesel generators. Continuous use of this pump station would require connection to the electric power grid. The hydraulic capacity of the Crossover Pipeline would be lowered as a result of the 1,078 feet gradient.
■ Flows from the RCS delivered in excess of daily demands (as seasonal or operating storage) should be stored in SVR. Use of the Olivenhain-Hodges reservoirs for seasonal or operating storage is not preferred by the Water Authority because it would add complexities with pumped-storage operation.
■ The RCS should be capable of a delivery gradient for Alternatives 5A and 5C at SVR of 920 feet to bypass operations of the SVPS during normal RCS operations. If normal RCS operations are interrupted, the SVPS could be needed to supply RCS deliveries as a backup. The SVPS would need power supply upgrades to meet the constant delivery of QSA supplies.
■ A new transmission line is needed for Alternatives 5A and 5C to deliver North County untreated water flows, via a new booster pump station, to the new FRS/day tank in the Twin Oaks area. With the DDVV isolation valve closed, the new transmission line would avoid Pipeline 5 operation conflicts during Olivenhain Reservoir operations. See Section 3.5 for details on the sizing of the new transmission line.
■ The new booster pump station to serve North County demands for Alternatives 5A,5C should be sized to deliver flow at 1,140 ft elevation to the new FRS/day tank in the Twin Oaks area and avoid having to use the Twin Oaks Valley Raw Water Pump Station to further lift flows to the TOVWTP. This is the preferred connection to the SVR from an operations standpoint. See Section 3.5 for details on the sizing of the new pump station.
■ Alternatives 5A and 5C should connect to the San Vicente Pipeline at or near the Access Vault Structure to maintain operational access to the structure. Flows from the RCS could then be delivered at SVSCF elevation. Excess QSA deliveries could be placed in SVR via the existing Reservoir Fill Chute. Maximum Fill Chute flow capacity is about 350 cfs. The Fill Chute would be modified to a Flow Control Valve. SVR could be supplied to the MNP without relying on SVPS.
■ The SVSCF overflow structure would need to be improved although it is not likely that water levels would exceed the overflow level during normal operation.

3.4 Evaluation of Untreated Water Supply Imbalances

An analysis comparing QSA supplies and untreated water demands was performed to identify potential supply imbalances. Supply imbalances occur when QSA supplies exceed untreated water demands. These imbalances would be addressed through use of operational and seasonal storage pools and by the reoperation of existing facilities or development of new facilities to move supplies into and out of such storage.

For the purpose of this study, potential imbalances between QSA supplies and untreated water demands were evaluated on a monthly basis. QSA supplies were limited by the capacity constraints developed in *Chapter 2.0 – Regional Conveyance System Operations and Sizing*. Untreated water demands were analyzed using projections provided by the Water Authority for year 2045. Four water demand and supply scenarios were evaluated to determine an upper range for new seasonal storage requirements, as well as the system's response to lower projected demands and extended annual maintenance shutdowns. This section documents the completion of that evaluation.

3.4.1 Untreated Water Demands

Monthly untreated water demand projections were provided by the Water Authority based on the 2015 Urban Water Management Plan (UWMP) – 2018 Demand Reset and used for determining imbalances between QSA supplies and deliveries to local water treatment plants and storage reservoirs. All untreated water demand projections herein are based on the 2018 Demand Reset of the 2015 UWMP. Should Phase B of this study be authorized, the untreated water demand projections would be updated to reflect the 2020 UWMP. Two untreated water demand projections were considered to provide a range for system response with year 2045 representing the high end of the range and year 2035 representing the low end of the range.

Year 2045 projections were used as the base projection and represent a high-demand projection where untreated water demands are greater than QSA supplies. Table 3-3 presents a monthly breakdown of the year 2045 untreated water demand projections for the "North Demand" and the "South Demand." The North Demand represents the untreated water demand for treatment plants served north of the DDVV, including the City of Oceanside, TOVWTP, the City of Escondido, Vallecitos Water District, Olivenhain Municipal Water District, Ramona Municipal Water District, and the City of Poway. The South Demand includes untreated water demands for Santa Fe Irrigation District, San Dieguito Water District, the City of Del Mar, the City of San Diego (Miramar Water Treatment Plant (WTP), Alvarado WTP, and Lower Otay WTP), Helix Water District, Sweetwater Authority, and East County WTP. This distinction is necessary since Alternatives 5A and 5C deliver QSA flows to SVR and therefore require pumping to meet North Demands with QSA supplies. Alternatively, the North Demands could be served by Metropolitan supplies as they are available. The demands presented include untreated water deliveries to Water Authority-owned treatment facilities at Twin Oaks and the Water Authority's share of treatment facilities at the R.M. Levy Water Treatment Plant (Helix).

To assess a narrower gap between supplies and demands, year 2035 untreated water demands projections were applied to address system response to a low demand water year. The year 2035 untreated water demand projections are 11.4-percent lower than year 2045. Table 3-4 provides a similar monthly breakdown of Member Agency demands and Water Authority owned treatment facilities for Year 2035. For both untreated water demand projections, monthly peaking factors are based on a running average of monthly deliveries using the most two years of data, 2017-2018. The monthly untreated water demands also consider projections for future member agency local supplies as provided in the UWMP.

TABLE 3-3
Untreated Water Demand Projections Year 2045

Untreated Water Demand – FY 2045	North Demand (AF/yr)	South Demand (AF/yr)	Total Demand (AF/yr)
July	15,066	18,298	**33,364**
August	15,806	20,295	**36,102**
September	14,383	17,471	**31,854**
October	13,140	15,922	**29,062**
November	10,724	13,349	**24,073**
December	8,452	12,239	**20,690**
January	6,587	9,580	**16,166**
February	6,626	10,575	**17,201**
March	6,815	10,504	**17,319**
April	11,509	13,938	**25,447**
May	11,979	14,604	**26,583**
June	13,810	16,341	**30,151**
Total	**134,897**	**173,116**	**308,013**

TABLE 3-4
Untreated Water Demand Projections Year 2035

Untreated Water Demand – FY 2035	North Demand (AF/yr)	South Demand (AF/yr)	Total Demand (AF/yr)
July	13,126	16,838	**29,964**
August	13,693	18,655	**32,348**
September	12,099	16,067	**28,167**
October	11,038	14,644	**25,683**
November	9,006	12,274	**21,280**
December	7,022	11,240	**18,263**
January	5,363	8,803	**14,167**
February	5,464	9,702	**15,166**
March	5,582	9,644	**15,226**
April	9,657	12,821	**22,478**
May	10,056	13,432	**23,488**
June	11,595	15,035	**26,630**
Total	**113,701**	**159,156**	**272,857**

3.4.2 Supply Imbalance Evaluations

Four scenarios, known as Case Evaluations, were considered to assess supply imbalances with untreated water demands. Each case assessed a different supply and demand assumption to determine seasonal storage requirements and other infrastructure or operational improvements necessary to implement the RCS.

Cases 1 and 2 considered years 2045 and 2035 projections, respectively, which represent the high and low demand scenarios for untreated water demands. The 2045 and 2035 demand projections were assessed against the RCS operated at maximum capacity during high demand months and then optimized during lower demand months. The goal of Cases 1 and 2 is to eliminate reliance on Metropolitan supplies by storing excess QSA supplies during the winter months such that the same seasonal supplies could be withdrawn from storage to offset summer supplies from Metropolitan.

Cases 3 and 4 provided the same assessment of 2045 and 2035 untreated water demands and QSA supplies but include a continuous 20-day shutdown. Cases 3 and 4 are intended to show how the RCS would respond to an extended maintenance outage. For purposes of this evaluation, the 20-day shutdown was scheduled for January, when monthly projected demands are at their lowest.

As noted in *Chapter 2.0 – Regional Conveyance System Operations and Sizing*, 2,900 AF of storage would be required during the month of May downstream of the AAC to make up for the lack of capacity in the AAC between the New River Siphon and its terminus at the WSM. While 900 AF of storage has been already been planned for in Imperial County, this evaluation has conservatively assumed the full 2,900 AF would be provided in San Diego County in addition to what is being provided in Imperial County. Each case evaluation stores excess QSA supplies during the low demand winter months to provide up to 2,900 AF of supply in May.

Table 3-5 summarizes of each of the supply and demand case evaluations.

TABLE 3-5
Summary of Supply-Demand Assumptions Case Evaluation

Case	Untreated Water Demand AF / yr	QSA Supplies AF / yr	RCS Capacity AF/ yr (cfs)
1	2045 Demands: 308,000	258,364	286,570 (396)
2	2035 Demands: 273,000	258,364	286,570 (396)
3	2045 Demands: 308,000	258,364	270,868 (396)(1)
4	2035 Demands: 273,000	258,364	270,868 (396)(1)

Note:
1. The lower RCS capacity shows a reduced delivery capability resulting from the continuous 20-day system outage for planned maintenance. Pipeline capacity does change because of the shutdown.

The following sections present the findings for the four case evaluations considered.

Case 1 Results
Case 1 results are illustrated in Figure 3-3 and summarized in Table 3-6 below.

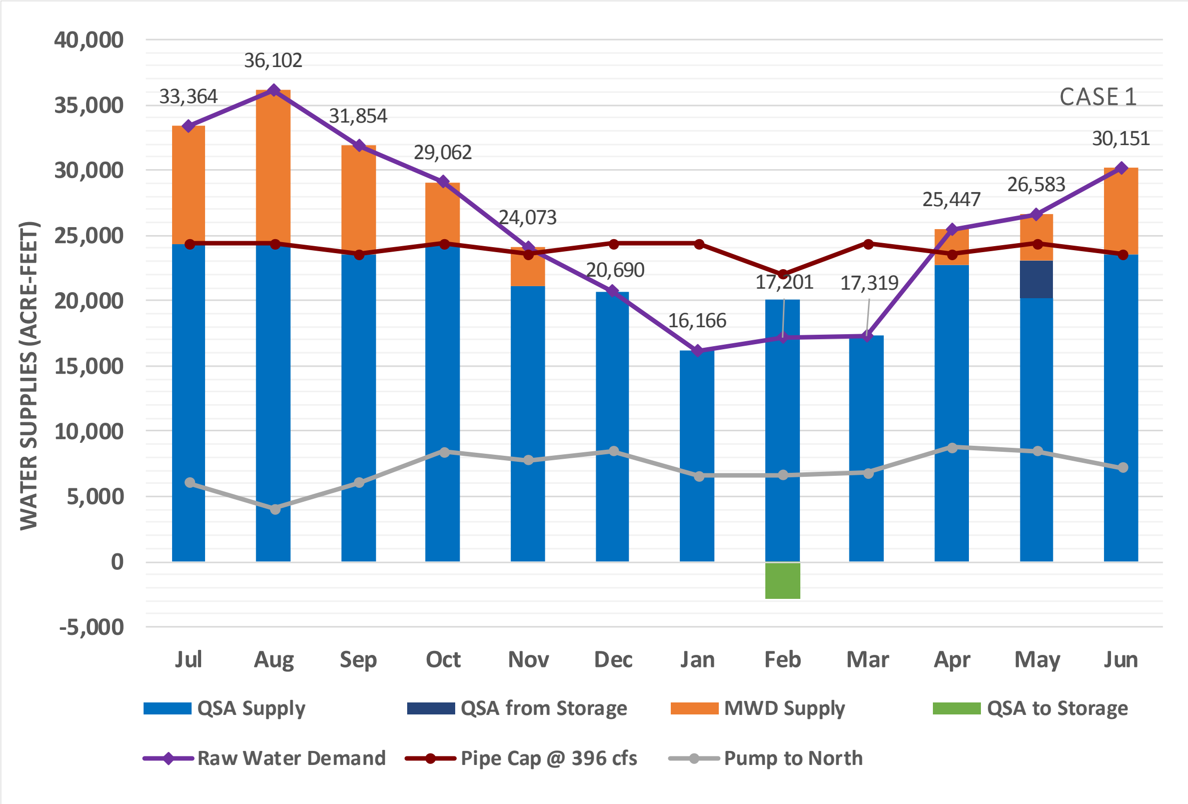

FIGURE 3-3
Case 1 Evaluation Results

Case 1 Summary. RCS facilities would be operated at or near capacity during the higher demand months between June and November. During these higher demand months, Metropolitan supplies would be required to meet the untreated water demands in excess of QSA supplies. During the low demand months, Metropolitan supplies could be curtailed entirely. Excess QSA supplies would be placed into seasonal storage during the lower demand months (such as February) to be used in May, when there is insufficient capacity in the AAC to meet the constant delivery of QSA supplies. For Alternative 5A and 5C, the monthly totals that would need to be pumped north of the DDVV are also illustrated.

TABLE 3-6
Case 1 Evaluation Results

Supply-Demand Summary	Untreated Water Demands (Year 2045)	308,000 AF
	Annual QSA Supplies	258,364 AF
	Annual Metropolitan Supplies	49,649 AF
Max. / Min. Monthly QSA Deliveries	Peak Month QSA Delivery (Jun – Nov)	24,340 AF
	Min. Month QSA Delivery (Jan)	16,170 AF
Storage Impacts	QSA Seasonal Storage Delivery	2,900 AF
Alternative 5A, 5C Pumping Requirements	Annual QSA Supply Pumped North	85,250 AF
	Peak Month QSA Pump North	147 cfs
	Min. Month QSA Pump North	66 cfs

Case 2 Results

Case 2 results are illustrated in Figure 3-4 and summarized in Table 3-7 below.

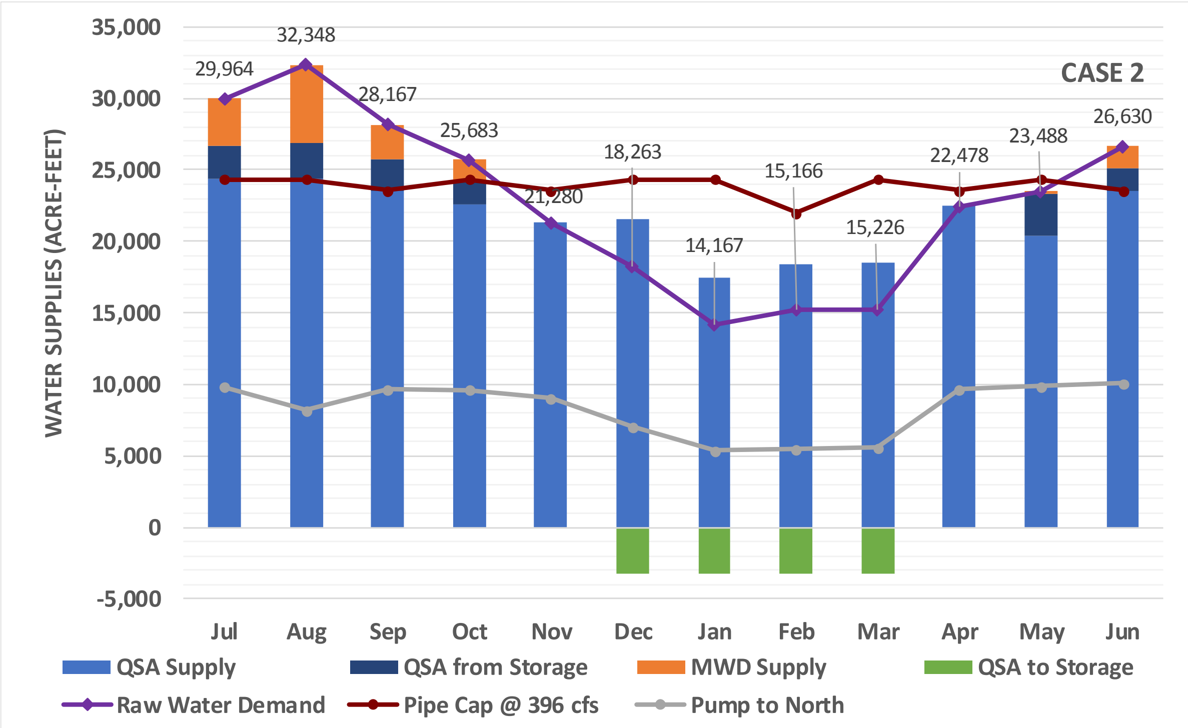

FIGURE 3-4
Case 2 Evaluation Results

Case 2 Summary. Similar to Case 1, the RCS would be operated at or near capacity during the higher demand months between June and September. During these higher demand months, Metropolitan supplies would be required to meet the untreated water demands in excess of QSA supplies. Because of the lower demand year, imbalanced QSA supplies would be placed in seasonal storage during the winter months between December and March. Seasonal storage supplies would then be withdrawn to offset Metropolitan deliveries during the months of May through October. This lower demand scenario would allow for Metropolitan supplies to be curtailed for much of the year but would result in more water needing to be pumped north of the DDVV for Alternatives 5A and 5C.

TABLE 3-7
Case 2 Evaluation Results

Supply-Demand Summary	Untreated Water Demands (Year 2035)	272,860 AF
	Annual QSA Supplies	258,364 AF
	Annual Metropolitan Supplies	14,493 AF
Max. / Min. Monthly QSA Deliveries	Peak Month QSA Delivery (Jun – Sep)	24,340 AF
	Min. Month QSA Delivery (Jan)	17,400 AF
Storage Impacts	QSA Seasonal Storage Delivery	12,980 AF
Alternative 5A, 5C Pumping Requirements	Annual QSA Supply Pumped North	99,200 AF
	Peak Month QSA Pump North	169 cfs
	Min. Month QSA Pump North	87 cfs

Case 3 Results

Case 3 results are illustrated in Figure 3-5 and summarized in Table 3-8 below.

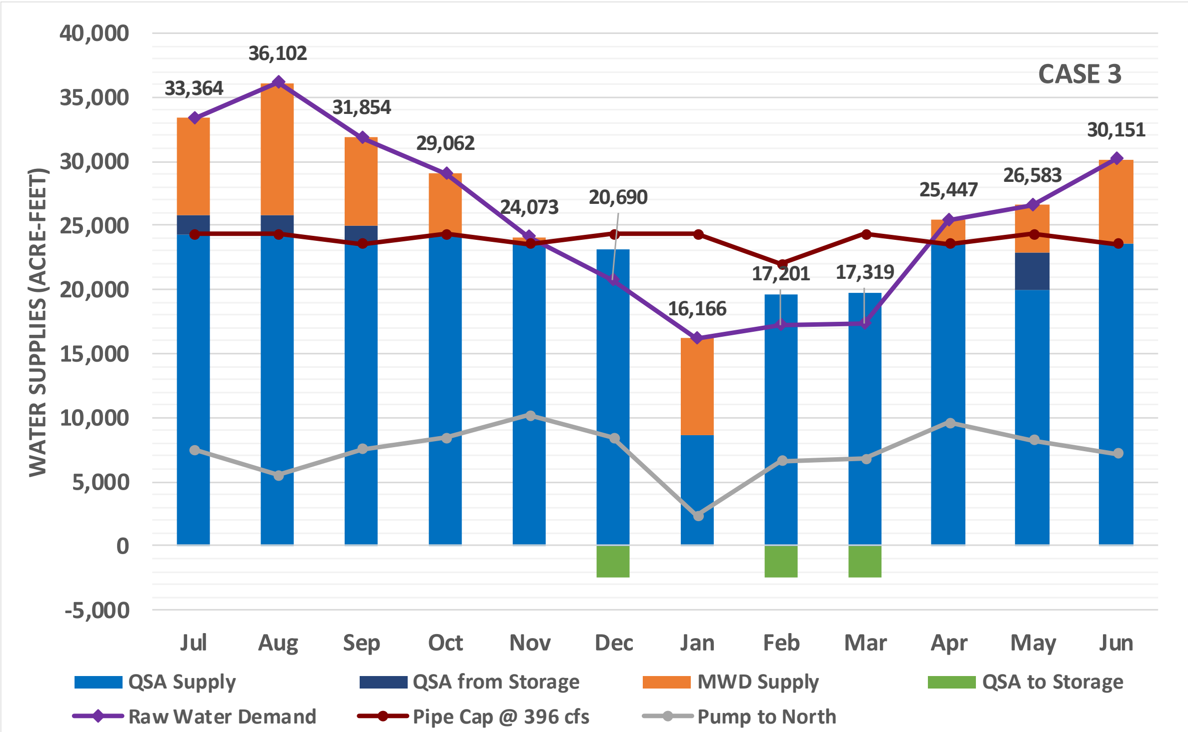

FIGURE 3-5
Case 3 Evaluation Results

Case 3 Summary. While QSA and Metropilitan supplies are the same as Case 1, a 20-day shutdown of QSA spplies in January is included in this evaluation. The RCS would be operated at or near to capacity between April and November, except for May. During the 20-day shutdown in January, Metropolitan supplies would be required. Because of capacity constraints resulting from the shutdown, a small volume of QSA supplies would need to be placed in seasonal storage. This same capacity constraint would cause an increase in peak pumping deliveries north of the DDVV for Alternatives 5A and 5C. Based on the above results, advanced planning would be required to assure seasonal QSA supplies are in storage prior to the high demand months.

TABLE 3-8
Case 3 Evaluation Results

Supply-Demand Summary	Untreated Water Demands (Year 2045)	308,000 AF
	Annual QSA Supplies	258,364 AF
	Annual Metropolitan Supplies	49,650 AF
Max. / Min. Monthly QSA Deliveries	Peak Month QSA Delivery (Jun – Sep)	24,340 AF
	Min. Month QSA Delivery (Jan)	8,635 AF
Storage Impacts	QSA Seasonal Storage Delivery	7,315 AF
Alternatives 5A and 5C Pumping Requirements	Annual QSA Supply Pumped North	88,530 AF
	Peak Month QSA Pump North	171 cfs
	Min. Month QSA Pump North (excluding Jan)	90 cfs

Case 4 Results

Case 4 results are illustrated in Figure 3-6 and summarized in Table 3-9 below.

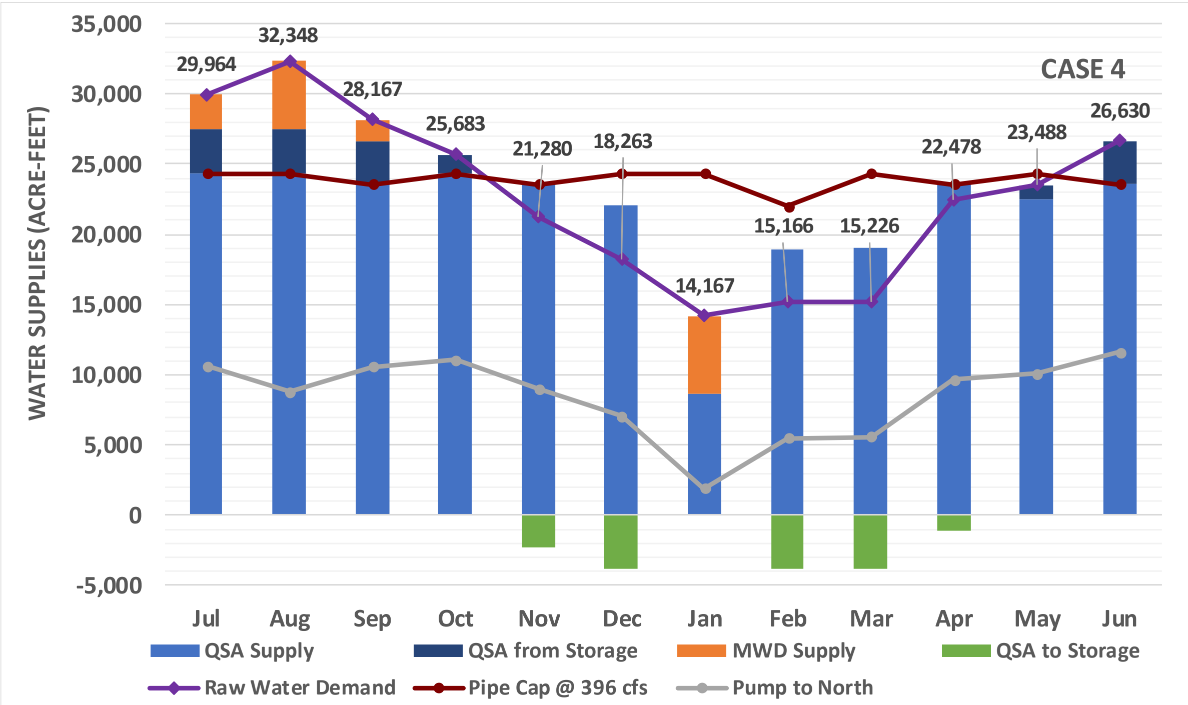

FIGURE 3-6
Case 4 Evaluation Results

Case 4 Summary. Since this low demand scenario also incorporates a 20-day shutdown, the RCS would need to be operated at or near capacity for a longer duration between April and November to address the need for Metropolitan supplies during the January shutdown. Because of both lower demands and capacity constraints resulting from the shutdown, a larger volume of QSA supplies would need to be placed in seasonal storage. During the higher demand months, Metropolitan supplies in conjunction with withdrawals from seasonal stroage would be required to meet the untreated water demands in excess of QSA supplies. Similar to Case 3, advanced planning would be required to assure seasonal QSA supplies are in storage prior to the high demand months.

TABLE 3-9
Case 4 Evaluation Results

Supply-Demand Summary	Untreated Water Demands (Year 2035)	272,860 AF
	Annual QSA Supplies	258,364 AF
	Annual Metropolitan Supplies	14,493 AF
Max. / Min. Monthly QSA Deliveries	Peak Month QSA Delivery (Jun – Nov)	24,340 AF
	Min. Month QSA Delivery (Jan)	8,635 AF
Storage Impacts	RCS Seasonal Storage Delivery	14,717 AF
Alternative 5A, 5C Pumping Requirements	Annual QSA Supply Pumped North	101,280 AF
	Peak Month QSA Pump North	200 cfs
	Min. Month QSA Pump North (excluding Jan)	91 cfs

Summary of Evaluation Findings

Table 3-10 summarizes the results of the four case evaluations.

TABLE 3-10
Summary of the Four Case Evaluations

	Case 1	Case 2	Case 3	Case 4
Supply-Demand Summary	308,000 AF	272,860 AF	308,000 AF	272,860 AF
	258,364 AF	258,364 AF	258,364 AF	258,364 AF
	49,649 AF	14,493 AF	49,650 AF	14,493 AF
Max. / Min. Monthly QSA Deliveries	24,340 AF	24,340 AF	24,340 AF	24,340 AF
	16,170 AF	17,400 AF	8,635 AF	8,635 AF
Storage Impacts	2,900 AF	12,980 AF	7,315 AF	14,717 AF
Alternative 5A, 5C Pumping Requirements	85,250 AF	99,200 AF	88,530 AF	101,280 AF
	147 cfs	169 cfs	171 cfs	200 cfs
	66 cfs	87 cfs	90 cfs	91 cfs

3.5 New Facility Requirements

This section discusses the improvements that would be required to the Water Authority's existing aqueduct system in order to implement the RCS, including a description of the improvements that would be required for Alternatives 3A, 5A, and 5C, and details on the sizing of the potential facilities.

3.5.1 Alternative 3A

The introduction of QSA supplies at the TOVDS based on a delivery gradient of 1,140 feet mimics current imported water supplies from Metropolitan. Accordingly, new facility requirements for Alternative 3A are limited to a new FRS/day tank that would be needed to provide hydraulic control, as well as blend Metropolitan and QSA supplies for gravity delivery to all local water treatment plants and storage reservoirs. The location and size of the FRS/day tank would be determined by the Water Authority as part of a separate engineering evaluation (Second Aqueduct Diversion Complex and Operations Study). A site layout for the FRS/day tank has not been completed to date. To meet the needs only of the RCS, a storage volume equal to 60 minutes of the peak flow through the system is anticipated to be enough. For the purposes of establishing a conservative budget for this study, the FRS / day tank has been assumed to have a storage capacity of 40 MG.

The length of the pipeline to connect the RCS to the FRS/day tank would need to be quantified once the facility has been sited.

3.5.2 Alternatives 5A and 5C

As discussed previously, a new transmission pipeline and pump station would be required to convey the QSA supplies to meet demands north of the DDVV. Table 3-11 summarizes the key parameters of the pipeline and pump station that would be required to transfer water from the SVR to the new FRS/day tank at the TOVDS.

TABLE 3-11
Overview of Aqueduct Improvements for Alternative 5A, 5C

Characteristic	Alternatives 5A, 5C
Pipeline, Miles	12.5
Pipeline Diameter, Inches	72
Total Pumping Head, Feet	490
Pump Stations, Each	1
Forebay/Suction Storage Tank, Capacity	1 (6 MG)

3.5.3 Description of Improvements

As shown schematically on Figure 3-7, approximately 12.5 miles of new 72-inch diameter welded steel pipeline would need to be constructed in an alignment generally parallel to the Water Authority's Second Aqueduct. This new pipeline would begin at a connection to Pipeline 5 just south of the DDVV, which operates at a static hydraulic gradient of 813 feet based on the invert elevation of the Miramar Hill vent tower. The pipeline would extend north to a new pump station that would be required to lift QSA supplies to the new FRS/day tank in the Twin Oaks area.

This study has assumed that the pump station would be located at a site that would take advantage of the available 813-foot suction side gradient, which could be achieved with potential sites north of Escondido Creek. The new pump station, and associated forebay tank, would provide approximately 490 feet of lift to convey the water to the new FRS/day tank at the TOVWTP at a hydraulic grade of 1,140 feet.

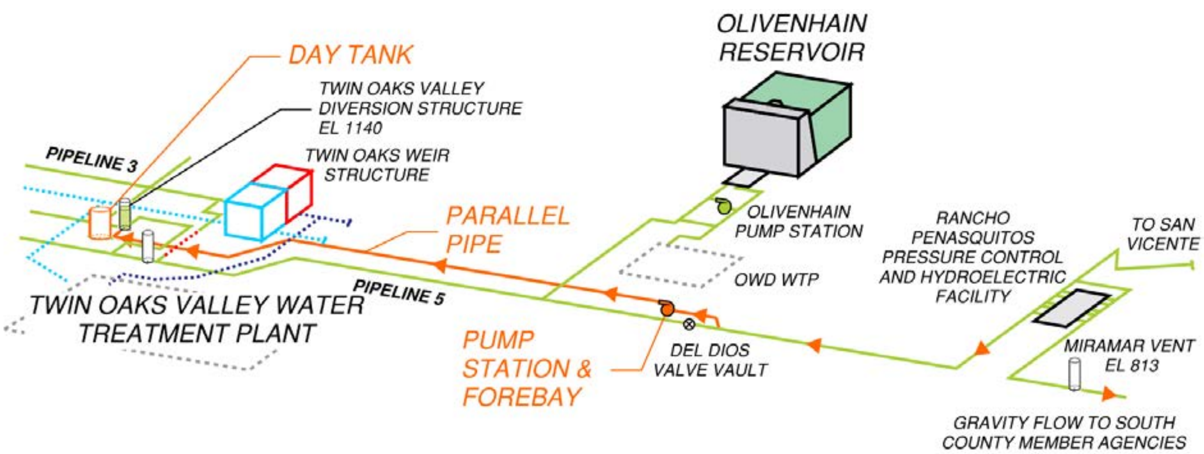

FIGURE 3-7
Aqueduct Improvements Schematic

From the DDVV, it is assumed that the new pipeline would head north parallel to Pipelines 3, 4, or 5 for 12.5 miles. The alignment would cross its lowest point at Escondido Creek, approximately elevation 260, and would peak crossing hilltops north of the community of San Elijo Hills. Additional right-of-way acquisition is anticipated for most, if not all, of this parallel pipeline.

For this conceptual study, the new pipeline has been noted as running parallel to Pipelines 3, 4, or 5 for 12.5 miles. However, a preliminary review of possible routes for this pipeline reveals several challenges for an alignment that is parallel to these existing pipelines the entire way, including elevation concerns when following Pipeline 5 or crossing Lake San Marcos while following Pipelines 3 or 4. This study was tasked with providing a schematic representation of the new pipeline to the Twin Oaks area to serve as the basis for the development of budgetary costs. A more detailed alignment evaluation would be required to refine this conceptual alignment.

3.5.4 Pipe Sizing

This section documents the evaluations completed to size the pipeline conveying water north from the DDVV to the TOVDS. The sizing information serves as the basis for the cost opinion, which is further described in *Chapter 6 – Risk, Cost Opinions, and Economic Comparison*. Criteria used for the preliminary hydraulic analysis and pipe sizing included flow rate, hydraulic losses, and permissible velocities.

Design Flow Rate. The system has been conceptually designed for a maximum flow rate of 220 cubic feet per second (cfs). This flowrate is based on the low untreated water demand scenario for peak pump to the north requirements and includes a 10 percent increase in pumping capacity to account for maintenance and unexpected pumping outages.

Hydraulic Losses. Manning's equation was used to calculate the hydraulic grade line (HGL) and friction losses throughout the system.

Manning's Equation: $H_{ft} = 4.66\ n^2\ L_{ft}\ Q_{cfs}^2\ /\ D_{ft}^{5.33}$

Where:

H = Friction loss, feet
n = Manning's n-value (0.012 for cement mortar lining)
D = Pipeline internal diameter, feet
L = Pipeline length, feet
Q = Flow rate, cubic feet per second

Permissible Velocities. Transmission pipelines are typically designed to have velocities of six to eight feet per second (fps) at the design flow rate to minimize surge potential and cavitation damage to the pipeline but could have velocities as high as 10 fps. For short periods of time under emergency operating conditions, a maximum pipeline velocity of up to 15 fps could also be considered.

Table 3-12 summarizes the key characteristics of the new transmission pipeline.

TABLE 3-12
Key Pipeline Design Criteria

Criteria	Item
Inner Diameter, Inches	72
Velocity, Feet per Second	7.8
Average Head Loss Per Mile, Feet	12.2

3.5.5 Pump Stations

This section presents the operating criteria and facility descriptions for the pump station proposed to convey water north from the DDVV location to the TOVWTP. The information serves as the basis for the cost opinions presented in *Chapter 6 – Risk, Cost Opinions, and Economic Comparison.*

The flow rate required to be pumped would vary throughout the year to meet varying demand scenarios. The design criteria presented below is based on the average flow for the peak month. A 10-percent increase in pumping capacity has been provided to account for maintenance and unexpected pumping outages. Table 3-13 presents the key design criteria for the pump station.

TABLE 3-13
Pump Station Design Criteria

Item	Criteria
Number of Pumps	3 duty, 1 standby
Pump Type	Vertical turbine, multiple stage, variable frequency drives
Rated Discharge, each pump	74 cfs
Rated Total Head	490 ft
Pump Efficiency (assumed)	85%
Rated Horsepower (hp), each pump	5,000 hp
Motor Type	Vertical, synchronous
Forebay/Suction Storage Tank, capacity	6 million gallons

The pump station attributes described above are consistent with the other pump stations being considered for the RCS. These attributes are also consistent with the SVPS, which has three 7,000 hp variable speed pumps that operate up to 440 cfs. Further evaluation of these specifics would be warranted if the project is advanced beyond the scope of this current study, including:

- Number of Pumps: The number of duty pumps should be further evaluated.
- Pump Type: Other pump types besides vertical turbine pumps should be considered. For instance, the Water Authority's SVPS uses horizontal split case pumps which could also provide benefits at this pump station.
- Motor Controller Type: Various combinations of motor controller types should be considered for each of the pumps at each pump station, including full voltage starters, soft starters, and variable frequency drives.

Phase B of this study, should it be authorized, would include the development of a typical conceptual building layout of the pump station. As part of that task, the attributes discussed above would continue to be refined.

Consistent with the other pump stations considered for the RCS, this pump station would include the following features: 1) a reinforced concrete structure with a steel framed superstructure and metal wall panels, 2) a traveling bridge crane and a gantry crane, and 3) typical auxiliary electrical and mechanical systems meeting the Water Authority's design standards. As this pump station is expected to be operated on a continuous basis, backup power generators would be anticipated to be required.

Further evaluation of system hydraulics would be required during subsequent phases of design, including a transient analysis, to mitigate any surge potential concerns resulting from this pump station. The transient analysis might result in an after bay, vent pipes, or surge control tanks. At this conceptual stage, an allowance has been included for surge control facilities within the pump station in the budgetary cost opinion documented in *Chapter 6 – Risk, Cost Opinions, and Economic Comparison*.

The pump station would be provided with a forebay, or suction side storage tank, that would be sized for normal startup and shutdown of the pump station and for unscheduled outages of individual pumps or the entire pump station. The forebay is not intended to provide operational storage for the RCS. The size and location of the forebay would be further evaluated as design concepts are refined.

Table 3-14 presents the forebay design criteria, which are based on the recommendations from previous studies. These recommendations would be further evaluated as part of the potential Phase B of this study.

TABLE 3-14
Forebay Design Criteria

Item	Criteria
Operational Storage	60 minutes at 100 percent pumped discharge
Operational Storage required for 220 cfs	6 million gallons
Storage Tank Dimensions	Height = 30-feet; Diameter = 210-feet
Type	Prestressed concrete tank

3.6 Impacts to SVR Operations

During conversations regarding the integration of the RCS, the City of San Diego (City) provided the following documents: the San Vicente Annual Operating Plan, Fiscal Year 2019 and the SVR Regulation Manual for the Expanded SVR. These documents were reviewed to identify potential impacts to the operation of the SVR. Further coordination would be required with the City in future phases of work, should they be authorized, to ensure the cycling of the Water Authority's QSA supplies through SVR would not impact the operation of the reservoir.

The following section describes the current operation of the SVR and notes where any potential impacts would be anticipated by the implementation of the RCS.

3.6.1 Reservoir Operations

The SVR is primarily operated as a water supply reservoir for the City and the Water Authority. The goals of the reservoir are to provide the Water Authority and the City with emergency storage, carryover and seasonal operation storage, and to maximize the ability to use local runoff and imported water storage programs. The reservoir stores water from two primary sources, imported water that the Water Authority receives and conveys from Metropolitan and local water runoff.

The primary means of filling the SVR is by gravity flow from the Water Authority's First or Second Aqueduct system, depending on the system operations. Both the First and Second Aqueducts are supplied by imported water from Metropolitan. The City is also able to release water from its Sutherland Reservoir into the SVR via the San Vicente Creek.

The Water Authority drafts water from the SVR by using the SVPS to lift water to the SVSCF. The water then flows from the SVSCF to the Second Aqueduct and/or the Moreno Lakeside Pipeline for delivery to the Water Authority's member agency turnouts. The City is able to draft water from the San Vicente Reservoir by gravity to the City's San Vicente Pipeline Numbers 1 and 2. The water then supplies either the Alvarado Water Treatment Plant or the El Capitan Reservoir.

3.6.2 Storage Capacity

While the City owns and operates the SVR and Dam, the Water Authority owns capacity in the reservoir equal to the increase in volume associated with the recent dam raise. Table 3-15 presents a summary of the water capacity ownership in the SVR.

TABLE 3-15
San Vicente Reservoir Capacity Ownership

Agency	Useable Storage Capacity (AF)	Dead Storage Capacity (AF)	Total Storage Capacity (AF)
Water Authority	154,384	3,279	157,663
City of San Diego	89,746	1,949	91,695
Total	**244,130**	**5,228**	**249,358**

The useable storage capacity is separated into the following storage pools: 1) Regional Emergency Pool (Water Authority), 2) Local Emergency Pool (City), 3) Carryover Pool (City and Water Authority), 4) Operational Pool (City and Water Authority), and 5) Flood Control Capacity (City and Water Authority).

Each year, the Water Authority and City develop a "guide curve" based on maintaining their target storage volumes for their individual storage pools within the reservoir. Figure 3-9, obtained from the 2019 Annual Operating Plan, presents the Water Authority's guide curve for the 2019 fiscal year. These represent the target storage volumes for each of the storage pools.

The City and Water Authority cooperate to provide an impound capacity (Flood Control Capacity) of 35,000 AF below the spillway crest by October 1st of every year. This impound capacity is provided to meet the reservoir's operating parameters cited in the Federal Emergency Management Agency flood plain studies. The share of impound capacity assigned to the City and Water Authority is determined each year. The runoff that enters the San Vicente Reservoir as part of the impound capacity is City-owned.

As shown on Figure 3-8, the Water Authority has approximately 20,000 AF of capacity available in 2019, less its impound capacity commitments.

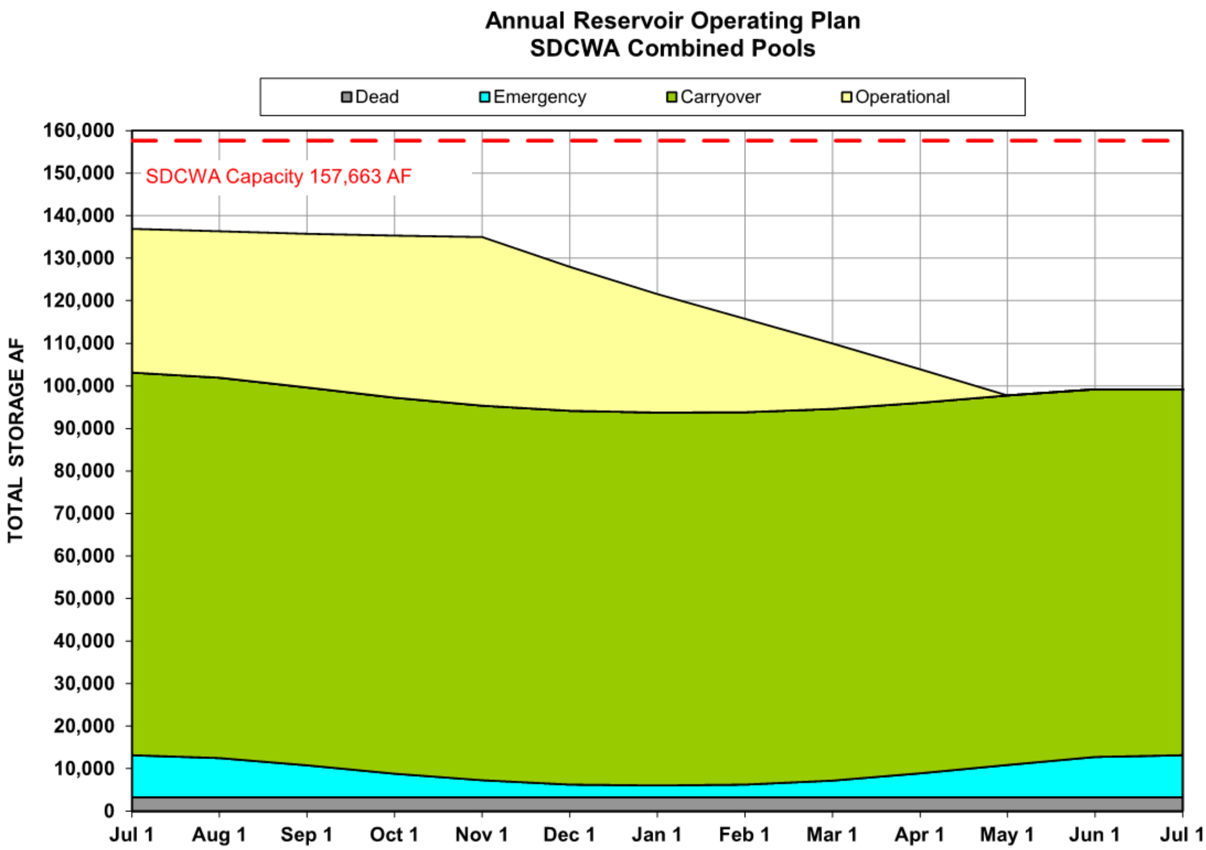

FIGURE 3-8
SDCWA San Vicente Reservoir 2019 Storage Volume Targets (San Vicente Reservoir Annual Operating Plan, Fiscal Year 2019)

The combined City and Water Authority guide curve for 2019 is presented on Figure 3-9, as obtained from the *2019 Annual Operating Plan*. As shown, the SVR has approximately 31,500 AF of capacity available on October 1, in addition to the 35,000 AF available to meet the impound capacity requirements.

Based on the volumes expected to be stored in the SVR, implementation of the RCS is not anticipated to impact the operation of the SVR for Alternative 3A or Alternatives 5A and 5C if the water is delivered to the San Vicente Pipeline Access Vault Structure. The RCS would be supplying the same QSA water currently delivered plus any additional seasonal storage requirements.

If Alternatives 5A or 5C were to discharge directly into the reservoir, then it is likely the operation of the reservoir would be impacted by the cycling of the Water Authority's full QSA alotment. The impact this connection point would have on reservoir operations would be further evaluated under Phase B, should it be authorized.

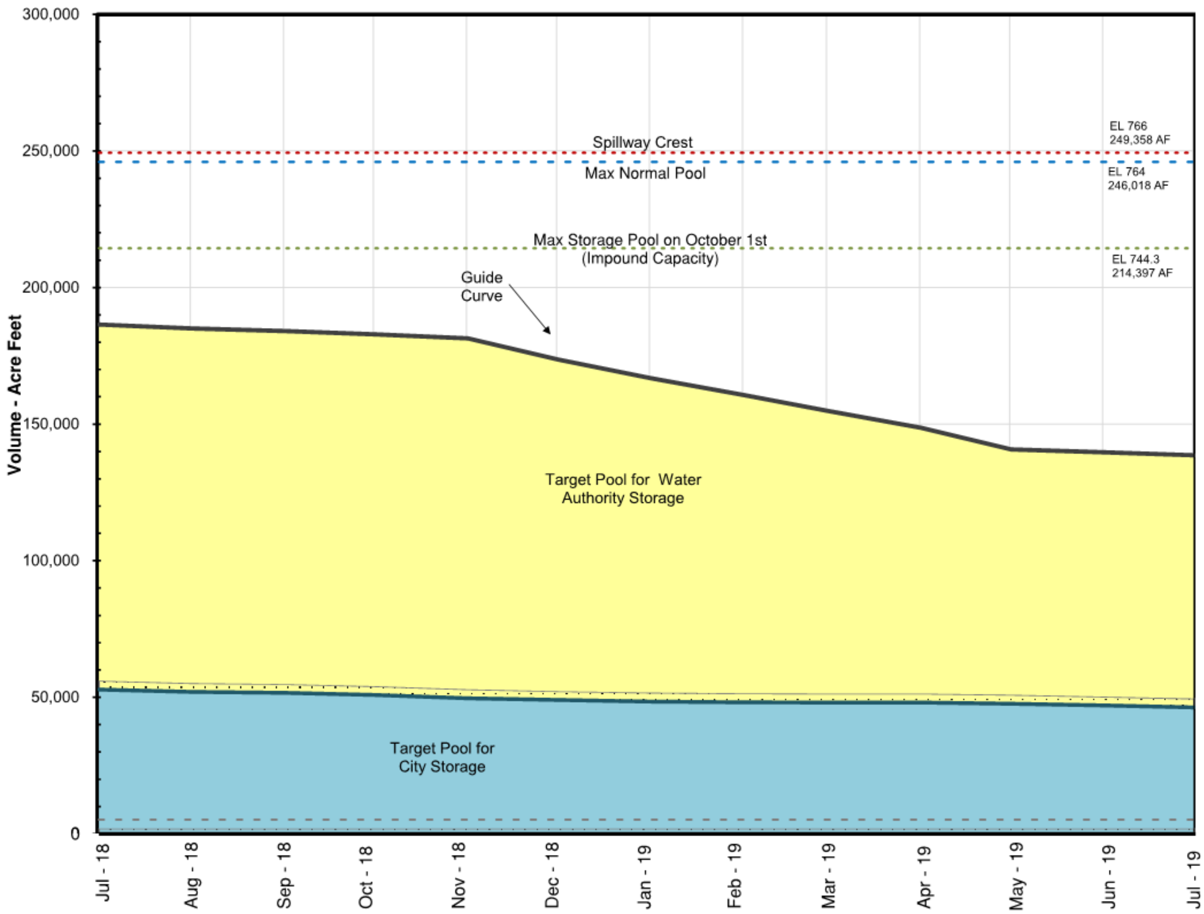

FIGURE 3-9
Combined San Vicente Reservoir 2019 Storage Volume Targets (San Vicente Reservoir Annual Operating Plan, Fiscal Year 2019)

3.6.3 Water Quality

While the RCS proposes difference connection points, the Water Authority's QSA supplies ultimately all come from the same source, the Colorado River. As described in *Chapter 4 – Treatment, Blending and Brine Management Options*, the RCS proposes to treat the QSA supplies for salinity prior to San Diego County to provide a similar water quality to that currently received by Metropolitan delivered water.

Turnover is a requirement to maintain water quality in reservoirs. Seasonal storage of QSA water from the RCS appears to be possible within the current capacity of the reservoir and would therefore be within the reservoir's current ability to turn over the water (import and drafting). Storage of QSA water delivered by the RCS would generally follow the same seasonal patterns as currently seen at SVR, where water would be stored in the winter when demands are low and withdrawn during the summer when demands are high.

3.6.4 Impacts on Existing SVR Facilities

As discussed in Table 3-2, the existing SVR fill chute has a maximum capacity of approximately 350 cfs. If the RCS connects to the San Vicente Pipeline Access Vault Structure as currently envisioned, then only the volume of water required for seasonal storage would be conveyed through the fill chute. The maximum flow rate to seasonal storage anticipated from the four case evaluations considered was between 80 and 90 cfs, which is within the capabilities of the existing chute. The fill chute would need to be changed to a flow control valve. However, if a delivery strategy that involves all the QSA supplies discharging directly to SVR before cycling out was ultimately selected, the capacity of the existing fill chute is anticipated to be undersized. Further, the SVPS would require power supply upgrades in order to convey all the QSA supplies out of the SVR.

Should the project progress, it will be imperative that the Water Authority closely monitors any planned or potential projects at the SVR that would impact the proposed RCS connection.

3.7 Impacts to Other Facility Operations

The Water Authority's *2020 Annual Operating Plan* and *2013 Master Plan Update* were reviewed to determine potential impacts the introduction of the RCS might have on the operation of the Water Authority's other existing facilities. The only other facility, besides the SVR, that a major operational impact would be anticipated for was at the RP PC&HF. This section describes the current operation of this facility and how it would be affected by the introduction of QSA deliveries.

3.7.1 Rancho Penasquitos Pressure Control and Hydro Facility Operations

The RP PC&HF serves several functions within the Water Authority's existing aqueduct system. First, the facility controls flow coming out of the SVR by way of the SVSCF and can bifurcate the flow both northward and southward in Pipeline No. 5. Flows to the south are controlled by pressure control (sleeve) valves, while flows to the north are controlled by isolating Pipeline No. 5 at the DDVV. In this function, the RP PC&HF does not typically generate hydroelectric power due to the difficulty of operating the units at this hydraulic gradient.

The second function the RP PC&HF serves is to generate hydroelectric power from the flows conveyed south in Pipeline No. 5 from the Twin Oaks area. The facility has a 4.5-megawatt turbine generator that could operate year-round.

Since Alternative 3A would discharge QSA flows in the Twin Oaks area, the flow to meet south county demands would be similar to the existing operation of the RP PC&HF, which would still be able to generate hydroelectric power as done today. However, Alternatives 5A and 5C as currently described would connect to the SVP on the SVR side of the RP PC&HF. Depending on the delivery gradient, the ability for the RP PC&HF to generate hydroelectric power could be impacted.

Regardless of the delivery location, the RP PC&HF would still be able to control pressure and flows without modifications.

It should be noted that impacts to the Water Authority's Emergency Storage Program operation was outside the scope of this study and would require evaluation during subsequent phases of work.

3.8 Conclusion

New Facility Requirements. The introduction of new supply alternatives from the RCS could be accomplished with modifications to the current aqueduct operating strategies, along with the implementation of new facilities. Alternative 3A is proposed to terminate at the new FRS/day tank, positioned near the TOVWTP, while Alternatives 5A and 5C are proposed to terminate either directly to the SVR or to the SVP at, or near, the existing Access Vault Structure. The later connection point is the preferred connection point for Alternatives 5A and 5C by the Water Authority's operations staff and serves as the basis for this planning study.

Impacts to San Vicente Reservoir Operations. The Water Authority's operations staff also prefers that QSA flows delivered in excess of daily demands (as seasonal or operating storage) be stored in SVR. Four scenarios considering two different demand projections (years 2035 and 2045) were evaluated to determine the range of volumes that could be required for seasonal storage and to provide the basis for the sizing of new facilities. Additional scenarios, such as multiple, consecutive wet years, should be evaluated during subsequent phases of work. Should Phase B move forward, additional coordination will be required with both the Water Authority operations staff and City of San Diego to further coordinate potential introduction into SVR so as not to impact existing operations or future projects.

Impacts to Other Facility Operations. The new FRS/day Tank would be sited and sized by the Water Authority as part of a separate engineering evaluation (Second Aqueduct Diversion Complex and Operations Study). To meet only the needs of the RCS, two concepts for sizing could be to 1) provide one hour of storage at the peak flow rate of the RCS or 2) provide one hour of storage above and below the tank mid-level. However, the FRS/day tank would also provide hydraulic flow control of the untreated water deliveries throughout the Water Authority service area and for blending with Metropolitan supplies. For the purposes of establishing a conservative budget, this study assumed the tank would provide 40 MG of storage.

Table 3-16 summarizes the new facilities and improvements that are anticipated for the integration of the RCS for Alternative 3A and for Alternatives 5A and 5C.

TABLE 3-16
Summary of New Facilities and Improvements Required to Integrate the RCS by Alternative

Alternative	New Facilities / Improvements
Alternative 3A	• FRS / Day Tank in the Twin Oaks area
Alternatives 5A and 5C	• FRS / Day Tank in the Twin Oaks area • 12.5 miles of transmission pipeline, pump station and forebay • Improvements to the SVSCF overflow structure • Modification of the SVR fill chute to a flow control valve

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4

TREATMENT, BLENDING AND BRINE MANAGEMENT OPTIONS

4.0 Chapter 4 Treatment, Blending and Brine Management Options

4.1 Introduction

4.1.1 Overview

This chapter documents the evaluations completed to assess the treatment, blending, and brine management strategies for the RCS. The strategies considered were identified through a collaboration between Water Authority and Black & Veatch.

4.1.2 Chapter Organization

Key operating parameters and project components affecting alternative decisions for the RCS are summarized below and discussed in the following chapter sections.

Summary of Previous Studies and Assumptions – This section summarizes treatment and brine management alternatives that were considered during previous studies beginning with the *1996 Water Transfer Study*. Further analysis and refinements to the treatment and brine management alternatives and associated costs were conducted under the *2013 Master Plan Update* and the *2017 Cost Update.*

Blending Strategies – This section reviews blending strategies to reduce the salinity of the Colorado River water to meet a target total dissolved solids (TDS) goal of 500 mg/L. The evaluation included revised blending analysis for SVR and TOVWTP.

Location of Treatment Facilities – This section provides treatment facility location information associated with Imperial County and San Diego County.

Treatment Facility Conceptual Design Criteria – This section provides conceptual design criteria associated with the major process components of the overall treatment facilities. This section also provides an alternatives comparison of reverse osmosis (RO) vs. electrodialysis reversal (EDR).

Treatment Facility Conceptual Site Layouts – This section provides conceptual treatment facility layouts and provides details for the various treatment components.

Brine Management Options – This section summarizes two primary alternative brine management options categorized as 1) Releasee to the Salton Sea (direct, tributary, and constructed wetlands) and 2) Other options (i.e. brine volume minimization followed by release to evaporation ponds).

Regulatory Requirements – This section describes the regulatory and permitting considerations associated with various Salton Sea-related alternatives.

Brine Management Facilities - This section presents the brine management facilities required for each potential treatment location in Imperial and San Diego Counties.

4.1.3 Summary of Previous Studies and Assumptions

1996 Water Transfer Study

This study considered various (1) water treatment options, (2) treatment locations in Imperial and San Diego Counties, (3) influent water quality data, and (4) brine management methods. Cost estimates were developed for each option. The respective ranges for these categories are shown below. For a more specific breakdown of the different alternative options, refer to the previous studies.

- Regional Conveyance Water: 432,600 - 496,200 AFY (386 – 443 mgd)
- Influent TDS: 710-885 mg/L
- Effluent Flow Rate: 400,000 AFY (357 mgd)
- Effluent TDS: 500-724 mg/L
- Quantity of Brine: 24,000-58,800 AFY (21.4 – 52.5 mgd)
- Brine Concentration: 3,500 mg/L
- Estimated Size of Evaporation Pond: 3,000 acres
- Cost of Evaporation Pond in 1996 dollars: $400 million
- Additional Brine Management Options: Conveyance via canal/pipeline to Yuma Desalter Drain, Gulf of California, local sewers, South Bay Ocean Outfall, San Diego River or to the Salton Sea via the Alamo River

2013 Master Plan Update

Several updates were made to the Regional Conveyance Study between the release of the 1996 and 2013 reports. As summarized below, the volume of water supplied from the Colorado River was decreased and expected influent TDS range was updated. An option for blending untreated streams of water was added to the list of available possibilities and the RO recovery rate increased to produce a concentrate with a TDS of 5,611 mg/L. The changes in flow rate and brine concentration affected the cost and size of the evaporation ponds as well.

- Regional Conveyance Water: 277,700 AFY (249 mgd)
- Influent TDS: 600-879 mg/L
- Streams Included in Untreated Blending: Regional Conveyance System, Rainfall, City Water
- Final Blending Flow Rate: 361,700 AFY (323 mgd)
- Final Blending TDS Range: 500-700 mg/L

- Treated Effluent Flow Rate: 259,000 AFY (231 mgd)
- Treated Effluent TDS: max. of 500 mg/L
- Quantity of Brine: 23,000 AFY (20.6 mgd)
- Brine Concentration: 5,611 mg/L
- Estimated Size of Evaporation Pond: 3,700 acres
- Cost of Evaporation Pond in 2012 dollars: $860 million
- Additional Brine Management Options: evaporation ponds or release to the Salton Sea

2017 Cost Update

Several updates were made to the RCS between the release of the 2013 and 2017 reports. An option to increase the pipeline capacity was studied to potentially accommodate use by the City of San Diego, which previously owned capacity rights in the AAC. As such, this option also impacted the cost and size of the evaporation ponds.

- Influent Flow Rate: 432,600 AFY (386 mgd)
- Influent TDS: 600 - 879 mg/L
- Effluent Flow Rate: 400,000 AFY (357 mgd)
- Effluent TDS: max. of 500 mg/L
- Quantity of Brine: 32,700 AFY (29.1 mgd)
- Brine Concentration: 5,611 mg/L
- Estimated Size of Evaporation Pond: 5,250 AFY
- Cost of Evaporation Pond in 2017 dollars: $1 billion

4.2 Blending Strategies

This section summarizes blending options that were evaluated to address the Colorado River's water quality for delivery to the SVR or TOVWTP. The purpose of the evaluation was to determine if blending alone, thereby forgoing the need for treatment, could reduce the TDS of the Colorado River water below a target concentration of 500 mg/L. The TDS target is based on the typical TDS of water delivered to the Water Authority via MWD and meets the secondary contaminant level (SMCL) established under State and United States Environmental Protection Agency (USEPA) drinking water standards.

4.2.1 San Vicente Reservoir

Previous Strategy

The *2013 Master Plan Update* identified several anticipated flow sources for blending in the SVR, as depicted in Figure 4-1. Flow sources included: rainfall/inflow, RCS and the City of San Diego Purification Project (Pure Water - future). A simple mass balance, as provided in

Table 4-1, shows this approach would not consistently meet the maximum acceptable target TDS of 500 mg/L.

TABLE 4-1
SVR Blending Strategy Mass Balance based on 2013 Master Plan

Flow Source	Flow Rate (AFY)	Min. TDS (mg/L)	Max. TDS (mg/L)
Regional Conveyance System	277,700	600	879
Rainfall/inflow	8,000	150	400
Pure Water	76,000	80	80
Total Flow	**361,700**		
Weighted Average	ppm	500	700

Updated Strategy

Due to updates in flow sources and flow rates, a new analysis was carried out to re-evaluate the previous blending strategy for SVR. Since the *2013 Master Plan Update* was prepared, the delivery location of purified water from the City of San Diego Water Purification Project changed from SVR to Miramar Reservoir; therefore the 76,000 AFY of City water is no longer considered as part of this blending option. The flow available from the Colorado River was also updated from 277,700 AFY to 279,500 AFY as discussed in *Chapter 2 – Regional Conveyance System Operations and Sizing*. The updated flow blending schematic and mass balance are provided in Figure 4-2 and Table 4-2, respectively. As shown, the updated blending strategy increased the TDS range and therefore does not meet the acceptable target TDS of 500 mg/l.

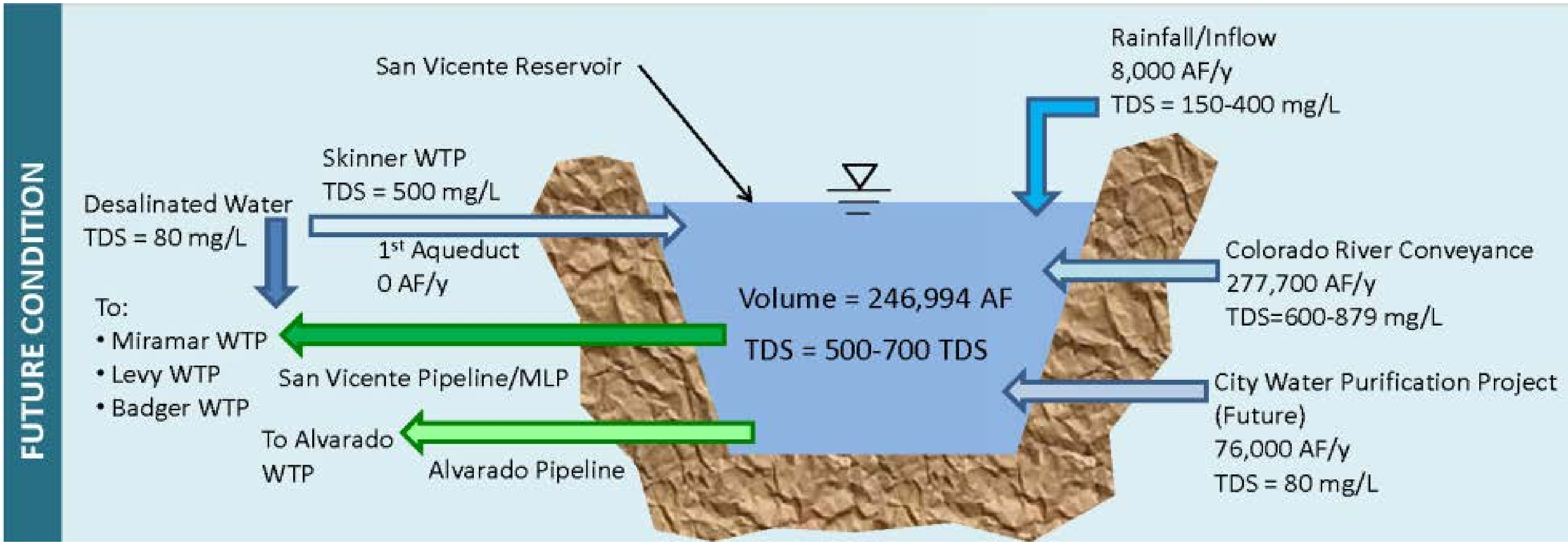

FIGURE 4-1
SVR Blending Strategy (*2013 Master Plan Update*)

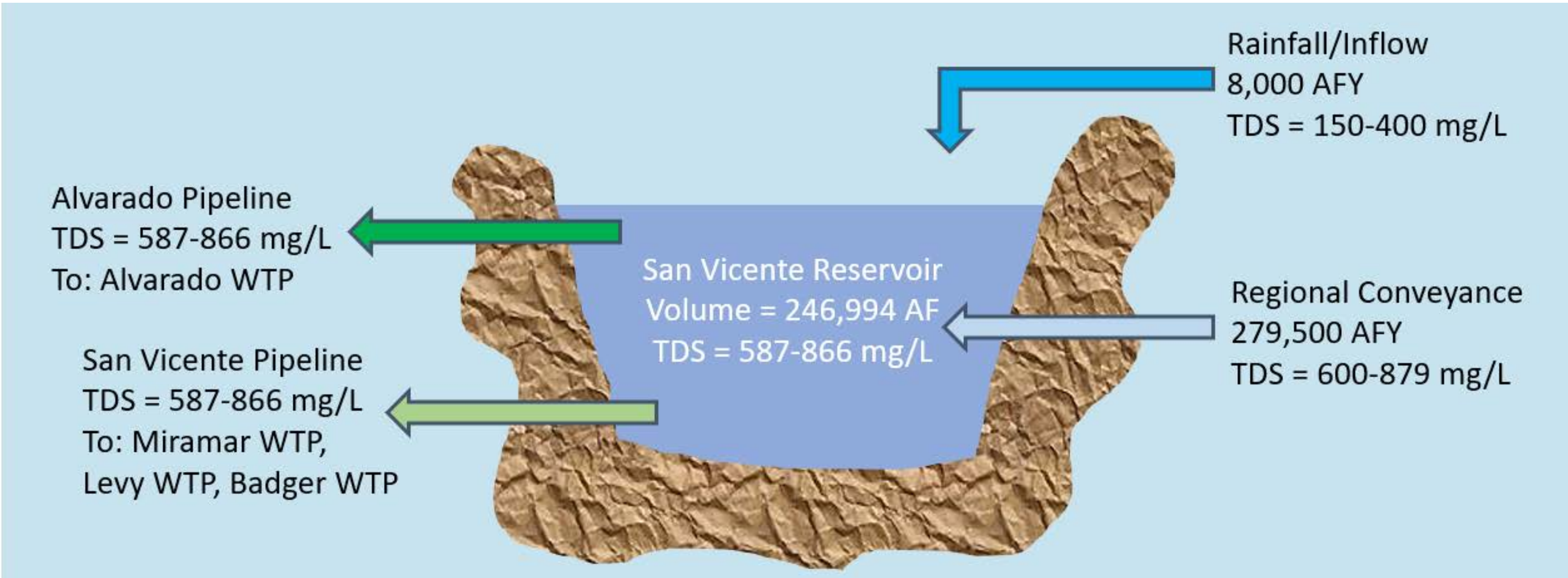

FIGURE 4-2
Updated SVR Blending Strategy

TABLE 4-2
Updated SVR Blending Strategy Mass Balance

Flow Source	Flow Rate (AFY)	Min. TDS (mg/L)	Max. TDS (mg/L)
Regional Conveyance System	279,500	600	879
Rainfall	8,000	150	400
Total Flow	**287,500**		
Weighted Average	ppm	587	866

4.2.2 Twin Oaks Valley Water Treatment Plant

Blending at TOVWTP was also evaluated. In this scenario, RCS water at 279,500 AFY and a TDS between 600 – 879 mg/L would be blended with water from MWD at varying flow rates between 0 and 56,000 AFY and a TDS between 400 – 600 mg/L. Due to the fluctuations in flow rate of MWD water, a precise total flow rate and weighted average cannot be calculated. As shown in Table 4-3, the anticipated range of TDS levels in the blended water would still exceed the limit of 500 mg/L, even at the optimal conditions. For this reason, blending at TOVWTP would not be considered a practical option.

TABLE 4-3
Blending Strategies for Colorado River Water at TOVWTP

Flow Source	Flow Rate (AFY)	Min. TDS (mg/L)	Max. TDS (mg/L)
Regional Conveyance System	279,500	600	879
MWD (Case 6)	0 - 56,000	400	600
Total Flow	**279,500 – 335,500**		
Weighted Average	ppm	566 - 600	832 - 879

4.2.3 Blending Summary and Recommendations

Based on the flow and mass balance results for SVR and TOVWTP blending strategies, it could be concluded that blending would not be able to reduce the TDS to acceptable levels. For this reason, blending is not a viable treatment option.

4.3 Location of Treatment Facilities

This section identifies potential treatment facility locations in Imperial and San Diego Counties. Previous studies for the RCS project identified a series of pipeline alternatives that could transport the RCS water to San Diego County. Alternatives 3A, 5A, and 5C, each have at least one feasible location for a treatment facility which would be discussed in this section. Figure 4-3 presents an overview map of the alternatives, including the treatment facility locations that were considered.

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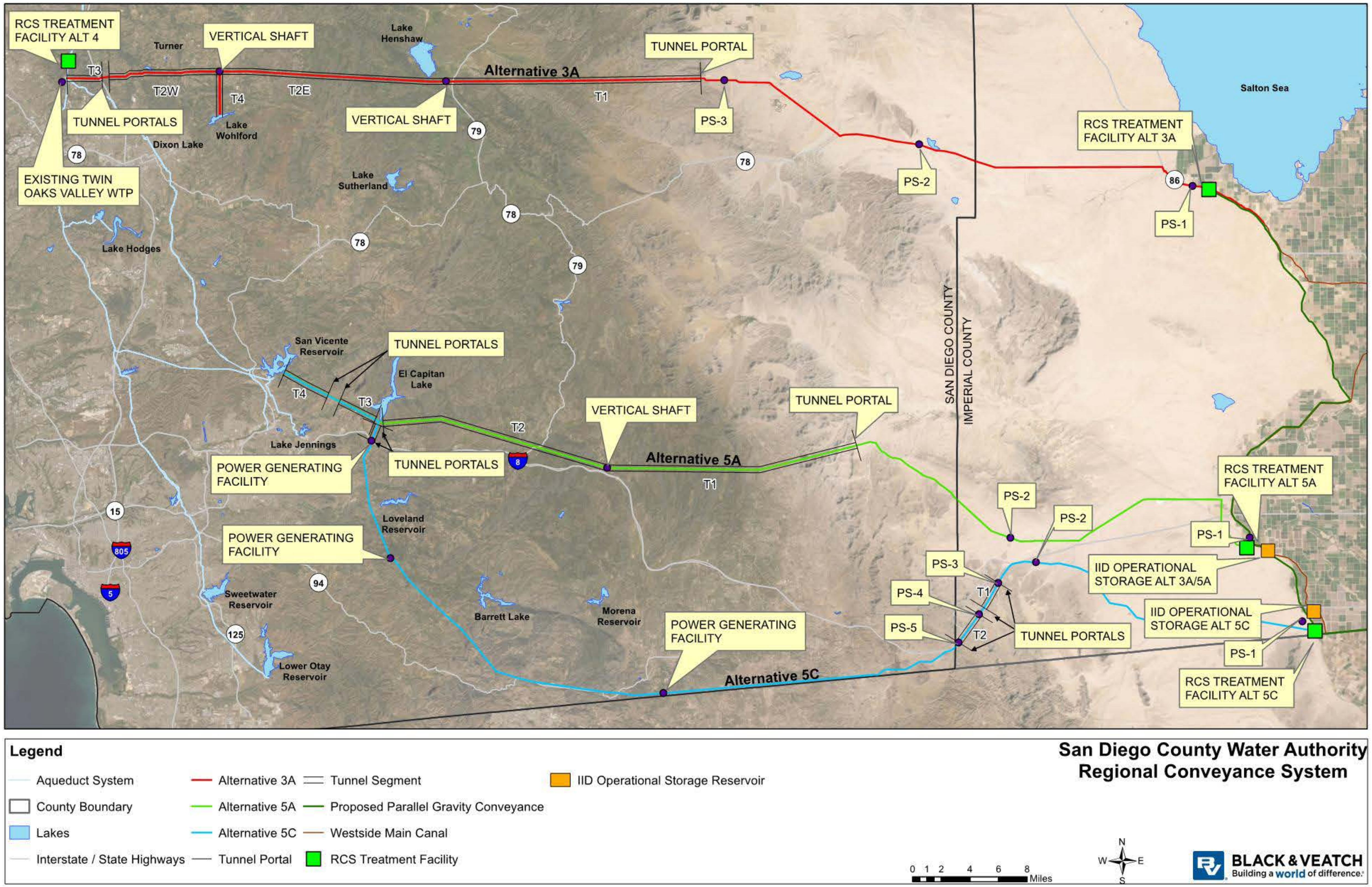

FIGURE 4-3
Overall Schematic of RCS Alternatives

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4.3.1 Imperial County Alternative 3A

The Imperial County treatment facility associated with Alternative 3A would be located near the Elmore Desert Ranch just west of State Highway 78, as shown on Figure 4-4. The 900 AF storage reservoir that would be required to address the lack of capacity in the AAC, as described in Chapter 2, would also serve as the forebay to the treatment plant. Raw water would be delivered to the treatment forebay by a new gravity conveyance system from the storage reservoir, which is anticipated to be located near the Fox Glove check facility along the WSM. This combined reservoir and treatment forebay would consist of approximately 900 AF of raw water storage and occupy approximately 200 acres.

This treatment location would require approximately 51 acres of land to accommodate the treatment facilities (both water and solids handling), and blended water forebay and associated Pump Station No. 1. Untreated raw water would bypass the treatment forebay, via a new gravity conveyance system, and blend with the treated water prior to being pumped. The blended flow would be pumped north along the Alternatives 3A corridor as shown on Figure 4-3. Access to the treatment facility would be off State Highway 78. Further process details of the treatment facility are presented in Section 4.5 of this Chapter. Brine management options for this treatment location are described in Section 4.6 of this Chapter.

4.3.2 Imperial County Alternative 5A

The treatment facility associated with Alternative 5A would be south of Interstate 8 and west of the Fox Glove Check facility along the WSM, which is owned and operated by IID, as shown on Figure 4-5. Similar to Alternative 3A, the 900 AF storage reservoir would also serve as the treatment facility's forebay. Raw water would be delivered to the treatment facility from the storage reservoir. For this treatment location, this combined reservoir would consist of approximately 900 AF of raw water storage and occupy approximately 200 acres.

This treatment location would require approximately 43 acres of land to accommodate the treatment facilities (both water and solids handling), and blended water forebay and associated Pump Station No.1. Untreated raw water would bypass the treatment forebay, via a new gravity conveyance system, and blend with the treated water prior to being pumped. The blended flow would be pumped northeast, where it would turn west and parallel State Highway 80 and follow the Alternative 5A corridor as shown on Figure 4-3. Access to the treatment facility would be off Interstate 8 at the Dunaway Road exit. The existing frontage road paralleling Interstate 8 heading east would be utilized, and a new roadway heading south to access this new treatment location would be required. Further process details of the treatment facility are presented in Section 4.5 of this Chapter. Brine management options for this treatment location are described in Section 4.6 of this Chapter.

4.3.3 Imperial County Alternative 5C

The treatment facility associated with Alternative 5C would be located northwest of the AAC terminus, and west of Signal Road, as shown on Figure 4-6. This alternative would also utilize the 900 AF storage reservoir as the treatment facility's forebay. Raw water would be delivered to the treatment facility by a new gravity conveyance system that extends from the storage reservoir. A new gravity conveyance system would be required to convey flows back to the WSM from the reservoir. For this treatment location, this combined reservoir would consist of approximately 900 AF of raw water storage and occupy approximately 200 acres.

This treatment location would require approximately 43 acres to accommodate the treatment facilities (both water and solids handling), and blended water forebay and associated Pump Station No. 1. Untreated raw water would bypass the treatment forebay, via a new gravity conveyance system, and blend with the treated water prior to being pumped. The blended flow would be pumped west along the Alternative 5C corridor as shown on Figure 4-3. Access to the treatment facility would be off Yuha Cutoff (State Route 98) and Signal Roads. A new roadway from Signal Road would be required. Further details of the treatment facility are presented in Section 4.5 of this Chapter. Brine management options for this treatment location are described in Section 4.6 of this Chapter.

4.3.4 San Diego County Alternative 3A

The San Diego County treatment facility associated with Alternative 3A would be located north of the existing TOVWTP on land currently owned by the City of Oceanside, as shown on Figure 4-7. For this option, pending coordination with the City of Oceanside, raw water would be delivered to the treatment forebay by a pressurized conveyance system that extends from Alternative 3A east of the Hidden Meadows community in Escondido, CA as shown on Figure 4-8. The pipeline would head in a northwest direction, passing north of Turner Lake, and crossing Interstate 15 north of Lawrence Welk Resort. This new pipeline would split from the main conveyance system and be sized to deliver approximately 134 mgd of raw water for treatment. This pipeline size would be 72-inches, which would result in a velocity of approximately 7 fps, similar to the design of the RCS, and be approximately 6.75 miles long. The remainder of the RCS flow, approximately 135 mgd, would continue west to the Twin Oaks Water Treatment Plant and would also be sized to 72-inches to limit the velocity to approximately 7 fps.

This treatment location would comprise of approximately 51 acres and consist of a raw water forebay and associated pump station, treatment facilities (both water and solids handling), and blended water forebay and associated pump station. The post treatment water, approximately 125 mgd, would be pumped via a new 1.4 mile long, 70-inch pipeline, that parallels the existing 2nd Aqueduct, to deliver flows to TOVWTP, where it would be blended with the bypass raw water, prior to delivery into the Water Authority aqueduct system. Blending would occur in a new day tank, as described in *Chapter 3*. This day tank would be utilized to blend three sources of water: 1) the post treated QSA water, 2) the bypassed QSA raw water, and 3) MWD supplies, when required. Access to the treatment facility would be off Silverleaf Lane and El Pass Road, just south of the existing City of Oceanside Water Treatment Plant. Details of the new treatment facility are presented in Section 4.5 of this Chapter.

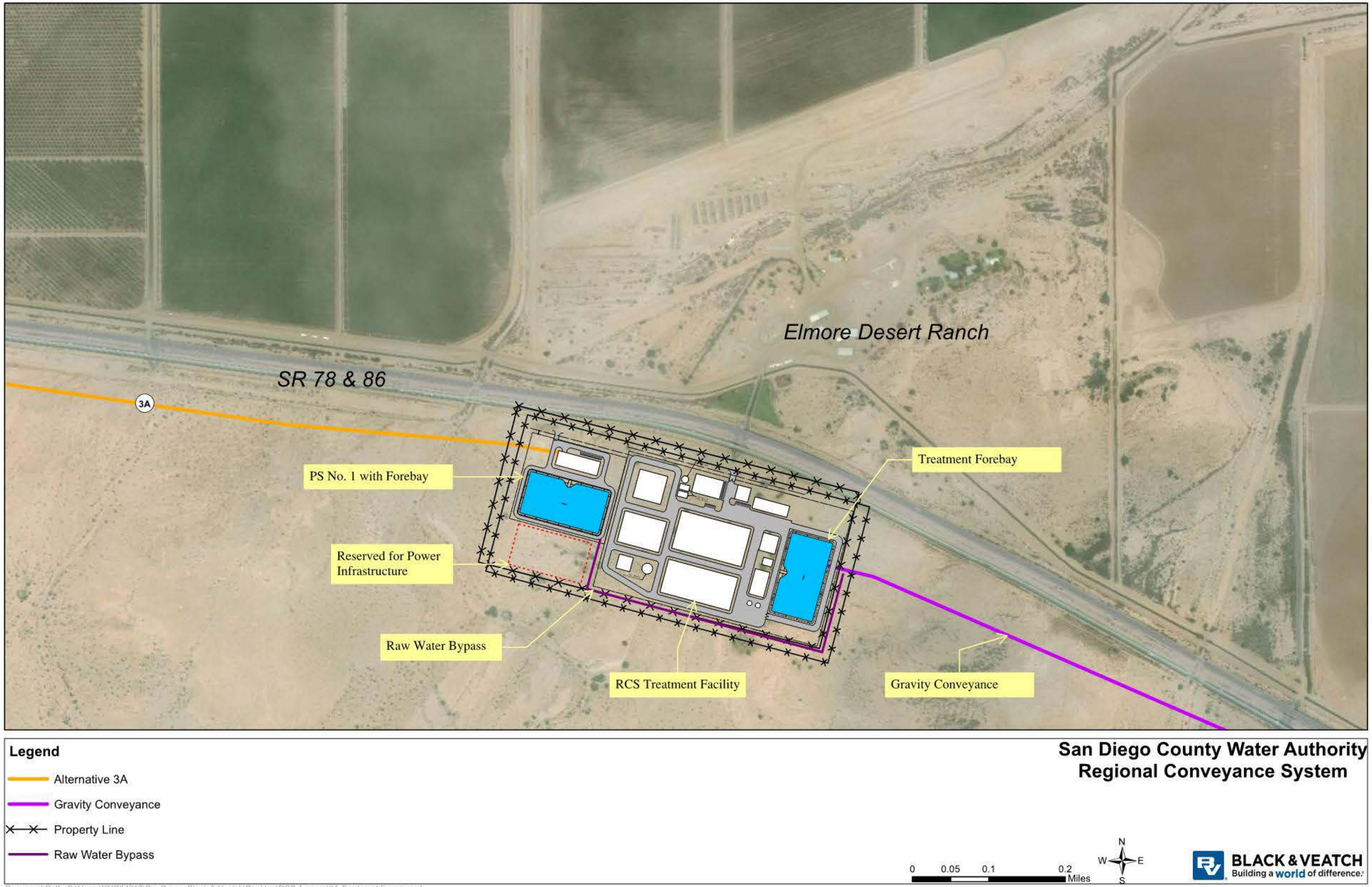

FIGURE 4-4
Alternative 3A Imperial Valley Treatment Location

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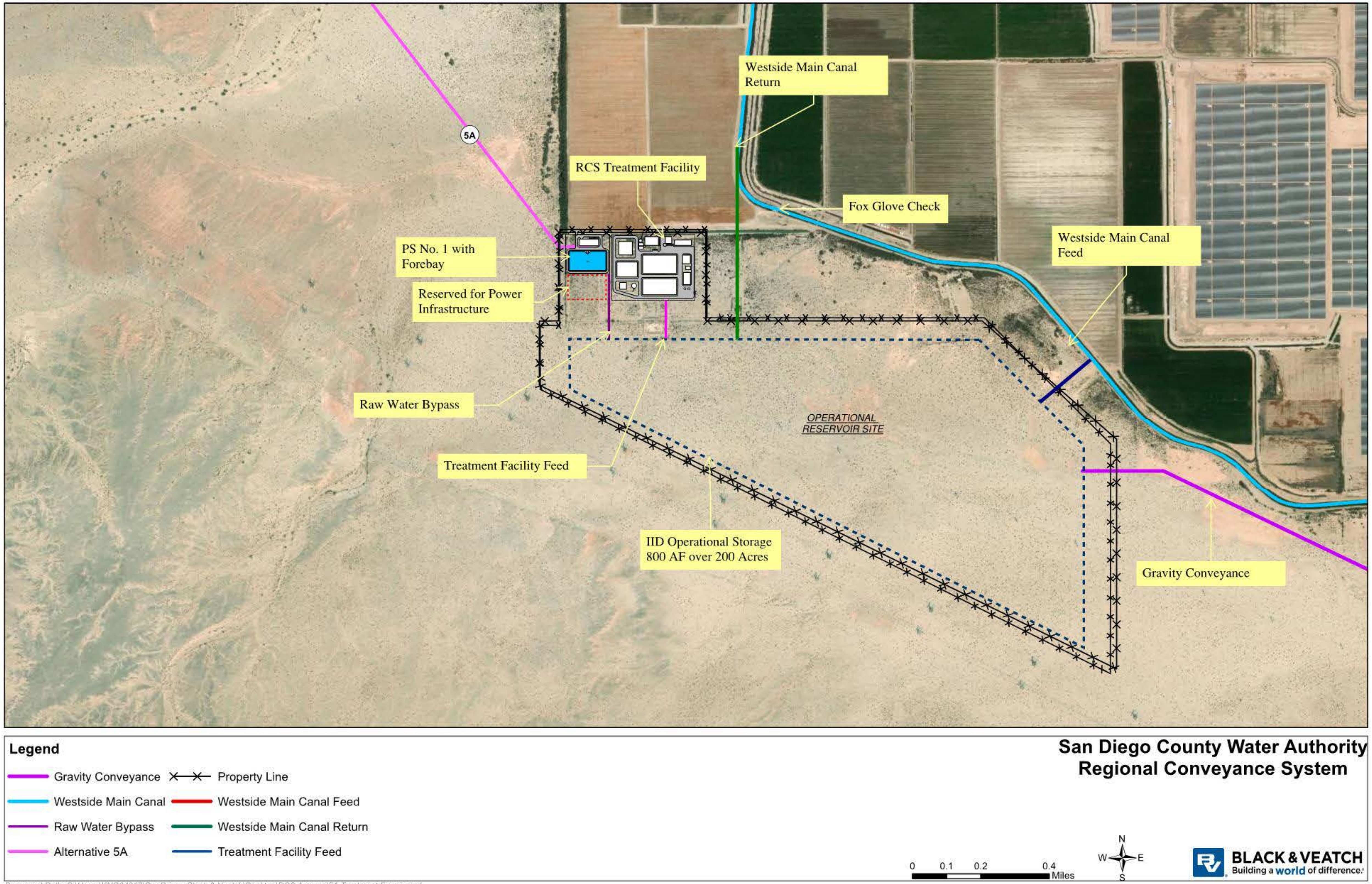

FIGURE 4-5
Alternative 5A Treatment Location

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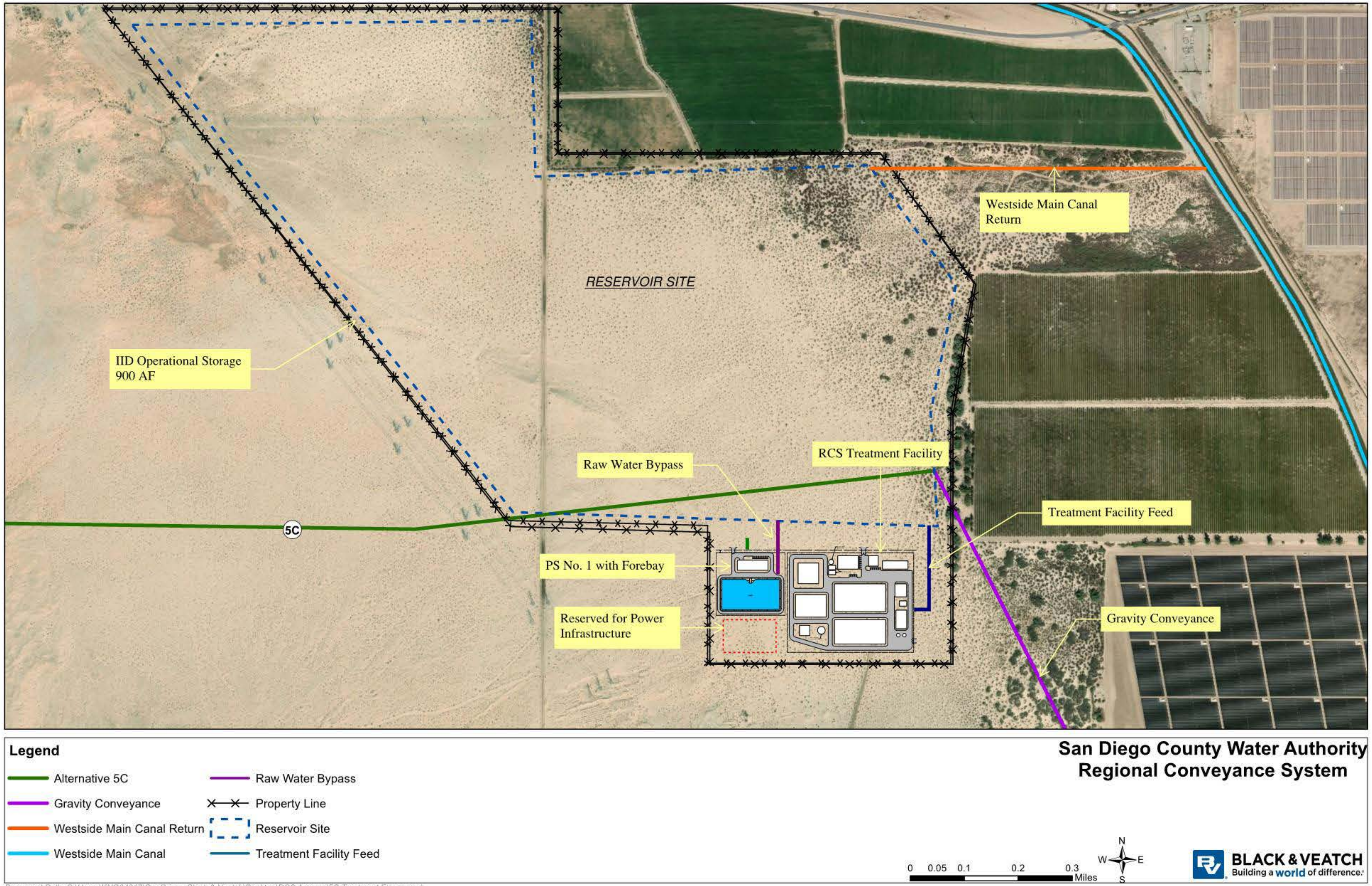

FIGURE 4-6
Alternative 5C Treatment Location

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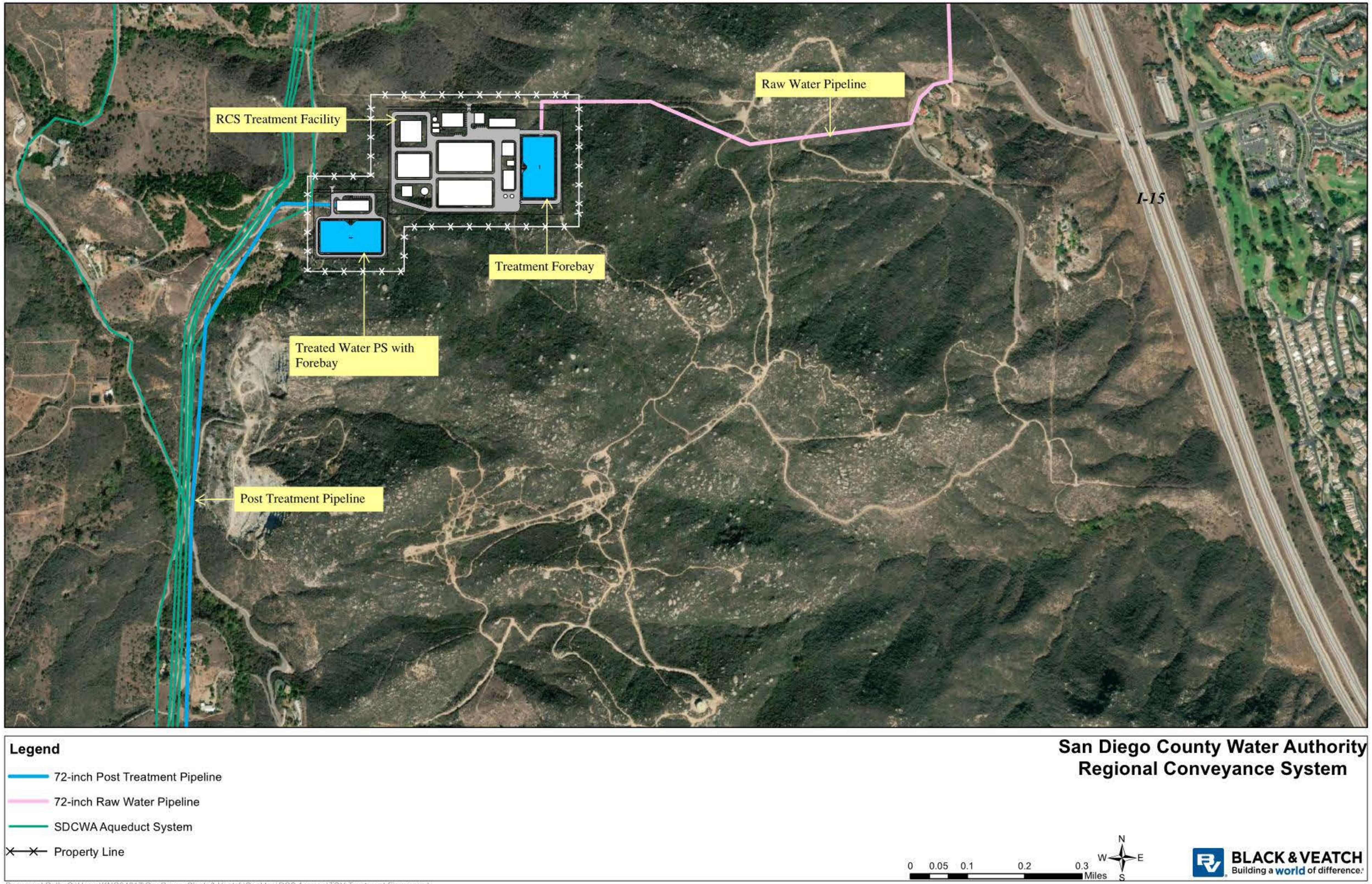

FIGURE 4-7
Alternative 3A San Diego County Treatment Location

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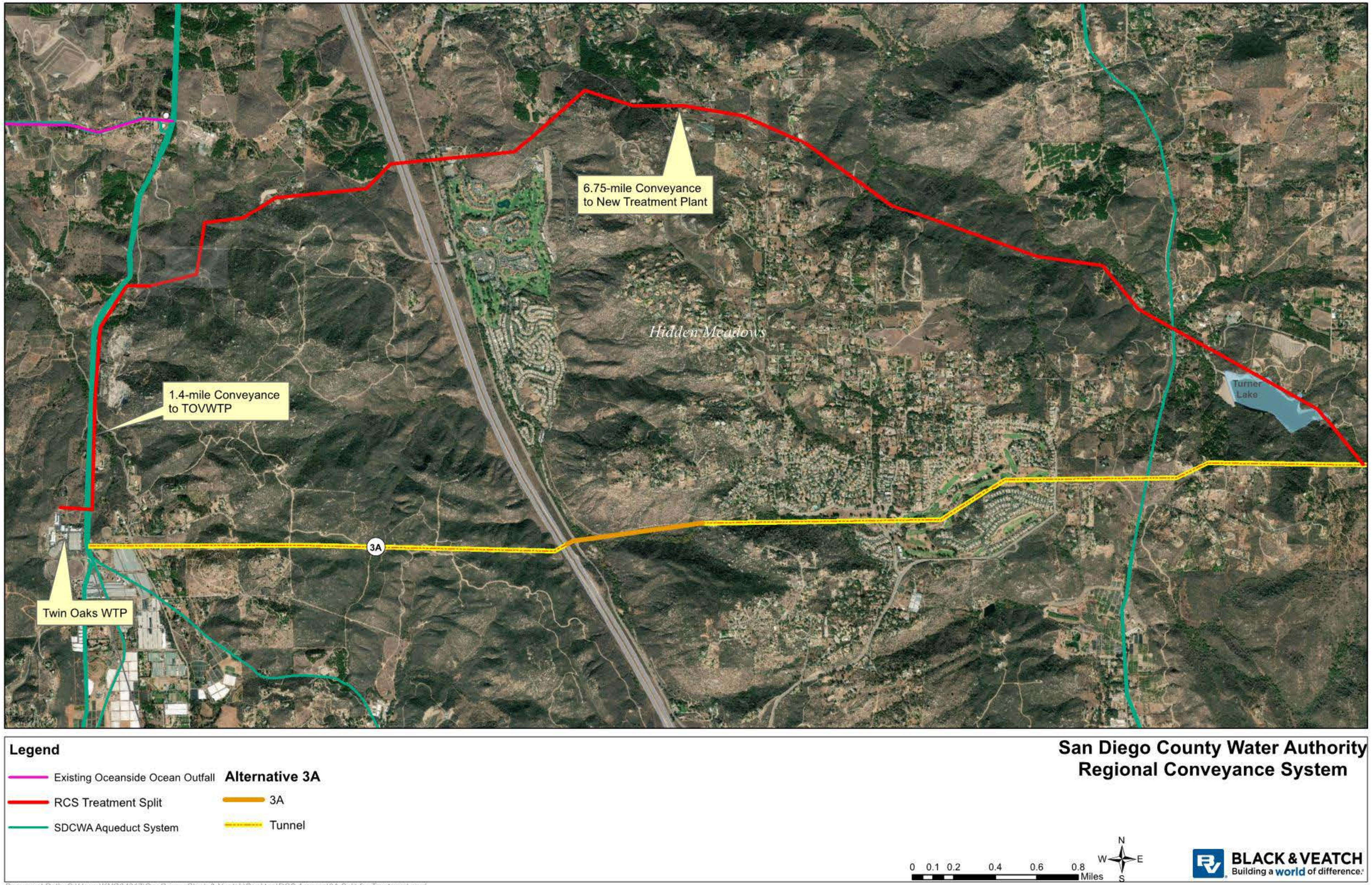

FIGURE 4-8
Alternative 3A San Diego County Treatment Option – Raw Water Pipelines

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4.4 Treatment Facility Conceptual Design

4.4.1 Treatment Facility Components Process Schematic/Mass Balance

Treatment Components

This section provides conceptual design criteria for the proposed treatment facilities consisting of the major components listed below.

- Invasive Mussel Treatment
- Traveling Water Screens
- Influent Storage Forebay (or Operational Storage Reservoir as described in Chapter 2) and Pump Station
- Pretreatment chemical addition (sodium hypochlorite and aqueous ammonia to produce chloramines for biofouling control)
- Automatic Strainers
- Membrane Filtration (MF) System
- MF clean in place (CIP) / Backwash System
- MF Break Tank / Transfer Pump Station
- Reverse Osmosis (RO) Cartridge Filters
- RO Pretreatment Chemicals (antiscalant and sulfuric acid for scale control, sodium bisulfite to quench free chlorine
- RO System
- RO CIP / Flush System
- Brine Storage Tank (Brine management Option 2 only)
- Brine Volume Minimization System (Brine management Option 2 only)
- Backwash Recovery System
    - Backwash Recovery Basin and Pump Station
    - Package Plate Settler Treatment Unit
- Solids Handling Equipment
    - Gravity Thickener
    - Belt Filter Press
    - Dewatered Sludge Holding Bed
- Waste Storage Tanks
- Finished Water Chemical Treatment (sodium hypochlorite and aqueous ammonia to produce chloramines for biofouling control; sodium hydroxide for corrosion control)

General Process Schematic

A general process flow schematic associated with the treatment facilities is provided in Figure 4-9. The QSA water would be treated for quagga mussel growth, passed through traveling water screens, and would flow by gravity to an influent storage forebay, or joint operational storage reservoir (as described in Chapter 2). This water would be pumped for treatment through MF, consisting of microfiltration or ultrafiltration, followed by RO. Prior to MF, the water would pass through automatic strainers to remove particles / debris that could damage and cause fouling of the downstream MF membranes. Filtrate produced from the MF system would be stored in a break tank, which would provide equalization between the MF and downstream RO process to allow continuous production capacity from the RO system when MF units undergo routine maintenance activities. Both the MF and RO system would include CIP systems consisting of batch tanks to prepare membrane cleaning solutions, chemical dosing pumps and recirculation piping. The RO system would also include a flush tank and pump system, which would store RO permeate for flushing the membranes following planned and unplanned shutdowns.

Backwash water produced from the MF system and the influent automatic strainers would be stored in a recovered water basin and pumped to a washwater treatment process consisting of package plate settler units with chemical pretreatment to enhance coagulation, flocculation and settling. Decant from the washwater treatment process would be returned to the influent of the automatic strainers. Settled sludge would undergo solids handling consisting of sludge thickening, mechanical dewatering, and sludge drying beds. Solids would be collected in roll off dumper and disposed off-site.

As discussed in Section 4.6 of this report, brine from the RO system would either be released to the Salton Sea directly (the ideal option) or via a constructed wetland or it could be further treated to reduce brine volume prior to release to evaporation ponds.

Lastly, finished water, comprising of a blend of bypass water, RO permeate, and possibly treated water from the brine volume minimization process, would be mixed in the blended water forebay, prior to being pumped by the blended water pump station for conveyance through the Alternative system.

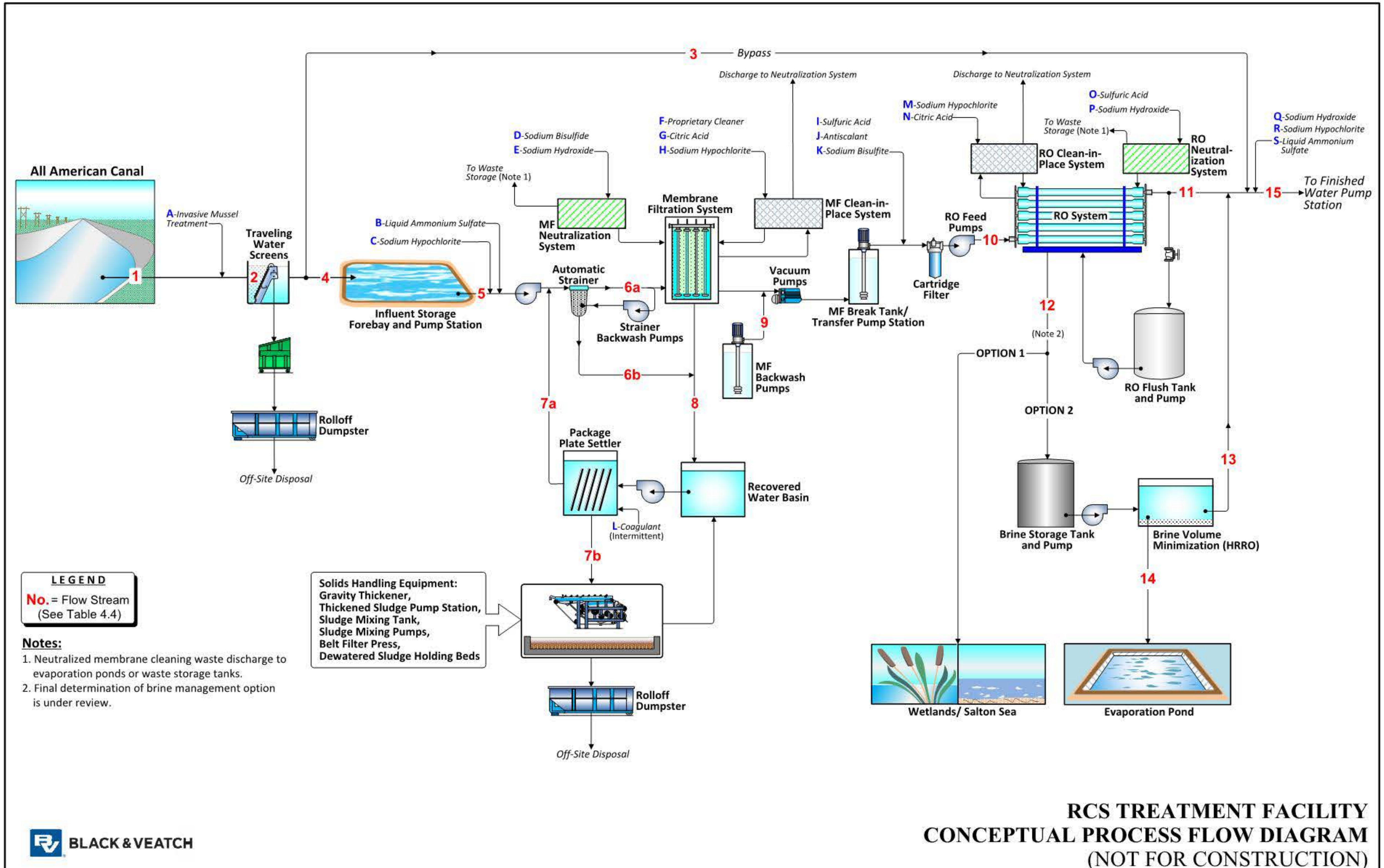

FIGURE 4-9
General Process Flow Schematic

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Mass Balance

A mass balance associated with the conceptual design of the treatment facilities is provided in Table 4-4. The table includes the estimated flows and TDS concentration for the various flow streams identified in Figure 4-9. The conceptual sizing of the various process components is based on a fixed influent flow of 269 mgd (301,500 AFY) and target treated water TDS of 500 mg/L under the maximum anticipated influent TDS concentration of 879 mg/L. The flow rates shown are based on the assumed recovery values associated with major processes as presented in Section 4.4.2. For purposes of this report, the term "recovery" for a given process is the ratio of usable treated water produced to total feed water within a 24-hour period.

TABLE 4-4
Conceptual Mass Balance RCS Treatment Facility

		TDS (mg/L)			Flow Rate	
Stream	**Description**	**Min(1)**	**Avg(2)**	**Max(3)**	**mgd**	**AFY**
1	Raw Water	570	662	879	269	301,500
2	Screened Water	570	662	879	269	301,500
3	MF Bypass	570	662	879	135	151,400
4	Forebay Influent	570	662	879	141	156,000
5	Forebay Effluent	570	662	879	141	156,000
6a	MF Feed	570	662	879	139	156,000
6b	Strainer Discharge	570	662	879	1.4	1,580
7a	Washwater Decant	570	662	879	6.8	7,700
7b	Package Plate Settler Discharge	570	662	879	0.1	160
8	Washwater Sludge	570	662	879	7	7,800
9	MF Backwash	570	662	879	5.6	6,250
10	RO Feed	570	662	879	134	150,000
11	RO Permeate	29	33	44	114	127,300
12	RO Concentrate/High Recovery Reverse Osmosis (HRRO) Feed /Salton Sea or Wetlands Feed	3,639	4,226	5,611	20	22,500
13	HRRO Permeate (optional)	364	423	561	10	11,200
14	HRRO Concentrate (optional)	6,913	8,029	10,661	10	11,200
15a	Finished Water Stream without HRRO	324	377	500	249	278,700
15b	Finished Water Stream with HRRO (optional)	324	377	500	259	289,900

Notes:

1. Minimum TDS reported for Colorado River Water sources 2009 to 2017 MWD Annual Water Quality Reports to Member Agencies.
2. Maximum TDS reported for Colorado River Water sources 2009 to 2017 MWD Annual Water Quality Reports to Member Agencies.
3. IID design feedwater value established for water transferred from the AAC in Imperial County, as presented in *1996 Water Transfer Study* and the *2017 Basin Plan*.

As shown, under minimum and average influent TDS conditions, the treatment facility could produce finished water TDS concentrations below 500 mg/L. Alternatively, the treatment facility could be operated at reduced capacity by taking membrane units offline and increasing the bypass flow to maintain target finished water TDS of 500 mg/L. The minimum and average TDS values used for the basis of the mass balance are based on Colorado River Water quality presented in MWD's 2009-2019 Annual Water Quality Reports to Member Agencies. If the Basin Plan is updated in 2020, Phase B, if authorized, should account for any updates. The maximum TDS used for the mass balance is based on the IID design feedwater value established for water transferred from the AAC in Imperial County as presented in *1996 Water Transfer Study*. This maximum TDS used for the mass balance also represents the maximum allowable discharge TDS to the Colorado River per the Basin Plan.

The mass balance provided in Table 4-5 shows the treatment facility is based on target finished water TDS of 500 mg/L, fixed influent flow 269 mgd (301,500 AFY) and maximum anticipated influent TDS concentration of 879 mg/L. As presented in Chapter 2, it is assumed the RCS could be offline approximately 20 days per year due to maintenance or other flow interruptions. As such, the conveyance system downstream of the treatment facility would be required to deliver a maximum flow of 296 mgd (331,563 AFY). Table 4-5 provides the influent flow and treated water flow required to achieve the maximum target flow associated with the downstream conveyance system for a range of influent TDS concentrations. Assuming a fixed RO capacity of 115 mgd (128,890 AFY), the influent TDS is required to be below 807 mg/L to meet the target finished water flow and TDS. If the TDS was above 807 mg/L, the system could be operated at design capacity with higher finished water TDS or reduced capacity to maintain the target TDS of 500 mg/L. It should also be noted that a short period of exceeding a finished water TDS of 500 is acceptable. The corrosivity of the finished water would need to be evaluated under future phases of the project, if authorized, to confirm if any post treatment would be required.

TABLE 4-5
Flow Requirements for Various Influent TDS Concentrations to Achieve Maximum Finished Water Flow for Downstream Conveyance System

Influent TDS Condition (mg/L)	Influent Flow Required (mgd)	Treated Water Required (mgd)	Bypass (mgd)	Finished Water TDS (mg/L)	Finished Water (mgd)
570	299	41	255	500	296
662	302	79	216	500	296
807 (1)	306	125	171	500	296

Note:

1. Influent water with a TDS level exceeding 807 mg/L would have a finished water TDS above 500 mg/L. In the most extreme case, where the influent TDS levels reach 879 mg/L, the finished water TDS would be approximately 500-600 mg/L.

4.4.2 Process Descriptions and Conceptual Design Criteria

This section provides conceptual design criteria associated with each major process component of the proposed treatment facility.

Invasive Mussel Treatment

Quagga and zebra mussels are an invasive species found in freshwater bodies throughout the United States. In addition to damaging ecosystems, these mussels clog pipes, destroy pumps, valves, and other treatment equipment, and are difficult and expensive to remove. Chemical treatment options to proactively combat these invasive species, which are present in the lower Colorado River basin, are discussed below.

- Chlorine can be used to maintain oxidant levels in water, which prevents mussels from attaching to structures, such as pipe walls. Maintaining Cl_2 at 0.5 mg/L for 14 days could kill about 95% of adult quagga and zebra mussels. Concerns with chlorine include the formation of disinfection byproducts (DBPs) such as trihalomethane (THM), which would need to be closely monitored and coordinated with downstream processes.
- Chlorine dioxide could kill virtually 100% of all veligers and reduce the reactivity of DBPs with free chlorine. ClO_2 must be made on site by electrolyte generation and requires more money for creation, equipment and storage than other options. Byproducts include chlorate $(ClO_3)^-$ and chlorite (ClO_2^-) and may have to be removed. Chlorite has current MCL of 1.0 mg/L.
- Potassium permanganate, while not as effective at mussel removal as chlorine, is simple to feed, has a low toxicity for wildlife, and no DBPs. Dosage is sensitive and could turn water pink if too much is added.

These treatment options would be compared in Phase B, if authorized, and ultimately one would be selected for recommendation at the future RCS treatment facility.

Traveling Water Screens

Traveling water screens (TWS) would be implemented to screen out debris such as twigs, leaves, fish, trash, etc. from entering the treatment facility and the bypass. Conceptual design criteria for a large TWS is provided in Table 4-6 below and would be described in more detail during Phase B of this evaluation, if authorized.

TABLE 4-6
Traveling Water Screen Conceptual Design Criteria

Design Parameter	Units	Value
Total Number of Units Required (Duty + Standby)	#	5+1
Capacity of Each	MGD	60
Screen Width	Feet	12
Screen Height	Feet	57
Screen Size	Inches	3/8

Influent Forebay and Pump Station

Conceptual design criteria for the influent forebay and pump station is summarized in Table 4-7. The purpose of the forebay would be to provide flow equalization of raw water prior to downstream treatment. A pump station comprised of vertical turbine pumps would deliver water from the forebay to the screening facility. Note the conceptual design of the pump station excludes the bypass flow therefore the bypass flow would be conveyed from the influent forebay to finished water pump station by gravity. This would require the influent forebay to be constructed at a higher elevation than that of the finished water pump station. As part of the final design, a detailed evaluation of this approach would need to be performed prior to finalizing the pump station design.

TABLE 4-7
Influent Storage Forebay and Pump Station Conceptual Design Criteria

Design Parameter	Units	Value
Influent Storage Forebay		
Retention Time	Minutes	60
Usable Volume Required	MG	5.8
Pump Station(1)		
Pump Units (Duty + Standby)	Quantity	10+1
Pump Type	-	Vertical Turbine
Capacity per Pump	GPM	10,000
Rated Head	Feet	100
Motor	HP	400

Note:
1. The use of Variable Frequency Drives (VFD) and generators would be evaluated during Phase B, if authorized.

Automatic Strainers

Conceptual design criteria for the automatic strainers is summarized in Table 4-8. The purpose of the automatic strainers would be to remove small particles and debris that could damage the downstream membranes. Based on the Water Authority's experience at TOVWTP and other facilities, the conceptual strainer design includes a pressurized backwash system to remove zooplanktons and other materials that could clog the strainers.

TABLE 4-8
Automatic Strainers Conceptual Design Criteria

Design Parameter	Units	Value
Number of Units (duty + standby)	#	8+1
Strainer Rating	Microns	500 (to be confirmed provided by membrane supplier)
Capacity of Each	MGD	33.3
Recovery	%	>99
Differential Pressure	PSI	2 (clean), 5 (dirty)
Backwash System	NA	Pressurized

Membrane System

Conceptual design criteria for the MF system is summarized in Table 4-9. The MF system would remove fine particulate matter thereby providing pretreatment to the downstream RO system, which requires feed water turbidity <0.15 NTU and silt density index (SDI) < 3.

The conceptual design is based on submerged or pressurized membranes. The former configuration, as employed at TOVWTP, contains membranes that are submerged in tanks and water is filtered from the outside-in using vacuum pumps. For the latter configuration, the membranes are housed in vessels and require a pressurized feed source. Submerged membranes could provide a possible lower cost for the given application. It should be noted the submerged system design could also eliminate the need for the pump station associated with the influent forebay by locating the membrane tanks at an appropriate elevation to allow water from the forebay to flow into the tanks by gravity. However, the required depth of the membrane tanks could make this cost prohibitive and would require further evaluation.

Though the submerged MF system has advantages for the given application, it is recommended a detailed evaluation of the two configurations be conducted during future design phase to compare economic and non-economic factors as part of the final design process. An important factor to consider when comparing the two alternatives is the number of membrane suppliers; which in the case of submerged membrane systems, is limited to two major suppliers.

Key design parameters that impact the cost of membrane systems is the maximum instantaneous flux and recovery. Because the flux has a direct impact on the footprint of the membrane system for a given filtrate capacity, for the purposes of the conceptual design, a conservative flux of 20 gallons per sq ft of membrane area per day (gfd) was assumed. Because several factors could impact membrane performance (e.g. raw water quality, selected membrane product, backwash recycle, cleaning regime, etc.) it is recommended that pilot testing be conducted should the project move forward as part of a competitive membrane procurement process to finalize the conceptual membrane design criteria. Ancillary equipment required for the membrane system (not shown in Table 4-9) would include: MF clean in place (CIP) / Neutralization system, air system (vacuum pumps, compressors, receiving tanks), and backwash system (BW).

TABLE 4-9
Membrane System Conceptual Design Criteria

Design Parameter	Units	Value
Membrane Type	-	Submerged or Pressurized, Hollow Fiber
Membrane Material	-	Polyvinylidene (PVDF) or Polyethersulfone (PES)
Net Filtrate Capacity	MGD	134
Maximum Instantaneous Flux	gfd	20
Recovery	%	96%
Backwash Frequency	Minutes	15-30
Backwash Duration	Seconds	90-120

MF Break Tank & Transfer Pump Station

Conceptual design criteria for the MF break tank and transfer pump station is summarized in Table 4-10. The MF break tank would provide storage of filtrate produced by the MF system, prior to being pumped to the RO system or used to backwash the membranes. The break tank would serve as a hydraulic and operational buffer between the MF and RO processes to limit the impact of short-term variations in filtrate flow that occur during MF backwash cycles.

The transfer pump station would convey MF filtrate from the MF break tank through the RO feed cartridge filters to the suction side of the RO high pressure pumps. The transfer pumps would be controlled by VFDs to maintain a constant pressure setpoint at the inlet to the cartridge filters.

TABLE 4-10
MF Break Tank and Transfer Pump Station Design Criteria

Design Parameter	Units	Value
Break Tank		
Quantity	-	1
Tank Type	-	Conventional Cast in Place or Prestressed Concrete
Retention Time	Minutes	30
Usable Tank Volume	MG	2.8
Transfer Pumps		
Quantity (duty + standby)	-	7
Pump Type	-	Vertical Turbine
Capacity per Pump	GPM	16,000
Rated Head	Feet	140

TABLE 4-10
MF Break Tank and Transfer Pump Station Design Criteria

Design Parameter	Units	Value
Motor	HP	887
Drive	-	VFD

RO Cartridge Filters

Conceptual design criteria for the RO feed cartridge filters is summarized in Table 4-11. The RO feed cartridge filters would prevent incidental particulate material present in the MF filtrate pipelines or MF break tank from passing through the RO feed pumps and damaging the membranes. Though the conceptual design considered standard size cartridge filters, due to the large capacity of the treatment facility, the final design could consider larger diameter cartridge filters which could reduce costs.

TABLE 4-11
Cartridge Filter Conceptual Design Criteria

Design Parameter	Units	Value
Quantity	-	14
Design Flow per Vessel	MGD	10
Cartridge Filter Element Diameter	Inches	3
Cartridge Filter Element Nominal Pore Size	Micron	5
Design Loading Rate	GPM/10-inch Equivalent	<3.5
Vessel Orientation	-	Horizontal
Vessel Material		FRP or 316 SS
Pressure Rating	PSI	150

RO System

Conceptual design criteria for the RO system is summarized in Table 4-12. The conceptual design recovery was determined using proprietary RO modeling software and aforementioned IID design feed water quality data. A 3-stage RO skid configuration was selected compared to a 2-stage system due to higher cross flow velocities under the design recovery conditions which could improve scaling/fouling performance. A nominal salt rejection of 95% was used for purpose of sizing the RO permeate capacity required to achieve target finished water TDS of 500 mg/L. The actual TDS rejection of membranes would be higher (> 98%) but the conservative value of 95% was used to account for impact of membrane ageing and high feed water temperatures which could reduce rejection.

Ancillary equipment required for the RO system (not shown in Table 4-11) would include: RO CIP / Neutralization system and RO flush system.

TABLE 4-12
RO System Conceptual Design Criteria

Design Parameter	Units	Value
RO Trains		
Quantity of RO Units (or Skids)	#	24
Quantity of Duty Units	#	23
Quantity of Spare Units	#	1
RO Units		
Design permeate flow per unit	mgd	5
Design permeate flow, total	mgd	115
RO membrane element diameter	inches	8
RO membrane element length	inches	40
RO membrane area per element	sq ft	440
RO element per vessel	#	7
Number of stages per RO unit	#	3
Number of pressure vessels per RO unit	#	125
RO System Recovery	%	85%
RO System flux	gfd	13
Nominal RO membrane TDS Rejection	%	95%

Backwash Recovery Basin & Pump Station

Conceptual design criteria for the backwash recovery basin is summarized in Table 4-13. The backwash recovery basin would allow approximately 30-minutes of backwash flow produced from the MF system and automatic strainers to be stored until it could be recycled to the head of the treatment facility. Operation of the backwash recovery system, consisting of the backwash recovery basin and pump station and packaged plate settler treatment units, would be required while the treatment facility is producing water. Though the treated water from the RCS treatment facility would be blended with bypass water and further treated at downstream drinking water facilities, the backwash recycle rate should be less than 10 percent of the raw water flow, in accordance with the Filter Backwash Recycle Rule to minimize potential impacts on MF performance. The flow of recycled water would be adjusted using pumps equipped with VFDs that are controlled by a signal from a recycle flowmeter. For conceptual design purposes it was assumed 98% of the backwash water produced by the MF system would be recovered and returned to the head of the plant daily. The remaining 2% backwash volume represents water associated with the concentrate the solids produced from the backwash recovery system and would be processed through the solids handling equipment.

TABLE 4-13
Backwash Recovery Basin & Pump Station Conceptual Design Criteria[1]

Design Parameter	Units	Value
Backwash Recovery Basin		
Quantity	#	1
Tank Type	-	Concrete Cast in Place or Prestressed Concrete
Retention Time	Minutes	30
Usable Tank Volume	Gallons	220,000
Transfer Pumps		
Quantity (Duty + Standby)	#	4+1
Pump Type	-	Vertical Turbine
Capacity per Pump	GPM	2,203
Rated Head	Feet	120
Motor	HP	105
Drive	-	VFD

Note:

1. Based on similar capacity submerged membrane system; actual backwash flow rate and sequencing is membrane manufacturer specific and would be further evaluated during future design phase.

Packaged Plate Settler Treatment Units

Conceptual design criteria for the packaged plate settler treatment units (PSUs) is summarized in Table 4-14. The PSUs would treat water pumped from the Backwash Recovery Basin. The PSUs would consist of rapid mix tank, flocculation tank, and plate settlers. Based on the specific characteristics of the backwash waste solids, dissolved air flotation (DAF) could provide more efficient solids removal than PSUs and therefore should be evaluated as part of the final design.

TABLE 4-14
Packaged Plate Settler Treatment Units Conceptual Design Criteria

Design Parameter	Units	Value
General		
Quantity (duty + standby)	#	4+1
Flow Rate per Unit	gpm	2,200
Influent TSS	mg/L	60-125
Target Effluent TSS	mg/L	10
Underflow Concentration Average	%	1
Rapid Mix Tanks		
Minimum Volume per Tank	Gallons	422

TABLE 4-14
Packaged Plate Settler Treatment Units Conceptual Design Criteria

Design Parameter	Units	Value
Minimum Mixer Drive Power	HP	1
Minimum Velocity Gradient @ 50F and Design Flow	1/seconds	500
Detention Time	Seconds	12
Flocculation Tanks		
Minimum Volume per Tank	Gallons	3,009
Minimum Mixer Drive Power	HP	1
Minimum Velocity Gradient @ 50F and Design Flow	1/seconds	100
Detention Time	Seconds	60
Plate Settlers		
Hydraulic Loading Rate	gpm/sq ft	0.5
Maximum Inlet Velocity at Design Flow Rate	fps	0.5
Minimum Effective Settling Horizontal Settling Area(1)	sq ft	4,118
Late Angle of Inclination as Measured from Horizontal	Degrees	55
Horizontal Center to Center Plate Spacing	Inches	2.44

Note:

1. Calculated using 80% of the total plate area.

Solids Handling Equipment

Conceptual design criteria for the solids handling equipment is summarized in Table 4-15. The solids handling equipment would be used to process the solids produced from the MF backwash recovery system. Solids handling equipment would include gravity thickeners, thickened sludge pump station, sludge mixing tanks, sludge mixing pumps, belt filter presses, and dewatered sludge holding beds. Dry solids would be stored in roll-off dumpsters and hauled offsite for proper disposal.

TABLE 4-15
Solids Handling Equipment Conceptual Design Criteria

Design Parameter	Units	Value
Gravity Thickener		
Quantity	#	4
Flow Rate per Unit	gpm	642
Maximum Daily Solids Load	PPD	38,576
Influent Solids Concentration	%	0.07-0.5
Underflow Solids Concentration	%	2-8
Thickened Sludge Pump Station		
Quantity (Duty + Standby)	#	4+1
Type of Pump	-	Rotary Lobe
Rated Capacity, Each	GPM	323
Rated Differential Pressure	PSIG	30
Motor Size	HP	25
Sludge Mixing Tank		
Quantity	#	4
Tank Capacity, Each	Gallons	17,000
Sludge Mixing Pumps		
Quantity (Duty + Standby)	#	4 +1
Pump Capacity, Each	GPM	1,600
Motor Size	HP	30
Belt Filter Press		
Quantity	#	8
Type	-	2-meter, 2-belt
Belt Tensioning Hydraulic Pump Motor	HP	1
Belt Press Washwater Booster Pump		
Quantity (Duty + Standby)	#	4+1
Capacity, Each	GPM	80
Motor Size	HP	15

TABLE 4-15
Solids Handling Equipment Conceptual Design Criteria

Design Parameter	Units	Value
Dewatered Sludge Holding Beds		
Type of Beds	-	Concentrate Bottom with Decanting Sieves
Quantity	#	8
Bed Size, Each (LXW)	Feet	100 x 58
Sludge Depth	Feet	2.5
Days of Sludge Storage	Days	30

Waste Disposal Alternatives

The conceptual design includes waste storage tanks to store neutralized spent cleaning solutions. The MF and RO systems would require periodic cleaning to remove foulants/scale which could reduce production. The frequency of CIPs is dependent on several factors, including feed water quality, pretreatment method employed, and membrane system operational parameters (e.g. flux, recovery). CIP frequency typically varies from once every 1 month to once every 3 months. Neutralized cleaning chemical waste streams are typically released to sewer or brine outfall. However, due to the remote location of the RCS treatment facility sites, the conceptual design includes storing the CIP waste solutions in large storage tanks which would be pumped out and hauled offsite for disposal. The conceptual design includes two tanks, each with a useable capacity of 100,000 gallons.

This membrane cleaning waste storage tank sizing is based on a total estimated cleaning waste volume of approximately 500,000 gallons per membrane system. Cleaning of the MF and RO systems would be conducted over a 5 to 10-day period and would require the tanks to be emptied every 1 to 4 days depending on whether the systems were cleaned at the same time or at different times.

The feasibility of hauling the spent CIP solution for offsite disposal should be further evaluated as part of Phase B, if authorized, and could differ specific to each treatment facility location. As an alternative, the spent CIP solution could be released to evaporation ponds. If brine management Option 1 (Evaporation Pond, see discussion below) is implemented the evaporation ponds could be upsized to accommodate the CIP solution flows. If brine management Option 2 is implemented (Wetlands/Salton Sea, see discussion below) a smaller, dedicated evaporation pond could be constructed to accommodate the CIP flows. For either Option, the evaporation ponds could also be utilized for emergency process tank overflows or diversion of treated water during plant startup and testing.

Finished Water Chemical Treatment

Conceptual design assumed chemical treatment of the finished water, including the addition of sodium hypochlorite and ammonia to form chloramines, to control biofouling in the downstream systems. Based on an initial evaluation, it is anticipated the finished water (consisting of raw water bypass, RO permeate, and HRRO permeate) would be slightly corrosive (LSI of +0.1 and CCPP of 1.7). As such, the conceptual design considered the use of a nominal dose of sodium hydroxide to meet typical water distribution quality corrosion characteristics (i.e. LSI =+0.5; CCPP of 4-10; pH > 8; AI > 12).

Brine Storage Tank & Pump Station

Conceptual design assumed a brine storage tank with a useable capacity of 420,000 gallons to provide 30 minutes of retention time of the concentrate produce from the RO system. Brine from the storage tank would be pumped to the downstream brine volume minimization system.

Brine Volume Minimization System

Conceptual design criteria for the brine volume minimization system is summarized in Table 4-16 and only applies to applications where evaporation ponds are utilized. The conceptual design overall recovery of 90 percent was determined using proprietary RO modeling software and estimated concentrate quality produced from the RO system. A High Recovery Reverse Osmosis (HRRO) system, specifically the Closed Circuit Reverse Osmosis (CCRO) ReFlex Max unit design (as manufactured by Desalitech), would be required compared to their standard ReFlex unit design in order to treat the primary RO concentrate. A nominal HRRO salt rejection of 90% was used for the purpose of sizing the HRRO permeate capacity required to achieve target finished water TDS of 500 mg/L. The actual TDS rejection of the HRRO system would be higher (> 95%) but the conservative value of 90% was used to account for impact of membrane aging and high feed water temperatures which could reduce rejection. Ancillary equipment required for the HRRO system (not shown in Table 4-15) include: HRRO clean in place (CIP) / Neutralization system and HRRO flush system.

TABLE 4-16
Brine Volume Minimization System Conceptual Design Criteria

Design Parameter	Units	Value
HRRO Trains		
Quantity of HRRO Units (Skids)	#	12
Quantity of Duty Skids	#	11
Quantity of Spare Units	#	1
HRRO Units		
Design Permeate Flow per Unit	MGD	1.2
Design Permeate Flow, Total	MGD	13.4

TABLE 4-16
Brine Volume Minimization System Conceptual Design Criteria

Design Parameter	Units	Value
HRRO Membrane Element Diameter	Inches	8
HRRO Membrane Element Length	Inches	40
HRRO Membrane Area per Element	sq ft	400
HRRO Element per Vessel	#	5
Number of Stages per HRRO Unit	#	1
HRRO System Recovery	%	50%
HRRO System Flux	GFD	7
Nominal HRRO Membrane TDS Rejection	%	90%

Chemical Systems

Conceptual design criteria for the chemical systems is summarized in Table 4-17. The table provides chemical name, dosing location, application and chemical strength. Note alternative chemicals and/or storage strength should be evaluated as part of the final design based on cost, permitting requirements, and other safety and operational considerations.

TABLE 4-17
Chemical Systems Conceptual Design Criteria

Chemical	Dosing Location	Dosing Location Description	Application	Storage Strength
TBD	TWS Feed	A	Invasive Mussel Treatment	TBD
Liquid Ammonium Sulfate	MF Feed	B	Chloramine Formation / Biofouling Control	40%
Sodium Hypochlorite	MF Feed	C	Chloramine Formation / Biofouling Control	12.5%
Sodium Bisulfate	MF Neutralization	D	Neutralizing Agent	TBD
Sodium Hydroxide	MF Neutralization	E	Neutralizing Agent	TBD
Proprietary Cleaner	MF CIP	F	MF/UF Membrane Cleaning	100%
Citric Acid	MF CIP	G	MF Membrane Cleaning	50%
Sodium Hypochlorite	MF CIP	H	MF/UF Membrane Cleaning	12.5%
Sulfuric Acid	RO Feed	I	Scale Control	93%
Antiscalant	RO Feed	J	Scale Control	100%
Sodium Bisulfate	RO Feed	K	Quench Free Chlorine, if Needed	25%
Coagulant	Package Plate Settler Rapid Mix /	L	Enhance Settling of Backwash Waste Solids	100%

TABLE 4-17
Chemical Systems Conceptual Design Criteria

Chemical	Dosing Location	Dosing Location Description	Application	Storage Strength
	Flocculation Tanks			
Sodium Hypochlorite	RO CIP	M	RO Membrane Cleaning	50%
Citric Acid	RO CIP	N	RO Membrane Cleaning	50%
Sulfuric Acid	MF Neutralization	O	Neutralizing Agent	TBD
Sodium Hydroxide	MF Neutralization	P	Neutralizing Agent	TBD
Sodium Hydroxide	Finished Water	Q	Stabilization / corrosion control	50%
Sodium Hypochlorite	Finished Water	R	Chloramine Boost / Biofouling Control	12.5%
Liquid Ammonium Sulfate	Finished Water	S	Chloramine Boost / Biofouling Control	40%

Power Supply

Preliminary power needs for the RCS treatment facility have been identified as described below. Details of these power needs are discussed in *Chapter 5 – Power Supply Alternatives*. The total power requirement for the treatment facility was determined based on preliminary design criteria for all process equipment, pumps, HVAC, lighting, and miscellaneous loads. New electric transmission lines and substations would be necessary to transmit the required power from existing electric utility facilities to the new treatment facility locations. Through this study several existing electrical facilities adjacent to the project area were identified to supply the required power. Preliminary alternatives were then developed to bring the power to the new treatment facility. It should be noted that the final transmission facilities and corridors would be based on IID and SDG&E preferences and/or limitations of capacity as part of future phases of this project. Below is a summary of the power supply alternatives for each treatment facility location (3A, 5A, and 5C) as presented in Chapter 5.

- Imperial County 3A – Power supplied by a loop-in / loop-out transmission line from IID's existing 161 kV Transmission line. A step-down substation would be provided.
- 5A – Power supplied from proposed new IID 230 kV substation. A transmission line from the substation to the treatment facility and a new step-down substation would be provided.
- 5C – Power supplied from proposed new IID 230 kV substation. A transmission line from the substation to the treatment facility and a new step-down substation would be provided.
- San Diego County 3A – Power supplied from a tap line on SDG&E's 230 kV transmission line. A new step-down substation would be provided.

4.4.3 Alternatives Evaluation of RO vs EDR

The main treatment processes considered for the RCS treatment facility conceptual design presented above are based on the use of RO technology with MF pretreatment to reduce the salinity of the Colorado River water. An alternative technology applicable for the desalination of brackish water is Electrodialysis Reversal (EDR). This section provides a general comparison of the two technologies from both a qualitative and quantitative perspective.

Introduction to Electrodialysis Reversal

EDR operates by applying direct current to non-sacrificial positive and negative electrodes passing that charge across a water stream flowing between ion exchange membranes. In doing so, charged ions are drawn from the feed stream, across the ion exchange membranes into a concentrate stream, thereby removing charged inorganics from the feed stream. In EDR the polarity of the charge is periodically reversed to remove and inhibit inorganic scale formation onto the ion exchange membranes. These ion exchange membranes are arranged in a stack formation with alternating anion and cation exchange membranes and are referred to as a membrane stack.

Potential Benefits of Electrodialysis Reversal

EDR is a charged based desalination process that removes charged ions from a feed stream and does not directly treat or filter the feed flow stream. Because the EDR process only removes ionized ions, it is not able to remove silica, which could be an advantage in brackish desalination applications because silica does not concentrate during the desalination process and therefore is not a recovery limiting constituent. Additionally, because ion removal is charge driven and not a pressure separation, flux driven membrane fouling is not as direct a concern with EDR as it typically is for RO. In addition, EDR could require less costly pretreatment than RO. In general, for influent TSS concentrations ≤ 30 mg/L no pretreatment is required. For TSS values above 30 mg/L, multimedia filtration (MMF) is required.

However, despite these benefits inorganic scale formation and membrane fouling have been observed in EDR processes and are operational concerns that must be managed. For example, previous EDR groundwater pilots in Southern California have reported that excessive calcium carbonate concentration in the EDR concentrate stream have led to significant scale formation at higher recoveries. For RO applications, the scale formation risk occurs at the surface of the membrane where separation occurs, however for high recovery EDR the location of the scale formation is not at the membranes surface but is within the brine stream. As a result, acid consumption for calcium carbonate is shifted to a different location in the process but is not eliminated.

Electrodialysis Reversal System Conceptual Design

While both RO and EDR technologies have merit for consideration for the given application, each require different design approaches due to how they perform desalination. For the RO system greater than 99 percent of dissolved salts are rejected by the membrane, resulting in a very low TDS RO permeate. In the case of this application the low TDS RO permeate could be blended with raw water to form a larger stream of moderately saline finished water, thereby reducing the RO system required footprint by approximately 50 percent.

In the case of an EDR design, the mechanism for desalination is not pressure driven as it is with the RO but rather is charge driven as previously discussed. The implication of this desalination mechanism is that ions are removed from the feed stream, driven by electrical power consumption, as the feedwater passes through the process, resulting in a less saline effluent. To increase removal of these ions either additional EDR stacks are required, or additional power application to membrane stacks would be required. In both scenarios, power in the form of electrons are being consumed to remove charged ions from the feed stream. This desalination process therefore allows for a feed water stream to be desalinated down to target salinity. This is different from RO which universally removes large percentages of inorganics and is less tunable from a finished water permeate salinity perspective. As a result, the design presented below has been prepared to maximize recovery of the RCS feedwater and meet the desired finished water salinity.

Conceptual design criteria for EDR system based on RCS influent flow of 269 mgd is provided in Table 4-18. In coordination with EDR vendors, the conceptual design is based on treating all the flow through EDR to target 500 mg/L effluent TDS. The alternative approach would have been to increase energy consumption and target a lower EDR salinity that could be blended with bypass, however in discussion with EDR vendors this approach was selected. For sizing of the facility, previous inland EDR pilot study conceptual design reports were reviewed and identified a typical facility production capacity density of 2,975 ft^2 per mgd of finished water. Because the production capacity density of RO is already more productive than EDR, this results in a significantly larger facility footprint for this approach.

TABLE 4-18
EDR Conceptual Design Criteria

Design Parameter	Units	Value
EDR Trains		
Number of Units	#	24
Number of Lines	#	8
Quantity of Spare Units	#	0
EDR Unit		
Electrical Stages	#	2
Hydraulic Stages	#	2
Cell Pairs per Stage	#	750
TDS Removal per Stage	%	32%
EDR Energy Consumption		
Pumping Power	kWh/kgal	1.6
Desalination Power	kWh/kgal	0.9
Total Power	kWh/kgal	2.5
Total Power Load	MW	626

TABLE 4-18
EDR Conceptual Design Criteria

Design Parameter	Units	Value
EDR Facility Design		
EDR Feed Flow	mgd	269
EDR Product Flow	mgd	256
EDR Brine Flow	mgd	13.4
EDR System Recovery	%	95%
EDR Finished Water TDS	mg/L	450
Projected Building Footprint	ft^2	762,000

Summary & Recommendations

The key findings of the RO vs EDR evaluation follow:

1. Based on the EDR configuration considered as part of this evaluation, the EDR footprint is estimated to be approximately three times greater than RO due to the need to treat the entire influent flow (unlike RO, which approximately 50% of the influent flow is bypassed and blended with permeate to meet the finished TDS target) to meet the target finished water flow and TDS requirements. Note that an EDR system for the given application could be designed with a lower foot-print by sizing the electrical capacity of the units or configuring units in series to achieve a lower effluent TDS allowing bypass. This approach would require overall higher power consumption.

2. EDR would require less costly pretreatment process than RO (i.e. MMF vs MF) however the capacity of the pretreatment process would be greater for EDR as the influent flow would require treatment.

3. The estimated unit energy required for EDR (kWh/1000 gallons treated) is similar to RO.

4. The estimated achievable recovery of EDR is higher than RO (i.e. 95% compared to 85%) resulting in a lower brine flow than RO (i.e. approximately 13 mgd compared to 20 mgd). Note with HRRO the RO brine flow is estimated to be approximately 10 mgd.

If the Water Authority is interested in further considering EDR, Black &Veatch could perform a comparison of the conceptual capital and Operation & Maintenance costs of EDR and RO as part of the Phase B evaluation, if authorized.

4.5 Treatment Facility Conceptual Site Layouts

This section provides conceptual layout of the proposed treatment facility associated with the RCS. The layout is based on existing treatment facilities that consist of similar processes and finished water capacity requirements as that proposed for the RCS treatment facilities. Information from these facilities were used as a guideline to estimate the footprint requirements for major process components as described below. These facility components are shown on Figure 4-10.

- **Influent Storage Forebay and Pump Station** – A reservoir and pump station (not shown on Figure 4-10) would store Colorado River water conveyed from the AAC. The forebay would provide an operational storage buffer to provide operators time to shut down the treatment facilities in the event there is a disruption in the influent flow supply. For planning purposes, the reservoir would be sized to provide 60 minutes of retention time.
- **Automatic Strainers** – This facility device is used to remove particles and debris serving as pretreatment to the MF membranes.
- **MF Building** – This building would contain the major equipment associated with the MF system including membrane skids, valve racks, air compressors and receivers, CIP system, and electrical and control room.
- **MF Break Tank and Transfer Pump Station** – This tank would be used to store MF/UF filtrate for use as feed to the downstream RO process and backwash water for the MF/UF system.
- **RO Building** – This building would contain the major equipment related to the RO system including high pressure pumps, membrane skids (pressure vessels, membrane elements, valves, piping, instrumentation, booster pumps, etc.), CIP system and flush system.
- **Brine Volume Minimization** – This building contains the equipment necessary to further treat the RO concentrate reducing the volume and providing additional permeate that is blended with RO permeate and bypass to produce finished water. The HRRO system includes brine storage feed tank, feed pumps, HRRO skids, recirculation pump, and CIP system.
- **Recovered Water Basin & Pump Station** – This system would store and pump backwash water produced from the MF system to the packaged plate settler units.
- **Packaged Plate Settler Treatment Units** – These units include rapid mix tank, flocculation tank, and plater settlers to remove solids from the MF backwash water.
- **Solids Handling Facility** – Settled sludge from the PSU's would undergo solids handling consisting of sludge thickening, mechanical dewatering, and sludge drying beds. Solids would be collected in roll off dumper and disposed off-site.
- **Chemical Storage and Feed Facilities** – This area would be used for chemical storage and feed facilities.
- **Membrane Cleaning Waste Storage Tanks** – These tanks would be used to store spent cleaning solutions generated from membrane CIPs. The contents of the tanks would be emptied and hauled offsite for disposal.
- **Maintenance Building** – This building is dedicated to all things related to the preventive and corrective maintenance of the treatment plant.

- **Electrical Building** – This facility contains all the necessary electrical equipment to power the entire water treatment facility.

Administration, Operations, and Laboratory Building – The footprint allotted for this building is approx. 26,000 square feet and could be designed with multiple levels. Due to the remote locations of the Water Treatment Facility (WTF), the building would include an onsite laboratory for processing water quality samples.

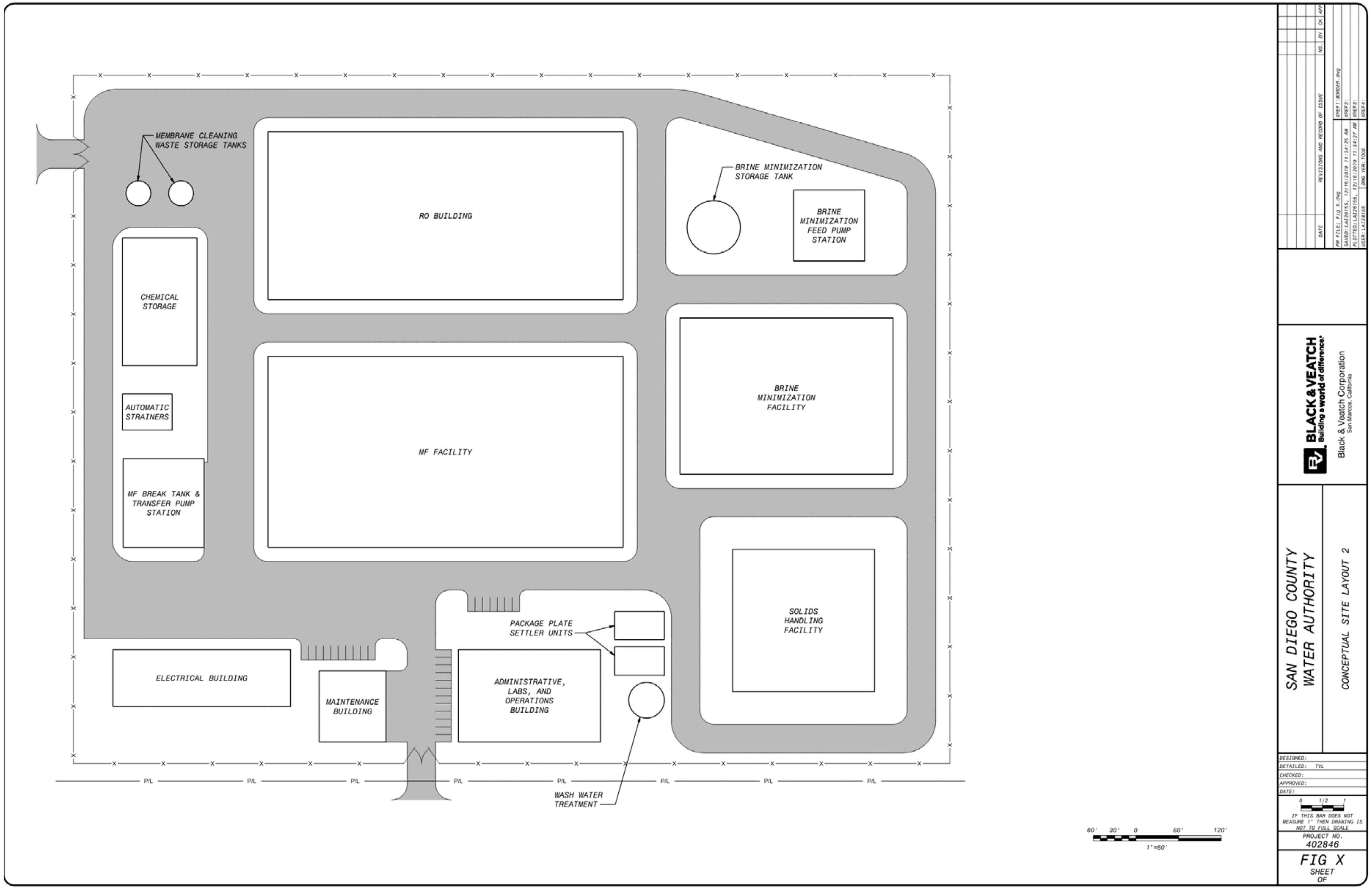

FIGURE 4-10
Conceptual Treatment Facility Layout

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4.6 Brine Management Options

The section includes an overview and regulatory requirements for brine management options described below.

- **Alternative 1 Conveyance to the Salton Sea:** This alternative applies to the treatment facilities associated with Alternatives 3A, 5A and 5C within Imperial County. Several options were evaluated to convey brine to the Salton Sea including: direct release, release to tributaries, and convey to constructed wetlands.
- **Alternative 2 Brine Volume Minimization followed by Evaporation Ponds:** This alternative applies to treatment facilities associated with Alternatives 3A, 5A and 5C within Imperial County. A desktop study of various brine volume minimization technologies was prepared as presented in Appendix D. As presented in Section 4, HRRO was selected as for conceptual design as this technology represents a current commercially viable option for brine volume minimization for the given application.
- **Alternative 3 Conveyance to Oceanside Outfall:** This alternative applies to the treatment facility associated with Alternative 3A located within San Diego County. Capacity in the Oceanside Outfall was evaluated as part of this alternative.

4.6.1 Overview of Brine Management Alternatives

This section provides a brief overview of each of the three brine management alternatives.

4.6.1.1 Alternative 1 Conveyance to the Salton Sea

Direct Release

The Salton Sea has historically struggled with managing the concentration of TDS in its water and has been making an effort to improve its existing conditions. One of the water quality objectives for the Salton Sea which can be found in the *Water Quality Control Plan for Region 7 of the Colorado River Basin*, is to reduce the present level of salinity and stabilize it at 35,000 mg/L (California Regional Water Quality Control Board, 2017) from its former concentration in 1992 of 44,000 mg/L. Assuming a brine concentration range of nearly 5,600 mg/L to 11,000 mg/L depending on the recovery rate leaving the RO system, this would still be significantly lower than their target TDS of the Salton Sea. Although permits and other regulations such as the *California Environmental Protection Agency's Policy for the Implementation of Toxics Standards, The Clean Water Act* and the *Colorado River Basin Plan* would need to be complied with, there is a high possibility that the brine would be permitted to be conveyed to the Salton Sea. This is discussed further in Section 4.6.2 of this chapter.

Tributaries of the Salton Sea

Tributaries of the Salton Sea such as the New River and San Felipe Wash were examined as means to manage brine as well. Unfortunately, the Colorado River Basin Plan lists a blended TDS objective of 4,000 mg/L for these bodies of water. With RCS brine concentration between 5,600 – 11,000 mg/L, it is unlikely that permission would be granted for release into these tributaries.

Partnership Opportunities for Constructed Habitats/Wetlands

Constructed wetlands and habitats are innovative solutions for managing waste flow. These systems are typically highly attractive to inland cities that are looking to remove contaminants from their waste flows, supply a fresh source of saltwater to a select vegetative area, or to gentrify their community. This project scope, when applied to the QSA water, would most likely call for the planning and construction of a large wetland area to support local wetland development and act as a communal space for recreation.

When brine is released into the wetland, the vegetation naturally filters the water containing toxins such as arsenic and selenium, which is supported by the rich source of microbial communities and various soils present in the wetland in addition to the plants (Arizona State University, 2014). While the quantities of these toxins are reduced, salinity increases due to evaporation, and a stream with a lower flow rate than the feed stream would exit the wetland. Regardless, the concentration of salt in the wetland would still be significantly less than the 35,000 ppm required to build habitats and cover playas.

From this point, that water could be sent to its final destination, such as a nearby lake, river, or sea. The wetland could also serve as a recreational area.

As part of this evaluation several potential partnering opportunities were identified related to conveying brine from the RCS treatment facilities to constructed wetlands.

- **State Salton Sea Management Program (SSMP)** – The SSMP is the State's phased approach to restoration, developed in the wake of Governor Brown's formation of the Salton Sea Task Force in 2015 and the establishment of near-term and mid-term goals for addressing exposed playa. The Species Conservation Habitat (SCH) Project is an example of one of the projects contained in the SSMP Phase I. The SSMP states that, "The Project (SCH) will create a series of gravity-fed ponds with islands and areas of varying water depths to serve as fish and avian habitat. The water sources for the Project will be brackish water pumped from the New River, runoff from the Existing Drainage Facilities, and saline water pumped from the Salton Sea." Water is available for Phase I of this project, however, demands for the later aspects of the project still need to be calculated to determine water availability for the longer term.

  There is a potential partnership with the State that could be further explored as there is a need for brine with a TDS below 35,000 ppm to build habitats and cover playas for Phase II of the SSMP. Prior to reaching an agreement with the State, many environmental regulations and permits would need to be obtained or met. Examples of these regulatory requirements include the Water Quality Control Plan for the Colorado River Basin – Region 7, California Environmental Protection Agency's Policy for Implementation of Toxics Standards and The Clean Water Act and would be discussed further in the regulatory part of this report.

- **The New River Improvement Project** - This project was initiated by the California-Mexico Border Relations Council (CMBRC) in 2009 to improve the quality of the New River. Historically, the New River has often been called the most polluted river in America, and for good reason. Industrial and domestic waste from Mexico is discharged into the river, creating an extensive list of water quality issues, such as pathogens, dissolved oxygen, toxicity, trash, mercury, and others (New River Improvement Project Technical Advisory Committee, 2011). Over the past several decades, The United States has been working with Mexico to improve the cleanliness of water that crosses the border and eventually ends up in the Salton Sea. One aspect of their New River Improvement Plan is to transform the ecology of the area by implementing several constructed wetlands, whose goal is to heal and rebuild the local ecosystems and river corridor conditions, as well as to supply cleaner water to the Salton Sea.

  There is the potential to blend the RCS brine with water in the New River, which could improve the water quality of the river. The anticipated exiting TDS concentration of brine leaving the RCS treatment plant would range from 3,600 to 5,600 mg/L, but when blended with the New River water would likely be lower than the 4,000 mg/L limit of the Colorado River Basin Plan for Region 7. The river water would reduce the TDS of the brine, but the brine could improve other river water quality parameters such as turbidity, DO, toxicity, etc.

- **New Constructed Wetlands** – In addition to the options noted above another alternative would be the creation of new wetlands associated with the development of the RCS project. Similar to the benefits described above, these new wetlands would allow for an innovative solution for managing the brine, while providing for fish and avian habitat and improve air quality along the sea shoreline. The addition of these new wetlands would add additional cost to the project and would need to be further evaluated under future phases of the project.

Note that neither the costs nor the benefits of these potential partnerships were accounted for in Phase A baseline cost estimates. Phase B, should it move forward would include further exploring these potential partnership projects in greater detail with the respective partners and stakeholders. Partnership opportunities are discussed in greater detail in *Chapter 8.0 - Partnership and Funding Opportunities*.

4.6.1.2 Alternative 2 Brine Volume Minimization followed by Evaporation Ponds

The previous studies identified evaporation pond size of greater than 5,000 acres. Due to sizable land requirements, a desktop evaluation (Appendix D) was conducted to evaluate innovative technologies that have the potential to reduce evaporation pond size requirements by either increasing evaporation rates or reducing the volume of brine that is released to the evaporation ponds. This section provides a summary of the desktop evaluation and conceptual design criteria for evaporation pond sizing based on the use of upstream brine volume minimization.

Summary of Brine Management Technologies

This section provides a summary of the desktop evaluation of developing technologies that could potentially be used to increase evaporation rates or reduce the volume of brine leaving the treatment facility, resulting in smaller evaporation ponds. Table 4-19 below highlights key points about each technology including purpose, summary of findings and whether the technology is recommended for further consideration in future phases of the project.

TABLE 4-19
Summary of Innovative Brine Management Technologies

Technology	Purpose	Summary of Findings	Recommended for Future Consideration
Wind Aided Intensified Evaporation (WAIV)	Reduce Evaporation Pond Size	Vertically packed wet surfaces that increase the surface area of an evaporation pond while decreasing its perimeter. Two pilot studies have been carried out.	Yes, as it has the potential to reduce the size of evaporation ponds by 50%-90%, though technology at infancy and with only few small-scale commercial applications.
SolarBee	Reduce Evaporation Pond Size	Solar powered reservoir re-circulator that prevents salt crystals from forming in evaporation ponds.	No, as this technology would not significantly increase evaporation rates.
Evaporation Cannon	Reduce Evaporation Pond Size	Quickly pressurizes and expands water to create a mist which continually rotates through either 90 or 180 degrees. Estimation predict that approximately 20 – 50% of the water fired through the canon would evaporate.	Yes, this technology could reduce the evaporation pond size by increasing the evaporation rate.
High Recovery Reverse Osmosis (HRRO)	Brine Volume Minimization	High recovery RO process designed to operate in batch mode with concentrate recycle to potentially reduce scaling and allow higher recovery than traditional RO.	Yes, as it could possible reduce the volume of brine produced for the RCS project by 50%.
High Efficiency Reverse Osmosis (HERO)	Brine Volume Minimization	RO system which uses ion exchange, CO_2 reduction and a pH increase for pre-treatment steps, resulting in recovery rates up to 98%.	No, as it has very limited uses for brackish water.
Flow Reversal Reverse Osmosis (FR-RO)	Brine Volume Minimization	Designed specifically to treat brackish water and prevent scaling, FR-RO periodically switches the feed and exiting sites within the pressure vessel.	Yes, however technology is still in its infancy and first scale application scheduled for 2020.
Vibratory Shear Enhanced Process (VSEP)	Brine Volume Minimization	A vibrating membrane system that the manufacturer claims to outperform RO in the following categories: fouling resistance, high solids, efficiency, dependability, cost, convenience, etc.	No, limited pilot test data which does not support manufacturers claims.
Membrane Distillation	Brine Volume Minimization	An evaporation process driven by the difference between partial pressures on one side of a porous hydrophobic membrane and the other. Advantages include high quality water, little pre-treatment and low pressures.	No, because this process has no full-scale applications for brackish water.

TABLE 4-19
Summary of Innovative Brine Management Technologies

Technology	Purpose	Summary of Findings	Recommended for Future Consideration
Dewvaporation	Brine Volume Minimization	Brackish water is evaporated by heated air which deposits fresh water as dew on the opposite site of a heat transfer wall. The energy needed for this process is supplied by the energy released from dew formation and this process does not have scaling issues.	No, because there are no full-scale applications of this technology.
Forward Osmosis (FO)	Brine Volume Minimization	The opposite of RO. The driving force for this process is osmotic pressure, which reduces energy costs.	No, limited full-scale applications at this time. Issues with recovery of draw solution.
Electrodialysis Reversal (EDR)	Brine Volume Minimization	The opposite of electrodialysis. This technology is not impacted by scaling and is used for the application of high-water recovery from highly saline water.	Yes, as it has achieved 95-98% recovery when combined with RO. Possible alternative to HRRO.
Electrodialysis Metathesis (EDM)	Beneficial Reuse	Amplifies water recapture by separating the charged stream components into partitions.	No, there are no municipal applications of this technology or commercial equipment providers currently.
Bipolar Membrane Electrodialysis (BMED)	Beneficial Reuse	Utilizes the principles of membrane electrodialysis in conjunction with a bipolar membrane to produce H^+ and OH^- ions from water.	No, as it is too energy intensive.
Electrochlorination	Beneficial Reuse	An electrolytic cell electrolyzes the diluted brine into sodium hypochlorite and hydrogen gas.	No, the concentration of salt in the RCS brine is too low for this process to be economically favorable.
Struvite Recovery (IX)	Beneficial Reuse	This process separates phosphate from the RO concentrate by using an ion exchange process to precipitate out a usable compound, struvite.	Not an economically favorable technology.

Evaporation Ponds

Background

Solar evaporation is a simple and well-established process used to remove water from a concentrated solution. It's easy implementation and operations processes make it an appropriate and attractive release method. When applied to the correct circumstances, such as low flow rates, warm, dry climates and cheap, flat land, evaporation ponds could be a cost and energy efficient way to treat brine (Giwa, 2016). Due to the complexity of the evaporation process, it was decided that the decision to proceed with an evaporation pond be re-evaluated.

Evaporation rates are affected by many variables, such as relative humidity, air temperature, barometric pressure, depth of the evaporation pond, surface area of the pond, brine concentration, wind, etc., making it difficult to estimate an accurate value for the rate of evaporation. However, it is certain that evaporation rates of brine are lower than evaporation rates of pure water and that varying evaporation rates between the summer and winter would require thoughtful sizing to accommodate the build-up of water in the cooler months. In addition to the difficulty associated with estimating values, there are many costs that accompany evaporation ponds that could quickly add up (Mickley, 2006). These costs include, but are not limited to, land, land clearing, building and maintaining a pipeline from the treatment facility to the evaporation pond, a pump to transport the brine to the pond, roadways, pricey pond liners and seepage monitoring systems, fencing, dike construction, release of sludge and dealing with precipitate buildup on the bottom of the pond, which could easily take expenses over budget.

Conceptual Size and Cost of Evaporation Pond

Conceptual sizing of the evaporation pond required to dispose of the brine from the HRRO process was conducted as part of this evaluation. In general, a similar approach was taken to size the evaporation ponds as used for the previous study. The exception was the revised approach includes a safety factor, to account for seasonal changes in evaporation rates and precipitation. The conceptual design criteria for evaporation pond is provided in Table 4-20.

TABLE 4-20
Evaporation Pond Conceptual Design Criteria

Parameter	Units	Value
Influent Flow	Mgd	10
Influent TDS	mg/L	11,000
Pan Evaporation Rate(1)	inches/year	86.21
Precipitation Rate(2)	inch/year	0.8
Pan Coefficient(3)	-	0.7
Salinity Factor(4)	-	0.995
Brine Concentration Factor(5)	%	1.1
Safety Factor(6)	Multiplier	1.5
Estimated pond size	Acre	3,390

Notes:

1. Value shown is based on Mexicali's annual pan evaporation rate (Western Regional Climate Center, 2019).
2. Imperial, CA (Average Weather in Imperial CA, Weatherspark, 2016.
3. Per guidelines provided in Membrane Concentrate Disposal: Practices and Regulation (Mickley & Associates, 2006).
4. Per guidelines provided in Membrane Concentrate Disposal: Practices and Regulation (Mickley & Associates, 2006).
5. Per guidelines provided in Membrane Concentrate Disposal: Practices and Regulation (Mickley & Associates, 2006).
6. Accounts for seasonal changes in evaporation rates and precipitation (Bond, 2008).

The new analysis as presented above was conducted to verify the cost and area needed for an evaporation pond fed with HRRO concentrate. The calculation yielded a total required pond size of 3,390 acres. In comparison, an evaporation pond fed with water from the RO concentrate would require 6,795 acres of land.

A cost analysis was carried out using the assumptions from the 2012 evaporation pond cost estimate, which could be found in the *2013 Master Plan Update*. With the exception of altering the land acquisition cost to $18,338 per acre, the costing information from 2012 was applied to the RO concentrate evaporation pond size of 6,795 acres and scaled up around 2.1% per year to account for inflation. This cost equates to $1.4B in 2019. When the 3,390-acre evaporation pond leaving the HRRO unit was run through the same financial equations, the resulting cost came out to be $719M in 2019. This is nearly 50% of the cost associated with the evaporation pond fed with RO concentrate. Factoring in the capital equipment costs of the HRRO, which is approximately $30M, initial results indicate implementing brine volume minimization prior to evaporation ponds could have significant cost savings. A more detailed opinion of the cost of HRRO including land, construction, O&M is presented in *Chapter 5 - Risk, Cost Opinions and Economic Analysis.*

4.6.1.3 Alternative 3 Release to Ocean Outfall

Utilizing the Oceanside Ocean Outfall (OOO) was also discussed as an option for brine management if treatment was carried out at the Alternative 3A San Diego County treatment facility. This would consist of a 12-mile pipeline from the new treatment facility to the outfall where the brine would then be carried to the Pacific Ocean. Upon further research, there is not sufficient capacity in the OOO to combine our flow with the existing releases. The OOO is rated at 24.4 mgd with a current combined release of 22.6 mgd (State Resources Control Board, 2011). The volume of brine needed to be managed is likely around 20 mgd if recovery runs at 85%, making this option infeasible. A new land and ocean outfall would be needed if the Alternative 3A San Diego County treatment plant location is chosen. See Section 4.7 of this chapter for further details on this option.

4.6.2 Regulatory Requirements

In order to get a better understanding of which brine management options are the most feasible, the regulatory and permitting information were explored. This section covers parts of *The Water Quality Control Plan for Region 7 of the Colorado River Basin* (Basin Plan), *The Clean Water Act* (CWA), and *The Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California* (California Toxics Report). Some additional information about the Colorado River water quality has also been included.

4.6.2.1 Colorado River Water Quality Considerations

The salinity of the Colorado River is subject to wide variation due to ongoing hydrologic conditions. As a result of natural and human induced causes, the river picks up salts and minerals during its 1,400-mile trip from Colorado to Mexico where the TDS increases from 50 mg/L to 879 mg/L. In 1975, the Colorado River Basin Salinity Control Forum established water quality standards for three locations along the Colorado River using the flow-weighted average annual salinity which enforces a maximum TDS of 879 mg/L at the Imperial Dam.

As part of determining the salinity reduction requirements, the project team reviewed several sources of Colorado River water quality.

In order to develop a baseline for the water quality of the untreated Colorado River water that would be needed to meet the regulations above after it is treated, the *2017 Water Quality Report to Member Agencies – Metropolitan Water District of Southern California* has been included. Any cell marked as ND indicates that no traces were detected while cells marked as NM indicate that the parameter of interest was not measured.

As presented in Table 4-21, most metals studied in the California Toxics Report were not detected in the untreated Colorado River water.

TABLE 4-21
Summary of General Mineral, Physical, Trace Metals Analyses of Colorado River Aqueduct Water Supplies from Appendix G: Colorado River Conveyance Alternative Report

Parameter	Unit	IID AAC Drop 4	IID AAC Design Feedwater
Arsenic	ppb	ND	NM
Cadmium	ppb	ND	NM
Chromium	ppb	ND	NM
Copper	ppm	ND	NM
Lead	ppb	ND	NM
Mercury	ppb	ND	NM
Nickel	ppb	ND	NM
Selenium	ppb	ND	NM
Total Dissolved Solids (TDS)	mg/L	762	879
Zinc	ppb	ND	NM

4.6.2.2 The Clean Water Act (CWA)

The CWA creates the baseline for regulating the release of contaminants into United States waters as well as water quality standards for surface water. Section 303(d) of CWA discusses water bodies deemed as being "impaired", meaning that it has already failed to meet specific water quality standards. The New River and Salton Sea are on the list of impaired bodies of water, which expresses two things: The Regional Water Quality Control Board may not allow or may limit the quantity of water that could be released to an impaired 303(d) body of water, and that the basin plan objectives may contain less rigid objectives for these specific locations. The associated risk that corresponds to the inability to obtain the required permits to convey brine to the Salton Sea either directly or via a tributary would dictate that an evaporation pond be used to handle the treatment plant effluent.

4.6.2.3 Water Quality Control Plan for Region 7 of the Colorado River Basin (Basin Plan)

The Water Quality Control Plan for the Colorado River Basin describes its water quality objectives for Region 7 as a whole and then breaks down some specific objectives for particular bodies of water. General basin plan objectives are blended average values and could be found below.

<u>General Basin Plan Objectives:</u>

- TDS: 4,000 mg/L
- pH: 6.0-9.0
- DO: 5.0 mg/L for warm water, 8.0 mg/L for cold water
- E. coli: 126/100mL for Rec I, 630/100mL for Rec II
- Salinity: 879 mg/L

Because the concentration of brine leaving the treatment facility exceeds the TDS limit in the Basin Plan objectives, it is highly unlikely that any bodies of water without specific objectives would permit the brine to be released into their system. As mentioned earlier, these basin plan objectives are blended averages. However, since they are impaired, it is improbable that approval would be granted to raise the overall TDS of the water. Fortunately, the basin plan does outline specific objectives for both the New River and the Salton Sea. These could be seen in detail below.

<u>Specific Basin Plan Objectives for the Salton Sea:</u>

- TDS: 35,000 mg/L
- Selenium: 0.005 mg/L yearly average

Since the probable concentration range for the RCS brine is between 5,600 mg/L and 11,000 mg/L, it is safely below the TDS objective for the Salton Sea. Despite the fact that selenium was not detected in the RCS water, TDS levels would increase significantly in the RO concentrate stream. Therefore, selenium levels should be verified before proceeding although it is unlikely that they would cause concern. It is presumable that RCS brine would be permitted in the Salton Sea.

<u>Specific Basin Plan Objectives for the New River:</u>

- BOD: 30 mg/L monthly grab sample
- COD: 70 mg/L
- pH: 6.0 – 9.0 weekly grab sample
- DO: 5.0 mg/L daily grab sample
- Fecal Coliform Organisms: 30,000 colonies/100 mL

The New River objectives do not list an additional TDS objective, consequently the 4,000 mg/L limit from the general basin plan objectives is assumed. It is doubtful that an impaired body of water such as the New River would authorize a flow with a TDS that exceeds their maximum TDS level to be blended with their water unless the blending took place before releasing to the river. This could also be said of the San Felipe wash and other tributaries into the Salton Sea.

4.6.2.4 Policy for Implementation of Toxics Standards for Inland Surface Waters, Enclosed Bays, and Estuaries of California (California Toxics Report)

Another water quality report that was reviewed was the California Toxics Report. This write-up covers the regulations on the releases of toxic pollutants into inland surface waters, enclosed bays, and estuaries in California and works hand in hand with the State's *Porter-Cologne Water Quality Control Act* and the CWA. Seen below is Table 4-22 that lists the limits for various toxic metals. The saltwater and human health columns apply to the Salton Sea while the freshwater and human health columns apply to the tributaries of the Salton Sea, such as the New River.

TABLE 4-22
California Toxics Report Blended Maximum Values for Saltwater, Freshwater and Human Health

Metal	Saltwater – Acute Toxicity (μg/L)	Saltwater – Chronic Toxicity (μg/L)	Freshwater – Acute Toxicity (μg/L)	Freshwater – Acute Toxicity (μg/L)	Human Health – Organisms Only (μg/L)
Arsenic	69	36	340	150	-
Cadmium	42	9.3	1.3	2.2	(1)
Chromium VI	1100	50	16	11	(1)
Copper	4.8	3.1	13	9	-
Lead	210	8.1	65	25	(1)
Mercury	1.80	0.94	1.4	0.77	0.051
Nickel	74	8.2	470	52	4600
Selenium	290	71	-	5	(1)
Zinc	90	81	120	120	-

Note:
1. USEPA is not promulgating human health criteria for these contaminants use the State's criteria for toxics.

As seen in the *2017 Water Quality Report to Member Agencies – The Metropolitan Water District of Southern California*, all the metals listed above, with the exception of arsenic, were not detected in the Colorado River near Lake Havasu and Lake Matthews. Arsenic was detected at low values with the maximum amount found at one reading to be 3.6 ppb. It should be noted again that TDS levels rise significantly in the RO concentrate stream, and although likely within the accepted range, should be evaluated as to remain under the California Toxics Report limits.

In conclusion, three reports were reviewed: Basin Plan, CWA, and the California Toxics Report. Based on these reports, managing brine in congruence with the Salton Sea would be the easiest scenario from a permitting standpoint. This is the only option where all the regulatory rules are met, since it had a specific basin plan objective that increased the allowable TDS up to 35,000 mg/L. Although not impossible, it is unlikely that authorization would be granted to release brine into any tributaries of the Salton Sea. However, mitigation could be relooked at in the future if the situation changes.

4.7 Brine Management Facilities

This section presents the brine management facilities required for each potential treatment location in Imperial and San Diego Counties.

4.7.1 Imperial Valley Alternative 3A

The Imperial Valley treatment facility associated with Alternative 3A could utilize two options for brine management as presented in Figure 4-11. The first and more practical solution would utilize a new 30-inch diameter pipeline to convey the brine to the Salton Sea for use in constructed wetlands as presented in the discussions above. Depending on what decisions the Water Authority makes to capture further QSA water (e.g. HRRO), brine management would range from 20 mgd without HRRO versus 10 mgd with HRRO. To cover this potential flow range, the pipeline was sized for 30-inch which provides for a minimum velocity of 3.2 fps and a maximum velocity of 6.3 fps. The pipeline would be approximately 2.4 miles in length.

The second and less likely alternative would include the construction of evaporation pond which would incorporate approximately 3,400 acres of land to dispose of brine. It's anticipated that the residual pressure from the treatment facilities, whether RO or HRRO would be sufficient to convey the brine concentrate for either of these brine management options.

4.7.2 Imperial Valley Alternative 5A

Similar to the Imperial Valley portion of Alternative 3A, the treatment facility associated with Alternative 5A could utilize two options for brine management as presented in Figure 4-12. The first and more practical solution would utilize a new 30-inch diameter pipeline to convey the brine to the Salton Sea for use in constructed wetlands as presented in the discussions above. Depending on what decisions the Water Authority makes to capture further QSA water, brine management would range from 20 mgd without HRRO versus 10 mgd with HRRO. To cover this potential flow range, the pipeline was sized for 30-inch which provides for a minimum velocity of 3.2 fps and a maximum velocity of 6.3 fps. The pipeline would be approximately 27.5 miles in length.

The second and less likely alternative would include the construction of evaporation pond which would incorporate approximately 3,400 acres of land to dispose of brine. It's anticipated that the residual pressure from the treatment facilities, whether RO or HRRO would be sufficient to convey the brine concentrate for either of these brine management options.

4.7.3 Imperial Valley Alternative 5C

Similar to Imperial Valley Alternatives 3A and 5A, the treatment facility associated with Alternative 5C could utilize two options for brine management as presented in Figure 4-13.

The first and more practical solution would utilize a new 30-inch diameter pipeline to convey the brine to the Salton Sea for use in constructed wetlands as presented in the discussions above. Depending on what decisions the Water Authority makes to capture further QSA water, brine management would range from 20 mgd without HRRO versus 10 mgd with HRRO. To cover this potential flow range, the pipeline was sized for 30-inch which provides for a minimum velocity of 3.2 fps and a maximum velocity of 6.3 fps. The pipeline would be approximately 31.7 miles in length.

The second and less likely alternative would include the construction of evaporation pond which would incorporate approximately 3,400 acres of land to dispose of brine. It’s anticipated that the residual pressure from the treatment facilities, whether RO or HRRO would be sufficient to convey the brine concentrate for either of these brine management options.

4.7.4 San Diego County Alternative 3A

Lastly the San Diego County Alternative 3A only has one practical solution for brine management, which consist of conveyance to an ocean outfall. Due to the existing topography around the proposed treatment facility and due to the high cost associated with purchasing land, an evaporation pond would not be a feasible option for brine management at this location. Instead a new pipeline would be required to convey the brine flow from the proposed treatment facility to an ocean outfall. As discussed in Section 4.1.6.3, the existing City of Oceanside Ocean Outfall was reviewed to determine if capacity within the existing system was available to take on this additional flow. However, it was determined capacity does not exist for the large volume of brine, and thus a parallel pipeline and outfall would need to be constructed as shown on Figure 4-13.

Depending on what decisions the Water Authority makes to capture further QSA water (e.g. HRRO), brine management would range from 20 mgd without HRRO versus 10 mgd with HRRO. To cover this potential flow range, the pipeline was sized for 30-inch which provides for a minimum velocity of 3.2 fps and a maximum velocity of 6.3 fps.

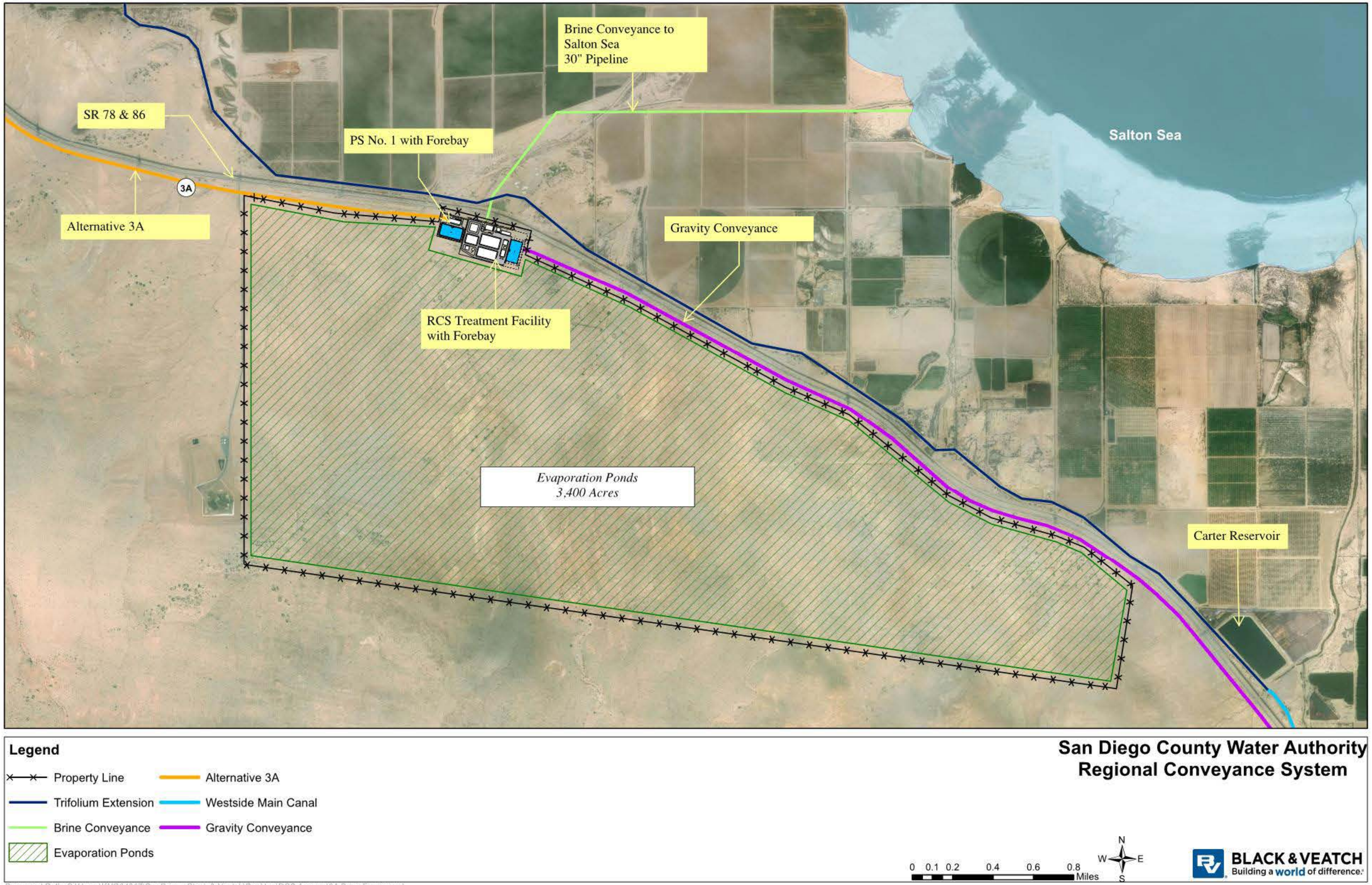

FIGURE 4-11
Brine Management Options – Alternative 3A Imperial Valley Treatment Option

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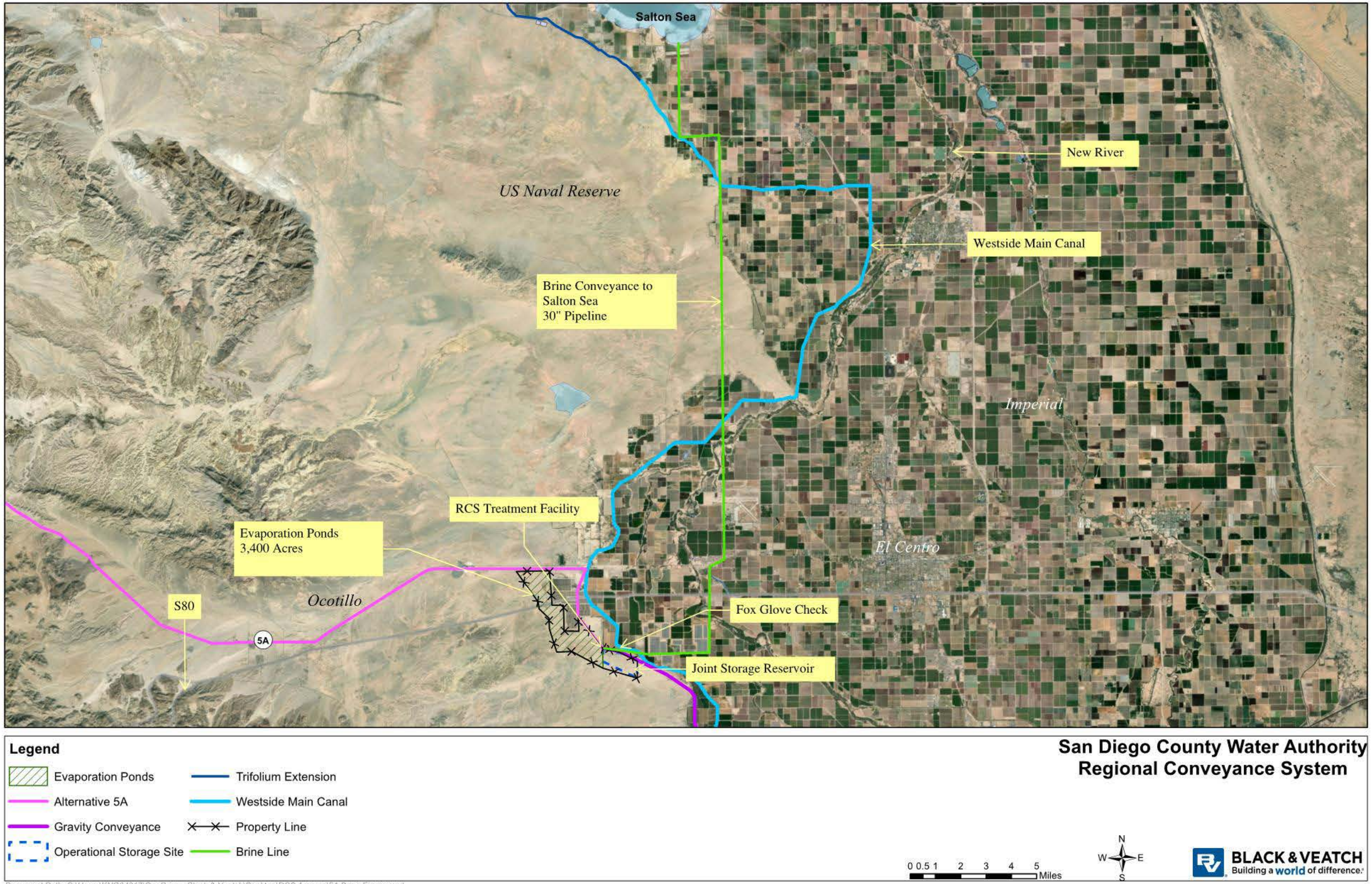

FIGURE 4-12
Brine Management Options – Alternative 5A

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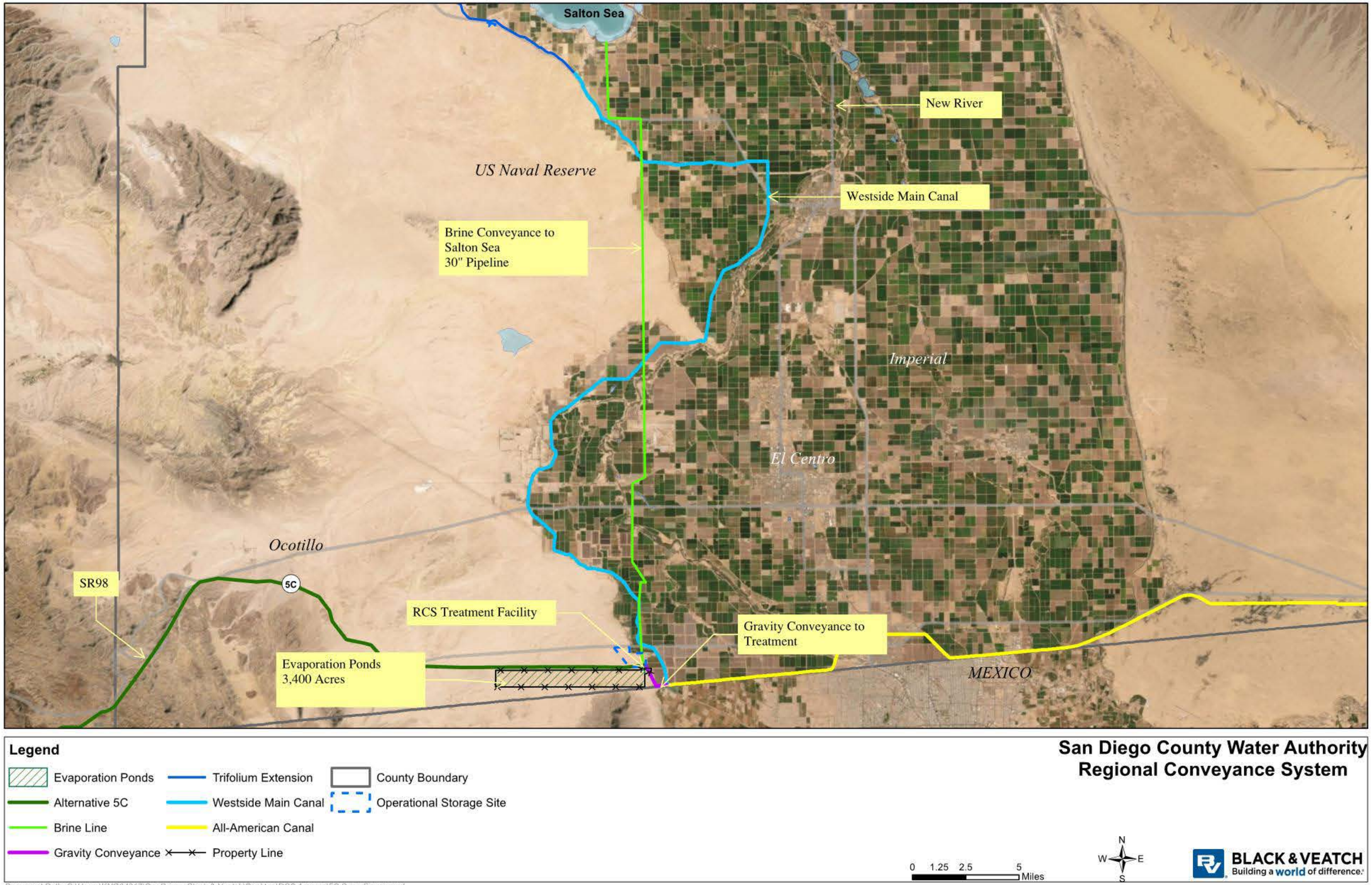

FIGURE 4-13
Brine Management Options – Alternative 5C

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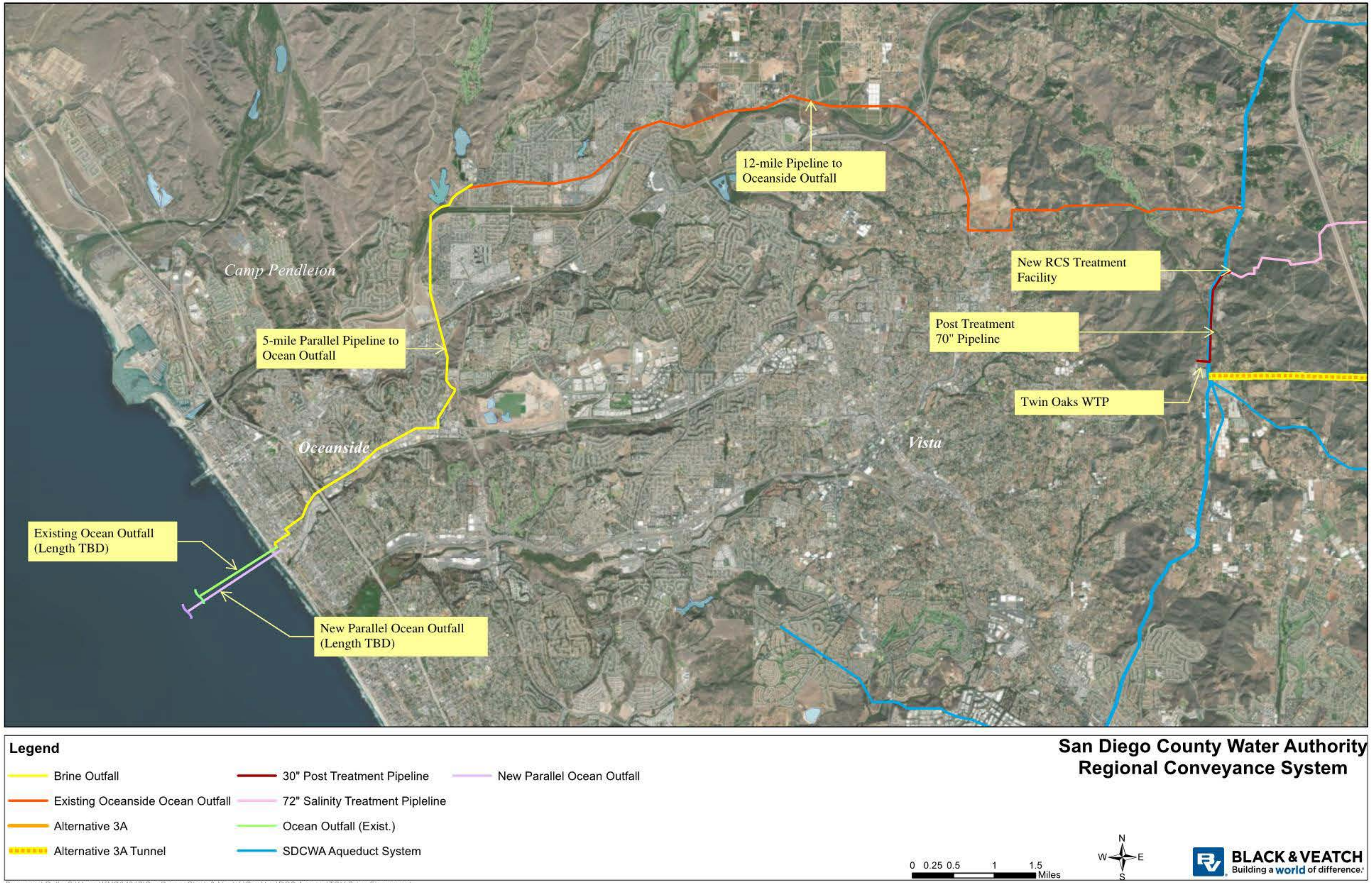

FIGURE 4-14
Brine Management Options – Alternative 3A San Diego County Treatment Location

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4.8 Conclusion

A preliminary investigation into the treatment of RCS water was conducted in this chapter, including a discussion on blending options, treatment facility locations, conceptual design, and brine management.

When blending options were evaluated at the SVR and TOVWTP, it was concluded that this method would not be able to reduce TDS levels below their target level of 500 mg/L and is no longer being considered as an option.

Four potential locations for an RCS treatment facility were discussed in this report, one in Imperial County for each alternative option and an additional facility location option for Alternative 3A in San Diego County. The facilities range in size from 43 to 51 acres and could potentially require 200 additional acres of land if evaporation ponds are required for brine management.

Research was conducted to determine if permitting could be obtained in order to convey RCS brine to the Salton Sea. Due to the Salton Sea's historic struggles with high TDS levels, whose target TDS is over three times more concentrated than RCS brine, and recent issues with air pollution from the exposed seabed, it seems likely that a permit could be obtained. However, since said permit is not guaranteed, analysis of brine management methods concluded that the best ways to manage RO concentrate are release to the Salton Sea, or the addition of a brine volume minimization technology. This would remove additional water from the RO concentrate, which would increase the quantity of final clean water and ultimately evaporate out the remaining water in the concentrated brine in an evaporation pond. Over a dozen brine volume minimization techniques were researched, and it was deduced that HRRO is the likely best option for RCS due to its recovery rates, energy use, small size and lower costs. EDR was compared to HRRO as another brine volume minimization technique, but the results were inconclusive about which method has the optimal cost:benefits ratio. Further analysis could be done in potential Phase B if the Water Authority would like to consider EDR.

During the conceptual design process, it was determined that the 301,500 AFY of raw water entering the treatment facility from the All-American Canal could produce 278,640 AFY of water without brine volume minimization, or 290,010 AFY of water with the additional of HRRO, all of which fall below the target TDS of 500 mg/L.

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5

POWER SUPPLY ALTERNATIVES

Chapter 5.0 Power Supply Alternatives

5.1 Introduction

5.1.1 Overview

This chapter describes the power supply alternatives for the RCS Alternative 3A. In addition, it discusses the power supply requirements and transmission systems for the treatment facilities associated with Alternatives 3A, 5A and 5C. The balance of technical evaluations for the pumping stations, pipelines and tunnels (including power supply) for Alternatives 5A and 5C were previously completed as part *2013 Master Plan*. Alternatives 5A and 5C costs have been updated to 2020 dollars in *Chapter 6.0 - Risk, Cost Opinions, and Economic Comparison*.

5.1.2 Chapter Organization

Key components for the RCS power supply alternatives are summarized below and discussed in the following chapter sections.

Power Supply Requirements – This section describes the overall proposed system and facility components requiring power.

Power Needs – This section provides a summary of the power needs per facility. Both temporary and permanent loads are identified.

Regional Transmission Facilities – This section identifies existing and proposed utility power supply alternatives for each power load requirement.

Rate Structures – This section reviews the rate schedules for both Imperial Irrigation District (IID) and San Diego Gas & Electric (SDG&E).

Renewable Energy Opportunities – This section provides an overview of renewable energy alternatives for the project. Further discussion is included in *Chapter 8 – Partnership and Funding Opportunities.*

5.2 Power Supply Requirements – Description of System

5.2.1 Description of System

The overall system would include a treatment facility, pumping stations, forebay, pipelines and tunnels, of which power would be required to be supplied from the local utility. The RCS alternatives that have been proposed would be constructed in both IID's and SDG&E's service regions. Each of these utilities have existing infrastructure, ranging from extra high voltage (500kV) to distribution voltages (12kV), that could be utilized to service the power needs at each respective facility location. Depending on the load that is required at each facility location, the utility in that service region would locate the nearby infrastructure that is capable of supporting that load and would work to integrate the facility into their transmission system. Figure 5-1 provides an overview of the system including all three alternatives and respective facilities. A description of each facility follows.

Treatment Facility. One treatment facility would be constructed to treat the Colorado River water to meet the maximum acceptable target TDS of 500 mg/L. The treatment facility would consist of influent storage forebay, strainer, MF, RO, cartridge filters, intermediate pumps, chemical systems, brine volume minimization system, backwash recovery system, and solids handling. The facility is further described in detail in *Chapter 4.0 – Treatment, Blending, and Brine Management Options*.

Each of the Alternative, 3A, 5A, and 5C, have one feasible location for the treatment facility in Imperial County. An additional treatment facility location north of the existing TOVWTP in San Diego County was evaluated for Alternative 3A and deemed infeasible as presented in *Chapter 4.0 – Treatment, Blending, and Brine Management Options* and will not be carried forward for potential future evaluations. As part of this evaluation, the necessary infrastructure improvements required to supply power were identified. In addition, anticipated energy rate structures for each major facility were determined based on conversations with Water Authority staff and the prospective energy providers, IID or SDG&E. The four potential treatment facility locations are shown on Figure 5-1.

Pipeline/Pump Stations. The pressurized portion of Alternative 3A would be comprised of a series of pipelines and tunnels that begins at Pump Station No. 1, located at the north terminus of the new gravity flow canal that extends approximately 47.1 miles from the end of the AAC. From there the RCS heads west via pipelines and pump stations and ends at a connection to existing Water Authority aqueduct facilities at the TODS. The alternative includes three intermediate pumping stations in series, to provide the energy required to overcome the changes in elevation and system head losses along the alternative and limit the maximum lift required to less than 1,000-ft. The pump type and configuration currently being considered includes three duty and one standby vertical turbine pumps with constant speed motors. The locations are shown on Figure 5-1 and further detail is provided in *Chapter 2.0 – Regional Conveyance System Operations and Sizing*. Alternatives 5A and 5C were previously evaluated as part of *2013 Master Plan* and not included in this power supply alternatives evaluation.

Each pumping station would be provided with a forebay that would be sized for normal startup and shutdown of the pumping station and for unscheduled outages of individual pumps or the entire pumping station.

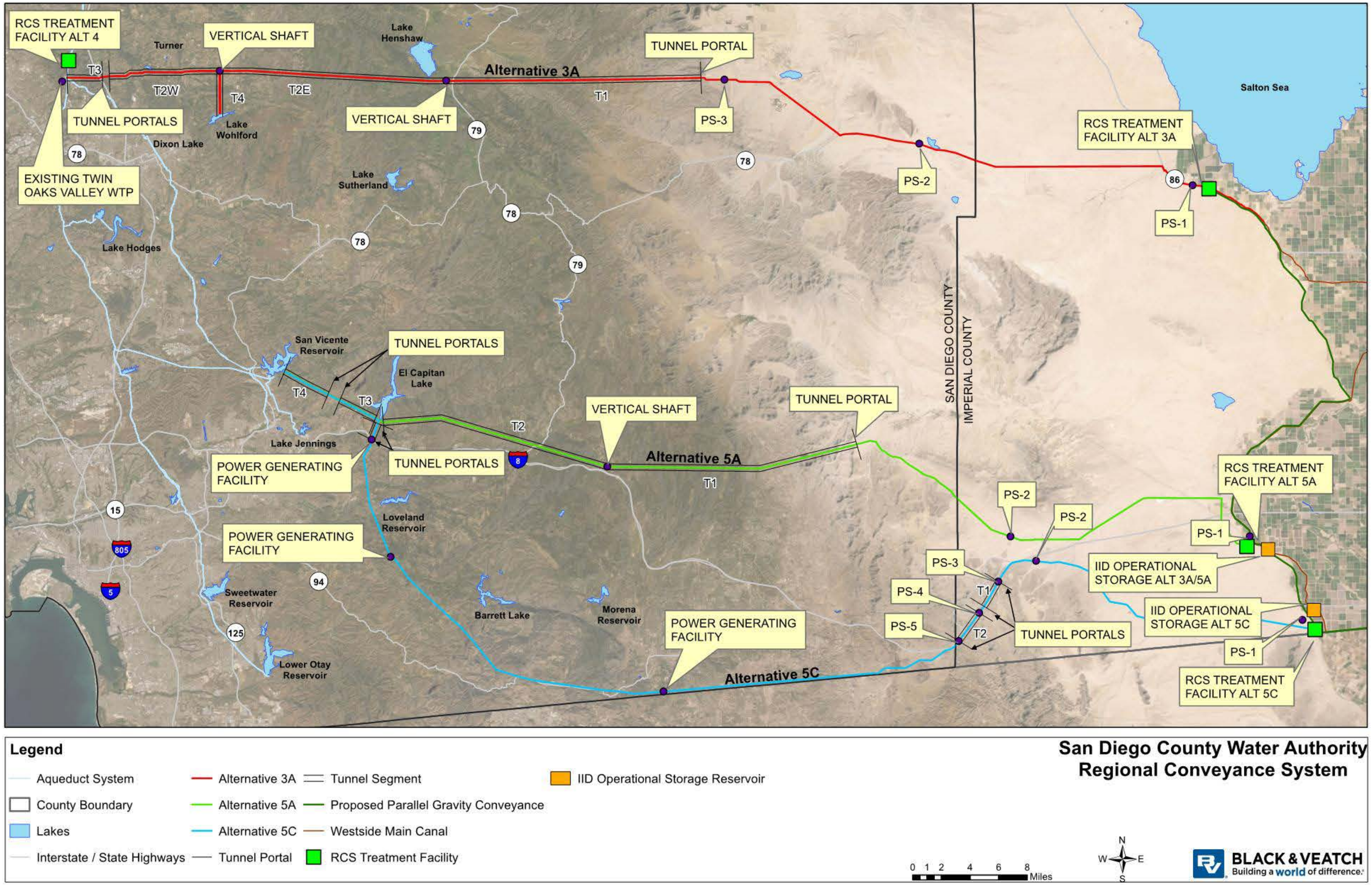

FIGURE 5-1
RCS Alternatives Overview Map

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Tunnel Portals. Alternative 3A has four defined tunnel segments (T1, T2, T3 and T4), as shown on Figure 5-1. All four tunnels along Alternative 3A are currently assumed to be located entirely within hard rock. Rocks of these types and strengths would require excavation using hard rock TBMs with disk and/or drilling and blasting. Tunnel shafts or portals are utilized for launching, assembling and retrieval of the TBMs. TBMs require a substantial amount of electrical power to run the large motors and hydraulic pumps onboard the TBM. In addition, power is required for dewatering purposes. The portals are shown on previous Figure 5-1.

5.3 Power Needs

5.3.1 Facility Power Needs

Preliminary power needs were identified for each facility and are listed in Table 5-1. The total power requirement was determined based on preliminary design criteria for all process equipment, pumps, TBMs, HVAC, lighting, and miscellaneous loads. The treatment facility and pumping station loads are based on new infrastructure which would meet and exceed the temporary demand loads during construction. At the tunnel portals, the power supply would be sized to accommodate the dewatering, TBMs, HVAC and lighting, all of which are considered temporary construction loads. As shown in Table 5-1 the total power requirement column is the larger of the permanent or temporary columns. The total power requirement column is what the power supply facilities would be sized for.

The main treatment processes considered for the treatment facility conceptual design would be based on the use of RO technology with MF pretreatment. As described in *Chapter 4.0 – Treatment, Blending, and Brine Management Options*, an alternative technology applicable for the desalination of brackish water is EDR. If requested by the Water Authority, Black & Veatch could perform a comparison of the conceptual capital and Operation & Maintenance (O&M) electrical costs (new power infrastructure and energy costs) of EDR as compared to RO as part of the Phase B scope of work, if authorized.

TABLE 5-1
Summary of Facility Power Needs

Facility	Power Requirement			Major Components
	Total (each)	Permanent (each)	Temporary (each)	
Treatment Facility	55 MW	55 MW	---	• MF (9 MW) • RO (9 MW) • Screening • Intermediate Pumps (35 MW) • HVAC • Lighting • Chemicals • Washwater Treatment • Brine Volume Minimalization
Pump Stations (3 total)	36 MW	36 MW	---	• Pumping Units (35 MW) • HVAC • Lighting • Misc. Loads

TABLE 5-1
Summary of Facility Power Needs

Facility	Power Requirement			Major Components
	Total (each)	Permanent (each)	Temporary (each)	
Tunnel Portals (7 total)	2 MW	<0.01 MW	1.99 MW	• Power to TBMs and associated systems • HVAC • Lighting • Dewatering

5.4 Regional Transmission Facilities

Electric transmission lines and substations are necessary to transmit power from existing electric utility facilities to the new treatment facilities, pumping stations and tunnel portals. We have identified several facilities adjacent to the project area and developed preliminary alternatives. However, the final transmission facilities and corridors would be determined based on IID and SDG&E preferences and/or limitations of capacity.

5.4.1 Existing Facilities

Based on input from IID, we have identified existing transmission facilities near the proposed facility power needs. Two existing substations (Imperial Valley and S/S2) are located in close proximity to the RCS Treatment Facility (TF) Alternative 1 (Alternative 5C) and Alternative 2 (Alternative 5A). Based on input from IID staff, IID is planning to build a new 230 kV switching station one mile north of the Imperial Valley Substation which should have enough capacity to provide power to these facilities. IID's 161 kV L transmission line runs close to proposed TF Alternative 3 (Alternative 3A) and proposed Pump Station (PS) 1. IID's 92 kV R transmission line runs about 2.3 miles south of proposed PS 2. The nearest facility to the proposed PS 3 is SDG&E's Narrow Substation, which is located southeast of proposed site of PS 3. There are few distribution lines in the surrounding areas. IID owned distribution lines are between 12.7 kV-13.2 kV with a maximum capacity of 5-6 MW, insufficient capacity to serve loads required by the proposed pump stations and TF.

5.4.2 Proposed Facilities

A dedicated transmission line (161 kV/92 kV voltage) and associated substation would be needed to provide power to each of the three proposed pump stations and TF.

The TF Alternatives 1 or 2, would be serviced from the proposed new 230 kV Substation of IID, while the TF Alternative 3 would be serviced by a loop-in-/loop-out transmission line from IID's existing 161 kV Transmission Line. TF Alternative 4, however would be located within SDG&E's service territory and a tap line from SDG&E's 230 kV transmission line and a new step-down substation would be required for this service.

Separate transmission lines (161 kV/92 kV) are assumed to be required to service each of the three pump stations. PS 1 would be serviced by the same loop-in/loop-out transmission line to service proposed TF for Alternative 3. PS 2 would be serviced by a tap line from IID's 92 kV R Line. PS 3 would require a new 92 kV transmission line to be built from SDG&E's

Narrow Substation. All three locations would require step-down Substations. Although electrical service providers could cross service area boundaries to provide service if they have nearby power infrastructure, since SDG&E owns the nearest substation, this study has assumed that SDG&E would provide power to PS 3 and the Tunnel Portal East.

Distribution lines would be required for connecting to the Tunnel Portal East, West and intermediate portals. The power for all the tunnel portals (with the exception of eastern-most tunnel portal) would need to be obtained from SDG&E since those areas are outside the IID service area. For purposes of this evaluation a distribution line length of three miles (or less) was assumed for the portals located within SDG&E's service area and is not shown on the figures until input can be obtained from SDG&E. Since the eastern-most tunnel portal would be in the SDG&E service territory that portal would be serviced by a distribution line from the existing SDG&E distribution line network. Table 5-2 provides a summary of the transmission and distribution line lengths and right-of-way widths required for Alternative 3A.

TABLE 5-2
Summary of Proposed Transmission and Distribution Lines

Alignment	Transmission Lines			Distribution Line
	230kV	161kV	92kV	12.7KV
3A	7.4 miles[(1)]	2.4 miles	12.5 miles	12.6 miles
Right of Way (Width)	100 feet	100 feet	100 feet	60 feet

Note:

1. Length determined based on selection of longest TF Alternative transmission line.

Further detail for the proposed transmission and distribution lines are provided per facility on Table 5-3. The existing and proposed transmission and distribution lines and substations are shown on Figures 5-2, 5-3, and 5-4.

TABLE 5-3
Breakdown of Proposed Transmission and Distribution Lines

Equipment/Facility Need Power	Power Source	Trans Line	Distrib Line	Notes
TF Alt 1 (Alternative 5C)	Proposed 230 kV Switch Station North of Imperial Valley S/S	7.4	-	230kV T-Line needed
TF Alt 2 (Alternative 5A)	Proposed 230 kV Switch Station North of Imperial Valley S/S	2.7	-	230kV T-Line needed
TF Alt 3 (Alternative 3A)	Loop-in/loop-out from IID 161kV T-Line	0	-	Same location as Pump Station 1
TF Alt 4 (Alternative 3A)	Tap from SDG&E 230 kV T-Line	5.0	-	North of Existing Twin Oaks WTP
PS 1	Loop-in/loop-out from IID Line 161 kV L Line	2.4	-	27 Miles from Imperial Valley Substation 11 Miles from Solar Substation

TABLE 5-3
Breakdown of Proposed Transmission and Distribution Lines

Equipment/Facility Need Power	Power Source	Trans Line	Distrib Line	Notes
PS 2	Tap with disconnect switches from IID 92 kV R Line	4.8	-	PS 2 North of IID 92kV R Line
PS 3	Narrows S/S: 92 kV T-Line	7.7	-	7.7 Miles from Narrows SS
T1 East Portal	Distribution Line from New Substation Near PS3	-	3.0	1.3 Miles from PS 3
T1/T2 Intermediate Portal	Tap from Nearby Distribution	-	3.0	19 Miles from PS 3
T2 West Portal	Tap from Distribution near T3 East Portal	-	3.0	-
T3 East Portal	Tap from Distribution near T3 East Portal	-	0.6	Close to T3 East Portal
T3 West Portal	Tap from Nearby Distribution	-	3.00	-

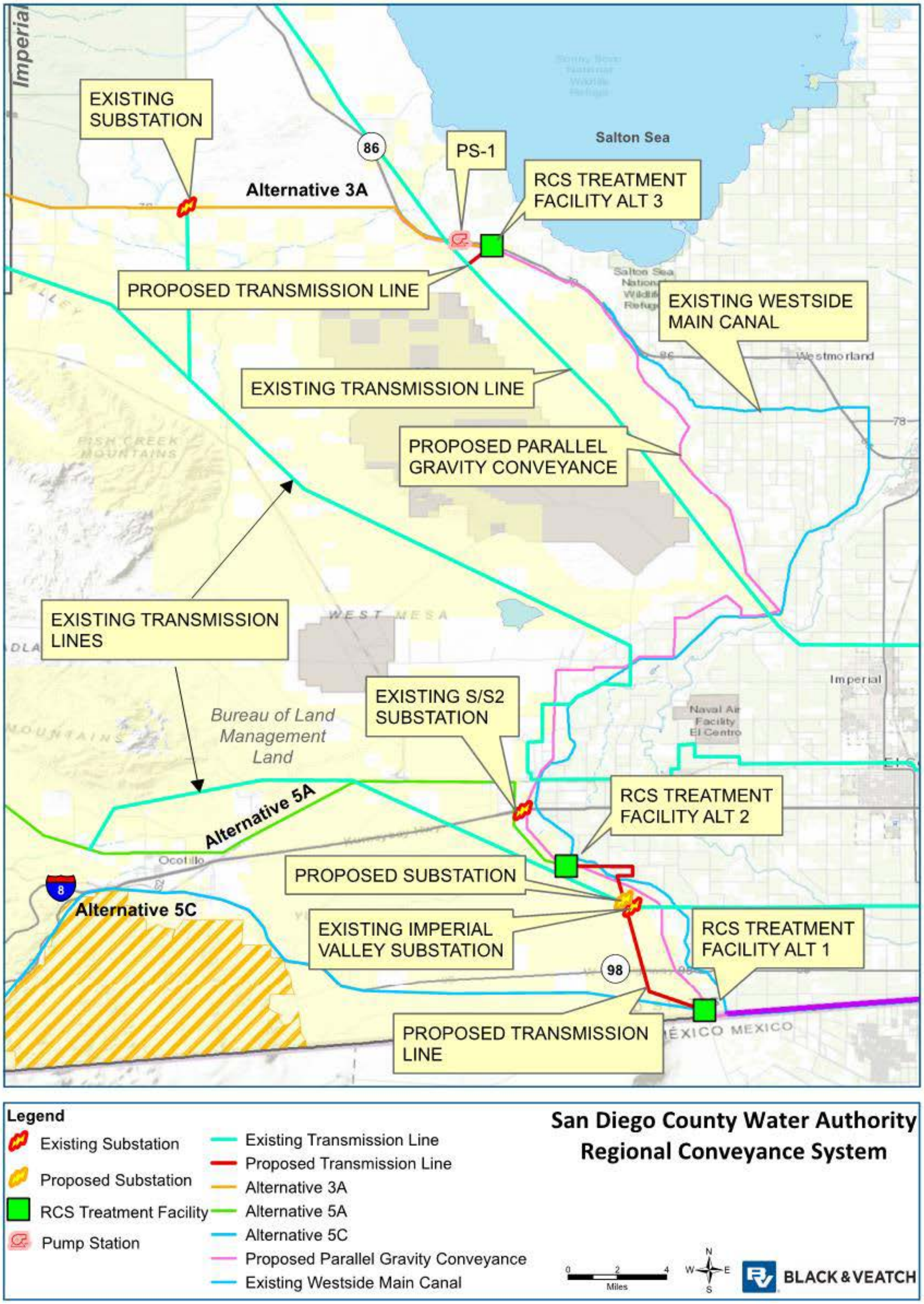

FIGURE 5-2
Proposed and Existing Transmission Facilities

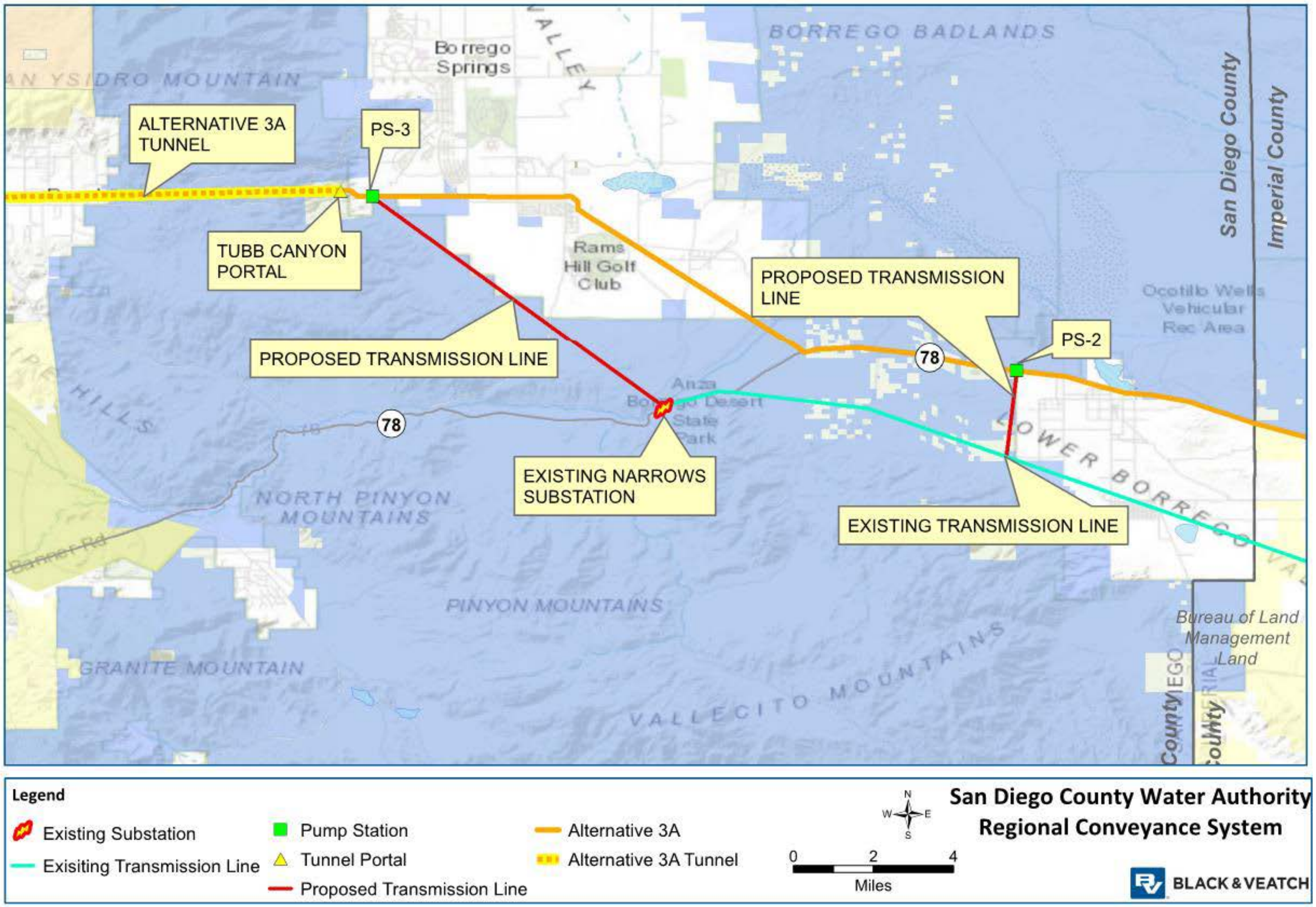

FIGURE 5-3
Proposed and Existing Transmission Facilities

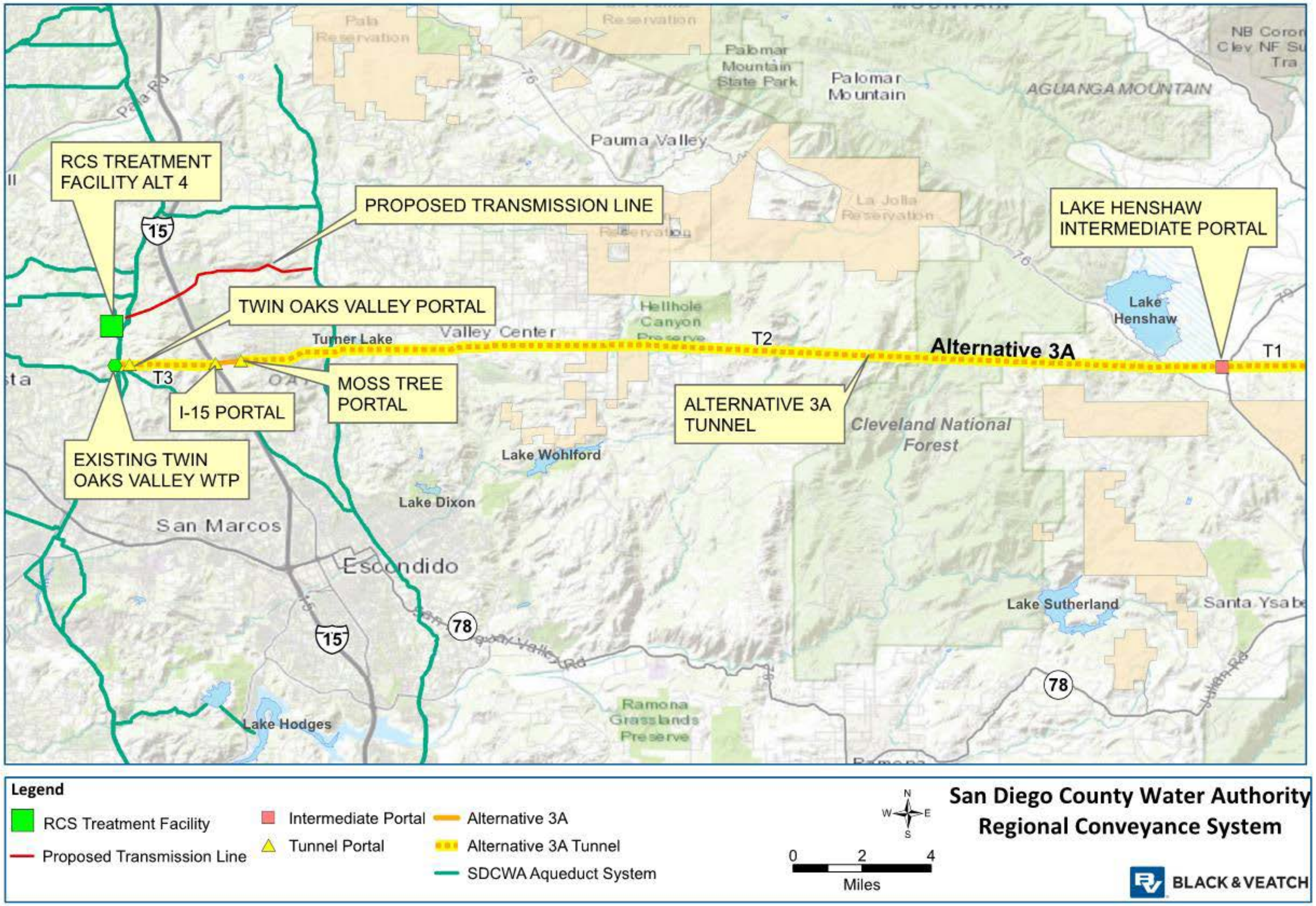

FIGURE 5-4
Proposed and Existing Transmission Facilities

The following design criteria apply to the proposed transmission lines of various voltage levels:

230 kV

- Single circuit transmission line, supported on galvanized single shaft tubular steel poles. Typical pole/structure details are provided in the Appendix E.
- Braced post insulators for tangent structures
- Suspension insulator string for angle and dead-end structures
- Single 795 kcmil aluminum conductor steel reinforced (ACSR) "Drake" conductor per phase
- Single 3/8 inch extra high strength (EHS) shield-wire
- 850 ft max wind span

161 kV / 92 kV

- Single circuit transmission line, supported on single wood poles (self-supporting for tangent, guyed for angle / dead-end structures). Typical pole/structure details are provided in the Appendix E.
- Post and /or braced post insulators for tangent structures
- Suspension insulator string for angle and dead-end structures
- Single 477 kcmil ACSR "Hawk" conductor per phase
- Single 3/8 inch EHS shield-wire
- 600 ft max wind span

12.7 kV

- Single circuit transmission line, supported on single wood poles (self-supporting for tangent, guyed for angle / dead-end structures). Typical pole/structure details are provided in the Appendix E.
- Pin insulators for tangent and angle structures
- Suspension insulator string for dead-end structures
- Single 4/0 ACSR "Penguin" conductor per phase
- Single 1/0 ACSR "Raven" conductor for Neutral
- 400 ft max wind span

A typical 230/12kV Substation layout has been developed to get a sense of the potential footprint and equipment that would be required to serve the respective facilities. The 230kV high side voltage was chosen due to the proximity of proposed TF Alternatives 1 and 2 to IID's proposed 230kV substation. With 230kV being the highest potential service voltage in the system, using this for the typical station layout is conservative in overall footprint and equipment costs. If an alternative TF is chosen with a lower service voltage (161kV or lower), then the substation could be adjusted accordingly. The layout could also be adjusted depending on the power needs of the facility it is servicing.

The current 230/12kV layout cuts the new 230kV IID Transmission Line to create a line in line out scenario with two positions dedicated to 230kV to 12kV Power Transformers. The total arrangement would be a four position ring bus configuration populated with the

Power Transformers, 12kV Switchgears, Disconnect Switches, Capacitor Voltage Transformers (CCVTs), Current Transformer/Potential Transformer (CT/PT) Metering Units, Circuit Breakers and the associated supporting steel and foundations. See Figure 5-5 for the typical substation layout.

5.5 Rate Structures

This section presents the rate structures used to calculate the energy costs for the proposed facilities. The rate structures are based on specific time-of-use (TOU) periods and vary by regional utility. The proposed facilities for the three alternatives are located within the IID and SDG&E service areas. As discussed in Section 5.4, IID owns and operates the substations and transmission lines within Imperial County and is assumed to be the energy service provider for the TF, PS-1, and PS-2 for Alternative 3A. PS-3 is located within SDG&E's service area and the nearest substation is SDG&E's Narrows Substation. Therefore, SDG&E would be the energy service provider for PS-3 and the eastern most tunnel portal. This was confirmed with IID.

The TF and PS-1 for both Alternatives 5A and 5C would be provided power by IID. The remaining pumping stations for Alternatives 5A and 5C would be provided power by SDG&E due to their proximity to SDG&E's transmission lines within Imperial County and since SDG&E owns the Imperial Valley Substation. This was verified with IID.

The pump station required for Alternatives 5A and 5C to pump water from San Vicente Reservoir to the Twin Oaks Valley Water Treatment Plant described in *Chapter 3.0 – Aqueduct Operations, and Integration of the RCS* would be within the SDG&E service area and would utilize SDG&E rates.

Power would also be required at the temporary facilities associated with constructing the RCS project, such as at tunnel portals and the intermediate shaft/vent location. These locations would require infrastructure to deliver them power but would not continue to use power after construction is finished. As such, power demands for temporary facilities were not included in the calculations for annual energy costs.

The TOU rate structures described below are used to calculate annual energy costs for the project, as documented in *Chapter 6.0 - Risk, Cost Opinions, and Economic Comparison*.

As mentioned previously, the potential treatment plant location at the TOVWTP in SDG&E's service area was deemed infeasible in *Chapter 4.0 – Treatment, Blending, and Brine Management Options* and was removed from further evaluation. As such, rate structures were not identified for a treatment plant in SDG&E's service area.

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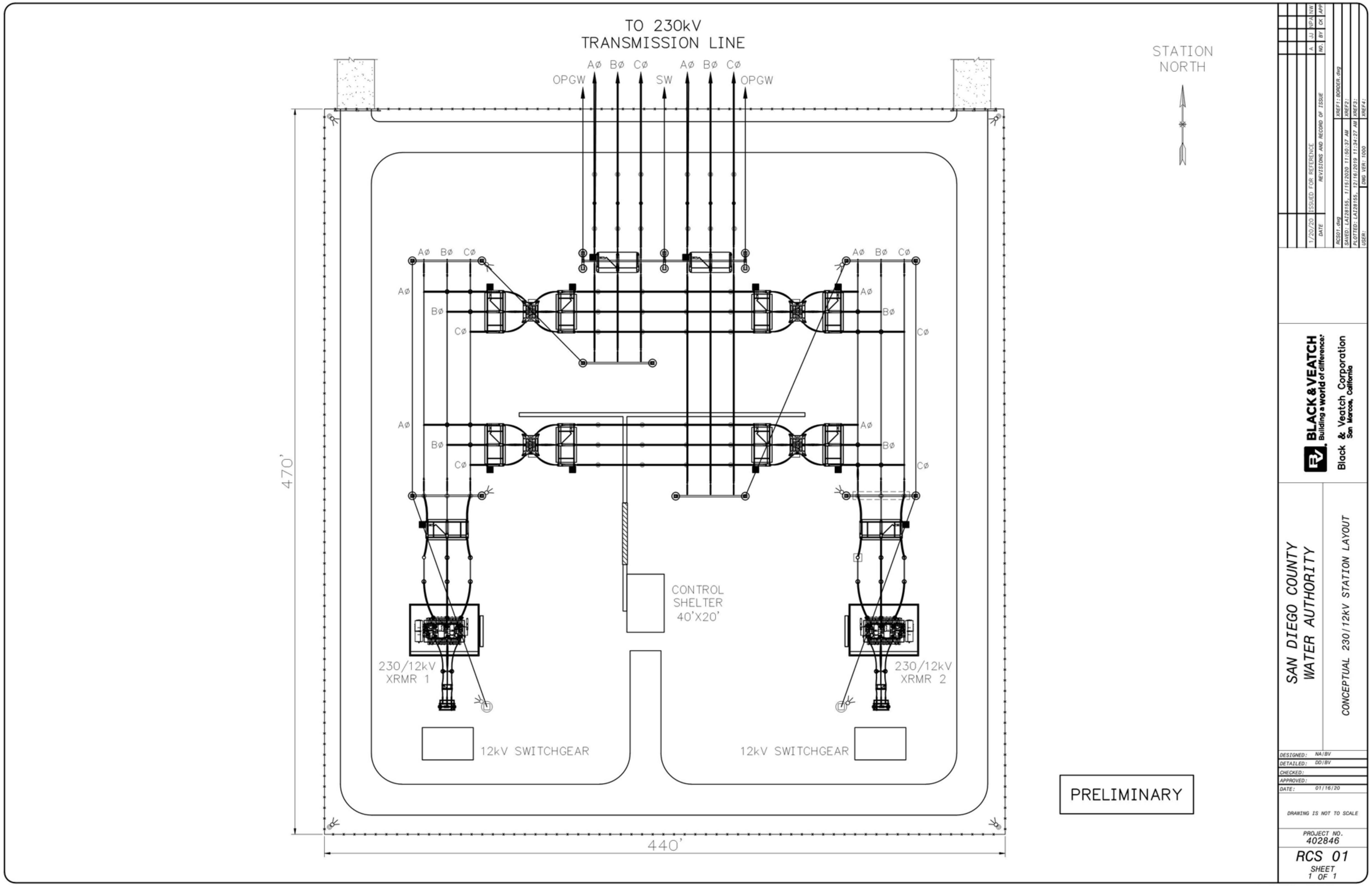

FIGURE 5-5
Typical Substation Layout

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5.5.1 Imperial Irrigation District (IID)

IID's service area is in the southeast portion of California and borders SDG&E's service area to the west and the California-Mexico border to the south. The three alternatives are proposed to begin at canal facilities located within IID's service area. IID's municipal rate structure is used to calculate the energy costs associated with pump stations and treatment plants served by IID.

IID's municipal rates as of December 2019 are presented in Table 5-4.

TABLE 5-4
IID Rate Structure – Municipal Demands (Updated December 2019)

Item	Rate
Basic Charges (Monthly)	
Charge per Meter ($/Month)	12.00
Public Benefit Charge	2.85%
California Energy Surcharge	0.00029
Utility Users/ Tax	N/A
Energy Charges	
Energy Charge ($/kWh)	0.1141
Energy Cost Adjustment Factors ($/kWh)	
Energy Cost Adjustment Billing Factor (January 2020)	-0.0175
Energy Cost Adjustment Renewables Factor (January 2020)	0.0181

Note:

1. IID's Rate Schedule for Municipal service can be found at the following website: (https://www.iid.com/energy/rates-regulations/rates)

IID's Energy Cost Adjustment (ECA) Billing Factor covers the fluctuating cost of power purchases, fuel, and transmission costs associated with generating and conveying power. Similarly, the Energy Cost Adjustment Renewables (ECA-R) Factor is applied to the basic energy charge to account for maintaining a renewable energy portfolio. The ECA and ECA-R Factors can be calculated monthly and are adjusted based on the district's historical power usage. The ECA and ECA-R Factors presented are the January 2020 rates and were provided by IID as suitable numbers to use for budgeting purposes.

It is understood that the ECA Factor is calculated based on seven variables and the ECA-R Factor is calculated based on two variables. However, when contacted, IID's Transmission Planning and Energy Department only provided the ECA and ECA-R Factors presented above to use for budgeting purposes and did not provide the variables by which they were calculated. Further coordination is recommended with IID during potential future phases of work to verify energy rate predictions.

5.5.2 San Diego Gas & Electric (SDG&E)

SDG&E's service area is bordered by IID's service area on the east and Pacific Ocean on the west. SDG&E's energy rate structures are specific to the application and time of use. A different rate structure is used to calculate the energy costs required for pump station as would be used to calculate for a treatment plant within SDG&E's service area. The sections below present SDG&E's TOU periods and the associated rate schedules.

Time-of-Use Periods

SDG&E's energy rates vary according to TOU rate structures, which adjust the rates based on peak and low energy usage periods of the day. The TOU periods include on-peak, off-peak, and super-off-peak periods during the summer and winter seasons. Table 5-5 presents the TOU periods as defined by SDG&E.

TABLE 5-5
SDG&E Time-of-Use Periods (Updated 2019)

Item	Summer (June 1 to October 31)	Winter (November 1 – May 31)
TOU Period - Weekdays		
On-Peak	4:00 p.m. to 9:00 p.m.	4:00 p.m. to 9:00 p.m.
Off-Peak	6:00 a.m. to 4:00 p.m. 9:00 p.m. to midnight	6:00 a.m. to 4:00 p.m. Excluding 10:00 a.m. to 2:00 p.m. in March and April; 9:00 p.m. to midnight
Super-Off-Peak	Midnight to 6:00 a.m.	Midnight to 6:00 a.m. 10:00 a.m. to 2:00 p.m. in March and April
TOU Period – Weekends and Holidays		
On-Peak	4:00 p.m. to 9:00 p.m.	4:00 p.m. to 9:00 p.m.
Off-Peak	2:00 p.m. to 4:00 p.m. 9:00 p.m. to midnight	2:00 p.m. to 4:00 p.m. 9:00 p.m. to midnight
Super-Off-Peak	Midnight to 2:00 p.m.	Midnight to 2:00 p.m.

Eight holidays are observed throughout the year. Table 5-6 presents the annual breakdown of hours by use period.

TABLE 5-6
Annual Hours per SDG&E Time of Use Period

Use Period	Summer	Winter	Total
On-Peak, Hours	765	1,060	1,825
Off-Peak, Hours	1,629	2,048	3,677
Super-Off-Peak, Hours	1,275	1,980	3,258
Total	3,672	5,088	8,760

Rate Schedules

SDG&E applies different rate structures depending on the facility using the energy and the quantity of energy used. Each rate structure includes a basic energy rate and a commodity rate to cover costs related to distribution and transmission. These rate structures are used to calculate the annual energy costs for each facility.

For the pump stations served by SDG&E, this study has assumed the use of the publically available rate schedules that correspond to the rate structures used by the Water Authority's

San Vicente Pump Station, which are the TOU-PA-3 ≥ 20 energy rate schedule with the EECC-CPP-D-AG schedule for commodity rates.

For the purposes of this evaluation, SDG&E's primary rates have been used. SDG&E's Electric Rates Rule 1 defines primary rates as applying to service that is taken at or above 2.00 kV but below 25.00 kV provided that the service is taken from regularly available service voltages. This study assumes a 230/12 kV stepdown substation owned by the utility provider would be provided at each facility and that service would be received at 12 kV. Each facility would then step down the power on site to whatever voltage was required. Further coordination with SDG&E would be required during potential subsequent phases of design to verify these assumptions.

Table 5-7 presents SDG&E's TOU-PA-3 ≥ 20 kW rate structure with the EECC-CPP-D-AG schedule for commodity rates.

TABLE 5-7
SDG&E TOU-PA-3 ≥ 20 kW Rate Structure (Effective June 2019) – Pump Stations

Item	Summer	Winter
Demand Charges		
Basic Service >200 kW ($/month)	126.24	-
Non-Coincident ($/kW each month)	-	-
On-Peak ($/kW each month)	-	-
Energy Charges ($/kWh)		
On-Peak (Primary)	0.09441	0.09441
Off-Peak (Primary)	0.09441	0.09441
Super Off-Peak (Primary)	0.09441	0.09441
Energy Charges – Commodity ($/kWh) – See Note 2		
On-Peak (Primary)	0.13042	0.07297
Off-Peak (Primary)	0.10485	0.06479
Super Off-Peak (Primary)	0.04178	0.05573

Notes:

1. SDG&E's current and effective tariffs are found at the following website: (https://www.sdge.com/rates-and-regulations/current-and-effective-tariffs)
2. Commodity rates are based on SDG&E's EECC-CPP-D-AG schedule effective June 2019.
3. Energy charges are based on the use of SDG&E's primary rates.

5.6 Renewable Energy Opportunities

This section describes renewable energy opportunities (i.e. hydro and pumped storage) as a function of the Alternative 3A corridor. Note that Phase A baseline cost estimates assume no partnerships and associated costs or benefits/funding opportunities of the renewable energy opportunities outlined in this section. Additional renewable energy opportunities including solar and wind are described in *Chapter 8 – Partnership and Funding Opportunities.*

AAC. During discussion with IID it was noted that with the potential increased flow rates for delivery of the RCS water through the AAC, existing hydroelectric facilities along the canal, which are currently owned and operated by IID, could see an increase in renewable energy productions. Since the AAC is currently operating near full capacity, this increase would likely be small. However, it is still recommended that this potential increase in generation be further evaluated under potential future phases of this work and coordinated with IID.

Lake Henshaw Tunnel Portal. Alternative 3A is the only alternative where a potential energy storage facility was identified. For Alternative 3A, a pumped storage facility connected to Lake Henshaw could be further examined. This facility would be constructed in a cavern approximately 1,500 feet below Lake Henshaw and would make use of the vent shaft that Would be required for construction of the tunnel section below the lake. New pumps/turbines would be constructed to send water up to Lake Henshaw when energy needs to be stored. When energy is needed, water would be withdrawn from Lake Henshaw to generate energy. The water would be pumped from the tunnel pipeline up to Lake Henshaw and then return to the tunnel pipeline after generating energy for delivery to San Diego county. These types of pumped storage facilities are less common than the type used at Lake Hodges and envisioned for San Vicente Reservoir but are used throughout the world. A pumped storage facility of this type appears to be technically feasible based on the information collected at the time of this study, but economic viability would need to be confirmed. Any future evaluation would be coordinated with the Vista Irrigation District in Phase B, if authorized.

Lake Wohlford. As discussed in *Chapter 2.0 – RCS Operations and Sizing*, Lake Wohlford may be utilized as terminal storage for Alternative 3A. Due to the higher elevation of Lake Wohlford at 1,480 feet, the Alternative 3A would be required to pump to this higher head in order to store the water. During the times when water is discharged from Lake Wohlford into the RCS system, excess head would be available for potential hydroelectric generation. Since the discharge of water from Lake Wohlford would be very infrequent, a detailed analysis would be required to determine if the cost of the energy recovery facility would be acceptable, in lieu of just burning the excess head via a pressure and flow control facility. This analysis should be further evaluated and coordinated with the City of Escondido. in Phase B, if authorized.

5.7 Conclusion

This section presents the conclusions and observations from the assessment of power supply alternatives for Alternative 3A and includes key takeaways, as well as items to consider during upcoming phases of work. The components described in this chapter form the basis for the power components of the conceptual cost opinions prepared and described in *Chapter 6.0 - Risk, Cost Opinions, and Economic Comparison*.

Power Needs. The preliminary power needs for the various proposed facilities were developed based on conservative assumptions and sizing (i.e. constant speed pumps). As the project progresses, further refinement would be made to the proposed facilities and associated power needs. At this conceptual stage of the project, the project scope included a common pump station layout that would be typical for all proposed pumping stations based on a consistent rated head and capacity at each. The pumping stations and intermediate pumps at the TF could be refined to consider Variable Frequency Drives. As well the pumping station locations could be refined based on the final preferred alternative to balance the pumping station costs with the costs of electrical infrastructure. This could be completed under the next phase of the project.

Regional Transmission Facilities. Electric transmission lines and substations would be necessary to transmit power from existing electric utility facilities to the new project facilities. Black & Veatch developed preliminary power supply alternatives based on information provided by IID and readily available information online for SDG&E. The final transmission facilities and corridors would be determined based on IID and SDG&E preferences and/or limitations of capacity and could be refined under the next phase of the project. Installation methods including the alternative of installing the electrical transmission/distribution lines within the pipe trench could be evaluated in the next phase of the project. Alternative power supply deliveries were not evaluated as part of this phase of the project but were previously evaluated as part of the *2013 Master Plan.* Natural gas powered turbines and associated natural gas deliveries could be evaluated in the next phase of the project as well, if authorized.

Rate Schedules. Historically, predicting changes in rate schedules from utility providers has been extremely difficult. As such, this report provided energy costs based on current, publicly available rate schedules and from limited information from the utility providers. During potential future project phases, it will be critical that the Water Authority monitors changes to the published rate schedules to assess their impact on the project. In future potential phases of work, additional coordination with the utility service providers is required to further verify the rates used. Future coordination could include negotiation of contractual rates during the preliminary and final design phases.

Renewable Energy Opportunities. Renewable energy opportunities (i.e. hydro and pumped storage) were evaluated as a function of the Alternative 3A corridor. An increase in renewable energy production along the existing IID AAC hydroelectric facilities could be realized with the potential increased flow for RCS water delivery. While this increase would likely be small as the canal currently runs near or at production, details for this potential generation should be further evaluated under potential future phases of this work. For Alternative 3A, a pumped storage facility connected to Lake Henshaw could be further examined, whether located in the tunnel vent shaft or utilizing PS3 to pump up to Lake Henshaw with turbines installed in the shaft. Pumped storage alternatives at both Lake Henshaw and Lake Wohlford could be further evaluated in Phase B, if authorized.

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6

RISK, COST OPINIONS AND ECONOMIC COMPARISON

Chapter 6.0 Risk, Cost Opinions and Economic Comparison

6.1 Chapter Overview

This chapter presents the project risks that were identified, the cost opinions that were developed, and an economic comparison of the alternatives to the status quo (transportation of QSA water MWD's conveyance facilities). The project risks are focused on high level risks that would have significant likelihood of occurrence and/or significant impacts if they occur. For a project of this scale, a fully developed risk register would likely include hundreds or thousands of identified risks and mitigation strategies. At this level of analysis, an abbreviated list of risks that would have significant impact on the project was developed.

A cost opinion for the RCS was generated with enough detail to capture the major facilities and provide reasonable estimates for use in the economic comparison. If the study proceeds to future phases, it is anticipated that the cost components would continue to be developed to provide a higher confidence level in their accuracy, as is typical in the planning of large-scale infrastructure projects. The project's costs were then applied to a financial model that included such items as annual operations and maintenance costs, financing costs, and other project related costs to enable a financial comparison. Note that Phase A baseline cost estimates assume no partnerships and associated benefits/funding due to the preliminary nature of this review and limited dialogue with potential partners.

6.2 Chapter Organization

The steps completed to identify project risks, develop cost opinions, and complete an economic comparison are summarized below and discussed in the following sections:

Project Risk Overview – This section presents an overview of how risks were identified for the RCS at this level of study. This section also describes the risk register prepared for the study.

Major Project Risks – This section presents the risks categories that were determined to impact the overall feasibility of the RCS. The major project risks described group individual risks identified in the risk register into overarching categories.

Comparison of Major Risks for Alternatives 3A, 5A, and 5C – This section presents a comparison of each alternative with the major project risk categories. The comparison includes the overall feasibility of each alternative, the uncertainty associated with the cost estimating at this planning level, and the operational risks.

Project Costs Overview – This section provides an overview of how the cost opinions for the capital and annual costs were developed. It also presents a summary of the final cost opinions for each of the major components. Capital and annual costs were estimated for the following:

- Canals
- Tunnels
- Pump Stations
- Treatment Facility (Salinity)
- Operational Storage
- Additional Staff at Kearny Mesa
- Soft Costs
- Pipelines
- Power Generating / Pressure Control Facilities
- Electric Distribution
- Environmental Mitigation and Permitting
- Office and Warehouses
- Energy Usage

Comparison with Prior Studies – This section presents a comparison of the costs developed during this study with the costs developed during previous studies. It also provides descriptions of factors that led to any differences in the cost opinions.

Economic Comparison – This section presents the findings from the economic model that was developed by the Water Authority.

6.3 Project Risk Overview

Considering the project is in the early planning stages, the risks focus on identifying issues on the overall feasibility of the alternative, the ability to develop reliable cost estimates which are necessary to evaluate alternatives, and operational risks. The risks present either a design, construction, agency coordinating, operating, permitting, public affair, or right of way challenge or fatal flaw.

The Risk Register, provided in Appendix F, lists and compares risks for the overall RCS and specific risks related to Alternatives 3A, 5A, and 5C.

A number of high level risks were identified and ranked in the risk register. The ranking was performed considering a combination of the probability of occurrence and likely impact of the risk. The ranking was reviewed and the most impactful were grouped into nine categories, known as the “major project risks”. The nine major project risks are described in Section 6.4.

It should be noted that this evaluation identified no risks that were deemed to be a fatal flaw for any of the three alternatives.

While not ranked high enough in the risk register to be defined as major project risks, the following key risks are also highlighted as important risks that should be addressed early due to their potential long lead time.

Renegotiation of QSA Agreements Past 2077. The 2003 QSA was executed concurrently with various interconnected agreements, one of which is the IID Water Transfer Agreement. The IID Water Transfer Agreement allows for an extension of 30 years, from 2047 to 2077, by mutual consent of the parties. Project risks include the inability to negotiate the IID Water

Transfer Agreement and other necessary related agreements with the various parties for the extension of the IID Water Transfer past 2077.

IID Water Transfer Agreement Price Uncertainty Post 2034. The cost per AF for the transfer water is based on the annual increase in the Gross Domestic Product Implicit Price Deflator as published by the Bureau of Economic Analysis of the United States Department of Commerce from now through 2034. Beginning in 2035, either the Water Authority or IID could, if certain criteria are met, elect a market price through a formula described in the water transfer agreement. The risk to the project includes uncertainty in the future cost of the water supply beginning in 2035.

Early coordination with stakeholders so that the agreements are finalized well before the start of construction could help mitigate both risks.

6.4 Major Project Risks

This section documents the major project risks grouped into three overarching categories: 1) key overall project risks, 2) risks due to cost uncertainty, and 3) risks to the operation of the system.

Key Overall Project Risks. Project risks for overall feasibility include the cost of future MWD supplies, inability to obtain certification for environmental permitting, and inability to obtain permits for brine management into the Salton Sea. These are described in more detail below in order of Risk ID shown on the Risk Register in Appendix F.

- MWD Supplies (Risk ID 4) – Project risks include cost uncertainty associated with cost of water from MWD for future water. In addition to maintaining engagement with MWD, establishing a model at the appropriate phase of planning and including appropriate contingency could help account for the risk.
- Certification for Environmental Permitting (Risk ID 7) – Construction of facilities for RCS could impact state parks, tribal lands, wilderness areas, and a national forest which could include an impact on sensitive habitats and/or species. Project risks include the inability to obtain certification of environmental documentation. Thorough planning and investigations begun as early in the process as possible to identify any specific concerns could minimize the severity of risk.

Major Cost Estimating Risks. Major risks associated with the ability to develop reliable cost estimates include the cost uncertainty associated with the agreement for treatment plant and pump station power, and the cost growth of tunneling due to unforeseen conditions. These are described in more detail below in order of Risk ID shown on the Risk Register in Appendix F.

- Tunneling (Risk IDs 1 through 3) – A preliminary geotechnical investigation was conducted for Alternatives 5A and 5C in 2001 and a geotechnical desktop study of the 3A Alternative was conducted as part of this study. Typically, geologic conditions would be further defined as the project proceeds to new phases and would include general mapping and detailed investigation at specific points along the alignment. However, the geology is estimated through interpolation between data points and could be different from what is predicted at the design phase. This is particularly critical for tunnel segments with geologic conditions defining the type of TBM, tunneling methods, and lining methods used. Field geological investigations during preliminary design and design phases would help to address this risk. While additional geological investigations improve the resolution of subsurface conditions, they are very expensive and must be balanced with risk. Including appropriate contingency at various project phases could minimize the severity of risk associated with this process.

- Treatment Plant Power Agreement (Risk ID 5) – There is a significant power demand to operate the treatment plant for RCS. Project risks include the cost uncertainty for this large-scale power. The baseline assumption is that power would be provided by one of the existing commercial energy providers, such as SDG&E and/or IID. Other potential energy providers could be investigated, such as private developers that lead to competition and lower energy rates. Establishing an agreement framework at the appropriate phase of planning and including appropriate contingency could minimize the severity of risk.

Key Operations Risks. Key challenges associated with the operations include possible terrorist activity and earthquakes for existing and new facilities. These are described in more detail below in order of Risk ID shown on the Risk Register in Appendix F.

- Terrorism on Canals (Risk ID 6) – The canals would supply RCS with water from the Colorado River. Project risks occur with terrorist activity on the open water system. Developing an early warning monitoring plan and mitigating through a storage system could minimize the severity of risk.
- Earthquakes on New Facilities (Risk IDs 8 through 10) – Project risks include earthquakes along the San Jacinto Fault Zone, Superstition Hills Fault Zone, Elmore Ranch Fault Zone, and Elsinore Fault Zone that could result in the new facilities' inability to deliver QSA supplies. Mitigating through appropriate storage could minimize the severity of the risk. Further, MWD's existing facilities are geographically distant from the proposed RCS facilities and offer redundancy to the system.

6.5 Comparison of Major Risks for Alternatives 3A, 5A, and 5C

Risks for the RCS are shown in the Risk Register in Appendix F. A comparison of the major risks along each alignment are included in Table 6-1, which identifies the risk ID from the Risk Register, the risk factor, and degree of severity for Alternatives 3A, 5A, and 5C. The major risks are grouped into the three overarching categories: 1) overall feasibility, 2) cost uncertainty, and 3) operations.

Table 6-1 rates the three alternatives based on each risk factor using a least, moderate, and most rating system to assess the relative risk of each alternative to that factor. A rating of "least" means that the alternative has the lowest risk for that category. Conversely, a rating of "most" means the alternative has the highest risk for that category. When all three alternatives were deemed to have the same risk based on this level of study, a rating of "equal" was used. Similarly, if two alternatives both had similar levels of risk while the third alternative had a lower risk, then the prefix "equally" was added to the rating. In this scenario, the alternative would receive a rating of "equally-most."

The ratings shown on Table 6-1 are intended to provide a relative comparison between the three alternatives. However, for the risk factor of Earthquakes on New Facilities, the risk of an earthquake for all three alternatives was high as they all cross numerous fault zones. For that factor, the ratings included a prefix of "high" followed by their relative comparison to each other. For example, Alternative 3A has the highest relative risk of the three alternatives, so it would receive a rating of "High-Most."

The sub-sections following Table 6-1 describe the three categories in more detail.

TABLE 6-1
Comparison of Major Risks for Alternatives 3A, 5A, and 5C

ID	Risk Factor	Alternative 3A	Alternative 5A	Alternative 5C
		Overall Feasibility		
4	MWD Supplies	Equal	Equal	Equal
7	Certification for Environmental Permitting	Least	Moderate	Most
		Cost Uncertainty		
1 through 3	Tunneling	Equally-Most	Equally-Most	Least
5	Treatment Plant Power Agreement	Equal	Equal	Equal
		Operations		
6	Terrorism on Canal Systems	Equal	Equal	Equal
8 through 10	Earthquakes on New Facilities	High-Most	High-Moderate	High-Least

6.5.1 Overall Feasibility

The risk of uncertainty in the cost of future MWD supplies are equal for all three alternatives.

Risk ID 7 - Since Alternative 3A crosses Anza-Borrego Desert State Park, potentially crosses a wilderness area but via a tunnel, crosses Borrego Springs Community, and crosses Cleveland National Forest, 3A is the least risky in comparison to the other alternatives in obtaining certification for environmental permitting. Alternative 5A crosses Anza-Borrego Desert State Park, Cuyapape Indian Reserve, and Cleveland National Forest making Alternative 5A a moderate risk in comparison to 3A and 5C in obtaining certification for Environmental permitting. Alternative 5C crosses Yuha Basin Area of Critical Environmental Concern (ACEC) which is home to threatened species, crosses through Campo Indian Reserve, and crosses Cleveland National Forest making corridor 5C the riskiest in comparison to 3A and 5A in obtaining certification for Environmental permitting.

Since each alternative would produce brine as a byproduct from the treatment process, the risk for brine management permitting into the Salton Sea would be equal for all three (3) alternatives.

6.5.2 Cost Uncertainty

Risk for tunneling cost uncertainty is directly correlated to the amount of tunneling involved with each alternative. The longer the tunneling segment, the greater the risk. Since 3A and 5A propose a similar amount of tunneling miles, they both have an equal risk. Alternative 5C has the least amount of tunneling and thus is the least risky among the other alternatives.

Since each alternative would require a similar amount of power to operate the treatment process, the treatment plant power agreement has an equal risk for all three alternatives.

6.5.3 Operations

Since existing and new canal systems would be utilized to convey water to each alternative from the Colorado River, each alternative would share the same risk if the canal system is affected by either terrorist activity or an earthquake.

Risk for earthquakes on new facilities, such as pipelines, tunnels, storage reservoirs, water treatment plant, and pump stations is directly correlated to the alternative's proximity to fault zones. Since 3A crosses and parallels the most fault zones: San Jacinto Fault Zone, Superstition Hills Fault, Elmore Ranch Fault and Elsinore Fault Zone, therefore, it has the greatest risk in comparison to Alternatives 5A and 5C. Alternative 5A crosses and parallels Elsinore Fault zone while 5C only crosses the Elsinore Fault Zone thus making 5C the lowest risk in relation to the other alternatives.

6.6 Project Costs Overview

This section provides an overall summary of capital and operations, maintenance, and replacement (OMR) costs. Following the summary tables, discussion is provided regarding each major type of facility indicating the approach used to generate costs.

Capital costs include construction costs, which are comprised of direct and indirect construction costs for each component of the project. Indirect costs cover the contractor's general conditions, overhead, profit, building permits, insurance, and bonding. Capital costs also include soft costs comprised of pre-construction costs, such as the initial studies, engineering, right of way and property acquisition, CEQA/NEPA, public outreach, legal, environmental mitigation, owner's representative, and staff support, as well as construction management costs during the construction period. OMR costs are annual costs that occur after construction is complete and the project is operational.

Capital and OMR costs were developed for the three RCS alternatives based on information gathered from prior reports and/or new information developed as part of this study. The opinion of probable construction cost was developed by applying unit costs to the quantities estimated for each alternative. Unit costs developed in prior studies (the *2013 Master Plan*

Update and the *2017 Cost Update)* were escalated to 2020 dollars based on the Engineering News Record (ENR) Construction Cost Indexes for the Los Angeles Market. The ENR Construction Cost Index for the Los Angeles Market was used because it typically has a higher escalation factor than the Water Authority's cost escalation model and results in more conservative costs. New unit costs were developed for new items based on recent estimates for similar projects. Adjustments and revisions were made to the quantities for Alternatives 5A and 5C where necessary. Quantities for Alternative 3A are based on the descriptions presented in this report.

Table 6-2 presents the estimated capital cost for each alignment. In accordance with the purpose of this study, the costs provided define an estimated range equivalent to a Class 4 Estimate using the Water Authority's Cost Estimating Guidelines (2019). As such, the cost estimates have a range of +50 percent to -30 percent accuracy. Estimated annual operations, maintenance and replacement (OMR) costs are summarized in Table 6-3 for Alternatives 3A, 5A, and 5C. All three alternatives consider an annual transfer volume of 279,500 AFY of QSA supplies. Detailed opinions of probable construction costs are provided in Appendix G of this report.

TABLE 6-2
Estimated Capital Costs(1)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Canals	$ 59,200,000	$ 10,900,000	$ 1,600,000
Pipelines	$ 359,000,000	$ 428,200,000	$ 1,033,600,000
Tunnels	$ 1,512,800,000	$ 1,431,400,000	$ 450,000,000
Pump Stations	$ 155,700,000	$ 156,000,000	$ 321,700,000
PGFs/PCFs	$ 0	$ 31,097,000	$ 134,874,000
Electric Distribution	$ 49,200,000	$ 39,300,000	$ 52,100,000
Treatment Facility(2)	$ 625,500,000	$ 760,700,000	$ 783,300,000
Operational Storage (IID, RCS, and Day Tanks)	$ 193,250,000	$ 108,250,000	$ 108,250,000
Office and Warehouse	$ 8,860,000	$ 8,860,000	$ 8,860,000
SUBTOTAL CONSTRUCTION	**$ 2,963,500,000**	**$ 2,974,700,000**	**$ 2,894,300,000**
Construction Management Soft Costs	$ 664,200,000	$ 662,100,000	$ 665,500,000
Pre-Construction Soft Costs(3)	$ 464,370,000	$ 450,680,000	$ 463,060,000
Contingency (10-30%)(4)	$ 861,643,000	$ 874,632,000	$ 835,795,000
TOTAL (2020 Dollars)	**$ 4,953,723,000**	**$ 4,962,119,000**	**$ 4,858,640,000**

Notes:

1. Expected accuracy range for a Class 4 Estimate is from -30 percent to +50 percent of the estimate.
2. Salinity treatment costs include brine management.
3. Includes the initial studies, engineering, right of way and property acquisition, CEQA/NEPA, public outreach, legal, environmental mitigation, owner's representative, and staff support.
4. Contingency varies between 10% and 30% for each component of work based on available information, level of design, and risk. See Appendix G of this report for details.

TABLE 6-3
Estimated Annual OMR Costs(1)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Energy Cost – Pumping	$ 82,000,000	$86,200,000	$ 219,800,000
Energy Cost – Treatment	$ 13,080,000	$13,080,000	$ 13,080,000
OMR	$ 17,414,000	$ 13,599,000	$ 26,092,000
Salinity Treatment (Excluding Energy)	$ 30,600,000	$ 32,949,000	$ 32,970,000
Energy Recovery	---	---	($ 33,400,000)
TOTAL ANNUAL COSTS (2020 Dollars)	**$ 143,094,000**	**$ 148,777,000**	**$ 258,205,000**

Note:

1. Expected accuracy range for a Class 4 Estimate is from -30 percent to +50 percent of the estimate.

The costs presented in Table 6-3 are based on the following assumptions:

- The salinity treatment would occur in Imperial Valley.
- Salinity treatment would consist of MF and RO.
- The brine would be conveyed to the Salton Sea via a 30-inch diameter pipeline for use in constructed wetlands.
- Operational storage for Alternative 3A would be provided in the Valley Center or Lake Wohlford area depending on partnership opportunities.

6.7 Canals

The WSM does not have sufficient capacity for the additional flow for the RCS project. Therefore, a new parallel canal would be constructed adjacent to the existing WSM from the end of the AAC terminus, extending to each alternative's connection point.

The new parallel canal would be constructed as a concrete-lined canal with a trapezoidal shape. Design criteria for canal construction are presented in *Chapter 2.0 - RCS Operations and Sizing*. Unit costs were developed for the parallel canal and are presented in January 2020 dollars. Quantities and cost items reflect the construction of a parallel canal for the WSM. Annual costs for OMR were added considering costs equivalent to 1 percent of capital costs. Capital and annual costs for canals are summarized in Table G-2 of Appendix G.

In general, the cost opinions developed for this study carry a contingency of 30%, which is typical for this level of planning estimate. In an effort to validate the contingency applied, each major facility component was reviewed to determine if the contingency assigned was in line with: 1) the quantity of unknowns and 2) the level of detail of the design. The cost opinion for the canals was developed by applying semi-detailed unit costs to an estimated quantity take-off. This, coupled with the lower level of unknowns, resulted in a recommendation to apply a contingency of 10% to the canal facilities.

6.8 Pipelines

Pipelines account for all of Alternatives 3A, 5A, and 5C which are not comprised of tunnels. Unit costs were developed for pipeline construction based on the wall thickness of the pipe and the construction method being utilized. For budgeting purposes, it was assumed that all pressured pipelines would be cement lined and coated. An evaluation of lining and coating material selection is recommended during subsequent design phases.

Pipe wall thicknesses were developed based on design pressures from the hydraulic profiles and pipeline invert elevations. Pipe wall thicknesses were used to estimate the quantity of steel required for the project.

Pipeline construction methods included open cut, shored, and trenchless construction for highway or railroad crossings. Four types of open trench construction were utilized for this report: 1) sloped trench sides with native backfill, 2) sloped trench sides with process backfill imported for the pipe zone, 3) sloped trench sides with localized blasting and process backfill imported for use in the pipe zone, and 4) the use of a shored trench box with native backfill. Accessories, crossings, and specials were developed in the *2001 Feasibility Study Cost Refinement* and used in this report including appurtenances, highway crossings, railroad crossings, river crossings, surface/utilities, and the San Vicente Outfall Structure. Additionally, the potential turnout for Borrego Springs associated with Alternative 3A was added for this report.

The pipeline would be 102-inch-diameter steel pipe. Design criteria for pipelines are presented in *Chapter 2.0 – Regional Conveyance System Operations and Sizing*. Quantities and cost items reflect the pipeline segment lengths developed in this report.

Annual costs for the pipeline include a labor cost that accounts for the Water Authority's staff time to operate and maintain the pipeline. A general OMR cost is also included that accounts for the labor, equipment, and loss of water associated with reoccurring shutdowns for inspections and routine lining replacements. Major shutdowns for inspection of the pipeline interior are assumed to occur no more frequently than every five years. The general OMR cost also accounts for minor replacements, replacement of valves, and general upkeep of facilities along the pipeline. Labor costs are based on staffing estimates and current labor rates provided by the Water Authority. Table 6-4 presents the Water Authority's estimated staffing plan for the pipeline.

TABLE 6-4
Estimated Staffing Plan - Pipelines

Item	Crews	No. per Crew	Total Staff
Mechanical Maintenance	2	3	6
Maintenance - Supervisor	1	1	1
Operation			
Operator	1	8	8
Supervisor	1	1	1

TABLE 6-4
Estimated Staffing Plan - Pipelines

Item	Crews	No. per Crew	Total Staff
Operation (Escondido)	1	1	1
Right-of-Way Maintenance	2	3	6
SCADA	1	2	2
Corrosion	1	2	2

General OMR costs are equivalent to 0.5 percent of the total capital costs. Capital and annual costs for pipelines are summarized in Table G-3 of Appendix G.

6.9 Tunnels

Each alternative would require large diameter tunnels, while only 3A and 5A would have long tunnels to convey water to San Diego County. Detailed tunnel parameters, construction methods, and costs were developed in the *2001 Feasibility Study Cost Refinement* based on geologic evaluations from the geotechnical studies completed in 2001 for Alternatives 5A and 5C. This detailed basis for tunnel construction was used for the costs presented in this report. Further, new tunnel parameters, construction methods, and costs were developed for Alternative 3A.

Tunnel unit costs were developed based on the best available information in order to establish a conservative budget for planning purposes. This section summarizes the assumed design parameters for the tunnels as they impact the development of unit costs:

- Controlling groundwater inflows to minimize the impact on water resources would be required at all areas of discontinuities (fault and joint zones) and areas underlying National Forests or Indian Reservations. The following design conditions were assumed for mitigation in these areas:
  - Tunnel sections under National Forests or Indian reservations (for Alternative 3A) or crossing jointed/faulted zones (for Alternatives 3A and 5A) would have an impermeable steel pipe as the final liner system.
  - A more extensive pre-excavation grouting program to control groundwater inflow would be required at all jointed zones equal to 1.5 times the assumed length of the crossing and at all fault zones equal to 3.0 times the assumed length of the crossing.
- Steel lining would be required at portals and extend into the tunnel until the overburden pressure is enough to resist the internal hydraulic pressure. Due to the slopes of the mountainous region surrounding the tunnel portals, the steel liner required is assumed to be 500 feet at each portal.
- The final lining system for the remaining portion of the tunnel is assumed to be shotcrete with rock dowels where needed.

- The tunnel is assumed to be excavated with an open-face, hard rock TBM.

Table 6-5 summarizes the design parameter assumed for the four tunnel segments associated with Alternative 3A for the purposes of establishing a conservative cost.

TABLE 6-5
Design Parameter Assumptions for Alternative 3A Tunnels

Item	T1	T2 (E and W)	T3	T4
Start	Tubb Canyon Portal	Lake Henshaw Vertical Shaft	I-15 Portal	Lake Wohlford Vertical Shaft
End	Lake Henshaw Vertical Shaft	Moss Tree Portal	Twin Oaks Valley Portal	Lake Wohlford Portal
Excavated Diameter	14 feet	14 feet	14 feet	14 feet
Excavation Method	TBM	TBM	TBM	TBM
Length	17.4 miles	24.0 miles	2.1 miles	3.0 miles
Fault Zone Crossings	280 feet	300 feet	200 feet	140 feet
Joint Systems	---	3,000 feet	4,000 feet	---
Steel Liner at Portals	500 feet	500 feet	1,000 feet	500 feet
Pre-Excavation Grouting	840 feet	5,400 feet	6,600 feet	420 feet
Reservation and National Forests	---	36,960 feet	---	---
Shotcrete Lined	91,305 feet	85,960 feet	5,925 feet	15,060 feet

Table 6-6 summarizes the design parameter assumed for the four tunnel segments associated with Alternative 5A for the purposes of establishing a conservative cost.

TABLE 6-6
Design Parameter Assumptions for Alternative 5A Tunnels

Item	T1	T2	T3 & T4
Excavated Diameter	14 feet	14 feet	14 feet
Excavation Method	TBM	TBM	TBM
Length	16.7 miles	17.6 miles	7.1 miles
Fault Zone Crossings	---	---	---
Joint Systems	9,650 feet	4,600 feet	---
Steel Liner at Portals	500 feet	500 feet	2,000 feet
Pre-Excavation Grouting	14,475 feet	6,900 feet	---
Shotcrete Lined	78,025 feet	87,830 feet	35,500 feet

Alternative 5C tunnel segments T1 and T2 would be constructed using drill and blast methods with an excavated diameter of 12 feet (horseshoe shaped) with a steel-lined finished diameter of 10 feet. The tunnel segments T3 and T4 are common between Alternative 5A and 5C.

See *Chapter 2.0 – Regional Conveyance System Operations and Sizing* for more details on the tunnels. Unit costs for the tunnels were developed by comparing the costs from the *2001*

Feasibility Study Cost Refinement, *2013 Master Plan Update*, and *2017 Cost Update* and Black & Veatch's tunnel construction cost data-base for similar projects. Quantities and cost items reflect the tunnel segment lengths discussed in *Chapter 2.0 - RCS Operations and Sizing*.

Annual costs for O&M were added considering costs equivalent to 0.5 percent of capital costs. O&M cost accounts for the labor, equipment, and loss of water associated with reoccurring shutdowns for inspections and routine lining replacements. Shutdowns are assumed to occur no more frequently than every five years. Capital and annual costs for tunnels are summarized in Table G-4 of Appendix G.

6.10 Pump Stations

The pump stations required for the three alternatives are described as follows.

Alternative 3A would require three pump stations with approximately 657 feet of pumping head each to overcome the static and dynamic losses in the system.

Alternative 5A would require two pump stations with approximately 800 feet of pumping head each to deliver the water to SVR. Alternative 5A would also require another pump station with approximately 490 feet of pumping head, in addition to other improvements to the Water Authority's existing aqueduct system, to convey the water from SVR north to the TOVWTP.

Five pump stations with approximately 800 feet of pumping head each would be required for Alternative 5C to deliver the water to SVR. Similar to Alternative 5A, Alternative 5C would also require another pump station with approximately 490 feet of pumping head, in addition to other improvements to the Water Authority's existing aqueduct system, to convey the water from SVR north to the TOVWTP.

Pump station costs include civil, structural, mechanical, and electrical costs along with the associated costs of the forebay.

Design criteria for the pump stations associated with Alternative 3A are presented in *Chapter 2.0 – RCS Operations and Sizing*. Design criteria for the pump stations associated with the aqueduct improvements are presented in *Chapter 3.0 – Aqueduct Operations and Integration of the RCS*. The pump stations for Alternatives 5A and 5C that would pump approximately 800 feet of head are based on the design criteria presented in the *2017 Cost Update*.

Unit costs were based on other recent, similarly sized projects. Quantities reflect those described in this report.

Annual costs for the pump stations include: 1) a cost for labor, 2) a cost for energy, 3) a cost for major equipment replacement, and 4) an annual OMR cost. The labor cost accounts for the Water Authority's staff time to operate and maintain the pump stations. Labor costs are based on staffing estimates and current labor rates provided by the Water Authority. Table 6-7 presents the Water Authority's estimated staffing plan for the pump stations. Energy costs were calculated using the latest SDG&E and IID energy rates based on continuous operation all year round, except for 20 days in January for annual maintenance. Major equipment replacement covers the cost to replace the main duty pumps and motors.

Annual OMR costs are comprised of two categories. The first is a general OMR category accounting for predictable annual costs, such as general upkeep and the replacement of motor bearing oil, mechanical seals, air filters, and instruments. The general OMR category cost was provided by the Water Authority based on their experience with other large pump stations. The second category accounts for more costly, unanticipated replacements, such as upgrading the SCADA to a new technology, replacing electrical gear or components thereto, replacements to structural components, or piping rehabilitation. This study assumed 10 percent of the capital cost of the non-structural pump station components are replaced every 20 years. Additional analysis is recommended to better determine what that cost would be.

TABLE 6-7
Estimated Staffing Plan – Pump Stations

Item	Staff	Unit
Mechanical Maintenance	1	per pump station
Electrical Maintenance	1	per pump station
Supervisor	1	per alternative

Capital and annual costs for pump stations are summarized in Table G-5 of Appendix G.

Similar to the canals, the cost opinion developed for the pump stations exceeded the level of detail typically associated with a Class 4 estimate in accordance with the Association for the Advancement of Cost Engineering. As such, a 20% contingency was applied to the pump stations capital costs in lieu of the typical 30% used elsewhere.

6.11 Power Generating/Pressure Control Facilities

No new PGF or PCF would be required on Alternative 3A. Therefore, the PGFs and PCFs are based on those described in the *2013 Master Plan Update* the PGFs located on Alternative 5C would be capable of recovering energy and reducing the pressure within the pipeline as the alignment transitions from higher elevations crossing the mountain range to lower elevations approaching El Capitan Reservoir. Three PGFs on Alternative 5C would each reduce approximately 800 feet of pressure head. A PCF is also needed on Alternatives 5A and 5C to reduce pressure in the pipeline at the San Vicente Reservoir Outfall Structure.

PGFs are designed for 800 feet of head at 487 cfs with vertical Pelton type turbines. PCFs are designed for the specific pressure reduction required for each alignment and would include pressure reducing sleeve valves. Unit costs from the *2001 Feasibility Study Cost Refinement*, *2013 Master Plan Update*, and *2017 Cost Update* were escalated based on the ENR Construction Cost Indexes for the Los Angeles Market. Quantities from the *2001 Feasibility Study Cost Refinement* were utilized.

Annual costs for O&M and equipment replacement were escalated based on actual cost escalation and updated power costs. Table 6-8 presents the estimated staffing plant at the PGFs.

TABLE 6-8
Estimated Staffing Plan – PGFs

Item	Crews	No. per Crew	Total Staff
Mechanical Maintenance	1	2	2
Electrical Maintenance	1	2	2

Capital and annual costs for pump stations are summarized in Table G-6 of Appendix G.

6.12 Electric Distribution

Electric transmission lines and substations are necessary to provide power to pump stations and treatment facilities and to transmit power generated at PGFs. Design criteria for electric transmission are presented in *Chapter 5.0 – Power Supply Alternatives*.

Unit costs for the electric transmission lines and substations required for Alternatives 5A and 5C were developed in the *2001 Feasibility Study Cost Refinement* and were escalated based on the ENR Construction Cost Indexes for the Los Angeles Market. New unit costs were developed for the electric transmission lines and substations required for Alternative 3A.

Quantities and cost items reflect the transmission line lengths developed in this report. This report assumes that SDG&E and IID would operate and maintain the electric transmission lines and substation once built. Annual energy costs were included with the pump stations and PGFs/PCFs annual costs. Capital and annual costs for electric transmission are summarized in Table G-7 of Appendix G.

6.13 Treatment Facility

Costs developed for salinity treatment are based on a facility located in the Imperial Valley utilizing MF and RO for the treatment. The brine would be conveyed to the Salton Sea via a 30-inch diameter pipeline for use in constructed wetlands.

TF costs include civil, structural, mechanical, and electrical costs along with the associated costs of the influent equalization forebay.

Design criteria for the TF is presented in *Chapter 4.0 – Treatment, Blending, and Brine Management Options*. Quantities reflect those described in *Chapter 4.0 – Treatment, Blending, and Brine Management Options*.

Annual costs for the TF include: 1) a cost for labor, 2) a cost for energy, 3) a cost for major equipment replacement, 4) a general O&M cost, and 5) a cost for chemicals. The labor cost accounts for the Water Authority's staff time to operate and maintain the TF. Labor costs are based on staffing estimates and current labor rates provided by the Water Authority. Table

6-9 presents the proposed staffing plan for the TF and associated brine pipeline, which is based on input from the Water Authority and Black & Veatch's experience.

TABLE 6-9
Estimated Staffing Plan –Treatment Facility

Item	Total Staff
Plant Manager	1
Chief Operator	1
Operations Manager	1
Lead Operators	5
Operators	15
Compliance Officer	1
Health & Safety Specialist	1
OT/System Integrator	1
Instrument Technicians	3
Process Control Engineer	1
Maintenance Manager	1
Maintenance Supervisors	2
Electrical Technicians	3
Mechanical Technicians	7
Laborers	8
Janitor	1
Brine Pipeline Maintenance	2

Energy costs were calculated using the latest IID energy rates based on continuous operation all year round, except for 20 days in January for annual maintenance. Major equipment replacement covers the cost to replace the MF membranes, RO membranes, and cartridge filters. General OMR costs account for the replacement of valves and pumps, general upkeep of facilities, and other more costly, unanticipated replacements, such as upgrading SCADA to new technology or replacing electrical gear. General OMR costs assume three (3) percent of the capital cost of the non-structural pump station components annually. Chemical costs include all of the continuous use chemicals associated with the treatment processes, such as MF clean in place (CIP) and maintenance wash chemicals and RO CIP chemicals.

Capital and annual costs for the TF are summarized in Table G-8 of Appendix G.

6.14 Environmental Mitigation and Permitting

Environmental mitigation and permitting would be required for regulatory approval of the Regional Conveyance System Project. All alternatives would require environmental mitigation and permitting; however, since Alternative 3A and 5A are constructed in tunnels for approximately 50 percent of their length, fewer environmental impacts are anticipated. Details on the environmental mitigation and permitting anticipated to be required are presented in *Chapter 7.0 – Environmental Review and Permitting.*

Costs for environmental permits were provided by the Water Authority.

Costs for environmental mitigation were based upon estimated land acquisition costs provided by the Water Authority. Other factors, such as environmental monitoring during and after construction, were estimated based prior experience on similar projects.

New quantities for land acquisition were developed for all three alternatives.

No annual costs for environmental mitigation and permitting are anticipated. Capital costs for environmental mitigation are summarized in Table G-9 of Appendix G.

6.15 Operational Storage

Costs developed for operational storage include: 1) the 900 AF of operational storage required for IID to negate the need for a parallel AAC, 2) the improvements in the Valley Center or Lake Wohlford area to provide operational storage for Alternative 3A, and 3) the 20 to 40-million-gallon, day storage tank required near the TOVWTP. As stated previously, the Phase A baseline cost estimates assume no partnerships and associated benefits/funding due to the preliminary nature of this review and limited dialogue with potential partners. Criteria for the 900 AF IID storage and the improvements to a lake in the Valley Center or Lake Wohlford area are provided in *Chapter 2.0 – RCS Operations and Sizing*. The 900 AF IID storage was priced as an open, earthen reservoir with a plastic liner. The improvements to lakes in the Valley Center or Lake Wohlford area were assumed to include a replacement of the dam, along with a new inlet/outlet structure.

Annual costs for operational storage include a labor cost that accounts for the Water Authority's staff time to operate and maintain the facilities and a general OMR cost that accounts for the replacement of valves, and general upkeep of facilities. Labor costs are based on staffing estimates and current labor rates provided by the Water Authority.

Table 6-10 presents the Water Authority's estimated staffing plan for the pipeline. This study assumes that the 900 AF IID storage reservoir would be maintained by the staff at the nearby TF and that the 20 to 40-million-gallon, day storage tank would be maintained by the staff at TOVWTP.

TABLE 6-10
Estimated Staffing Plan – Operational Storage

Item	Crews	No. per Crew	Total Staff
Operators	1	3	3

Capital and annual costs for operational storage are summarized in Table G-10 of Appendix G.

6.16 Office and Warehouse

Due to the distance of the proposed facilities from Water Authority's other locations, additional office and warehouse space would be required for operation and maintenance. The warehouse would include a storage yard for the equipment, tools, spare parts (i.e., valves, instrumentations, etc.), and vehicles necessary to maintain the facilities. It is assumed that the office and warehouse would be located at, or near, the proposed TF for each alternative.

Unit costs developed for the offices would include all furnishings, while the storage yard would include lighting and basic canopies for storage or parking.

Annual costs for the office and warehouse would include labor, energy, and general OMR. Table 6-11 presents the estimated staffing plan for the office and warehouse.

TABLE 6-11
Estimated Staffing Plan – Office and Warehouse

Item	Crews	No. per Crew	Total Staff
Warehouse Technician	1	1	1
Warehouse Manager	1	1	1

Capital and annual costs for the office and warehouse are summarized in Table G-11 of Appendix G.

6.17 Additional Staff at Kearny Mesa

The RCS would require additional staff at the Water Authority's office in Kearny Mesa in addition to the staff described at each of the individual facilities. The Water Authority provided an estimate on the number of additional staff required, as presented in Table 6-12. Labor costs are based on staffing estimates and current labor rates provided by the Water Authority.

TABLE 6-12
Estimated Staffing Plan – Additional Staff at Kearny Mesa

Item	Total Staff
Human Resources	1
Finance - Accounting	1
Finance - Budgeting	1
Finance - Payroll	1
IT Supervisor	1
IT Analyst	1
Water Resources Principal	1
Water Resources Senior	1
Engineer (P.E.)	2
Engineering Senior Engineer	1
RCS Operations and Maintenance Manager	1

Annual costs for the additional staff required at Kearny Mesa are summarized in Table G-12 of Appendix G.

6.18 Energy Usage

Energy usage and the corresponding energy costs were evaluated for the pump stations and TF. Similarly, energy production and the corresponding value of the energy produced at the power generating facilities was estimated. Energy usage was estimated assuming the conveyance system would operate continuously throughout the year, with the exception of a 20-day period every January, during which the system would be offline for maintenance. As such, the remainder of the year the system would convey an additional 10-percent flow to offset for maintenance and system outages.

Energy costs for the pump stations and the TF were based on current rates obtained from SDG&E and IID, as presented in *Chapter 5.0 – Power Supply Alternatives*. The rates used to calculate the energy recovered at the power generating facility were based on the energy recovery rates used in the *2013 Master Plan Update* escalated to 2020 dollars and are presented in Table 6-13.

TABLE 6-13
SDG&E Energy Service Provider Commodity Rate Structure (July 2012 escalated to 2020 dollars) – Power Generating Facilities

Item	Summer	Winter
Energy Charges ($/kWh)		
On-Peak	0.10084	0.09525
Off-Peak	0.08167	0.08750
Super Off-Peak	0.05892	0.06501

Table 6-14 summarizes the estimated annual energy usage cost for the pump stations on all three alternatives. Alternative 3A would have three pump stations, two with IID as the energy service provider and the other served by SDG&E. Alternative 5A would have two pump stations, one with IID as the energy service provider and the other serviced by SDG&E. Alternative 5C would also have one pump station served by IID and four pump stations served by SDG&E. Furthermore, Alternatives 5A and 5C would both have an additional pump station associated with the improvements to the Water Authority's existing aqueduct system, which would also be serviced by SDG&E.

TABLE 6-14
Estimated Annual Energy Cost – Pump Stations (2020 Dollars)

	Alternative 3A	Alternative 5A		Alternative 5C	
Item	**RCS Pump Stations**	**RCS Pump Stations**	**Pump Station on Aqueduct(1)**	**RCS Pump Station**	**Pump Station on Aqueduct(1)**
SDG&E Energy Costs (TOU-PA-3 ≥ 20 kW Rate Structure – June 2019)					
No. of Pump Stations	1	1	1	4	1
Basic Service Charge	$1,514.88	$1,514.88	$1,514.88	$1,514.88	$1,514.88
Energy Charge ($/kWh)					
On/Off Peak	$19,446,611	$23,667,000	$7,481,000	$98,785,000	$7,481,000
Commodity Rates ($/kWh)					
On Peak	$4,224,651	$6,892,000	$1,625,000	$21,460,000	$1,625,000
Off Peak	$7,201,750	$8,764,000	$2,770,000	$36,583,000	$2,770,000
Super Off Peak	$3,845,976	$4,681,000	$1,479,000	$19,537,000	$1,479,000
IID Energy Costs (Municipal Rate Schedule)					
No. of Pump Stations	2	1	---	1	---
Customer Charge	$288	$144	---	$144	---
Energy Charge ($/kWh)	$47,004,731	$28,603,000	---	$29,847,000	---
Energy Cost Adjustment Factor					
Non-Renewable Factor	($7,209,315)	($4,387,000)	---	($4,578,000)	---
Renewable Factor	$7,456,491	$4,537,000	---	$4,735,000	---
Annual Energy Cost	**$82,000,000**	**$86,200,000**		**$219,800,000**	

Note:
1. A new pump station near the existing Del Dios Valve Vault is required to move water north from SVR.

Table 6-15 summarizes the estimated annual energy cost for the TF for all three alternatives. IID would be the energy service provider for the TF in all three alternatives.

TABLE 6-15
Estimated Annual Energy Cost – Treatment Facility - AL-TOU ≥ 12 MW Rate Structure (June 2019)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Customer Charge	$144	$144	$144
Energy Charge ($/kWh)	$13,011,000	$13,011,000	$13,011,000
Energy Cost Adjustment Factor			
Non-Renewable Factor	($1,996,000)	($1,996,000)	($1,996,000)

TABLE 6-15
Estimated Annual Energy Cost – Treatment Facility - AL-TOU ≥ 12 MW Rate Structure (June 2019)

Renewable Factor	$2,064,000	$2,064,000	$2,064,000
Annual Energy Cost	**$13,080,000**	**$13,080,000**	**$13,080,000**

Table 6-16 summarizes the estimated annual energy produced at the PGFs. SDG&E would be the energy service provider for the PGFs. Alternative 5C would have three PGFs, while Alternatives 3A and 5A would have none.

TABLE 6-16
Estimated Annual Energy Produced – Power Generating Facilities (2020 Dollars)[(1)]

Item	Alternative 3A	Alternative 5A	Alternative 5C
Commodity Rates ($/kWh)			
On Peak	---	---	($8,600,000)
Off Peak	---	---	($14,960,000)
Super Off Peak	---	---	($9,860,000)
Annual Energy Cost	**---**	**---**	**($33,400,000)**

Note:

1. The energy cost is calculated based on the SDG&E Energy Service Provider commodity rate schedule utilized in the 2013 Master Plan Update escalated to 2020 dollars.

Tables 6-17 summarizes the total annual energy cost for each of the three alternatives summarizing all of the major loads.

TABLE 6-17
Summary of the Total Estimated Annual Energy Cost for the RCS (2020 Dollars)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Energy Cost			
Pump Stations	$82,000,000	$86,200,000	$219,800,000
Treatment Facility	$13,080,000	$13,080,000	$13,080,000
Energy Recovery - PGF	---	---	($33,400,000)
Annual Energy Cost	**$95,080,000**	**$99,280,000**	**$199,480,000**

6.19 Soft Costs

Soft costs were primarily developed by the Water Authority. In some cases, such as for construction management, the cost was developed as a percentage of the construction cost for each major facility based on the Water Authority's past experience on projects of similar scope. Other soft costs, such as environmental mitigation, were developed in more detail.

Table 6-18 presents the soft costs estimated for each of the alternatives with the construction management itemized as its own line item.

TABLE 6-18
Summary of Soft Costs (2020 Dollars)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Phase A Planning Study	$2,600,000	$2,600,000	$2,600,000
Phase B Planning Study	$1,300,000	$1,300,000	$1,300,000
Procure Preliminary Designer	$50,000	$50,000	$50,000
Procure Env. Consultant	$50,000	$50,000	$50,000
Preliminary Design	$2,000,000	$2,000,000	$2,000,000
Environmental Work	$3,900,000	$3,900,000	$3,900,000
Right-of-Way: Title, Appraisals, Phase 1, Survey, Acquisition, and Legal	$15,532,000	$14,828,000	$21,318,000
Property Acquisition/Land Costs	$17,060,000	$9,620,000	$16,390,000
CEQA / NEPA / Permits	$5,200,000	$5,200,000	$5,200,000
Public Outreach	$2,810,000	$2,810,000	$2,810,000
Legal Review	$6,250,000	$6,250,000	$6,250,000
Environmental Mitigation/Monitoring	$17,323,500	$10,785,500	$17,852,500
Owners Representative/Program Management	$31,644,000	$31,644,000	$31,644,000
Staff Support - Programmatic	$31,644,000	$31,644,000	$31,644,000
Design and Bid Phase	$327,000,000	$328,000,000	$320,050,000
Subtotal (Pre-construction)	**$464,370,000**	**$450,680,000**	**$463,060,000**
Construction Management	$664,200,000	$662,100,000	$665,500,000
Total Soft Costs (2020 Dollars)	**$1,128,600,000**	**$1,112,800,000**	**$1,128,600,000**

6.20 Comparison with Prior Studies

Table 6-19 presents a comparison of the probable construction costs presented in prior studies with the costs prepared as part of this report using 2020 dollars. The costs presented in Table 6-19 represent direct and indirect capital construction costs escalated to 2020 dollars and do not include capital, pre-construction soft costs, or construction management soft costs. Table 6-19 also contains the post construction annual OMR costs, whereas 2013 and 2017 annual costs do not include replacement costs.

TABLE 6-19
Opinion of Probable Construction and Annual OMR Cost Comparison (2020 Dollars)

Study	Alternative 3A		Alternative 5A		Alternative 5C	
	Construction Costs(1)	Annual OMR Costs(1)	Construction Costs(1)	Annual OMR Costs(1)	Construction Costs(1)	Annual OMR Costs(1)
2013 Report(2)	-	-	$2,057.5M	$78.0M	$2,394.5	$146.6M
2017 Report(3)	-	-	$2,386 M	$134.4 M	$2,818 M	$285.5 M
2020 Report(4), (5)	$3,825 M	$143 M	$3,849 M	$149 M	$3,730 M	$258 M

Notes:

1. All values include contingency but do not include pre-construction or construction management soft costs.
2. Cost for Average Annual Flow of 280,200 AF/y and using blending for treatment.
3. Cost for Average Annual Flow of 400,000 AF/y.
4. Cost for Average Annual Flow of 279,500 AF/y (QSA water) and 22,000 AF/y capacity for the potential partnership with Borrego Valley or another off-taker.
5. Cost includes TF and associated treatment OMR costs.

It should be noted that the difference in capital costs between the three alternatives is within the range of accuracy of this level of planning estimate. A full economic comparison, including annual costs, is discussed in detail in Section 6.21.

As can be seen in Table 6-19, the construction costs increased for Alternatives 5A and 5C from the *2017 Cost Update* to this 2020 Report. Several major factors that have driven the increase in cost are noted:

- Salinity Treatment – The *2017 Cost Update* assumed that salinity treatment would be accomplished through blending at the SVR. However, after further evaluation of anticipated flow sources, it was determined that blending alone would not consistently meet the maximum acceptable target TDS. Therefore, this 2020 Report includes costs for a salinity treatment facility.
- Operational Storage – The *2017 Cost Update* assumed the AAC would have sufficient capacity to convey the QSA water to the WSM. However, based on conversations with IID during this evaluation, it was determined that a 900 AF reservoir was required in order to convey the required volume of water through the AAC.
- Office and Warehouse – The *2017 Cost Update* did not include an office or warehouse for Water Authority operation staff in Imperial County to operate and maintain the RCS system. Based on conversations with Water Authority Staff, an office and a warehouse in Imperial County would be necessary for operation.
- Construction Management – The *2017 Cost Update* included construction management with other soft costs (such as administrative and engineering) at 25 percent of the total construction cost. For this 2020 Report, the Water Authority individually estimated the construction management costs for each component of the system based on past experience on similar projects. The result was a significant increase in the estimated cost for construction management.

- Aqueduct Improvements – The *2017 Cost Update* did not estimate costs for improvements to the Water Authority's existing aqueduct system in order to convey the QSA water from SVR to meet demands to the north. This 2020 Report evaluated new strategies to convey water north from SVR and included costs for improvements to the aqueduct system.
- OMR Costs – The *2017 Cost Update* generally estimated the OMR costs based on a percentage of the construction cost for each facility. This study estimated the OMR costs in greater detail, including providing estimates for 1) the labor to operate and maintain each facility, 2) the labor required to operate and maintain the entire RCS, 3) major equipment replacements (such as pumps), and 4) chemicals and consumables (such as membranes) at the TF.

6.21 Economic Comparison

Based on the cost information presented in this study and the detailed project schedule, a life-cycle cost analysis was performed to compare the three alternatives.

Net present value (NPV) is one of many capital budgeting methods used to evaluate potential capital projects in which an entity might want to invest. NPV uses discounted cash flows in the analysis, which makes the NPV more precise than of many of the other capital budgeting methods as it considers both risk and time variables.

NPV analyses use several assumed variables to evaluate the forecasted cash flows of a project by discounting them back to the present (2020) using project schedule and a pre-determined, weighted average cost of capital (known as a discount rate). When comparing two or more projects (in this case, alternatives), the project with the lower NPV is the preferred. Given the set of assumptions and time horizon, should the resulting NPV values fall within a narrow range, it could be appropriate to continue forward to future phases of study with those alternatives.

Table 6-20 presents a comparison of the NPV analysis. The NPV reflects all costs related to the development, planning, and construction of the RCS alternatives, as well as annual OMR costs related to RCS operations, from 2045 to 2112. All alternatives make use of the same financing, escalation, and project schedule assumptions.

TABLE 6-20
Net Present Value Comparison (2020 – 2112)

	Capital NPV(1)	OMR NPV(2)	Total NPV
Alternative 3A	$7,068 M	$18,037 M	$25,106 M
Alternative 5A	$7,148 M	$18,633 M	$25,781 M
Alternative 5C	$6,947 M	$31,124 M	$38,071 M

Notes:

1. Includes costs of financing (principal, interest, and cost of issuance).
2. Does not include cost of supply; however, it does include the costs associated with assumed treatment losses (20,000 AFY). The supply costs are equal under each alternative, thus has no bearing on the NPV comparison.

While the NPV analysis above details a single outcome, a sensitivity analysis of forecasted capital expenditures was performed. The sensitivity analysis details the estimated NPV range of increasing and decreasing capital costs by 40%. The costs presented for the RCS include contingencies which provides an additional layer of conservatism. Additionally, as a majority of the capital costs are expected to be debt funded, an analysis of interest rate sensitivity was performed. The baseline assumption for financing is 5%, as such interest rates of 4% and 6% were reviewed. These two assumptions were then combined to provide a high/high and a low/low estimate.

TABLE 6-21
NPV Capital Sensitivity Comparison(1)

	+40% to Capital	-40% to Capital	Lower 4% Debt	Higher 6% Debt	High Capital & High Debt	Low Capital & Low Debt
Alternative 3A	$9,344 M	$4,793 M	$6,133 M	$8,041 M	$10,635 M	$4,164 M
Alternative 5A	$9,433 M	$4,863 M	$6,185 M	$8,146 M	$10,756 M	$4,213 M
Alternative 5C	$9,165 M	$4,728 M	$6,026 M	$7,904 M	$10,434 M	$4,108 M

Note:

1. Includes costs of financing (principal, interest, and cost of issuance)

The cost per acre-foot is another point of comparison for analysis. While the NPV values shown on Table 6-21 include capital and OMR costs, the cost per acre-foot calculation on Table 6-22 includes the cost of supplies in addition to capital and OMR costs. These costs reflect the total dollars spent to deliver 277,700 acre-feet to the Water Authority's system. The RCS is as much a supply project as it is a transportation project, as it enables the Water Authority to maintain its reliable and independent QSA supplies, rather than falling back on to MWD reliance and its stressed Bay-Delta.

TABLE 6-22
NPV Equivalent Unit Cost ($/AF)(1)

	Cost per AF	Notes
Alternative 3A	$1,697	---
Alternative 5A	$1,733	---
Alternative 5C	$2,384	Higher unit cost due to high pumping (O&M) costs.
MWD Reliance	$2,691	MWD Full Service Tier 1 to replace 200,000 AF of IID deliveries in 2048. Remaining 77,700 AF to be "exchanged" at the MWD Transportation rates. Reflects annual adjustments of 5.1% based on historical increases (2003-2022). No rate adjustments (increases) have been assumed for MWD's planned Cal Water Fix or Recycled Water Program costs (capital or O&M).
Local Supply Development	$2,594	Cost of $3,000/AF, based on a new desalination project. MWD Full Service Tier 1 to replace 200,000 AF of IID deliveries in 2048. Remaining 77,700 AF to be "exchanged" at the MWD Transportation rates. All costs are escalated at escalated at 3%. 77,700 AF to be maintained based on exchange agreement (Canal Supply + MWD Transportation). Assumes no double costs are incurred, leading up to 2047 QSA contract expiration.

Note:

1. Includes all costs (including upfront construction of RCS) to deliver 277,700 AF to the Water Authority between 2045 and 2112.

As demonstrated, the RCS project would be forecasted to provide significant savings. While costs will continue to shift leading up to potential design and construction, the savings gap is not trivial and provides substantial margin. The Water Authority will continue to develop and update its economic analysis. Should the RCS project advance to future phases, Water Authority staff will look at annual cost impacts, leading up to construction, rather than lifecycle project NPV. Early analysis shows important NPV savings, and it is necessary to understand how these savings can be realized across the generational spectrum, not simply next generation water users.

6.22 Conclusion

This chapter documents the comparison of project risks, cost opinions, and an economic comparison of the alternatives to the status quo. The high level, qualitative comparison of project risks completed for this study noted slight advantages to Alternatives 3A and 5A as compared to Alternative 5C. These advantages mostly manifest in the lower pumping head and small quantity of cut-and-cover construction methods required for Alternatives 3A and 5A. No fatal flaws were identified for any of the alternatives as a part of this study.

A quantitative comparison of project risks is recommended during the next phase of work to better compare the alternatives.

The opinions of probable construction costs developed for each of the alternatives were all within five percent of each other. This cost delta is within the range of accuracy of this level of planning estimate.

The estimated annual cost developed for Alternative 5C is nearly double the cost to operate Alternatives 3A and 5A. The cost difference is due mostly to the increased pumping head required to convey water over the mountain range as opposed to tunneling through the mountains. This increased annual cost difference has a significant impact on the economic comparison of the projects over a 100-year life cycle, particularly after the capital financing costs have been paid off.

An economic evaluation was performed for the RCS alternatives. In addition, an economic analysis was performed for various MWD supply options and local supply development for the equivalent QSA supplies that would be replaced if the QSA is not extended past 2047 and 2077. Based on the NPV analysis completed, along with preliminary sensitivity analyses performed, the RCS Alternatives 3A and 5A are cost competitive to all other options explored.

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7

ENVIRONMENTAL REVIEW AND PERMITTING

Chapter 7.0 Environmental Review and Permitting

7.1 Introduction

7.1.1 Overview

This chapter describes the environmental review and permitting requirements for the RCS Alignment 3A. Alignments 5A and 5C environmental constraints and permitting were previously described in *2017 Supplemental Colorado River Conveyance Alternative Report Update*.

7.1.2 Chapter Organization

The environmental review and permitting processes for the RCS are summarized below and discussed in the following chapter sections.

Environmental Requirements – This section provides an overview of the environmental requirements for the project.

Regulatory and Environmental Permitting Process – This section identifies the content and steps required to be undertaken to complete the California Environmental Quality Act (CEQA) and National Environmental Policy Act (NEPA) processes, as well as related consultation and permitting actions.

Summary of Environmental Issues – This section lists the anticipated environmental issues to be encountered for the project. The list is preliminary and would be revisited in development of the final work plans.

Risks Identified that Could Impede Environmental Review and Permitting, and Process Success – This section describes the land acquisition risks that could impede the progress of the project. Specifically, lands are identified that are particularly sensitive and therefore a risk factor for the project.

Incidental Environmental Benefits of the RCS Project – This section identifies several environmental benefits that could result from the RCS project.

Schedule and Budget Estimates - This section provides an overview of the environmental process schedule and preliminary environmental budgets.

7.2 Environmental Requirements

The RCS Project being contemplated by the Water Authority would require acquisition of rights-of-way (ROW)and easements or purchased lands for development of the Alignment 3A, a 132 mile-long system. Assuming an average ROW width of 100-feet, the route includes approximately 1,700 acres, with additional lands required for appurtenant access roads, tunnel portals, a vertical shaft from the tunnel to the ground surface in the vicinity of Lake Henshaw, and powerline corridors to each of the three pump stations and the treatment facility. The linear system crosses a variety of Federal, State, County, City and private lands, and borders or is in close proximity to other lands that include habitat preserves, State and local parks, a National Landmark, a military reservation, and tribal reservations.

Regulatory coordination and land acquisition would be accomplished in tandem with the environmental review processes required to support the Water Authority's project approval decision-making. This would involve performing all the environmental and engineering studies required and preparation of the related permit applications with supporting environmental assessments and mitigation proposals. Processing of these applications would require completion of documentation in compliance with CEQA, including an Environmental Impact Report (EIR) for which the Water Authority is the lead agency. NEPA documentation would include an Environmental Impact Statement (EIS), with the Bureau of Reclamation as the Federal lead agency. A joint EIR/EIS document should be prepared under terms of a Memorandum of Understanding (MOU) to coordinate between the two lead agencies, and all Federal and State permitting agencies that would need to rely upon the document to support their individual decision-making requirements.

7.3 Regulatory / Environmental Permitting Processes

A linear project of this size and complexity involves a range of environmental permitting requirements at Federal, State and local levels. The following section identifies the content and steps required to be undertaken to complete the CEQA and NEPA processes, and related consultation and permitting actions.

7.3.1 Development of the Project Description

The Project Description includes a statement of the purpose and need for the project, goals and objectives, identification of all project components, project operations, and proposed construction schedule, with supporting maps and design drawings showing the project location, boundaries, and land ownership. The Project Description also includes an inventory of all required project approvals and the Responsible Agencies (CEQA) and Cooperating Agencies (NEPA) that would need to rely upon the EIR/EIS as a basis for their individual decision-making.

The Project includes – canals, pipelines, tunnels, pump stations, electric transmissions lines, storage facilities, and a treatment facility with associated brine management. For each of these there are also construction access, staging areas, and disposal sites. A complete list of the project description elements to be addressed is provided below.

1. Project Sizing and Operation

- Canal alignment, width and capacity
- Pipeline alignment and diameter
- Tunnel alignment and capacity
- Treatment facility and associated brine management
- Pump stations capacity, power requirements, and source(s)

- Storage facility size and capacity
- Monthly/seasonal operating requirements to meet water delivery needs
- Other operational parameters

2. Water Supply

- Water Authority Quantification Settlement Agreement (QSA) water supplies
- Water delivery system changes – place of diversion

3. Tunnels

- Likely tunneling conditions (geologic and geotechnical)
- Water control during construction

4. Powerline Alignments to Pump Stations and RCS Treatment Facility

- Voltage requirements
- Alternative corridors
- Source(s) of power (Imperial Irrigation District (IID) and/or San Diego Gas and Electric (SDG&E))

7.3.2 Staged or Program EIR

Water Authority staff have suggested a somewhat unique approach to the environmental review process intended to refine the ultimate project alignment and details, and possibly to streamline the subsequent project specific CEQA and NEPA processes. The approach includes early preparation of a high level CEQA document, either as a Staged EIR or a Program EIR as discussed further below. This approach that would examine a range of configurations, would provide an opportunity for an extensive scoping and consultation process, consider the whole range of potential environmental effects, and develop recommended mitigation strategies, or identify potential impacts that could not be mitigated.

Rather than supporting decision-making for project approval, the outcome of that process would be the selection of the preferred project alignment (or alignments) that would be approved for examination in the subsequent project-specific environmental review processes and permitting. This could streamline the subsequent EIR/EIS process by refining the Project Description and incorporating ideas and strategies developed in the agency consultation process. The program-level assessments of multiple alignments determined to be infeasible or undesirable would form the basis of the required assessment of alternatives in the subsequent joint EIR/EIS, and could help streamline the Federal NEPA process for consideration of alternatives.

The RCS project does qualify for preparation of a Staged EIR as defined in the CEQA Guidelines section 15167:

> *(a) Where a large capital project will require a number of discretionary approvals from government agencies and one of the approvals will occur more than two years before construction will begin, a staged EIR may be prepared covering the entire project in a general form. The staged EIR shall evaluate the proposal in light of current and contemplated plans and produce an informed estimate of the environmental consequences of the entire project.*

The aspect of the project before the public agency for approval shall be discussed with a greater degree of specificity.

(b) When a staged EIR has been prepared, a supplement to the EIR shall be prepared when a later approval is required for the project, and the information available at the time of the later approval would permit consideration of additional environmental impacts, mitigation measures, or reasonable alternatives to the project.

As detailed above, the RCS is a large capital project that would be subject to numerous discretionary approvals and would require timelines for final engineering design, land acquisition and final permitting. This process is expected to take more than two years after project approval. This process would also allow the use of high-level analyses and consideration of alternatives. In this case the aspect of the project, subject to agency approval, would be limited to selection of the action(s) to be taken to the next level of consideration, with the test for adequacy limited to "an informed estimate of the environmental consequences of the entire project". The subsequent EIR would then be more narrowly focused on the preferred alternative as the Proposed Action and would include sufficient detailed information rising to the "substantial evidence in the record" as the test for legal adequacy for deciding on project approval. Finally, a Staged EIR clearly recognizes the need to prepare a subsequent project-specific EIR, with no option to perform a less detailed assessment.

Alternatively, the RCS project also qualifies for preparation of a Program EIR as defined in the CEQA Guidelines section 15168 for a first stage examination of alternative alignments and selection of a preferred action to carry forward in a more detailed project-specific EIR.

(a) General. A program EIR is an EIR which may be prepared on a series of actions that can be characterized as one large project and are related either:

(1) Geographically,

(2) A logical parts in the chain of contemplated actions,

(3) In connection with issuance of rules, regulations, plans, or other general criteria to govern the conduct of a continuing program, or

(4) As individual activities carried out under the same authorizing statutory or regulatory authority and having generally similar environmental effects which can be mitigated in similar ways.

Advantages of the Program EIR are listed in the Guidelines to include "a more exhaustive consideration of effects and alternatives than would be practical in an EIR on an individual action". This process would allow for a more thorough consideration of cumulative impacts that might be slighted in a case-by-case analysis, and consideration of broad policy alternatives and program-wide mitigation measures.

Program EIRs are commonly used for master plans and community General Plans which include a wide variety of subsequent actions. The Water Authority prepared a Program EIR for its Regional Water Facilities Master Plan which covered dozens of separate projects that were all part of the water delivery and treatment system, but each of the projects in the

Master Plan also have some independent utility, meaning that if some components were not developed, other components would still be needed.

The RCS project could be viewed in a similar manner, as a large set of components that are all part of the larger water delivery and treatment system, but there is no independent utility to those components, and the Water Authority would not build one component unless it intended to build all the other components. In that sense, it really is a single project, with numerous connected parts that all must function together. The CEQA Guidelines include details of how a Program EIR is to be used with later activities, and says in part:

> *(c) Use with Later Activities. Subsequent activities in the program must be examined in the light of the program EIR to determine whether an additional environmental document must be prepared.*
>
> *(1) If a later activity would have effects that were not examined in the program EIR, a new Initial Study would need to be prepared leading to either an EIR or a Negative Declaration.*
>
> *(2) If the agency finds that pursuant to Section 15162, no new effects could occur or no new mitigation measures would be required, the agency can approve the activity as being within the scope of the project covered by the program EIR, and no new environmental document would be required.*

In this case, there is no plan to use the Program EIR in a way that allows approval of subsequent actions or avoids the need for a more detailed EIR for the selected preferred Proposed Action. In practice, either form of CEQA document could suffice, but it does appear that the Staged EIR could be more applicable to the RCS project.

Either a Staged or Program EIR could be completed within 18 to 24 months and could be prepared based mainly upon desktop technical studies with limited supporting field investigations to reduce costs and timing for this initial review. This could also help limit the scope of later field-intensive investigations needed to support the project-specific environmental review processes that lead to consideration of project approval and implementation.

7.3.3 Project Specific Joint CEQA & NEPA Documentation Process

The primary steps in completion of the joint EIR/EIS document are outlined below. These would need to be completed in close coordination between the two Lead Agencies.

- Confirm all details of the Project Description/Proposed Action and Alternatives
- Notice of Preparation (NOP) to prepare an EIR filed with the State Clearinghouse and County Clerk and mailed to Responsible Agencies, and Notice of Intent to prepare an EIS filed for publication in the Federal Register, and mailed to all Cooperating Agencies
- Consultation with all applicable resource agencies and tribes

- Draft EIR/EIS with Notice of Availability/Notice of Completion (NOA/NOC) to State Clearinghouse and County Clerk and mailed to Responsible Agencies, and Notice of Availability of the EIS filed for publication in the Federal Register, and mailed to all Cooperating Agencies and the EPA
- Responses to Comments mailed to Responsible Agencies and commenting agencies and Cooperating Agencies at least 10 days prior to project approval decision
- Final EIR/EIS, including Draft Environmental Impact Report (DEIR)/Draft Environmental Impact Statement (DEIS) with any and all changes made in response to comments received, and copies of the comments and responses, and Mitigation Monitoring and Reporting Program (MMRP)
- Water Authority Decision (Statement of Findings of Fact, FEIR Certification including MMRP, and Project Approval) and Notice of Determination (NOD) filed with the State Clearinghouse and County Clerk, and Federal Record of Decision filed for publication in the Federal Register, mailed to all Cooperating Agencies, and the EPA

7.3.4 Consultation

Concurrent with, and as an essential part of both the NEPA and CEQA environmental review and permitting processes, formal consultation must be undertaken to complete Federal and State requirements and to obtain related permits and agreements. Consultation would be required with a variety of resource agencies. A detailed list is provided in Table 7-1. In addition, the table lists all project approvals, permitting and land acquisition. Other agencies and required approvals could be identified during the formal scoping process.

Other informal consultations that would be necessary include the County of San Diego, affected municipalities, local landowners and interested citizens, and local and regional environmental groups.

TABLE 7-1
Resources Agencies, Environmental Permitting, and Discretionary Approvals

Agency	Approvals/Permitting Description
	Federal
Bureau of Reclamation	NEPA Lead Agency – Lower Colorado River Watermaster and QSA Water Transfer, all project elements
Bureau of Land Management	Consultation for ROW easements on BLM lands for canal, pipeline and pump station alignments in Imperial Valley
National Forest Service	Consultation for canal, pipeline and/or tunnel segments easement for on State parks land, including Cleveland National Forest
U.S. Army Corps of Engineers	Consultation and application for wetlands/Waters of the U.S. – Individual or Nationwide Permit – CWA §404, stream channel crossings including desert dry washes, and potential water storage reservoir sites
US Fish and Wildlife Service	Endangered Species Act §7 consultation, including the U.S. Fish and Wildlife Service (for the federal ESA) and the California Department of Fish and Wildlife (for the State CESA), to obtain a Biological Opinion (USFWS) and Consistency Determination (CDFW)
Department of Defense, U.S. Navy	Consultation for easement / ROW for canal segment crossing southeast corner of the U.S. Naval Parachute Drop Zone in vicinity of canal segment near mile 25
National Park Service	Consultation for possible easement / ROW for canal / pipeline segment along northern edge of the San Felipe Creek National Natural Landmark on the south side of Highway 78 west of intersection of Highway 86 between miles 4 and 10
	State (California)
State Water Resources Control Board (Regional Water Quality Control Boards)	Consultation and application for Water Quality Certification with the Regional Water Quality Control Board (RWQCB) pursuant to the Clean Water Act, section 401 (Region 7 office for Imperial Valley segments, and Region 9 office for San Diego County segments), QSA Water Transfers
Department of Fish and Game	• California Endangered Species Act (CESA) consultation • Streambed Alteration Agreements F&G Code §1600, including desert dry washes
Department of Parks and Recreation	• Special Use Permit • Subsurface easement for canal, pipeline and/or tunnel segments on or under State parks land, including Ocotillo Wells State Vehicular Recreation Area and Anza-Borrego Desert State Park
San Dieguito River Valley Regional Open Space Park Joint Powers Authority (also known as the San Dieguito River Park)	Subsurface easement for pipeline segment under Park land between miles 46 and 47
State Historic Preservation Office	Cultural Resources Consultation - NHPA §106 - to satisfy requirements of the National Historic Preservation Act (Section 106), usually including development of a formal agreement in the form of a MOU

TABLE 7-1
Resources Agencies, Environmental Permitting, and Discretionary Approvals

Agency	Approvals/Permitting Description
Native American Heritage Commission	Tribal and Cultural Resources Consultation
	Local
Water Authority	CEQA Compliance
Water Authority Member Agencies	Consultation/agreement for use of or interconnection to facilities.
County of San Diego	Permits and Stormwater Design
Imperial Irrigation District	Conveyance alignment, lands and power supply – transmission interconnection for pump stations, treatment facility and tunnel portals
San Diego Gas & Electric	Power supply – transmission interconnection for pump stations and tunnel portals
Tribes	Native American Consultation with all regional tribes to be determined in consultation with the Native American Heritage Commission

7.3.5 Scope of Environmental Review

The scope of the EIR/EIS would cover the full range of resource topics typically identified in the States' CEQA Guidelines and the Bureau of Reclamations' NEPA Handbook. While there is significant overlap between CEQA and NEPA, there are important differences that would need to be accounted for in the joint document, for example in the treatment of alternatives and assessment of environmental justice under NEPA, and mandatory assessment of unavoidable impacts, growth inducing impacts, and mitigation measures under CEQA.

The EIR/EIS would address each of the following topics as they are applicable to the Proposed Project:

- Aesthetics
- Agriculture and Forestry Resources
- Air Quality and Greenhouse Gas Emissions
- Biological Resources – all topics – terrestrial and aquatic and wetlands/Waters of the U.S.
- Cultural Resources
- Tribal Cultural Resources
- Energy
- Environmental Justice (NEPA)
- Geology, Geotechnical, Geochemistry (including geologic hazards and Paleontology)
- Hazards and Hazardous Materials
- Hydrology and Water Quality (including hydrogeology)
- Land Use and Planning
- Mineral Resources
- Noise
- Population and Housing
- Public Services
- Recreation
- Transportation
- Utilities and Service Systems
- Wildfire
- Alternatives Assessment
- Mitigation Measures/MMRP (CEQA)
- Other CEQA mandated discussions including energy impacts, significant unavoidable impacts, irreversible environmental changes, and growth inducing effects
- Responses to Comments
- Findings of Fact (CEQA) and Record of Decision (NEPA)

7.3.6 Other Project Permits

The permitting process would be extensive for the project. The following tables provide a detailed list identifying the project permits required beyond environmental permitting. Owner and Contractor obtained permits are included. Table 7-2 summarizes all the design and construction permits that would be required for the tunnels, pipelines, and pump stations. Table 7-3 summarizes all the design and construction permits that would be required for the treatment facility and brine management pipeline.

TABLE 7-2
Tunnel, Pipeline and Pump Station Permits

Agency	Permit	Notes
	Federal	
US Department of State	Presidential Permit for Border Crossing	• Required for border crossing if Mexico connection is incorporated into the project • Additional coordination required with General Services Administration and the Department of Homeland Security's Bureau of Customs and Border Protection as well as Federal Environmental Agencies listed in Table 7-1
	State (California)	
Caltrans	Encroachment Permits	• Required for all state highway crossings
State Water Resources Control Board, Regional Boards (San Diego District 9 and Colorado River District 7)	National Pollutant Discharge Elimination System Permit/Report of Waste Discharge Construction General Permit	• General Construction Permit would include Storm Water Pollution Prevention Plan (SWPPP), hydrostatic test water discharges and tunnel dewatering
SWRCB, Division of Water Rights	• Temporary Permit to Appropriate Water • Drinking Water Supply Permit for Public Water Systems	• Only required if diverting water to be used during construction. Contractor would likely obtain water for Tunnel boring machine cooling and hydrostatic test water from nearest water agency • Potential modification to existing Water Authority Permit
	Local	
County of San Diego	• Blasting and Traffic Control Permits • Stormwater Design	• Contractor to obtain permits as required • Stormwater Design includes Storm Water Quality Management Plan (SWQMP), Municipal Separate Storm Sewer System (MS4) Compliance, Best Management Practice (BMPs), etc. for the pump stations
Imperial County	• Blasting and Traffic Control Permits • Stormwater Design	• Contractor to obtain permits as required • Stormwater Design includes SWQMP, MS4 Compliance, BMPs, etc. for the pump stations
Air Pollution Control District (San Diego County)	Back-Up Engine Generator Permit	• Required for temporary (during construction) or permanent back up power
Air Pollution Control District (Imperial County)	Back-Up Engine Generator Permit	• Required for temporary (during construction) or permanent back up power during construction
Union Pacific Railroad	Encroachment Permit	• Required for railroad crossings

TABLE 7-3
Treatment Facility and Brine Management Pipeline Permits

Agency	Permit	Notes
	Federal	
U.S. Army Corps of Engineers	Consultation and application for wetlands/Waters of the U.S. – Individual or Nationwide Permit – CWA §404	Stream channel crossings including desert dry washes and brine conveyance to the Salton Sea
	State (California)	
State Water Resources Control Board, Regional Boards (San Diego District 9 and Colorado River District 7)	• National Pollutant Discharge Elimination System Permit/Report of Waste Discharge Project Specific Permit • National Pollutant Discharge Elimination System Permit/Report of Waste Discharge Construction General Permit	• NPDES permit would include brine conveyance to Salton Sea if this brine management alternative is selected • General Construction Permit would include SWPPP and hydrostatic test water discharges
SWRCB, Division of Drinking Water	Domestic Water Supply Permit	• Required for new drinking water treatment facilities • Water would be treated at Water Treatment Plants to meet the same water quality standards applicable to existing input water supplies
	Local	
County of San Diego (Planning & Development Services)	• Traffic Control Permit • Stormwater Design	• Contractor to obtain permit as required • Stormwater Design includes SWQMP, MS4 Compliance, BMPs, etc.
County of San Diego (DEH)	Onsite Wastewater Treatment System Permit	• Required for construction of onsite septic system for disposal of sewer waste • Permit only required from San Diego or Imperial County not both, pending final Facility location
County of San Diego Certified Unified Program Agency	Unified Program Facility Permit	• Required for chemical storage • Permit only required from San Diego or Imperial County not both, pending final Facility location
Imperial County (Planning & Development Services)	• Traffic Control Permit • Stormwater Design	• Contractor to obtain permit as required • Stormwater Design includes SWQMP, MS4 Compliance, BMPs, etc.
Imperial County (PHD)	Septic System Permit (Disposal of sewer to septic system)	• Required for construction of onsite septic system for disposal of sewer waste

TABLE 7-3
Treatment Facility and Brine Management Pipeline Permits

Agency	Permit	Notes
		• Permit only required from San Diego or Imperial County not both, pending final Facility location
Imperial County Certified Unified Program Agency	Unified Program Facility Permit (Chemical Storage)	• Required for chemical storage • Permit only required from San Diego or Imperial County not both, pending final Facility location
Air Pollution Control District (San Diego County)	Back-Up Engine Generator Permit	• Required for temporary (during construction) or permanent back up power during construction • Required from San Diego or Imperial County not both, pending final Facility location
Air Pollution Control District (Imperial County)	Back-Up Engine Generator Permit	• Required for temporary (during construction) or permanent back up power during construction • Required from San Diego or Imperial County not both, pending final Facility location
Union Pacific Railroad	Encroachment Permit	• Required for railroad crossings (if encountered for Brine Management Pipeline)
Fire Department	Chemical Storage	• Required for chemical storage
San Diego Gas & Electrical	Electrical Service	• Design coordination required for facility electrical service
Imperial Irrigation District	Electrical Service	• Design coordination required for facility electrical service, if located within district boundaries
Local City (Imperial County)	Potable Water Service	• Design coordination required for facility potable water service, if treatment plant located within City boundaries

7.4 Summary of Environmental Issues

Key environmental issues that are expected to be encountered for the project are identified below in Table 7-4. This list is preliminary and would be revisited in development of the final work plans following scoping, and throughout the study process in reaction to findings of the technical analyses and consultation processes.

TABLE 7-4
Summary of Environmental Issues

Resource	Potential Effects	Canal and Pipeline Segments And Tunnel Portals	Tunnel Segment	Power Lines to Pump Stations And RCS Treatment Facility	Pump Stations and RCS Treatment Facility
Aesthetics/Visual Quality	Vegetation removal; construction grading and excavation; night lighting (if needed), facilities design.	X	NA	X	X
Agriculture and Forestry	Potential for disturbance or loss of farmed lands in IID.	X	NA	NA	X
Air Quality & Greenhouse Gas Emissions	During Construction: dust control and vehicle track-out sediment; stabilization of storage piles and tunneling materials; emissions from construction equipment and vehicles. Operations: Quantification of GHG emissions for operation of the RSS compared to current use of the CRA.	X	X	X	X
Biological Resources	Habitat loss for canal and pipeline segments, tunnel portals and powerline alignments, including desert dry wash crossings and riparian lands and wetlands. Evaluation of potential habitat disturbance affecting special-status species. Evaluation of brine management and potential impacts or habitat benefits along the Salton Sea shoreline.	X	NA	X	X
Cultural Resources	Potential alteration or destruction of historic resources and/or Native American cultural resources, including artifacts, village sites and burial sites for development of the canal and pipeline segments and powerline pole footings, and at tunnel portals. The tunnel system is too deep to encounter cultural resources. The current alignment does not encroach on tribal lands. Evaluate potential impacts, if any, on the San Felipe Creek National Natural Landmark.	X	NA	X	X

TABLE 7-4
Summary of Environmental Issues

Resource	Potential Effects	Canal and Pipeline Segments And Tunnel Portals	Tunnel Segment	Power Lines to Pump Stations And RCS Treatment Facility	Pump Stations and RCS Treatment Facility
Tribal Cultural Resources	Required tribal consultation, and evaluation of potential impacts on significant tribal cultural resources including sacred places or objects.	X	NA	X	X
Energy	Evaluation of energy demands and sources for construction and operation, energy conservation opportunities, and the potential effects of the project on local and regional energy resources.	X	X	X	X
Environmental Justice	Assessment of whether the proposed Project has the potential to have any disproportionate adverse effects on economically disadvantaged communities.	X	X	X	X
Geology and Soils (*including geologic hazards and paleontology*)	Excavation and grading for canal, pipeline and tunnel segments. Seismic risks for tunneling through the Elsinore Fault Zone. Evaluation of rock integrity and stability for tunnel. Evaluation of potential for tunneling to intercept groundwater, and impact aquifers. Grading and excavation of undisturbed sedimentary rock has the potential to encounter significant fossils.	X	X	X	X
Hydrology and Water Quality	Effects on storm drainage, and erosion control measures to be used during construction, and post-construction in restored/backfilled areas, and flood hazards. Protection of water quality would be required throughout construction. Assessment of water quality must consider all potentially affected surface water sources, erosion and sediment control, and effects on underlying and overlying groundwater. Groundwater pumping that could be required for dewatering during construction must be evaluated for quality, and possible need for treatment prior to brine management. Evaluation of the desalination process and conveyance of brine water to the Salton Sea and shoreline.	X	X	X	X

TABLE 7-4
Summary of Environmental Issues

Resource	Potential Effects	Canal and Pipeline Segments And Tunnel Portals	Tunnel Segment	Power Lines to Pump Stations And RCS Treatment Facility	Pump Stations and RCS Treatment Facility
Land Use and Planning	Inventory of underlying and overlying land ownership, including Federal, State, County, and City lands, Tribal lands (if any), and private landowners. Identify any conflicts with established land use and resource management plans. Tunnel segments would not impact overlying land uses.	X	X	X	X
Mineral Resources	Although unlikely for the properties under consideration, the potential for disturbance or loss of important mineral resources must be considered. Potential recovery of mineral resources through use of the tunneling tailings should also be evaluated. It is also possible that the tunneling rock residual could be utilized for shoreline stabilization projects on the exposed playa at the Salton Sea.	X	X	NA	X
Noise and Vibration	Construction traffic noise and vibration could affect neighboring properties during construction, and for tunneling operations at the tunnel portals. Concrete batch plant operations during construction of the canal segments can also result in significant noise, particularly during more sensitive nighttime hours.	X	X	X	X
Population and Housing	The proposed project would not affect population or housing demands locally or regionally.	NA	NA	NA	NA
Public Safety and Hazards	Construction activities for large scale projects always present safety hazards for workers and nearby residents, including traffic conflicts, fire, spills, and collapse of excavations.	X	X	X	X
Public Services	Evaluate potential impacts on fire and police protection services, schools, parks and other public facilities.	X	X	X	X
Recreation	Potential conflicts during construction with existing recreational uses at the Ocotillo Wells State Vehicular Recreation Area needs to be considered.	X	NA	X	X

TABLE 7-4
Summary of Environmental Issues

Resource	Potential Effects	Canal and Pipeline Segments And Tunnel Portals	Tunnel Segment	Power Lines to Pump Stations And RCS Treatment Facility	Pump Stations and RCS Treatment Facility
Transportation	A Traffic Control Plan would be required to be implemented throughout the construction period and would need to account for current federal and State guidelines for traffic management, road safety, and uniform traffic devices.	X	X	X	X
Utilities and Service Systems	Project design would need to account for other existing utility systems, including easements, rights-of-way, and possible need to relocate conflicting infrastructure.	X	X	X	X
Wildfire	The project does include installation of new power lines in remote areas which are known to pose a potential fire hazard. The wildfire evaluation would consider the potential to exacerbate wildfire risks, or to impede any emergency response or evacuation plans.	X	NA	X	X

7.5 Environmental Review and Permitting Risks

Land acquisition would involve complicated regulatory and environmental permitting and ROW approval processes, but does appear to be entirely feasible with consultation, identification of mitigation offsets, and possibly alternative designs and/or minor adjustments to the conveyance alignment. Success would require a combination of carefully managed environmental studies, permitting and all related consultation and environmental review processes.

Acquiring lands and permanent easements that are under the jurisdiction of State and Federal government agencies and that have special-status designation (endangered species habitat, State and regional parks, a National Natural Landmark, designated habitat preserves and a federal military reservation) would be complicated.

Lands identified as particularly sensitive and therefore a risk factor for the Project are identified below.

1) The pipeline alignment parallel to the northern boundary of the eastern segment of the Highway 78 ROW appears to avoid encroachment into the San Felipe Creek National Natural Landmark under the jurisdiction of the National Park Service (NPS). Avoidance is the preferred strategy, and the pipeline ROW should avoid any encroachment on the Landmark. NPS consultation should be undertaken at an early stage in the Project to verify the relationship of the Project alignment and the Landmark property. There is potential for the proposed alignment to impact designated critical habitats for California

gnatcatcher (Polioptila californica; CAGN), and Peninsular bighorn sheep (Ovis canadensis; PBHS). If habitat could not be avoided, design features would need to be implemented that avoid impacts on applicable Primary Constituent Elements of designated critical habitat throughout the alignment.

2) The tunnel segments ROW under Anza-Borrego State Park and Cleveland National Forest are located at considerable depth and would be in dense bedrock but would still be linked to groundwater or other resources within the overlying Park/Forest. Early consultation with the State Department of Parks and Recreation, the Anza-Borrego State Park Superintendent and the National Forest Service is recommended to verify this understanding of the relationship between the tunnel and the overlying Park/Forest lands.

3) A short segment of the canal ROW in the Imperial Valley parallel to IID's existing Westside Main Canal crosses the southeastern corner of the U.S. Naval Parachute Drop Zone (as does the existing IID canal). Early consultation with the U.S. Navy is essential to determine their willingness to grant a ROW encroaching on their land, and any potential impacts to the property or function of the military reservation.

4) Particularly for the segments of the tunnel through Lake Henshaw and Valley Center, the tunnel system could pose potential impacts to the overlying aquifer(s) that could be intercepted by tunneling. The geologic impact assessment would include evaluation of hydrogeology and potential groundwater impacts and would identify design features and mitigation measures that could be included in the Project to assure local groundwater users that their resource and wells would not be adversely impacted.

5) The long lead time for development of the RCS poses a risk that other land development projects could occur in the interim, causing a need for alignment adjustments, interfering with planned locations of tunnel portal or other project features, or in a worst-case, rendering a segment of the alignment infeasible.

7.6 Incidental Environmental Benefits

There are several environmental benefits that could result from the RCS Project. These would be assessed further as a part of the environmental review documentation, and include:

1) The Colorado River reach between Parker Dam and Imperial Dam would experience higher flows with conveyance of 279,500 acre-feet annually to the changed point of diversion at Imperial Dam for transport through the new RCS, rather than being diverted at Parker Dam for conveyance via the Colorado River Aqueduct as occurs at present. Higher flows would benefit habitat and water quality conditions in that river segment.

2) The proposed RCS Treatment Facility near the Salton Sea would produce a brine byproduct of an estimated 20 million gallons per day, with estimated salinity between 5,600 to 11,000 TDS. TDS in the Salton Sea is now approaching 60,000 TDS. That brine byproduct , totaling approximately 22,500 acre-feet per year, could be used in a Salton Sea Shoreline habitat restoration project similar to the Species Conservation Habitat project now in early implementation stages around the New River delta area on the

south shore of the sea, and could offer a relatively high quality habitat conditions supporting tilapia, desert pupfish, and multiple species of resident and migratory birds.

3) As the Salton Sea becomes smaller with the change in agricultural drainage inflow, the shoreline is predicted to stabilize with an exposed playa of up to 75,000 acres. There are numerous projects and studies being undertaken to stabilize the exposed playa to minimize windblown dust and PM10 particulate matter attributable to the exposed shoreline. The residual rock materials from the tunneling should be substantial and presents a question regarding suitable disposal sites. The potential for this material to be utilized for exposed playa shoreline stabilization presents a possible solution with benefits to the RCS and the Salton Sea and surrounding communities.

7.7 Schedule and Budget Estimates

Table 7-5 sets out a sequence of tasks to be undertaken with assumed time required for completion of concurrent sets of tasks and the total time needed to complete environmental review and subsequent permitting. The feasibility level estimate is approximately 5 to 7 years to complete the environmental and permitting process.

TABLE 7-5
Task Sequence and Timeline / Schedule

Timeframe	Tasks
Day 0-180 *(6 months)*	• Agency consultation regarding scope and access to lands for biological and cultural surveys for all canal and pipeline segments, tunnel portals, and powerline alignments to pump stations • Project Description, including all project components, staging areas, construction footprint and final Project footprint • Develop a reasonable range of Project Alternatives for assessment in the Program EIR • Prepare Notice of Preparation and schedule scoping meetings

Day 180-730
(18 months)
	• Conduct general biological surveys – all project lands, transmission line routes (estimated one year)
	• Conduct desktop cultural surveys – all project lands, transmission line routes. (Estimated six months with ongoing tribal consultation)
	• Conduct geological and geotechnical assessments
	• Conduct all related technical assessments to support Program EIR primary resource impact analyses, including all topics listed in Table 7-4 above.
	• Program EIR
	    • Initiate Tribal Consultation
	    • Prepare Scoping Summary
	    • Prepare Draft Program EIR
	    • Complete public review draft Program EIR
	    • 45-day comment period and Responses to Comments and Final Program EIR, including Mitigation Monitoring and Reporting Plan
	    • Prepare CEQA Findings of Fact
	    • Water Authority Board Selection of Preferred Project for subsequent project-specific environmental review and permitting

Day 730-1,460 *(24 months)*	• Project Specific EIR/EIS • Consultation between Water Authority and the Bureau of Reclamation and development of a MOA for preparation of a joint CEQA and NEPA document • Prepare Notice of Preparation and Federal Register Notice of Intent and schedule scoping meetings • Initiate Tribal Consultation • Prepare Scoping Summary • Conduct protocol level biological and cultural surveys, and site specific geological and

TABLE 7-5
Task Sequence and Timeline / Schedule

	geotechnical investigations • Prepare Draft EIR/EIS • Prepare Draft Biological Assessment for USFWS and CDFW Endangered Species Act consultation • Prepare Draft Cultural Resources Assessment for SHPO and Tribal Consultation • Complete Biological Opinion (USFWS) and Consistency Determination (CDFW) • Complete Cultural Resources Protection Plan and Historic Properties Management Plan • Complete public review draft EIR/EIS
Day 1,460-1,825 *(12 months)*	• 60-day comment period and Responses to Comments and Final EIR/EIS, including CEQA Mitigation Monitoring and Reporting Plan • Prepare CEQA Findings of Fact, and NEPA Record of Decision • Water Authority Board Project Approval • Draft and Final Permit Applications
Day 1825-2555 *(24 months)*	• Permitting Coordination and Completion • Land Acquisition (ROW, easement and/or purchase)

Like the schedule, the total budget required is equally difficult to accurately determine this far in advance of any scoping or coordination with key agencies. For feasibility-level planning purposes, estimated costs are between $8,000,000 and $12,000,000. This estimate includes environmental permitting, and other than preliminary design information required for the EIS/EIR. The estimate does not include design, legal services, or public outreach. Refined estimates at the task-level could be developed as the Project Description is refined and after engagement in scoping with the relevant agencies to define the extent of field investigations, resource management study details, and mitigation planning.

7.8 Conclusion

The following next steps are recommended to continue to advance the environmental process.

Pre-consultation with Key Agencies. High level meetings with materials such as presentations, maps, and brief Project Descriptions that define the purpose and need for the project to convey what is being considered, planned next steps, and what is needed from stakeholders. These meetings will inform the Water Authority what is needed from the project team.

- Federal and State Legislators – Brief federal and state governments on the RCS.
- Water Authority Member Agencies - The Water Authority would continue coordination with member agencies on potential use of or interconnection to facilities.
- IID – The Water Authority has already started coordination.
- Bureau of Reclamation – Prior to initiating any formal environmental review process, it would be beneficial to coordinate with the BOR. BOR would serve as the Federal Lead Agency.
- State Park - The tunnel beneath Anza Borrego State Park is only feasible if the Park Superintendent (or higher in State government) gives approval.
- National Park Service and U.S. Navy - The alignment either crosses or is adjacent to National Park Service managed lands (San Felipe Creek National Natural Landmark),

and the U.S. Navy parachute training zone – Coordinating with these agencies early would be beneficial.

- Cleveland National Forest - Coordinating early would be beneficial.

Biological Baseline Development. Baseline biological and cultural surveys to identify potential conflicts with specially designated habitat areas, sacred tribal lands, and potential lands to acquire for mitigation. Due to the length of time over which environmental review would extend, getting a first year of biological baseline data would also be valuable, particularly if data could be collected in a wet year.

Initial Geotechnical Evaluation. Performing borings to obtain geotechnical information along the tunnel alignment would be informative to understand the potential to intercept fault lines and avoid aquifers.

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8

PARTNERSHIP AND FUNDING OPPORTUNITIES

Chapter 8.0 Partnership and Funding Opportunities

8.1 Introduction

8.1.1 Overview

This chapter describes partnership opportunities that could be considered for the RCS. Since this project evaluation is in the feasibility phase, the partnerships identified and described in this chapter are based on a cursory review of agencies, governments entities, and other organizations that may share common interests in water supply, environmental, resiliency, and other regional benefits. These partnerships were identified through collaboration between the Water Authority and Black & Veatch. Consideration was given to compatible partnership projects that could provide mutual benefits for both the Water Authority and the partner. In addition, regional partnerships that aligned with the Governor's Water Resilience Portfolio were evaluated including: funding and infrastructure, groundwater replenishment, storage, reuse, and water use efficiency. Energy storage and production opportunities were also reviewed. Note that Phase A baseline cost estimates assumes no partnerships and associated benefits or funding due to the preliminary nature of this review and limited dialogue with potential partners.

Because this study includes three alternative alignments, some partnerships may not apply to all alternatives. Therefore, each partnership is linked to either an individual alternatives or multiple alternatives.

If this study moves to Phase B, more detailed explorations of partnerships would be performed in collaboration with the respective potential partners and stakeholders.

8.1.2 Chapter Organization

This chapter first outlines overall strategies that were used to identify potential partnerships and then describes each potential project partnership by location along with specific considerations, advantages, challenges, and benefits to each partner.

8.2 Partnership Opportunities

8.2.1 General Approach

Past studies of the RCS looked purely at alternative conveyance of the Water Authority's QSA supplies or a "single-use" concept. This study builds and expands upon past work to evaluate the potential for multi-use projects, partnerships, and funding to provide regional benefits and reduce cost and risk to the Water Authority and its member agencies. There are several potential multi-use opportunities along each of the three alignments under study that appear to be compatible with the objectives of the RCS that could help address regional issues. These potential multi-use projects include operational storage, renewable energy storage and production, binational projects, and habitat restoration. Potential partners include local, state and federal agencies and organizations, Native American tribes, Mexico, and private entities. Water Authority staff and Black & Veatch engaged in a focused workshop to discuss the partnership opportunities identified to date.

8.2.2 Project Benefits to Potential Partners

The RCS provides unique project elements that have the potential to benefit other organizations in addition to conveying the Water Authority's QSA water to San Diego County. Key project elements that could benefit other agencies that were considered include:

- The conveyance system (canals, pump stations and pipelines) to move water from the All-American Canal to San Diego County
- Brine generated from a treatment facility
- Storage facilities needed for operation of treatment facilities
- Energy demand to power the treatment facilities and pump station (could engage an energy partner)
- Energy recovery where available from sufficient pressure (head)
- Energy storage through pumped storage facilities that could be added to the project

8.2.3 Partnership Timing

All partnerships considered would need to work within the timeline of the project. Since the RCS is targeted to be on-line by 2045, partnerships would need to be viable in that time frame.

8.2.4 Potential Partnerships

The following sections describe the potential partnerships that were explored thus far. Note that this list is not exhaustive and other partnership opportunities could emerge. In some cases, there could be opportunities to provide recreational benefits as part of the RCS. In nearly every case, the RCS would provide positive economic impacts by creating both short-term and long-term jobs.

8.2.4.1 Imperial Irrigation District

The IID operates a vast series of canals in which water is supplied from the Colorado River via the All-American Canal. Farmers comprise the largest customer base for IID, most of which receive water deliveries over a 24-hour period or a 12-hour period. Flood irrigation techniques have historically been used by farmers but under the QSA and as part of IID's conservation programs, an increasing number of farmers are utilizing sprinkler irrigation and pumpback systems, among other measures, for conservation. Providing deliveries to farmers in shorter than 12-hour timeframes such as 8-hour timeframes, could provide operational flexibility and support conservation efforts. Storage facilities located in strategic locations would enable IID to refine delivery times to their customers which would in turn drive even more efficient irrigation practices and conserve water.

Because storage would be needed to provide a consistent flowrate to the RCS treatment plant, both the RCS and IID could benefit from a shared storage reservoir in Imperial Valley. For the RCS, the reservoir would be located on the west side of the Westside Main Canal (WSM) and could be placed near the Fox Glove Check, where sufficient elevation exists to optimize the gravity flow into and out of the reservoir for the IID operation. This storage strategy could also give IID more flexibility in how they operate the All-American Canal (AAC) from the New River Siphon to the west terminus of the AAC as well as WSM. By optimizing the operation of the AAC, capacity could be made available for Water Authority use, thus not requiring the construction of a parallel canal from the New River Siphon to the terminus of the AAC. This is discussed in detail in *Chapter 2.0 - Regional Conveyance System Operations and Sizing.*

8.2.4.2 Borrego Springs

Borrego Springs is located along the Alternative 3A alignment. The Borrego Springs community relies on groundwater to supply water to the area. Because the groundwater basin is critically over-drafted, mandatory reductions in groundwater use will greatly impact the community's current water use. A Groundwater Sustainability Plan (GSP) is currently under development for the Borrego Groundwater Basin to reduce drafting by 70% by year 2040 (19,100 AF to 5,700 AF).

Because Alignment 3A passes through this community, there is a potential partnership between the Water Authority and offtakers in Borrego Springs. It could be possible to upsize the RCS (*see Chapter 3.0 - Aqueduct Operations and Integration of the RCS)* slightly to provide a conduit to Borrego Springs to convey water for storage in the groundwater basin, use the water directly for either non-potable uses, or to send to new treatment facilities.

The benefit to Borrego Springs is primarily a cost-effective way to convey water supplies provided by the potential partners to that community. The Water Authority and Borrego Springs could benefit by attracting funding since Borrego Springs is a disadvantaged community. Previous legislative efforts to fund a similar project through federal funding were ultimately not successful but demonstrate legislators' desire to support Borrego Springs.

Because the water transmitted through the RCS would likely go through treatment prior to conveying to San Diego County, part of the partnership would include treatment of the Borrego Springs water since it would share the same pipeline. Treatment would include treatment costs, water loss (through the treatment process), and brine management.

In addition to the partnership with the Water Authority, Borrego Springs or other offtaker(s) would also need to partner with another agency to secure the water supply to transmit through the RCS. The Water Authority, or others, could also explore with Borrego Springs, the potential to temporarily store water in the large groundwater basin and then convey that water when needed. This strategy would optimize the use of the RCS infrastructure to provide this shared benefit.

8.2.4.3 Salton Sea

The Salton Sea is a uniquely challenged water body with a long history of efforts to improve the sea's water quality and the surrounding environment. Environmental Groups (Non-Government Organizations) and the State government share a desire to improve the existing Salton Sea environmental conditions. The State has already begun investing in the Salton Sea. In addition, NGO's may begin fundraising for additional environmental mitigation.

California State's Salton Sea Management Program (SSMP) is led by the California Natural Resources Agency (CNRA). The State's SSMP is a phased approach to Salton Sea restoration. Under phase one of the SSMP, the State is to address 30,000 acres of exposed playa (dry shoreline of the Salto Sea due to evaporation and supply losses) over a 10-year period with a mix of habitat and air quality projects. Annual milestones in acreage coverage are to be met under an updated water rights order (WRO 2017-0134) approved in 2017 by the State Water Board based on guidelines outlined by the Water Authority, IID, Imperial County, environmental organizations, and the State.

This program is currently behind schedule but represents the State's commitment to investing in programs that address the environmental challenges related to the Salton Sea.

Because the RCS would include treatment facilities to reduce the salt concentration in the Colorado River water, a new water supply that could benefit the Salton Sea would be generated if the project moves forward.

The average salt concentration in the Colorado River is 570 mg/L, with a maximum of 879 mg/L. Through the treatment process, that concentration would be reduced to about 500 mg/L to match the typical background salt concentration in existing water supplies in San Diego County. The treatment process would generate brine concentrations of approximately 3,500 mg/L to 11,000 mg/L depending on the recovery rate of the chosen treatment system. The anticipated brine management flow rate is approximately 22,500 AFY. While brine concentrations at this range are not attractive to typical streams and groundwater basins, this concentration is significantly lower than the Salton Sea, with a salt concentration having been as high as 44,000 mg/L in the early 1990's. The current Water Quality Control Plan for Region 7 of the Colorado River Basin has set a goal to stabilize the salinity to 35,000 mg/L. Sending the brine stream from the treatment facilities to the Salton Sea would have a net benefit to the sea's concentration. In addition, the brine could be used to create new habitat on the playa surrounding the Salton Sea prior to introducing it to the main sea body. The habitat would receive the brine at a near constant inflow to constructed wetlands that would support plant and aquatic life and possibly create recreational uses. The wetlands would be constructed in areas of the playa that are currently dry, thereby reducing the area subject to wind scouring, resulting in improved air quality in the area. This strategy could also improve bio-diversity within the region and support erosion control efforts. *Chapter 4.0 – Treatment, Blending and Brine Management Options* provides more details on the brine management options related to the Salton Sea.

This could also be a benefit to the Federal government, which is a significant landowner within the Salton Sea. The Federal government has an existing Memorandum of

Understanding (MOU) with the State on Salton Sea issues, but there has been little action taken in relation to the MOU.

8.2.4.4 New River

The New River is largely supplied with flows that come across the border from Mexico. This river has significant water quality issues that are well documented and primarily a function of wastewater. Currently, there are strategies in place, such as the *Strategic Plan: New River Improvement Project*, to improve the water quality within the New River.

The basin plan in the New River area regulates the salt concentration that could be introduced to the river or surrounding unlined canals. That concentration limit is 4,000 mg/L, which is lower than the brine concentration coming from the RCS treatment plant, thereby eliminating the possibility to send brine directly to the New River.

A possible win-win solution could be to utilize the brine flow in conjunction with the treatment strategies envisioned in the *Strategic Plan: New River Improvement Project*. As part of this strategic plan, a treatment plant would be built to treat New River flows from Mexico. The salt concentration in the New River is relatively modest, well below the 4,000 mg/L limit defined in the basin plan. However, the New River water quality is high in many other parameters that the brine would not be. Mixing the effluent from the proposed New River treatment plant with the brine would reduce the salt concentration from the brine to below 4,000 mg/L and reduce the concentration of other harmful constituents in the treatment plant effluent (such as virus, pathogens, and coliform). The mixed water would have a net result that would be even better water quality in the New River and meet the discharge requirements, allowing the water to be sent to the New River. *Chapter 4.0 - Treatment, Blending, and Brine Management Options* provides more details on the brine management option related to the New River.

8.2.4.5 Member Agencies

Several member agencies (MA) asked that local operational flexibility and storage be evaluated as part of this study. Many of the MAs have existing storage facilities that are near one or more of the RCS alternative alignments. In some cases, it could be useful to the MA to be able to connect directly to the RCS to put water into or withdraw out of their existing reservoir. This strategy was reviewed at a cursory level, for all the reservoirs in San Diego County as shown on Figure 8-1. The circles indicated on Figure 8-1 are associated with the information presented in Table 8-1, which summarizes the potential partnership opportunity for each reservoir.

It should be noted that the partnership opportunities associated with Turner Reservoir, Lake Wohlford and San Vicente Reservoir are further defined in *Chapter 3 – Aqueduct Operations and Integration of the RCS.*

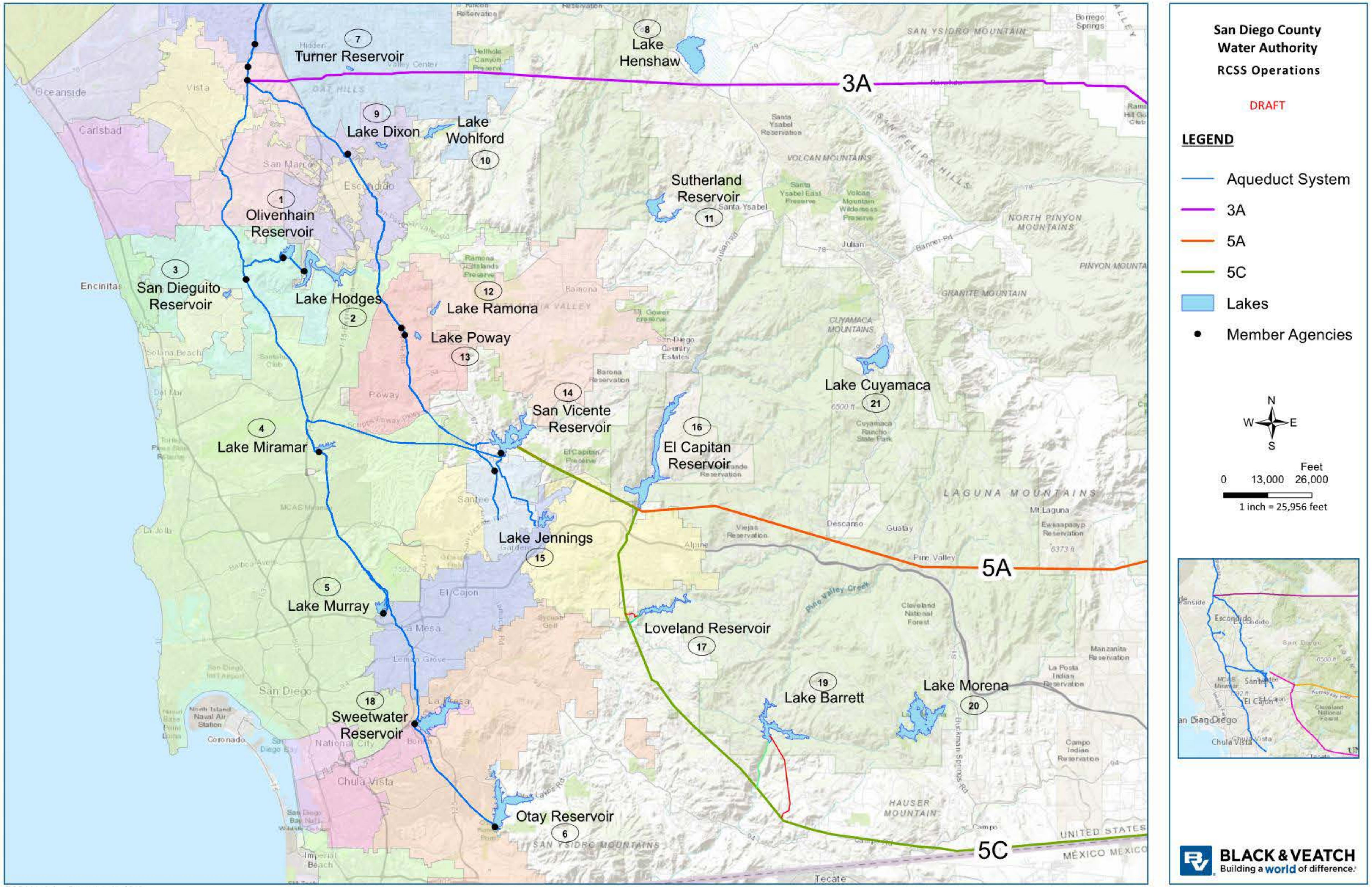

FIGURE 8-1
Member Agency Reservoir Map

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TABLE 8-1
Potential Member Agency Storage Partnerships

Reservoir ID	Name	Owner	Capacity (AF)	Elevation (ft)	New Partnership Opportunity	Partnership Benefit
1	Olivenhain Reservoir	Water Authority	24,700	1,080	None identified	Existing ESP Facility
2	Lake Hodges	City of San Diego	30,250	315	None identified	Existing ESP and Pumped Storage Facility
3	San Dieguito Reservoir	(Joint) Santa Fe Irrigation District & San Dieguito Irrigation District	883	250	None identified	NA
4	Lake Miramar	City of San Diego	7,180	714	None identified	NA
5	Lake Murray	City of San Diego	4,820	536	None identified	NA
6	Otay Reservoir (Upper and Lower)	City of San Diego	49,500	485	None identified	NA
7	Turner Reservoir	Valley Center MWD	1,730	1,071	Potential shared storage for Water Authority providing aqueduct operational flexibility	Improved recreational use
8	Lake Henshaw	Vista Irrigation District	53,400	2,690	Potential partnership for energy storage	Revenue
9	Lake Dixon	City of Escondido	2,610	1,043	None identified	NA
10	Lake Wohlford	City of Escondido	2,800 – 6,500, depending condition of the dam	1,460 – 1,480 depending on the condition of the dam	Potential shared storage for Water Authority providing aqueduct operational flexibility	Improved recreational use
11	Sutherland Reservoir	City of San Diego	29,700	2,057	None identified	NA
12	Lake Ramona	Ramona MWD	12,000	1,341	None identified	NA
13	Lake Poway	City of Poway	3,320	938	None identified	NA
14	San Vicente Reservoir	City of San Diego	249,350	766	Existing Partnership	Existing Partnership
15	Lake Jennings	Helix Water District	9,790	700	None identified	NA
16	El Capitan Reservoir	City of San Diego	113,000	750	Potential regional storage	Additional supplies to improve operation flexibility
17	Loveland Reservoir	Sweetwater Authority	25,400	1,355	Potential regional storage	Connection to and from the Water Authority's aqueduct system to convey flows between Sweetwater Authority's Loveland and Sweetwater reservoirs[1]
18	Sweetwater Reservoir	Sweetwater Authority	27,700	237	None identified	NA
19	Barrett Lake	City of San Diego	37,900	1,607	Potential regional storage	Additional supplies to improve operation flexibility[2]
20	Lake Morena	City of San Diego	50,200	3,039	None identified	NA
21	Lake Cuyamaca	Helix Water District	8,190	4,635	None identified	NA

Notes:

1. Requires transferring water between RCS Alignment 5C and Loveland reservoir. Based on preliminary analysis, this option would require several miles of additional pipeline and tunnel construction.
2. Requires transferring water between RCS Alignment 5C and Barrett reservoir. Based on preliminary analysis, this option would require 5 to 10 miles of additional pipeline and tunnel construction.

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8.2.4.6 Energy

Projects with significant energy demands and potential for energy recovery often present opportunities for partnerships with energy providers, private investors, and utilities. The identified study alternatives would have a treatment plant and at least two large pump stations. These facilities would have significant energy demands, making them attractive to energy providers to enter into long-term power purchase agreements and public private partnerships (P3s) to create cost certainty for the facility owners and revenue streams to the energy provider.

Renewable Energy

The Imperial Valley is the home of many renewable energy facilities, including wind, solar, and geothermal power plants, as indicated on Figure 8-2. Many of these generating facilities are near the RCS energy needs. Partnerships could be created if the project proceeds to enter into power purchase agreements with existing plant owners or with owners of new facilities that could be built through P3s to optimally meet the energy needs.

In addition, a potential ancillary benefit would be the covering of exposed playa along the Salton Sea shoreline by the energy developer (wind, solar, geothermal). This would be accomplished with the development of these facilities along the shoreline, thereby reducing the exposed playa. These partnerships would be further explored if the project proceeds to future phases.

Energy Recovery

For Alternatives 5A and 5C, there are opportunities for energy recovery due to their unique topography. In each case, an in-line hydro-electric facility similar to the Water Authority's Rancho Penasquitos Pressure Control and Hydroelectric Facility would likely be cost effective and produce renewable energy. See *Chapter 2.0 - Regional Conveyance System Operations and Sizing*, Figure 2-1, for location of these facilities along Alternatives 5A and 5C.

Also based on discussions with IID staff, the additional flow required to deliver the QSA water through IID existing facilities could allow for the additional creation of power through IID's existing hydroelectric facilities on AAC. This opportunity would be further explored if the project proceeds to future phases.

Energy Storage

A cursory review of potential pumped energy storage was completed as part of Phase A. With the renewable mandates and goals within California, energy storage is now an essential part of the state's energy portfolio. To date, most energy storage has been contracted with battery developers. Moving forward, pumped storage will likely play a greater role to meet the large energy storage demands on the horizon. The Water Authority is very knowledgeable of energy storage through their two local projects, the Lake Hodges Pumped Storage project and the proposed San Vicente Energy Storage Facility with the City of San Diego. Alternative 3A is the only alternative where a potential energy storage facility was identified. For Alternative 3A, a pumped storage facility connected to Lake Henshaw could be further examined. This facility would be constructed in a cavern approximately 1,500 feet below Lake Henshaw and make use of the vent shaft that would be needed for construction of the tunnel section below the lake. New pumps/turbines would be constructed to send water up to Lake Henshaw when energy needs to be stored. When energy would be needed, water would be withdrawn from Lake Henshaw to generate energy. The water would be pumped from the tunnel pipeline up to Lake Henshaw and then return to the tunnel pipeline after generating energy for delivery to San Diego County. These types of pumped storage facilities are less common than the type used at Lake Hodges and envisioned for San Vicente but are used throughout the world. A pumped storage facility of this type appears to be technically feasible based on the information collected at the time of this study.

The commercial market for energy storage is emerging and not well defined. At this time, it would not be possible to determine the economic feasibility since the project would not be on-line until 2045 and energy storage is dependent on many factors including regulations, energy storage mandates, and the commercial market. If this project proceeds to future phases, energy storage should be further evaluated to determine the viability of incorporating it into this program. Furthermore, the Water Authority would engage with Vista Irrigation District, the owner of Lake Henshaw, should this option be explored in potential future phases.

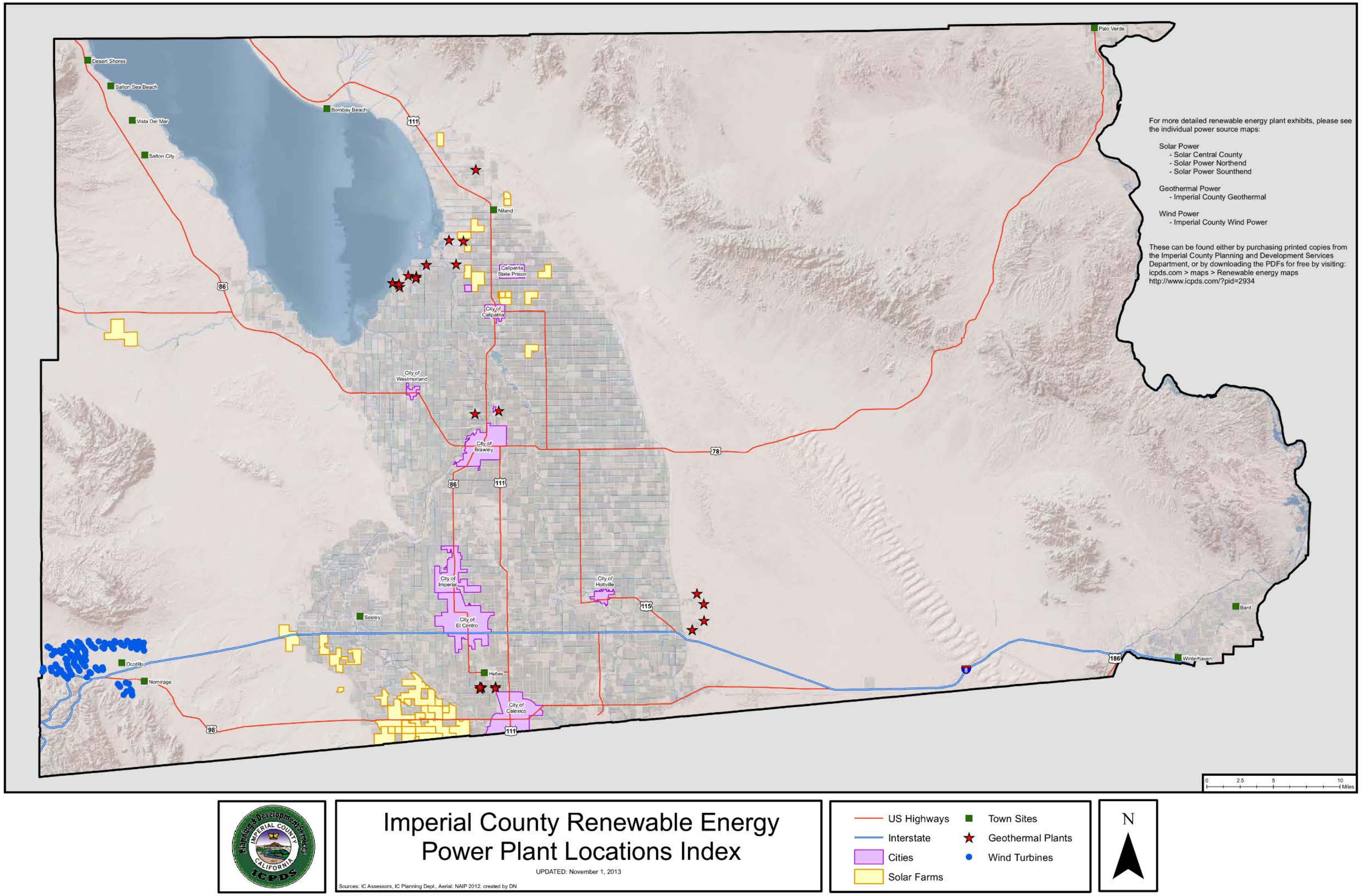

FIGURE 8-2
Imperial County Renewable Energy Power Plant Locations

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8.2.4.7 Bi-National

Due to the proximity of the RCS to the border, a partnership could be envisioned to create a connection between the United States (US) and Mexico for emergency supplies, a concept that is currently under discussion between the two countries. If implemented, the RCS would have facilities near the US-Mexico border at the west terminus of the AAC. Emergency connections between the AAC and Mexico have been explored in the past to provide mutual benefits to mitigate catastrophic events such as earthquakes. In the event of damage to Mexico's water infrastructure, water could be sent from the AAC to Mexico. In the event of there being a two-way connection, conversely, water could be sent from Mexico's aqueduct system to the US if there were damaged to the AAC.

8.2.4.8 Native American

Each of the RCS alternatives pass through or nearby Native American tribal lands. Should any of the tribes be interested in becoming a member agency to the Water Authority, connections to their respective service areas could be provided off the RCS. Figures 8-3 and 8-4 show the RCS Alternatives and the neighboring tribal lands.

8.2.4.9 State

The Governor's Water Resilience Portfolio directs the California Natural Resources Agency, California Environmental Protection Agency and the California Department of Food and Agriculture to identify and assess a suite of complementary actions to ensure safe and resilient water supplies, flood protection and healthy waterways for the State's communities, economy and environment. The RCS provides a conduit to convey water directly from the AAC to the San Diego region that avoids the San Andreas Fault, the largest active fault in California. While other major water supplies from the State Water Project and the Colorado River do pass through or more closely to the San Andreas Fault, the RCS provides an alternative to that specific risk. However, the alignment of the RCS would cross other existing faults (e.g. San Jacinto and Elsinore) and would need to be designed to limit potential damage associated with these fault crossings.

In the event that a major earthquake along the San Andreas Fault disrupted flows into Southern California, the RCS could provide a means to continue to deliver water to San Diego County and potentially other water agencies in Southern California, while repairs were being made.

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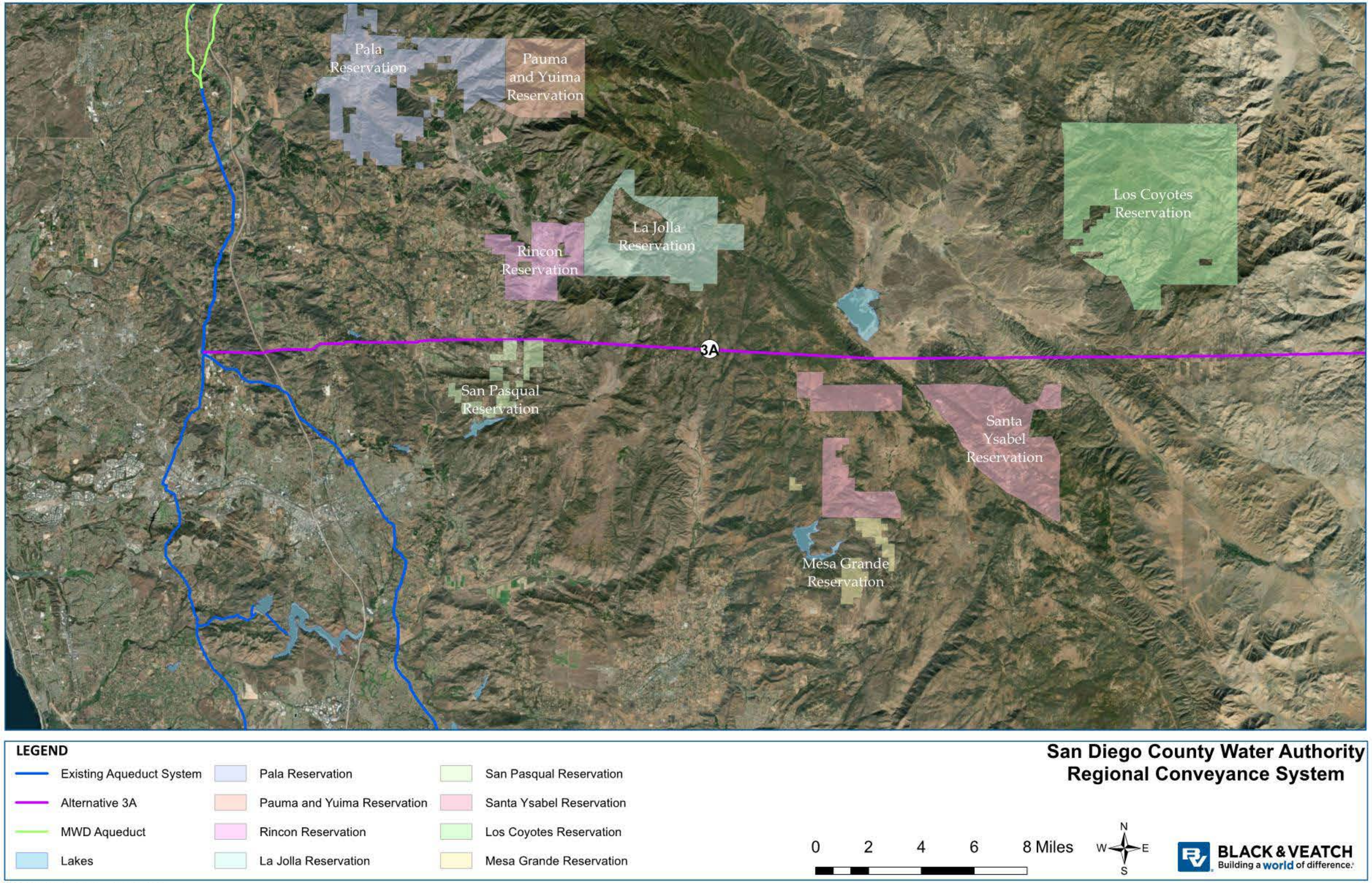

FIGURE 8-3
Location of Native Americans Lands Neighboring the RCS Alternative 3A

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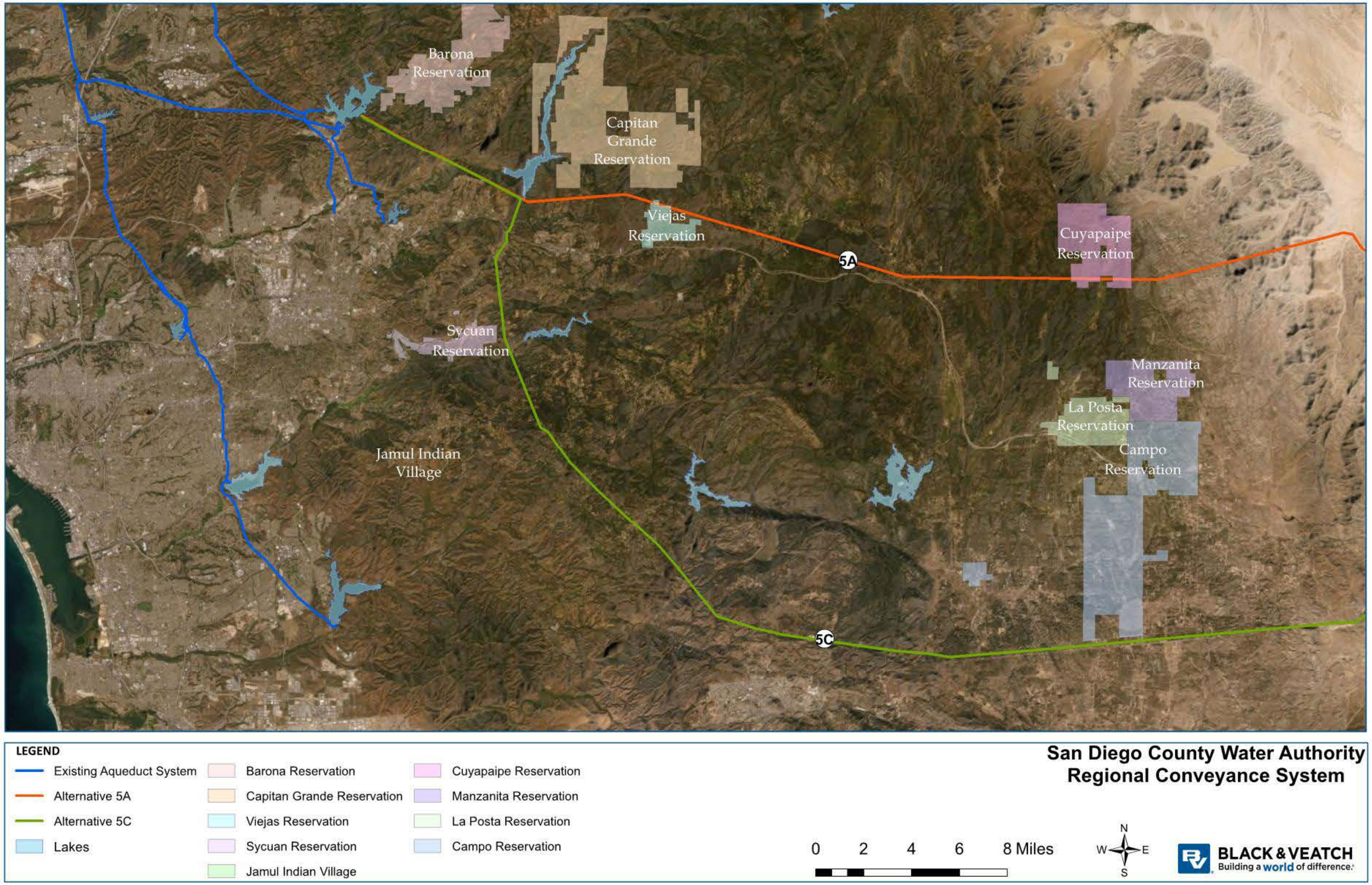

FIGURE 8-4
Location of Native Americans Lands Neighboring the RCS Alternatives 5A and 5C

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8.2.4.10 Public Private Partnerships

P3sprovide strategic advantages related to large municipal projects. Two key advantages are risk transfer and financing, where the private partner would assume much of the project risk and would likely finance the project, putting the debt on their balance sheet. In return the public entity would enter into a long-term contract to pay for commodity or service. This long-term contract would include the private partners return on the investment, which could add cost to the overall project, but is offset, at least partially, by the cost associated with risk transfer. The Water Authority has experience with these types of arrangements having collaborated with Poseidon Water to develop the Carlsbad Desalination Plant and is currently working with Brookfield Renewables to develop the San Vicente Energy Storage Facility.

Because of the diversity of types of facilities that make up the RCS, it is unlikely that a single entity would develop the entire RCS as in a P3 relationship. However, a group of developers could team up to provide the entire system. If the project moves to the next phase, it may be desirable to test the private market and collect input on the best strategies to advance a P3.

There are several pieces of the overall system that would likely be candidates for P3s, including the treatment plant, the power for the facilities, the energy recovery facilities, and potentially some of the individual conveyance systems. The pipelines, pump stations, and treatment plant could be developed with a "rent to own" relationship with a private entity. In this case, the private partner would develop the projects and charge the Water Authority a fee for a set number of years. At some point in the future, the ownership of the facilities would change from the developer to the Water Authority, which would then own and operate the facilities. These water facilities would likely be very prescriptive with the Water Authority providing preliminary design requirements to the developers. The energy facilities would likely be implemented very differently. The Water Authority could issue Requests for Offers (ROFs) to provide energy. Private energy developers would compete to provide energy facilities at the lowest price without dictating the exact energy generation details.

8.3 Funding Opportunities

Many of the partnerships identified include funding opportunities. Table 8-2 below provides a summary of funding sources that may be available for the applicable partnerships.

TABLE 8-2
Summary of Potential Funding Sources

Partnership	Funding Source	Driver
Borrego Springs	Local, State, and Federal Governments	Borrego Springs is a disadvantaged community with significant water supply challenges
Salton Sea	State and Federal Governments	The Salton Sea is an impaired water body that could realize water quality, air quality, and habitat restoration benefits through strategic management of brine from the treatment plant

TABLE 8-2
Summary of Potential Funding Sources

Partnership	Funding Source	Driver
New River	Local, State, and Federal Governments	The New River has water quality challenges that could be improved by strategic blending with brine from the treatment plant
Energy Storage and Production	Private Developers and Energy Providers	Energy recovery and energy storage opportunities utilizing RCS facilities could generate income streams through P3s and power purchase agreements
Binational	US Federal Government	Provides an emergency connection to Mexico from the AAC as part of the RCS and would support ongoing discussions between the countries
Native American Tribes	Federal Funding	Provides needed, reliable water supplies to disadvantaged communities
P3	Public Private Partnership	Opportunity to transfer risk (i.e. - construction, financing rates, operation) from taxpayers to private sector

8.4 Conclusion

Table 8-3 provides a high-level summary of the potential partnerships, potential benefits to each partner, and funding opportunities. The preliminary partnerships developed are cursory and need to be coordinated with the identified partners if the project moves to future phases. Note that Phase A baseline cost estimates assume no partnerships and associated benefits and funding due to the preliminary nature of this review and limited dialogue with potential partners. If this project moves to Phase B, it is envisioned that the Water Authority would meet with potential partners to explore opportunities at a deeper level. Partnerships that provide regional benefits could attract local, state, and federal grant funding for the project. Where the RCS solves challenges related to the environment, disadvantaged communities, and water resiliency, the likelihood to attract funding increases. In addition, opportunities associated with P3s would be further evaluated because such a partnership structure allows for the transfer of risk connected with the development and long-term operations of the RCS.

TABLE 8-3
Potential Partnership and Funding Opportunities

Agency	Issues	Solution Provided by RCS	Timing	Water Authority/Member Agency Benefit	Partner Benefit	Funding Source
Imperial Irrigation District						
	• Storage/water conservation	• Construction of new joint operational forebay/reservoir facilitates water conservation with farmers and frees capacity in the AAC from the Siphon to west terminus	• With implementation of RCS program	• Forebay for new RCS treatment facility • Flow balancing by IID allows for RCS capacity within AAC and potentially limits need and cost for parallel gravity system	• Delivery flexibility to match farmers schedules and improve irrigation techniques to save water • Terminal storage on west side of service area creates system wide flexibility	• None identified at this time
Borrego Springs						
	• Mandatory water restrictions due to groundwater basin depletion	• Provide infrastructure for delivery of water to community • Potential to recharge the groundwater basin	• With implementation of RCS program	• Possible seasonal storage in groundwater basin • Delivery flexibility • Brings funding opportunities	• New water supply • Recharge groundwater basin • Positive community impact (economic / environmental)	• Federal • State • Local
Salton Sea						
	• Habitat restoration (Exposed Playa)	• Provides water for Salton Sea Management Plan	• With implementation of RCS program	• Cost effective brine management alternative • Brings funding opportunities	• Helps meet State restoration efforts and long-term plan	• Federal • State
	• Water quality improvements	• Brine management provides better water quality to Salton Sea	• With implementation of RCS program	• Provides cost effective management alternative	• Better water quality in Salton Sea	• Federal • State
New River						
	• Habitat restoration	• Provide water for Habitat Restoration	• With implementation of RCS program	• Cost effective brine management alternative • Brings funding opportunities	• Improved river water quality	• Federal • State
Member Agencies						
	• Untreated water connections to existing reservoirs	• Where the RCS passes existing reservoirs, provide connections to the deliver water	• With implementation of RCS program	• Expanded use of regional storage facilities	• Improved flexibility in reservoir operation	• None identified at this time
Energy						
	• Large commercial energy demand	• Consistent, long-term energy pricing with incentives • Energy recovery and energy storage potential	• With implementation of RCS program	• Cost effective energy • Potential revenue source	• Long-term energy customer • Revenue	• Private
Bi-national						
	• Emergency connection	• Project turnout (conveyance) to provide emergency water	• With implementation of RCS program	• Brings funding opportunities	• Emergency infrastructure	• Federal
Native American						
	• Reliable water supply	• Where the RCS passes tribal lands, water could be supplied	• With implementation of RCS program	• Serving new member agencies	• Reliable water supply to disadvantaged communities	• Federal

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TABLE 8-3
Potential Partnership and Funding Opportunities

Agency	Issues	Solution Provided by RCS	Timing	Water Authority/Member Agency Benefit	Partner Benefit	Funding Source
State						
	• Water resiliency	• Additional conveyance provides water delivery redundancy to San Diego County	• With implementation of RCS program	• Water supply redundancy	• Avoids the San Andreas fault and frees up supplies outside San Diego County for other Southern California water agencies should existing conveyance systems be disrupted	• State
Public Private Partnerships						
	• Project risk and large funding requirements	• Transfer risk to private entities and move funding to private partner balance sheets	• With implementation of RCS program	• Risk transfer of some facilities • Reduced impact to balance sheet	• Revenue source	• Private partner

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9

SCREENING CRITERIA AND EVALUATION

Chapter 9.0 Screening Criteria and Evaluation

9.1 Introduction

9.1.1 Overview

This study is being completed in two steps. This report documents the initial step, known as Phase A, which consists of evaluations and data collection necessary to fully describe the three alternatives being considered and compare them to the continued use of MWD facilities to convey the Water Authority's QSA supplies to San Diego, known as the status quo.

The screening evaluation documented herein, known as the initial or coarse screening, uses the data collected on each alternative to develop and prioritize screening criteria to achieve a ranking of alternatives. The highest-ranking alternatives are then compared to the status quo to determine if viable alternatives exist to convey the QSA water to San Diego County without the use of MWD facilities.

This chapter presents the technical analysis completed for the coarse screening to compare and rank Alternatives 3A, 5A, and 5C based on the information presented in the previous chapters. Final or fine screening would occur during Phase B of this study, should it be authorized, and would consist of more detailed evaluations to identify the preferred alternative. Other items documented in this chapter include the evaluation's goals, the screening process, and, ultimately, the results of the evaluation.

9.1.2 RCS Objectives

Alternatives were assessed to determine their viability by confirming there are no known fatal flaws and ensuring they meet the RCS's objectives, which are as follows:

1. Alternative provides a cost competitive solution that makes the best use of ratepayer funds as compared to the status quo.
2. Alternative provides resilient improvements that are consistent with the Water Authority's other investments.
3. Alternative provides multiple benefits by incorporating approaches that meet multiple needs and align with the Governor's Water Resilience Portfolio objectives.

If all the alternatives evaluated as part of Phase A are deemed viable during this coarse screening, then the two (2) highest-ranking could move forward to Phase B, for more detailed evaluations, if the Water Authority elects to authorize it. In other words, Phase A is intended to help inform the Water Authority on the viability of RCS, while Phase B would be intended to define a preferred RCS and alternatives for the environmental studies necessary for preparation of CEQA and NEPA reviews.

9.1.3 Chapter Organization

The tasks completed during the screening evaluation to assess the three alternatives and achieve a ranking are summarized below and discussed in the following sections:

Screening Criteria – Screening criteria were developed to assess and compare the relative feasibility of each pipeline alignment based upon its ability to satisfy the Water Authority's objectives. The screening criteria was limited to the data collected on each alternative, as documented in Chapters 2 through 8. To objectively assess the screening criteria, a scoring system was developed to facilitate a quantitative comparison of the three alternatives.

Weighting Factors – Weighting factors were assigned to each of the screening criteria based upon its relative importance to the Water Authority at this conceptual level. Weighting factors emphasize certain criteria while limiting factors of less importance to the feasibility of the proposed RCS.

Decision Model – A spreadsheet-based decision model was used to evaluate each alternative based upon the screening criteria, the scoring system, and the weighting factors and assign a ranking.

Results – The results of the screening evaluation were presented, including a ranking of alternatives.

Conclusion – Outcomes and observations of the screening evaluation were noted, including the alternatives recommended for further evaluation, if any.

These steps are described in more detail in the subsequent sections.

9.2 Screening Criteria

This section describes the screening criteria used to assess and compare the three alternatives being considered, including details on the scoring system and the definitions of each screening criteria.

The screening criteria included the following:

- Cost
- Environmental Considerations:
  - *Land Impacts*
  - *Energy*
- Regulations/Institutional Coordination
- Land Use
- Community Impacts / Public Affairs
- Operation and Maintenance:
  - *Mechanical Equipment*
  - *Canals*
- Partnership Opportunities

Risk was assessed qualitatively for Phase A of the Study and was limited to identifying overarching, high level risks. As such, this screening evaluation did not include a criterion assessing risk, as meaningful information was not available. Phase B of this Study, should it be authorized, would assess risk quantitatively and a risk screening criterion would be added at that time.

9.2.1 Scoring System

For the purposes of the coarse screening evaluation, a rating or scoring system was developed to compare each pipeline alternative. Each reach was assigned a rating score for each criterion based upon its ability to satisfy the Study's objectives using a scale of 1 to 5. Although each factor has a unique scoring system, general definitions of the scoring system used for this Study have been developed and are shown in Table 9-1.

TABLE 9-1
Scoring System Definitions

Score	Requirements
5	The alternative satisfies the Study's objectives with either no negative impacts, or relatively low negative impacts in regard to the criterion.
3	The alternative satisfies Study's objectives but with an increasing level or degree of negative impacts related to the criterion. Where possible, consideration should be given to minimize this moderate impact during subsequent phases of implementation.
1	The alternative satisfies the Study's objectives, but significant disadvantages exist. This alternative could have a significant increase in cost, documentation, and/or permitting to the scope associated with this criterion.

The following sections define each of the screening criteria.

9.2.2 Cost

This study includes a detailed economic analysis comparing each alternative to the continued use of MWD facilities to convey QSA supplies to San Diego. The economic analysis used the estimated capital costs required to construct the facilities, the ongoing annual costs required to operate and maintain the facilities, and various economic factors (such as inflation, interest, discount rates, etc.) to determine the NPV for each alternative over a one-hundred-year life cycle. The NPV for the status quo included projections on the increase in fees charged by MWD for the delivery of QSA supplies. Details on the analysis are presented in *Chapter 6.0 – Risk, Cost Opinions, and Economic Comparison.*

This cost criterion was assessed in two parts using the results of the economic analysis documented in *Chapter 6.0*. The first part was a pass or fail test that compared each alternative to the status quo. In order to pass, the alternative needed a NPV comparable to the status quo. Alternatives which passed the first test were ranked based on their NPV. This NPV analysis was of the "base case" project description that included total capital costs and total annualized operating costs of the alternatives in 2020 Dollars. The NPV assumed no partnerships and that the Water Authority financed the construction. This criterion is important to determine the financial feasibility of each alternative to ensure fiscally responsible use of rate payer funds.

Due to the level of accuracy of cost opinions at this conceptual level, the lowest NPV and anything within 10% of the lowest NPV were assigned a score of 5. This is because 10% was deemed to be within the range of accuracy at this level of cost opinion. Table 9-2 presents the scoring system for Costs.

TABLE 9-2
Screening Criteria: Cost

Score	Requirements
5	Within 10% of the Lowest NPV
3	Next Lowest NPV
1	Highest NPV

9.2.3 Environmental Considerations

At this conceptual-level, a detailed environmental constraints analysis has not been completed. The intent of this study is to prepare detailed descriptions of all the proposed facilities such that subsequent environmental studies and analyses could be completed.

While detailed environmental impacts are not known, this screening evaluation considers environmental factors to the extent that they could be estimated based on the information available. To that end, two screening criteria were developed to estimate the relative impact each alternative might have on the environment: 1) Land Impacts and 2) Energy. These two criteria are defined below.

Land (Surface) Impacts

While it is understood that habitat quality and special-status species are not uniform, the land surface impacts expected to be encountered over pipeline alignments of this length are generally anticipated to be proportional to the length and type of construction methods employed. For instance, alignments with significantly more open-trench construction are anticipated to have larger surface impacts than alternatives that are mostly tunneled.

Potential surface disturbances and habitat loss resulting from construction of pipelines, canals, tunnel portals, pump stations, power generation facilities and treatment plants (including brine management) were determined for each alternative. For the purposes of this evaluation, the quantity of land disturbance was assumed to be equal to the permanent right of way (excluding tunnel subsurface easements) required for each alternative.

Table 9-3 presents the scoring system for Environmental Considerations – Land (Surface) Impacts.

TABLE 9-3
Screening Criteria: Environmental Considerations – Land (Surface) Impacts

Score	Requirements
5	Least amount of surface disturbance
3	Next least amount of surface disturbance
1	Highest amount of surface disturbance

Energy

This criterion assessed the long-term greenhouse gas (GHG) impacts anticipated for each alternative, which, for the purposes of this evaluation, was completed by comparing the annual energy consumption for each alternative. This criterion assumed all energy would be provided from the local utility providers with no distinction between fuel types at each point of use. It was assumed that the renewables portfolio of the energy service providers would be constant, regardless of alternative. Therefore, the more energy used, the more GHG that would be emitted. Detailed information of the actual fuel sources was not known at the time of this writing and should be considered during subsequent evaluations.

Energy consumption was based on annual operations and maintenance needs and did not include emissions related to construction activities or for the operation of backup power generators. Table 9-4 presents the scoring system for Energy.

TABLE 9-4
Screening Criteria: Environmental Considerations – Energy

Score	Requirements
5	Least energy consumed per year
3	Next least energy consumed per year
1	Highest energy consumed per year

9.2.4 Regulatory Agency and Institutional Coordination

Various Federal, State and Local government agencies require permits and approvals for construction impacting their jurisdiction. Examples of such agencies include, but are not limited to, Native American Reservations, California Department of Transportation (Caltrans), the California Department of Water Resources Division of Safety of Dams (DSOD), member agencies (like the City of San Diego), and the Federal Energy Regulatory Commissions (FERC). Other agencies, like the U.S. Army Corp of Engineers, U.S. Fish and Wildlife Service, the California Department of Fish and Wildlife, and the California State Water Resources Control Board were not included in the coarse screening as an evaluation of the impacts of the three alternatives on these agencies' jurisdictions have not been completed yet. However, Phase B of this study, should it be authorized, would include scope to gather information and assess the impact of the remaining alternatives on each of these agencies' jurisdictions. The fine screening would then compare the remaining anticipated impacts of the remaining alternatives.

The regulatory agency and institutional coordination criterion assessed the impact to the RCS associated with the coordination that would be required with the various entities for each of the alternatives. Coordination with these agencies could add to the complexity, increase the overall scope to meet their requirements, and, in some cases, could cause delays due to extended review times.

Some entities are anticipated to be more impactful than others, either due to their extended review process or the anticipated increase in scope to meet the requirements they impose.

Therefore, this assessment assigned a score to each agency assessing the anticipated difficulty of obtaining construction approvals.

A weighted score was determined for each alternative based on the number of jurisdictions impacted and the anticipated difficulty of obtaining construction approvals from that agency. For the weighted score, the higher the score the less favorable the alternative. In other words, the more entities an alternative must coordinate with and the more impactful coordinating those entities are anticipated to be, the worse the overall score.

The assigned score assessing the anticipated difficulty of obtaining construction approvals with each agency is as follows:

- **Native American Reservations.** Alternatives with construction within Indian Reservations/Tribal Lands received a score of 5 (high relative impact) to account for the cost, documentation, and permitting that would be added to the scope. An effort was made to avoid Indian Reservations/Tribal Lands during the identification of alignments to the extent possible. Alternative 3A only crosses Indian Reservations/Tribal Lands for the tunnel connecting to Lake Wohlford. The boundaries of Indian Reservations have been revised since the alignments were originally identified in the 1996 Water Transfer Study, resulting in Alternative 5A also crossing the Campo Indian Reservation in Tunnel T1. Scope would be included in Phase B of this study – should it be authorized – to further evaluate the alternatives to determine if minor modifications in the alignment could avoid crossing Indian Reservations/Tribal Lands.
- **Caltrans.** Linear encroachments in State highways received a score of 3 (average relative impact) based on additional permitting requirement from Caltrans.
- **Division of Safety of Dams (DSOD).**
    - Alternatives with a new dam, or modifications to an existing dam, received a score of 5 (high relative impact). This includes DSOD approvals for modifications to existing reservoir outlet structures.
    - If DSOD approvals are needed but the permitting process has already started and concepts verbally agreed upon by DSOD for modifications to an existing dam, then the alternative received a 3 (average relative impact).
    - If no permit from DSOD is required, then the alternative received a score of 1 (low relative impact).
- **Member Agency Coordination.** Alternatives requiring coordination and approvals from the Water Authority’s member agencies for added agreements related to the shared use of reservoir storage received a score of 3 (average relative impact).
- **FERC.** Alternatives requiring approvals from FERC due to the addition of new power generating facilities received a score of 3 (average relative impact).

Table 9-5 presents the scoring system for this factor. The highest weighted score corresponds with a criterion score of (1). Conversely, the lowest weighted score corresponds with a criterion score of (5).

TABLE 9-5
Screening Criteria: Regulatory Agency and Institutional Coordination

Score	Requirements
5	Lowest Weighted Score
3	Next Lowest Weighted Score
1	Highest Weighted Score

9.2.5 Land Use

The land use criterion assessed each alternatives' compatibility with the current land use through which it would be constructed. The less compatible the alternative is with the current land use, the more difficult it is anticipated to be to construct. Difficulties could include more expensive construction techniques to avoid or mitigate the potential impacts to surface features, revisions to the design to avoid impacts, or other impacts not listed. Further, much of the cut-and-cover portions of the alignments were selected due to the undeveloped nature of the land. Privately owned undeveloped areas could undergo development prior to the implementation of the RCS, which would result in a totally different landscape for construction of the conveyance facilities.

Land use types were determined and measured through publicly available GIS information obtained through either the San Diego Association of Governments' (SANDAG) website or through the websites of the governing agencies.

Similar to the Regulatory Agency and Institutional Coordination criterion, land use was assessed by calculating a weighted score. A score was assigned to each land use type based upon the anticipated difficulty that is anticipated to construct within. Then the weighted score was calculated based on the length constructed within each land use type and the difficulty score assigned to that land use.

For the weighted score, the higher the score the less favorable the alternative. The score assigned to each land use type to assess the relative difficulty that is anticipated to construct within is as follows:

- **Bureau of Land Management (BLM) Land.** BLM oversees 15 million acres of public lands in California. The BLM develops extensive land use plans to provide the framework to guide decisions for approved uses on BLM-managed lands. Based on coordination conducted during prior studies, it appears the BLM would approve the construction of a below grade pipeline through their lands as long as it met the intentions of the applicable land use plans. Each alternative crosses significant stretches of BLM land where the BLM would be the only jurisdiction requiring coordination.
    - Open cut construction in BLM Land receives a score of 1 (low relative impact)

- Trenchless construction in BLM Land was not scored (no impact)

- **Private Property.** Private land has the risk of development occurring between the completion of this study and the beginning of construction. Further, privately held lands are typically smaller in acreage. Therefore, there would be an increase in the number of parties that would need to be coordinated with.
  - Open cut construction in private property receives a score of 3 (average relative impact)
  - Trenchless construction in private property was not scored (no impact)
- **Area of Critical Environmental Concern Land.** The BLM provides the ACEC designation to areas where special management and direction is needed to protect human life from natural hazards and prevent irreparable damage to wildlife resources, natural systems, and important historic, cultural, and scenic values. Surface impacts within ACEC designated areas are anticipated to be very difficult.
  - Open cut construction in ACEC Land receives a score of 5 (high relative impact)
  - Trenchless construction in ACEC Land was assumed to minimize the impacts to the land use and was therefore not scored (no impact)
- **State Parks.** California State Parks are internationally significant conservation areas and home to a wide variety of species that require protection. Surface impacts within State Park designated areas are anticipated to be very difficult. While trenchless construction would minimize the surface impacts, tunnels crossing beneath State Parks would still be subject to their review and requirements, which could include additional studies and an extended review process.
  - Open cut construction in State Parks receives a score of 5 (high relative impact)
  - Trenchless construction in State Parks receives a score of 3 (average relative impact)
- **National Parks/Forests.** National Parks and National Forests are also internationally recognized for the environments they protect. Obtaining a permit associated with surface impacts within National Park and Forest designated areas, would be very difficult. While trenchless construction would minimize the surface impacts, tunnels crossing beneath National Parks and Forests would still be subject to their review and requirements, which could include additional studies and an extended review process. Linear facilities constructed within National Parks and National Forests require detailed studies and robust designs to mitigate any potential impact to groundwater resources.
  - Open cut construction in National Parks/Forests receives a score of 5 (high relative impact)
  - Trenchless construction in National Parks/Forests receives a score of 3 (average relative impact)

- **Indian Reservations/Tribal Lands.** Indian Reservations are lands held in trust by the United States government for a federally recognized Indian tribe. The land is federally managed by the United States Bureau of Indian Affairs rather than the state government in which they are physically located. Construction of a linear pipeline within an Indian Reservation is anticipated to be extremely difficult, regardless of the construction method, as the tribes retain the rights to the water and minerals underlying their lands. Linear facilities constructed within reservation lands require detailed studies and robust designs to mitigate any potential impact to groundwater resources.
    - Open cut construction in an Indian Reservation receives a score of 5 (high relative impact)
    - Trenchless construction in an Indian Reservation receives a score of 5 (high relative impact)
- **Endangered Species Act (ESA) Critical Habitat.** The ESA requires the designation of critical habitat for species when an area contains the physical or biological features essential to the species conservation. Surface impacts resulting in a need for a permit within ESA designated critical habitat areas are anticipated to be very difficult to obtain.
    - Open cut construction in ESA Critical Habitat land receives a score of 5 (high relative impact)
    - Trenchless construction in ESA Critical Habitat land was assumed to minimize the impacts to the land use and was therefore not scored (no impact)
- **Military Bases.** A military base is a facility owned and operated by branch of the military. Bases shelter military equipment, personnel, and other items of national security. As such, military bases have a high-level of security. Construction of any facility that requires permanent maintenance through a military base would be extremely difficult, both to construct and to operate.
    - Open cut construction in a military base receives a score of 5 (high relative impact)
    - No trenchless construction was considered beneath a military base

Table 9-6 presents the scoring system for land use. The highest weighted score corresponds with a criterion score of (1). Conversely, the lowest weighted score corresponds with a criterion score of (5).

TABLE 9-6
Screening Criteria: Land Use

Score	Requirements
5	Lowest Weighted Score
3	Next Lowest Weighted Score
1	Highest Weighted Score

9.2.6 Community Impacts and Public Affairs

The community impacts and public affairs criterion assessed the impact each alternative was anticipated to have on the community. Potential community impacts could include impacts on traffic, utilities, parks, agricultural land, or schools. For the purpose of this evaluation, these impacts were assessed based on the length of pipelines and canals constructed through rural residential developed areas, agricultural lands and parklands.

While each type of community impact was not calculated individually, the more open-trench construction through rural residential, agricultural, or park lands, the greater the impact to the community is expected to be as compared to construction through undeveloped lands or construction via trenchless construction methods.

Table 9-7 summarizes the scoring system for community impacts and public affairs.

TABLE 9-7
Screening Criteria: Community Impacts and Public Affairs

Score	Requirements
5	Shortest length through rural residential, agricultural and park lands
3	Next shortest length through rural residential, agricultural and park lands
1	Longest length through rural residential, agricultural and park lands

9.2.7 Operation and Maintenance (O&M)

The RCS proposes the construction of a number of new facilities that would require operating by the Water Authority's staff (or by others). Some of these new facilities are similar to facilities currently owned and operated by the Water Authority, such as the pump stations, while others would be new or unique to the Water Authority's infrastructure, such as canals. Each alternative proposes a widely varying quantity of new facilities. To assess the relative impacts each alternative would have on the Water Authority's operation staff, two criteria were developed: 1) mechanical facilities and 2) canals.

Based on the level of study, there does not appear to be a quantifiable difference in the operational reliability/facility resilience between the alternatives. The three alternatives provide similar levels of flexibility, redundancy, and shutdown impact to the Water Authority's system. They also appear to have similar levels of vulnerability to man-made or natural disasters. Further evaluation of facility resilience is recommended during future phases of work.

Mechanical Facilities

This criterion assessed the impact of each alternative on the Water Authority's operation staff based on the total number of proposed facilities with large, rotating mechanical equipment, such as pump stations, power generating facilities, and water treatment plants (common to all). This evaluation did not attempt to differentiate between the difficulties of operating the various mechanical facilities based on facility type, size, or location. Instead, all proposed mechanical facilities with large rotation equipment were tabulated.

Alternatives with a higher number of new mechanical facilities were deemed to have a larger impact on the Water Authority's operation staff. Conversely, alternatives with a lower number of new mechanical facilities were deemed to have a lesser impact on the Water Authority's operation staff. Table 9-8 summarizes the scoring system for O&M – Mechanical Facilities.

TABLE 9-8
Screening Criteria: O&M – Mechanical Facilities

Score	Requirements
5	Lowest number of new mechanical facilities
3	Next lowest number of new mechanical facilities
1	Highest number of new mechanical facilities

Canals

This criterion assessed the impact of each alternative on the Water Authority's operation staff based on the length of new canals. While not inherently complex, canals are new to the Water Authority's infrastructure and offer unique operational challenges, such as increased maintenance concerns with security and stormwater/flash flood risks compared with buried conduits. Impacts on O&M is based on the total length of canals to address increased maintenance concerns with security and stormwater/flash flood risks compared with buried conduits. Additionally, there is a wide differential in the length of new canals being proposed between the alternatives. Alternative 3A is much further from the AAC and requires significantly more canals for the Water Authority to maintain than Alternatives 5A and 5C, which are much further south. Table 9-9 summarizes the scoring system for O&M – Canals.

TABLE 9-9
Screening Criteria: O&M – Canals

Score	Requirements
5	Least length of new canals
3	Next least length of new canals
1	Most length of new canals

9.2.8 Partnership Opportunities

The partnership opportunities criterion assessed each alternatives' potential to facilitate partnership opportunities. These partnerships offer the opportunity to meet one of the study's objectives by incorporating approaches that meet the needs of other entities and further enhance the RCS's ability to meet State objectives regarding water resilient infrastructure. Potential partnerships are defined in *Chapter 8.0 – Partnership and Funding Opportunities*.

For purposes of this screening evaluation partnership opportunities were looked at in two ways for the benefits they could bring to the RCS:

- Partnerships that might: 1) improve operational reliability, including seasonal and operational storage, 2) provide environmental benefits, 3) improve water quality, or 4) afford streamlined or expediated permitting were assigned a high score of 5.
- Partnerships that might: 1) provide funds for construction and/or operations (includes favorable power supply, renewable energy) or 2) provide opportunities for enhanced supplies and/or supply reliability were assigned a medium score of 3.
- There was no low score for this criterion.

If a partnership opportunity met both benefits, then a score of 5 was assigned. The potential partnerships for each alternative were scored and totaled and the alternatives were ranked from highest to lowest. Table 9-10 summarizes the scoring system for Partnership Opportunities.

TABLE 9-10
Screening Criteria: Partnership Opportunities

Score	Requirements
5	Highest total score
3	Next highest total score
1	No scores of 1 were assigned for Partnership Opportunities

9.2.9 Summary

Table 9-11 summarizes the scoring system for each screening criteria.

TABLE 9-11
Screening Criteria: Scoring System Summary

Criteria	Scoring Range 5	3	1
Cost	Lowest NPV	Next Lowest NPV	Highest NPV
Environmental Considerations – Land (Surface) Impacts	Least amount of surface disturbance	Next least amount of surface disturbance	Most amount of surface disturbance
Environmental Considerations – Energy	Least energy consumed per year	Next least energy consumed per year	Most energy consumed per year
Regulatory Agency and Institutional Coordination	Lowest Weighted Score	Next Lowest Weighted Score	Highest Weighted Score
Land Use	Lowest Weighted Score	Next Lowest Weighted Score	Highest Weighted Score
Community Impacts and Public Affairs	Shortest length through rural residential, agricultural and park lands	Next shortest length through rural residential, agricultural and park lands	Longest length through rural residential, agricultural and park lands
O&M – Mechanical Facilities	Lowest number of new mechanical facilities	Next lowest number of new mechanical facilities	Highest number of new mechanical facilities
O&M – Canals	Least length of new canals	Next least length of new canals	Most length of new canals
Partnership Opportunities	Highest total score	Next highest total score	Lowest total score

9.3 Weighting Factors

To account for the difference in relative importance that each evaluation factor contributes to the overall evaluation, weighting factors reflecting the Water Authority's priorities for the RCS were assigned to each screening factor. Table 9-12 presents the weighting factors used in this analysis. As a note, these weighting factors were modeling after the Water Authority's Carryover Storage Project (CSP) to ensure consistency with past Water Authority practices.

TABLE 9-12
Weighting Factors

Screening Factor	Factor Weight
Cost	25%
Environmental Considerations	25%
- *Land (Surface) Impacts*	*(60% of the Environmental Factor Weight; or 15% total)*
- *Energy*	*(40% of the Environmental Factor Weight; or 10% total)*
Regulatory Agency and Institutional Coordination	10%
Land Use	10%
Community Impacts and Public Affairs	10%
Operation and Maintenance	10%
- *Mechanical Facilities*	*(80% of the O&M Factor Weight; or 8% total)*
- *Canals*	*(20% of the O&M Factor Weight; or 2% total)*
Partnership Opportunities	10%

9.4 Decision Model

In order to achieve a ranking of feasible alternatives, a spreadsheet-based decision model was developed. The decision model used the data collected in Chapters 2 through 8 to compare and assess the quantitative and qualitative characteristics of each alternative.

This model applies the scoring methodology described in the previous sections of this chapter to calculate a total "unweighted" score. The weighting factors are then applied to produce a scoring result that is compared to achieve a ranking of alternatives to determine preferences. Table 9-13 presents a summary of the decision model inputs and results.

TABLE 9-13
Summary of Decision Model

Criteria	Alternative 3A				Alternative 5A				Alternative 5C			
	Sum	Raw Score	Weighting	Weighted Score	Sum	Raw Score	Weighting	Weighted Score	Sum	Raw Score	Weighting	Weighted Score
RCS Objectives Met	Y	Y			Y	Y			N	N		
Cost NPV ($B)	25.1	5	25%	**125**	25.8	5	25%	**125**	38.1	1	25%	**25**
Environmental Considerations – Land Impact (Acres)	1,699	3	15%	**45**	1,455	5	15%	**75**	2,002	1	15%	**15**
Environmental Considerations – Energy (Elec – GWH/yr)	774	3	10%	**30**	735	5	10%	**50**	1,097	1	10%	**10**
Regulatory Agency / Institutional Coordination (Score)	11	3	10%	**30**	11	3	10%	**30**	12	1	10%	**10**
Land Use Impacts (Weighted miles)	8.2	5	10%	**50**	41.1	1	10%	**10**	35.5	3	10%	**30**
Community Impacts / Public Affairs (Miles)	1.2	5	10%	**50**	7.5	3	10%	**30**	9.3	1	10%	**10**
O&M: Mechanical Facilities (No.)	4	5	8%	**40**	5	3	8%	**24**	11	1	8%	**8**
O&M: Canals (Miles)	42.2	1	2%	**2**	13.0	3	2%	**6**	1.5	5	2%	**10**
Partnerships (Score)	25	5	10%	**50**	14	3	10%	**30**	14	3	10%	**30**
Weighted Score				**422**				**380**				**148**

9.5 Screening Evaluation Results

As mentioned previously, the goal of the screening evaluation was two-fold. The first goal was to utilize various technical and qualitative analysis to select the two (2) highest scoring alternatives for more detailed consideration during Phase B of the study, should it move forward. The second goal was to compare the highest-ranking alternatives with the status quo to determine if viable alternatives exist.

The initial ranking of alternatives based on the screening evaluation results shown in Table 9-14 above are as follows:

- Alternative 3A – 422 (weighted score)
- Alternative 5A – 380 (weighted score)
- Alternative 5C – 148 (weighted score)

Based on the results of the screening evaluation, Alternatives 3A and 5A are clearly preferred over Alternative 5C. Alternatives 3A and 5A score as well as, or more favorably than Alternative 5C in virtually every criterion except for O&M – Canals, which is the lowest weighted of the screening criteria. Table 9-14 summarizes the characteristics of each alternative compared to the other alternatives for each screening criteria. In general, alternatives noted as “least” are more favorable for a particular screening criterion as compared to the other alternatives, and alternatives noted as “most” are less favorable. Alternatives in between are noted as “middle.” For partnership opportunities criterion, the alternative with the most potential opportunities was noted as the “favored” alternative, while the others were noted as “not favored.”

TABLE 9-14
Key Characteristics of Each Alternative

Screening Criteria	3A	5A	5C
Cost	• Least	• Nearly Least	• Most
Environmental Considerations			
Land Impacts	• Middle	• Least	• Most
Energy	• Middle	• Least	• Most
Regulatory Agency and Institutional Coordination	• Least (tied)	• Least (tied)	• Most
Land Use	• Least	• Most	• Middle
Community Impacts and Public Affairs	• Least	• Middle	• Most
O&M			
Mechanical Facilities	• Least	• Middle	• Most
Canals	• Most	• Middle	• Least
Partnership Opportunities	• Favored	• Not favored (tied)	• Not favored (tied)

9.5.1 Comparison to the Status Quo

As mentioned earlier, each of the alternatives were compared to the status quo to determine their viability by meeting the RCS’s objectives. This section compares the alternatives with each of the RCS’s objectives.

Provides a cost competitive solution

Chapter 6 – Risk, Cost Opinions, and Economic Comparison presents the economic comparison that was completed by the Water Authority. The economic comparison included a lifecycle assessment of each of the alternatives to the status quo. As documented in Chapter 6, Alternatives 3A and 5A both provide cost competitive solutions to the status quo.

Alternative 5C did not compare favorably to the status quo and was therefore screened from further consideration in this evaluation.

Provides resilient improvements

All three alternatives provide resilient improvements designed to the match the Water Authority's other capital investments. As the conveyance infrastructure would be new and built with similar materials to those used in MWD's existing conveyance infrastructure (e.g. Colorado River Aqueduct System), it is anticipated that the RCS would be at least as resilient as the status quo. The detailed project descriptions for each alternative were developed following the Water Authority's design guidelines, providing for a robust and resilient system consistent with other investments. Pressure pipelines are assumed to be lined and coated welded steel pipe. Tunnel lining and ground stabilization depends on many factors still unknown but would be designed at a minimum to permit transfer of internal and external hydrostatic and rock loads to occur between the final lining and the surrounding rock mass without excessive deformation to ensure a long lifespan with minimal maintenance. Mechanical facilities, such as pump stations and water treatment facilities, would be designed consistent to the Water Authorities other facilities of similar type and size.

Not only would the RCS consist of resilient materials and a resilient design, it also increases the resiliency of the Water Authority's overall system by adding an independent connection to import QSA supplies that is geographically distant from MWD's existing facilities. The RCS would be resilient to natural disasters that occur outside the region, such as an earthquake on the San Andreas Fault.

Provides multiple benefits and aligns with State objectives

All three alternatives offer approaches that provide opportunities to meet the needs of multiple entities, including the Water Authority's member agencies, disadvantaged communities, and others, as described in *Chapter 8.0 – Partnership and Funding Opportunities*. They also align with the Governor's Water Resilience Portfolio objectives by providing the Water Authority and its member agencies another outlet to convey QSA supplies into its service area. However, it should be noted that Alternative 5C would create additional GHG due to energy use as compared to the other RCS alternatives. So, while the specific 5C alternative provides multiple benefits and aligns with some of the State's objectives, it does not meet all the objectives that the other alternatives do.

Summary of the comparison to status quo

Table 9-15 summarizes the comparison of each alternative to the RCS's objectives to determine their viability.

TABLE 9-15
Summary of Alternatives in Meeting the RCS's Objectives

Objective	Alternative		
	3A	5A	5C
Provides a cost competitive solution	Yes	Yes	No
Provides resilient improvements	Yes	Yes	Yes
Provides multiple benefits and aligns with State objectives	Yes	Yes	No

9.6 Sensitivity Analysis

An analysis was performed to test the sensitivity of the results to changes in the weighting factors. This sensitivity analysis was performed by varying the weighting factors described previously to determine what changes would be required to affect the outcome. As can be seen in Table 9-15 above, Alternative 3A scored better than Alternative 5C in every criterion except O&M – Canals. Similarly, Alternative 5A scored better than Alternative 5C in every criterion except O&M – Canals and Land Use Impacts.

To change either of the top two (2) alternatives in the rankings, the weighting factors for both O&M – Canals and Land Use Impacts would have to be significantly increased and other factors decreased accordingly. An example of the necessary changes to the weighting factors that would have to be made are as follows:

- Increase O&M – Canals to 30% (from 2%), an increase in weighting of 1,400%
- Increase Land Use Impacts to 30% (from 10%), an increase in weighting of 200%
- Decrease Cost, Environmental – Land (Surface) Impacts, Community Impacts / Public Affairs, O&M – Mechanical Facilities, Regulatory Agency / Institutional Coordination, and Partnerships all to 5%

The sensitivity analysis was conducted during the screening analysis workshop between Black & Veatch and the Water Authority's staff. As shown, reasonable changes to the weighting system do not affect the outcome of ranking of alternatives. In order to affect the result, extreme changes to the weighting factors would be required, as presented above. These extreme changes do not reflect he Water Authority's goals for the study or past practices.

9.7 Conclusion

Two alternatives – Alternatives 3A and 5A – met all the RCS's objectives and offer viable alternatives to convey QSA supplies to San Diego without the use of MWD facilities. These two alternatives are also the highest ranked alternatives, offering many potential benefits over Alternative 5C, as described above. The results of the screening analysis show that

Alternatives 3A and 5A warrant further, more detailed evaluation in Phase B of the study, should it be authorized.

There are many items that require more detailed study to 1) further enhance the project definition of each alternative and 2) determine the preferred alternative, including but not limited to the following:

- An environmental constraints analysis assessing factors such as habitat quality and potential special-status species disturbances
- Research on existing utilities impacted by each alternative
- Subsurface exploration, including a geotechnical evaluation and dewatering studies
- Further refinements to the facility layouts and alignments
- A comparison of the implementation of each alternative
- A property acquisition analysis
- A quantitative analysis and risk register update
- Consideration for socio-economic, air quality, and non-physical environmental impacts
- A more detailed comparison of regulatory agency and institutional requirements/permits

As noted earlier, Indian Reservation/Tribal Lands were avoided to the extent possible. As currently described, Alternative 3A crosses Indian Reservation Lands for the tunnel connecting the pipeline alignment to Lake Wohlford for operational storage. Further evaluation would be conducted in Phase B, should it be authorized, comparing tunneling through the reservation land to an alignment that avoids the reservation boundaries. Similarly, Phase B would also evaluate Alternative 5A to determine if the Campo Indian Reservation could be avoided.

It is recommended that these items be studied in subsequent phases of the design. Some of these items could be studied during Phase B of the study, should it be authorized, while others are recommended for evaluation only on the preferred RCS alternative that would be selected during Phase B.

10

CONCLUSIONS

Chapter 10.0 Conclusions

10.1 Overview

This chapter presents a summary of the key findings and results from the engineering, environmental, social, and economic evaluations that were conducted to determine the viability of three RCS alternatives against the status quo of continued use of MWD facilities to convey QSA supplies to San Diego County.

10.2 Regional Conveyance System Flows

The RCS would be designed to convey the Water Authority's annual QSA supply, plus any additional supplies resulting from potential partnership opportunities, less treatment losses to meet salinity goals. For the purposes of sizing the RCS facilities, an annual post-treatment supply of 20,000 AF has been assumed for all potential partnership opportunities, with the point of delivery being Borrego Valley. In reality, these supplies would be developed by each potential partner, using the RCS to a point of delivery near such partner.

Table 10-1 summarizes RCS conveyance requirements, differentiating RCS flows and design capacity. This table illustrates the volumes of pre-treatment flows conveyed from the end of the AAC to the influent side of the treatment plant, and the post-treatment flows conveyed to Borrego Valley with the balance delivered to the Water Authority.

For sizing considerations, a 10-percent surcharge was added to determine RCS design capacity. The larger design capacity allows for system downtimes and assures annual RCS deliveries could be achieved if a planned or unplanned outage occurs during any given year. The increased conveyance size would also improve operational flexibility to meet seasonal variations in demand and reduce in-region seasonal storage needs during low demand winter months.

TABLE 10-1
Summary of RCS Conveyance Requirements

Conveyance Reach	RCS Flows (AFY)	RCS Design Capacity (AFY)
AAC to Treatment Plant Influent	301,500	331,70
Treatment Plant Effluent to Borrego Valley	278,700	306,570
Borrego Valley to Twin Oaks/SVR	258,700	286,570

10.3 Canals, Pipelines, and Tunnels

Gravity Flow Canals. The capacity across IID's existing canal system to meet RCS flow requirements was evaluated based on 10-years of recent IID-provided flow data for the AAC and WSM canal that captures delivery trends resulting from the conversions of agricultural lands to solar farms along the western reaches of the AAC. Based on the flow data, and as verified by IID, there is sufficient capacity in the AAC upstream of the New River Siphon to convey RCS flows.

The flow data also revealed that there is insufficient capacity within the AAC downstream of the New River Siphon during peak irrigation cycles, and at all times within the WSM canal. To address the AAC capacity constraint, an evaluation of coordinated operations combining RCS flows with peak and non-peak irrigation cycles was performed. The evaluation revealed that coordinated operations, coupled with 2,900 AF of new reservoir storage downstream of the New River Siphon meets all RCS flow requirements.

This storage requirement was split to include a new 900 AF open reservoir in the IID service area and utilization of 2,000 AF of available space in SVR. The new reservoir would be consistent with IID plans to develop operational storage near Foxglove Check, offering a potential partnership opportunity for compatible and joint reservoir operations. The new reservoir would have a footprint of 200 acres and is assumed to be earthen with a plastic liner.

The WSM canal capacity constraint would be addressed through construction of a parallel canal extended as needed along the WSM, Thistle, and Trifolium Extension canals. The parallel canal would be constructed as an open concrete-lined canal within new dedicated easements.

Pipelines and Tunnels. The pressurized portion of the RCS would be constructed as a series of cement mortar lined and coated welded steel pipelines and hard rock tunnels. Pipeline size would be 102-inches, resulting in a peak velocity of 7.5 fps and dynamic head losses of 7.8 feet per mile. The pipelines would be installed using cut-and-cover methods with sloped trench sides to limit construction costs. At select locations, shoring or other methods could be imposed to minimize environmental impacts. Trenchless construction methods would also be applied for major road and railroad crossings.

Each alternative would contain varying amounts of hard rock tunnels based on topography and pumping requirements. For this study phase, the tunnels are assumed to be excavated with an open-face, hard rock TBM. The tunnel ground stabilization and final lining would vary based on the conditions encountered and the potential impacts to groundwater resources. Canal, pipeline and tunnel characteristic are shown in Table 10-2.

TABLE 10-2
Canal, Pipeline, and Tunnel Characteristics

Characteristic	Alternative 3A	Alternative 5A	Alternative 5C
Gravity Flow Canals, miles	46.7	8.8	1.5
Pressure Pipelines, miles	38.8	34.8	81.2
Pressure Tunnels, miles	46.5	41.4	10.6
Overall Alignment Length, miles	132.0	85.0	93.3
AAC Operational Storage, capacity	900 AF	900 AF	900 AF
RCS Terminal Storage, capacity	3,500 – 4,000 AF	-	-

Terminal storage would be required to balance daily RCS operations and meet Water Authority reliability needs for periods of scheduled and unscheduled system outages. Alternatives 5A and 5C would utilize existing storage in SVR to meet this requirement. For Alternative 3A, new reservoir storage is required near the RCS system westerly terminus in north San Diego County. For Phase A cost estimating, 3,500 to 4,000 AF of new storage is provided.

10.4 Pump Stations and Hydroelectric Plants

Table 10-3 presents the key design criteria for the pump stations and hydroelectric associated with each alternative.

TABLE 10-3
Pump Stations and Hydroelectric Facilities

Characteristic	Alternative 3A	Alternative 5A	Alternative 5C
Minimum Elevation, feet (MSL)	-218	-5	-35
Maximum Elevation, feet (MSL)	1,140	1,150	4,050
Total Pumping Head, feet	1,980	1,555[(1)]	4,225[(1)]
Total Hydroelectric Head, feet	0	0	2,350
Pump Stations, each	3	2	5
Power Generating Facilities, each	0[(2)]	0	3
Forebays, capacity	3 (40 AF each)	2 (40 AF each)	5 (40 AF each)
Afterbays, capacity	0	0	3 (40 AF each)
AAC Operational Storage, capacity	900 AF	900 AF	900 AF

Notes:

1. The total pumping head and number of pump stations do not include improvements to convey water from SVR north to Twin Oaks Valley, which would add a pump station with 490 feet of total pumping head to Alternatives 5A and 5C.
2. Opportunities for micro-hydro may exist at canal drop structures and pipeline turnout facilities. These opportunities should be further evaluated during subsequent phases of design.

10.5 Aqueduct Operations

The Water Authority's existing aqueduct system was largely built to provide north-to-south gravity flow conveyance of imported water supplies from MWD. The integration of new supply alternatives from the RCS could be accomplished with modifications to the current aqueduct operating strategies, along with the implementation of new facilities.

Alternative 3A is proposed to terminate at a new 40 MG FRS/Day Tank located in the Twin Oaks Valley area that would connect to existing untreated water aqueduct pipelines upstream of the Twin Oaks Valley Diversion Structure, allowing gravity operation of RCS flows similar to flows from MWD.

Alternatives 5A and 5C are proposed to terminate at the SVR with connections to the San Vicente Pipeline (SVP) at or near the existing access vault structure. To meet north county member agency demands, a new 200 cfs pump station and approximately 12.5 miles of 72-inch diameter pipeline would be required to convey RCS flows from the Del Dios area to a new 40 MG FRS/Day Tank in Twin Oaks Valley.

Seasonal storage of RCS flows may be required to balance monthly demand variations and are compatible with planned use of Water Authority-owned storage accounts in SVR. Alternatives 5A and 5C would also impact power generation estimates for the Rancho

Penasquitos Pressure Control and Hydroelectric Facility due to the lower delivery gradient of this alternative, but pressure and flow control features would still be maintained without modification. Table 10-4 summarizes the new facilities and improvements related to the integration of RCS flows.

TABLE 10-4
Summary of New Facilities and Improvements Required to Integrate RCS Flows

Alternative	New Facilities / Improvements
Alternative 3A	• 40 MG FRS / Day Tank in the Twin Oaks area
Alternative 5A/5C	• 40 MG FRS / Day Tank in the Twin Oaks area • 12.5 miles of transmission pipeline, 200 cfs pump station and forebay • Improvements to the SVSCF overflow structure • Modification of the SVR fill chute to a flow control valve

10.6 Treatment, Blending, and Brine Management Options

Blending Options. Blending evaluations at the SVR and TOVWTP determined that blending alone would not reduce TDS levels below the target level of 500 mg/L.

RCS Treatment Locations. Four locations were considered for an RCS treatment facility to address high salinity within the existing QSA water. These locations coinciding with each alternative’s connection to a new canal parallel to the WSM canal in Imperial County, and one location in San Diego County specific to Alternative 3A. The treatment facility would require between 43 and 51 acres of land.

Conceptual Design Criteria. A treatment process train of Membrane Filtration (MF) followed by Reverse Osmosis (RO) would require an annual influent flow of 301,500 AF to produce 278,700 AF of effluent water meeting the target TDS of 500 mg/L. The addition of brine volumes minimization techniques could increase effluent output to 289,900 AF.

Brine Management Options. Brine management was primarily focused on conveyance to the Salton Sea, providing both cost savings benefits and a potential source water for mitigation of exposed playa lands. Due to uncertainty regarding permitting and approvals, an alternative approach using brine volume minimization technology along with evaporation ponds was considered. Multiple brine volume minimization techniques were researched, landing on High Recovery Reserve Osmosis (HRRO) as the likely best option due to its recovery rates, energy use, small size and lower costs. Electrodialysis reversal (EDR) was compared to HRRO as another brine volume minimization technique, but the results were inconclusive about which method has the optimal cost benefit ratio. The higher concentrated brine would be disposed in evaporation ponds, requiring 200 acres of additional land acquisition.

10.7 Power Supply Alternatives

Power Requirements. Table 10-5 presents a summary of the facility power requirements for Alternative 3A. The power requirements for Alternatives 5A and 5C were considered in similar detail during prior studies. In general, the power requirements for Alternatives 5A and 5C are similar to Alternative 3A, with the exception that pump station demands would be larger due to higher rated total pumping head for Alternatives 5A and 5C.

TABLE 10-5
Summary of Alternative 3A Power Requirements

Facility	Power Requirement: Total	Permanent	Temporary	Major Components
Treatment Plant	55 MW	55 MW	---	• MF (9 MW), RO (9 MW), Screening • Intermediate Pumps (35 MW) • HVAC, lighting, chemical feed • Washwater Treatment • Brine Volume Minimalization
Pump Stations (3 total)	36 MW	36 MW	---	• Pumping Units (35 MW) • HVAC, lighting, misc. loads
Tunnel Portals (7 total)	2 MW	<0.01 MW	1.99 MW	• Power to TBMs and associated systems • HVAC, lighting, dewatering

Transmission Lines. A new dedicated transmission line (191 kV/92 kV) and associated step-down substation would be needed to provide power to each of the proposed pump stations and the treatment facility. New distribution lines (12.7 kV) would provide power for the tunnel portals. Table 10-6 provides a summary of the transmission and distribution line lengths and right-of-way (ROW) widths required for Alternative 3A.

TABLE 10-6
Summary of Proposed Transmission and Distribution Lines for Alternative 3A

Alternative	Transmission Lines: 230 kV	161 kV	92 kV	Distribution Lines: 12.7 kV
3A	7.4 miles(1)	2.4 miles	12.5 miles	12.6 miles
ROW (width)	100 feet	100 feet	100 feet	60 feet

Note:

1. Length determined based on selection of longest treatment plant transmission line.

SGE&E and IID were contacted to provide their current rate schedules, which were used as the basis of determining annual power costs in this report. It should be noted that future changes in rate schedules have historically been extremely difficult to predict and follow-up coordination with the utility service providers is required to verify the applied rates.

10.8 Risk, Cost Opinions and Economic Comparison

Risk Analysis. A qualitative risk analysis was completed and a risk register was developed. The identified risks focused on issues related to feasibility, the ability to develop accurate costs estimates, and operational concerns. The risk analysis compares the risks for each alternative based on the probability of occurrence and the likely impact of the risk. The qualitative comparison of project risks noted slight advantages to Alternatives 3A and 5A as compared to Alternative 5C. These advantages mostly manifest in the lower pumping head and smaller quantity of cut-and-cover construction methods required for Alternatives 3A and 5A. No fatal flaws were identified for the alternatives as a result of this study.

Cost Opinions. An opinion of probable construction cost developed for each facility component is presented in Table 10-7, including capital construction costs, construction management costs, pre-construction costs, and assigned contingencies for each alternative. The resulting costs for each alternative are within five percent of one another.

TABLE 10-7
Estimated Capital Costs(1)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Canals	$ 59,200,000	$ 10,900,000	$ 1,600,000
Pipelines	$ 359,000,000	$ 428,200,000	$ 1,033,600,000
Tunnels	$ 1,512,800,000	$ 1,431,400,000	$ 434,600,000
Pump Stations	$ 155,700,000	$ 156,000,000	$ 321,700,000
PGFs/PCFs	$ 0	$ 31,097,000	$ 134,874,000
Electric Distribution	$ 49,200,000	$ 39,300,000	$ 52,100,000
Water Treatment(2)	$ 625,500,000	$ 760,700,000	$ 783,300,000
Operational Storage (IID, RCS, Day Tanks)	$ 193,250,000	$ 108,250,000	$ 108,250,000
Office and Warehouse	$ 8,860,000	$ 8,860,000	$ 8,860,000
SUBTOTAL CONSTRUCTION	**$ 2,963,500,000**	**$ 2,974,700,000**	**$ 2,878,900,000**
Construction Management Soft Costs	$ 664,200,000	$ 662,100,000	$ 665,500,000
Pre-Construction Soft Costs(3)	$ 464,370,000	$ 450,680,000	$ 463,060,000
Contingency (10-30%)(4)	$ 861,643,000	$ 874,632,000	$ 835,795,000
TOTAL (2020 Dollars)	**$ 4,953,723,000**	**$ 4,962,119,000**	**$ 4,858,640,000**

Notes:

1. Expected accuracy range for a Class 4 estimate is from -30 percent to +50 percent.
2. Water treatment costs include brine management.
3. Pre-construction soft costs include the initial studies, engineering, right of way and property acquisition, CEQA/NEPA, public outreach, legal, environmental mitigation, owner's representative, and staff support.
4. Contingency varies between 10% and 30% for each component of work based on available information, level of design, and risk.

Annual OMR Costs. Annual OMR costs were estimated for operation of the RCS and are presented in Table 10-8.

TABLE 10-8
Estimated Annual OMR Costs(1)

Item	Alternative 3A	Alternative 5A	Alternative 5C
Energy Cost – Pumping	$ 82,000,000	$86,200,000	$ 219,800,000
Energy Cost – Treatment	$ 13,080,000	$13,080,000	$ 13,080,000
OMR	$ 17,414,000	$ 13,599,000	$ 26,092,000
Salinity Treatment (Excluding Energy)	$ 30,600,000	$ 32,949,000	$ 32,970,000
Energy Recovery	---	---	($ 33,400,000)
TOTAL ANNUAL COSTS (2020 Dollars)	**$ 143,094,000**	**$ 148,777,000**	**$ 258,205,000**

Economic Comparisons. The alternatives were compared on a per acre-foot basis with supply options providing greater reliance on MWD or development of new local supplies. Cost calculations include the cost of supplies, capital costs, and OMR costs as detailed in Table 10-9. All alternatives make use of the same financing, escalation, and project schedule assumptions.

TABLE 10-9
NPV Equivalent Unit Cost ($/AF)(1)

	Cost per AF	Notes
Alternative 3A	$1,697	
Alternative 5A	$1,733	
Alternative 5C	$2,384	Higher unit cost due to high pumping (O&M) costs.
MWD Reliance	$2,691	MWD Full Service Tier 1 to replace 200,000 AF of IID deliveries in 2048. Remaining 77,700 AF to be "exchanged" at the MWD Transportation rates. Reflects annual adjustments of 5.1% based on historical increases (2003-2022). No rate adjustments (increases) have been assumed for MWD's planned Cal Water Fix or Recycled Water Program costs (capital or O&M).
Local Supply Development	$2,594	Cost of $3,000/AF, based on a new desalination project. MWD Full Service Tier 1 to replace 200,000 AF of IID deliveries in 2048. Remaining 77,700 AF to be "exchanged" at the MWD Transportation rates. All costs are escalated at escalated at 3%. 77,700 AF to be maintained based on exchange agreement (Canal Supply + MWD Transportation). Assumes no double costs are incurred, leading up to 2047 QSA contract expiration.

Note:

1. Includes all costs (including construction of RCS) to deliver QSA supplies to the Water Authority between 2045 - 2112.

As demonstrated, the RCS project is forecasted to provide significant savings. While costs will continue to shift leading up to potential design and construction, the savings gap is not trivial and provides substantial margin.

10.9 Permitting and Environmental Considerations

The proposed alternatives cross a variety of Federal, State, County, City, and private lands, and borders or is in close proximity to other lands that include habitat preserves, State and local parks, a National Landmark at San Felipe Creek, a military reservation, and tribal reservations. As such, significant regulatory coordination is anticipated.

In an attempt to refine the project alignment and details, as well as to streamline the subsequent project specific CEQA/NEPA processes, the Water Authority has suggested completing a Staged Environmental Impact Report (EIR) or a Programmatic EIR. This would be followed by a project specific joint EIR/Environmental Impact Statement (EIS). As part of this review and permitting process, formal consultation would be undertaken with a variety of impacted resource agencies, as identified in Chapter 7.

An extensive list of other permits would also be required to construct the tunnels, pipelines, pump stations, treatment facility, and brine management pipelines. To complete the entire process, a sequence of tasks would need to be completed concurrently with the entire time needed to complete the environmental review and subsequent permitting estimate to take between 5 and 7 years.

10.10 Partnership and Funding Opportunities

Preliminary partnership were identified and a high-level summary of the potential benefits and associated funding opportunities to each partner were developed. The preliminary partnerships are cursory and need to be coordinated with the identified partners if the project moves to future phases. Under Phase B, the Water Authority would meet with potential partners to explore opportunities. Partnerships that provide regional benefits could attract local, State, and Federal grant funding for the project. Where the RCS solves challenges related to the environment, disadvantaged communities, and water resiliency, the likelihood to attract funding increases. Potential partnerships, funding opportunities, and benefits would also be quantified. In addition, opportunities associated with P3s would be further evaluated as this would allow for the transfer of risk connected with the development and long-term operations of the RCS.

10.11 Screening Evaluation

A detailed screening evaluation was performed to achieve a ranking of the alternatives. Screening criteria used in the evaluation were developed during focused workshops with Water Authority staff and are consistent with previous major projects completed by the Water Authority. Weighting factors were then assigned to each of the screening criteria based upon relative importance at the conceptual level of design. A final ranking was determined based on the combined weighted scores. Results of the screening evaluation are shown in Table 10-10.

TABLE 10-10
Screening Evaluation Results

Screening Criteria	3A	5A	5C
Cost	• Least	• Nearly Least	• Most
Environmental Considerations			
Land Impacts	• Middle	• Least	• Most
Energy	• Middle	• Least	• Most
Regulatory Agency and Institutional Coordination	• Least (tied)	• Least (tied)	• Most
Land Use	• Least	• Most	• Middle
Community Impacts and Public Affairs	• Least	• Middle	• Most
O&M			
Mechanical Facilities	• Least	• Middle	• Most
Canals	• Most	• Middle	• Least
Partnership Opportunities	• Favored	• Not favored (tied)	• Not favored (tied)
Weighted Scoring	422	380	148
Meets Project Objectives			
Provides a cost competitive solution	Yes	Yes	No

TABLE 10-10
Screening Evaluation Results

Provides resilient improvements	Yes	Yes	Yes
Provides multiple benefits and aligns with State objectives	Yes	Yes	No

10.12 Report Summary

The following list presents some of the key findings of this study:

- The region will continue to need QSA water through 2112.
- All three RCS alternatives are viable from a technical and engineering perspective.
- Alternatives 3A and 5A are economically competitive and provide long-term reliability and low cost water to the region.
- Alternative 5C is not economically competitive with Alternatives 3A and 5A and will not be recommended for further study.
- Alternatives 3A and 5A could be integrated without major changes to current Water Authority operations.
- Potential multi-agency, multi-use partnerships and other agreements could significantly reduce the cost and enhance the value of each RCS alternative and provide regional benefits to San Diego, California and the Southwest.
- Blending RCS deliveries with other supplies in existing reservoirs is no longer feasible and treatment of RCS supplies would be required to match the salinity of water currently delivered by MWD.
- Due to the decades-long process for designing, permitting, and building a major conveyance system, continuing to Phase B of the current study is recommended to retain the RCS as a viable option.

As discussed in the key findings summarized above, Alignments 3A and 5A are viable alternatives to the current status quo for the Water Authority. If approved to move forward with Phase B, these alternatives would be further defined to identify the preferred option, while providing enough project definition and detail to consider the project CEQA ready, which would allow for future permitting. Under the Phase B work, detailed coordination with potential partners, identification of potential funding, and quantification of partnership benefits would assist in the updates of the RCS costs estimates. Ongoing coordination with IID and Water Authority member agencies would also be critical to ensure delivery of the QSA supplies to the San Diego region.

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B

IID FLOW DATA

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Cana	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check			
	January	January	January	January	January	January	January	January	January	January
2019	2,766	978	1,254	354	517			482	375	107
2018	2,785	1,216	1,166	460	647	109	756	584	475	109
2017	2,612	757	1,030	287	404	66	470	498	432	66
2016	2,958	1,253	1,030	530	579	144	723	602	458	144
2015	2,859	1,175	1,236	465	568	142	710	578	436	142
2014	2,679	1,114	1,047	450	525	139	664	634	495	139
2013	3,127	1,202	1,361	456	628	118	746	638	520	118
2012	3,188	1,399	1,321	599	668	132	800	683	551	132
2011	3,283	1,381	1,485	588	656	137	793	693	556	137
2010	2,346	1,135	921	442	556	137	693	491	354	137
Max 2013-2019	3,127	1,253	1,361	530	647	144	756			
Max 2010-2019	3,283	1,399	1,485	599	668	144	800			
								693	556	144
	February	February	February	February	February	February	February			
2019	3,709	1,394	1,670	634	634	126	760			
2018	3,537	1,645	1,786	667	812	166	978			
2017	3,315	1,570	1,449	652	762	156	918			
2016	4,643	2,060	2,180	1,009	840	211	1,051			
2015	4,038	1,791	1,722	804	742	245	987			
2014	3,778	1,610	1,667	636	777	197	974			
2013	3,804	1,584	1,376	660	716	208	924			
2012	4,807	1,805	1,941	726	912	167	1,079			
2011	3,531	1,588	1,439	618	799	171	970			
2010	3,520	1,228	1,652	385	660	183	843			
Max 2013-2019	4,643	2,060	2,180	1,009	840	245	1,051			
Max 2010-2019	4,807	2,060	2,180	1,009	912	245	1,079			
	March	March	March	March	March	March	March			
2019	4,534	1,998	2,000	940	872	186	1,058			
2018	4,970	2,132	2,179	994	921	217	1,138			
2017	5,074	2,278	2,113	995	1,057	226	1,283			
2016	5,439	2,340	2,262	1,043	1,063	234	1,297			
2015	5,092	2,429	2,204	1,081	1,075	273	1,348			
2014	4,774	2,200	2,044	976	931	293	1,224			
2013	4,956	2,170	2,072	974	981	215	1,196			
2012	5,767	2,452	2,403	1,099	1,057	296	1,353			
2011	5,693	2,328	2,450	975	1,066	287	1,353			
2010	5,701	2,373	2,275	1,005	1,042	326	1,368			
Max 2013-2019	5,439	2,429	2,262	1,081	1,075	293	1,348			
Max 2010-2019	5,767	2,452	2,450	1,099	1,075	326	1,368			
	April	April	April	April	April	April	April			
2019	5,673	2,429	2,594	1,121	1,065	243	1,308			
2018	5,448	2,421	2,513	1,123	1,068	230	1,298			
2017	5,418	2,399	2,412	1,098	1,093	208	1,301			
2016	5,135	2,279	2,417	1,042	973	264	1,237			
2015	5,567	2,426	2,269	1,114	1,047	265	1,312			
2014	5,476	2,332	2,251	1,052	1,025	255	1,280			
2013	5,655	2,342	2,596	1,027	1,076	239	1,315			

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Cana	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check
2012	6,326	2,700	2,693	1,248	1,145	307	1,452
2011	6,100	2,622	2,620	1,152	1,111	359	1,470
2010	5,551	2,532	2,329	1,067	1,096	369	1,465
Max 2013-2019	5,673	2,429	2,596	1,123	1,093	265	1,315
Max 2010-2019	6,326	2,700	2,693	1,248	1,145	369	1,470
	May	May	May	May	May	May	May
2019	5,851	2,539	2,502	1,148	1,147	244	1,391
2018	5,847	2,473	2,730	1,148	1,095	230	1,325
2017	5,539	2,461	2,484	1,121	1,063	277	1,340
2016	5,295	2,414	2,480	1,086	1,091	237	1,328
2015	5,129	2,250	2,272	1,101	904	245	1,149
2014	5,513	2,391	2,538	1,103	1,032	256	1,288
2013	5,349	2,420	2,307	1,103	1,033	284	1,317
2012	6,173	2,591	2,644	1,199	1,084	308	1,392
2011	6,050	2,613	2,484	1,137	1,142	334	1,476
2010	5,243	2,494	2,247	1,069	1,035	390	1,425
Max 2013-2019	5,851	2,539	2,730	1,148	1,147	284	1,391
Max 2010-2019	6,173	2,613	2,730	1,199	1,147	390	1,476
	June	June	June	June	June	June	June
2019	5,775	2,302	2,580	1,140	940	222	1,162
2018	5,777	2,460	2,656	1,172	1,057	231	1,288
2017	5,342	2,363	2,512	1,091	1,058	214	1,272
2016	5,368	2,377	2,498	1,095	1,058	224	1,282
2015	4,957	2,107	2,274	941	916	250	1,166
2014	5,547	2,434	2,334	1,106	1,038	290	1,328
2013	5,259	2,347	2,288	1,058	1,052	237	1,289
2012	5,979	2,662	2,515	1,197	1,178	287	1,465
2011	5,515	2,564	2,449	1,162	1,097	305	1,402
2010	5,162	2,426	2,204	1,032	982	412	1,394
Max 2013-2019	5,777	2,460	2,656	1,172	1,058	290	1,328
Max 2010-2019	5,979	2,662	2,656	1,197	1,178	412	1,465
	July	July	July	July	July	July	July
2019	5,590	2,472	2,387	1,142	1,078	252	1,330
2018	4,849	2,208	2,222	1,007	966	235	1,201
2017	5,314	2,222	2,383	1,017	973	232	1,205
2016	4,695	2,200	2,249	922	1,046	232	1,278
2015	5,080	2,057	2,058	951	849	257	1,106
2014	5,302	2,351	2,406	1,110	974	267	1,241
2013	4,773	2,028	2,135	896	882	250	1,132
2012	5,806	2,660	2,488	1,138	1,206	316	1,522
2011	5,906	2,574	2,472	1,138	1,134	302	1,436
2010	5,251	2,545	2,223	1,052	1,079	414	1,493
Max 2013-2019	5,590	2,472	2,406	1,142	1,078	267	1,330
Max 2010-2019	5,906	2,660	2,488	1,142	1,206	414	1,522

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Cana	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check
	August	August	August	August	August	August	August
2019	5,263	2,391	2,247	1,135	1,002	254	1,256
2018	4,922	2,097	2,273	928	956	213	1,169
2017	4,616	1,950	2,025	856	912	182	1,094
2016	4,324	1,948	2,040	773	927	248	1,175
2015	4,674	2,092	2,177	866	1,006	220	1,226
2014	4,359	2,103	1,929	966	882	255	1,137
2013	4,520	2,003	2,094	948	813	242	1,055
2012	4,731	2,011	2,235	922	856	233	1,089
2011	5,393	2,311	2,292	984	1,027	300	1,327
2010	4,914	2,188	1,943	870	949	369	1,318
Max 2013-2019	5,263	2,391	2,273	1,135	1,006	255	1,256
Max 2010-2019	5,393	2,391	2,292	1,135	1,027	369	1,327
	September	September	September	September	September	September	September
2019 m		2,090	2,089	983	921	186	1,107
2018	4,090	1,800	1,901	821	820	159	979
2017	4,253	1,920	1,799	871	844	205	1,049
2016	4,464	1,772	2,005	758	806	208	1,014
2015	4,389	1,796	1,901	850	736	210	946
2014	4,039	1,753	1,746	687	865	201	1,066
2013	4,557	1,713	1,955	733	815	165	980
2012	4,792	2,060	2,143	859	950	251	1,201
2011	5,142	2,243	2,021	991	992	260	1,252
2010	4,961	2,048	1,770	861	848	339	1,187
Max 2013-2019	4,557	2,090	2,089	983	921	210	1,107
Max 2010-2019	5,142	2,243	2,143	991	992	339	1,252
	October	October	October	October	October	October	October
2019	4,267	1,817	1,914	809	808	200	1,008
2018	4,527	1,800	2,064	804	841	155	996
2017	4,427	1,790	1,975	744	862	184	1,046
2016	4,166	1,721	1,796	747	800	174	974
2015	4,695	1,833	2,134	821	825	187	1,012
2014	2,823	1,800	1,545	769	793	238	1,031
2013	4,778	1,765	1,810	774	803	188	991
2012	4,696	2,090	1,893	924	895	271	1,166
2011	4,408	1,953	1,893	783	938	232	1,170
2010	4,297	1,649	1,599	641	772	236	1,008
Max 2013-2019	4,778	1,833	2,134	821	862	238	1,046
Max 2010-2019	4,778	2,090	2,134	924	938	271	1,170
	November	November	November	November	November	November	November
2019	3,258	1,426	1,573	547	720	159	879
2018	3,884	1,467	1,787	653	695	119	814
2017	3,606	1,526	1,462	687	683	156	839
2016	3,967	1,549	1,515	635	778	136	914
2015	4,111	1,539	1,581	651	729	159	888
2014	3,685	1,423	1,514	647	638	138	776
2013	3,474	1,459	1,313	619	679	161	840
2012	3,539	1,596	1,390	724	756	116	872

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Cana	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check
2011	3,608	1,538	1,462	601	770	167	937
2010	3,845	1,430	1,624	558	692	180	872
Max 2013-2019	4,111	1,549	1,787	687	778	161	914
Max 2010-2019	4,111	1,596	1,787	724	778	180	937
	December	December	December	December	December	December	December
2019	2,369	843	935	323	437	83	520
2018	2,781	1,068	1,228	396	589	83	672
2017	3,404	1,272	1,494	556	553	163	716
2016	3,108	1,139	1,303	445	550	144	694
2015	3,182	1,323	1,266	518	639	166	805
2014	2,352	1,005	1,207	386	452	167	619
2013	3,065	1,333	1,197	576	609	148	757
2012	2,640	1,202	1,100	455	570	177	747
2011	3,236	1,441	1,287	638	682	121	803
2010	3,100	1,553	1,316	648	743	162	905
Max 2013-2019	3,404	1,333	1,494	576	639	167	805
Max 2010-2019	3,404	1,553	1,494	648	743	177	905

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Canal	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check	Maximum CFS Discharge Delivered AAC	Maximum CFS Discharge Delivered AAC Above CM	Maximum CFS Discharge Delivered AAC Above WSM
J-19	2,766	978	1,254	354	517	107	624	482	375	107
F-19	3,709	1,394	1,670	634	634	126	760	630	504	126
M-19	4,534	1,998	2,000	940	872	186	1,058	825	639	186
A-19	5,673	2,429	2,594	1,121	1,065	243	1,308	1,045	802	243
M-19	5,851	2,539	2,502	1,148	1,147	244	1,391	1,049	805	244
J-19	5,775	2,302	2,580	1,140	940	222	1,162	1,042	820	222
J-19	5,590	2,472	2,387	1,142	1,078	252	1,330	1,036	784	252
A-19	5,263	2,391	2,247	1,135	1,002	254	1,256	1,063	809	254
S-19	m	2,090	2,089	983	921	186	1,107	823	637	186
O-19	4,267	1,817	1,914	809	808	200	1,008	820	620	200
N-19	3,258	1,426	1,573	547	720	159	879	699	540	159
D-19	2,369	843	935	323	437	83	520	490	407	83
J-18	2,785	1,216	1,166	460	647	109	756	584	475	109
F-18	3,537	1,645	1,786	667	812	166	978	718	552	166
M-18	4,970	2,132	2,179	994	921	217	1,138	930	713	217
A-18	5,448	2,421	2,513	1,123	1,068	230	1,298	1,023	793	230
M-18	5,847	2,473	2,730	1,148	1,095	230	1,325	1,027	797	230
J-18	5,777	2,460	2,656	1,172	1,057	231	1,288	1,033	802	231
J-18	4,849	2,208	2,222	1,007	966	235	1,201	932	697	235
A-18	4,922	2,097	2,273	928	956	213	1,169	906	693	213
S-18	4,090	1,800	1,901	821	820	159	979	751	592	159
O-18	4,527	1,800	2,064	804	841	155	996	795	640	155
N-18	3,884	1,467	1,787	653	695	119	814	715	596	119
D-18	2,781	1,068	1,228	396	589	83	672	529	446	83
J-17	2,612	757	1,030	287	404	66	470	498	432	66
F-17	3,315	1,570	1,449	652	762	156	918	858	702	156
M-17	5,074	2,278	2,113	995	1,057	226	1,283	929	703	226
A-17	5,418	2,399	2,412	1,098	1,093	208	1,301	1,048	840	208
M-17	5,539	2,461	2,484	1,121	1,063	277	1,340	976	699	277
J-17	5,342	2,363	2,512	1,091	1,058	214	1,272	1,036	822	214
J-17	5,314	2,222	2,383	1,017	973	232	1,205	987	755	232
A-17	4,616	1,950	2,025	856	912	182	1,094	968	786	182
S-17	4,253	1,920	1,799	871	844	205	1,049	785	580	205
O-17	4,427	1,790	1,975	744	862	184	1,046	785	601	184
N-17	3,606	1,526	1,462	687	683	156	839	723	567	156
D-17	3,404	1,272	1,494	556	553	163	716	694	531	163
J-16	2,958	1,253	1,030	530	579	144	723	602	458	144
F-16	4,643	2,060	2,180	1,009	840	211	1,051	863	652	211
M-16	5,439	2,340	2,262	1,043	1,063	234	1,297	913	679	234
A-16	5,135	2,279	2,417	1,042	973	264	1,237	992	728	264
M-16	5,295	2,414	2,480	1,086	1,091	237	1,328	985	748	237
J-16	5,368	2,377	2,498	1,095	1,058	224	1,282	1,072	848	224
J-16	4,695	2,200	2,249	922	1,046	232	1,278	966	734	232
A-16	4,324	1,948	2,040	773	927	248	1,175	922	674	248
S-16	4,464	1,772	2,005	758	806	208	1,014	757	549	208
O-16	4,166	1,721	1,796	747	800	174	974	802	628	174
N-16	3,967	1,549	1,515	635	778	136	914	736	600	136
D-16	3,108	1,139	1,303	445	550	144	694	645	501	144
J-15	2,859	1,175	1,236	465	568	142	710	578	436	142
F-15	4,038	1,791	1,722	804	742	245	987	853	608	245
M-15	5,092	2,429	2,204	1,081	1,075	273	1,348	989	716	273
A-15	5,567	2,426	2,269	1,114	1,047	265	1,312	1,154	889	265
M-15	5,129	2,250	2,272	1,101	904	245	1,149	984	739	245
J-15	4,957	2,107	2,274	941	916	250	1,166	972	722	250

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Canal	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check	Maximum CFS Discharge Delivered AAC	Maximum CFS Discharge Delivered AAC Above CM	Maximum CFS Discharge Delivered AAC Above WSM
J-15	5,080	2,057	2,058	951	849	257	1,106	1,031	774	257
A-15	4,674	2,092	2,177	866	1,006	220	1,226	818	598	220
S-15	4,389	1,796	1,901	850	736	210	946	802	592	210
O-15	4,695	1,833	2,134	821	825	187	1,012	808	621	187
N-15	4,111	1,539	1,581	651	729	159	888	803	644	159
D-15	3,182	1,323	1,266	518	639	166	805	735	569	166
J-14	2,679	1,114	1,047	450	525	139	664	634	495	139
F-14	3,778	1,610	1,667	636	777	197	974	813	616	197
M-14	4,774	2,200	2,044	976	931	293	1,224	945	652	293
A-14	5,476	2,332	2,251	1,052	1,025	255	1,280	1,061	806	255
M-14	5,513	2,391	2,538	1,103	1,032	256	1,288	1,061	805	256
J-14	5,547	2,434	2,334	1,106	1,038	290	1,328	1,111	821	290
J-14	5,302	2,351	2,406	1,110	974	267	1,241	1,081	814	267
A-14	4,359	2,103	1,929	966	882	255	1,137	878	623	255
S-14	4,039	1,753	1,746	687	865	201	1,066	812	611	201
O-14	2,823	1,800	1,545	769	793	238	1,031	787	549	238
N-14	3,685	1,423	1,514	647	638	138	776	737	599	138
D-14	2,352	1,005	1,207	386	452	167	619	549	382	167
J-13	3,127	1,202	1,361	456	628	118	746	638	520	118
F-13	3,804	1,584	1,376	660	716	208	924	850	642	208
M-13	4,956	2,170	2,072	974	981	215	1,196	943	728	215
A-13	5,655	2,342	2,596	1,027	1,076	239	1,315	1,108	869	239
M-13	5,349	2,420	2,307	1,103	1,033	284	1,317	1,083	799	284
J-13	5,259	2,347	2,288	1,058	1,052	237	1,289	1,047	810	237
J-13	4,773	2,028	2,135	896	882	250	1,132	996	746	250
A-13	4,520	2,003	2,094	948	813	242	1,055	914	672	242
S-13	4,557	1,713	1,955	733	815	165	980	666	501	165
O-13	4,778	1,765	1,810	774	803	188	991	853	665	188
N-13	3,474	1,459	1,313	619	679	161	840	711	550	161
D-13	3,065	1,333	1,197	576	609	148	757	686	538	148
J-12	3,188	1,399	1,321	599	668	132	800	683	551	132
F-12	4,807	1,805	1,941	726	912	167	1,079	876	709	167
M-12	5,767	2,452	2,403	1,099	1,057	296	1,353	1,118	822	296
A-12	**6,326**	**2,700**	**2,693**	**1,248**	**1,145**	**307**	1,452	1,236	**929**	**307**
M-12	6,173	2,591	2,644	1,199	1,084	308	1,392	1,188	880	308
J-12	5,979	2,662	2,515	1,197	1,178	287	1,465	1,133	846	287
J-12	5,806	2,660	2,488	1,138	1,206	316	1,522	1,157	841	316
A-12	4,731	2,011	2,235	922	856	233	1,089	949	716	233
S-12	4,792	2,060	2,143	859	950	251	1,201	1,168	917	251
O-12	4,696	2,090	1,893	924	895	271	1,166	986	715	271
N-12	3,539	1,596	1,390	724	756	116	872	674	558	116
D-12	2,640	1,202	1,100	455	570	177	747	687	510	177
J-11	3,283	1,381	1,485	588	656	137	793	693	556	137
F-11	3,531	1,588	1,439	618	799	171	970	731	560	171
M-11	5,693	2,328	2,450	975	1,066	287	1,353	1,034	747	287
A-11	6,100	2,622	2,620	1,152	1,111	359	1,470	1,243	884	359
M-11	6,050	2,613	2,484	1,137	1,142	334	1,476	1,235	901	334
J-11	5,515	2,564	2,449	1,162	1,097	305	1,402	1,102	797	305
J-11	5,906	2,574	2,472	1,138	1,134	302	1,436	1,109	807	302
A-11	5,393	2,311	2,292	984	1,027	300	1,327	1,094	794	300
S-11	5,142	2,243	2,021	991	992	260	1,252	952	692	260
O-11	4,408	1,953	1,893	783	938	232	1,170	869	637	232
N-11	3,608	1,538	1,462	601	770	167	937	718	551	167
D-11	3,236	1,441	1,287	638	682	121	803	712	591	121

	Maximum CFS Discharge AAC Station 2900	Maximum CFS Discharge AAC Drop 5	Maximum CFS Discharge East Highline Canal	Maximum CFS Discharge Central Main Canal	Maximum CFS Discharge Westside Main Canal	Maximum CFS Discharge AAC Above WSM	Maximum CFS Discharge New River Check	Maximum CFS Discharge Delivered AAC	Maximum CFS Discharge Delivered AAC Above CM	Maximum CFS Discharge Delivered AAC Above WSM
J-10	2,346	1,135	921	442	556	137	693	491	354	137
F-10	3,520	1,228	1,652	385	660	183	843	594	411	183
M-10	5,701	2,373	2,275	1,005	1,042	326	1,368	1,120	794	326
A-10	5,551	2,532	2,329	1,067	1,096	369	1,465	1,229	860	369
M-10	5,243	2,494	2,247	1,069	1,035	390	1,425	1,234	844	390
J-10	5,162	2,426	2,204	1,032	982	412	1,394	1,167	755	412
J-10	5,251	2,545	2,223	1,052	1,079	414	1,493	1,235	821	414
A-10	4,914	2,188	1,943	870	949	369	1,318	1,075	706	369
S-10	4,961	2,048	1,770	861	848	339	1,187	1,070	731	339
O-10	4,297	1,649	1,599	641	772	236	1,008	917	681	236
N-10	3,845	1,430	1,624	558	692	180	872	813	633	180
D-10	3,100	1,553	1,316	648	743	162	905	881	719	162
Max 2013-2019	5,851	2,539	2,730	1,172	1,147	293	1,391			
Max 2010-2019	6,326	2,700	2,730	1,248	1,206	414	1,522	1,243	929	414

C

GEOTECHNICAL DESKTOP STUDY - RCS ALIGNMENT 3A

December 4, 2019
Revision April 8, 2020
Kleinfelder Project No. 20201537.001A

Mr. John T. Bekmanis, PE
Senior Project Manager, Water
Black & Veatch
300 Rancheros Drive, Suite 250
San Marcos, California 92069

SUBJECT: Geotechnical Desktop Study
Regional Conveyance System Study (RCS) – Alignment 3A
San Diego and Imperial Counties, California

Dear Mr. Bekmanis:

Kleinfelder, Inc. (Kleinfelder) is pleased to present this geotechnical desktop study for the proposed Regional Conveyance System (RCS) study–Alignment 3A. The purpose of our services was to provide preliminary geotechnical and geologic information along the alignment in support of B&V's Pre-Design Study of the RCS.

This report presents the results of our background review, geologic reconnaissance, technical analyses, and conclusions regarding the geologic hazards and their impacts to the proposed alignment. We have also provided a preliminary scope and rough order magnitude (ROM) cost estimate to conduct a design-level geotechnical investigation for advanced design of the RCS. The ROM cost to conduct a design-level geotechnical investigation is based on our past experience in pricing and implementing geotechnical studies of similar size projects, comparable site conditions and design requirements for tunnels, cut and cover pipelines, and canal conveyance systems.

We appreciate the opportunity to be of service to you on this project and are prepared to provide the recommended additional services. Please call us if you have any questions concerning this report.

Sincerely,

KLEINFELDER

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GEOTECHNICAL DESKTOP STUDY
REGIONAL CONVEYANCE SYSTEM STUDY
ALIGNMENT 3A
SAN DIEGO and IMPERIAL COUNTIES, CALIFORNIA
KLEINFELDER PROJECT NO. 20201537.001A

DECEMBER 4, 2019
Revision April 8, 2020

A Report Prepared for:

Mr. John T. Bekmanis, PE
Sr. Project Manager, Water
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GEOTECHNICAL DESKTOP STUDY
REGIONAL CONVEYANCE SYSTEM STUDY – ALIGNMENT 3A
SAN DIEGO and IMPERIAL COUNTIES, CALIFORNIA

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December 4, 2019
Revision April 8, 2020
Kleinfelder Project No. 20201537.001A

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1 EXECUTIVE SUMMARY

Based on the results of our study, the proposed project is feasible from a geotechnical standpoint. The tunnel, pipeline, and parallel canal segments extend approximately 127.4 miles within San Diego and Imperial Counties, California. The western portion of the project will involve tunneling roughly 44.5 miles through a wide range of granitic and metasedimentary rocks at a maximum tunnel depth of up to approximately 3,500 feet beneath the Cleveland National Forest. The proposed cut and cover trench pipeline segment will be approximately 39.9 miles extending between Borrego Springs and SR 86 in Imperial Valley. The proposed canal segments parallel to the existing West Side Main and All American Canals will be roughly 43.0 miles in length.

The tunnel excavations are anticipated to be excavated utilizing TBM methods that will cross several faults and fault zones. Key issues for tunnel design and construction will involve the nature and characterization of the various rock masses, including rock strength, rock weathering and abrasivity, active fault crossings, poor ground conditions at fault crossings, hydrostatic pressures at tunnel depth, in-situ temperature, and control of groundwater inflows during tunneling. Although faults, groundwater and various geologic hazards are present along the alignment, they are not considered fatal flaws provided time in the construction schedule and reasonable costs are incorporated into the project.

The non-tunnel portions of the alignment are primarily in the eastern half of the project and are anticipated to encounter various alluvial soil types as well as local shallow bedrock. Key issues for the non-tunnel portions of the alignment include soil and bedrock excavatability, liquefaction potential, re-use of excavated materials for trench backfill, corrosion potential, shallow groundwater, canal side wall stability, and fault crossings.

2 INTRODUCTION

2.1 GENERAL

Kleinfelder has completed the authorized Geotechnical Desktop Study for the Regional Conveyance System Study project. This report includes the results of our geotechnical and geologic assessment of Alignment 3A, along with a rough order of magnitude (ROM) cost estimate for future geotechnical study and analysis which is included in Appendix A The ROM cost is based on our past experience in pricing and implementing field geotechnical investigations for projects of similar size, geotechnical settings and design requirements.

2.2 PROJECT DESCRIPTION

We understand the project consists of conveying Colorado River water from Imperial Irrigation District (IID) facilities in Imperial County to the Water Authority's 2nd Aqueduct system near the Twin Oaks Valley Water Treatment Plant in San Marcos. Alignment 3A will include approximately 44.5 miles of tunnel and 39.9 miles of cut-and-cover pipeline and/or canals and associated pumping systems. The pipelines will consist of a 102-inch diameter cement mortar lined and coated steel (CMLC) pipe. The alignment will traverse various federal, state, county and local agency right of ways.

Alignment 3A is one of three routes being studied by SDCWA and this report focuses on the northernmost alignment which had not previously been studied to this level geotechnically. The project alignment is shown on Figure 1, Project Alignment Map. A summary of Alignment 3A characteristics and facilities is provided below in Table 1.

Table 1

Summary of Alignment 3A Characteristics

Characteristic	Corridor 3A
Min. Surface Elevation (MSL)	-180
Max. Surface Elevation (MSL)	4,535
Canal Segment, mi.	43.0
Pipeline Segment, mi.	39.9
Tunnel Segment, mi.	44.5
Total Length, mi.	127.4

Characteristic	Corridor 3A
Pump Stations (PS)	3
Power Generating Facilities (PGF)	0
Pressure Control Facilities (PCF)	1

2.3 SCOPE OF SERVICES

Our scope of services included studying Alignment 3A to the same level of effort geotechnically as the two previously studied Southern Alignments 5A and 5C. Our services consisted of reviewing available and provided documentation pertaining to the geologic conditions, including results of prior geotechnical investigations conducted for the project. A field geologic reconnaissance was conducted to document exposed geologic conditions and terrain, observe various geologic units, and other features affecting site access and construction, and field checking of data compiled from the photographic analysis. We did not conduct any subsurface field explorations or laboratory testing during this phase of the project. Our services included preparation of this report and developing a recommended scope of work to conduct a subsurface geotechnical investigation for advancement of the design, along with rough order of magnitude (ROM) costs.

3 LITERATURE REVIEW

Background literature review for the project included research and review of existing documents pertinent to the geology in the area of Alignment 3A, including geology, soils, groundwater, available well logs, aerial photographs, flood information, landslide, faulting, and seismicity. The literature review included published and unpublished literature available in our files and the files of public agencies, including the United States Geological Survey, California Geological Survey, California Department of Water Resources, San Diego County, United States Bureau of Reclamation, Flood Insurance Rate Maps, and other documents compiled by Black & Veatch for the project.

Other documents researched and reviewed included published reports pertaining to historic tunnel projects and feasibility studies for tunnels in similar terrain, planning and design for tunnels, and constructability issues for tunnels in hard rock and faulted terrain.

A listing of documents reviewed and referred in this report is provided in the reference section of this report.

4 FIELD GEOLOGIC RECONNAISSANCE

Kleinfelder performed a geologic reconnaissance along accessible portions of the project tunnel, cut and cover and canal alignment. The reconnaissance was performed by two experienced geologists from our San Diego office between September 17 and 23, 2019. The first day was with members of the design team from Black and Veatch and personnel from the San Diego County Water Authority. The following days the Kleinfelder geologists performed geologic observation along the alignment. Most of observations were made from the roadways of cut and natural outcrops. Photos were taken of most of the stops along with notes of geologic materials, structure and conditions. This information was used along with review of published geologic maps, reports, topographic maps and aerial photography to perform our preliminary geologic interpretation of the geologic conditions along the alignment. Photographs taken during the field reconnaissance are included in Appendix B.

5 GEOTECHNICAL AND GEOLOGIC CONDITIONS

5.1 REGIONAL GEOLOGY

The project alignment is positioned within two separate geomorphic provinces as depicted on Figure 2, Regional Geologic Map. The western portion of the alignment which includes the entire tunnel segment and approximately 20 miles of the western side of cut and cover section are located within the Peninsular Ranges Geomorphic Province. The eastern portion of the alignment which includes the remainder of the cut and cover section and the entire canal section are located within the Salton Trough Geomorphic Province. This eastern province is also referred to as the Colorado Desert.

5.1.1 Peninsular Ranges Geomorphic Province

The Peninsular Ranges Geomorphic Province (CGS, 2002) is characterized as an assemblage of north-to-northwest-trending, high-relief ranges stretching south from the Santa Monica Mountains in Los Angeles, through San Diego County, and well into Baja California, Mexico. Some of the notable ranges of southern California include the Santa Ana Mountains, the Laguna Mountains, Mount Palomar and the Cuyamaca Mountains. The development of this mountain system is closely tied to the transform tectonics of the San Andreas Fault System (SAFS).

Locally within San Diego County this Province includes two geomorphic subzones, that are also aligned in north-to-northwest trending belts, roughly parallel to the Pacific coastline. On the west is the relatively narrow and low-relief coastal plain. On the east and comprising the majority of the province is the high-relief mountainous zone. The tunnel section of the alignment is positioned entirely within the mountainous subzone and the non-tunnel sections are positioned within the low-relief desert of the Salton Trough.

The mountainous subzone is 40 to 50 miles wide and is composed mostly of Cretaceous granitic rocks of the Southern California Batholith. The granites are inset with numerous isolated patches of Jurassic to Triassic metamorphic roof pendants that are remnants of the former sedimentary cover into which the batholith intruded. The batholith surface trends downward toward the west and underlies the sedimentary cover of the coastal plain. The Southern California Batholith developed during late Jurassic to Cretaceous time when the Farallon tectonic plate was undergoing subduction beneath the North American Plate. This style of plate tectonic interaction ultimately gave way to the transform tectonics of the current SAFS sometime after the Pacific Plate spreading center was consumed below the North American Plate. Topographic relief in excess of 6,000 feet above mean sea level (MSL) are common within the county with a high of approximately 4,535 feet MSL along the tunnel section at Angel Mountain near mile 58.7.

The desert subzone occurs along the eastern edge of the county and extends eastward into Imperial County. This desert basin developed in response to crustal extension and related faulting within the southeastern portion of the SAFS. Elevations at the eastern side of this subzone are approximately 1,000 feet MSL and descend relatively gently to the east. This area is typically underlain by relatively recent alluvial deposits and terrestrial sediments.

5.1.2 Salton Trough Geomorphic Province

The Salton Trough Geomorphic Province (CGS, 2002), is an elongated topographic and structural depression extending along a northwest/southeast alignment bound on the west by the Peninsular Ranges Geomorphic Province, the north by the Coachella Valley, the east by the San Andreas fault, and to the south by the Gulf of California, as presented in Figure 2, Regional Geologic Map. The Salton Trough is a region of transition from extensional tectonics of the East Pacific Rise to the transform tectonic environment of the SAFS (Powell, 1993). Late Cenozoic extension associated with the opening of the Gulf of California formed this deep structural depression.

The Salton Trough is an actively growing rift valley in which sedimentation has almost kept pace with basement down drop from tectonism. As rifting continued, the Colorado River Delta continued to fill the trough, and conditions gradually changed from marine, to deltaic, to subaerial (sediments deposited on land rather than under water) river and lake deposits. The northern end of the gulf was subsequently cut off by growth of the delta, resulting in the closed basin present today, a large portion of which is below sea-level. During the last 3.3 million years, the basin has undergone cycles of filling with freshwater lakes and desiccation as the Colorado River changed course, alternately flowing north or south (Alles, 2011). The maximum Cenozoic sediment thickness within the basin is approximately 15,000 feet The sediment is underlain at depth by Mesozoic age crystalline and metamorphic basement rock.

The last large ancient lake to fill the basin was Lake Cahuilla (Waters, 1981) which existed up to approximately 300 years ago. It formed by flooding of the Colorado River into the northern Salton Trough area. The lake was approximately 100 miles long by 40 miles wide. The old shoreline of Lake Cahuilla can be traced along the Santa Rosa Mountains to the north and has an average elevation of about +1040 feet above MSL. The Lake Cahuilla deposits represent the most recent in a series of Late Pleistocene to Holocene fresh to brackish water lakes which have occupied the closed Salton Trough.

The Salton Sea was accidentally created by the engineers of the California Development Company in 1905. In an effort to increase water flow into the area for farming, irrigation canals were dug from the Colorado River into the valley. Due to concerns of silt buildup, a cut was made in the bank of the Colorado River to further increase the water flow. The resulting outflow overwhelmed the engineered canal, and the river flowed into the basin for two years, filling the historic dry lake bed and creating the modern sea, before repairs were completed. The uncontrolled water flow incised two channels in the recent Pleistocene sediments which resulted in the formation of the New River and Alamo River west and east of El Centro, California. Both of these rivers flow north and empty into the Salton Sea.

5.1.3 Geologic Formations

Table 2 below lists the various geologic formations that underlie portions of the project alignment.

Table 2

Geologic Formations Along Alignment 3A

Formation Symbol	Age	Description	Rock Types
Qal	Late Pleistocene to Holocene	Unconsolidated alluvial stream, river and fan deposits	sand, gravel & conglomerate
Ql	Late Pleistocene to Holocene	Lake deposits of ancient Lake Cahuilla	clay, silt & sand
Qt	Pleistocene	Nonmarine terrace deposits	silt, sand & gravel
Qc	Pleistocene	Nonmarine sedimentary deposits	sand, gravel & fanglomerate
Pc	Pliocene	Undivided nonmarine sedimentary rocks	claystone, siltstone & sandstone
bi	Mesozoic	Basic intrusive rock	gabbro
gr	Mesozoic	Undivided intrusive granitic rocks	tonalite, granodiorite & diorite
gr^t	Mesozoic	Intrusive granitic rock	tonalite
gr^g	Mesozoic	Intrusive granitic rock	granodiorite
JTrv	Jurassic-Triassic	Metasedimentary rock	tuff, breccia, agglomerate, & volcanics
Ju	Jurassic	Marine sedimentary & metasedimentary rock	argillite, skate, graywacke & limestone
ms	Pre-Cretaceous	Metasedimentary rock	schist, quartzite & gneiss
gr-m	Pre-Cenozoic	Granitic & metamorphic rock	schist, diorite, gneiss, phyllite, gabbro & monzonite

5.1.4 Tunnel Section Geology

The tunnel section of the alignment occurs within the basement rock complex of the mountainous subzone. The geologic formations that are present along the alignment at tunnel depth include all the Mesozoic granitic and metamorphic rocks (bi, gr, gr^t, gr^g, JTrv, Ju, ms, and gr-m) listed in Table 1. Figure 3, Tunnel Alignment Sections, displays the approximate tunnel alignment and demarcates the nine profile sections shown in Figures 4-12, Tunnel Plan and Profile Segments. These profile segments depict the anticipated location of occurrence of these various geologic units along the alignment shown in Figure 3. We observed the surface conditions of many of these units during our geologic field reconnaissance. These units were typically moderately to highly weathered with few outcrops of slightly to fresh exposure. Many outcrops exhibited decomposition to a soil consistency. It should be noted that much of the tunnel is positioned deep within the rock mass where the weathering grade is expected to be in the slightly weathered to fresh condition. Weathered rock is anticipated within the near surface of the portal locations but will improve away from the openings. Much of the rock also exhibited appreciable fracturing and jointing.

The rock at the Tubbs Canyon T1 East Portal consist of a moderately to highly weathered granitic intrusive rock (gr). This unit also exhibits a planar foliation that dips 30 to 50 degrees to the east. This is in the direction of the tunnel opening and is considered a discontinuity of potential structural weakness.

The Intermediate Vent Shaft east of Lake Henshaw is mapped to be directly underlain by relatively young alluvial type sediments (Qc). This unit is relatively thin and is likely several tens of feet thick and likely overlies weathered tonalite (gr^t).

The I-15 T2 West and T3 East Portals occur within a low-lying drainage just west of the old Highway 395 fill embankment. This location is likely also underlain by unconsolidated alluvial deposits at least several tens of feet in thickness which in turn overlies weathered granodiorite (gr^g).

The T3 West Portal is near the base of a west descending hillside which likely is underlain by moderately to highly weathered granodiorite (gr^g) or metamorphic rock (Ju)

5.1.5 Cut and Cover and Canal Sections Geology

The cut and cover and canal sections of the alignment east of tunnel, all the way to the All American Canal (AAC) near the International border consist of a variety of relatively recent sediments consisting of various unconsolidated alluvial deposits, lake deposits (Qal, Ql, Qt and Qc) and consolidated Pliocene sedimentary rock (Pc). The alluvial deposits (Qal, and Qt) are thickest on the west in the Borrego area where they are several in excess of one hundred to several hundred feet in thickness and generally thin to the east toward the Salton Sea. These materials are typically comprised of coarse angular to subangular sands, gravels and conglomerate.

The lake deposits (Ql) occur toward the eastern side of the cut and cover section and along the entire length of the canal section. This unit was deposited during the Pleistocene from the several ancient lakes that once filled the Salton basin and consists of a variety of unconsolidated clay, silt and sand.

Both the Ql and Qal deposits overlie Pliocene age sedimentary rock (Pc) at depth. Along the alignment, this unit is comprised of beds of claystone, siltstone and fine sandstone with some gravel. Faulting has deformed and folded these units where they outcrop at the surface along the mid-portion of the cut and cover section near the San Jacinto fault.

Based on site observations and our prior experience in the vicinity, anticipated soil conditions within excavations parallel to the existing AAC are expected to consist of fine-grained, soft to medium stiff Ql materials. Toward the west end where the AAC becomes the West Side Main Canal (WSMC), the soils along the alignment transition to sandier, loose to medium dense Qal materials. The sandier Qal materials were deposited over the clayey and silty Ql materials and will be predominant soil type along excavations along the WSMC. However, some fine-grained silts and clay soils were also observed near the surface along the WSMC.

Ql and Qal materials can be excavated using conventional construction equipment and can be used as engineered fill soils for proposed canal embankments. Excavation of surficial soft or loose soils and subgrade preparation will be required prior to placing proposed embankments. Slope stability analysis of temporary and finished slopes should be conducted during advancement of the project design.

Permeability in the existing soils varies from low in the stiffer clay soils too high in the coarser grained sandy materials. Erosion potential is considered high, especially within unlined canals. In addition, washouts of proposed canal embankments may occur where the WSMC traverses existing natural drainage courses during high flow flash floods. Accordingly, proper drainage improvements to the natural grades should be incorporated into design of proposed canal embankments.

5.2 FAULTING

Southern California straddles the boundary between two global tectonic plates known as the North American Plate (on the east) and the Pacific Plate (on the west). Active faults associated with this plate boundary cross through some of the most densely populated and developed areas of southern California. The main plate boundary faults are the Imperial and San Andreas faults, which stretch northwest from the Gulf of California in Mexico, through the desert region of the Imperial Valley, crossing the San Bernardino region, and traversing up into northern California where it eventually trends offshore near San Francisco (Jennings, 1994; Jennings and Bryant, 2010).

Within southern California, the plate boundary is actually a complex system of numerous faults known as the San Andreas Fault System (SAFS) that span a 150-mile wide zone from the main San Andreas fault in the Imperial Valley westward to offshore of San Diego (Powell et al., 1993; and Wallace, 1990). This zone of faulting is depicted on Figure 13, Regional Fault Map and Earthquake Epicenters. The major faults east of San Diego (from east to west) include the San Andreas, San Jacinto, and Elsinore faults. Major faults through and west of San Diego include the Palos Verdes-Coronado Bank, San Diego Trough, San Clemente and Rose Canyon faults.

The project alignment is crossed by several faults spanning from the mid-portion of the tunnel section all the way to the east side at the canal section as depicted on Figure 13. Active faults cross the alignment at eight locations. The tunnel section is crossed by an active segment of the Elsinore Fault Zone just west of Lake Henshaw near mile marker 56.6. The cut and cover section is crossed by an active segment of San Jacinto Fault Zone near the airport at Ocotillo Wells. This fault section last ruptured during the 1968 Borrego Mountain Earthquake.

The canal section is crossed by six active faults. Four of these faults are short strands of the Elmore Ranch fault which ruptured as part of the 1987 Superstition Hills earthquake. These faults cross an approximate 3-mile span of the canal between Kane Springs and Elmore Ranch. The next active fault is the Superstition Hills fault which crosses the proposed canal section near Salt Creek Slough 2.5 miles west of Forrester Road. The sixth fault crosses the canal near the International Border. This fault is a short active segment associated with the rupture for the 2010 El Mayor-Cucapah Earthquake in Baja California.

5.3 SEISMICITY

The SAFS is a transform plate boundary dominated by right-lateral fault displacement (Wallace, 1990; Weldon and Sieh, 1985) with the Pacific Plate moving northwest relative to the North

American Plate. The significance of this lateral faulting is that transform plate interactions typically generate much smaller maximum magnitude earthquakes than convergent or subduction plate boundaries. Thus in southern California where expected maximum moment magnitudes for most faults are typically in the M7.0 to 7.5 range, with only a few faults (San Andreas fault, possibly some thrust faults of the Transverse Ranges) capable of generating earthquakes in the M8 range, such as the 1906 San Francisco and 1857 Fort Tejon earthquakes, on the San Andreas fault itself.

Most of the seismic energy and associated fault displacement within the SAFS occurs along the fault structures closest to the plate boundary (i.e., on the Elsinore, San Jacinto, and San Andreas faults) (Powell et al., 1993). Approximately 49 millimeters/year (mm/yr), or about 1.9 inches/year, of overall lateral displacement has been measured geodetically as fault slip across the plate boundary. Combined, the Elsinore, San Jacinto, and San Andreas faults account for up to approximately 41 mm/yr (1.6 inches/year) of the total plate displacement (84 percent). The remaining 8 mm/yr (0.3 inches/year) or 16 percent is accommodated across the faults to the west (Bennett et al., 1996). At the latitude of San Diego, most of this, about 5-8 mm/yr, is accommodated by the coastal and offshore system of faults, including the Rose Canyon fault (Rockwell et al., 1991).

Figure 13, also displays the epicenters of regional earthquakes from 1800 to the present in excess of 4.0 Mw. A multitude of earthquake events are shown throughout the Salton Trough between the San Jacinto Fault Zone and San Andreas Fault Zone. This area of California is one of the most active seismic regions in the country and project will experience a variety of local and regional earthquake events during its lifetime.

6 GROUNDWATER

6.1 DESERT ALLUVIAL GROUNDWATER

The project alignment within the desert subzone is a shallow cut/cover conveyance crossing several alluvial valleys and groundwater basins, including the Imperial Valley (California Department of Water Resources (DWR), 2004a), Ocotillo Valley (DWR, 2004b) and the Borrego Valley (DWR, 2004c). The primary aquifer system in each of these groundwater basins consists of Quaternary alluvium of Pleistocene to Holocene age, and to a lesser extent, Tertiary alluvium primarily found underlying the Quaternary deposits. These alluvial deposits are composed largely of unconsolidated to semi-consolidated layers of gravel, sand, silt, and clay, generally ranging over 1,000 feet thick.

Within the project alignment the Imperial Valley Groundwater Basin extends from the southern terminus to the Salton Sea. Much of the land surrounding the Salton Sea is below present sea level due to crustal thinning and subsidence. As a result of this subsidence, the Salton Trough, and Imperial Valley has been filled with alluvial sediments shed from the local mountain ranges, as well as the ancestral and modern Colorado River. The Imperial Valley basin contains up to 2,000 feet of Tertiary to Holocene alluvium comprising two major aquifers. The upper aquifer is 200 to 450 feet thick and is underlain by a 60- to 280-foot thick aquitard (DWR, 2004a). At the southern end of the alignment the groundwater is reported by the U.S. Geological Survey (USGS) to be from 31 to 100 feet below the ground surface (bgs) (USGS, 2019). As the project alignment nears the Salton Sea the depth to groundwater shallows to 6 to 8 feet bgs and is of poor quality (Dynamic Consulting Engineers, 2010). Locally, displacement of alluvial layers along the San Jacinto and Superstition Hill faults can affect the depth of the groundwater as these faults form a barrier to groundwater flow.

To the west of the Salton Sea, the project alignment turns westward crossing the Ocotillo Valley and Borrego Valley Groundwater Basins (DWR, 2004 b and c). These two groundwater basins have a similar geology and geologic history but are separated by the active San Jacinto fault (Borrego Mountain section). The fault intersects the project alignment at Ocotillo Wells. These two groundwater basins are alluvium-filled valleys underlain by crystalline bedrock. The alluvium comprises three aquifers, an upper, middle, and lower. The upper aquifer is comprised of Holocene to Pleistocene age alluvial, fan, playa and eolian deposits, and ranges up to 1,000 feet thick at the north end of the basin (DWR, 2004c). Specific yields for these deposits range from 15 to 25 percent (DWR, 2004c). This upper aquifer is the principal source of groundwater in Borrego Valley and well yields are as much as 2,000 gallons/minute (Mitten, et al., 1988). The middle and lower aquifers are Pleistocene age and include moderately consolidated layers of sand, gravel, and boulders. Thickness ranges to 700 feet for the middle aquifer and the lower aquifer reaches 1,800 feet thick (Mitten and others 1988). Groundwater in the upper aquifer is approximately 205 feet bgs in the Ocotillo Valley Groundwater Basin (DWR, 2019), which is a drop of about 125 feet from an historical high of 80 feet bgs. The upper aquifer groundwater continues to deepen as the project alignment crosses the Borrego Valley Groundwater Basin. Across Borrego Valley the groundwater ranges from approximately 241 feet to 312 feet bgs. Groundwater over-usage in this basin has stressed the aquifer affecting its quality and depleted its groundwater storage (Faunt, et al., 2015).

6.2 TUNNEL HYDROGEOLOGY

The central mountain subzone is 40 to 50 miles wide and is composed mostly of Cretaceous-age igneous rocks of the Peninsular Range Region. The igneous are inset with numerous isolated patches of Jurassic to Triassic metamorphic rocks that are remnants of the former sedimentary cover into which the batholith intruded. The igneous rocks include mostly granites, granodiorite, diorite, tonalite, and gabbro. The intense heat associated with these plutonic magmas metamorphosed the ancient sedimentary rocks into which the plutons intruded. These metasedimentary rocks are now preserved in the Peninsular Ranges, and along the project alignment as slates, schist, quartzites, and gneiss. The western portion of the project alignment extends from the Tubbs Canyon Portal (Approximate PM 37.9) on Figure 12 to the Twin Oaks Valley Portal at approximately PM 82.3.

The groundwater potentiometric surface generally mimics the topography as a subdued expression of the ground surface; that is, although the depth to groundwater is nearest the canyon bottoms and is generally deeper beneath the ridgelines and mountain peaks, hydraulic gradients are from the ridges to the valleys. This is generally the case in all crystalline and metamorphic rock terrains, where steep hillsides facilitate rapid runoff of precipitation to canyon bottoms, where water is directed as runoff to larger tributaries. Infiltration is generally less on hillsides and more within canyons and valleys, where the flow gradients are lower and residence time is greater.

The interaction between surface water and groundwater systems is governed largely by lithology, geologic structures (e.g., faults, joints, unconformities, etc.), weathering conditions, and in-situ stress. Conceptually, groundwater flow within a rock mass occurs in two possible ways: 1) primary porosity of the rock matrix, and 2) secondary porosity such as fractures, although certain types of joints can develop with cooling of a rock mass so would be primary porosity features. Hydraulic conductivity (K) describes the ability of a formation to transmit water, and the connected pore space represents the effective porosity. In general, the hydraulic conductivity of a rock mass tends to decrease with depth coinciding with reduction in weathering effects, fewer discontinuities, and increasing lithostatic pressure.

Primary porosity is the connected void spaces of the intact rock, i.e., spaces between grains and cement or interlocking crystalline minerals comprising the rock. In poorly cemented, granular sedimentary rock, the primary porosity can be comparable to that of unconsolidated sediments. Conversely, for well-cemented or fine-grained sedimentary, metamorphic, and crystalline igneous rock, the primary porosity is low and reduces or prevents water transmission. Weathering processes alter the primary porosity of all rocks. Where cement or crystalline minerals are removed, the primary porosity could increase. In most cases, it is assumed that weathering of crystalline rock tends to increase their primary porosity by altering rock chemically, accentuating defects in the rock (i.e. fractures), and general opening of discontinuities.

Secondary porosity is the connected void spaces formed from discontinuities (e.g., joints, shears, faults, fractures, bedding, etc.) and geologic structures. Rock mass with persistent discontinuity systems with wide apertures open or infilled with coarse material will have a high secondary porosity. In some cases, such conduits may be further enhanced over time as flow occurs, water pressures build acting to prop open the fracture, finer-particles infilling the system are flushed away, and weathering of the surrounding intact rock walls increases their local primary porosity. The orientations of the discontinuities are also important. In general, near-vertical discontinuities often are better connected to the surface as the normal stress that reduces the joint opening tends to be lower in a gravitational stress field than the normal stress acting on near-horizontal discontinuities. At some critical depth, the state of stress becomes so great that fracture openings are inhibited or eliminated altogether.

Depending on the style of faulting, lithology, net displacement, and other factors, faults typically impose a high degree of anisotropy to groundwater flow. In most cases, faults act as a barrier to flow across the fault, and as a conduit for flow parallel to the fault. These established relationships are suggested within the tunnel portion of the project alignment based on the current geologic maps.

The hydrogeology of the project alignment is dominated by water flow along fractures in the igneous and metamorphic bedrock terrain of the mountains. The bedrock comprising the majority of the tunnel portion project alignment contains little to no porosity within the rock mass in which groundwater could reside. However, the mountains have been subjected to numerous episodes of deformation, including faulting and uplift, which has fractured the bedrock. This fracturing and faulting have created fissures within the bedrock that allows groundwater to reside and move through the rock mass along preferred fracture systems. Several of these faults exists at PM 41.8, PM 45.5, PM 46.8 (San Felipe fault) PM 49.6 to 57.1 (Elsinore Fault Zone), and PM 62.2. As the water is stored in the fractured bedrock it will move down through fractures (and faults) if the fractures are interconnected and have sufficient capacity to accommodate the water volume (see jointed zone, PM 66.1 to 66.6). If the water-laden fracture/fault intersects a canyon wall, the groundwater will discharge at the ground surface as a spring or seep and join with surface runoff in the canyon's stream.

6.3 GROUNDWATER SUMMARY

The analyses of groundwater conditions within the 125-mile long project alignment is based on limited field data and existing published hydrogeological studies. Preliminary interpretation of the hydrogeology of the project alignment indicates a high likelihood of proposed tunnel construction affecting groundwater and possibly surface water resources, if these impacts are not properly planned for and mitigated during the design and construction phases of the project. The distribution of the effects cannot be predicted at this time, but the hydrogeologic model should represent the groundwater flow along faults as being favored both vertically and horizontally. In general, the water chemistry of the surface water differs from the deep groundwater at tunnel depth, which suggests that the deeper groundwater at the tunnel depths is not hydraulically connected to surface waters. Although the chemistry differences suggest hydrological separation, the information does not preclude impacts to shallow groundwater and surface water if water is allowed to drain in an uncontrolled manner into a deep tunnel excavation.

Tunneling will tend to provide a conduit for groundwater to drain into the excavation as the advancing tunnel intersects fractures and faults within the crystalline rock terrain. Based on the general experience of the groundwater system within the crystalline bedrock, the near-surface water resources near fault zones will most likely respond more rapidly to annual precipitation and may respond to tunnel construction. Deeper groundwater systems may be less well connected to the shallow groundwater zones and may respond to precipitation recharge more slowly, although shallow and deep zones may be connected along fault zones, which can act as vertical conduits to groundwater flow. The magnitude of potential impacts to shallow groundwater resources and surface water would depend upon the total volume of groundwater that flows into the tunnel during construction and the potential rate of recharge due to precipitation. Since the deeper rock zones generally exhibit lower hydraulic conductivity than shallower zones, recharge from shallow zones vertically downward will likely exceed the rate of drainage/leakage from rock mass surrounding the tunnel lining.

Supplemental data from groundwater monitoring of water wells, springs and seeps within the tunnel hard-rock aquifer and the desert groundwater basins has been completed as a joint effort between the USGS and the California State Water Resources Control Board for the California Groundwater Ambient Monitoring and Assessment (GAMA) Program. The GAMA program is designed to provide unbiased assessment of untreated, shallow groundwater quality, with its main goal to improve groundwater monitoring and to increase the availability of groundwater-quality data to the public. Within the project alignment, the GAMA program collected groundwater data from wells and springs between December 2008 to March 2012 (Davis and Shelton, 2014; Parsons et al., 2014; and Wright and Belitz, 2011). The groundwater collected during the GAMA program was tested for; standard field water-quality parameters, volatile organic compounds, pesticides, perchlorate, inorganic constituents (ions, nutrients, dissolved solids, arsenic, chromium, iron, etc.), and radioactive constituents (tritium, radon, uranium isotopes, carbon-14). This testing provides a baseline for comparison to future groundwater chemistry of the project alignment.

7 GEOLOGIC HAZARDS

7.1 FAULT RUPTURE

The project alignment crosses through one of the most heavily faulted regions of California within the heart of the San Andreas Fault System. Active faults associated with the Elsinore Fault Zone, San Jacinto Fault Zone, Elmore Ranch fault and Superstition Hills fault all cross the project alignment. As such, fault rupture is a notable hazard for the proposed project and design measures will be necessary to mitigate against this condition. The specific locations of active fault crossings occur at the tunnel section just west of Lake Henshaw on the main structure of the Elsinore fault, at the San Jacinto fault along the central portion of the cut and cover pipeline section at Ocotillo Wells, several small fault segments of the Elmore Ranch fault along the canal section south of Kane Springs, the Superstition Hills fault of the canal section at Salt Creek Slough and at the southern end of the canal section near the international border.

7.2 SEISMIC SHAKING

Like all of southern California, the project alignment is located within a seismically active area and will experience ground shaking as a result of earthquakes on nearby or distant faults. The seismic exposure of the alignment is dominated by the several faults within the eastern San Andreas Fault System and include the Elsinore Fault Zone, the San Jacinto Fault Zone, the Imperial fault and the San Andreas Fault Zone.

7.3 LANDSLIDES AND SLOPE STABILITY

Landslides are deep-seated ground failures (several tens to hundreds of feet deep) in which a large arcuate or block shaped section of a slope detaches and slides downhill. Landslides can cause damage to structures both above and below the slide mass. Several formations within the Southern California region are particularly prone to landslides. These formations generally have high clay content and mobilize when they become saturated with water. Hard rock material such as granite and metamorphic rock can also be prone to landslide slope failure on steep slope along discontinuity surfaces typically in highly fractured zones. These failures are generally of the block failure type. The tunnel section of the alignment is composed of moderate to high relief topography which could be prone to landsliding in certain areas. Areas of most concern to the project would be at the portal locations as these areas will penetrate the ground surface at areas of topographic relief. Our review of the available geologic maps do not show mapped landslide terrain within or nearby the portal locations. The Tubbs Canyon T1 East Portal is an area of relatively steep topographic relief. Additionally, the texture of the granitic rock has a planar foliated fabric that dips to the east in the same direction of the slope. These conditions would make the site more prone to slope instability given the relief and structural discontinuities. The conditions at the T2 and T3 portals on the west end of the tunnel consist of weathered granitic rock of moderately steep topography. The susceptibility to landslide failure at these locations would be moderate to low.

The cut and cover and canal section of the conveyance alignment crosses through gently sloping to relatively flat topography that is not typically prone to landslide failure and would therefore have a nominal hazard with respects to landsliding. Temporary slope stability with respects to construction of the pipeline sections and canal work will need to be analyzed to determine safe slope cut gradients.

7.4 LIQUEFACTION AND SEISMIC SETTLEMENT

Liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confinement, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and some silts. Liquefaction is most prevalent in loose to medium dense, sandy and gravely soils below the groundwater table, but can also occur in non-plastic to low plasticity finer grained soil.

Another type of seismically induced ground failure that can occur as a result of seismic shaking is dynamic compaction, or seismic settlement. Such phenomena typically occur in unsaturated, loose granular material or poorly compacted fill soils. The alluvial soils along the west and east side of the cut and cover section are of the characteristic prone to seismic settlement.

8 CONSTRUCTABILITY

8.1 KEY CONSTRUCTABILITY ISSUES FOR RCS TUNNEL

The primary emphasis of this tunnel constructability evaluation is to identify, describe, and quantify challenging technical constraints that may impact tunnel design and construction for Regional Conveyance System (RCS) study. Since the RCS includes a tunnel beneath the Peninsular Ranges including the Cleveland National Forest, and the Anza Borrego Desert State Park, the conditions for tunneling will include conditions associated with deep tunnels through crystalline and metamorphic rock mountains. Such conditions have been encountered in other tunnels in Southern California and include extremely high groundwater pressures, high temperatures, high groundwater inflows and unavoidable impacts to water resources. Other challenging conditions may include severely unfavorable geology, such as wide fault zones, squeezing ground and intersections with active faults. Active faults intersecting the tunnel at depth is a constraint that requires special design and construction methods that are briefly addressed in this section. Any one of these conditions or a combination of the conditions can represent design or construction challenges that need careful evaluation and planning to minimize the constructability constraints, avoid significant delays and protect construction workers. The most challenging conditions are related to groundwater pressures, high temperatures, squeezing ground and high groundwater flows and are expected in the areas where the tunnels are deepest below the ground surface. Thus, the depth of the tunnel below ground surface impacts constructability.

A constructability evaluation assimilates and interprets available geotechnical data for a preliminary tunnel alignment passing beneath the Peninsular Ranges from Tubbs Canyon near Borrego Spring Road to the San Marcos area. The tunnel location is shown on Figure 12. For this constructability evaluation, key topics of this section are as follows:

- Constructability and Ground Conditions (rock mass conditions, weathering);
- Tunnel Design and Construction issues (hydraulic head, flush water flows and sustained flows, hydraulic conductivity, temperature, and fault displacement);
- Hydrogeologic Conditions and Impacts on Water Stakeholders;
- Construction Difficulties (Groundwater flow controls, Fault Zones, and state of rock stress); and
- Mitigation Methods for Tunneling.

To assist in the interpretation of the RCS Tunnel constructability the following information is considered:

- Summaries of four case histories of tunnel construction in Southern California mountain ranges;
- Two feasibility studies conducted in the Santa Ana Mountains; and
- Evaluating and interpreting available geotechnical data geological/geotechnical data along the RCS 3A tunnel alignment. (Geologic Profiles) to locate potential constructability concerns.

Because there are no direct field data currently available along the alignment, the primary source of geotechnical data for constructability purposes is derived from case history information of four tunnel construction projects, and general experience of tunneling within the Peninsular Ranges Province of California and other southern California mountain ranges including the San Bernardino Mountains, Santa Ynez Mountains, and Santa Ana Mountains.

8.2 HISTORICAL TUNNEL PROJECTS AND FEASIBILITY STUDIES

Historical tunnel projects in Southern California stand as examples of tunnel conditions that are typical and have served as the basis for many mitigation requirements for tunnel design, safety regulations, and construction methods in the industry. The United States Forest Service's (USFS) past experiences with these tunnel projects have been very influential in the developing the concerns that the USFS, California State and Native American People now express regarding impacts of constructing tunnels beneath forest lands (i.e. Cleveland National Forest and San Bernardino National Forest) and Native American lands. Significant case histories are summarized below covering a long period of tunnel industry development, evolution of design and construction methods and general industry changes with respect to constructability constraints. Conditions encountered during construction of these tunnels is likely directly applicable to construction of the RCS tunnel. The geotechnical conditions of the five case histories are summarized to reflect the changes in public perception, agency (USFS) concerns, private landowners with water rights, and Native Americans and impacts on their land holdings or reservations. The six case histories include the San Jacinto tunnel, Tecolote tunnel, Central Pool Augmentation tunnel feasibility study, the Cowles Mountain tunnel, the Arrowhead East tunnel, and Irvine-Corona Expressway tunnels. Case histories of construction for the San Vicente Tunnel and the Authority's first aqueduct tunnels were reviewed and found to be too shallow and too different conditions to represent the construction difficulties anticipated for the RCS Alignment 3A such as high water pressures, high ground pressures and squeezing ground, high temperatures, wide faults zones and active faults. Therefore, those tunnels are not included in this discussion.

In addition to four historical tunnel construction case histories, two tunnel feasibility studies are reviewed for comparison of feasibility conditions recognized during field investigations. Both of these feasibility studies are within the northern end of the Peninsular Ranges and within the Cleveland National Forest.

8.2.1 San Jacinto Tunnel (1933-1939), San Jacinto Mountains, San Bernardino National Forest and Mt. San Jacinto State Park

On April 8, 1933, Metropolitan Water District (MWD) started construction of the San Jacinto Tunnel, a critical link in the Colorado River aqueduct bringing water from the Colorado River to Lake Matthews for distribution to southern California water users. The San Jacinto Tunnel was driven 13 miles through the San Jacinto Mountains with two headings, one starting near Cabazon in Banning Pass on the east and the second beginning near Potrero Ranch on the west using drill and blast techniques for mining an 18-foot-diameter tunnel through the predominantly granitic rock. The maximum rock overburden was approximately 2,600 feet above the tunnel crown. Unstable sections were supported with horseshoe and circular steel sets and gunite for temporary tunnel support. Some sections of the tunnel were self-supporting and not lined initially.

During construction, significant water inflows were encountered, the first being only 160 feet east of the 815-foot deep vertical Protrero shaft, where the western tunnel heading encountered the first of many faults to be intersected along the tunnel alignment. Water flowing into the tunnel was estimated at 7,500 gallons per minute (gpm) and rapidly filled the excavation and rose up the Protrero shaft access to within 150 feet of ground surface while workers fled the tunnel to safety at the top of the shaft. A detailed geologic study was initiated in 1935 to evaluate whether additional faults could be encountered along the alignment. Results of geologic mapping discovered a total of 21 northwest-trending faults with northeast dip were along the planned tunnel path with some other east-west trending faults being mapped. As the tunnel construction advanced, miners discovered that the occurrence of water was on the northeast side of the faults within interconnected joints and fractures associated with the faults. The narrow fault gouge (clay) acted as a diaphragm that prevented water flow across the fault and acted as a groundwater barrier. As tunnel headings neared the fault zones, the jointing and fracturing in the rock became more frequent and closely spaced carrying more water.

The maximum instantaneous flush flow reached 16,000 gpm and included 3,000 cubic yards of sand flowing into the tunnel. Cumulatively, the peak water flow from all headings was approximately 40,000 gpm. Maximum in-situ groundwater pressures were measured as high as 600 pounds per square inch (psi) with the typical groundwater pressures reading 150 to 300 psi. Efforts to reduce the groundwater pressures during construction and shut off the water flow included driving pioneer tunnels parallel to the main tunnel to reduce groundwater pressures and perform pressure grouting ahead of the tunnel excavation by injecting cement into drill holes at pressures up to 1,500 psi. These measures helped for construction advancement.

By the second year of construction, 1935, the San Jacinto water seepage had become a concern for the San Jacinto Valley landowners as springs, creeks and streams that once flowed year-round at the edge of the mountains began to dry up. Groundwater levels in wells of San Jacinto Valley began dropping, but this was also during a period of increasing water diversions and well pumping for agriculture in the valley, so the causal relations to the tunneling and decline in groundwater levels in the valley were not clear. At the same time, many springs on the Soboba Reservation higher than the tunnel elevation had dried up and streams began to be intermittent. In October of 1935, the Riverside County Board of Supervisors (RCBS) passed a resolution demanding that MWD prevent further water losses into the tunnel. In response to the RCBS and several ensuing lawsuits, and upon completion of the tunnel excavation in 1938, MWD began grouting the leaking cracks in the tunnel and lined the tunnel with concrete reducing the size of the tunnel to 16 feet tall and 16 feet wide. The total seepage was reduced to 540 gpm. But as water levels began to recover, groundwater pressures began to build up behind the concrete, and leakage into the tunnel increased. By 1940, the leakage into the tunnel was back up to approximately 8,000 gpm. Again in 1944, a citizen's group, San Jacinto River Protective Committee, filed four new lawsuits against MWD. MWD ultimately responded by implementing a new grouting program to seal the tunnel by drilling holes through the tunnel lining 2 to 15 feet into rock, and pressure grouting the rock behind the concrete. Oversight was provided by the San Jacinto River Conservation District (formerly the Protective Committee). Upon completion of the grouting work in 1947, leakage continued at a rate of approximately 2,500 gpm. Both sides agreed that not all of the leakage could be stopped.

By 1946, all of the springs and streams on the Soboba Reservation had dried up. In 1950, the Soboba people filed litigation in the Indian Claims Commission against the United States for failing to protect the Soboba Reservation water resources. Negotiations continued until 1991, resulting in a compromise settlement of claims against the United States. Soboba invited MWD for direct negotiations with respect to the tunnel leakage in 1998 and again in 2000. With MWD declining, Soboba filed suit against MWD resulting in decade of negotiations that resulted in Soboba Reservation receiving an allocation to water rights and construction of water projects for the Soboba people.

8.2.2 Tecolote Tunnel (1950-1956), Santa Ynez Mountains, Los Padres National Forest

Tecolote Tunnel was constructed through the Santa Ynez Mountains beneath the Los Padres National Forest to tap into Lake Cachuma on the Santa Ynez River for transporting water to the City of Santa Barbara on the south side of the mountains. It was constructed by the United States Bureau of Reclamation (USBR) using drill and blast methods and supported by steel 6-inch horseshoe H-beam ribs with sheet metal panning and wood lagging for interim support. The tunnel diameter was 7 feet and the length was 6.4 miles. The maximum rock cover over the tunnel was approximately 2,300 feet. The tunnel was driven through a thick sequence of Tertiary- and Cretaceous-age sedimentary rocks. North of the Santa Ynez Mountain crest, the sedimentary rocks are cut by a major left lateral reverse slip fault known as the Santa Ynez fault, which extends for 90 miles east-west within the Transverse Ranges. The tunnel crosses the Santa Ynez fault at a nearly perpendicular angle. Other smaller faults were mapped throughout the tunnel. The faults resulted in zones of squeezing ground, which overstressed and deformed the tunnel supports requiring replacement several times during construction. Steeply dipping north-south trending joint sets were prevalent in the Cretaceous and Eocene sandstone and siltstone and served as the principal sources of water flow into the tunnel excavation. In addition to large volumes of water, many of the sedimentary formations carried methane gas with minor hydrogen sulfide gas escaping from solution in the groundwater. Several explosions occurred as a result of the methane gas accumulation due to inadequate ventilation of the tunnel.

Groundwater inflows to the tunnel caused the most difficulty for tunnel construction. Smaller flows in the range of 400 to 1,000 gpm under 375 psi were able to be sealed off by grouting. Control of the largest sustained flow ranging from 1,200 to 2,800 gpm for 16 months required a combination of grouting and advancing 3-foot by 5-foot drainage drifts parallel to the main tunnel to control water flows at the tunnel heading. Grouting pressures ranged from 1,300 to 2,000 psi to overcome static water pressures of 230 to 250 psi. Peak water flows at the outlet (south) portal were 9,100 gpm, which decreased to 6,500 gpm when the north and south tunnel headings holed through. The maximum water temperatures were 117 degrees Fahrenheit (°F), which necessitated miners to cool down in mine carts filled with cool water to keep working.

In advance of tunnel construction, a water resources monitoring program was implemented in order to identify and quantify potential effects of draining water from Tecolote tunnel during construction on local springs and spring fed streams in the San Ynez Mountains. In all, flows of 125 springs and streams were measured monthly from 1948 through 1951 as a calibration (baseline readings) and continued through construction. During the monitoring period, 18 measuring sites increased in flow in direct response to the July 21, 1952 Arvin-Tehachapi earthquake. Measured flows also increased in succeeding years after the September 1955 Refugio fire possibly in response to losses of evapotranspiration in the burned area. During the Tecolote Tunnel construction, one spring “failed”, which is the source of Hot Spring Creek. Other than the one spring being affected by the tunnel construction, there was no evidence that the tunnel construction affected other springs or spring-fed streams being monitored (Rantz, S. E. 1962).

8.2.3 Central Pool Augmentation Project Feasibility Study (2006-2008), Santa Ana Mountains, Cleveland National Forest

Since the 1980s, MWD has carried out several Central Pool Augmentation Project (CPA) studies to evaluate the environmental impacts and feasibility of a water delivery system from Lake Matthews to their Central Pool Service Area in Orange County including a tunnel through the northern Santa Ana Mountains (Woodward-Clyde Consultants, 1989, 1992a and 1992b; MWD, 1992 and 1994; Kleinfelder, 2008a). These studies characterized the general geologic and hydrogeologic conditions of the study areas; evaluated geotechnical and construction considerations with respect to the proposed tunnel; identified possible environmental impacts of tunneling through the Santa Ana Mountains and provided recommendations for mitigating those impacts; provided alternate alignments and locations for the proposed tunnel, pipeline, and water treatment plant; and provided preliminary cost estimates for the construction of the tunnel.

The CPA tunnel project is part of a water delivery system that includes a pipeline from Lake Matthews in western Riverside County to a delivery destination in southeastern Orange County. The project includes a water treatment plant in Eagle Valley, pipeline conveyance across Temescal Valley, and a treated water tunnel beneath the Santa Ana Mountains and the Cleveland National Forest. The 12-foot-diameter pipeline-tunnel alignment extends 10 miles through the between east and west portals. The east portal is planned in Bedford Canyon, and the west portal is on a ridge above Agua Chinon Wash in Orange County. The pipeline gradient from east to west would support gravity flow of water to avoid the need for active pumping.

From 2006 to 2008, Kleinfelder conducted a field investigation of the design and construction feasibility for the pipeline alignment through the northern Santa Ana Mountains. The field investigation was conducted under a Special Use Permit issued by the USFS allowing field investigations within the Cleveland National Forest. Key areas of concern addressed in this study included potential high pressures from groundwater (anticipated up to 2,000 feet of groundwater head) and relatively unknown rock mass conditions at the tunnel invert. The objectives of the field program were to: 1) Recover and characterize rock core from the deepest section of the tunnels beneath the mountains; 2) Measure hydraulic conductivity of the in-situ rock mass from shallow depths down to the tunnel invert; 3) Install vibrating wire pressure transducers (VWPT) to measure in-situ water pressures and temperatures at varying depths to the tunnel invert under maximum ground cover in excess of 2,500 feet below Bedford Ridge; 4) Evaluate if there is connection between deeper and shallower water within the rock mass; 5) Measure in-situ orientations of rock discontinuities, and 5) Characterize the rock discontinuities, strength, and groundwater according to industry-wide classification systems used for tunnels.

Two continuous vertical rock core holes were completed to 2,200 and 2,500 feet, all within the Bedford Canyon Formation, a sequence of alternating meta-sandstone and meta-shale (argillite, slate, and pebbly mudstone) turbidite sediments deposited during the Jurassic Period. Five VWPTs were installed in each of the core holes at approximate 500-foot intervals for measuring the maximum groundwater pressures at the depth of each VWPT and monitoring changes that might occur with variations in seasonal and longer-term weather patterns. The rock mass was characterized based on rock quality designation (RQD), rock mass rating (RMR), rock mass quality (Q), and geological strength index (GSI) classifications. Hydraulic conductivities were measured and flow rates were estimated in Lugeons.

Findings indicated the following results of the field investigations:

- Measured VWPT pressures indicate maximum water pressures on the order of 500 to 600 psi at the tunnel depths of 2,200 and 2,500 feet below ground surface. This is approximately 60 percent of the expected pressures if the deep groundwater was directly connected to the shallowest groundwater along a constant hydraulic gradient. The data indicate vertical separation between the groundwater at the tunnel depth versus the first encountered shallow water.
- Temperature readings indicate maximum in-situ temperatures of 26 and 29 degrees Celsius (°C) at 2,200 and 2, 500 feet, respectively.
- Measured hydraulic conductivities (K) decrease with depth. Near surface K values range from $5x10^{-3}$ to $1x10^{-5}$ centimeters per second (cm/s), while test results from greater depths (2,200 to 2,500 feet) range from $1x10^{-6}$ to $8x10^{-7}$ cm/sec. These data indicate very low groundwater flow potential at the tunnel depth.

- Rock mass classifications using RMR indicate mostly Poor (21-40) with some Fair (41-60) and lesser Very Poor (<21) rock mass.

8.2.4 Cowles Mountain Tunnel Construction (1992-1993), San Diego

The Cowles Mountain Tunnel was constructed within the City of San Diego through granitic rock of the Peninsular Ranges for the San Diego County Water Authority. It was approximately 6,800 ft long with a rough-cut diameter of 11 feet 3 inches. The tunnel was excavated with a “shuffle shoe” tunnel boring machine (TBM) with 360 gripper shoe configuration that allowed continuous advancement of the TBM in strokes equivalent to the hydraulic thrust reaches. The TBM was fitted with a steel rib erector and upper decks for installing rock bolts. The tunnel was lined with welded 96-inch steel pipe segments welded in the tunnel. The anulus between the pipe and tunnel diameter was backfilled with light weight cellular grout.

The Cowles tunnel was constructed through granitic rocks the majority of which was granodiorite with lesser lengths of adamellite, diorite, tonalite and the same rock with various degrees of weathering. The geotechnical design summary report created for the project bidding indicated that the average rock unconfined compressive strength (UCS) would be on the order of 24,000 psi. As the tunnel was being constructed, the rate of tunnel advancement was 40% slower than expected. Tests on the rock during construction indicated that the average rock strength was approximately 60% higher than assumed with ranges of 10,000 to 69,000 psi for UCS. The contractor claimed and received additional compensation for the differing site conditions (DSC). The rock was highly abrasive with the high strength causing the tunneling progress to be slower than expected.

Groundwater was not an issue for construction. The maximum overburden cover was relatively small at 280 feet. The maximum groundwater flush flow was 150 gpm in a fractured bedrock zone.

8.2.5 Arrowhead Tunnel East (1997-2008), San Bernardino Mountains, San Bernardino National Forest

The Inland Feeder Project is a water delivery project tapping into the California Aqueduct water delivery system at Devil Canyon in the San Bernardino Mountains and transferring it through 44 miles of tunnels and cut and cover pipelines to blend with Colorado River Aqueduct water at Diamond Valley Lake in Riverside County. The Arrowhead West and Arrowhead East tunnels carry the water along the south side of the San Bernardino Mountains for approximately 10 miles to City Creek, all within the San Bernardino National Forest. The Arrowhead Tunnels had been planned and designed since the late 1980s with construction beginning in 1997. The USFS was intimately involved in the design and construction phases of the project with the goal of protecting the water resources and habitat of the forest, especially after the history of the San Jacinto Tunnel (summarized above).

Both of the Arrowhead Tunnels were advanced by 19-foot diameter custom-built tunnel boring machines (TBM). The newer technology of using a TBM for excavating the tunnel was believed to afford better control of water inflow into the tunnel excavation and thus protection of the surface water resources. The Arrowhead West Tunnel was 4 miles (6.4 km) long and the Arrowhead East was 5.8 miles (9.3 km) long. On the Arrowhead East Tunnel, the contractor had completed 8,000 feet of TBM mining when MWD shut down construction due both to water inflows increasing without appropriate controls and concerns about water impacts expressed by the San Manuel Band of Indians and the USFS. In 2003 a new contractor was selected to complete the tunnel construction after the alignment was modified and the tunnel lining and TBM were redesigned. Two Herrenknecht TBMs were designed and constructed to new specifications for the project including the ability to drill 26 100- to 150-foot long probe holes for pre-excavation grouting ahead of the TBM, and the ability to operate in either open or closed mode for mining both high strength crystalline rock and soft rock and fault gouge that are prone to squeezing. The TBM would also withstand 10 bar (145 psi) pressure at a standstill and 3 bar (44 psi) of pressure when in operating mode. With overburdens ranging from 1,100 to 2,070 feet above the tunnel, measured groundwater pressures were as high a 30 bar (435 psi).

The USFS Special Use Permit included groundwater control measures to be implemented during construction to protect the groundwater resources impacted by tunnel construction. If water flow from probe holes drilled ahead of the TBM exceeded 0.3 gpm, the mining operation would cease and the probe holes would be grouted to stem the flows. Additionally, groundwater outflow at the tunnel portal could not exceed 520 gpm (33 liter/second) as mandated by the project's Special Use Permit. To prevent groundwater losses into the lined tunnel, a new bolted and single rubber gasket segmental tunnel lining system was designed to withstand up to 40 bar (580 psi) of water pressure. A program of contact grouting around the concrete lining was carried out after erection of the segmental lining system and included annular grout and inflatable collar grouting to prevent water from traveling along the tunnel annulus.

A rigorous groundwater monitoring program had been implemented before construction to establish groundwater baseline data. The monitoring program with adaptations for TBM progress was continued during construction to document any effects recorded at springs and streams and associated habitat, and documented within an extensive monitoring well system installed during exploration and construction of the tunnels. Effects of tunneling on the groundwater elevations were documented in detail. Temporary mitigation measures for enhancing spring flow and habitats were implemented where needed during construction and after completion until baseline conditions were re-established.

In accordance with the permit requirements with the USFS, temporary mitigation methods for surface water resources were implemented on the Arrowhead Tunnels project, where supplemental surface water was needed during certain times of the year. Mitigation methods were applied based on the specific needs of each of the field site(s) that were affected by tunnel construction. For example, where supplemental water was needed in low quantities (e.g., a few gpm), a storage tank was set up and water was delivered to the tank by a water truck. Small diameter pipelines with regulated flows were extended from the water tank to appropriate discharge locations at spring sites. The flow rates and water tank volumes were instrumented to track water use and set up maintenance schedules. Where supplemental water was required in larger volumes than was practical for water truck delivery, a temporary, automated pumping system was constructed. The system utilized a connection to the local municipal water supply, and a pump system that was capable of lifting water to a temporary tank for distribution by discharge pipes at selected sites. The automated pumping system monitors the water level in the tank on the ridge, and triggers pumping as needed to maintain water flow in the discharge pipeline.

8.2.6 Irvine-Corona Expressway Tunnels Feasibility Study (2008-2011), Santa Ana Mountains, Cleveland National Forest

The Irvine-Corona Expressway Tunnels (ICE) project is a conceptual transportation corridor proposed between Interstate-15 near Cajalco Road in Corona, and the interchange of the SR-133 and SR-241 toll roads in Irvine, California. The tunnels would pass beneath the northern Santa Ana Mountains beneath the Cleveland National Forest and the Irvine Ranch Nature Conservancy to the west.

The transportation tunnel concepts evaluated in the ICE Tunnels feasibility evaluation proposed an 11.5-mile-long, large- to medium-diameter tunnels accommodating highway and rail traffic. The tunnel alignment and profile were developed, as much as practical, to avoid assumed high groundwater pressures, while constraining road grades to no more than five (5) percent.

The study included field rock coring exploration at five sites, groundwater modeling to estimate flow and impacts to the groundwater resources, ventilation modeling of conceptual tunnel designs, tunnel lining design concepts based on existing technologies and itemized construction costs and operations models for estimating operation costs. The study provided information, supported by scientific data, engineering studies, available technology, and professional opinions for use in evaluating the overall feasibility of the ICE Tunnels.

Based on the findings, the highway and rail tunnel system is technically feasible to construct beneath the Santa Ana Mountains, and to operate as a viable transportation alternative to the SR-91 Freeway between Orange and Riverside counties. There were no fatal flaws identified that would prevent the project from being designed, constructed and operated. However, the findings identified significant constructability challenges related to the following:

- Large tunnel diameter (52.5 feet) is at the cutting edge of feasibility for a TBM construction method;
- Groundwater pressures of 25 bar (360 psi) or more depending on the tunnel gradient and depth below the ground surface;
- Avoidance/mitigation of groundwater impacts to the Cleveland National Forest, and potential mitigation requirements;
- Tunnel ventilation requiring at least one mid-tunnel vent shaft within the National Forest or a horizontal vent tunnel parallel to the 11.5 mile-long main tunnels;
- Tunnel configuration and grade with a rail component would reduce the tunnel grade to no more than 3 percent, while increasing the overall depth of the tunnels and groundwater pressures; and
- Construction costs that exceed $6.9 billion, not including costs of financing.

With respect to design loads on the lining system, groundwater pressures dominate the tunnel lining design rather than rock loads. Based on the current state-of-the-art limit (Arrowhead Tunnels), a maximum groundwater head criterion of 800 feet (25 bar) was selected for design and construction of a watertight tunnel lining system consistent with the 5 percent road grade assumptions. Pressures above this limit would require the development and testing of lining systems specific to the ICE project and would exceed the current state-of-the-practice for watertight lining systems.

8.3 KEY TUNNEL CONSTRUCTABILITY ISSUES BASED ON PROJECT HISTORIES

Based on past tunnel project case histories for design and construction of tunnels in southern California, the following issues are recognized as critical for evaluating constructability of the RCS tunnel in with challenging conditions for design and construction of tunnels in general within mountainous terrain:

- Ground Conditions, both soil and rock mass classifications:

- Faulted Ground, poor tunneling conditions,
- In-Situ Stress Field
- Squeezing ground conditions affecting tunneling methods and rates of advancement.

- Elevated Groundwater Temperatures
- Active Fault Displacement
- Rock Strength and Abrasiveness
- Gassy Ground
- Corrosive Ground
- High Groundwater Pressures
- High Water Inflows, flush flows and sustained flows into tunnel
- Impacts on Groundwater and Surface Water Resources, water resources depletion.

These tunnel construction conditions are common in most long mountain (i.e. deep, in excess of 1,000 ft) tunnels in southern California and are described below along with similar anticipated conditions along the tunnel alignment with mitigation strategies that should be considered.

8.3.1 Ground Conditions

In the tunnel industry, ground condition is a term used to describe how the ground responds during or shortly following underground excavation. The ground conditions affect the constructability with respect to the mining and support requirements and are related to the geomechanical properties of the geologic units or rock mass conditions, the in-situ stress, groundwater conditions and the excavation method. There are different descriptors that are applied to soil (Tunnelman's Ground Classification) and rock (Squeezing Degree). The typical industry descriptors are listed in Table 3 below. In some conditions, e.g. where the rock mass is faulted or weathered, the rock mass may be reduced to intermediate geomaterials that behave more similar to soil. Therefore, we've adopted descriptive terms compiled by Singh and Goel (1999), which include terms that are commonly used for describing ground conditions relative to design and construction methods. It is anticipated that virtually all of the ground conditions described will be encountered in the RCS tunnel.

Table 3

Descriptors for Ground Conditions

Ground Condition Description	Potential Materials	Excavation Behavior	Design and Construction Considerations
Self supporting	Unfractured to slightly fractured, hard rock mass	• Adequate stand-up time to install support • Does not require initial support	• Identify potential wedges, rock blocks in crown and walls requiring reinforcing as necessary during mining
Firm	Stiff, cohesive or strongly cemented soil or soil-like material	• Adequate stand-up time to install support • Does not require initial support	• Identify potential zones where degree of cementation is less that have the potential to run or flow
Non squeezing	Slightly to moderately fractured, hard rock mass with a stress to strength ratio less than 1	• Adequate stand-up time to install support • Does not require initial support	• Install tunnel support with delay necessary to allow release of strain-energy within rock mass
Ravelling	Intensely to very intensely fractured rock mass or stiff, cohesive or weakly to moderately cemented soil under moderate to high stress	• Blocks drop from the face, crown or walls shortly after excavation. • Inadequate stand-up time to install support • Requires initial support, limiting unsupported spans, and/or rapid installation of support	• Install initial support shortly after excavating to prevent overbreakage • Heavy crown and wall pressures should be considered in design
Mild squeezing	Slightly to moderately fractured, soft to hard rock mass with a stress to strength ratio greater than 1 and less than 5	• Inadequate stand-up time to install support • Excavation deforms plastically decreasing the tunnel diameter (closure) on the order of 1 to 3%.	• Install initial support shortly after excavating to prevent heaving in invert of tunnel • Install tunnel support with little delay • Side pressure should be considered in design
Moderate squeezing	Intensely to very intensely fractured, or soft rock mass with a stress to strength ratio greater than 1 and less than 5	• Inadequate stand-up time to install support • Rate of closure is more rapid than mild squeezing ground with a closure magnitude on the order of 3 to 5%	• Initial support should be installed as early as possible to reduce the rate of closure or to limit closure • Tunnel excavation diameter should be increased to allow for desired closure • Wall pressure should be considered in design • Instrumentation is essential

Ground Condition Description	Potential Materials	Excavation Behavior	Design and Construction Considerations
High (Heavy) squeezing	Rock mass or soil with a stress to strength ratio greater than 5	• Inadequate stand-up time to install support • Rate of closure is more rapid than moderate squeezing ground with a closure magnitude > 5% • Excavation deforms irregularly resulting in irregular cross-section	• Initial support should be installed as early as possible to reduce the rate of closure or to limit closure • Tunnel excavation diameter should be increased to allow for acceptable closure • Invert support should be installed as early as possible to mobilize support capacity • TBM steering may be difficult • Instrumentation is essential
Swelling	Rock mass or soil with expansive clay minerals that have natural moisture contents near or less than their liquid limit	• Expansive clays absorb water and expand volumetrically resulting in some degree of tunnel closure or swelling pressure where support is placed in advance of swelling	• Tunnel excavation diameter should be increased to allow for expected swelling • Measures should be made to limit moisture being absorbed by swelling clay during and following construction • Tunnel closure should be measured
Running	Decomposed to highly weathered, very intensely fractured to earthlike unsaturated rock mass or cohesionless soil or soil-like material	• Blocks, grains or particles fall or “run” into tunnel from the face, invert, crown or walls	• Forepoling, grouting or other ground improvements may be necessary to stabilize ground and reduce the risk of mining-in-place • Excavated volumes and advance should be monitored closely
Flowing	Decomposed to highly weathered, very intensely fractured to earthlike saturated rock mass or cohesionless soil or soil-like material, usually under water pressure	• Mixture of rock or soil and water material flows into tunnel like a viscous fluid from the face, invert, crown or walls	• Forepoling, grouting or other ground improvements may be necessary to stabilize ground and reduce the risk of mining-in-place • Dewatering ahead of excavation to reduce water pressure • Excavated volumes and advance should be monitored closely

Ground Condition Description	Potential Materials	Excavation Behavior	Design and Construction Considerations
Rock bursting, Slabbing, Spalling	Unfractured to very slightly fractured, hard rock mass under moderate to high stress	• Portions of massive, unsupported rock explode, elastically deform rapidly, or pop from unsupported areas of the face, invert, crown or walls	• Rock anchors installed in portions of tunnel where slabbing is evident or where there is a delay before installing support • Micro-seismic monitoring essential

Source: Singh et al., 1999.

8.3.2 Faulted Ground, Poor Tunneling Conditions

Faults can pose significant construction difficulties for tunnels by altering the conditions of the rock mass being mined and increasing water flows into the tunnel. The difficulties can include ground conditions that require use of additional tunnel support (e.g. squeezing, raveling, flowing ground), tunnel boring machine rescue (ground or water conditions trap the TBM or shield requiring hand mining for its release and continued operation), excessive water inflow requiring additional pumping or grouting of ground. All of these conditions can cause schedule delays and increased costs. Such difficulties are not fatal flaws unless the project owner cannot spare additional time in the schedule and costs to mitigate them. Therefore, faults should be anticipated, evaluated and accounted for when considering tunnel constructability and tunneling methods and tunnel lining design.

Geologic formations that once were intact and strong become mechanically sheared and brecciated, altered, decomposed, and weak after being subjected to faulting. The degradation of the rock mass may result in face instability during mining, higher lithostatic loads on the tunnel lining system, and facilitate higher groundwater pressures and flows in and adjacent to the faults.

The RCS tunnel will encounter several mapped faults such as the Elsinore fault zone and the San Felipe fault zone that will likely be associated with severely faulted ground. Other areas may also exist where severe jointing or faulting, not mapped in the available literature, may be encountered. Such conditions can be identified and evaluated by rock coring methods that penetrate the fault zones.

8.3.3 In-Situ Stress and Rock Bursting

The in-situ stress conditions are important for construction as stresses retained in the rock mass affect tunnel mining and support requirements. Anisotropic stress fields may result in TBM steering difficulties, instabilities in short spans that are temporarily unsupported resulting in rock bursting, or overstressing of tunnel supports. In-situ stress is governed by the lithostatic stress, which is the overlying weight of the rock mass (i.e., the average unit weight including the intact rock, joints, groundwater and infill), and in some cases tectonic stresses caused by active faults or other geologic structures (e.g., antiforms, synforms, etc.). Deep tunnels often are subject to high in-situ stress conditions.

Along the tunnel alignment, significant length of the tunnel will be in excess of 2,000 feet deep in high strength rock which may foster moderate to high in-situ stress in the rock mass. Field investigations employing hydrofracturing of in-tact rock at depth will help to measure and define the in-situ state of stress in the rock for design and anticipating potential constructability issues.

8.3.4 Squeezing Ground Occurrence

For the RCS tunnel, squeezing is likely an important factor in tunnel construction. Squeezing occurs where the rock mass strength (σ_c) is substantially less than the reconfiguration of the stress (i.e., post-excavation stress) around the openings at the excavation face and sidewalls, the rock surrounding the TBM or lining can deform inward elastically and plastically (i.e., tunnel closure) following excavation. If this deformation is not accounted for in the design, the TBM may become trapped in the squeezing ground, or the lining could become overstressed. Although the mechanisms are different, the ground response from swelling is similar to squeezing, as swelling can result in tunnel closure and TBM entrapment. In general, substantial lengths of tunnel with ground conditions that describe soil and intermediate geomaterials occur in areas of lower in-situ stress.

With respect to the constructability of the RCS tunnel, the most adverse ground conditions are likely zones of heavy (high) squeezing in proximity to faults where the rock mass surrounding the tunnel “squeezes” causing tunnel closure (convergence) of 5 percent or more. In such conditions, it may be necessary to install temporary reinforcing to maintain safety and control the rate of closure, or allow some degree of deformation to occur before installing the final support. The excavation diameter within these zones should carefully consider the ground load and tolerable deformation for the tunnel lining system.

Squeezing ground conditions are expected to occur in the deeper sections of tunnel and in proximity to wide fault zones, especially branches of the Elsinore fault or intensely jointed rock that are intersected by tunnel. In order to overcome the squeezing ground conditions, geologic investigations must thoroughly evaluate ground in-situ stresses within lengths of tunnel with high overburdens and at major fault zone crossings. An enlarged bore and/or alternative construction methods may need to be compatible with or capable of overcoming or avoiding squeezing pressures. In some cases, ground improvement may be feasible to stabilize squeezing ground ahead of tunnel excavation. Future design and construction planning should include contingencies for conducting TBM rescue in the event that one becomes frozen (entrapped) due to squeezing ground.

8.3.5 Elevated Groundwater Temperatures

Faults have the potential to act both as groundwater conduits and as barriers that often result in significant variations in groundwater pressures from one side of the fault to the other. These variations in groundwater pressures are especially critical when unexpectedly encountered during tunnel mining. Also, high temperature groundwater may be channeled upward along faults to shallower depths requiring special controls to enable workers to work in the hot tunnel environment. Both the Arrowhead Tunnels and the Tecolote tunnel (117 degrees Fahrenheit) encountered hot groundwater within fault zones intersecting the tunnels.

Along the RCS tunnel there are no known hot springs or hot water, however, there are hot springs and hot water wells associated with the Elsinore fault zone, and the San Felipe fault zone (e.g. Warner Springs). If water temperatures reach levels high enough to adversely affect tunnel miners, then the constructability of the tunnel in such sections may be affected. In high temperature zones, artificial cooling of the tunnel and tunnel workers will need to be included in the construction methods. Groundwater temperatures can be measured during geotechnical investigations to help anticipate the underground conditions and if elevated temperatures may be encountered.

8.3.6 Active Fault Displacement

Fault displacements result from differential movement across a fault during an earthquake due to tectonic forces shearing the Earth’s crust. Depending on the size of the earthquake (i.e. magnitude representing energy release), the displacement sometimes propagates to the ground surface causing surface rupture and displacement of features straddling the fault such as geomorphic features (e.g. streams, flat surfaces) or man-made structures (e.g. roads, buildings, pipelines, etc.). Tunnels also are subject to fault displacement causing offset of the tunnel structure below ground due to relative displacement across a fault or fault zone.

For the RCS tunnel, the most significant active fault intersected by the alignment is the main trace of the Elsinore fault zone. The zone also includes lesser fault traces that are active and belong to the Elsinore fault zone. Based on the location of the fault intersection, it is likely that there will be high groundwater pressures, high groundwater flows and squeezing ground conditions in the fault gouge. All of these conditions occurring at the same place will complicate the tunnel construction and raise the risk of the TBM getting stuck in the fault zone. It is likely that the tunnel may need to be mined using sequential excavation methods to widen the tunnel and build in systems to accommodate the anticipated fault displacement.

Restoration of a tunnel requires realignment or smoothing of the offset of the tunnel and repair of the lining system. Through planning during design of a tunnel, fault displacements can either be accommodated in the design or if the displacement is large enough to damage the lining system, the tunnel can be designed to minimize the required repair efforts to restore service of the tunnel. Fault displacements can be accommodated by design for specified displacement magnitude and slip direction, which needs to be determined by field investigations by geologists familiar with paleo-seismology and field interpretation of past faulting events on the active fault of concern. Mitigation for fault displacement has included use of enlarged tunnel sections and/or fault chambers or special tunnel lining systems designed to survive a fault displacement such as a ductile lining. Potential displacements need to be quantified so that the tunnel lining and hydraulics can be designed to accommodate the faulting with minimal repair to restore normal operation.

8.3.7 Rock Strength and Abrasiveness

Cowles Tunnel in San Diego encountered high strength rock that caused slow tunneling progress with the TBM constructed for the project. The granodiorite encountered in the Cowles Tunnel had UCS as high as 69,000 psi, which slowed progress for the TBM by 40%. The RCS tunnel will likely have very high strength rock similar to the Cowles Tunnel and must be considered for the construction schedule and budgeting.

The abrasiveness of the geologic units affects the amount of wear on the various pieces of mining equipment. Mining in abrasive materials requires more frequent tooling replacements to avoid overwearing vital components of the TBM cutterhead and cuttings circulation system. A widely used measure of abrasivity is the quartz content measured in petrographic thin section analysis as percent of quartz and by the Cherchar Abrasivity Index. Granitic rocks usually have a high quartz content and are classified as highly abrasive. Highly abrasive rock can cause delays in tunneling progress due to machinery break down or excessive wear requiring shut down of tunneling while worn parts are replaced.

A large proportion of the rock terrain crossed by the RCS tunnel includes Quartz Monzonite and Monzonite, both of which will be classified as highly abrasive on mechanized equipment for tunnel construction. Wear on tunneling equipment will need to be monitored closely and frequent maintenance and parts replacement should be anticipated during construction.

8.3.8 Gassy Ground

Gassy ground results from the migration of flammable, toxic, or asphyxiating gases into the tunnel during construction or operation. The gas emanates from geologic materials (e.g., from oxidation of minerals, e.g. pyrite releasing hydrogen sulfide gas), groundwater containing dissolved gas flowing into the tunnel, or petroleum occurrence in formations. Tunnel alignments have been successfully constructed through gassy ground in southern California with proper procedures as required by the California Division of Safety and Health (Cal/OSHA). A more detailed discussion of requirements for gassy ground is presented in the California Code of Regulations (CCR). Gassy ground can cause underground deaths due to deprivation of oxygen and to fires or explosions and is thus highly regulated. Depending on the potential occurrence of gassy ground, such conditions can be mitigated by implementation of sufficient ventilation protocols.

Based on the limited data available at this time, the potential for gassy ground along the tunnel alignment appears to be small. However, once a tunnel project is nearing construction and results from field investigations are available, the CCR Subchapter 20 Article 8 requires a tunnel classification be obtained from Cal/OSHA with respect to flammable gas or vapors. Depending upon the Cal/OSHA classification, various gas monitoring and ventilation methods may be required during tunnel construction and operation.

8.3.9 Radon Gas Hazard

Radon is a naturally occurring colorless, odorless and tasteless radioactive gas derived from the decay process of uranium and thorium occurring in rocks, soils and groundwater. It is a known carcinogen increasing the risk of lung cancer. Generally, the higher risk occurs in areas where ventilation is poor and radon, if present, can concentrate at levels above the permissible exposure limit (PEL). OSHA's Ionizing Radiation standards including radon are covered in 29 CFR 1910.1096 and 1926.53. Radon can be concentrated in confined workspaces such as caves, mines and tunnels creating a "hazardous atmosphere". Regulations for protection from hazardous atmospheres require that workers employ control measures such as respiratory protection, self-contained breathing apparatus or sufficient ventilation to mitigate the hazard in the workspace.

The proposed tunnel alignment will penetrate several rock types that could contain radioactive substances that could release radon gas directly into the atmosphere of the tunnel or from water that could release radon gas dissolved in the groundwater. The early detection of potential radon gas can be evaluated by testing groundwater samples for content of radionuclides. Such testing is usually included in the scope of the geotechnical exploration program defining the subsurface conditions for defining tunneling conditions for tunnel boring machine design and lining system design. Testing for radon gas should also be implemented during construction of the tunnels to develop appropriate mitigation measures should radon gas be present.

8.3.10 Corrosive Groundwater

Corrosive groundwater can damage components of the TBM, and over time may deteriorate the concrete compromising the performance of the tunnel structure. Although relatively high sulfate concentration is the primary cause of corrosive groundwater, gases such as carbon dioxide and hydrogen sulfide that dissolve into groundwater form acids that may also damage construction materials. Planning for corrosive groundwater can avoid damage by using corrosion resistant materials or applying corrosion protection to tunnel linings and associated facilities. Currently, there is no known corrosive water along the tunnel alignment.

Corrosive groundwater can be mitigated using corrosion resistant concrete mix and admixtures. As information and data is collected along the tunnel alignment, project-specific designs could consider the effects of corrosion on the tunnel structures and components, if it is a constructability concern.

8.3.11 Groundwater Inflows to Tunnel

Groundwater flows into tunnels under construction occur whenever a tunnel is being advanced through ground (i.e. rock) saturated with water. As the tunnel advances in rock, it intersects defects in the rock that carry water allowing water flows from the rock mass into the tunnel opening. The degree of interconnectedness of fractures and the aperture (i.e. width of the fractures) determine the amount of water available for flowing into the tunnel opening. The hydraulic gradient or pressure of the water closely control the rate of water flow. As described in the tunnel construction case histories the rate of flow can reach very high quantities especially from rock highly fractured such as in a fault zone. Fault zones were the primary areas where high groundwater flows were encountered in the San Jacinto tunnel, Tecolote tunnel and the Arrowhead tunnels. In the San Jacinto tunnel, inflows reached as high as 16,000 gpm with 3,000 cy of sand. The Tecolote tunnel experienced flows as high as 2,800 gpm with the cumulative flows out the tunnel portal as high as 9,100 gpm.

Hydraulic conductivity is a measure of the potential flow of water from a water bearing isotropic aquifer. In fractured rock, which is fracture flow controlled, the hydraulic conductivity is averaged over the interval of fractures being measured and is called effective hydraulic conductivity because the flow is non-isotropic. The hydraulic conductivity of the geologic units interacting with the tunnels are important as these affect the potential for inflows during construction and operation, and the groutability of the geologic units to block the flows from entering the tunnel. Table 4 gives relative hydraulic conductivities and equivalent units in Lugeons of various lithologies as published in literature. A typical fractured crystalline rock usually has hydraulic conductivity in the range of low to moderate flow. Some fault zones may be classified as very high groundwater flow potential.

At the greater depths anticipated for the RCS tunnel, the rock mass generally should have low to very low hydraulic conductivity. The shallow zones should have moderate to low hydraulic conductivity. Therefore, groundwater flow through the rock mass is generally expected to occur at a slower rate at depth than near the ground surface. However, locally, more intensely fractured zones (e.g. fault zones) may have higher hydraulic conductivity and allow more rapid water flows through the affected rock. Where crossed by the tunnels, such fault zones could introduce relatively high-water flows into the tunnels, combined with squeezing ground, and unstable ground causing significant hazards and/or difficulty during construction. High groundwater flow and associated unstable ground conditions can be controlled through proper planning and application of various methods to control groundwater flow.

Geotechnical investigations should focus exploration on areas that are suspected to produce high groundwater flows such as at fault intersections of the Elsinore fault zone and San Felipe fault zone or where major joint sets are recognized through use of aerial photograph analysis intersecting the joint sets recognized as lineaments through the terrain.

Table 4

Hydraulic Conductivity Correlations

Descriptor	Hydraulic Conductivity (K) cm/sec	Lugeon	Generalized Lithology or Conditions
Very High	$10 – 10^{-1}$	>50	Sediments comprised of gravel Intensely fractured (karstic) limestone or basalt Rock mass with many open joints
High	$10^{-1} – 10^{-3}$	5-50	Sediments comprised of sand Intensely fractured igneous or sedimentary rock Rock mass with only some open joints
Moderate	10^{-3} - 10^{-5}	1-5	Sediments comprised of fine sand, or interlayers of silt or clay Coarse- to medium-grained sedimentary rocks Fractured sedimentary, igneous, and metamorphic rocks Rock mass with small joint openings, openings with impervious infill, or few joints
Low	10^{-5} - 10^{-7}	0.01 – 1	Sediments comprised predominantly of silt or clay Fine-grained sedimentary and igneous rock, metamorphic rock Rock mass with tight joints, openings with impervious infill, or few joints
Very Low	$<10^{-7}$	<0.01	Sediments comprised of homogeneous clay Shale and evaporite Rock mass with tight joints, openings with impervious infill, or few joints

Sources: Isherwood, 1979; Goodman, 1981; Jaeger et al., 2007; Domenico and Schwartz, 1990; USBR, 1998; Fell et al., 2005; Freeze and Cherry, 1979.

A commonly used method for mitigating tunnel flooding using a TBM in open face mode of operation is through drilling probe holes in front of the TBM shield and applying pre-excavation grouting to seal off water flow paths. According to the Tunnel Safety Orders of the CCR, Cal-OSHA requires a minimum of 20 feet of tested ground ahead of the excavation face in tunnels where there is a likelihood for dangerous accumulations of water, gas or mud within 200 feet of the working area. Another option for constructing the RCS tunnel is using a TBM that applies a positive face pressure at the front of the TBM cutter head, which holds back the water. This method is practical as long as the TBM operating pressure is greater than the groundwater pressure in the vicinity of the excavation. Once the tunnel is completed, a cast in place or gasketed tunnel lining system is designed to prevent leakage through the lining system.

8.3.12 Groundwater Pressures

The groundwater pressures are one of the key features to consider when designing and constructing a watertight tunnel lining. The feasibility for watertight linings are reported to be limited to magnitude of water pressure less than about 40 bar (580 psi), based on design specifications for the Hallandsas Tunnel in Sweden. The Arrowhead Tunnels lining systems were proof tested up to the 27 bar (390 psi) to meet the anticipated design requirements (Swartz et al., 2002). For design of tunnel linings that are leak proof, the Arrowhead tunnels lining system was able to withstand approximately 27 bar of pressure. Beyond that level of pressure, the tunnel linings may leak. Table 5 indicates the relationship of groundwater head to groundwater pressure expressed in units of psi and bar for reference.

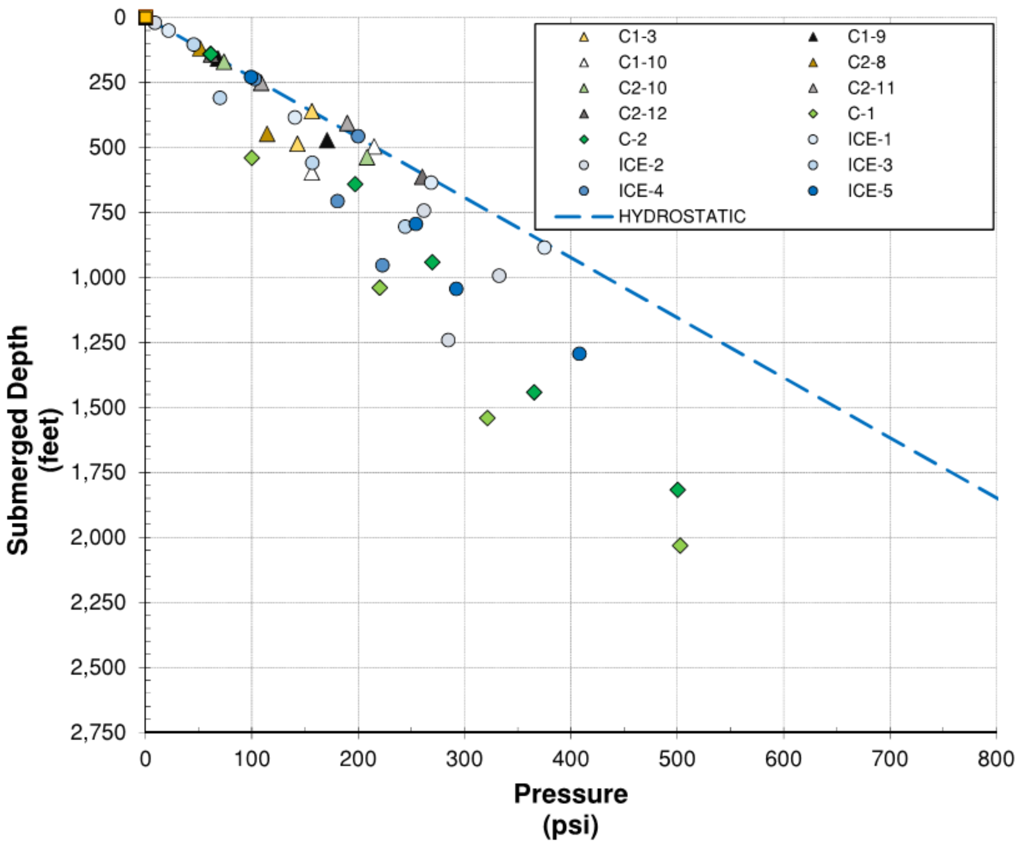
Table 5
Unit Equivalents for Groundwater Pressures

Approximate Groundwater Pressures		
feet-head	**psi**	**bar**
<175	<75	<5
175-350	75-150	5-10
350-850	150-370	10-25
850-1,175	370-510	25-35
>1,175	>510	>35

Also, during construction, potential inflows are generally proportional to groundwater pressure outside the tunnel lining. Therefore, the higher the groundwater pressure at the tunnel elevation, the greater the potential water flows into the tunnel opening if the lining is not watertight.

Based on the depth versus groundwater pressure trends observed from the instruments monitored in core holes for the Arrowhead tunnels, CPA tunnel and the ICE tunnel projects, data indicate that pressures measured at various depths in the rock masses are generally lower than the straight line of the hydraulic gradient. This indicates that the water column is partially interrupted in most of the observed cases. Therefore, the anticipated water pressure is not directly proportional to the height of saturated rock above deep tunnels. We would expect such conditions to exist along the RCS tunnel.

The maximum overburden along the RCS tunnel is approximately 2,700 to 3,000 feet between the highest ground surface and the tunnel crown. If the groundwater table occurs in the bedrock over the tunnel between 200 and 300 feet below ground surface, theoretically, as much as 1,150 psi (80 bar) of hydraulic head could exert pressure at the tunnel elevation. More likely at such depths, the pressure will be less than the straight hydraulic head anticipated and could be on the order of 750 psi (52 bar) based on the empirical data shown in Figure 8.3.1.



Arrowhead Tunnel West: C1-3, C1-9 & C-10
Arrowhead Tunnel East: C2-10, C2-11 & C2-12
Central Pool Augmentation: C1 & C-2
Irvine Corona Expressway: ICE-1 to 5

Figure 8.3.1 - Hydraulic Pressure Gradients in Deep Tunnels

The maximum groundwater pressure that watertight tunnel lining systems have been designed for in California is approximately 360 psi (25 bar) for the Arrowhead Tunnels. The purpose of the watertight lining was to prevent uncontrolled leakage of groundwater into the tunnel as one of the measures to protect the groundwater system and surface water resources from impacts. For constructability of RCS tunnel, similar design concepts should be assumed with the objective of avoiding or mitigating impacts to the hydrogeology and hydrology along the entire length of the tunnel alignment. The Arrowhead tunnels lining system was a precast, segmental, gasketed and bolted lining system. Since the depth of the RCS tunnel indicates that the groundwater pressures could be twice the state of practice gasketed segmental lining used for the Arrowhead tunnels, development and testing of lining systems for pressures greater than 25 bar (360 psi) may be needed to mitigate groundwater impacts. Also, where groundwater pressure exceeds 25 bar, other precautions may be necessary including a well-developed grouting strategy to help prevent leakage during tunnel construction. See the description alternatives under Impacts on Groundwater and Surface Water Resources.

8.3.13 Impacts on Groundwater and Surface Water Resources

Based on the case histories of long-tunnel design and construction through southern California's mountains summarized above, some key tunnel feasibility issues are related directly to groundwater resources. In most cases, the tunnels have, by necessity, passed beneath national forests, national monuments and state parks including San Jacinto Mountains National Monument (and Mount San Jacinto State Park), Los Padres National Forest, San Bernardino National Forest and the Cleveland National Forest. The USFS manages these national forests and relies on their past experience with tunnels beneath their forests to guide them in forming opinions and policies about proposals for construction and operation of future tunnels beneath their national forests. It also involves impacts to Native American lands, water rights and private landowners. The proposed tunnel alignment also traverses beneath or in close proximity to several Native American lands including the Los Coyotes, Santa Ysabel, and San Pasqual Indian reservations. The key question raised by potentially impacted stakeholders is whether the tunnel design, construction methods and other mitigation methods will adequately protect water resources along the tunnel alignment.

During the construction of early tunnels in southern California, the practices of tunnel construction treated groundwater as a nuisance and, as a common practice, it was drained from the tunnel to decrease water inflows and pressures in the tunnel during construction (e.g., San Jacinto Tunnel and Tecolote Tunnel). The construction practices for controlling groundwater were not geared to water resources protection and they led to impacts on groundwater resources and surface water resources with the most notable case history being the San Jacinto Tunnel. The practices of letting the water flow freely to depressurize the groundwater system resulted in springs drying up and streams stopped flowing after construction.

As environmental sensitivity and protection of resources has become a greater priority than in the past, more advanced design and construction methods, monitoring programs, and mitigation strategies have been established to mitigate water resources impacts

The hydrologic and hydrogeologic conditions along and adjacent to the tunnel alignment pose three major constructability challenges as follows: 1) In order to protect the groundwater and surface water resources in proximity to the tunnel, the groundwater system needs to be protected during tunnel construction to limit losses of water available to sustain the habitat above the tunnel (i.e. maintain spring and stream flows); 2) Conducting tunneling activities while maintaining high groundwater pressures and preventing flows of water into the tunnel make tunneling more difficult and risky for ; and 3) If impacts occur to the water resources, the tunnel owner/contractor may be held liable for mitigating impacts to the natural habitat.

Methods applicable to protecting groundwater resources both during tunnel construction and after completion of the construction include the following for protecting hydrologic and hydrogeologic conditions:

- Understanding how the surface water and groundwater systems are connected;
- Understanding how tunnel construction and operation affects these systems;
- Use of a TBM that can operate in closed-mode to maintain face stability and counteract in-situ groundwater pressures thereby limiting inflows.
- Pre-excavation grouting of the rock ahead of the tunnel excavation and radial grouting through the lining system can reduce or prevent groundwater drainage into the tunnel. Reducing inflow into the tunnel during construction will reduce the hydrologic and hydrogeologic impacts along the tunnel alignment.
- Using a segmental, precast, concrete lining with bolted and gasketed joints as the final lining could control groundwater inflows to the tunnel during and after excavation up to certain pressures, as discussed above.
- Although less effective in protecting groundwater and surface water resources, a lining system that allows enough leakage to reduce groundwater pressures on the lining system may be considered as an alternative to control and manage water inflows.
- Collars can be installed along the tunnel lining to prevent water from migrating along the lining / bedrock interface.
- Contact grouting can be implemented to fill and seal the interface (aperture) between the tunnel lining and the bedrock.
- In areas where groundwater pressures and rock mass permeability are unfavorable, other options may include:
    - Specifying thresholds for short-term (transient) and long-term (static) groundwater inflows that mitigate impacts to acceptable levels; and,
    - Close coordination between the tunnel contractor and designer to successfully implement specified tunnel construction methods and designs that mitigate potential impacts to these systems.

- Develop a long term (e.g. decade long) monitoring program to measure all groundwater systems, spring and stream flows along the RCS tunnel to establish baselines prior to construction for each including seasonal changes and diurnal fluctuations.
- Conduct surveys of habitats dependent on water resources to establish biological baselines prior to tunnel construction and establish thresholds for triggering mitigation measures.
- In the event of changes in the water resources and biological baseline data during construction, be prepared with a mitigation plan that will supplement stream and spring flows of the affected areas to sustain the habitat in its natural condition. Water supplementation may continue for years until natural water resources have recovered to levels able to sustain the native habitats.

8.3.14 Portal Construction Issues

Portals will be constructed either as permanent or temporary structures to access the tunnel for construction and operation. Depending on the terrain and tunnel alignment, portals can be either in hillsides or on flat ground. Portals are excavated openings into the tunnel alignment daylighting the tunnel to allow access for workers, mining equipment, construction materials or ventilation of the workspace underground. Portals used for launching TBMs can be several hundred feet long and represent large horizontal excavations for equipment assembly or they can be vertical openings (i.e. shaft) to allow vertical access over the crown of the tunnel. The key constructability issues include many of the ground conditions and construction methods associated with tunnel excavation including shoring support, running or flowing ground, squeezing ground, groundwater control, dewatering, rock bolting etc. In addition, portals in hillsides may encounter landslides or result in unstable cut slopes both on the sides of the portal excavation and over the crown of the excavation. Conventional slope stabilization methods may be employed for stabilizing the portal cut slopes depending on the topography and the cause of slope instability. Proper specifications for portal backfill will be necessary to reduce potential for long-term surface settlement. For the RCS tunnel and portals, more information will be needed to address specifics of portal constructability on a site-specific basis.

8.4 EXCAVATION AND TUNNEL SUPPORT CONSIDERATIONS

The excavation and support of tunnels is largely governed by the ground conditions, and groundwater pressures and inflows during tunnel construction and/or operation. Typically, in long tunnels, using TBM and a pre-cast concrete lining system is the most economical because of cost and schedule. However, in most tunneling projects, appurtenant tunnel components (i.e., cross passages, utility chambers, etc.) are constructed using a variety of methods (e.g., drill and blast, mechanized mining using a shield and roadheader, etc.) and support systems (e.g., shotcrete and rockbolts, steel sets, truss systems, etc.).

The ground conditions should be carefully considered in the TBM selection and design. Based on the anticipated ground conditions, the more adverse ground conditions (i.e., squeezing, high groundwater pressure) will likely require a TBM that can operate in closed-mode [e.g., an Earth Pressure Balance (EPB) TBM, Slurry TBM, or Crossover TBM]. Such TBM technologies have been successfully used to mine tunnels subjected to groundwater pressures as high as 11 to 15 bar (Hallandsas Tunnel, Sweden and Lake Mead Tunnel, Nevada). To avoid the risks of the TBM becoming frozen (entrapped), the TBM and lining system should be designed such that the thrust necessary to overcome shield friction from squeezing ground can be accommodated.

9 CONCLUSIONS

Based on the results of our geologic reconnaissance, literature review and engineering evaluations, the proposed project is feasible from a geotechnical standpoint. The tunnel alignment will involve approximately 127.4 miles of tunnel, pipeline and canal alignment within San Diego and Imperial Counties, California. The western portion of the project will involve tunneling roughly 44.5 miles through a wide range of granitic and metasedimentary rocks at a maximum tunnel depth of up to approximately 3,500 feet beneath the Cleveland National Forest. The proposed cut and cover trench pipeline segment will be approximately 39.9 miles extending between Borrego Springs and SR 86 in Imperial Valley. The proposed canal segments parallel to the existing West Side Main and All American Canals will be roughly 43.0 miles in length.

The tunnel excavations are anticipated to be excavated utilizing TBM methods that will cross several faults and fault zones. Based on our site reconnaissance, research and literature review, and our experience with similar projects, we anticipate the key issues for tunnel design and construction will involve the nature and characterization of the various rock masses, including rock strength, rock weathering and abrasivity, active fault crossings, poor ground conditions at fault crossings, hydrostatic pressures at tunnel depth, in-situ temperature, and control of groundwater inflows during tunneling. These listed conditions can affect designs, schedule and cost but do not render the proposed tunnel geotechnically infeasible with the anticipated underground conditions. In addition, no fatal flaws or seismic hazards are anticipated for the proposed pump stations, reservoir, or treatment plant. These facilities will be further evaluated as part of Phase B.

The non-tunnel portions of the alignment are primarily in the eastern half of the project and are anticipated to encounter various alluvial soil types as well as local shallow bedrock. Key issues for the non-tunnel portions of the alignment include soil and bedrock excavatability, liquefaction potential, re-use of excavated materials for trench backfill, corrosion potential, shallow groundwater, canal side wall stability, and fault crossings. Summarized below are some design and construction considerations:

<u>Tunnel Design and Construction</u>

- Faulted ground.
- Poor tunneling conditions in some sections.
- In-situ stress field.
- Squeezing ground conditions affecting tunneling methods and rates of advancement.

- Elevated groundwater temperatures.
- Active fault displacement.
- Rock strength and abrasiveness.
- Gassy ground.
- Corrosive soil conditions.
- Elevated Temperature
- High groundwater pressures.
- High water inflows, flush flows and sustained flows into tunnel.
- Impacts on groundwater and surface water resources, water resources depletion.

Pipeline/Canal Design and Construction

- Dewatering adjacent to existing canal.
- Stability of temporary and finished slopes in wet/dewatered areas and in loose/dry soil conditions.
- Shoring of deep excavations where layback is not permissible.
- Excavating beneath existing bridges and roadways.
- Corrosive soil conditions.
- Active fault displacements.
- Liquefaction potential in loose granular soils.

10 LIMITATIONS

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

This report may be used only by clients and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than three years from the date of the report.

The work performed was based on project information provided by Black & Veatch and the San Diego County Water Authority.

Conclusions and recommendations contained in this report are based on our field observations, review of existing site surface and subsurface information, and our present knowledge of the proposed project. A design-level geotechnical investigation is recommended to validate the assumptions presented in this report.

The scope of services for this report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

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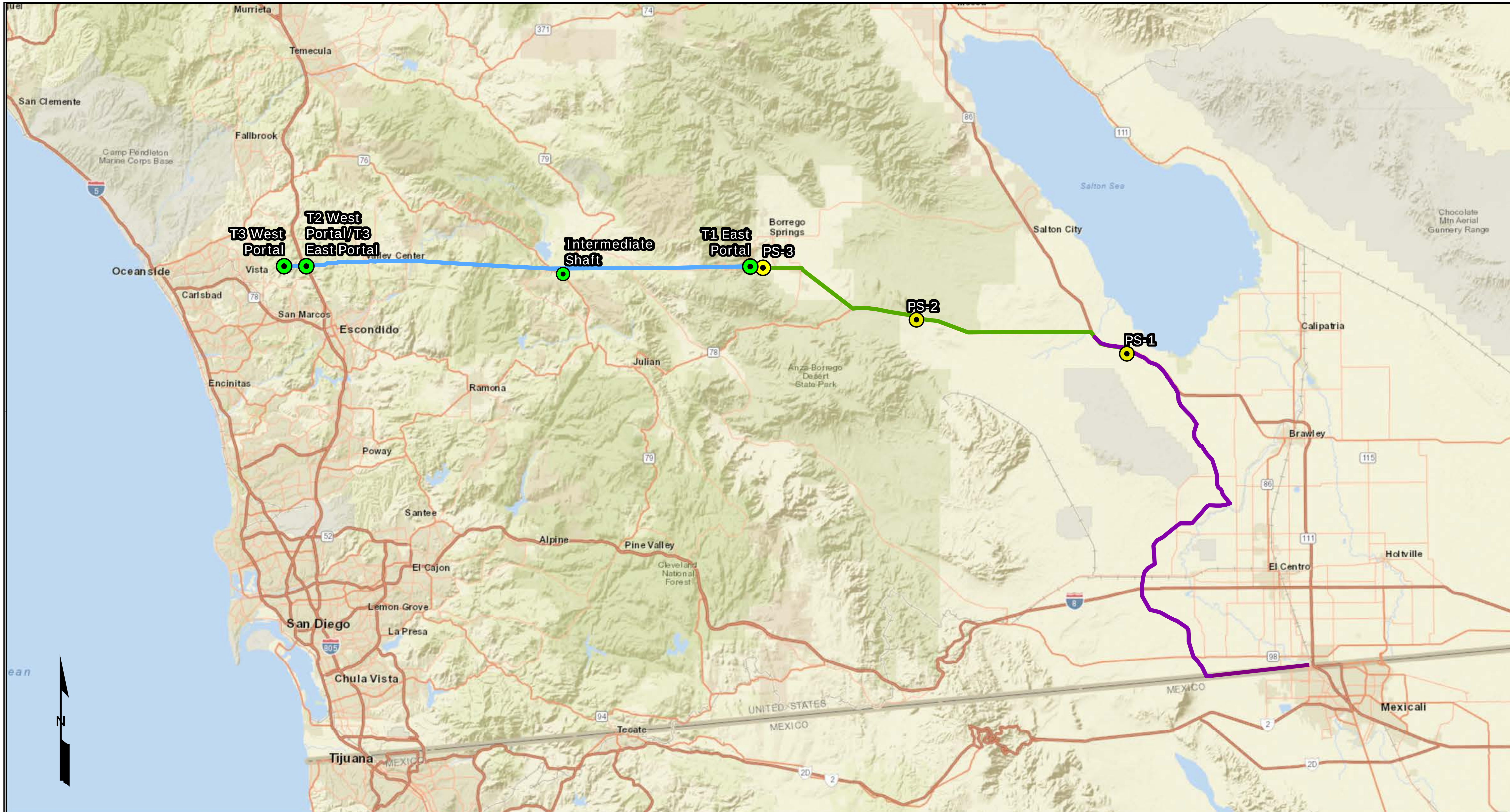
4M-160 to 4M-164	4M-214 to 4M-219	5M-14 to 5M-17	5M-86 to 5M-89
5M-118 to 5M-122	5M-194 to 5M-197	6M-95 to 6M-98	6M-172 to 6M-175
6M-207 to 6M-210	7M-30 to 7M-34	7M-117 to 7M-121	8M-23 to 8M-28
8M-64 to 8M-68	9M-16 to 9M-20	9M-91 to 9M-94	9M-129 to 9M-132
10M-38 to 10M-41	10M-78 to 10M-82	11M-12 to 11M-15	11M-48 to 11M-51
11M-123 to 11M-127	11M-159 to 11M-163	12M-128 to 12M-131	12M-162 to 12M-166
12M-198 to 12M-202	13M-69 to 13M-73	13M-125 to 13M-131	14M-152 and 14M-153
14M-170 to 14M-176	15M-74 to 15M-80	15M-136 to 15M-142	16M-36 to 16M-40
16M-70 to 16M-75	16M-141 to 16M-146	16M-185 to 16M-191	17M-22 to 17M-25
17M-89 to 17M-91	17M-214 to 17M-219		

FIGURES



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Legend

- Approximate Portal and Shaft Locations
- Approximate Pump Station Locations
- Approximate Tunnel Alignment
- Approximate Cut and Cover Alignment
- Approximate Canal alignment

0 5 10 20 30 Miles

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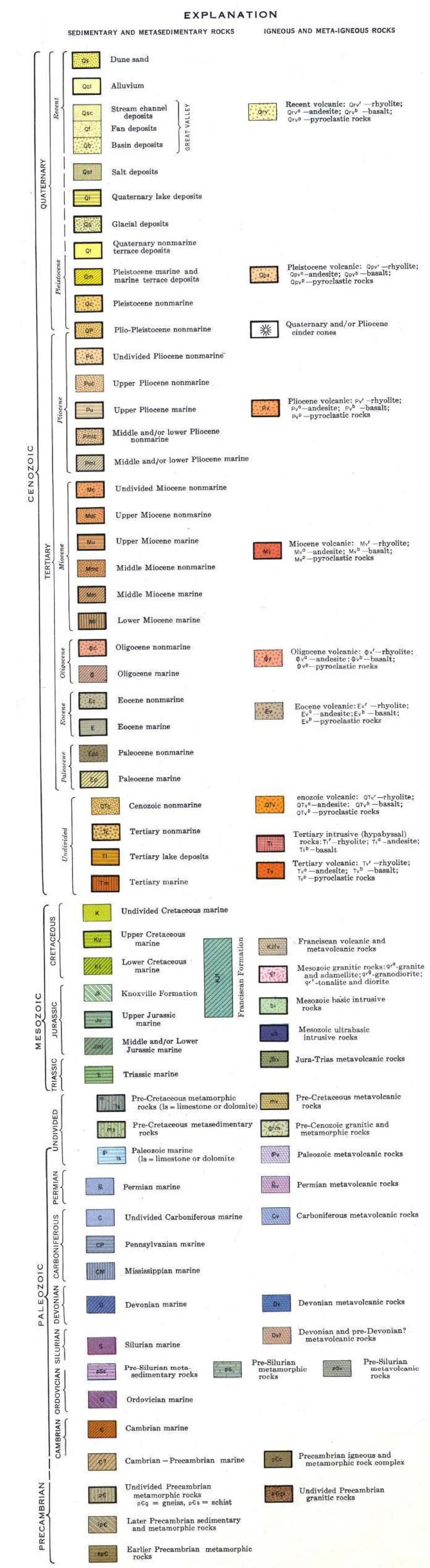
PROJECT NO. 20201537	PROJECT ALIGNMENT MAP	FIGURE
DRAWN BY: T.Cisney		1
CHECKED BY: S.Rugg	SDCWA Regional Conveyance System Study	
DATE: 11-13-2019	San Diego and Imperial Counties, California	
REVISED: -		



LEGEND

- Approximate Pump Station Locations
- Approximate Portal and Shaft Locations
- Approximate Canal Alignment
- Approximate Cut and Cover Alignment
- Approximate Tunnel Alignment
- Approximate Geomorphic Province Boundary
- Approximate Subzone Boundary

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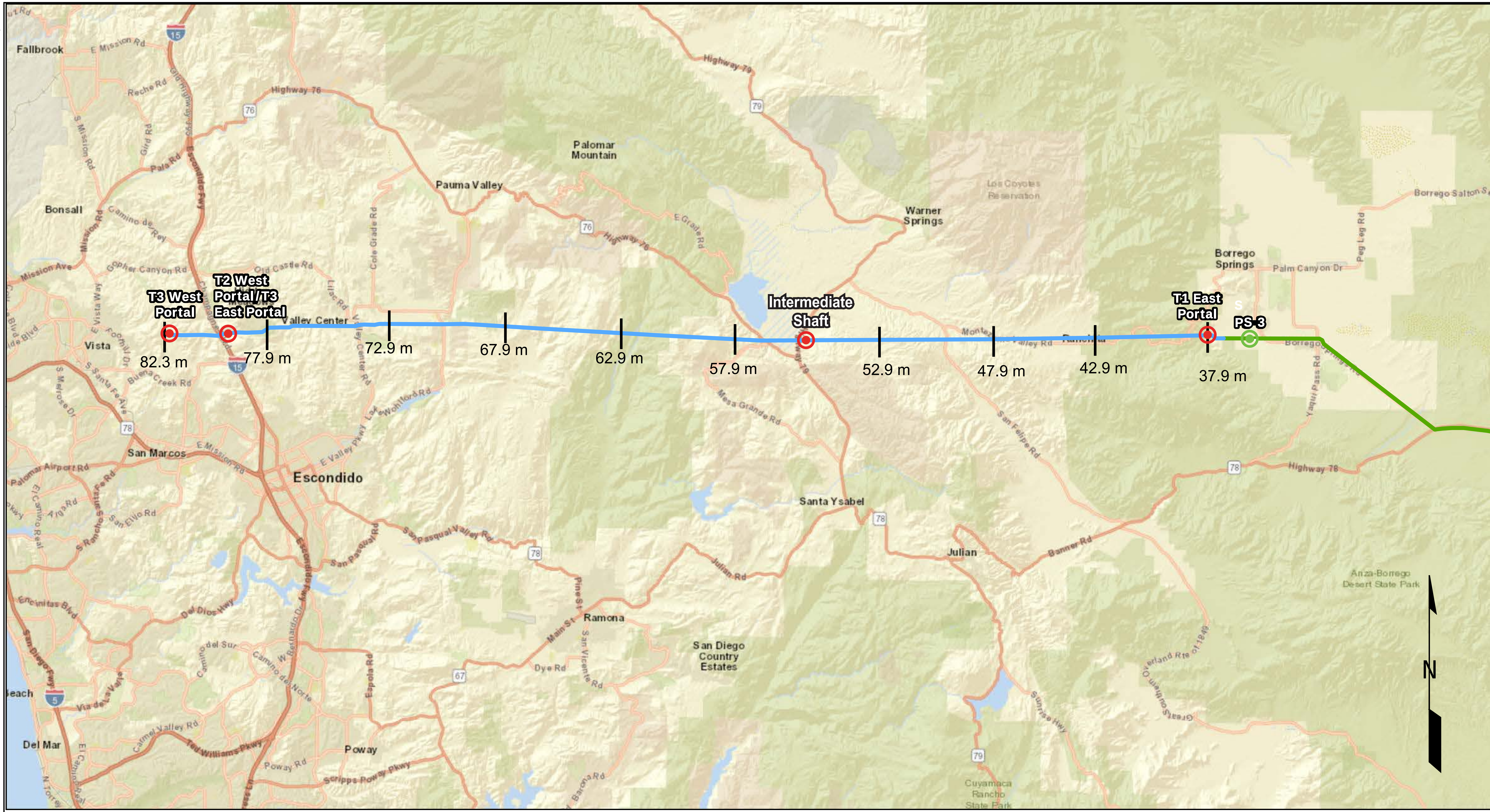
CHECKED BY:	S.Rugg

REGIONAL GEOLOGIC MAP

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CALIFORNIA

FIGURE:

2



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Legend

- Approximate Pump Station Locations
- Approximate Portal and Shaft Locations
- Approximate Tunnel with Mile Markers
- Approximate Cut and Cover Alignment

0 2 4 8 12 Miles

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CHECKED BY:	S.Rugg
DATE:	11-11-2019
REVISED:	-

TUNNEL ALIGNMENT SECTIONS

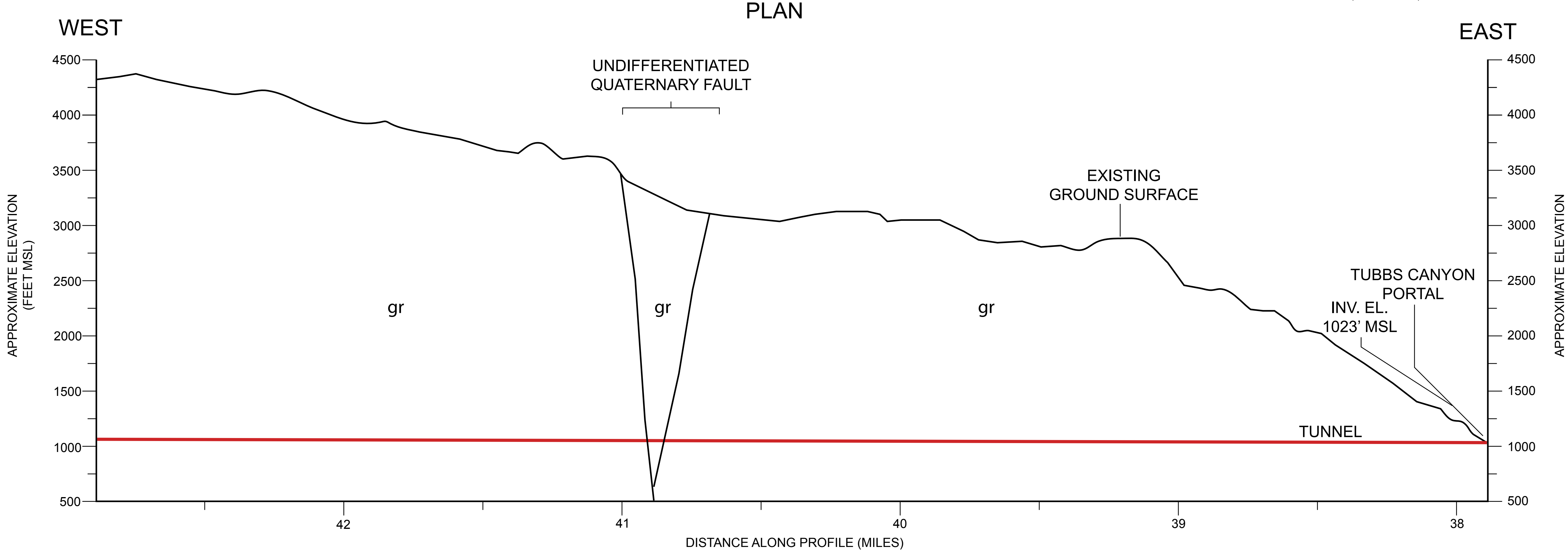
SDCWA REGIONAL CONEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CALIFORNIA

FIGURE

3

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY

PLAN



PROFILE

LEGEND

SYMBOLS	GEOLOGIC UNITS
—?— — —?— APPROXIMATE GEOLOGIC CONTACT	gr - UNDIFFERENTIATED MESOZOIC GRANITIC ROCKS

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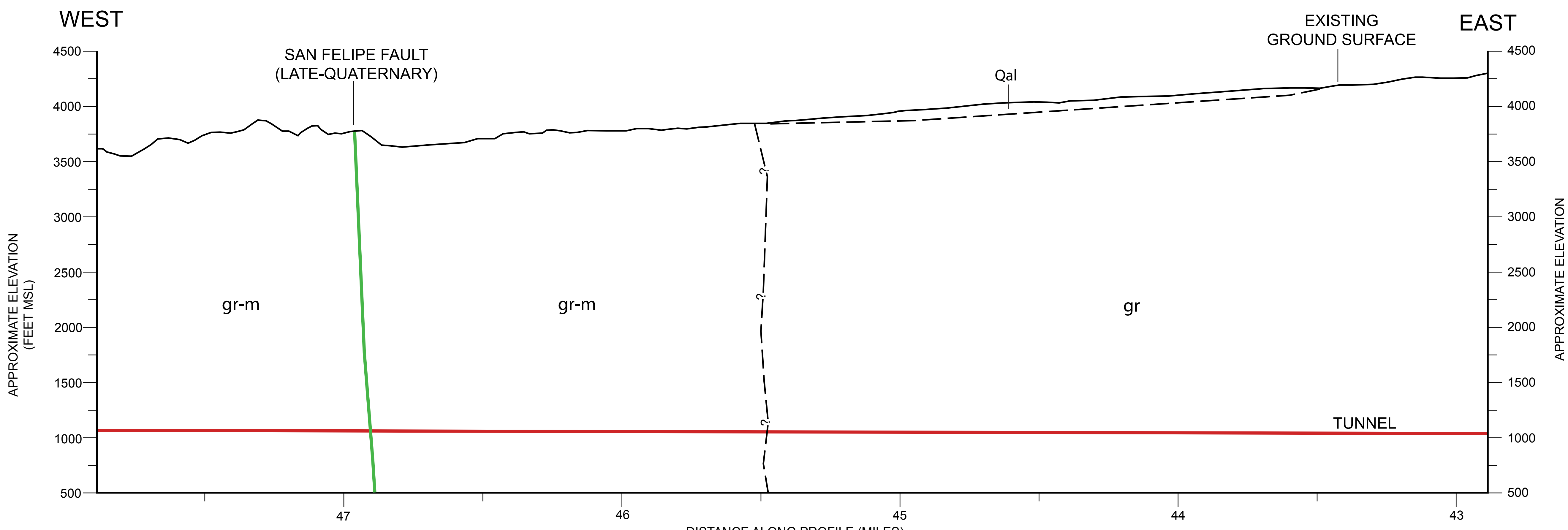
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 37.9 M TO 42.9 M

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
4

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY

PLAN



PROFILE

LEGEND

SYMBOLS

—?— — —?— APPROXIMATE GEOLOGIC CONTACT

LINEAMENT FEATURE

GEOLOGIC UNITS

Qal - ALLUVIAL DEPOSITS

gr - UNDIFFERENTIATED MESOZOIC GRANITIC ROCKS

gr-m - PRE-CENOZOIC GRANITIC & METAMORPHIC ROCKS

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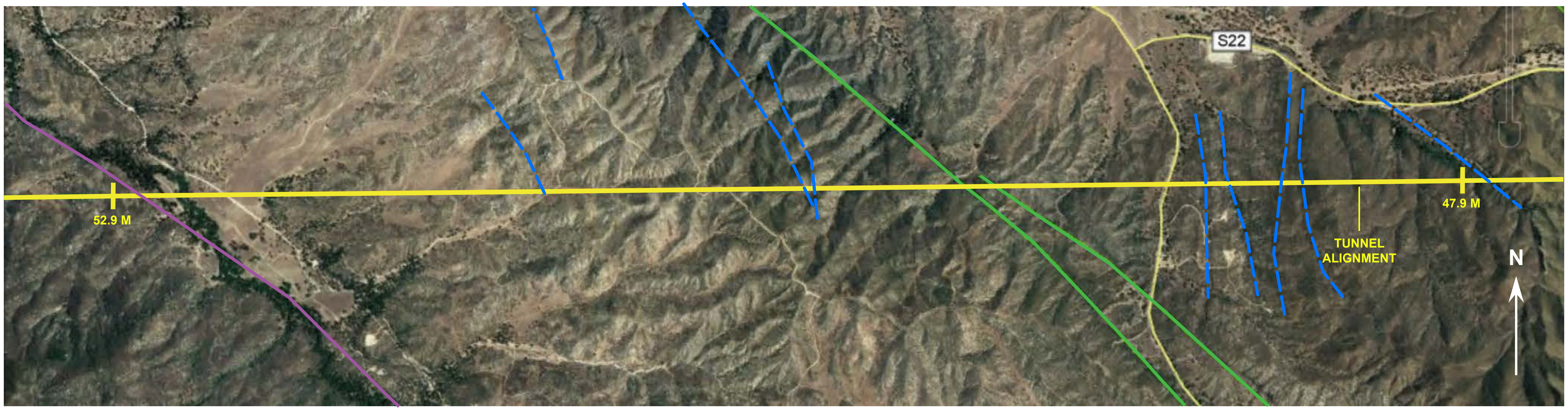
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 42.9 M TO 47.9 M

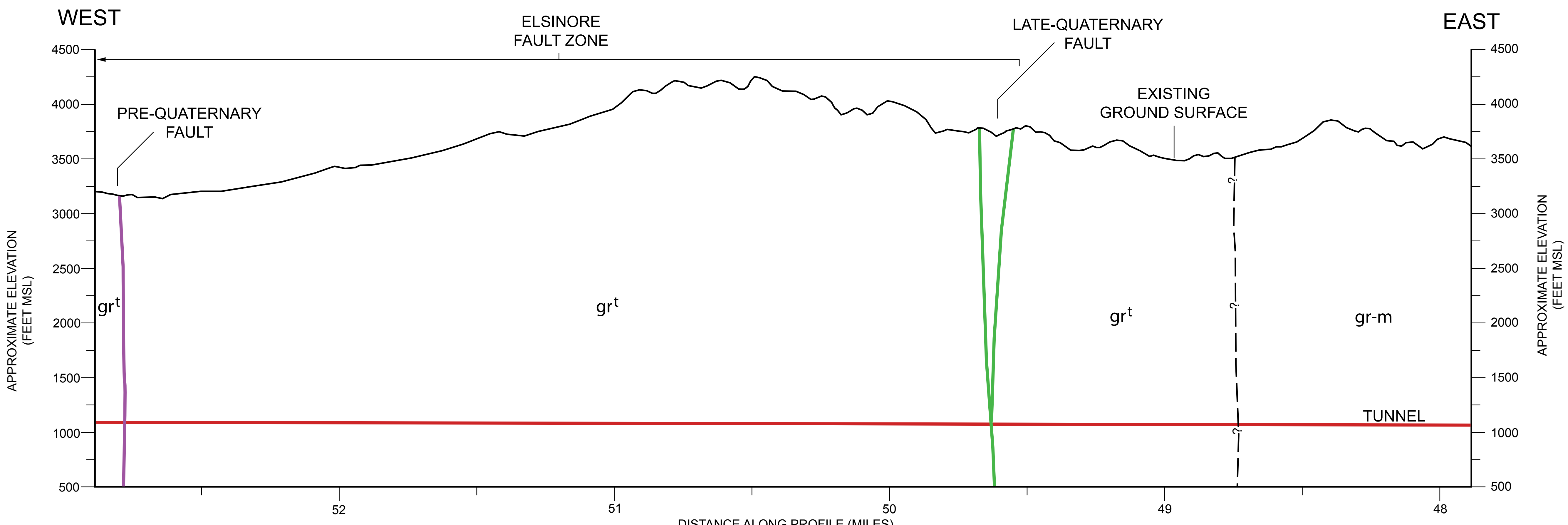
SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE 5



SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY

PLAN



PROFILE

LEGEND

SYMBOLS	GEOLOGIC UNITS
—?— — —?— APPROXIMATE GEOLOGIC CONTACT	gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS
LINEAMENT FEATURE	gr-m - PRE-CENOZOIC GRANITIC & METAMORPHIC ROCKS

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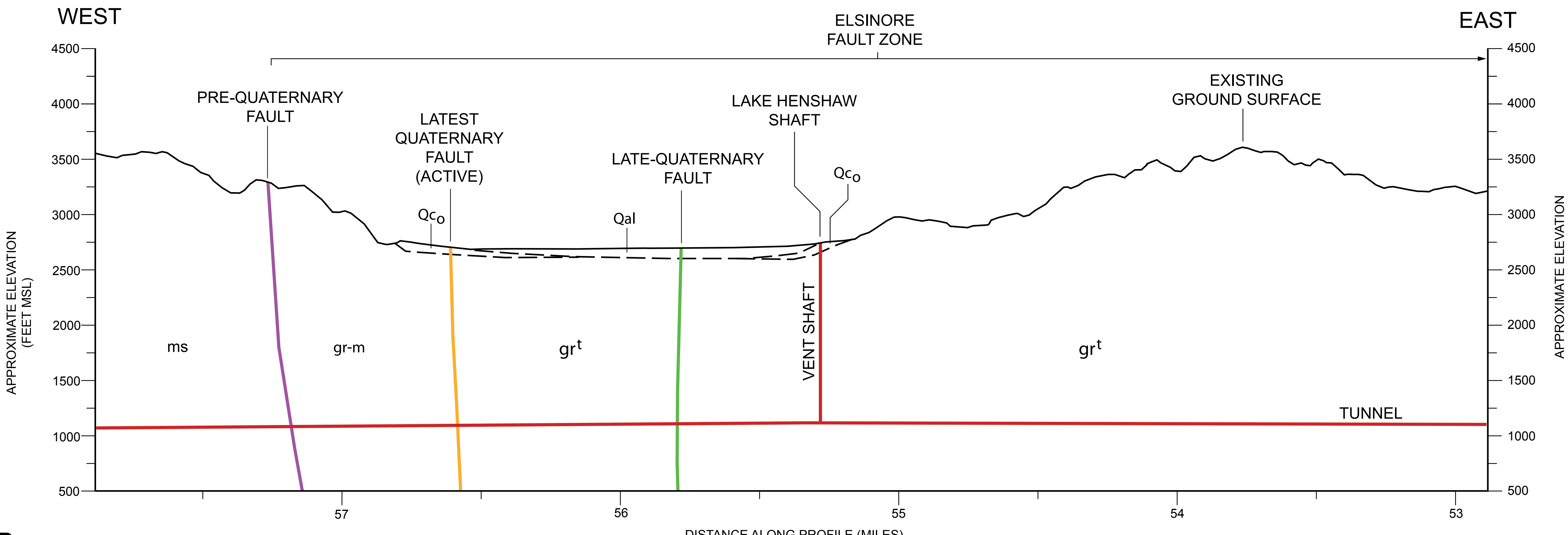
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 47.9 M TO 52.9 M

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
6

PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS

—?— — —?— APPROXIMATE GEOLOGIC CONTACT

— — — — LINEAMENT FEATURE

GEOLOGIC UNITS

Qal - ALLUVIAL DEPOSITS

Qc_o - OLD ALLUVIAL DEPOSITS

gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS

ms - PRE-CRETACEOUS METASEDIMENTARY ROCKS

gr-m - PRE-CENOZOIC GRANITIC & METAMORPHIC ROCKS

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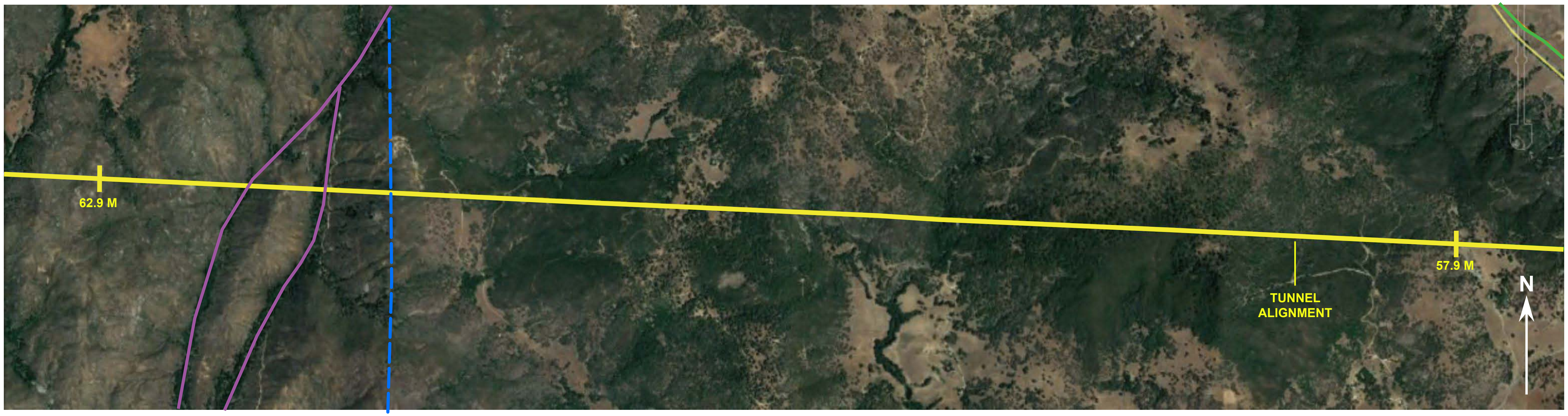
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 52.9 M TO 57.9 M

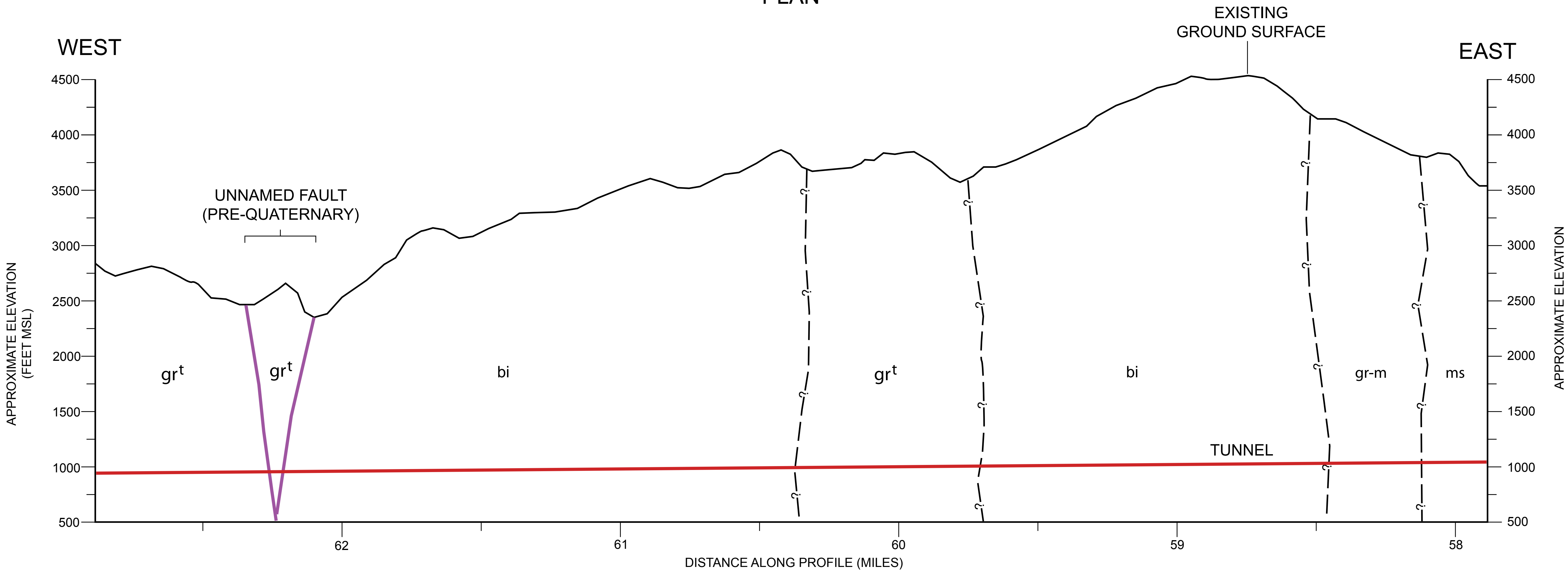
SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
7



PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS

APPROXIMATE GEOLOGIC CONTACT

LINEAMENT FEATURE

GEOLOGIC UNITS

- bi - MESOZOIC BASIC INTRUSIVE ROCKS
- gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS
- ms - PRE-CRETACEOUS METASEDIMENTARY ROCKS
- gr-m - PRE-CENOZOIC GRANITIC & METAMORPHIC ROCKS

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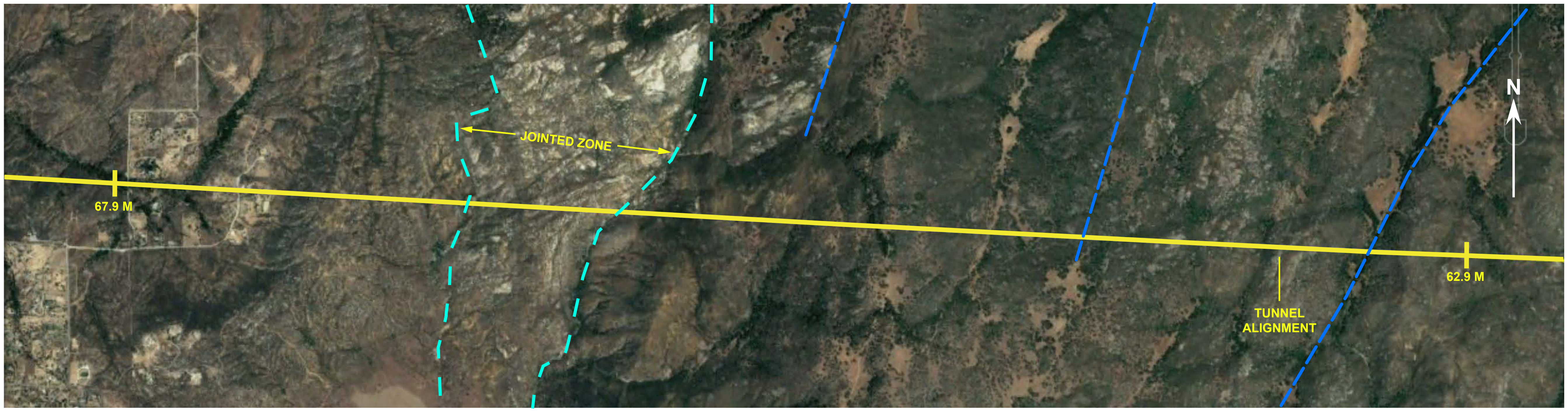
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 57.9 M TO 62.9 M

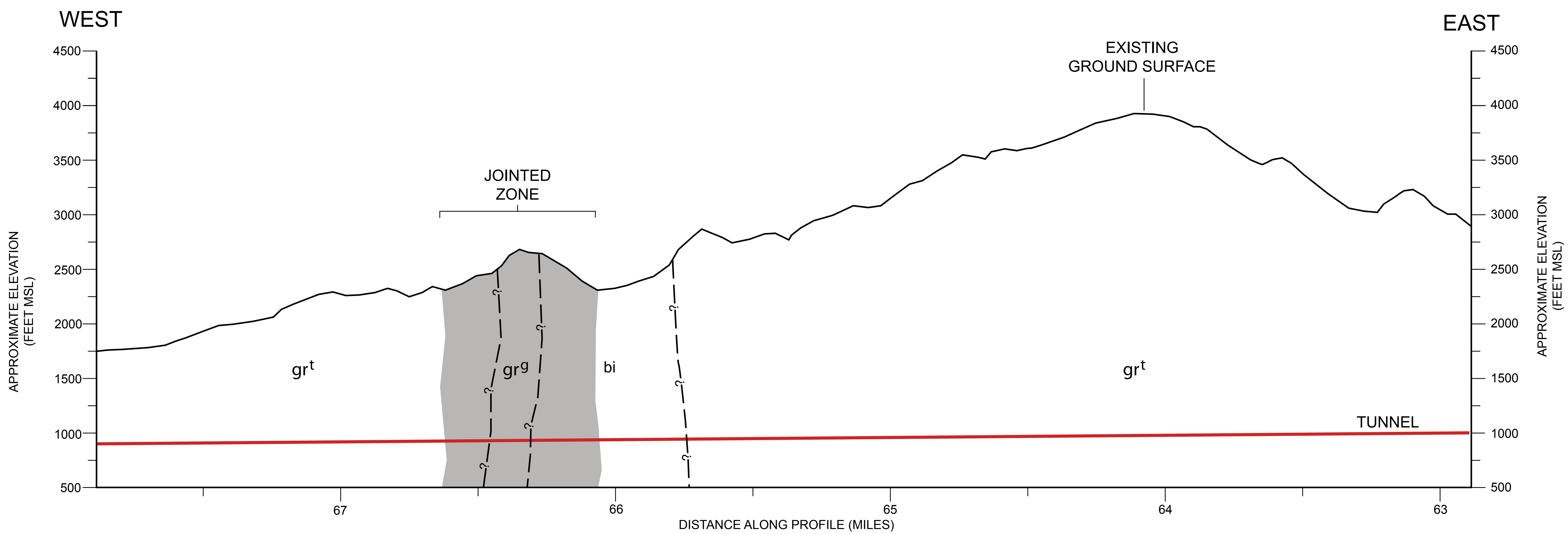
SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
8



PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS	GEOLOGIC UNITS
—?— — —?— APPROXIMATE GEOLOGIC CONTACT	bi - MESOZOIC BASIC INTRUSIVE ROCKS
LINEAMENT FEATURE	gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS
	gr^g - MESOZOIC GRANODIORITE INTRUSIVE ROCKS

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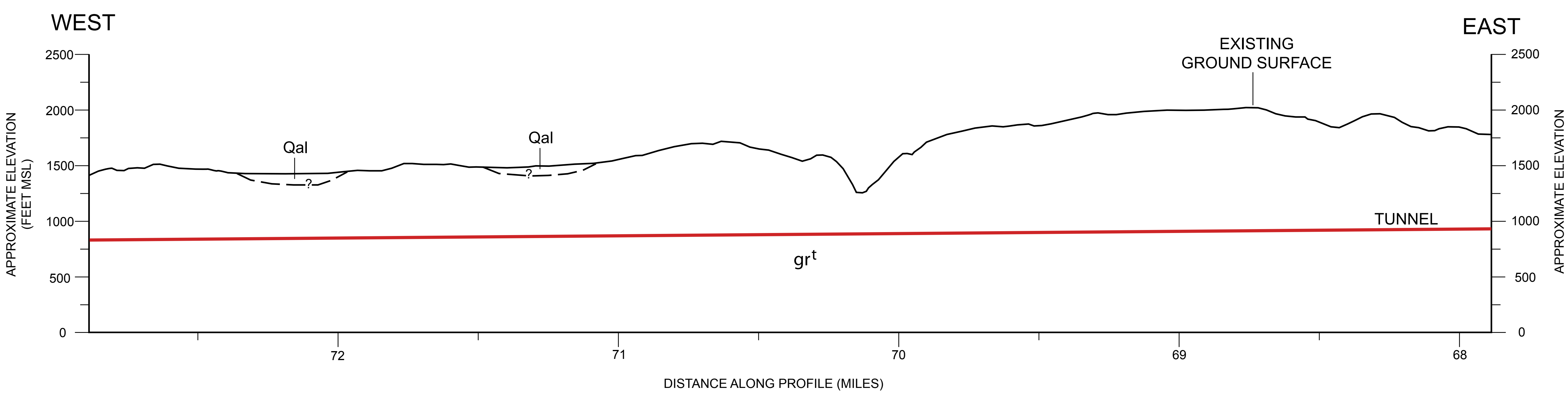
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 62.9 M TO 67.9 M

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
9

PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS	GEOLOGIC UNITS
—?— — —?— APPROXIMATE GEOLOGIC CONTACT	Qal - ALLUVIAL DEPOSITS
	gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS

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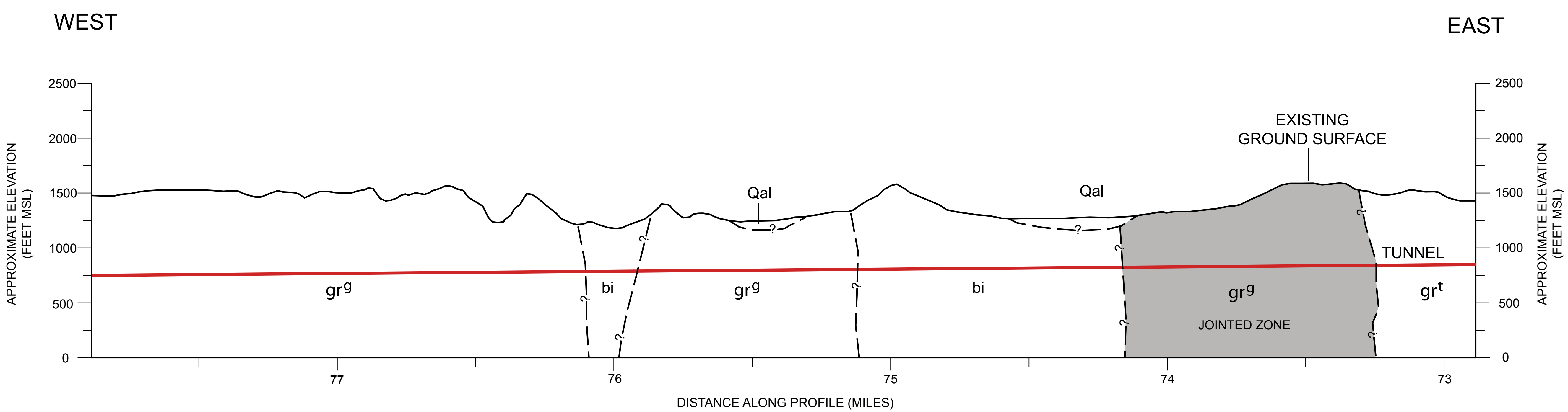
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 67.9 M TO 72.9 M

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
10

PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS

—?— — —?— APPROXIMATE GEOLOGIC CONTACT

GEOLOGIC UNITS

- Qal - ALLUVIAL DEPOSITS
- bi - MESOZOIC BASIC INTRUSIVE ROCKS
- gr^t - MESOZOIC TONALITE TO DIORITE INTRUSIVE ROCKS
- gr^g - MESOZOIC GRANODIORITE INTRUSIVE ROCKS

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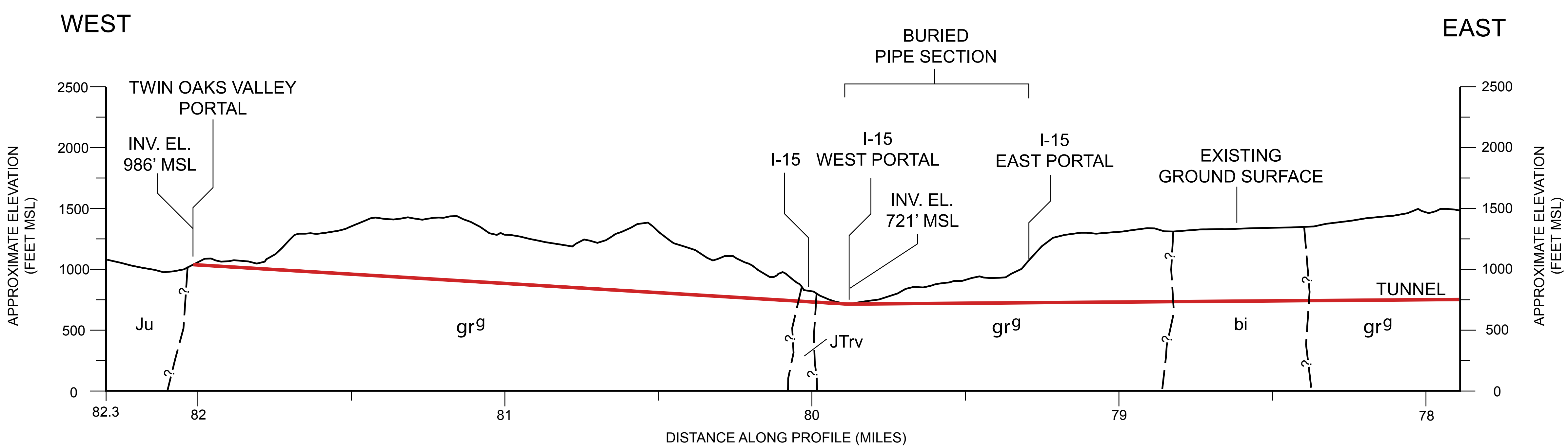
TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 72.9 M TO 77.9 M

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
11

PLAN

SEE REGIONAL GEOLOGIC MAP, FIGURE 2, FOR SURFACE GEOLOGY



PROFILE

LEGEND

SYMBOLS

—?— — —?—
APPROXIMATE
GEOLOGIC CONTACT

GEOLOGIC UNITS

- bi - MESOZOIC BASIC INTRUSIVE ROCKS
- gr^g - MESOZOIC GRANODIORITE INTRUSIVE ROCKS
- JTrv - JURASSIC-TRIASSIC METAVOLCANIC ROCKS
- Ju - JURASSIC METASEDIMENTARY ROCKS

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TUNNEL PLAN AND PROFILE
TUNNEL SEGMENT 77.9 M TO 82.3 M

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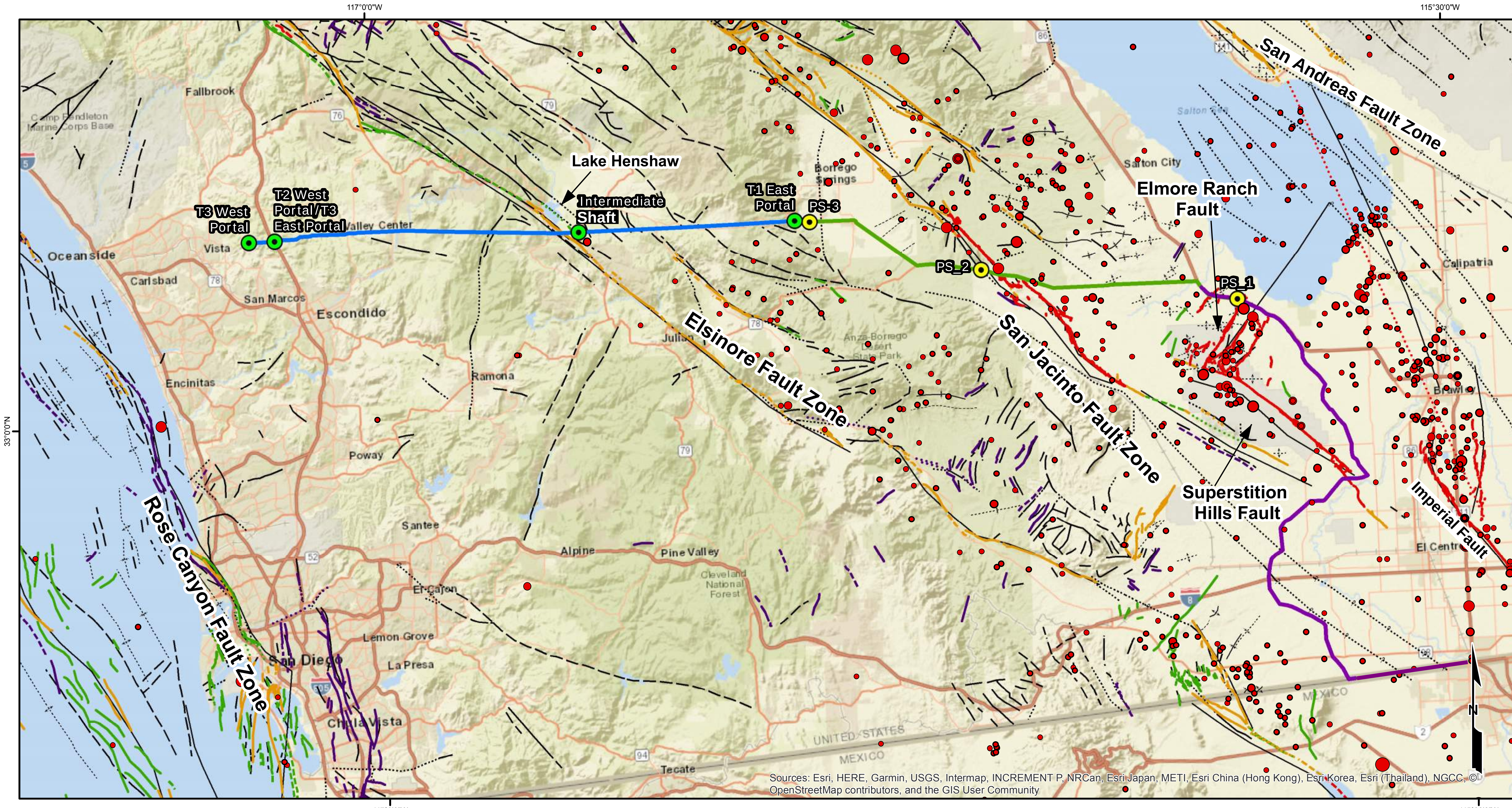
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE
12



Faulting Legend

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed were Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

Quaternary Faults (Bryant, 2005; USGS, 2009)

- Tunnel Alignment
- Cut and Cover Alignment
- Canal Alignment
- Approximate Portal Locations
- Approximate Pump Station Locations

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DRAWN BY:	T.Cisney
CHECKED BY:	S.Rugg
FILE NAME:	Figure 4- EQ 20201537.MXD

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - MAY 2019)

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CALIFORNIA

FIGURE 13

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APPENDIX A
Scope and Fee Estimate for Geotechnical Investigation

APPENDIX A

Scope and Fee Estimate for Geotechnical Investigation

Recommended Scope of Work (Next Steps) – Geotechnical Investigation

Based on information provided, we anticipate approximately 44 miles of tunnel and 83 miles of cut-and-cover pipeline and/or canals along the tunnel alignment. Based on our background review and site reconnaissance, the tunnel portions of the alignment will be excavated in a wide range of granitic and metasedimentary rocks both above and below the groundwater table. The tunnels will also cross several faults and fault zones with anticipated poor ground conditions.

The primary objective of the geotechnical investigation is to provide characterization of the soil and rock conditions along the proposed tunnel alignment. The information is key to predicting ground behavior, addressing excavation and ground support issues, and establishing baseline conditions for tunnel contractor bid preparation. Ground characterization is also essential to identify very hard and/or abrasive rock, to estimate reasonable TMB excavation performance, to estimate groundwater inflows and establish reasonable groundwater control measures, and to assess potential risks such as landslides, seismic shaking, active faults, squeezing ground, and poor ground conditions.

Key geotechnical/geologic/hydrogeological issues to be addressed as part of the investigation for the tunnel portions of the alignment include:

- Rock Strength, Abrasiveness and Hardness;
- Occurrence and degree of Rock Mass Weathering;
- Distribution, Frequency and Characteristics of Rock Mass Discontinuities;
- Poor Ground Conditions
    - Squeezing Ground
    - Faulted Ground
    - Ground Conditions at Areas of Low Overburden
- Active Fault Crossing and Displacement
- Slope Stability at Portal sites
- Flush Flows, Sustained Flows and Depletion of Groundwater Resources;
- Hydrostatic Loads on Tunnel Support and Lining;
- Excessive Rock Loads on Tunnel Supports under High Overburden;
- Groundwater Inflows at Fault Crossings

- Elevated Groundwater Temperatures
- Gassy Ground

The non-tunnel portions of the alignment are primarily in the eastern half of the project and are anticipated to encounter various alluvial soil types as well as local shallow bedrock. Key issues for the non-tunnel portions of the alignment include soil and bedrock excavatability, liquefaction potential, re-use of excavated materials for trench backfill, corrosion potential, shallow groundwater, canal side wall stability, and fault crossings.

Phased Approach

We recommend the geotechnical investigation be divided into phases. The first phase would include additional detailed geologic mapping, development of an exploration plan and limited investigations at key points of interest including portals, shafts, and other areas of concern such as fault crossings.

The second phase would include a design-level geotechnical investigation. Investigation tasks would include deep rock core for the tunnel alignment, shallow borings for the pipeline and canal portions of the alignment, hydraulic conductivity testing, well installations, geophysical surveys, laboratory testing of soil and rock, and report preparation. This phase could also be further subdivided into additional phases which could allow for refinements planning and alignment adjustments as preliminary information is obtained, reviewed and analyzed.

A more detailed discussion of the recommended scope of work is presented below.

Geologic Mapping and Exploration Planning

Detailed geologic mapping is recommended to supplement the reconnaissance mapping performed as part of the Desktop Study. The mapping will provide more detailed geologic information along the alignment as well as focused mapping at portals and shaft locations and areas of anticipated tunnel fault crossings. The geologic mapping will focus more detailed characterization of the rock mass at the surface, delineating the lateral extent of the various rock formations, characterizing the contacts between the various units, and evaluating ground conditions at fault crossings. The mapping will also include joint line surveys at selected outcrops. The line surveys will provide important discontinuity data which can be evaluated together with discontinuity data collected from the later boreholes.

Candidate borehole locations and equipment accessibility will also be identified during the mapping. The results of the geologic mapping will be used to plan and refine a detailed scope of work for the subsurface investigation.

Geophysical Surveys

Geophysical surveys using seismic refraction surveys are recommended at portal areas to evaluate depth of soil and alluvium, depth of weathering, possible landslide materials, evaluate bedrock rippabilitiy and to obtain rock seismic velocities as an indicator or rock mass quality. Seismic refraction surveys may also be used for portions of the pipeline and canal alignment where shallow bedrock is anticipated. Seismic refraction surveys can be completed as part the recommended initial phase of exploration.

Subsurface Investigation

There is currently no accepted industry standard for the number of borings, their spacing and depths for tunnel projects. Each project must be evaluated individually based on the nature of the project and anticipated geologic conditions. Tunnel projects in urban settings generally require more closely spaced borings while more widely spaced borings may be enough in remote areas. However, enough investigation is required to characterize the ground for the project and to make predictions on anticipated ground conditions and behavior along the entire project alignment. This is most important for the construction contractor to know how to bid a tunnel construction project and avoid unanticipated conditions that could cause claims or delays in the construction schedule.

<u>Rock Core Borings – Tunnel Alignment</u>

For exploration planning purposes we estimate 1 deep rock core boring for every 1.1 mile of tunnel alignment. Based on the tunnel profile we estimate the rock core borings will range from approximately 200 feet in depth to as deep as 3,400 feet. Except at the portals, anticipated depths range from approximately 1,400 to 2,900 feet below the existing ground surface (bgs) between mile 40 and 50; 1,600 to 3,400 feet bgs between mile 50 and 60; 950 to 2,900 feet bgs between mile 60 and 70; and 350 to 800 feet bgs between mile 70 and 82. A summary of the number of rock core borings and depths estimated for this project is presented in the table below.

Summary of Rock and Soil Borings

Rock Core and Soil Borings	Depth to 500 ft	1,000 to 3,000 ft (av. 2,000)	3,000 ft	3,000 to 4,000 ft (av. 3,500)	25 to 50 feet	Total No. of Borings/ Footage
Rock Core Borings - 1 Mile Spacing – 44-Mile Tunnel Alignment						
No. of Rock Core Borings	12	26	2	1		41
Rock Core Boring Footage	3,750	48,550	6,100	3,400		61,800
Soil/Rock Core Borings - 1/2 Mile Spacing – 83-Mile Pipeline and Canal Alignment						
No. of Soil/Rock Core Borings	0	0	0	0	162	162
Soil/Rock Core Boring Footage	0	0	0	0	8,100	8,100

The number of rock boring borings may be reduced following the geologic mapping and preparation of the field exploration plan. However, we do not recommend eliminating some of the deeper borings as these exploration points are key to evaluating the maximum hydrostatic pressures at tunnel depth.

We anticipate many of the proposed drill locations can be planned along or adjacent to existing roadways or forest service access roads. However, due the remoteness of some areas, helicopter access will be required for some of the borings.

Because the proposed rock core borings are deep, it is important that the equipment be sized appropriately for the length and weight of drill rod string as well as the fluid circulation system capacity to lift cuttings efficiently. It is also important to match drilling equipment to the task and select a drilling company with a proven track record for achieving similar project goals. Kleinfelder has successfully completed several deep rock core boring projects in southern California using a drilling subcontractor with experience in deep rock core drilling up to 3,900 feet and experience in remote helicopter drilling operations. Rock core borings should be drilled using diamond core drill rigs equipped with HQ3 drill rods and core barrel and wireline triple tube coring systems.

Rock Core Sampling and Logging

Rock core samples will be HQ3 size. The HQ-size core size typically results in good core recovery and is practical for laboratory testing. The core samples will be obtained from the HQ3 core barrel in 5-foot runs. Core evaluation will be performed in the field by the geologists responsible for collecting and logging the recovered core. Field logs will be prepared as the core is recovered, cleaned, and placed in wooden core boxes. Physical characteristics of the rock lithology and discontinuities for each core run will be recorded on core logs in accordance with USBR and International Society of Rock Mechanics (ISRM) suggested methods. The field logs will include: driller and coring information such as depth and duration of core run, drill fluid gains or losses, drilling conditions, length of core runs, core recovery, and rock quality designation (RQD); descriptions of lithology, mineral content, texture, and weathering/alteration; descriptions of discontinuities including type, dip with respect to core axis, spacing, condition of discontinuities and other observations made by the field geologist.

Each core run is photographed under natural lighting prior to transfer to wooden core boxes for transport and storage. After each core box is filled and labeled, the core will again be photographed in natural light to document the condition of the core. Rock core recovered from the field investigation will be transported to our laboratory for further review and for selection of samples for laboratory testing. After field logs are completed, they will be checked (QC) against the boxed cores by the supervising CEG. After the QC review, the logs will be converted to final drill logs using the gINT logging program.

Corehole Video Log

Following completion of coring activities, we recommend a down-hole Optical Televiewer (OTV) or Acoustical Televiewer (ATV) survey be performed in each boring. The main purpose of the OTV/ATV survey is to obtain in-situ orientations of rock discontinuities (joints, shears, bedding) and observe aperture and filling of discontinuities in situ. Features such as lithologic changes, joint systems, bedding or other features can also be viewed in situ and correlated to the core. While the OTV/ATV log is being run, a permanent continuous optical record is created that is oriented with reference to magnetic North. The orientations of the discontinuities are then plotted on stereonets using the computer program ROCKPACK III by C.F. Watts (2001). Stereonets provide a two-dimensional representation of the three-dimensional discontinuity data. These stereonets can be used for analyses related to tunnel crown stability and structural reinforcement needs.

Additional Down-Hole Geophysical Surveys

Kleinfelder is aware of the need to accurately characterize the hydrogeologic regime along the tunnel alignment. Recognition of separation between upper and lower groundwater regimes is important because of the large potential hydraulic head over the tunnel if the groundwater is one interconnected system. We propose several geophysical surveys in the rock core borings to analyze differing hydraulic conditions at greater depths. We propose to run a suite of tests with down-hole geophysical tools to identify entry points of formation groundwater and flow directions within the borehole (vertical gradients). The geophysical tools proposed are Fluid Temperature, Fluid Conductivity, E-Log Gamma, and Heat Pulse Flow Meter. The Fluid Temperature and Fluid Conductivity, especially when plotted together, can highlight anomalies in temperature or conductivity that result from water entering the corehole through fractures. Such zones can be identified and viewed with OTV/ATV. Different hydraulic zones can also be tested for flow direction (either up- or down-hole gradients) and volume of flow from one zone to another (evidence of separate hydraulic zones) using the Heat Pulse Flow Meter. This tool releases a heat tracer at time zero and then detects effects at specified distances from the source. Direction (up- or down-hole) and distance are determined from the time data, and flow velocity and flow volume can be calculated based on the corehole diameter. In our opinion, these surveys are prudent to include with the OTV/ATV survey. These data, when used together, provide corroborative evidence for understanding the groundwater regime within the rock mass that the tunnel will penetrate.

Hydraulic Conductivity Testing

Hydraulic conductivity testing should be conducted using both single and dual packer systems to isolate and straddle selected test sections, both during and upon completion of drilling as may be appropriate based on field conditions. The test sections will be selected during drilling based on the general character of the core recovered (or lack of core depending on degree of fracturing) and drilling characteristics such as loss of drilling fluid or increase of drilling fluid (groundwater inflow, indicating increased hydraulic conductivity or a vertical hydraulic gradient). Tabulated test results for packer testing will include elapsed time; gauge and transducer pressure; meter reading; water quality; and flow rate. The tabulated data will be used to calculate an "effective coefficient of permeability" of the rock mass at the test interval. Results of the packer permeability testing will be tabulated in individual summary sheets and will be reported on the corehole log sheets in terms of hydraulic conductivity. To estimate the flow of water from the rock mass in a tunnel, the hydraulic conductivity from all test sections should be evaluated collectively.

Transducer Installations

We recommend vibrating wire pressure transducers be installed in several of the rock core borings. The pressure transducers will be used to evaluate and monitor hydraulic pressure within the subsurface above the tunnel. The pressure transducers will allow measuring true water pressures at different depths. The number and locations of rock core borings with transducers installed will be determined at the time of the field investigation. For preliminary planning we estimate up to 6 rock core borings may be selected for transducer.

Soil/Rock Core Borings - Pipeline and Canal Alignment

For exploration planning purposes we estimate 1 shallow soil and/or rock boring for every ½ mile of pipeline or canal alignment. We propose to explore the subsurface conditions along the proposed 83-mile-long pipeline and canal portion of the alignment by drilling 162 borings with truck-mounted, track-mounted, and all-terrain 4WD drill rigs using auger drilling and rock core methods. We anticipate some of the sites will require an all-terrain drill rig for access. The shallow borings are estimated to be 25 to 50 feet deep, depending the invert depth of the pipe/canal. Of these borings, we estimate several may encounter shallow granitic or metamorphic bedrock at or near the ground surface. The remainder of the borings are anticipated to encounter recent alluvium, alluvial fan deposits, older alluvium, and/or Pleistocene sediments.

In general, most borings in alluvial soils or Pleistocene materials will be drilled using a hollow-stem auger drill rig. Borings encountering shallow groundwater, running sands or flowing ground beneath the water table will be converted to mud rotary drill techniques to achieve the required penetration. Borings in areas underlain by hard metamorphic and granitic bedrock will be drilled using wireline diamond bit core drilling methods.

Samples will be collected from the borings using Standard Penetration test (SPT) and/or Modified California samplers (MCS) driven 18 inches into the soil using a 140-pound hammer falling 30 inches. Disturbed samples and bulk samples will also be obtained from each boring.

Laboratory Testing

Laboratory testing will be designed to define the general rock mass engineering properties throughout the depth of each rock core boring. Samples collected for testing will include both high- and low-strength samples to represent the range of strengths encountered. Specialized testing may be required for lower strength materials including clay gouge, sheared rock or weak rock. This diversity of sample depths should represent a range of rock conditions roughly equivalent to the conditions expected during tunneling from a weathered rock portal to the heart of the mountain.

For soil samples, laboratory testing will be performed on samples obtained during our field investigation to assess the engineering and index properties of the subsurface materials. We anticipate the tests will include unit weight, moisture content, direct shear strength, grain size analysis, modified proctor, Corrosion, and Atterburg limits. The proposed testing program is shown in the accompanying table.

Laboratory Testing Summary

Test Name	Purpose
Rock	
Unconfined Compression (UC)	Evaluate the highest rock strengths
Confined Compression (CC, Hoek triaxial cells)	Evaluate weaker rock strengths
Direct Shear	Evaluate strength of weak discontinuities
Cerchar Abrasion Test	Evaluate wear of rock on TBM cutters
Brazilian Tensile Strength	Evaluate strength of rock in tension
Punch Test	Evaluate strength of confined rock for cutter design
Petrographic Analysis	Identify minerals, especially quartz content, and micro discontinuities, fractures and weathering.
Point Load Test	Field index test for estimating rock strength ranges
Soil	
Moisture Determination/Unit Weight	In situ Moisture/Dry Density
Direct Shear Strength	Evaluate soil strength
Sieve Analysis	Characterization of grain size distribution for pipeline and canal
Resistivity, pH, Sulfate Content, Chloride Content	Corrosivity Evaluation
Modified Proctor	Maximum Density for trench backfill
Atterberg Limits	Characterization of soils for pipeline and canal
Expansion Index	Characterization of soils for pipeline and canal

Geotechnical Report

A Geotechnical Data Report (GDR) will be prepared summarizing the field investigations, laboratory testing, methods utilized, and data collected for the project. The GDR will include the following: 1) a discussion of exploration and testing methods, including conditions and difficulties encountered in the field or laboratory that affect results; 2) maps and figures as aids in documenting field exploration and testing; 3) presentation of all field data and laboratory test results data; 4) core logs; and 5) summary tables and figures of laboratory and field test results. Summary tables and figures in the GDR will include:

- Plots of fracture density, aperture size, hydraulic conductivity, JRC, point load strength, and other key parameters versus depth of corehole;
- Stereonet plots of discontinuity poles for primary and secondary joint/foliation systems;

- Correlation plots of geophysical data, hydraulic conductivity, strength data, aperture size, and fracture density;
- Summary plots of strength test data, including tensile strength, unconfined compression strength versus depth and confined compressive strength versus depth;
- Histograms of permeability, fracture density and orientation, strength data, and other key parameters; and
- Summary tables of strength, permeability and discontinuity data, including discontinuity characteristics (roughness, surface shape, orientation, spacing aperture, filling etc.).

A draft copy of the GDR can be presented for review and comment. The final GDR will be signed and stamped by a California Registered Civil or Geotechnical Engineer, and a California Certified Engineering Geologist.

Estimated Fee

We have prepared a rough order of magnitude estimated cost for the proposed geotechnical investigation outlined in the preceding section of this report. The ROM cost to conduct a design-level geotechnical investigation is based on our past experience in pricing and implementing geotechnical studies of similar size projects, comparable site conditions and design requirements for tunnels, cut and cover pipelines, and canal conveyance systems. Based on the number and depth of the proposed borings we estimate an approximate fee of **$53.7M** as outlined in the table below.

Alignment	3A
Tunnel Miles	44.5
Pipeline and Canal Miles	83
Tunnel Boring Spacing	1.1 mile
Estimated number of tunnel borings	41
Estimated rock core footage - tunnel	61,800
Pipeline and Canal Boring Spacing	1/2 mile
Estimated number of pipeline/canal borings	162
Estimated soil/rock core footage - pipeline/canal	8,100
Estimated Geotechnical Investigation Cost	**$53,717,240**

This estimate is for planning purposes and does not account for several unknowns including drill rig access, number of helicopter sites, permit requirements, etc. The total cost could also be reduced if the number of borings can also be reduced.

APPENDIX B
Project Photos

THE FOLLOWING PHOTOS 1 - 64 WERE TAKEN FROM EAST TO WEST ALONG SR86 AND SR78 STARTING AT THE NORTH END OF THE WEST SIDE CANAL (WSC) TO THE PROPOSED TUNNEL PORTAL NEAR BORREGO SPRINGS AT TUBBS CANYON. THIS STRECTH OF THE PROJECT IS COMPRISED OF CUT AND COVER PIPELINE CONSTRUCTION.

PHOTO 1. MP 0.0 Northern End West Side Canal (WSC).

PHOTO 2: MP 0.0 Lacustrine deposits east of WSC.

PHOTO 3. MP 0.0 Lacustrine deposits east of WSC.

PHOTO 4: MP 0.0 close-up of lacustrine deposits.

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PHOTO 5: MP 0.45 looking north, east of SR86.

PHOTO 6: MP 0.84 looking north, east of SR86.

PHOTO 7: MP 1.4 looking north, east of SR86.

PHOTO 8: MP 2.5 looking north, east of SR86.

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PHOTO 9: MP 3.6 looking north, east of SR86.

PHOTO 10: MP 4.6 looking north, east of SR86 at Elmore Ranch.

PHOTO 11: MP 0.5.3 looking north, east of SR86.

PHOTO 12: MP 5.7 looking north, east of SR86.

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FILE NAME:			

PHOTO 13: MP 6.4 looking north, east of SR86.

PHOTO 14: MP 6.8 looking north, east of SR86.

PHOTO 15: MP 7.5 looking north, east of SR86.

PHOTO 16: MP 8.4 looking west to north of SR78 near intersection of SR86.

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PHOTO 17: MP 8.4 in San Felipe Creek south of SR78. Lacustrine deposits in outcrop

PHOTO 18: MP 8.4. Close up of outcrop in Photo 17.

PHOTO 19: MP 9.1. Outcrop of lacustrine deposit on north side of SR78.

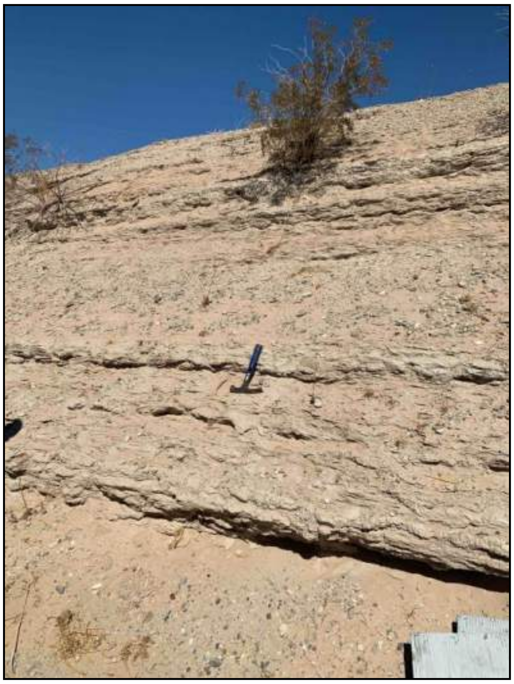

PHOTO 20: MP 9.7. Roadcut of Brawley Formation on north side of SR78.

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FIGURE

B5

PHOTO 21: MP 9.7. Overview of Roadcut of Brawley Formation on north side of SR78.

PHOTO 22: MP 9.7. View west of Roadcut.

PHOTO 23: MP 11.2. Roadcut of Brawley Formation on south side of SR78. Bedding is steeply dipping to the south.

PHOTO 24: MP 11.2. Close up Roadcut in Photo 23.

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PHOTO 25: MP 13. View looking west on SR78
Across drainage feature.

PHOTO 26: MP 15.5. Pole Line Road north
of SR78.

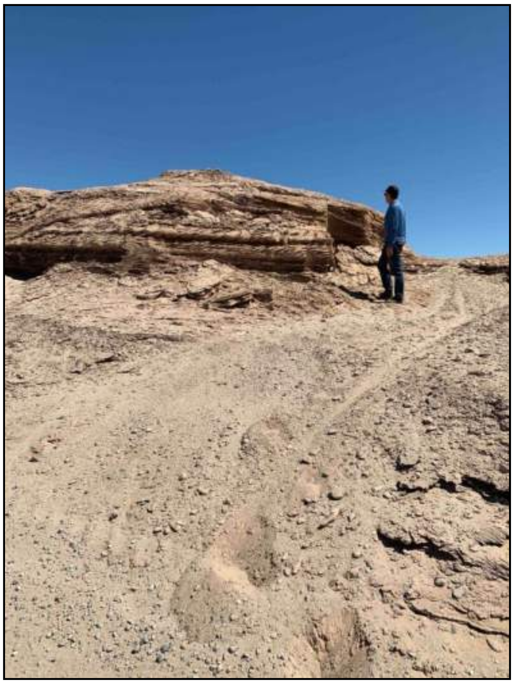

PHOTO 27: MP 15.5. Outcrops of Ocotillo Formation
approximately 0.5 miles north of SR78

PHOTO 28: MP 15.5. Close up of sandstone in
Photo 27.

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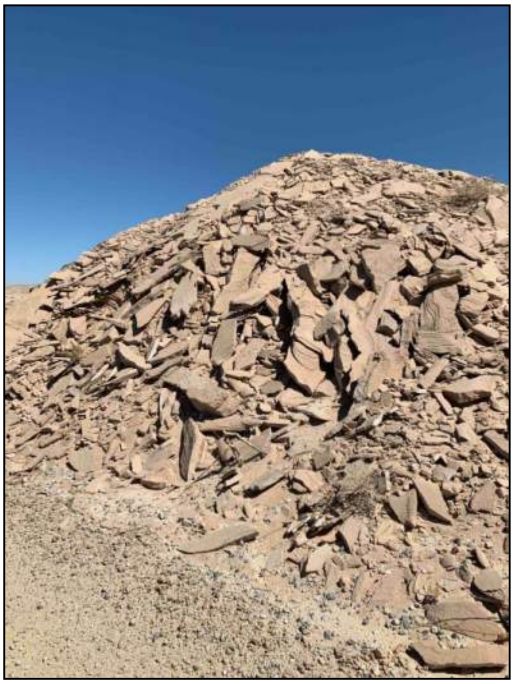

PHOTO 29: MP 15.5. Ocotillo Formation 1.5 miles north of SR78.

PHOTO 30: MP 15.5. Close up of sandstone in Photo 29.

PHOTO 31: MP 15.5. Landscape north of SR78.

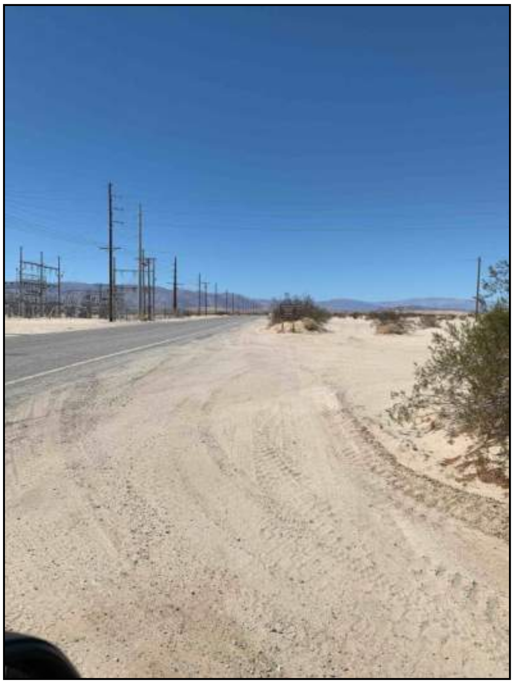

PHOTO 32: MP 15.5. View west from SR78l.

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	FILE NAME:		

PHOTO 33: MP 16.6. Landscape north of SR78).

PHOTO 34: MP 18 . View west along SR78.

PHOTO 35: MP 18.7. Cahuilla Road north of SR78.

PHOTO 36: MP 18.7. Conglomerate 1 mile north of SR78.

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FIGURE

B9

PHOTO 37: MP 20.7. View west along SR78.

PHOTO 38: MP 20.7. Landscape 0.5 miles north
Of SR78.

PHOTO 39: MP 23.7. Wolfe Well Road north of SR78.

PHOTO 40: MP 24.5. SR78 at Ocotillo Wells near crossing of San Jancinto fault.

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PHOTO 41: MP 24.5. View of roadcut on south side of SR78.

PHOTO 42: MP 24.7. View west along SR78 near airport.

PHOTO 43: MP 26. View west along SR78.

PHOTO 44: MP 26. Landscape north of SR78.

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PHOTO 45: MP 29.8. Private Road north of SR78.

PHOTO 46: MP 29.8. Landscape northeast of SR78.

PHOTO 47: MP 31.5. Roadcut of alluvial fan deposits on east flank of San Felipe Creek.

PHOTO 48: MP 31.5. Close up of alluvial fan deposits on east flank of San Felipe Creek.

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FIGURE

B12

PHOTO 49: STOP 28.

PHOTO 50: STOP 28.

PHOTO 51: MP 31.5. View northeast of roadcut of at crest on east flank of San Felipe Creek.

PHOTO 52: MP 31.5. Boulders and cobbles in alluvium.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B13

PHOTO 53: MP 32.6. Roadcut of alluvial fan deposits on west flank of San Felipe Creek.

PHOTO 54: MP 40.2. View west approximately 0.5 Miles north of alignment down private road near S3. View is toward proposed tunnel portal in hillside.

PHOTO 55: MP 41.2. View west 0.5 miles north of alignment toward tunnel portal.

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PHOTO 57. View towards west of Tubbs Canyon Portal area, MP 37.9.

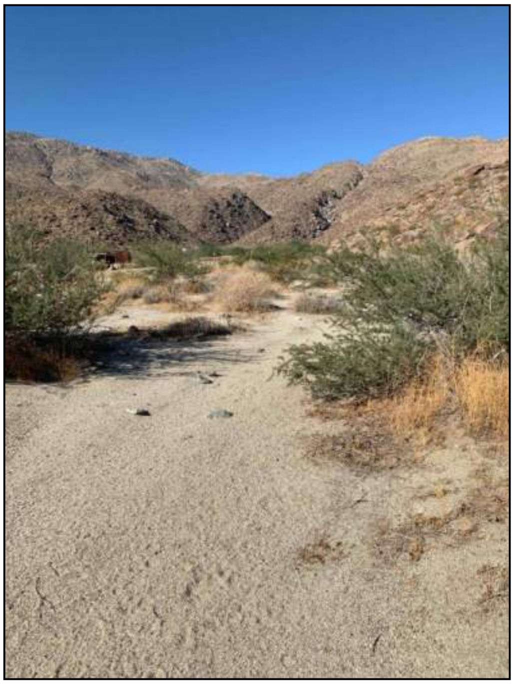

PHOTO 58: Tubbs Canyon Portal area.

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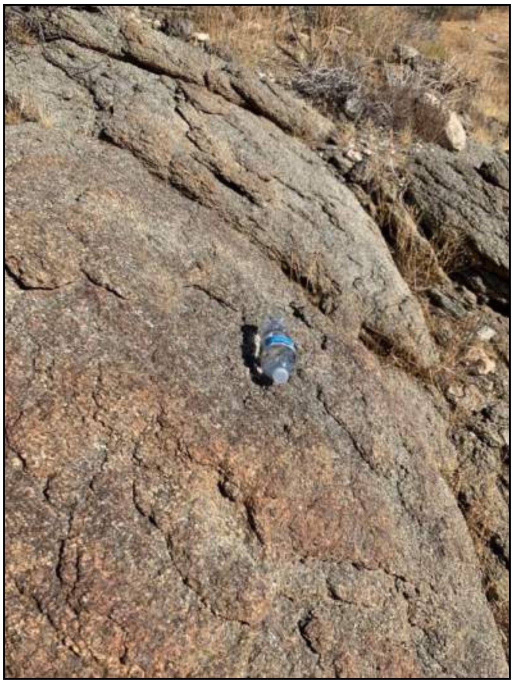

PHOTO 59: Highly weathered granitic rock near Tubbs Canyon Portal.

PHOTO 60: Closeup of granitic rock near Tubbs Canyon Portal.

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PHOTO 61: Outcrop of granitic rock near Tubbs Canyon Portal showing east dipping foliation.

PHOTO 62: Closeup of outcrop near portal location.

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THE FOLLOWING PHOTOS 63 -104 WERE TAKEN AT ROADCUTS ALONG MONTEZUMA VALLEY ROAD (S22) BETWEEN BORREGO SPRINGS AND THE OVERLOOK NEAR PROJECT MILE POST MP 43.3. MOST OF THESE PHOTOS ARE WELL NORTH OF THE PROJECT TUNNEL ALIGNMENT. THE PHOTOS ARE ARRANGED IN ORDER FROM THE NORTH TO SOUTH ALONG S22.

PHOTO 63: Outcrop of highly weathered metamorphic rock near north side of S22.

PHOTO 64. Closup of the outcrop in photo 63.

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PHOTO 65: Closeup of outcrop of metamorphic rock.

PHOTO 66: Outcrop of metasedimentary rock.

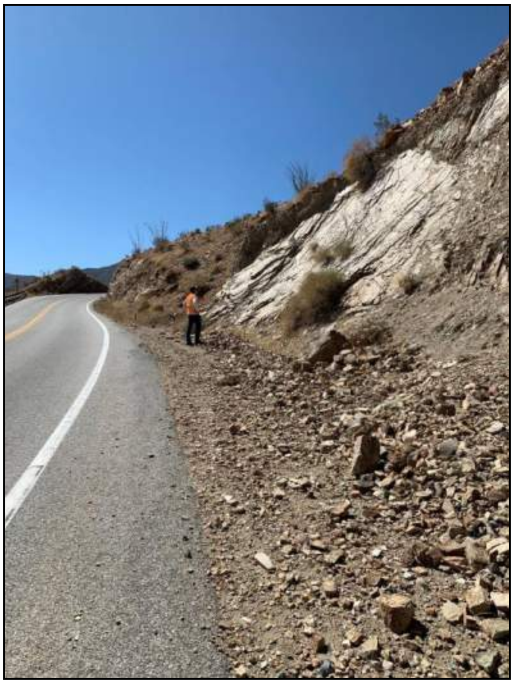

PHOTO 67: Outcrop of metasedimentary rock displaying steep bedding dipping east.

PHOTO 68: Outcrop highly weathered metamorphic rock.

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FIGURE

B18

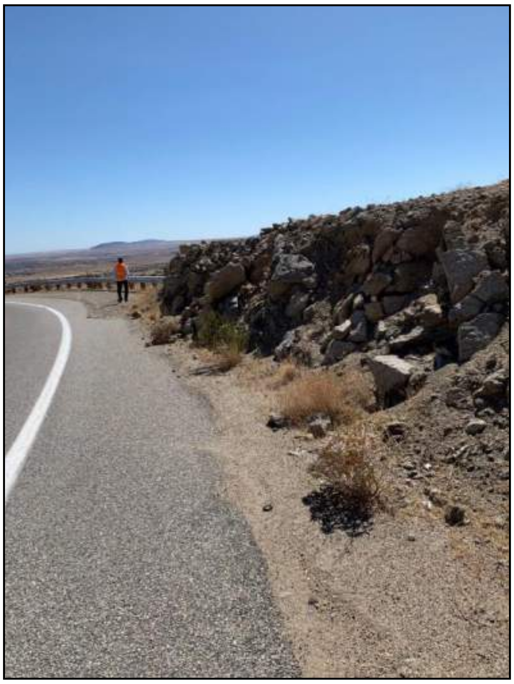

PHOTO 69: Moderately weathered granitic rock.

PHOTO 70: Closeup of the outcrop in photo 69.

PHOTO 71: Outcrop of metasedimentary rock.

PHOTO 72: Outcrop of metasedimentary rock.

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PROJECT PHOTOS

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B19

PHOTO 73: Sheared rock likely due to nearby faulting.

PHOTO 74: Outcrop of granitic rock.

PHOTO 75: Closeup of granitic rock displaying east dipping foliated texture.

PHOTO 76: Granitic rock with east dipping foliation.

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FILE NAME:			

PHOTO 77: Outcrop of weathered granitic rock surface showing overall east dipping structure.

PHOTO 78: Outcrop highly weathered granitic rock.

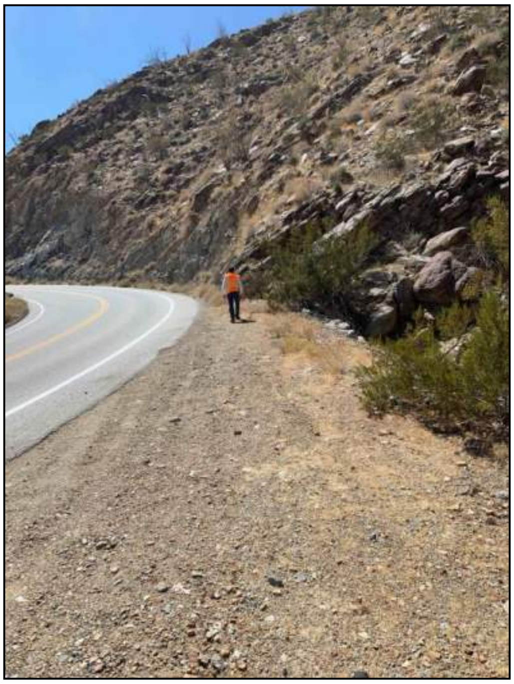

PHOTO 79: Granitic rock with east dipping structure.

PHOTO 80: Closeup of foliated texture.

PHOTO 81: Highly fractured granitic rock.

PHOTO 82: Close up of outcrop in photo 81.

PHOTO 83: Steep cut in granitic rock. Notice weathered brown natural ground surface in background and fresh to slightly weathered gray rock in cut face.

PHOTO 84: Highly fractured granitic rock in road cut.

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FIGURE

B22

PHOTO 85: Cut in granitic rock.

PHOTO 86: Closeup of cut on photo 85.

PHOTO 87: Cut exposing granitic rock

PHOTO 88: Granitic road cut.

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FILE NAME:			

PHOTO 89: Steep cut in granitic rock.

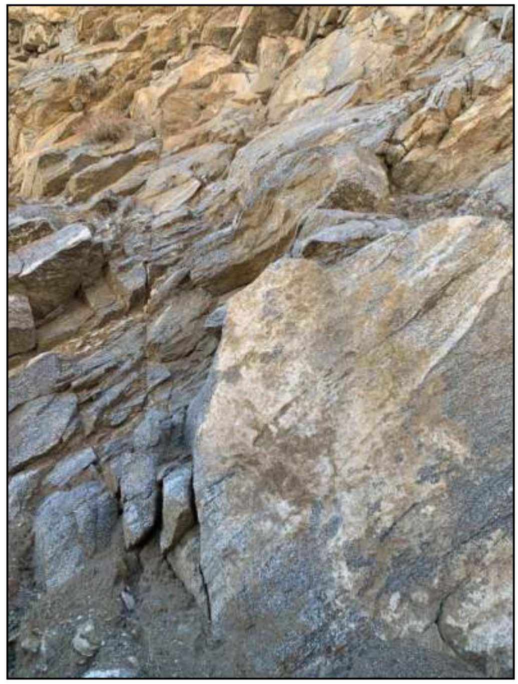

PHOTO 90: Closeup of cut in photo 89.

PHOTO 91: Moderately to slightly weathered granitic rock.

PHOTO 92: Closeup of outcrop in photo 91.

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FIGURE

B24

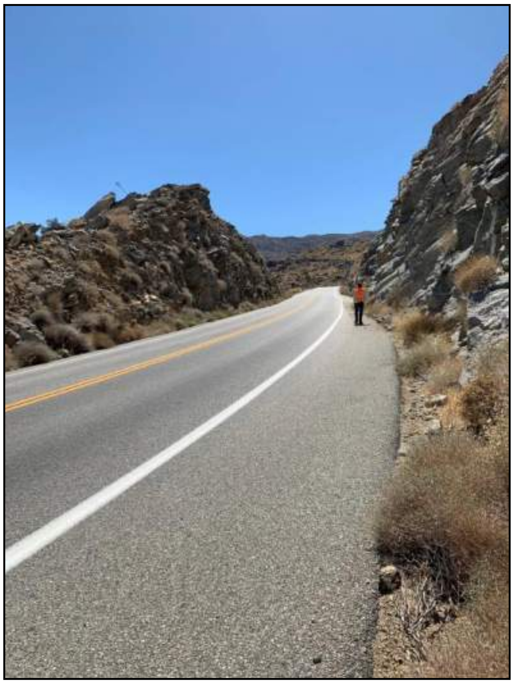

PHOTO 93: Slot cut in granitic rock.

PHOTO 94: Cut in moderately weathered granitic rock.

PHOTO 95: Outcrop of granitic rock displaying steep east dipping foliation.

PHOTO 96: Close up of rock in photo 95.

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DRAWN:	12/02/2019		**B25**
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FILE NAME:			

PHOTO 97: East dipping foliation in granitic rock.

PHOTO 98: Closeup of cut in photo 97.

PHOTO 99: Highly contorted foliation in granitic exposure.

PHOTO 100: Closeup of rock in photo 99.

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DRAWN BY:	TC		B26
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FILE NAME:			

PHOTO 101: Cut in granitic rock with east dipping Foliation.

PHOTO 102: Closeup of cut in photo 101.

PHOTO 103: Closeup of cut in photo 101.

PHOTO 104: View looking east at desert overlook turnoff. This location is above the Tubbs Canyon Portal near MP 43.3.

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	DRAWN:	12/02/2019		B27
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	FILE NAME:			

THE FOLLOWING PHOTOS 105 - 136 WERE TAKEN AIONG THE STRETCH OF S22 BETWEEN PROJECT MP 43.3 AND THE EAST SIDE OF RANCHITA NEAR MP EEN BORREGO SPRINGS AND THE OVERLOOK NEAR PROJECT MILE POST MP 47.5. S22 MEANDERS ALONG THE SECTION AND CROSSES OVER THE TUNNEL ALIGNEMENT AT SEVERAL LOCATIONS.

PHOTO 105: Highly weathered and fractured granitic rock.

PHOTO 106: Possible fault in granitic rock. The front of the truck is at the locus of a steep east dipping fault structure. Notice the highly sheared rock material on the right side of photo.

PHOTO 107: East dipping foliation in granitic rock.

PHOTO 108: Close up of photo 107.

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FIGURE

B28

PHOTO 109: Outcrop along S22- Near Alignment (33.211389, -116.444444).

PHOTO 110: Outcrop along S22- Near alignment (33.21778, -116.465556).

PHOTO 111: Outcrop granitic rock displaying bouldery surface expression.

PHOTO 112: Close up of photo 111.

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FIGURE

B29

PHOTO 113: Outcrop along S22- Western end of road.

PHOTO 114: Outcrop along S22.

PHOTO 115: Granitic outcrop displaying boulder surface expression.

PHOTO 116: Highly weathered to decomposed granitic rock.

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	DRAWN:	12/02/2019		**B30**
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	CHECK BY:	SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
	FILE NAME:			

PHOTO 117: Landscape along S22.

PHOTO 118: Highly weathered to decomposed granitic material.

PHOTO 119: Area of decomposed granitic material.

PHOTO 120: Landscape east of Ranchita.

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FILE NAME:			

PHOTO 121: Cut of highly to moderately weathered granitic rock.

PHOTO 122: Close up of cut in photo 121.

PHOTO 123: Cut in moderately to highly weathered granitic rock.

PHOTO 124: Cut in moderately to highly weathered granitic rock.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B32

PHOTO 125: Outcrop along S22.

PHOTO 126: Outcrop along S22.

PHOTO 127: Cut in moderately to highly weathered granitic rock.

PHOTO 128: Cut in moderately to highly weathered granitic rock.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B33

PHOTO 129: Cut in moderately to highly weathered granitic rock.

PHOTO 130: Cut in moderately to highly weathered granitic rock.

PHOTO 131: Cut in moderately to highly weathered granitic rock.

PHOTO 132: Cut in moderately to highly weathered granitic rock.

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	DRAWN:	12/02/2019		B34
	DRAWN BY:	TC		
	CHECK BY:	SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
	FILE NAME:			

PHOTO 133: Cut in moderately to highly weathered granitic rock.

PHOTO 134: Cut in moderately to highly weathered granitic rock.

PHOTO 135: Cut just east of Ranchita.

PHOTO 136: View looking west from east side of Ranchita.

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FIGURE

B35

THE FOLLOWING PHOTOS 137 - 144 WERE TAKEN IN THE RANCHITA AREA.

PHOTO 137: Ranchita, East End, on Via Oak Grove Lane.

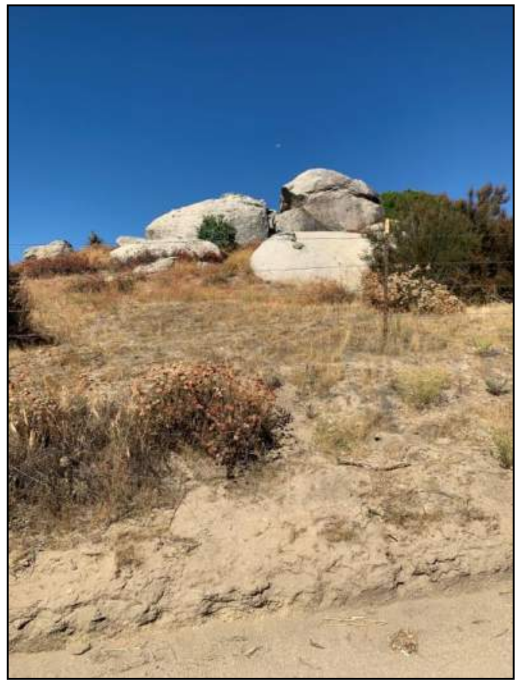

PHOTO 138: Outcrop of granitic rock on Via Oak Grove Lane.

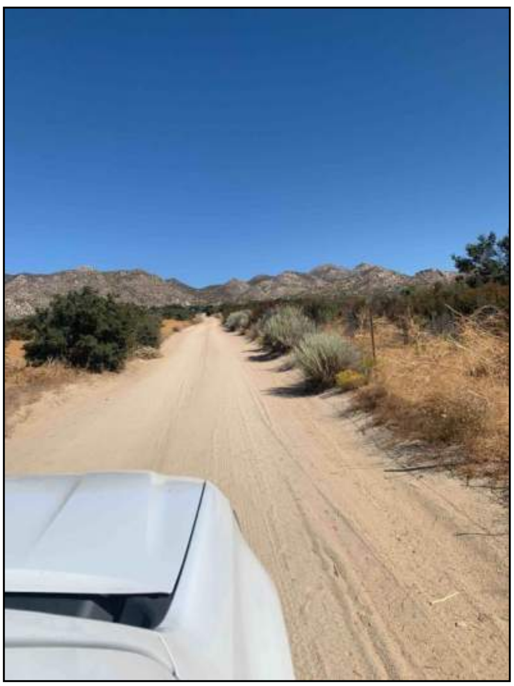

PHOTO 139: North end of Via Oak Grove Lane.

PHOTO 140: View to west of Road in Photo 139.

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FIGURE

B36

PHOTO 141: Ranchita, middle portion, on Lease Road.

PHOTO 142: Decomposed grainitic rock in cut on Lease Road.

PHOTO 143: Ranchita, Western portion, on Tye Road.

PHOTO 144: Ranchita, western side of Tye Road.

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	FILE NAME:			

THE FOLLOWING PHOTOS 145 - 176 WERE TAKEN ALONG S22 AND SR79 BETWEEN IN THE RANCHITA AREA AND THE EAST MARGIN OF LAKE HENSHAW. MUCH OF THIS STRETCH IS NORTH OF THE PROJECT TUNNEL ALIGNMENT. ALL PHOTOS ARE PRESENTED IN SEQUENTIAL ORDER FROM EAST TO WEST.

PHOTO 145: Cut in highly weathered metamorphic rock west of Ranchita on S22.

PHOTO 146: Closer view of the outcrop in photo 145.

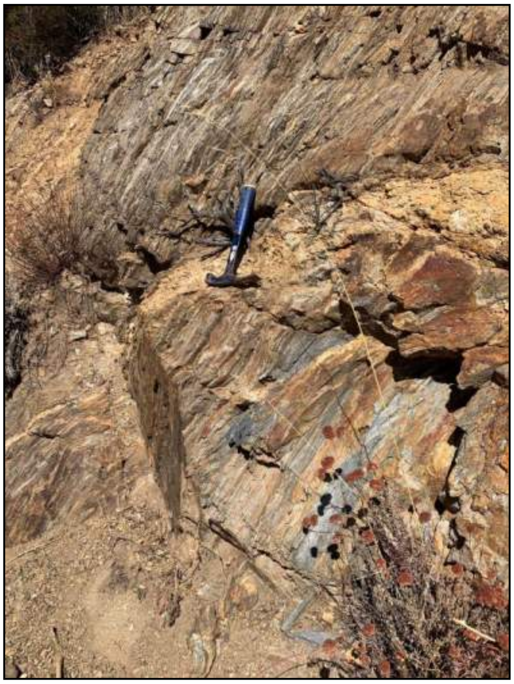

PHOTO 147: Outcrop of metasedimentary rock on SR22.

PHOTO 148: Cut into highly weathered metamorphic rock.

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PROJECT NO.	20201537	**PROJECT PHOTOS**	FIGURE
DRAWN:	12/02/2019		**B38**
DRAWN BY:	TC		
CHECK BY:	SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
FILE NAME:			

PHOTO 149: Highly weathered metamorphic rock Along S22 west Ranchita.

PHOTO 150: Cut on north side of S22 west of Ranchita. The cut is in a contact zone with both metamorphic and granitic rock.

PHOTO 151: Highly weathered to decomposed metamorphic rock.

PHOTO 152: Cut on south side of S22 west of Ranchita.

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	DRAWN: 12/02/2019		B39
	DRAWN BY: TC		
	CHECK BY: SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
	FILE NAME:		

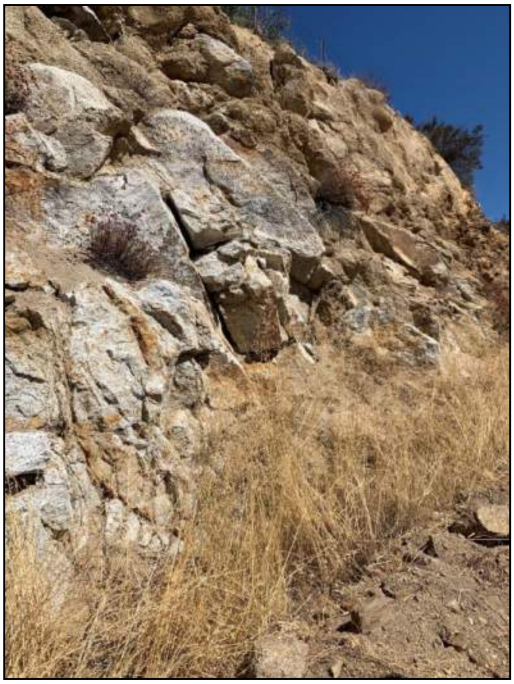

PHOTO 153: Granitic rock along S22.

PHOTO 154: View looking west along S22.

PHOTO 155: Cut containing both metamorphic and granitic rock.

PHOTO 156: Slot cut into granitic rock west of Ranchita.

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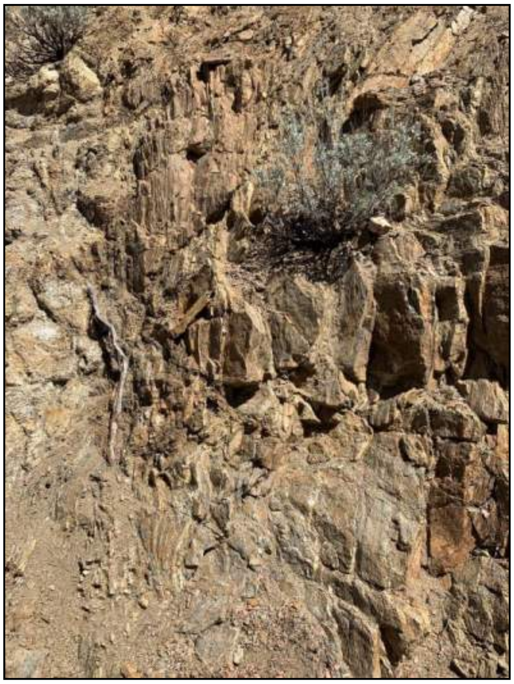

PHOTO 157: Highly weathered metasedimentary rock.

PHOTO 158: Same location as photo 157 looking West.

PHOTO 159: Cut into decomposed granitic rock.

PHOTO 160: Cut into decomposed granitic rock.

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DRAWN:	12/02/2019		B41
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CHECK BY:	SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
FILE NAME:			

PHOTO 161: Cut into decomposed granitic rock.

PHOTO 162: Cut into decomposed granitic rock.

PHOTO 163: Cut into decomposed granitic rock.

PHOTO 164: Cut into decomposed granitic rock.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B42

PHOTO 165: Cut into decomposed granitic rock.

PHOTO 166: Cut into decomposed granitic rock.

PHOTO 167: Cut into decomposed granitic rock.

PHOTO 168: Photo on San Felipe Road (S2) at tunnel crossing north of S22.

PHOTO 169: Area along Matagual Rd to Boy Scout Camp near tunnel alignment.

PHOTO 170: Adjacent area on Matagual Road of tunnel alignment.

PHOTO 171: Near alignment, in Boy Scout Camp area.

PHOTO 172: Near alignment, along Highway 79.

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PHOTO 173: Entrance to residential area near Highway 79 near alignment crossing.

PHOTO 174: Sign limiting access.

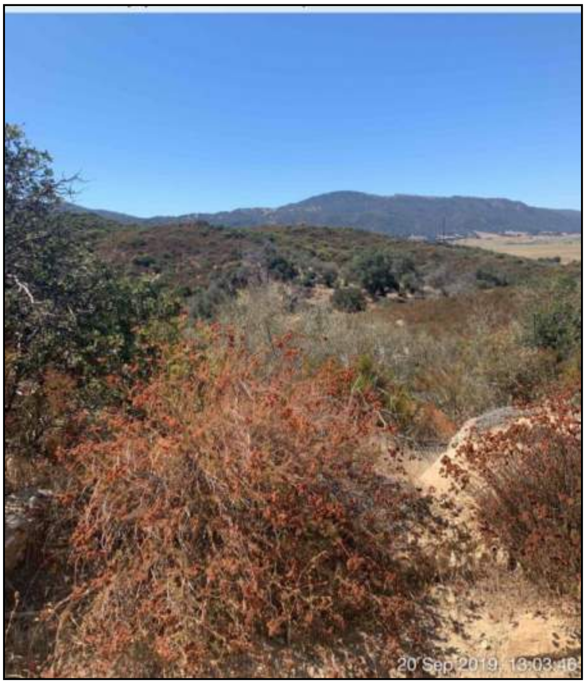

PHOTO 175: Areas with limited site access, near Highway 79, on hiking trail near MP 59.5.

PHOTO 176: Rock outcrop in area of photo 175.

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FILE NAME:		

THE FOLLOWING PHOTOS 177 - 196 WERE TAKEN ALONG S79, SR76 AND VALLEY CENTER DRIVE BETWEEN THE EAST MARGIN OF LAKE HENSHAW AND EAST VALLEY CENTER. MUCH OF THIS STRETCH IS NORTH OF THE PROJECT TUNNEL ALIGNMENT.

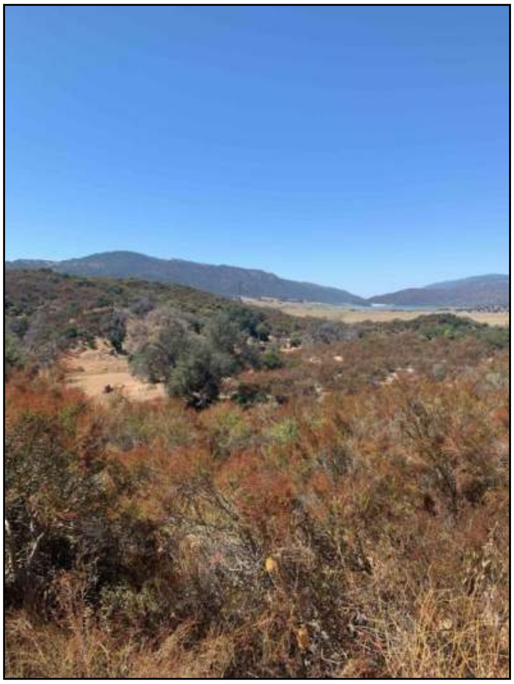

PHOTO 177: View looking west to Lake Hesshaw east of intermediate shaft.

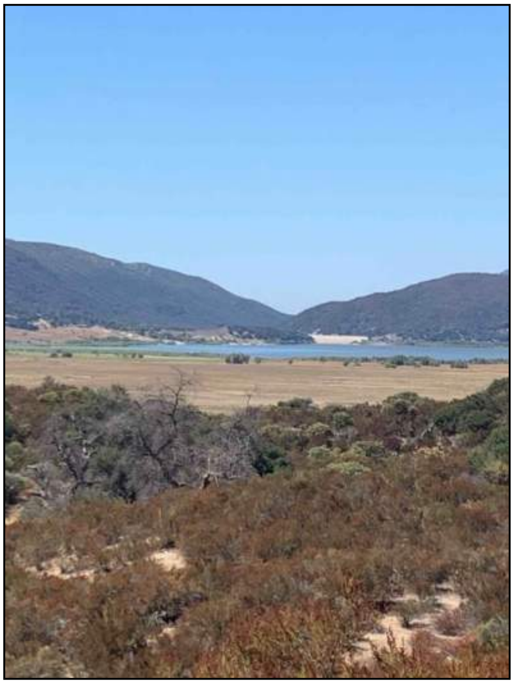

PHOTO 178: West looking view of Lake Henshaw Basin just east of intermediate shaft.

PHOTO 179: View looking west across SR79 near Intermediate shaft location.

PHOTO 180: View looking east from same location of Photo 179.

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	FILE NAME:			

PHOTO 181: SR76 along south side of Lake Henshaw with road cut of granitic material.

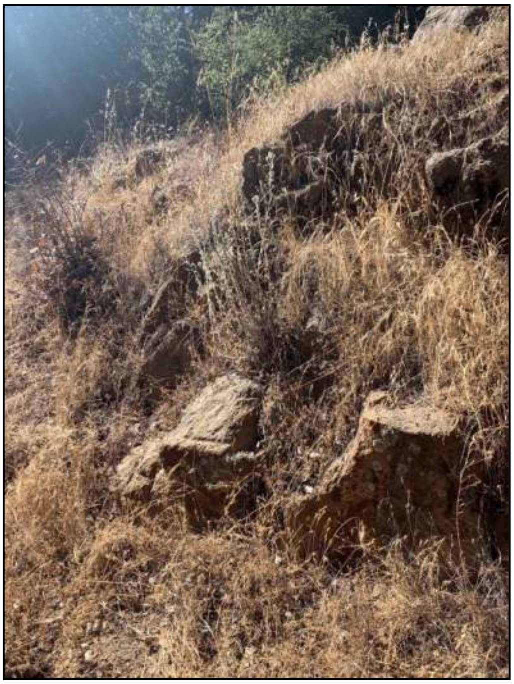

PHOTO 182: Close up of slope in photo 181.

PHOTO 183: Near Lake Henshaw and Joise's Saloon above tunnel alignment.

PHOTO 184: View looking south of same area in Photo 183.

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FIGURE

B47

PHOTO 185: View looking east from SR76 aross Lake Henshaw basin.

PHOTO 186: Near Lake Henshaw, looking west at Intersection of SR76 and Center Loop Road.

PHOTO 187: View looking east across Lake Henshaw Basin toward intermediate shaft.

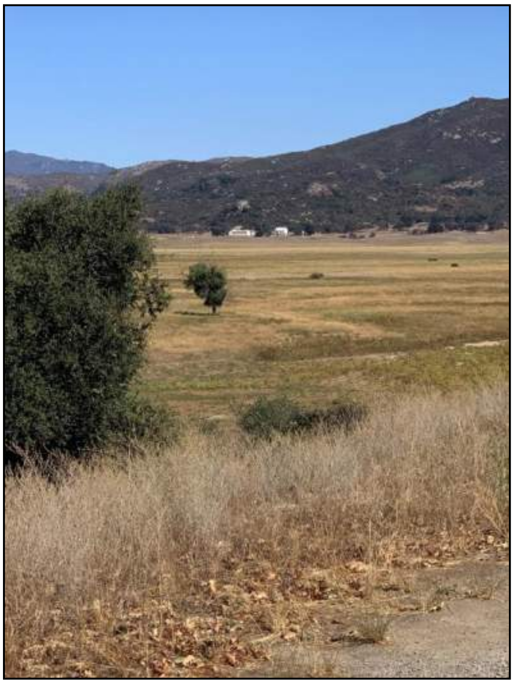

PHOTO 188: Close up of view in photo 187.

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FIGURE

B48

PHOTO 189:. View looking east from west side of Lake Henshaw.

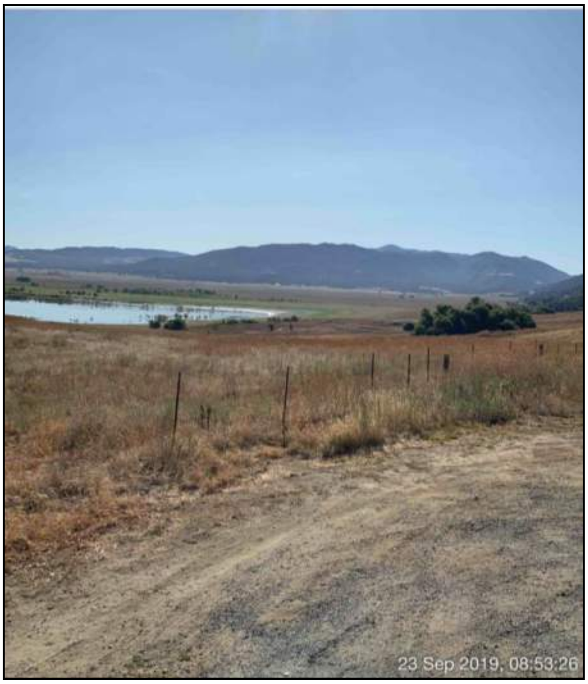

PHOTO 190: Close up of area in photo 189.

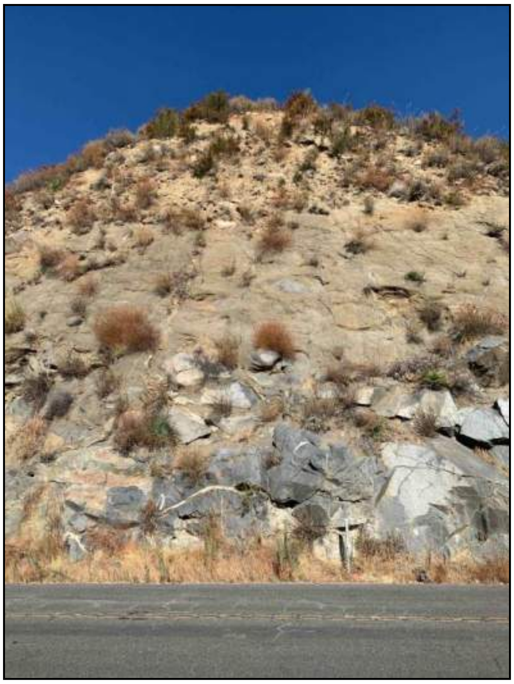

PHOTO 191: Cut of granitic material west of Lake Henshaw.

PHOTO 192: Cut of granitic rock west of Lake Henshaw.

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FIGURE

B49

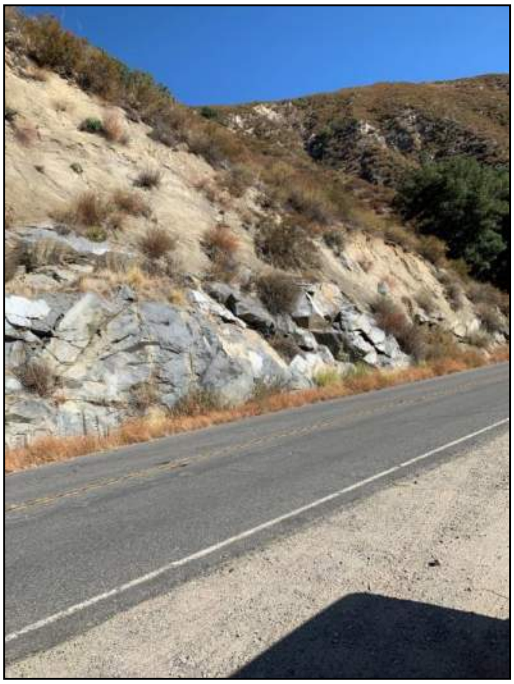

PHOTO 193: View looking east on SR76 of granitic rock.

PHOTO 194: Highly weathered granitic rock along Highway 76.

PHOTO 195: Closer view of the rock outcrop shown in photo 194.

PHOTO 196: Highly weathered granitic rock in Cut on SR76.

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FIGURE

B50

THE FOLLOWING PHOTOS 197 - 212 WERE TAKEN IN THE VALLEY CENTER AREA IN THE PROXIMITY OF VALLEY CENTER DRIVE.

PHOTO 197: View looking east in area of tunnel alignment near North Lake Wohlford Road and Valley Center Drive.

PHOTO 198: Outcrop of slightly weathered granitic rock on Nyemi Pass Place near Valley View Casino.

PHOTO 199:Outcrop of slightly weathered granitic rock on Nyemi Pass Place near Valley View Casino.

PHOTO 200: Outcrop of slightly weathered granitic rock on Nyemi Pass Place near Valley View Casino.

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FIGURE

B51

PHOTO 201: Outcrop of moderately weathered granitic rock near MP 78.8.

PHOTO 202: : Highly weathered granitic rock near MP 78.8.

PHOTO 203: Close up of outcrop in photo 202.

PHOTO 204 : Area near tunnel alignment on Calle De Vista.

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FIGURE

B52

PHOTO 205: Area near tunnel alignment on Calle De Vista.

PHOTO 206: Area near tunnel alignment on Calle De Vista.

PHOTO 207: Valley Center Road near area of tunnel Alignment.

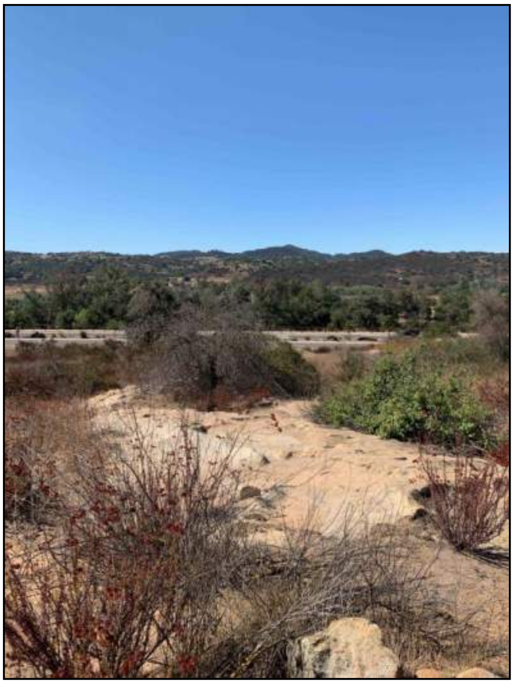

PHOTO 208: View on east in area above that in photo 207.

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PROJECT PHOTOS

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B53

PHOTO 209: View looking east of site in photo 208.

PHOTO 210: View looking northeast of site in photo 208.

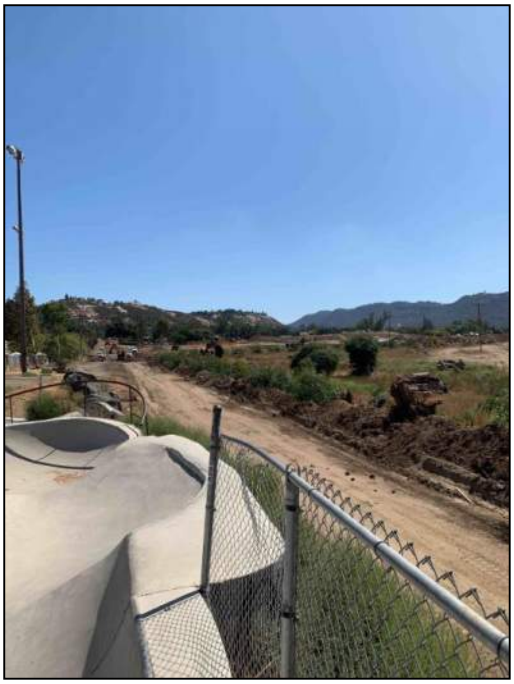

PHOTO 211: View looking east along tunnel alignment at Valley Center skate park.

PHOTO 212: View looking west along tunnel alignment at Valley Center skate park.

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PROJECT NO.	20201537	**PROJECT PHOTOS**	FIGURE
DRAWN:	12/02/2019		**B54**
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FILE NAME:			

THE FOLLOWING PHOTOS 213 - 236 WERE TAKEN BETWEEN THE FRONTAGE ROAD (OLD HIGHWAY 395) EAST OF INTERSTATE 15 AND THE END OF THE TUNNEL ALIGNMENT TO THE WATER AUTHORITY'S 2nd AQUEDUCT SYSTEM NEAR THE TWIN OAKS VALLEY WATER TREATMENT PLANT (TOVWTP) IN SAN MARCOS.

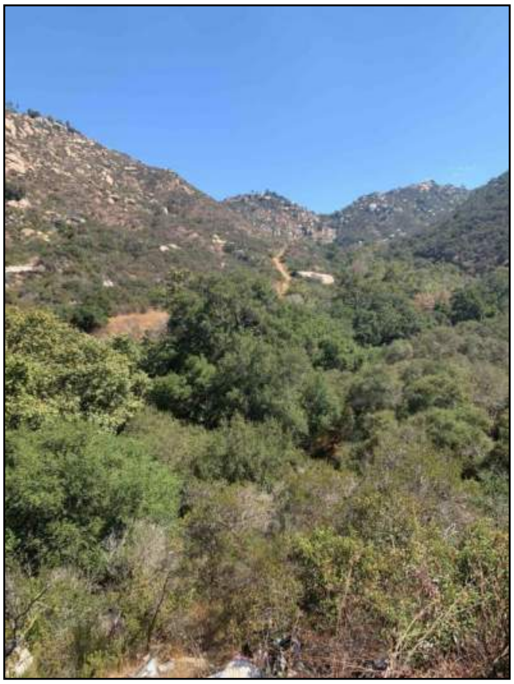

PHOTO 213: View looking east from Old Highway 395 along cut and cover section up to portal area at MP 79.3.

PHOTO 214: View looking south along Old Highway 395 at pipeline crossing.

PHOTO 215: View looking east north of pipeline Crossing along Old Highway 395.

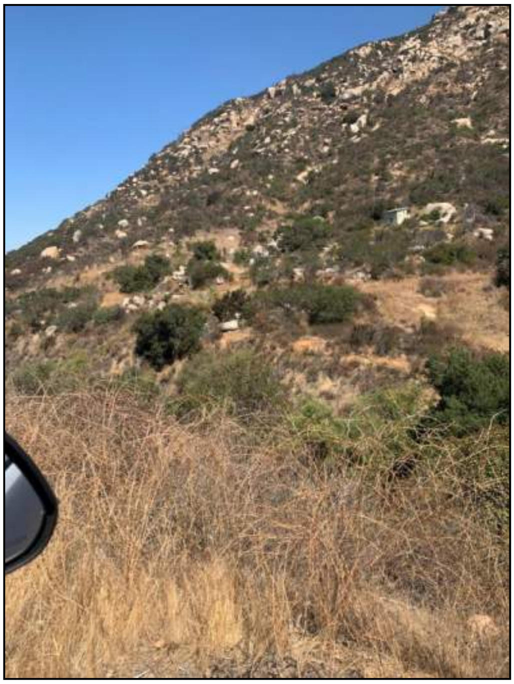

PHOTO 216: View looking east showing boulder granitic rock near pipeline crossing.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B55

PHOTO 217: View of hillside east of Old Highway 395.

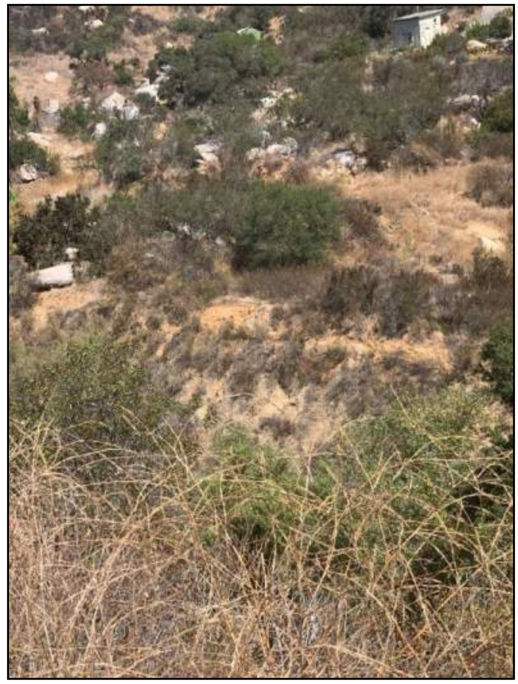

PHOTO 218: View looking down fill embankment slope on east side of Old Highway 395.

PHOTO 219: View looking west from I-15 just south of tunnel alignment.

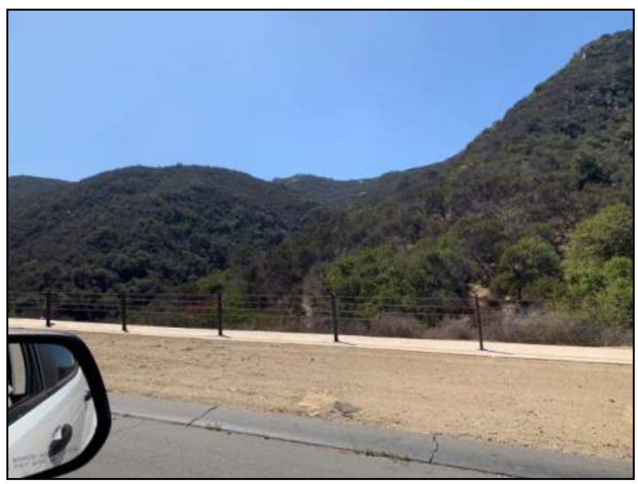

PHOTO 220: View looking west from I-15 just south of tunnel alignment.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

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PHOTO 221: View looking west from I-15 just near tunnel alignment.

PHOTO 222: View looking west from I-15 just near tunnel alignment.

PHOTO 223: Photo looking north from Twin Oak Valley Road of rock quarry north of TOVWTP.

PHOTO 224: Close up of quarry in photo 223.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B57

PHOTO 225: View looking south from N Twin Oaks Valley Road of rock quarry.

PHOTO 226: Outcrop of slightly weathered hard granitic rock at 2nd Aqueduct location at TOVWTP.

PHOTO 227: Close up of rock in photo 226.

PHOTO 228: View looking east from TOVWTP of west portal location on hillside.

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PROJECT NO.	20201537	**PROJECT PHOTOS**	FIGURE
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DRAWN BY:	TC		
CHECK BY:	SHR	SDCWA REGIONAL CONVEYANCE SYSTEM STUDY SAN DIEGO AND IMPERIAL COUNTIES, CA	
FILE NAME:			

PHOTO 229: View looking southwest from Twin Oak Valley Road of adjacent area south of TOVWTP.

PHOTO 230: Private drive east of Twin Oaks Valley Road downslope of west portal.

PHOTO 231: View looking west from I-15 just near tunnel alignment.

PHOTO 232: View looking south from Twin Oaks Crest Drive near west portal location.

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PROJECT PHOTOS

SDCWA REGIONAL CONVEYANCE SYSTEM STUDY
SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B59

PHOTO 233: View looking south from Twin Oaks Crest Drive near west portal location.

PHOTO 234: View looking south from Twin Oaks Crest Drive near west portal location.

PHOTO 235: View looking east from bottom of Twin Oaks Crest Drive near west portal location.

PHOTO 236: Cut of highly weathered granite along east side of Twin Oaks Valley Road near TOVWTP.

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SAN DIEGO AND IMPERIAL COUNTIES, CA

FIGURE

B60

D

INNOVATIVE BRINE MANAGEMENT TECHNOLOGIES

Innovative Brine Management Technologies

Introduction

This section provides information collected from a desktop literature review of innovative brine management technologies. The technologies described herein are categorized as: enhanced evaporation, brine volume minimization, and beneficial reuse. A brief description of each category is provided below. For each technology the literature review provides process description, full scale applications and general cost information.

- Enhanced Evaporation - These technologies are designed to increase the evaporation rate of the brine thereby reducing the foot-print required for traditional evaporation ponds.
- Brine Volume Minimization – These technologies reduce the volume of brine by either 1) modifying the traditional RO process to increase recovery or 2) by further concentrating the brine produced from the traditional RO process.
- Beneficial Reuse – These technologies are designed to produce a beneficial reusable product. To date a variety of uses for concentrate have been identified including: the production of marketable mineral salts, industrial uses, brine shrimp production, electricity production, irrigation of halophytes and aquaculture rehabilitation (Howe 2004). Though beneficial reuse of concentrate is not currently practiced in the main stream several innovative technologies due exist.

Enhanced Evaporation

1.0 Wind Aided Intensified Evaporation (WAIV) Process Description

PROCESS DESCRIPTION

FIGURE A-1
WAIV System from The American Membrane Technology Association's Solutions Issue, Winter 2016-17

This process increases the surface area of an evaporation pond while minimizing its perimeter. By packing vertically mounted, wet surfaces and subjecting them to natural, dry winds, reports done on pilot studies of WAIV systems claim that they can reduce an evaporation pond size by up to 90% (AMTA, 2016). There have been several case studies carried out on this technology, one of which is located in Alamogordo, NM. This smaller scale facility is a pilot for a larger 4 mgd project that is not complete at this time. The pilot handles 2.5 mgd of brine with an evaporation pond paired with electrodialysis metathesis (EDM) and WAIV (Cappelle, 2016) and the system was able to achieve 99% reduction in evaporation pond size. However, it should be noted that WAIV itself was only able to reduce the pond size by 50%. Another study carried out in El Paso, TX proved that evaporation rates with WAIV reached 50% to 90% improvement in comparison to an open pond of twice the size while using minimal energy (Cappelle, 2016).

FULL APPLICATIONS / COSTS

While there are obvious benefits from incorporating WAIV into a treatment process, the technology is still new and has only been carried out at the pilot level. The study in El Paso, which implemented an advanced water purification facility (AWPF) with concentrate enhanced recovery reverse osmosis (CERRO) and WAIV, amortized the capital costs with a 20-year equipment life and 4% interest rate, resulting in a normalized cost of \$4/1,000 gallons. The study in Alamogordo, which implemented zero discharge desalination (ZDD), CERRO and WAIV, estimated the total costs to be \$2.39/1,000 gallons (Vasu, 2016). In addition, highly concentrated brine has been known to clog the WAIV membranes, decreasing their efficiency. Since the RCS project is still decades in the making, WAIV would be considered a viable option, as it is predicted that they will become more mainstream in the coming years.

2.0 SolarBee

PROCESS DESCRIPTION

Solar Bee is a solar powered reservoir circulator which seeks to increase dissolved oxygen (DO) levels, typically applied to waste water or salt ponds.

FULL SCALE APPLICATION/COSTS

This technology can help to control algae blooms in natural ponds, prevent salt crystals from forming in brine ponds, and minimize the hydrophobic resin that appears on top of wastewater ponds. This in turn can assist in improving evaporation rates by 20% in saline solutions, 100% in wastewater solutions, provide energy savings and completely mix water tanks. The pricing for a single solar evaporation unit, including delivery and installation, runs around $45,000. Units are typically spaced every 3 to 5 acres.

FIGURE A-2
SolarBee System from the Medora Corporation website

3.0 Evaporation Cannon

PROCESS DESCRIPTION

This technology seeks to enhance evaporation by creating a mist and continually rotating through either 90 or 180 degrees into the air. Using specialized nozzles and high-water pressure, water droplet size is greatly and quickly reduced before traveling through a fan at a high discharge velocity. While they can be implemented on their own, evaporation canons have the ability to be controlled by external operation systems that sync them with local weather conditions. Factors that impact evaporation rates include air temperature, relative humidity, wind speed, water droplet size and spray coverage area (Wet Earth, 2015).

FULL SCALE APPLICATION/COSTS

FIGURE A-3
Evaporation Cannon System from the Wet Earth website

Upon speaking with Wet Earth, an evaporation pond vendor, it was predicted that approximately 50% of the water fired from the canon during the summer would evaporate, while the evaporation rate would drop down to around 20% during the winter. These rates would increase to around 65% in the summer and 30% in the winter if the cannons were run only during daylight hours, which would cost less per gallon evaporated. Costing and operations information from Wet Earth stated that each unit costs $75,600, total power consumption is 82 kW and water consumption is 20 L/s.

This technology has the potential to greatly reduce the size of evaporation ponds, therefore, once more detailed information is gathered from the vendor, a thorough cost-benefit analysis will be carried out and included in the final report.

Brine Volume Minimization

4.0 High Recovery Reverse Osmosis

PROCESS DESCRIPTION

When RO is conducted on a larger industrial level, these systems are highly sensitive to imbalanced flows and fluxes, as well as reoccurring fouling, large operating space and wasted energy. Chemical softening can be used to combat scaling, but price, the complexity of varying feed water chemistry and additional space for chemical storage and equipment can take RO systems from high efficiency and low cost to the opposite if not implemented or monitored correctly. Recent advancements in RO technology have introduced the idea of a High Recovery Reverse Osmosis System (HRRO) in order to combat these obstacles.
The HRRO system contains a cluster of identical membranes and a recirculation pump which optimizes operating conditions, allowing for flexibility in the system. To conserve both energy and space, the HRRO unit recirculates the concentrate back through the system to continuously increase the recovery rate. HRRO combines aspects of traditional RO with simple filtration and semi-batch processes through constant monitoring by a software system that periodically flushes out the concentrate at the ideal time, resulting in less maintenance and down time. A limitation associated with this technology is related to the vast range of components that can be present in water samples. It is imperative that feed content be known prior to operation of the HRRO system in order for its software to

calculate the optimal process conditions, otherwise performance could be drastically impaired. Due to its adaptability, HRRO can be applied to everything traditional RO is used for, including the treatment of brackish water. Other factors such as saturation relief, flow reversal and semi-batch configuration can be manipulated to improve efficiency but are still impacted by the same limitations. This technology is on the brink of municipal application and will therefore be researched further as a potentially feasible application.

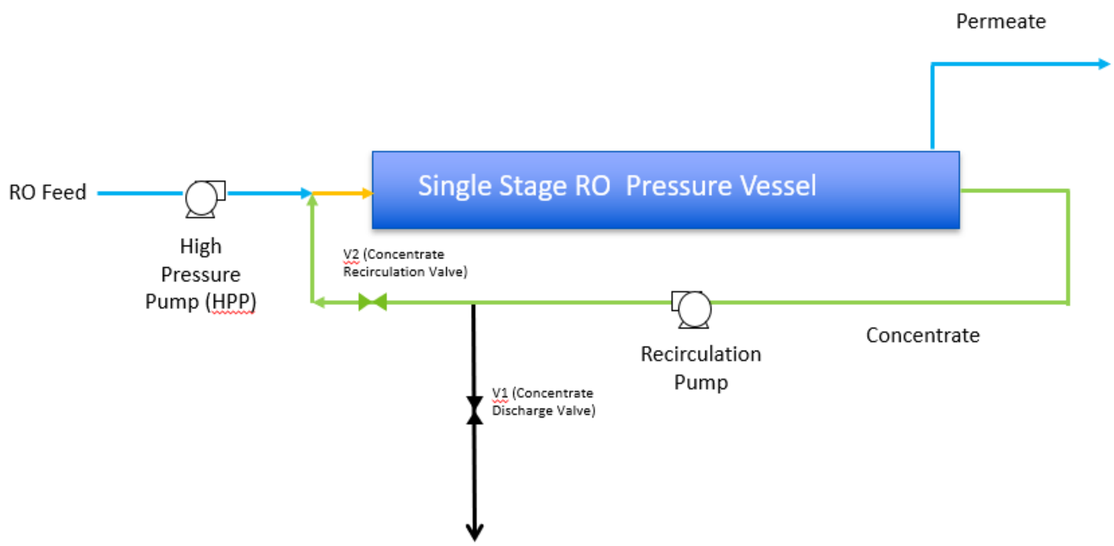

FIGURE A-4
Closed Circuit Reverse Osmosis Process Schematic from Desalitech

FULL SCALE APPLICATIONS / COSTS

As of 2017, Desalitech, a manufacturer of HRRO systems they call Closed Circuit Reverse Osmosis (CCRO), stated that 58 units have been installed with a capacity range of 15 to 1200 gpm and includes 2 municipal potable water applications. The capital cost of a CCRO system for the RCS project was estimated by B&V to be approx. $30M.

5.0 High Efficiency Reverse Osmosis (HERO)

PROCESS DESCRIPTION

General Electric's Water and Process Technologies group are pioneering a new reverse osmosis (RO) process that they market as being "High Efficiency". This system, shown in Figure A-5, uses 3 pre-treatment steps which consist of ion exchange, CO_2 reduction, and finally the pH is raised over 10 to improve the solubility of silica (GE Water and Process Technology, 2008). This procedure achieved a recovery rate over 90% with brackish ground water with a flux of 30 gfd (Sethi, 2006).

FULL SCALE APPLICATIONS / COSTS

While HERO has been applied to numerous semiconductor systems (Eagle, 2008), there has been no full-scale application for the treatment of brackish water. Black & Veatch previously carried out a paper study in 2004 for the Southern Nevada Authority comparing the economy of the HERO application with traditional RO and found that a HERO system achieving 98% recovery would cost between $873 per AF less and $112 per AF more than traditional RO. In addition to the little information known about applying HERO to a brackish water treatment facility outside of the desktop study, its high chemical consumption rates and high upfront capital costs make it an unwise investment for the Colorado River project. Other applications for HERO include the semiconductor industry to treat water for the purposes of reducing brine volume and cooling blowdown water.

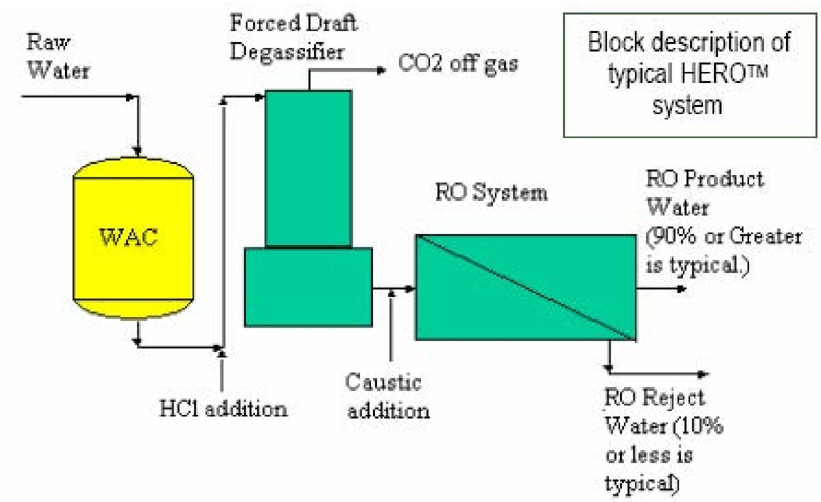

FIGURE A-5
High Efficiency Reverse Osmosis

6.0 Flow Reversal Reverse Osmosis

<u>PROCESS DESCRIPTION</u>

AdEdge brought a new desalination technology to market called Flow Reversal Reverse Osmosis (FR-RO), designed specifically to treat brackish water and prevent scaling, which causes membrane plugging and decreases recovery rates. As illustrated in Figure A-6, the location of the feed is switched periodically with the exiting site of the concentrate inside the pressure vessel (AdEdge). Flow reversal avoids the previous issues of supersaturated solutions precipitating onto the membrane because the solution does not have time to reach its saturation point before the flow is reversed. AdEdge states the following benefits of their technology: flexibility of control, 100% fallback redundancy, capital savings through a single integrated system, operations and maintenance savings driven by chemistry, CIP and membrane life, and others. Due to the nature of river water, if this brine management method was selected, a real time water quality sensor would need to be installed to monitor the saturation level of the water so that optimum flow reversal efficiencies could be reached. However, this technology is still in its infancy and will not be considered for this reason.

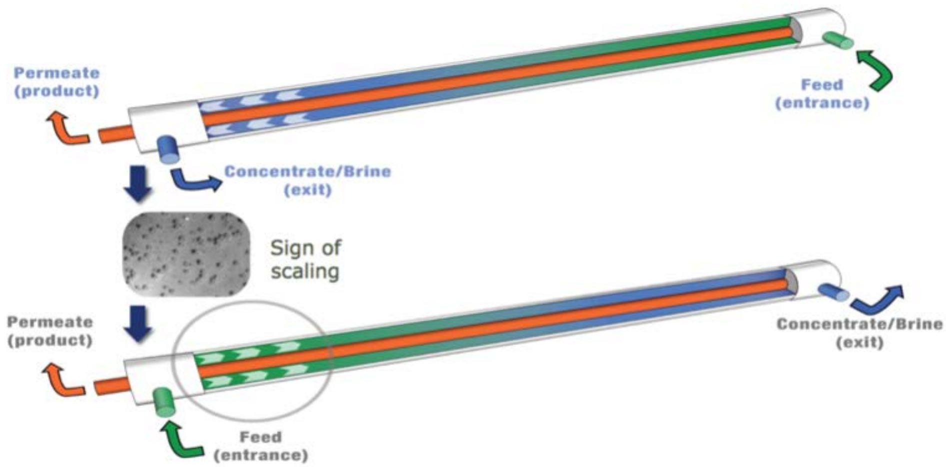

FIGURE A-6
Flow Reversal RO System from the AdEdge website

<u>FULL SCALE APPLICATIONS / COSTS</u>

There is one full scale municipal application that processes approximately 1 mgd starting in Arizona on 2020. Adedge states that the cost of FR-RO is about 10-20% higher than traditional brackish RO.

7.0 Vibratory Shear Enhanced Process

<u>PROCESS DESCRIPTION</u>

The Vibratory Shear Enhanced Process (VSEP®) is a vibrating membrane system manufactured by New Logic Research in Emeryville, CA. The basic components of the VSEP system (See Figure A-7) include: a drive system, membrane module, torsion spring and vibration control system. Similar to conventional RO systems, VSEP can be operated in two stage configurations to increase recovery but operates with many different membrane sizes including microfiltration, ultrafiltration, nanofiltration and RO (New Logic Research). Additionally, New Logic Research claims the following 8 reasons why VSEP outperforms conventional systems: high filtration rates, fouling resistance, high solids, high efficiency, dependability, compact design, convenience and low cost.

FIGURE A-7
VSEP System from New Logic

FULL SCALE APPLICATIONS / COST

Initial applications of the VSEP process have been limited to chemical processing and manufacturing markets and although a recent study evaluated the technology's ability to reduce RO concentrate volume. However, the volume of water used in this pilot study was only 1.2 mgd and no costing information was provided. Due to the lack of data on similar systems, this alternative will not be considered further.

8.0 Membrane Distillation

PROCESS DESCRIPTION

Membrane distillation, as illustrated in Figure A-8, is an evaporation process driven by the difference between partial pressures on one side of a porous hydrophobic membrane and the other (Song et al. 2007). Evaporation occurs if the vapor pressure at the solution side is greater than the vapor pressure at the condensate side. Membrane distillation units are available in several configurations, but direct contact membrane distillation is the most suitable for desalination. In this process, hot brine is passed on one side of a permeable hydrophobic membrane as a colder aqueous distillate stream flows on the other side. Distribution of water vapor evaporated from the hot brine at the brine-membrane interface takes place through the gas-filled hydrophobic membrane pores and the water vapor is condensed in the cold distillate on the other side of the membrane.

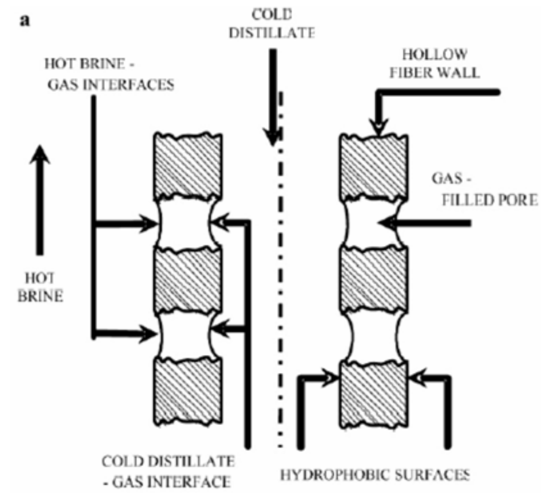

FIGURE A-8
Membrane Distillation System

There are several advantages of this process, such as the production of high-quality water, which does not require extensive pretreatment and can be distilled at relatively low temperatures, low pressures and low-grade heat. This process has been used in industrial application, but there is no full-scale application for brackish groundwater concentrate management. Therefore, this management option will no longer be pursued.

FULL SCALE APPLICATIONS /COSTS

This process has been used in industrial application, but there is no full-scale application for brackish groundwater concentrate management. A preliminary study by Sirkar in 2003 suggested that the cost of implementing membrane distillation would be about half the cost of the RO process. Due to the lack of application and information on membrane distillation, it will no longer be considered as an option.

9.0 Dewvaporation

PROCESS DESCRIPTION

In this process, illustrated in Figure A-9, brackish water is evaporated by heated air which deposits fresh water as dew on the opposite site of a heat transfer wall. The dewvaporation contacting tower focuses on a relatively non-traditional and innovative heat-driven process using air as a carrier gas and operating at atmospheric pressure throughout the device (Hamieh et al. 2001). The energy needed for this process is supplied by the energy released from dew formation and the heat source can be formed from low temperature solar heat, waste heat, or combustible fuels. This process does not have a scaling problem since the evaporation occurs at the air-liquid interface and not at the heat transfer wall. To date there are no reported full-scale applications of a dewvaporation process in action and due to the associated risks with implementing a brand-new technology, this method will not be pursued.

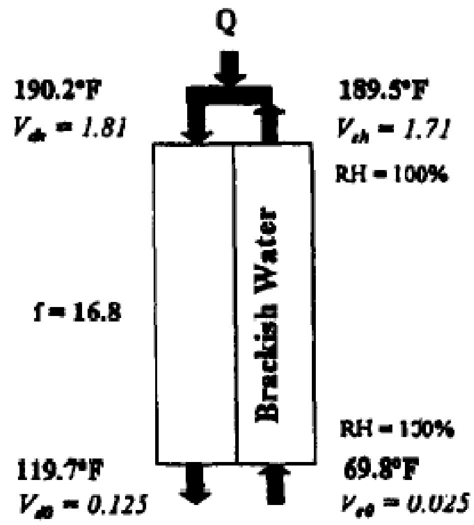

FIGURE A-9
Dewvaporation System

FULL SCALE APPLICATIONS / COSTS

While no full-scale applications exist yet, the reported operating cost of this process is $3.5/1000 gallons while using natural gas as a heat source, while costing $12/1000 gallons when using vapor compression evaporators as a heat source (Jordahl, J., 2006).

10.0 Forward Osmosis

PROCESS DESCRIPTION

An innovative method to further concentrate brine is forward osmosis (FO), a process essentially the opposite from RO. As illustrated in Figure A-10, when solutions of different concentrations are separated by a semi-permeable membrane, the solvent would move across the membrane from the lower solute concentration side to the higher concentration solute side. The driving force for this movement is the osmotic pressure gradient across the membrane caused by the differences in solute concentrations. The key advantage of FO is a reduced energy use due to the lack of external pressure required to drive the water across the membrane. The main challenges exist in the manufacturing of high-performance FO membranes and the selection of easily separable draw solutions with a high osmotic pressure (Cath et al., 2006). In addition, the water flux in FO process is often much lower than the flux expected from the bulk osmotic pressure difference and membrane permeability. There are still no full-scale facilities that have reported using FO for a brackish groundwater concentrate volume minimization. For this reason, FO will not be considered as a realistic application.

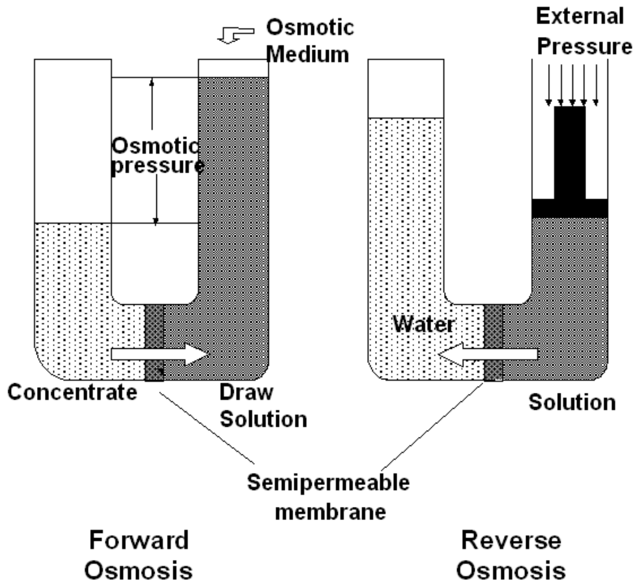

FIGURE A-10
Comparison of Forward Osmosis to Reverse Osmosis

FULL SCALE APPLICATIONS/COSTS

Forward osmosis has an assortment of different applications, such as volume minimization of sanitary landfill leachate (York et al., 1999; Osmotek Inc., 2003), concentration of fruit juices (Petrotos et al., 1998), desalting (McGinnis, 2002; Cath et al., 2005; McCutcheon, et al., 2005; McCutcheon et al., 2006) and emergency water supply equipment for homeland security operations (Cohen, 2004)). To date there are no full-scale facilities reported that use FO for brackish groundwater concentrate volume minimization. Preliminary cost estimates conducted by the project team (Adham et al., 2007) showed the inclusion of Forward Osmosis/Struvite Recovery in a treatment train of Membrane Bioreactor/Reverse Osmosis/Zero Liquid Discharge resulted in significant reduction ($2.49/1000 gal. vs. $3.07/1000 gal) in O&M cost based on a 10 MGD treatment capacity.

11.0 Electrodialysis Reversal (EDR) Process

PROCESS DESCRIPTION

Electrodialysis (ED) became commercially available in 1953 for the desalination of high salinity water. The process works by generating a direct current (DC) field across a stack of flat sheet ion exchange membranes arranged alternately in a cation / anion configuration. An electric field is applied by a pair of electrodes (anode / cathode) placed on the outside of the membrane stack. Cations are attracted to the cathode and pass through the cation transfer membranes only while anions are attracted towards the anode and pass through anion transfer membranes only. Because the cathode and anode transfer membranes are alternated ions become trapped resulting in a concentrate and demineralized product stream. Systems are configured in stages to increase removal. In order to remove scaling which occurs on the membrane surface the DC voltage is reversed several times per hour. The ion exchange membranes are also periodically cleaned by either CIP or stack disassemble.

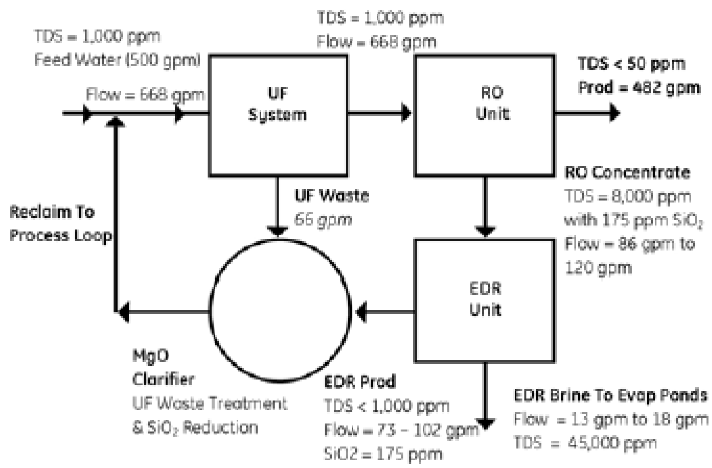

FIGURE A-11
A Schematic of The Full Scale EDR System Used for RO Concentrate Volume Minimization in Buckeye, AZ (REAHL, 2006).

FULL SCALE APPLICATIONS/COST

EDR performance is not typically impacted by certain scaling causing compounds (e.g. silica), this process can be utilized for higher water recovery from highly saline water. Therefore, the application of EDR for recovering water from RO brine is also technically feasible. For example, in Buckeye, AZ, EDR was applied to treat 8,000 ppm concentrate from RO process to achieve the total recovery of the RO-EDR process about 97% (Reahl, 2006). A schematic of the process train is included in Figure A-11. There are other reported applications for achieving 95-98% combined recovery from the RO-EDR process (Reahl, 2006).

There are few full-scale applications of EDR processes for concentrate treatment from the brackish water RO process. The estimated capital cost for the concentrate containing 7000 mg/L of TDS can be $3.25/1000 gal of treated water (Reahl, 2007). Since this process is an energy intensive process, the energy consumption of the process for the feed containing 6000-7000 mg/L of TDS can be about 15 kWh/1000 gal of treated water (Reahl, 2007). Note it is recommended a cost comparison of EDR to HRRO for the purpose of reducing brine volume be considered moving forward.

Beneficial Reuse

12.0 Electrodialysis Metathesis

PROCESS DESCRIPTION

Electrodialysis Metathesis (EDM) is a type of zero discharge desalination (ZDD) process that amplifies water recapture and salt retrieval to their highest efficiencies. This process separates the charged stream components, including minerals such as gypsum and $MgCl_2$ and collects them in partitions that make up the EDM stack, which is illustrated in Figure A-12.

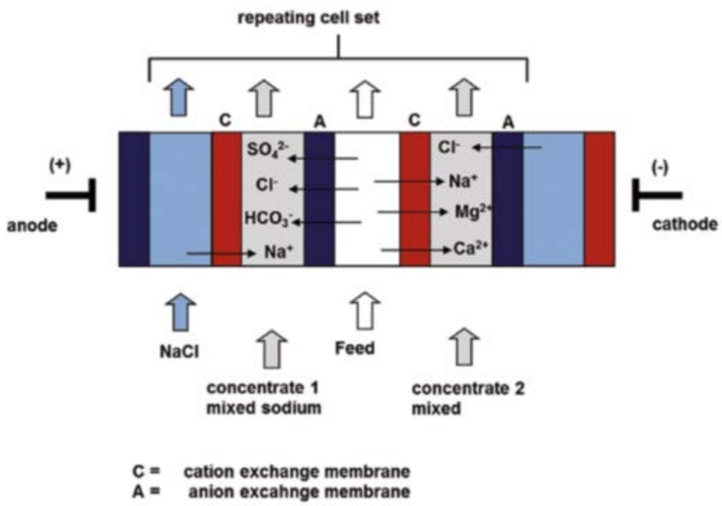

FIGURE A-12
EDM STACK FROM THE AMERICAN MEMBRANE TECHNOLOGY ASSOCIATION'S SOLUTIONS ISSUE, WINTER 2016-17

One of the compartments would hold a concentrated sodium solution, the second contains the concentrated chloride solution, the third comprises of the feed stream and the last one is filled with NaCl, which provides the necessary ions for a proper reaction. This arrangement causes ions in the feed to separate into highly concentrate streams, one containing high concentrations of chloride with the cations and one containing high concentrations of sodium with the anions (Veerapeneni, 2016), although imperfections in the membrane and impurities in the water would prevent this process from being 100% efficient. With this being said, a small-scale demonstration study done at the Beverly Hills Water Treatment Facility (BHWTF) with a RO concentrate TDS of 3190 mg/L achieved an overall recovery of 98%. The flow rate of the EDM system was not listed.

Recovery rates drop with the precipitation of various compounds such as $CaSO_4$, which clog the membranes and damage the EDM stack, requiring the addition of expensive pre-treatment chemicals (AIChE, 2012). Furthermore, an elaborate controls system is required to keep the system at the ideal conditions and a lot of salt is required in order to keep the system in operation. In addition, at the time of this review, there are no commercial system providers for EDM. For these reasons, this process will no longer be considered.

FULL SCALE APPLICATIONS/COST

In Black & Veatch's 2015 *Demonstration of a New Electrodialysis Technology to Reduce Energy Required for Salinity Management* report, three sites were studied: the Beverly Hills Water Treatment Facility, the Arcadia Water Treatment Plant in Santa Monica, and the Santa Rosa Water Reclamation Facility (SRWRF) operated by the Rancho County Water District (RCWD). The treatment costs for these facilities ranged from $2.30 to $4.50 / 1,000 gallons of RO concentrate. EDM is a new technology that has yet to be fully implemented at a treatment facility. A market readiness evaluation claimed that the technology lacked commercial backing for further research to bring it to market.

13.0 Bipolar Membrane Electrodialysis

PROCESS DESCRIPTION

Bipolar membrane electrodialysis (BMED) utilizes the principles of membrane electrodialysis, applying enhanced ionic mobilities under an applied potential and limiting the movements of ions using selective membranes, in conjunction with a bipolar membrane, which is used to split water into H^+ and OH^-. The advantage of the BMED process is that the water dissociation is accelerated up to 50 million times compared to the rate of water dissociation in aqueous solutions (Wilhelm, 2001). Therefore, in the BMED process, produced H^+ and OH^- ions can be used to generate acid and base from salts without production of hydrogen and oxygen gases.

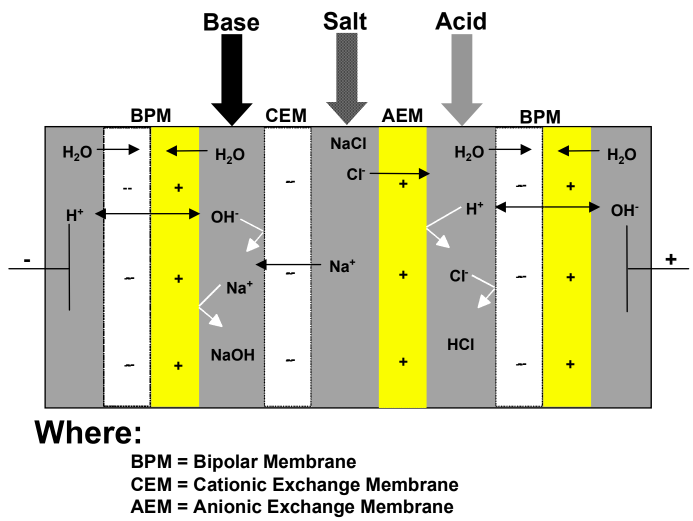

FIGURE A-13
Bipolar Membrane Electrodialysis

As illustrated in Figure A-13, To dissociate a salt in this approach, a DC voltage is applied to the electrodes. The electrical potential developed becomes the driving force to move ions, with the membranes forming barriers to ions of opposite charge. Anions attempting to migrate to the anode would pass through the adjacent anion membrane but would be rejected by the first cation membrane. Similarly, cations trying to migrate to the cathode would pass through the cation membrane but would be rejected by the anion membrane. Hence, the membranes develop a salt compartment and ion concentrating compartment. When solutions are pumped through these respective compartments, transfer of ions through the membranes de-mineralizes one and the other becomes concentrated with those ions (Wilhelm, 2001).

FULL SCALE APPLICATIONS/COST

Although there are potentially more uses for BMED, studies have only studied it for the purpose of producing acid and bases generated from RO concentrate via a wastewater reclamation facility. Since this process is very energy intensive, it is not recommended to treat the brine from the Colorado River water. Scaling up an initial cost estimation based on a bench scale performance where annualized capital cost was $0.1/1000 gal water and O&M cost was $0.4/1000 gal, the estimated capital cost for a 19.9 mgd system would be $44.9M and O&M costs would exceed $179M per year.

14.0 Electrochlorination

PROCESS DESCRIPTION

In order to treat the excessive quantity of salt in brine, some water treatment plants opt to use electrolytic cells to split NaCl to produce hypochlorite as shown in the reaction below. In this process, an electrolytic cell electrolyzes the diluted brine into sodium hypochlorite. The process is illustrated in Figure A-14.

$$NaCl + H_2O + 2e^- = NaOCl + H_2$$

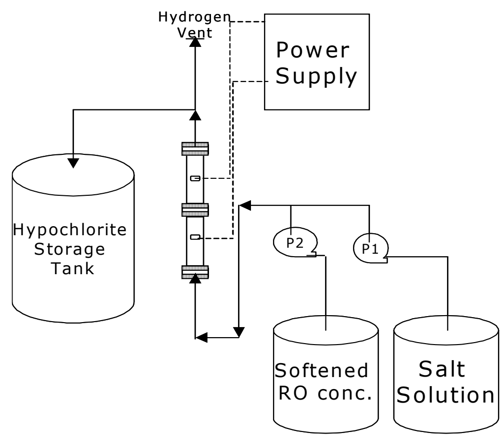

FIGURE A-14
Electrochlorination System Schematic

FULL SCALE APPLICATIONS/COST

A major advantage of this method is the entire portion of the concentrate can be converted to beneficial chlorine product, so no other disposal methods need to be used for concentrate treatment. A bench scale evaluation discussed by S. Adham in the paper Evaluation of Reverse Osmosis concentrate Minimization and Beneficial Reuse discusses the success of this process when salt concentrations are high, but reveals that the operational costs are extremely high; annualized capital cost could be $0.90/1000 gal water (total capital cost for 19.9 mgd concentrate was $399 million and O&M cost could be $5.46/1000 gal (O&M costs for 19.9 mgd concentrate was estimated over $2 billion over the course its lifetime). Since the brine concentration of interest is about 1%, and the associated costs are too expensive, electrochloronation will not be considered.

15.0 Struvite Recovery

PROCESS DESCRIPTION

The objective of struvite (PO_4^{3+}) recovery is to separate phosphate from the RO concentrate by using an ion exchange process to precipitate out a usable compound, struvite. In this process, phosphate is adsorbed out of the feed stream using PLE resin beds. Once the resin is saturated with phosphate, brine is applied to regenerate the bed and struvite is precipitated out with the addition of magnesium. This process can decrease the amount of scaling in pipes and pumps, reducing shutdown time and creating a valuable product from the brine stream.

FULL SCALE APPLICATIONS/COST

There is no full-scale application of struvite recovery process till to date for RO concentrate management. Bench scale testing was successful in utilizing RO concentrate to recover struvite, although a cost-benefit analysis exhibited that the payback of the struvite recovery did not deem the technology to be economically favorable in comparison to an evaporation pond. It was stated by the authors that this process be implemented in conjunction with other brine management processes to make it financially viable (Kumar, 2013). For this reason, struvite recovery will not be researched any further.

Appendix D References

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E

TYPICAL ELECTRICAL TRANSMISSION LINE POLE/STRUCTURE DETAILS

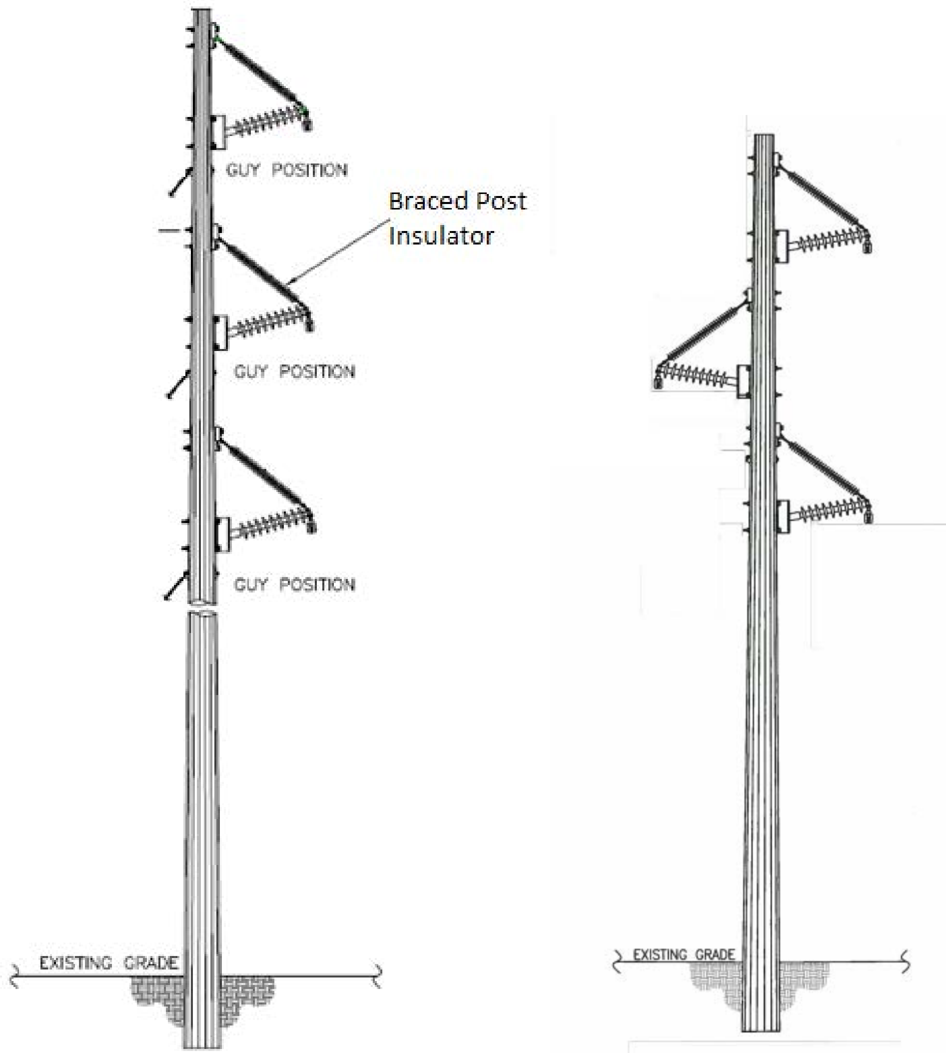

Fig: Typical wood pole tangent structure (161KV/92KV).

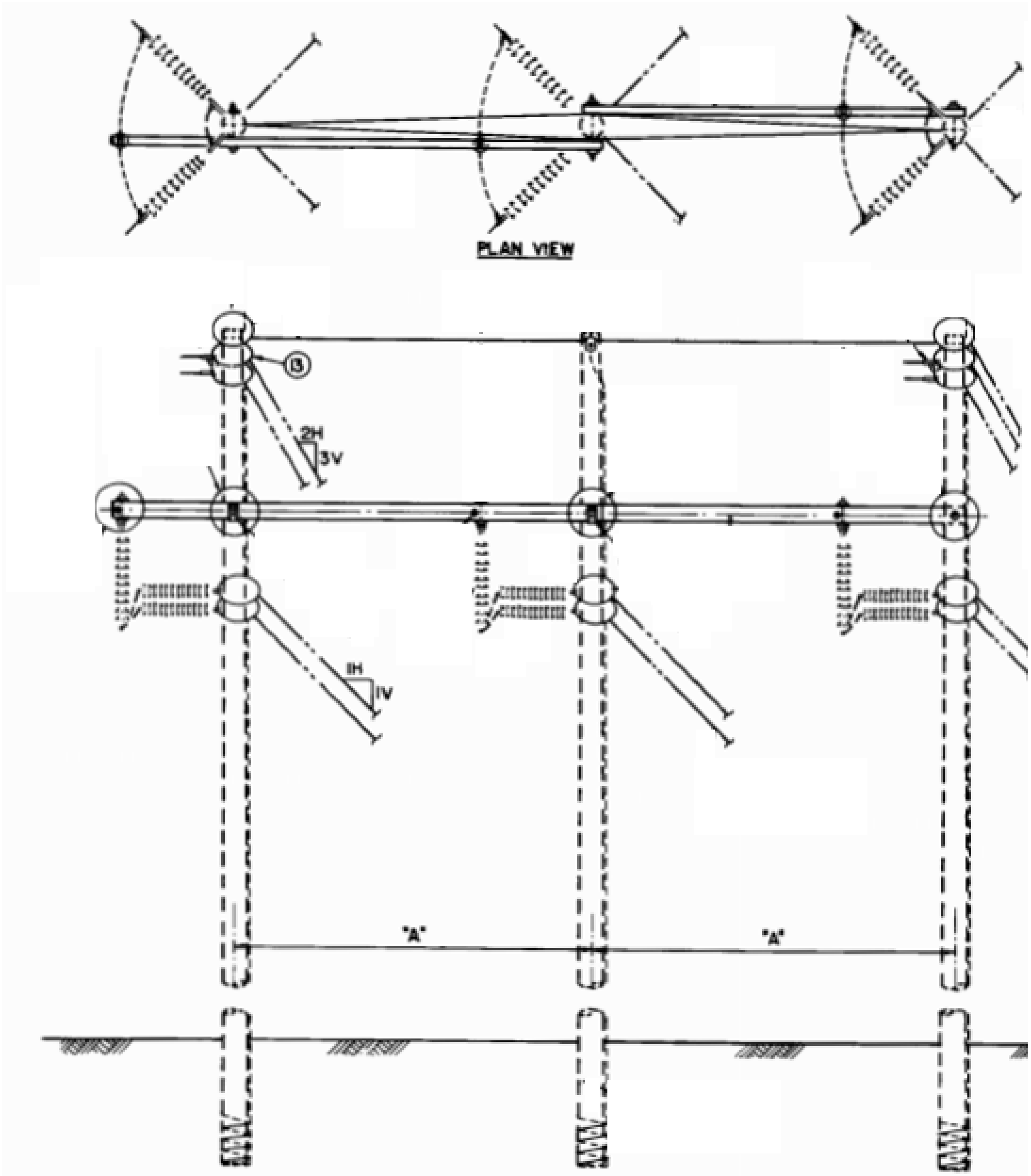

Fig: Typical dead-end structure (161KV/92KV).

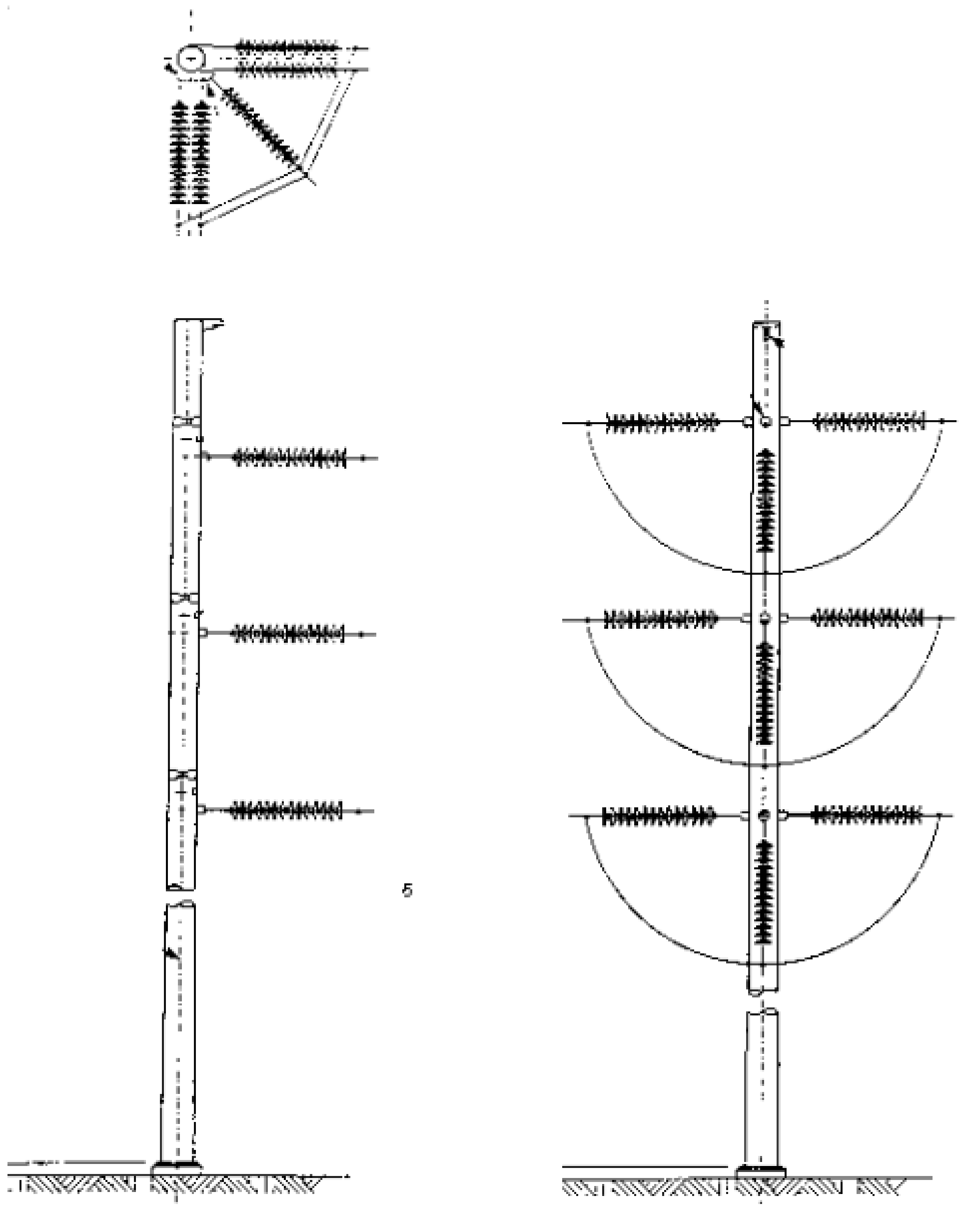

Fig: Typical dead-end structure (230kV/161KV/92KV).

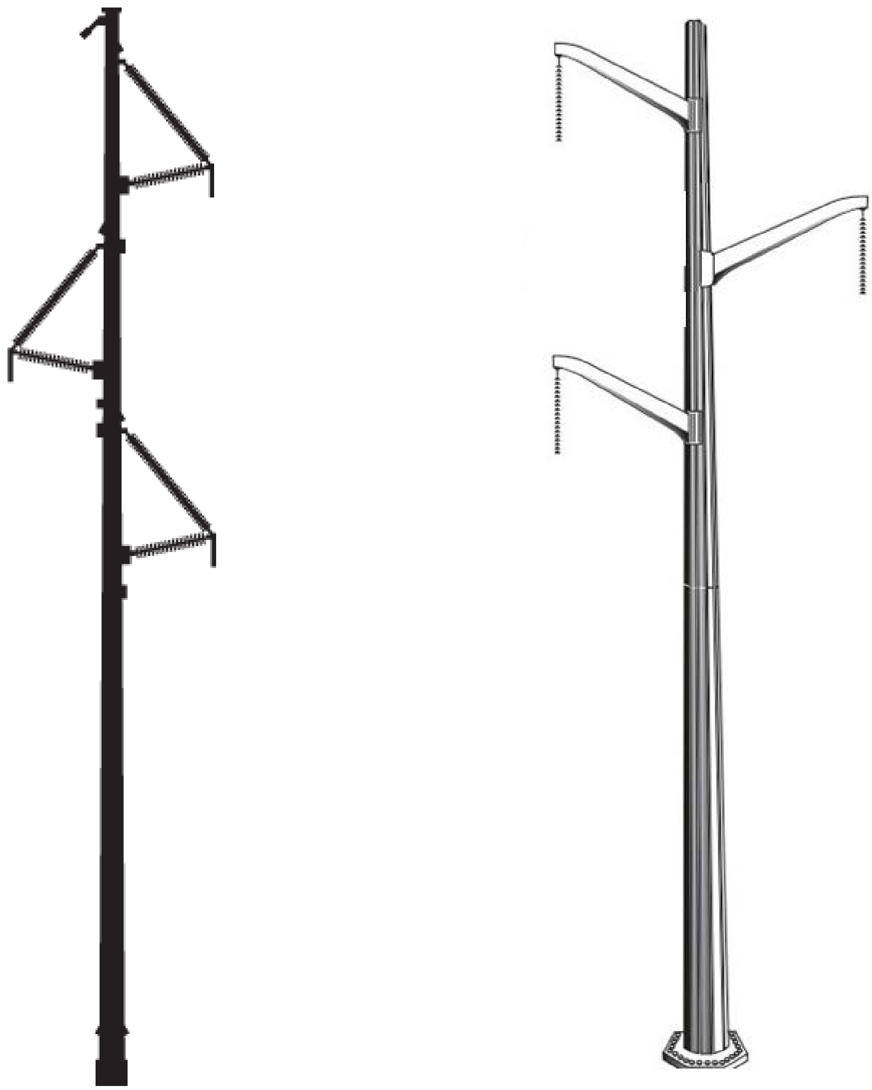

Fig: Typical steel pole tangent structure (230kV)

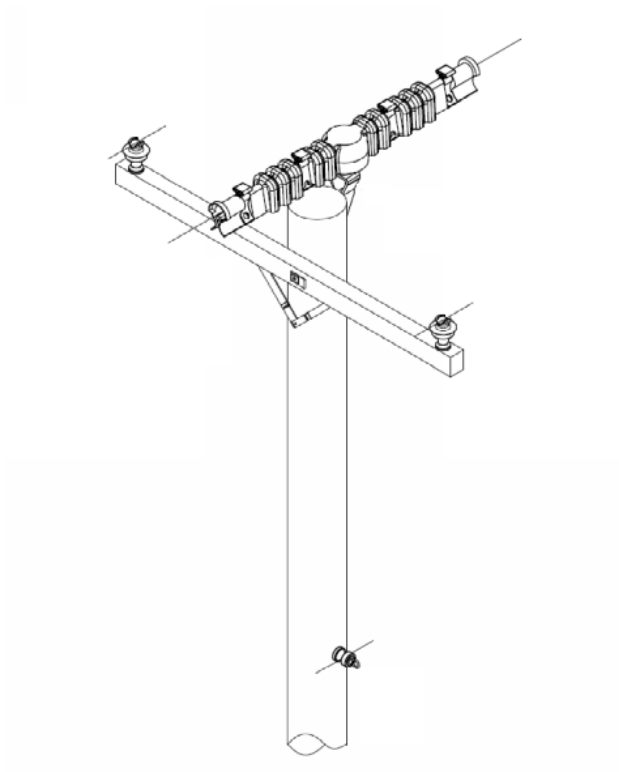

Fig: Typical tangent wood pole for distribution (12.7KV)

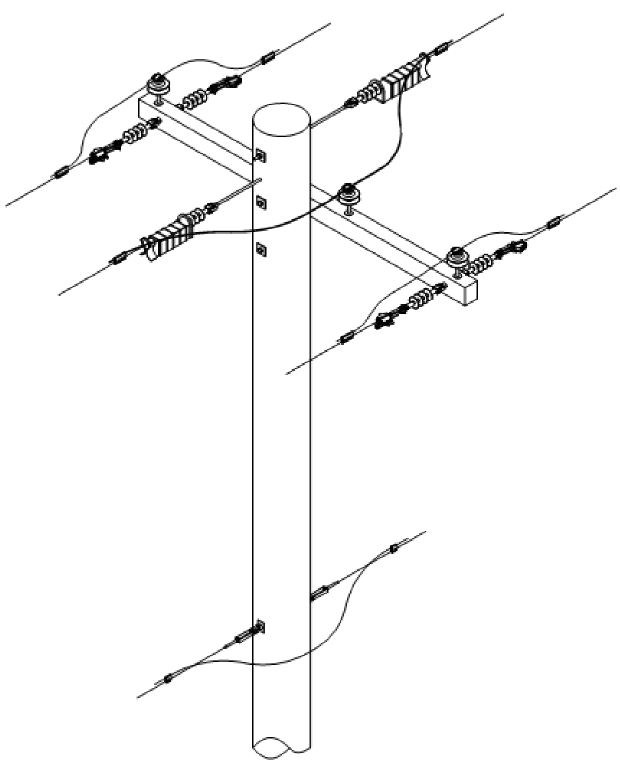

Fig: Typical dead-end wood pole for distribution (12.7KV)

F

RISK REGISTER

Risk Identification									Qualitative Analysis				Risk Monitoring and Control			
ID	Risk Category	Group	Alternate Conveyance Options	Category	Description	Raised By	Date Raised	Source	Impact Level	Probability Level	Risk Matrix Score	Qualitative Impact	Risk Strategy	Response Plan	Response Status	Owner
1	Construction	Tunneling	Conveyance Alternative 3A	3A Tunneling	Cost growth due to unforeseen conditions-geology, groundwater, hazardous materials.				4	4	16	High- Develop Mitigation Plan	Mitigate	Perform field geological investigations during preliminary design and design phases. Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
2	Construction	Tunneling	Conveyance Alternative 5A	5A Tunneling	Cost growth due to unforeseen conditions-geology, groundwater, hazardous materials.				4	4	16	High- Develop Mitigation Plan	Mitigate	Perform field geological investigations during preliminary design and design phases. Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
3	Construction	Tunneling	Conveyance Alternative 5C	5C Tunneling	Cost growth due to unforeseen conditions-geology, groundwater, hazardous materials.				4	4	16	High- Develop Mitigation Plan	Mitigate	Perform field geological investigations during preliminary design and design phases. Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
4	Agency Coordination	Overall RCS	Common	Metropolitan Water District Supply Cost	Uncertainty associated with cost of future Metropolitan Water District supplies.				5	3	15	High- Develop Mitigation Plan	Mitigate	Maintain engagement with MWD. Also establish a model at the appropriate phase of planning and include appropriate contingency.		
5	Agency Coordination	Overall RCS	Common	Treatment Plant	Cost uncertainty associated with agreement for Treatment Plant power use with San Diego Gas & Electric / Imperial Irrigation District.				5	3	15	High- Develop Mitigation Plan	Mitigate	Establish agreement framework at appropriate phase of planning and include appropriate contingency.		
6	Operations	Overall RCS	Common	Canal Systems	Terrorist activity on open water canal system could affect safety and result in loss of use.				5	3	15	High- Develop Mitigation Plan	Mitigate	Develop early warning monitoring plan and mitigate through storage system.		
7	Permits	Overall RCS	Common	Conveyance	Inability to obtain certification of environmental document as a result of potential range of issues.				5	3	15	High- Develop Mitigation Plan	Mitigate	Some alignment segments could require no environmental mitigation, some segments could require 1:1 mitigation, and limited segments could require higher mitigation ratios of 2:1 or 3:1. For the purpose of feasibility level cost estimating, a 1:1 mitigation ratio for all disturbed lands was assumed.		
8	Operations	Conveyance	Conveyance Alternative 3A	Conveyance	Earthquakes on pipelines, tunnels, storage reservoirs, WTP, and pump stations may result in loss of flow. (San Jacinto Fault Zone, Superstition Hills Fault Zone, Elmore Ranch Fault Zone, and Elsinore Fault Zone)				5	3	15	High- Develop Mitigation Plan	Mitigate	Minimize impact through design. Evaluate likely outage period and mitigate through appropriate storage.		
9	Operations	Conveyance	Conveyance Alternative 5A	Conveyance	Earthquakes on pipelines, tunnels, storage reservoirs, WTP, and pump stations may result in loss of flow. (San Jacinto Fault Zone and Elsinore Fault Zone)				4.5	3	14	High- Develop Mitigation Plan	Mitigate	Minimize impact through design. Evaluate likely outage period and mitigate through appropriate storage.		
10	Operations	Conveyance	Conveyance Alternative 5C	Conveyance	Earthquakes on pipelines, tunnels, storage reservoirs, WTP, and pump stations may result in loss of flow. (Elsinore Fault Zone)				4.25	3	13	High- Develop Mitigation Plan	Mitigate	Minimize impact through design. Evaluate likely outage period and mitigate through appropriate storage.		
11	Agency Coordination	Overall RCS	Conveyance Alternative 3A	3A Pump Stations	Cost uncertainty associated with agreement for pump station power use with San Diego Gas & Electric / Imperial Irrigation District.				4	3	12	Moderate-Actively Manage	Mitigate	Establish agreement framework at appropriate phase of planning and include appropriate contingency.		
12	Agency Coordination	Overall RCS	Conveyance Alternative 5A	5A Pump Stations	Cost uncertainty associated with agreement for pump station power use with San Diego Gas & Electric / Imperial Irrigation District.				4	3	12	Moderate-Actively Manage	Mitigate	Establish agreement framework at appropriate phase of planning and include appropriate contingency.		
13	Agency Coordination	Overall RCS	Conveyance Alternative 5C	5C Pump Stations	Cost uncertainty associated with agreement for pump station power use with San Diego Gas & Electric / Imperial Irrigation District.				4	3	12	Moderate-Actively Manage	Mitigate	Establish agreement framework at appropriate phase of planning and include appropriate contingency.		
14	Permits	Brine Management	Common	Brine Management-New River	May not be able to obtain permit to convey brine water to the New River.				4	3	12	Moderate-Actively Manage	Avoid	Determine feasibility at appropriate phase of planning and choose alternate brine management option if necessary.		
15	Operations	Overall RCS	Common	Existing All-American Canal	Earthquakes along Imperial Fault Line could result in loss of flow in All-American Canal.				5	2	10	Moderate-Actively Manage	Mitigate	Evaluate likely outage period and mitigate through appropriate storage.		

Risk Identification									Qualitative Analysis				Risk Monitoring and Control			
ID	Risk Category	Group	Alternate Conveyance Options	Category	Description	Raised By	Date Raised	Source	Impact Level	Probability Level	Risk Matrix Score	Qualitative Impact	Risk Strategy	Response Plan	Response Status	Owner
16	Agency Coordination	Conveyance	Common	New Pipeline Parallel to All-American Canal	Feasibility of obtaining Caltrans ROW for conveyance system.				5	2	10	Moderate-Actively Manage	Avoid	Determine feasibility at appropriate phase of planning.		
17	Public Affairs	Overall RCS	Common	Conveyance	Stakeholders along river may object and consider RCS "another straw".				5	2	10	Moderate-Actively Manage	Avoid	Continue engaging with stakeholders on the RCS to describe the purpose of the project and garner stakeholder input.		
18	Operations	Overall RCS	Common	Pump Stations	Pump stations introduce operational risk versus gravity flow systems.				3	3	9	Moderate-Actively Manage	Mitigate	Design Regional Conveyance System pump stations for maximum reliability and mitigate through San Diego county storage.		
19	Construction	Tunneling	Conveyance Alternative 3A	3A Tunneling	Cost growth due to unforeseen conditions-paleontology.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
20	Construction	Tunneling	Conveyance Alternative 3A	3A Tunneling	Cost growth due to unforeseen conditions-archelogy.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
21	Construction	Tunneling	Conveyance Alternative 5A	5A Tunneling	Cost growth due to unforeseen conditions-paleontology.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
22	Construction	Tunneling	Conveyance Alternative 5A	5A Tunneling	Cost growth due to unforeseen conditions-archelogy.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
23	Construction	Tunneling	Conveyance Alternative 5C	5C Tunneling	Cost growth due to unforeseen conditions-paleontology.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
24	Construction	Tunneling	Conveyance Alternative 5C	5C Tunneling	Cost growth due to unforeseen conditions-archelogy.				3	3	9	Moderate-Actively Manage	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
25	Construction	Overall RCS	Common	Conveyance	Financial Risk associated with assumptions on cost of money, future economic conditions (inflation), discount rate, ability to finance or sell bonds.				4	2	8	Moderate-Actively Manage		Develop further in the economic analysis workshop.		
26	ROW	Conveyance	Common	All-American Canal Siphon & Parallel Conveyance	Feasibility of obtaining ROW.				4	2	8	Moderate-Actively Manage	Avoid	Resolve issue at earliest reasonable phase.		
27	Agency Coordination	Conveyance	Common	All-American Canal Siphon & Parallel Conveyance	Agency coordination (e.g. Homeland Security) may affect feasibility.				4	2	8	Moderate-Actively Manage	Avoid	Resolve issue at earliest reasonable phase.		
28	Operations	Pump Stations	Common	Pump Stations	Remote pump stations may result in slower operator response times affecting system reliability.				4	2	8	Moderate-Actively Manage	Mitigate	Develop appropriate operations management strategy and additional operational staff. Mitigate through water storage.		
29	Operations	Pump Stations	Common	Pump Stations	Remote pump stations create vulnerability with respect to terrorist attack affecting system reliability.				4	2	8	Moderate-Actively Manage	Mitigate	Provide for appropriate security and mitigate with systems storage.		
30	Agency Coordination	Overall RCS	Conveyance Alternative 3A	New System Storage Reservoir	Division of Safety of Dams permit and implementation for new dam may affect cost.				4	2	8	Moderate-Actively Manage	Avoid	Early review with DSOD. Establish appropriate cost contingency.		
31	Agency Coordination	Overall RCS	Common	Conveyance	1) Transfer supply cost certainty post 2035 2) Renegotiation of the QSA/IID water transfer agreement to allow for extension post 2077.				4	2	8	Moderate-Actively Manage	Mitigate	Engage with parties and stakeholders early and ensure agreements are in place well in advance of construction start.		
32	Permits	Brine Management	Common	Brine Management-Salton Sea	May not be able to obtain permit to convey brine water to the Salton Sea.				2	3	6	Low-Monitor	Avoid	Determine feasibility at appropriate phase of planning.		
33	Permits	Brine Management	Common	Brine Management-Habitat	May not be able to obtain permit to convey brine water to a new habitat restoration.				2	3	6	Low-Monitor	Avoid	Determine feasibility at appropriate phase of planning.		
34	Agency Coordination	Conveyance	Conveyance Alternative 5A	Westside Main Canal Parallel Conveyance	Agency coordination (e.g. Homeland Security) may affect feasibility.				3	2	6	Low-Monitor	Avoid	Resolve issue at earliest reasonable phase.		
35	Public Affairs	Overall RCS	Common	Conveyance	Greenhouse Gas (GHG) Emissions increase from project if renewable energy plan is not met - public opposition to potential increase in GHG.				3	2	6	Low-Monitor	Mitigate	During design phase, develop an achievable Renewable Energy Plan which is defendable during permitting phase and achievable during operations phase.		

Risk Identification									Qualitative Analysis				Risk Monitoring and Control			
ID	Risk Category	Group	Alternate Conveyance Options	Category	Description	Raised By	Date Raised	Source	Impact Level	Probability Level	Risk Matrix Score	Qualitative Impact	Risk Strategy	Response Plan	Response Status	Owner
36	Climate Change	Overall RCS	Common	Colorado River	Climate change affects Colorado River flow to the extent that Quantity Settlement Agreement flow isn't available.				5	1	5	Low-Monitor	Accept	Common to both Colorado River Aqueduct and Regional Conveyance System mitigation. High Priority Status of QSA supplies largely insulates from cutbacks. Monitor during each potential phase.		
37	Design	Brine Management	Common	Brine Management-Evaporation Ponds	Cost uncertainty associated with preliminary concept.				2	2	4	Low-Monitor	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
38	Design	Brine Management	Common	Brine Management-Zero Liquid Discharge Facilities	Cost uncertainty associated with preliminary concept.				2	2	4	Low-Monitor	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
39	Design	Brine Management	Common	Brine Management-Outfall Facilities	Cost uncertainty associated with preliminary concept.				2	2	4	Low-Monitor	Mitigate	Include appropriate contingency at various project phases (e.g. Feasibility phase has highest contingency)		
40	ROW	Conveyance	Conveyance Alternative 5A	Westside Main Canal Parallel Conveyance	Feasibility of obtaining ROW.				2	2	4	Low-Monitor	Avoid	Resolve issue at earliest reasonable phase.		
41	Agency Coordination	Conveyance	Conveyance Alternative 3A	Westside Main Canal Parallel Conveyance	Agency coordination (e.g. Homeland Security) may affect feasibility.				2	2	4	Low-Monitor	Avoid	Resolve issue at earliest reasonable phase.		
42	Agency Coordination	Overall RCS	Conveyance Alternative 3A	Lake Wohlford	Agency coordination for use of existing water storage may affect feasibility.				2	2	4	Low-Monitor	Avoid	Establish agreement framework at appropriate phase of planning.		
43	Agency Coordination	Overall RCS	Conveyance Alternative 3A	Turner Reservoir	Agency coordination for use of existing water storage may affect feasibility.				2	2	4	Low-Monitor	Avoid	Establish agreement framework at appropriate phase of planning.		
44	Operations	Pump Stations	Common	Pump Stations	Reliability of power for Regional Conveyance System				2	2	4	Low-Monitor	Mitigate	Design Regional Conveyance System pump stations for maximum reliability and mitigate through San Diego county storage.		
45	Operations	Conveyance	Conveyance Alternative 3A	Conveyance	Heavy rain may result in flooding.				3	1	3	Low-Monitor	Mitigate	Design Regional Conveyance System to accommodate extreme flooding events.		
46	Operations	Conveyance	Conveyance Alternative 5A	Conveyance	Heavy rain may result in flooding.				3	1	3	Low-Monitor	Mitigate	Design Regional Conveyance System to accommodate extreme flooding events.		
47	Operations	Conveyance	Conveyance Alternative 5C	Conveyance	Heavy rain may result in flooding.				3	1	3	Low-Monitor	Mitigate	Design Regional Conveyance System to accommodate extreme flooding events.		

G

COSTS OPINION DETAILS

BLACK & VEATCH

San Diego County Water Authority
Regional Conveyance System Study
AACE International Level 4

Notes:

1 Unit costs are presented in January 2020 dollars.
2 Unit costs developed in the 2013 Master Plan Update were escalated from 2012 to 2020 using the ENR Construction Cost Indexes for the LA Market (2012 - 10270.93; January 2020 - 12144.49).
3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-1 Cost Opinion Summary

ESTIMATED CAPITAL COSTS			
ITEM	**Alternative 3A**	**Alternative 5A**	**Alternative 5C**
Canals	$ 59,200,000	$ 10,900,000	$ 1,600,000
Pipelines	$ 359,000,000	$ 428,200,000	$ 1,033,600,000
Tunnels	$ 1,512,800,000	$ 1,431,400,000	$ 450,000,000
Pumping Plants	$ 155,700,000	$ 156,000,000	$ 321,700,000
Power Generating/Pressure Control Facilities	$ 0	$ 31,097,000	$ 134,874,000
Electric Distribution	$ 49,200,000	$ 39,300,000	$ 52,100,000
Treatment Facility	$ 625,500,000	$ 760,700,000	$ 783,300,000
Operational Storage	$ 193,250,000	$ 108,250,000	$ 108,250,000
Office and Warehouse	$ 8,860,000	$ 8,860,000	$ 8,860,000
SUBTOTAL	$ 2,963,510,000	$ 2,974,707,000	$ 2,894,284,000
Construction Management	$ 664,200,000	$ 662,100,000	$ 665,500,000
Pre-Construction Costs[1]	$ 464,370,000	$ 450,680,000	$ 463,060,000
Contingency (10-30%)	$ 861,643,000	$ 874,632,000	$ 835,795,000
TOTAL (2020 Dollars)	$ 4,953,723,000	$ 4,962,119,000	$ 4,858,639,000

[1] Pre-Construction costs include the initial studies, engineering, right of way and property acquisition, CEQA/NEPA, public outreach, legal, environmental, owners representative, and staff support.

ESTIMATED ANNUAL COSTS			
ITEM	**Alternative 3A**	**Alternative 5A**	**Alternative 5C**
Energy Cost - Pumping	$ 82,000,000	$ 86,200,000	$ 219,800,000
Energy Cost - Treatment	$ 13,080,000	$ 13,080,000	$ 13,080,000
O&M and Replacement	$ 17,414,000	$ 16,867,000	$ 25,755,000
Water Treatment (excluding energy)	$ 30,600,000	$ 32,630,000	$ 32,970,000
Energy Recovery	-	-	($ 33,400,000)
TOTAL ANNUAL COSTS (2020 Dollars)	**$ 143,094,000**	**$ 148,777,000**	**$ 258,205,000**

San Diego County Water Authority
Regional Conveyance System Study
AACE International Level 4

Notes:
1 Unit costs are presented in January 2020 dollars.
2 Unit costs developed in the 2013 Master Plan Update were escalated from 2012 to 2020 using the ENR Construction Cost Indexes for the LA Market (2012 - 10270.93; January 2020 - 12144.49).
3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-2 Cost Summary - Canals

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Concrete Lined Channel		LF	246,576	$ 51,396,301	46,465	$ 9,685,165	7,800	$ 1,625,832
Siphons		LS	19	$ 7,790,000	3	$ 1,230,000	0	$ -
Subtotal				**$ 59,200,000**		**$ 10,900,000**		**$ 1,600,000**
Contingency	10%			$ 5,920,000		$ 1,090,000		$ 160,000
Total Capital Costs (2020 Dollars)				**$ 65,100,000**		**$ 12,000,000**		**$ 1,800,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance[1]	$ 10,000	per mile	46.7	$ 467,000	8.8	$ 88,002	1.5	$ 14,773
Total Annual Costs				**$ 467,000**		**$ 88,000**		**$ 15,000**

[1]Canal O&M unit cost provided by the Water Authority.

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Notes:

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3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-3 Cost Summary - Pipelines

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
102-inch Diameter Pipe (50 ksi)								
0.500-inch wall	$ 1,210	LF	139,475	$ 168,820,540	134,300	$ 162,556,720	239,950	$ 290,435,480
0.625-inch wall	$ 1,516	LF	38,970	$ 59,094,108	23,600	$ 35,787,040	63,725	$ 96,632,590
0.750-inch wall	$ 1,821	LF	26,300	$ 47,893,352	25,850	$ 47,073,884	47,375	$ 86,271,770
0.875-inch wall		LF	-		-		22,215	$ 44,321,591
1.000-inch wall		LF	-		-		31,625	$ 67,439,680
1.125-inch wall		LF	-		-		12,350	$ 28,788,344
1.250-inch wall		LF	-		-		4,900	$ 12,694,920
1.375-inch wall		LF	-		-		3,960	$ 11,287,194
1.500-inch wall		LF	-		-		2,500	$ 7,775,800
Trenching - Main Alignments								
Type 1E1B1 (Open Trench/Use Backfill)	$ 218	LF	-		17,424	$ 3,791,462	63,888	$ 13,902,029
Type 1E1B2 (Open Trench/Process Backfill)	$ 479	LF	-		76,032	$ 36,398,039	163,152	$ 78,104,125
Type 1E2B2 (Open Trench/Localized Blasting/Process Backfill)	$ 725	LF	4,750	$ 3,443,750	1,056	$ 765,600	159,984	$ 115,988,400
Type 1E3B2 (Wide Trench/Process Backfill)	$ 479	LF	-		65,472	$ 31,342,756	20,064	$ 9,605,038
Type 1E3B1 (Wide Trench/Native Backfill)	$ 218	LF	149,996	$ 32,639,184		$ -		$ -
Type 2 (Shored Trench)	$ 268	LF	49,999	$ 13,399,665	-		21,648	$ 5,801,664
72-inch Diameter Pipe (42 ksi) - Aqueduct Improvements								
0.375-inch wall	$ 629	LF	-	-	14,000	$ 8,801,811.20	14,000	$ 8,801,811
0.500-inch wall	$ 836	LF	-	-	21,585	$18,053,694.00	21,585	$ 18,053,694
0.625-inch wall	$ 972	LF	-		28,775	$27,980,810.00	28,775	$ 27,980,810
0.750-inch wall	$ 1,087	LF	-		1,850	$ 2,010,284.00	1,850	$ 2,010,284
Trenching - Aqueducts								
Type 2 (Shored Trench)	$ 268	LF		$ -	66,210	$17,744,280.00	66,210	$ 17,744,280
Accessories/Crossings/Specials								
Appurtenances	$ 50	LF	204,745	$10,237,250.00	226,194	$ 11,309,700	494,946	$ 24,747,300
Highway Crossings	$ 3,154	LF	250	$ 788,535	150	$ 473,121	4250	$ 13,405,095
River Crossings	$ 3,154	LF	-		-		450	$ 1,419,363
Surface/Utilities	$ 98	LF	204,745	$ 20,052,725	226,194	$ 22,153,440	494,946	$ 48,475,011
San Vicente Outfall Structure	$ 1,944,640	LS	-		1	$ 1,944,640	1	$ 1,944,640
Borrego Springs Turnout	$ 2,584,000	LS	1	$ 2,584,000	-		-	
Subtotal				**$ 359,000,000**		**$ 428,200,000**		**$ 1,033,600,000**
Contingency	30%			$ 107,700,000		$ 128,460,000		$ 310,080,000
Total Capital Costs (2020 Dollars)				**$ 466,700,000**		**$ 556,660,000**		**$ 1,343,680,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Labor	$ 3,529,000	LS	1	$ 3,529,000	1	$ 3,529,000	1	$ 3,529,000
O&M	0.5	% of Capital	1	$ 1,795,000	1	$ 2,141,000	1	$ 5,168,000
Total Annual Costs				**$ 5,324,000**		**$ 5,670,000**		**$ 8,697,000**

Staffing	**Crews**	**No. per Crew**	**Total Staff**	**Hourly Rate**	**Burden**	**Annual Hours**	**Total Annual Staff Cost**
Mechanical Maintenance	2	3	6	$ 34.39	1.60	2,080	$ 686,700
Operation							
Operator	1	8	8	$ 40.88	1.60	2,080	$ 1,088,389
Supervisor	1	1	1	$ 45.12	1.60	2,080	$ 150,159
Operation (Escondido)	1	1	1	$ 40.88	1.60	2,080	$ 136,049
Right-of-Way Maintenance	2	3	6	$ 36.13	1.60	2,080	$ 721,444
Maintenance - Supervisor	1	1	1	$ 56.34	1.60	2,080	$ 187,500
SCADA	1	2	2	$ 45.12	1.60	2,080	$ 300,319
Corrosion	1	2	2	$ 38.90	1.60	2,080	$ 258,918
Total Annual Staffing Cost							**$ 3,529,000**

Notes: The estimated staffing plan was provided by the Water Authority based on prior experience operating similar facilities. Rates were provided by the Water Authority.

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Notes:

1 Unit costs are presented in January 2020 dollars.
2 Unit costs developed in the 2012 Appendix G - Colorado River Conveyance Alternative Report were escalated from 2012 to 2020 using the ENR Construction Cost Indexes for the LA Market (2012 - 10270.93; January 2020 - 12144.49).
3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-4 Cost Summary - Tunnels

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Corridor 5A Tunnels								
T1 (Bow Willow Portal to Vent)	$ 5,860	LF			88,176	$ 516,731,308		
T2 (Vent to El Capitan)	$ 5,491	LF			92,928	$ 510,310,090		
T3 & T4 (El Capitan to San Vicente)	$ 8,307	LF			37,488	$ 311,408,077		
Corridor 5C Tunnels								
T1 (In-Ka-Pah Gorge PS3 to PS4)	$ 7,445	LF					12,100	$ 90,079,902
T2 (In-Ka-Pah Gorge PS4 to PS5)	$ 7,655	LF					6,336	$ 48,499,926
T3 & T4 (El Capitan to San Vicente)	$ 8,307	LF					37,488	$ 311,408,077
Corridor 3A Tunnels								
T1 (PS3 to Vent)	$ 5,216	LF	92,085	$ 480,283,427				
T2 (Vent to Moss Tree Portal)	$ 6,265	LF	126,720	$ 793,874,062				
T3 (I-15 to Twin Oaks WTP)	$ 7,823	LF	11,125	$ 87,033,003				
T4 (Tunnel Vent 2 to Lake Wohlford)	$ 5,452	LF	15,700	$ 85,597,349				
Intermediate Tunnel Shafts								
Alignment 5A Intermediate Shaft	$ 30,000	FT Deep			3,100	$ 93,000,000		
Alignment 3A Intermediate Shaft 1	$ 30,000	FT Deep	1,650	$ 49,500,000				
Alignment 3A Intermediate Shaft 2	$ 30,000	FT Deep	550	$ 16,500,000				
Subtotal				**$ 1,512,800,000**		**$ 1,431,400,000**		**$ 450,000,000**
Contingency	30%			$ 453,840,000		$ 429,420,000		$ 135,000,000
Total Capital Costs (2020 Dollars)				**$ 1,966,640,000**		**$ 1,860,820,000**		**$ 585,000,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance	0.1	% of Capital		$ 1,513,000		$ 1,431,000		$ 450,000
Total Annual Costs				**$ 1,513,000**		**$ 1,431,000**		**$ 450,000**

Note: Labor costs for day to day staffing of the tunnels are covered under pipelines.

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Notes:

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Table G-5 Cost Summary - Pumping Plants

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Pumping Plants on Main Regional Conveyance System								
800 Foot TDH Pumping Plant	$ 52,675,000	LS	0	$ -	1	$ 52,675,000	1	$ 52,675,000
700 Foot TDH Pumping Plant	$ 49,336,445	LS	1	$ 49,336,445				
Forebay - 40 acre-feet				$ -				
Earth Excavation	$ 6.00	CY	60,000	$ 360,000	60,000	$ 360,000	60,000	$ 360,000
Compacted Backfill	$ 12.00	CY	60,000	$ 720,000	60,000	$ 720,000	60,000	$ 720,000
Plastic Liner	$ 34.00	SY	25,000	$ 850,000	25,000	$ 850,000	25,000	$ 850,000
Outfall Structure	$ 185,000	LS	1	$ 185,000	1	$ 185,000	1	$ 185,000
Spillway Structure and Pipeline	$ 320,000	LS	1	$ 320,000	1	$ 320,000	1	$ 320,000
Sitework	$ 120,000	LS	1	$ 120,000	1	$ 120,000	1	$ 120,000
Pumping Plant for Improvements to Aqueduct System								
500 Foot TDH Pumping Plant	$ 33,525,000	LS	-		1	$ 33,525,000	1	$ 33,525,000
6 Million Gallon Suction Storage Tank	$ 12,000,000	LS	-		1	$ 12,000,000	1	$ 12,000,000
Subtotal per 800 Foot Pumping Station						**$ 55,230,000**		**$ 55,230,000**
Number of 800 Foot Pumping Station			-		2	**$ 110,460,000**	5	**$ 276,150,000**
Subtotal per 700 Foot Pumping Station				**$ 51,891,445**				
Number of 700 Foot Pumping Station			3	**$ 155,674,335**	-		-	
Subtotal per 500 Foot Pumping Station (Aqueduct)						**$ 45,525,000**		**$ 45,525,000**
Number of 500 Foot Pumping Station (Aqueduct)			-		1	**$ 45,525,000**	1	**$ 45,525,000**
Subtotal				**$ 155,700,000**		**$ 156,000,000**		**$ 321,700,000**
Contingency	20%			$ 31,140,000		$ 31,200,000		$ 64,340,000
Total Capital Costs (2020 Dollars)				**$ 186,840,000**		**$ 187,200,000**		**$ 386,040,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Staffing	Varies	LS	1	$ 950,810	1	$ 950,810	1	$ 1,718,679
Energy - 700/800 Foot Pumping Stations	Varies	LS	1	$ 82,000,000	2	$ 72,800,000	5	$ 206,400,000
Energy - 500 Foot Pumping Station (Aqueduct)	Varies	LS	-	-	1	$ 13,400,000	1	$ 13,400,000
Major Equipment Replacement	Varies	LS	1	$ 3,000,000	1	$ 2,400,000	1	$ 5,400,000
Operations & Maintenance	Varies	LS	1	$ 1,514,250	1	$ 1,515,000	1	$ 3,054,250
Total Annual Costs				**$ 87,470,000**		**$ 91,070,000**		**$ 229,970,000**

Staffing						**Alternative 3A Labor**		**Alternative 5A Labor**		**Alternative 5C Labor**	
Item	**Staff**	**Unit**	**Hourly Rate**	**Burden**	**Annual Hours**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Mechanical Maintenance	1	per PS	$ 39.88	1.60	2,080	3.00	$ 398,162	3.00	$ 398,162	6.00	$ 796,324
Electrical Maintenance	1	per PS	$ 37.03	1.60	2,080	3.00	$ 369,708	3.00	$ 369,708	6.00	$ 739,415
Supervisor	1	per Alternative	$ 54.97	1.60	2,080	1.00	$ 182,940	1.00	$ 182,940	1.00	$ 182,940
Total							**$ 950,810**		**$ 950,810**		**$ 1,718,679**

Notes: The estimated staffing plan was provided by the Water Authority based on prior experience operating similar faclities. Rates were provided by the Water Authority.

Major Equipment Replacement Costs			**Alternative 3A**			**Alternative 5A**			**Alternative 5C**		
Item	**Unit Cost**	**Unit**	**Quantity per PS**	**No. of PS**	**Annual Cost**	**Quantity per PS**	**No. of PS**	**Annual Cost**	**Quantity per PS**	**No. of PS**	**Annual Cost**
Pump Replacement - RCS Pumping Plants	$ 10,000,000	Every 30 Years	3	3	$ 3,000,000	3	2	$ 2,000,000	3	5	$ 5,000,000
Pump Replacement - Aqueduct Pumping Plants	$ 4,000,000	Every 30 Years	-	-	-	3	1	$ 400,000	3	1	$ 400,000
Total					**$ 3,000,000**			**$ 2,400,000**			**$ 5,400,000**

Notes: Properly sized, low speed vertical turbine pumps can last 50+ years before replacement, with the motor being re-wound every 20 years. However, based on conversations with the Water Authority, it was assumed pumps are replaced every 30 years.

Annual O&M Costs			**Alternative 3A**			**Alternative 5A**			**Alternative 5C**		
Item	**Unit Cost**	**Unit**	**Quantity per PS**	**Life Span**	**Annual Cost**	**Quantity per PS**	**Life Span**	**Annual Cost**	**Quantity per PS**	**Life Span**	**Annual Cost**
General O&M	$ 375,000	each	3	N/A	$ 1,125,000	3	N/A	$ 1,125,000	6	N/A	$ 2,250,000
Unanticipated Replacements (Note 2)	10.00	% of non-structural components	1	20	$ 389,250	1	20	$ 390,000	1	20	$ 804,250
Total					**$ 1,514,250**			**$ 1,515,000**			**$ 3,054,250**

Notes:

1) General O&M covers replacement of motor bearing oil, mechanical seals, air filters, and intruments. This cost was provided by the Water Authority as an average annual cost for O&M at their large pump stations.

2) Unanticipated replacements covers replacement of structural or piping rehabs, electrical equipment, and upgrades to the SCADA system.

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Table G-6 Cost Summary - Power Generating/Pressure Control Facilities

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Pressure Generating/Pressure Control Facility	$ 31,096,540	LS			1	$ 31,096,540	1	$ 31,096,540
Forebay - 40 ac-ft								
Earth Excavation	$ 6	CY	-		0		60,000	$ 360,000
Compacted Backfill	$ 12	CY	-		0		60,000	$ 720,000
Plastic Liner	$ 34	SY	-		0		25,000	$ 850,000
Intake Structure	$ 1,167,020	LS	-		0		1	$ 1,167,020
Spillway Structure & Pipeline	$ 291,460	LS	-		0		1	$ 291,460
Sitework	$ 107,380	LS	-		0		1	$ 107,380
Subtotal per Facility						**$ 31,096,540**		**$ 34,592,400**
Number of Power Generating Facilities					-		3	$ 103,777,200
Number of Pressure Control Facilities					1	$ 31,096,540	1	$ 31,096,540
Subtotal						**$ 31,096,540**		**$ 134,873,740**
Contingency	30%					$ 9,328,962.0		$ 40,462,122.0
Total Capital Costs (2020 Dollars)						**$ 40,000,000**		**$ 175,000,000**

Notes: No PGF were required for Alignment 3A. Therefore, the cost for the PGF/PCF was escalated from the 2012 and 2017 reports to 2020 dollars.

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance								
Labor	Varies	LS	-	-	1	$ 329,000	1	$ 493,000
Energy Recovery	$ (33,400,000)	LS	-	-	-	-	1	$ (33,400,000)
Maintenance	Varies	LS			1	$ 402,000	1	$ 1,846,000
Total Annual Costs								**$ (31,061,000)**
Total Annual Costs (without energy recovery)				**$ -**		**$ 731,000**		**$ 2,339,000**

Notes: O&M costs were escalated based on the 2012 and 2017 reports.

Staffing	**Crews**	**No. per Cr**	**Total Staff**	**Hourly Rate**	**Burden**	**Annual Hours**	**Total Annual Staff Cost**
Mechanical Maintenance	1	2	2	$ 39.88	1.60	2,080	$ 265,441
Electrical Maintenance	1	2	2	$ 37.03	1.60	2,080	$ 246,472
Total Annual Costs							**$ 512,000**

Notes: PGFs will share the same supervisor as the pumping stations.

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Notes:

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Table G-7 Cost Summary - Power Delivery

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
69 kV Substation	$ 236,480	EA		$ -	0	$ -	3	$ 709,440
69 kV Transmission Line	$ 236,480	MI		$ -	0	$ -	10	$ 2,364,800
230 kV Substation (90 MW load)	$ 23,385,507	EA		$ -	1	$ 23,385,507	0	$ -
230 kV Substation (220 MW load)	$ 29,231,293	EA		$ -	0	$ -	1	$ 29,231,293
230 kV Transmission Line	$ 667,131	MI	7.4	$ 4,936,771	23.8	$ 15,877,724	29.6	$ 19,747,085
161/92 kV Transmission Line	$ 307,274	MI	14.9	$ 4,578,389		$ -		$ -
12.7 kV Distribution Line	$ 234,386	MI	12.6	$ 2,953,270		$ -		$ -
Alignment 3A Substations	$ 12,254,297	LS	3	$ 36,762,892		$ -		$ -
Subtotal				**$ 49,200,000**		**$ 39,300,000**		**$ 52,100,000**
Contingency	30%			$ 14,760,000		$ 11,790,000		$ 15,630,000
Total Capital Costs (2020 Dollars)				**$ 63,960,000**		**$ 51,090,000**		**$ 67,730,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance	-	-	-	-	-	-	-	-
Contingency	-	-	-	-	-	-	-	-
Total Annual Costs	-	-	-	-	-	-	-	-

Note: SDG&E/IID are assumed to operate and maintain the facilities.

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Notes:

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Table G-8 Cost Summary - Salinity Treatment Facility

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
MF Plant	$ 2.26	MGD	141,000,000	$ 318,660,000	141,000,000	$ 318,660,000	141,000,000	$ 318,660,000
RO Plant	$ 1.87	MGD	135,000,000	$ 252,450,000	135,000,000	$ 252,450,000	135,000,000	$ 252,450,000
Solids Handling	$ 25,000,000	LS	1	$ 25,000,000	1	$ 25,000,000	1	$ 25,000,000
Screening	$ 15,000,000	LS	1	$ 15,000,000	1	$ 15,000,000	1	$ 15,000,000
Influent EQ Forebay - 5.8 Million Gallons	$ 1,450,000	LS	1	$ 1,450,000	1	$ 1,450,000	1	$ 1,450,000
Brine Management								
Brine Pipeline - 30-inch		LF	12,672	$ 12,925,440	145,200	$ 148,104,000	167,375	$ 170,722,500
Subtotal				**$ 625,500,000**		**$ 760,700,000**		**$ 783,300,000**
Contingency	30%			$ 187,650,000		$ 228,210,000		$ 234,990,000
Total Capital Costs (2020 Dollars)				**$ 813,150,000**		**$ 988,910,000**		**$ 1,018,290,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance								
Chemicals	$ 11,373,113	LS	1	$ 11,373,113	1	$ 11,373,113	1	$ 11,373,113
Major Equipment Replacements (RO membranes, e	$ 2,657,273	LS	1	$ 2,657,273	1	$ 2,657,273	1	$ 2,657,273
Major Equipment Replacements (MF membranes, e	$ 3,888,534	LS	1	$ 3,888,534	1	$ 3,888,534	1	$ 3,888,534
R&R general equipment	Varies	LS	1	$ 2,837,000	1	$ 4,865,000	1	$ 5,204,000
Labor	$ 9,843,934.43	LS	1	$ 9,843,934	1	$ 9,843,934	1	$ 9,843,934
Treatment - Energy	$ 13,080,000	LS	1	$ 13,080,000	1	$ 13,080,000	1	$ 13,080,000
Total Annual Costs				**$ 43,680,000**		**$ 45,710,000**		**$ 46,050,000**

Staffing	**Crews**	**No. per Crew**	**Total Staff**	**Hourly Rate**	**Burden**	**Annual Hours**	**Total Annual Staff Cost**
Plant Manager	1	1	1	$ 81.60	1.60	2,080	$ 271,565
Chief Operator	1	1	1	$ 73.93	1.60	2,080	$ 246,039
Operations Manager	1	1	1	$ 66.98	1.60	2,080	$ 222,909
Lead Operator (5)	1	5	5	$ 53.63	1.60	2,080	$ 892,403
Operator (15)	1	15	15	$ 48.59	1.60	2,080	$ 2,425,613
Compliance Officer	1	1	1	$ 65.34	1.60	2,080	$ 217,452
Health & Safety Specialist	1	1	1	$ 59.20	1.60	2,080	$ 197,018
OT/System Integrator	1	1	1	$ 63.75	1.60	2,080	$ 212,160
Instrument Tech (4)	1	4	4	$ 57.75	1.60	2,080	$ 768,768
Process Control Engineer	1	1	1	$ 75.78	1.60	2,080	$ 252,196
Maintenance Manager	1	1	1	$ 66.98	1.60	2,080	$ 222,909
Maintenance Supervisor (2)	1	2	2	$ 63.75	1.60	2,080	$ 424,320
Electrical Tech (4)	1	4	4	$ 57.75	1.60	2,080	$ 768,768
Mechanical Tech (10)	1	10	10	$ 51.05	1.60	2,080	$ 1,698,944
Laborer (12)	1	12	12	$ 46.25	1.60	2,080	$ 1,847,040
Janitor	1	1	1	$ 39.88	1.60	2,080	$ 132,721
Brine Pipeline	1	2	2	$ 48.59	1.60	2,080	$ 323,415
Total							**$ 11,120,000**

Note: Staffing plan shown was provided by the Water Authority by escalating the desal plants current staffing. However, we recommend a staffing plan of 54 for the treatment plant (including 2 for the brine pipeline). The annual labor cost shown was scaled down to 54 (including 2 for the brine pipeline) when applied.

General O&M Costs			**Alternative 3A**	**Alternative 5A**	**Alternative 5C**
Item	**Unit Cost**	**Unit**	**Annual Cost**	**Annual Cost**	**Annual Cost**
General O&M	3%	% of non-structural components	$ 2,836,693	$ 4,864,693	$ 5,203,693
Total			**$ 2,837,000**	**$ 4,865,000**	**$ 5,204,000**

Note: General O&M covers structural and piping repairs, general upkeep, and other miscellaneous expenses and was based on prior experience in similar facilities.

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Notes:

1 Unit costs are presented in January 2020 dollars.
2 Unit costs developed in the 2013 Master Plan Update were escalated from 2012 to 2020 using the ENR Construction Cost Indexes for the LA Market (2012 - 10270.93; January 2020 - 12144.49).
3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-9 Cost Summary - Environmental Mitigation

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Environmental Mitigation	Varies	LS	1	$ 17,323,500	1	$ 10,785,500	1	$ 17,852,500
Subtotal				**$ 17,300,000**		**$ 10,800,000**		**$ 17,900,000**
Contingency	30%			$ 5,190,000		$ 3,240,000		$ 5,370,000
Total Capital Costs (2020 Dollars)				**$ 22,500,000**		**$ 14,040,000**		**$ 23,270,000**

Notes: Environmental mitigation costs include the price of mitigation land, in addition to monitoring during construction and post construction activities.

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Total Annual Costs				**$ -**		**$ -**		**$ -**

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Notes:

1 Unit costs are presented in January 2020 dollars.
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3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-10 Cost Summary - Storage Facilities

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
40 MG Day Storage Tanks	$ 52,000,000	LS	1	$ 52,000,000	1	$ 52,000,000	1	$ 52,000,000
900 ac-ft IID Operational Storage	$ 56,250,000	LS	1	$ 56,250,000	1	$ 56,250,000	1	$ 56,250,000
RCS Operational Storage								
Dam Raise - Lake Wohlford	$ 65,000,000	LS	1	$ 65,000,000	-		-	$ -
Inlet/Outlet Structure - Lake Wohlford	$ 20,000,000	LS	1	$ 20,000,000	-	$ -	-	$ -
Subtotal				**$ 193,250,000**		**$ 108,250,000**		**$ 108,250,000**
Contingency	30%			$ 57,975,000		$ 32,475,000		$ 32,475,000
Total Capital Costs (2020 Dollars)				**$ 251,225,000**		**$ 140,725,000**		**$ 140,725,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance								
Labor - Lake Wohlford	$ 352,000	LS	1	$ 352,000				
O&M - IID 900 ac-ft	0.25	% of Capita	1	$ 140,625	1	$ 140,625	1	$ 140,625
O&M - Lake Wohlford	0.25	% of Capita	1	$ 212,500		$ -		$ -
O&M - 40 MG Storage Tanks	0.25	% of Capita	1	$ 50,000		$ -		$ -
Total Annual Costs				**$ 705,000**		**$ 141,000**		**$ 141,000**

Staffing	**Crews**	**No. per Cr**	**Total Staff**	**Hourly Rate**	**Burden**	**Annual Hours**	**Total Annual Staff Cost**
Operators	1	3	3	$ 35.25	1.60	2,080	$ 351,936

Notes: Staffing estimate is for the operation of Lake Wohlford. The 900 ac-ft operational storage reservoir required for IID is assumed to be maintained either by the staff at the adjacent treatment plant or by IID. Similarly, the 40 MG storage tank is assumed to be operated by the Water Authority's staff at the nearby Twin Oaks WTP.

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Notes:
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3 Unit costs developed in the 2017 Update to the 2013 Master Plan were escalated from 2017 to 2020 using the ENR Construction Cost Indexes for the LA Market (August 2017 - 11962.32; January 2020 - 12144.49).

Table G-11 Cost Summary - Office and Warehouse

Estimated Capital Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Warehouse	$ 260	SF	10000	$ 2,600,000	10000	$ 2,600,000	10000	$ 2,600,000
Offices	$ 380	SF	2000	$ 760,000	2000	$ 760,000	2000	$ 760,000
Storage Yard	$ 125	SF	20000	$ 2,500,000	20000	$ 2,500,000	20000	$ 2,500,000
Equipment / Tools	$ 1,000,000	LS	1	$ 1,000,000	1	$ 1,000,000	1	$ 1,000,000
Vehicles	$ 2,000,000	LS	1	$ 2,000,000	1	$ 2,000,000	1	$ 2,000,000
Subtotal				**$ 8,860,000**		**$ 8,860,000**		**$ 8,860,000**
Contingency	30%			$ 2,658,000		$ 2,658,000		$ 2,658,000
Total Capital Costs (2020 Dollars)				**$ 11,518,000**		**$ 11,518,000**		**$ 11,518,000**

Estimated Annual Costs			**Alternative 3A**		**Alternative 5A**		**Alternative 5C**	
Item	**Unit Cost**	**Unit**	**Quantity**	**Cost**	**Quantity**	**Cost**	**Quantity**	**Cost**
Operations & Maintenance	$ 1,000,000	LS	1	$ 1,000,000	1	$ 1,000,000	1	$ 1,000,000
Staffing	$ 349,939	LS	1	$ 349,939	1	$ 349,939	1	$ 349,939
Energy	$ 100,000	LS	1	$ 100,000	1	$ 100,000	1	$ 100,000
Total Annual Costs				**$ 1,450,000**		**$ 1,450,000**		**$ 1,450,000**

Staffing	**Crews**	**No. per Cr**	**Total Staff**	**Hourly Rate**	**Burden**	**Annual Hours**	**Total Annual Staff Cost**
Warehouse Tech	1	1	1	$ 47.40	1.60	2,080	$ 157,747
Warehouse Manager	1	1	1	$ 57.75	1.60	2,080	$ 192,192
Total							**$ 349,939**

Notes: The estimated staffing plan was provided by the Water Authority based on prior experience operating similar facilities. Rates were provided by the Water Authority.

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Table G-12 Summary of Additional Staff at Kearny Mesa

Item	Crews	No. per Crew	Total Staff	Annual Salary	Burden	Annual Hours	Total Annual Staff Cost
HR	1	1	1	$ 48.56	1.60	2,080	$ 161,608
Accounting	1	1	1	$ 39.88	1.60	2,080	$ 132,721
Budgeting	1	1	1	$ 59.20	1.60	2,080	$ 197,018
Payroll	1	1	1	$ 35.25	1.60	2,080	$ 117,312
IT Supervisor	1	1	1	$ 68.65	1.60	2,080	$ 228,467
IT Analyst	1	1	1	$ 56.34	1.60	2,080	$ 187,500
Water Resources Principal	1	1	1	$ 73.93	1.60	2,080	$ 246,039
Water Resources Senior	1	1	1	$ 68.65	1.60	2,080	$ 228,467
Engineer (P.E.)	1	1	2	$ 68.65	1.60	2,080	$ 456,934
Engineering Senior Engineer	1	1	1	$ 75.78	1.60	2,080	$ 252,196
O&M Manager	1	1	1	$ 85.00	1.60	2,080	$ 282,880
Total							**$ 2,490,000**

Notes: The estimated staffing plan was provided by the Water Authority based on prior experience operating similar facilities.
Rates were provided by the Water Authority.

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Table G-13 Summary of Pre-Construction Costs and Construction Management

Pre-Construction Costs	**Alternative 3A**	**Alternative 5A**	**Alternative 5C**
Phase A Planning Study	$ 2,600,000	$ 2,600,000	$ 2,600,000
Phase B Planning Study	$ 1,300,000	$ 1,300,000	$ 1,300,000
Procure Preliminary Designer	$ 50,000	$ 50,000	$ 50,000
Procure Env. Consultant	$ 50,000	$ 50,000	$ 50,000
Preliminary Design	$ 2,000,000	$ 2,000,000	$ 2,000,000
Environmental Work	$ 3,900,000	$ 3,900,000	$ 3,900,000
Right-of-Way: Title, Appraisals, Phase 1, Survey, Acquisition, and Legal	$ 15,532,000	$ 14,828,000	$ 21,318,000
Property Acquisition/Land Costs	$ 17,063,817	$ 9,623,077	$ 16,386,755
CEQA / NEPA / Permits	$ 5,200,000	$ 5,200,000	$ 5,200,000
Public Outreach	$ 2,810,000	$ 2,810,000	$ 2,810,000
Legal Review	$ 6,250,000	$ 6,250,000	$ 6,250,000
Environmental Mitigation/Monitoring	$ 17,323,500	$ 10,785,500	$ 17,852,500
Owners Representative/Programmatic Management	$ 31,644,000	$ 31,644,000	$ 31,644,000
Staff Support - Programmatic	$ 31,644,000	$ 31,644,000	$ 31,644,000
Design + Bid + NTP	$ 327,000,000	$ 328,000,000	$ 320,050,000
Total	**$ 464,370,000**	**$ 450,680,000**	**$ 463,060,000**

Notes: Programmatic costs were primarily developed by the Water Authority. In some cases, the cost was developed as a percentage of the construction cost for each component, while other costs, such as environmental mitigation, were developed in more detail.

Construction Management	**Alternative 3A**	**Alternative 5A**	**Alternative 5C**
Construction Management	$ 664,200,000	$ 662,100,000	$ 665,500,000

Notes: Construction management was primarily developed by the WA as a percentage of the estimated capital cost of each facility based on the WA's experience on facilities of similar scope.

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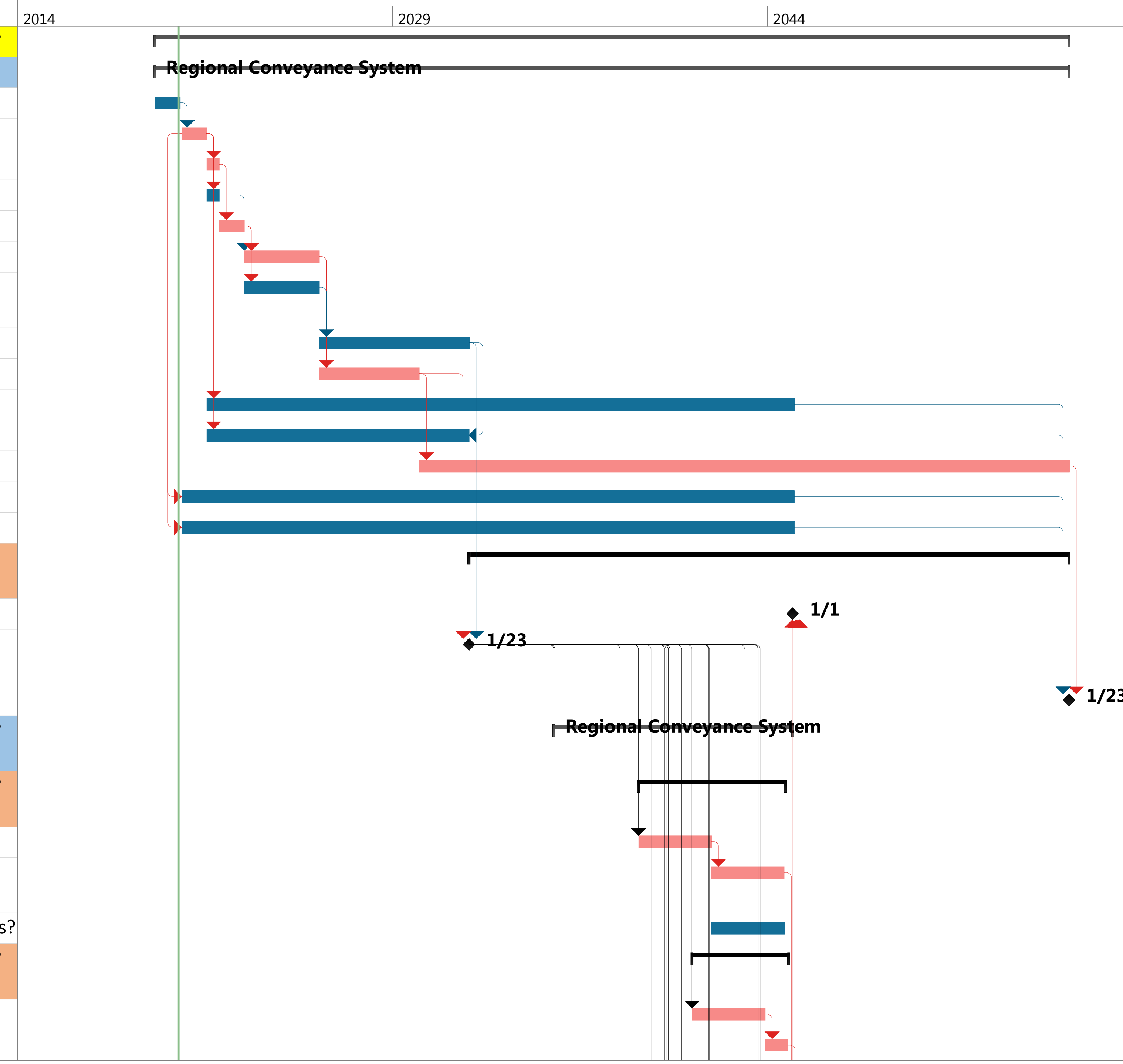
ALTERNATIVE SCHEDULES

Alternative 3A - Schedule

SDCWA
RCS Study - Phase A
Alternative 3A

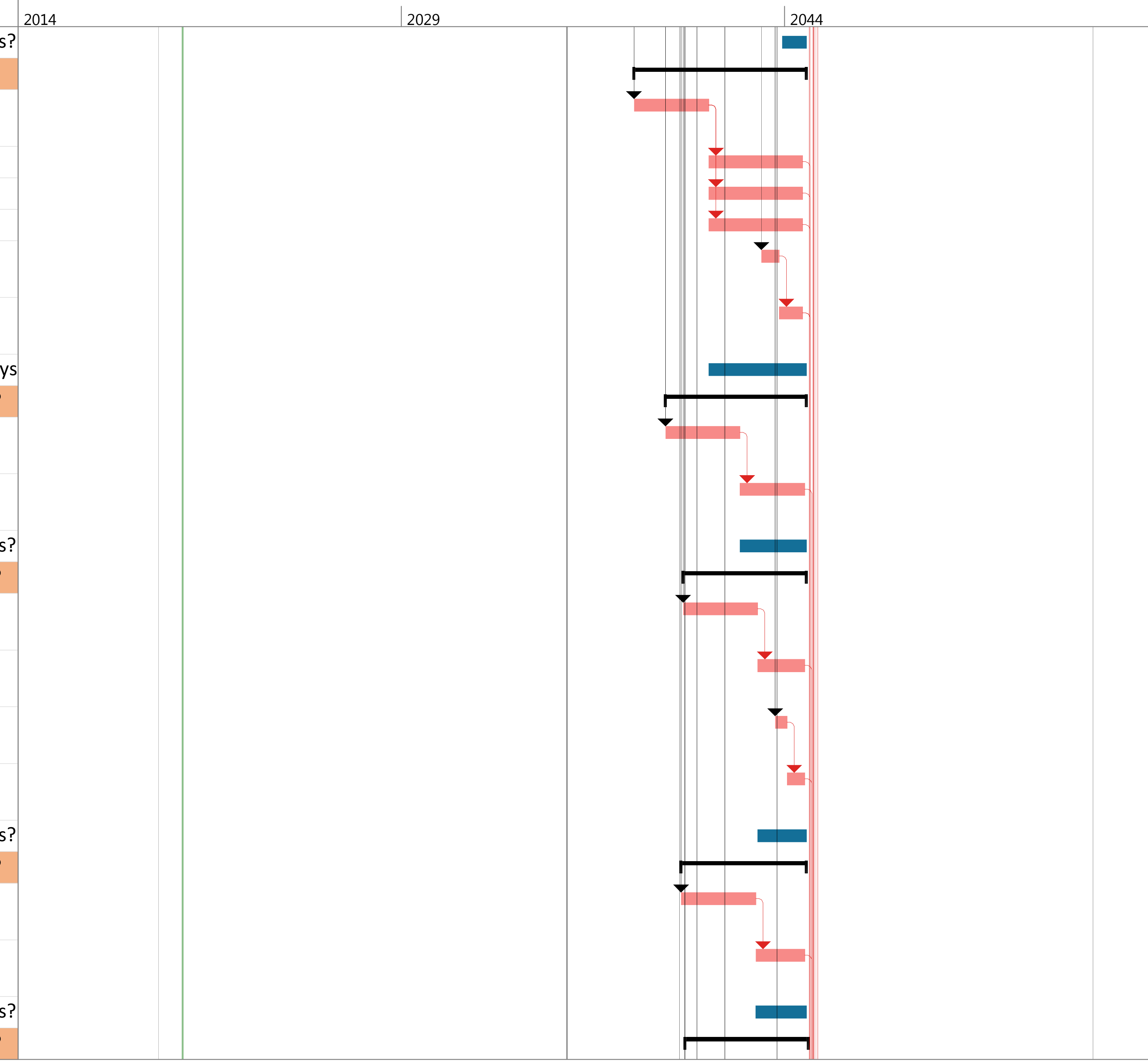
ID	WBS	Task Name	Start	Finish	Duration
1	**A**	**Regional Conveyance System**	**Mon 7/1/19**	**Sun 1/23/56**	**9541 days?**
2	**A-1**	**Initial Program Development**	**Mon 7/1/19**	**Sun 1/23/56**	**9541 days**
3	A-1.1	Phase A Planning Study	Mon 7/1/19	Tue 6/30/20	365 edays
4	A-1.2	Phase B Planning Study	Fri 7/24/20	Fri 7/23/21	364 edays
5	A-1.3	Procure Preliminary Designer	Sat 7/24/21	Sun 1/23/22	183 edays
6	A-1.4	Procure Env. Consultant	Sat 7/24/21	Sun 1/23/22	183 edays
7	A-1.5	Preliminary Design	Mon 1/24/22	Mon 1/23/23	364 edays
8	A-1.6	Environmental Work	Tue 1/24/23	Fri 1/23/26	1095 edays
9	A-1.7	Right-of-Way: Title, Appraisals, Phase 1, Survey, Acquisition, and Legal	Tue 1/24/23	Fri 1/23/26	1095 edays
10	A-1.8	Land Acquisition	Sat 1/24/26	Fri 1/23/32	2190 edays
11	A-1.9	CEQA / NEPA / Permits	Sat 1/24/26	Wed 1/23/30	1460 edays
12	A-1.10	Public Outreach	Sat 7/24/21	Mon 1/23/45	8584 edays
13	A-1.11	Legal Review	Sat 7/24/21	Fri 1/23/32	3835 edays
14	A-1.12	Environmental Mitigation/Monitoring	Thu 1/24/30	Sun 1/23/56	9495 edays
15	A-1.13	Owner's Representative/Programmatic Management	Fri 7/24/20	Mon 1/23/45	8949 edays
16	A-1.14	Staff Support - Programmatic	Fri 7/24/20	Mon 1/23/45	8949 edays
17	**A-1.200**	**Project Constraint Milestones**	**Fri 1/23/32**	**Sun 1/23/56**	**6262 days**
18	A-1.200.1	End of Construction Tasks	Sun 1/1/45	Sun 1/1/45	0 days
19	A-1.200.4	Initial Program Substantially Complete (project work can start)	Fri 1/23/32	Fri 1/23/32	0 days
20	A-1.200.3	Initial Program Complete	Sun 1/23/56	Sun 1/23/56	0 days
21	**A-2**	**3A Alignment (North) Design, Construction, Commissioning**	**Wed 6/13/35**	**Sun 1/1/45**	**2493 days?**
22	**A-2.1**	**WMC Parallel Conveyance (Canal MP 0 to MP -52.5)**	**Mon 11/1/38**	**Mon 9/12/44**	**1530 days?**
23	A-2.1.1	Parallel Conveyance Design + Bid + NTP	Mon 11/1/38	Mon 9/30/41	760 days
24	A-2.1.2	Construct Parallel Conveyance (Canal MP 0 to MP -52.5)	Mon 9/30/41	Mon 8/29/44	760 days
25	A-2.1.5	Construction Management	Mon 9/30/41	Mon 9/12/44	770.88 days?
26	**A-2.2**	**Imperial Valley Operational Storage (900 AF)**	**Mon 12/24/40**	**Mon 11/7/44**	**1010 days?**
27	A-2.2.1	Joint Op Storage Design + Bid + NTP	Mon 12/24/40	Mon 11/23/43	760 days
28	A-2.2.2	Joint Op Storage Construction	Mon 11/23/43	Mon 10/24/44	240 days



Project: MS Project Scheduling
Date: Wed 6/10/20

Task | Project Summary | Manual Task | Start-only | Deadline | Manual Progress
Split | Inactive Task | Duration-only | Finish-only | Critical
Milestone | Inactive Milestone | Manual Summary Rollup | External Tasks | Critical Split
Summary | Inactive Summary | Manual Summary | External Milestone | Progress

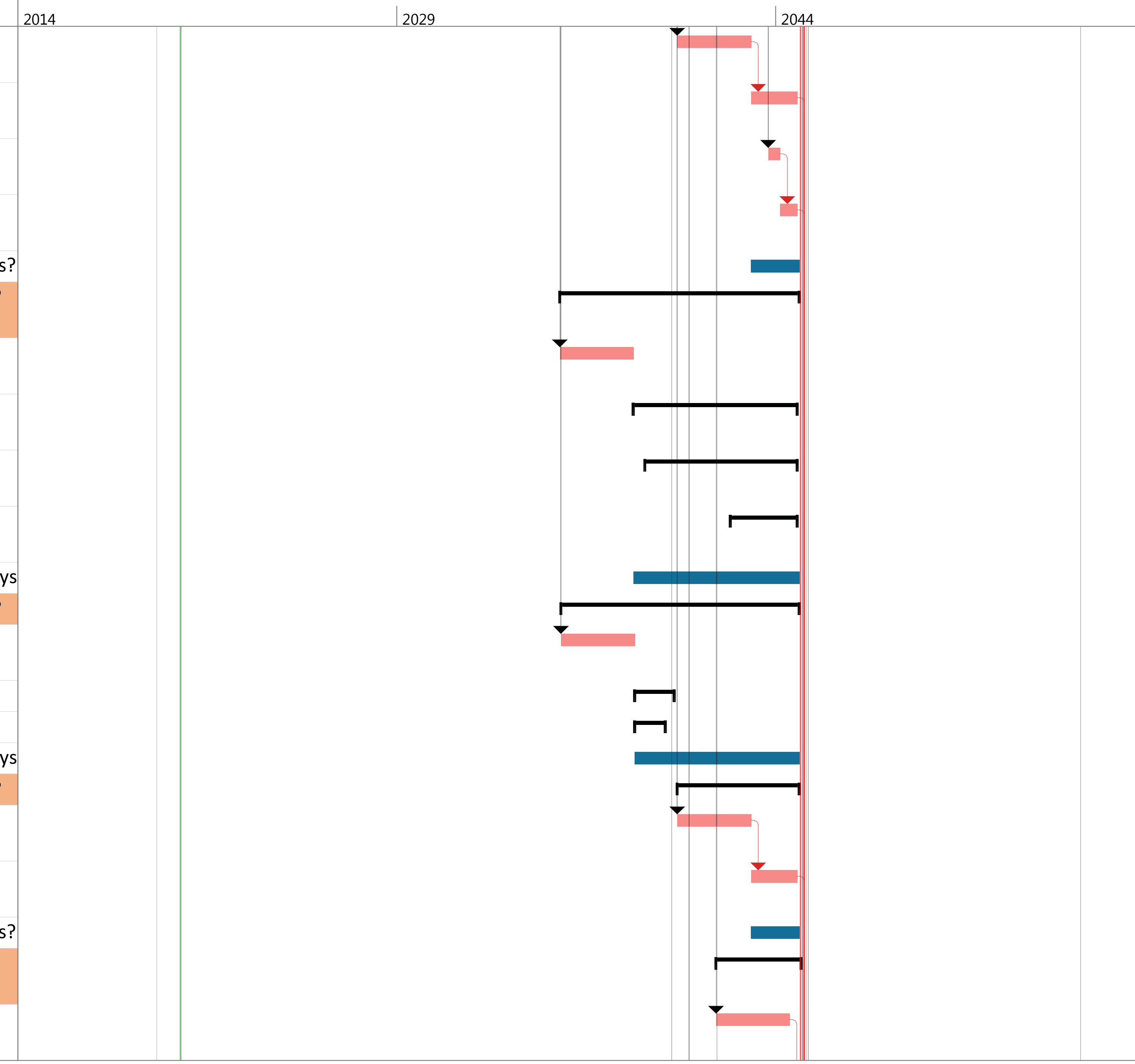
ID	WBS	Task Name	Start	Finish	Duration
29	A-2.2.5	Construction Management	Mon 11/23/43	Mon 11/7/44	250.88 days?
30	**A-2.3**	**Salinity Treatment Plant + PS 1 + Forebay**	**Mon 2/8/38**	**Mon 11/7/44**	**1760 days**
31	A-2.3.1	Salinity Treatment + PS 1 + Forebay Design + Bid + NTP	Mon 2/8/38	Mon 1/7/41	760 days
32	A-2.3.2	Salinity Treatment Construction	Mon 1/7/41	Mon 9/12/44	960 days
33	A-2.3.3	Brine Management Construction	Mon 1/7/41	Mon 9/12/44	960 days
34	A-2.3.4	PS1 + Forebay Construction	Mon 1/7/41	Mon 9/12/44	960 days
35	A-2.3.5	Detailed Design of Salinity Treatment and PS1 Operating Power Facilities + NTP	Mon 2/2/43	Mon 10/12/43	180 days
36	A-2.3.6	Construct Salinity Treatment and PS 1 Operating Power Facilities	Mon 10/12/43	Mon 9/12/44	240 days
37	A-2.3.7	Construction Management	Mon 1/7/41	Mon 11/7/44	1000.88 days
38	**A-2.4**	**Pipeline Reach 1 (MP 0 to MP 21.4)**	**Mon 5/2/39**	**Mon 11/7/44**	**1440 days?**
39	A-2.4.1	Pipeline Reach 1 Design + Bid + NTP	Mon 5/2/39	Mon 3/31/42	760 days
40	A-2.4.2	Pipeline Reach 1 Construction (MP 0 to MP 21.4)	Mon 3/31/42	Mon 10/10/44	660 days
41	A-2.4.3	Construction Management	Mon 3/31/42	Mon 11/7/44	680.88 days?
42	**A-2.5**	**Pump Station 2 (MP 21.4) and Forebay**	**Mon 1/9/40**	**Mon 11/7/44**	**1260 days?**
43	A-2.5.1	PS 2 Design + Bid + NTP	Mon 1/9/40	Mon 12/8/42	760 days
44	A-2.5.3	PS 2 Construction	Mon 12/8/42	Mon 10/10/44	480 days
45	A-2.5.4	Detailed Design of PS 2 Operating Power Facilities + NTP	Mon 8/17/43	Mon 2/1/44	120 days
46	A-2.5.5	Construct PS 2 Operating Power Facilities	Mon 2/1/44	Mon 10/10/44	180 days
47	A-2.5.6	Construction Management	Mon 12/8/42	Mon 11/7/44	500.88 days?
48	**A-2.6**	**Pipeline Reach 2 & 3 (MP 21.4 to MP 37.9)**	**Mon 12/12/39**	**Mon 11/7/44**	**1280 days?**
49	A-2.6.1	Pipeline Reach 2 & 3 Design + Bid + NTP	Mon 12/12/39	Mon 11/10/42	760 days
50	A-2.6.2	Pipeline Reach 2 & 3 Construction (MP 22.7 to MP 37.9)	Mon 11/10/42	Mon 10/10/44	500 days
51	A-2.6.3	Construction Management	Mon 11/10/42	Mon 11/7/44	520.88 days?
52	**A-2.7**	**Pump Station 3 (MP 36.6) and Forebay**	**Mon 2/6/40**	**Mon 12/5/44**	**1260 days?**



Project: MS Project Scheduling
Date: Wed 6/10/20

Task
Split
Milestone
Summary
Project Summary
Inactive Task
Inactive Milestone
Inactive Summary
Manual Task
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
External Tasks
External Milestone
Deadline
Critical
Critical Split
Progress
Manual Progress

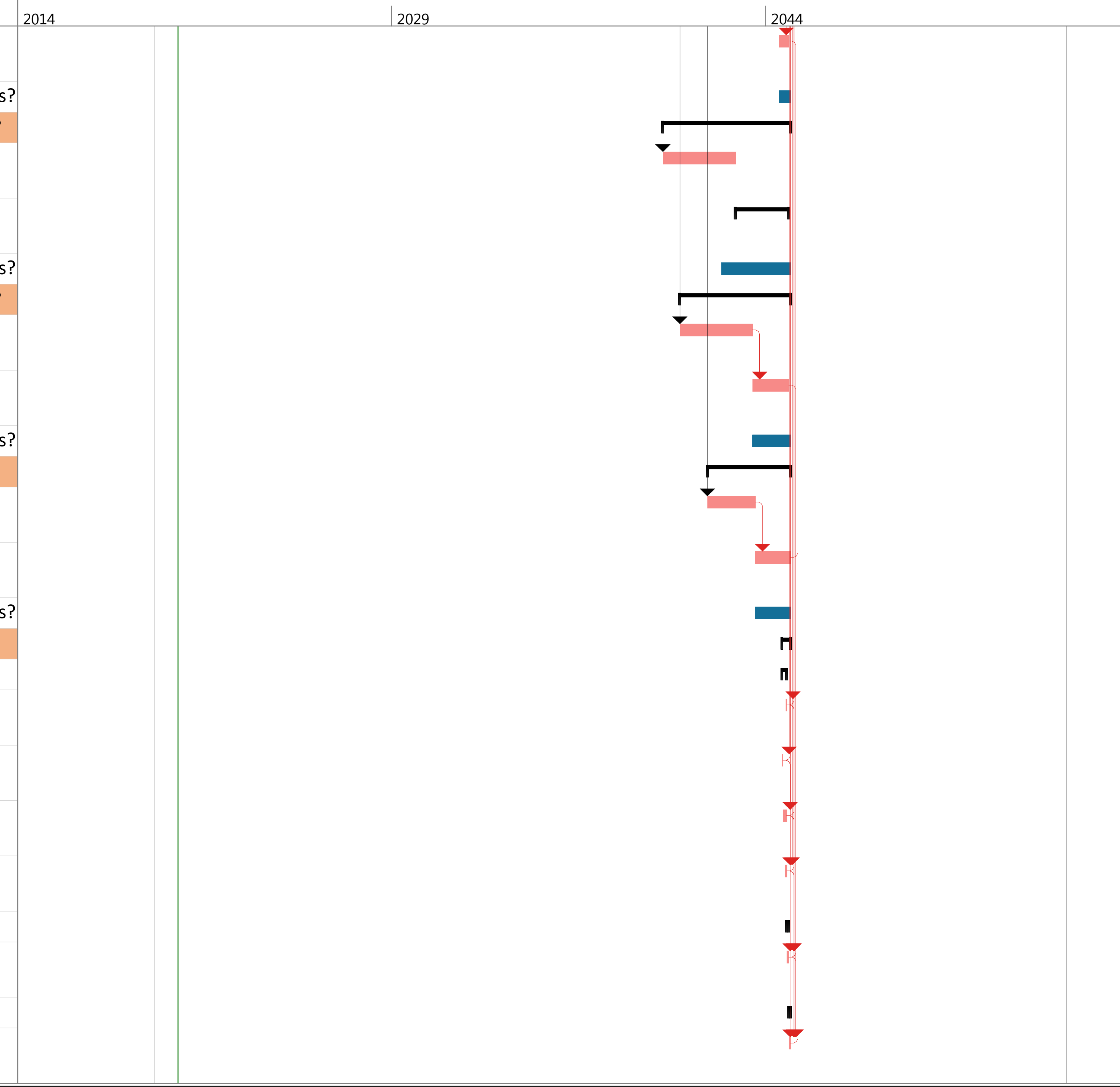
ID	WBS	Task Name	Start	Finish	Duration
53	A-2.7.1	PS 3 Design + Bid + NTP	Mon 2/6/40	Mon 1/5/43	760 days
54	A-2.7.3	PS 3 Construction	Mon 1/5/43	Mon 11/7/44	480 days
55	A-2.7.4	Detailed Design of PS 3 Operating Power Facilities + NTP	Mon 9/14/43	Mon 2/29/44	120 days
56	A-2.7.5	Construct PS 3 Operating Power Facilities + NTP	Mon 2/29/44	Mon 11/7/44	180 days
57	A-2.7.6	Construction Management	Mon 1/5/43	Mon 12/5/44	500.88 days?
58	**A-2.8**	**Tunnel 1 & 2 & 4 (MP 37.9 to MP 72.0) and (Wohlford Shaft to Lake Wohlford)**	**Wed 6/13/35**	**Mon 12/5/44**	**2473 days?**
59	A-2.8.1	Tunnel Design + Bid + NTP	Wed 6/13/35	Wed 5/12/38	760 days
60	**A-2.8.5**	**Excavation and Lining Tunnel 1 (MP 37.9 to MP 55.3)**	**Wed 5/12/38**	**Mon 11/7/44**	**1693 days**
67	**A-2.8.9**	**Excavation and Lining Tunnel 2 (MP 79.3 to MP 55.3)**	**Thu 10/28/38**	**Mon 11/7/44**	**1572 days**
75	**A-2.8.15**	**Excavation and Lining Tunnel 4 (Wohlford Shaft to Lake Wohlford)**	**Thu 3/13/42**	**Mon 11/7/44**	**692 days**
78	A-2.8.14	Construction Management	Wed 5/12/38	Mon 12/5/44	1713.88 days
79	**A-2.9**	**Vertical Shafts**	**Mon 7/2/35**	**Mon 12/5/44**	**2460 days?**
80	A-2.9.1	Vertical Shafts Detailed Design + Bid + NTP	Mon 7/2/35	Mon 5/31/38	760 days
81	**A-2.9.5**	**Construct Henshaw Shaft (MP 55.3, 1685 ft)**	**Mon 5/31/38**	**Fri 12/23/39**	**409 days**
85	**A-2.9.8**	**Construct Wohlford Shaft (MP 72.0, 480 ft)**	**Mon 5/31/38**	**Thu 8/18/39**	**318 days**
89	A-2.9.7	Construction Management	Mon 5/31/38	Mon 12/5/44	1700.88 days
90	**A-2.11**	**Lake Wohlford Operational Storage Improvements**	**Mon 2/6/40**	**Mon 12/5/44**	**1260 days?**
91	A-2.11.1	Lake Wohlford Detailed Design + Bid + NTP	Mon 2/6/40	Mon 1/5/43	760 days
92	A-2.11.2	Lake Wohlford Construction	Mon 1/5/43	Mon 11/7/44	480 days
93	A-2.11.3	Construction Management	Mon 1/5/43	Mon 12/5/44	500.88 days?
94	**A-2.12**	**Pipeline Reach 4 & 5 (MP 79.3 to MP 79.8) and (MP 82.0 to 82.3)**	**Mon 8/19/41**	**Sun 1/1/45**	**880 days?**
95	A-2.12.1	Pipeline Reach 4 & 5 Detailed Design + Bid + NTP	Mon 8/19/41	Mon 7/18/44	760 days



Project: MS Project Scheduling
Date: Wed 6/10/20

Task	Project Summary	Manual Task	Start-only	Deadline	Manual Progress
Split	Inactive Task	Duration-only	Finish-only	Critical	
Milestone	Inactive Milestone	Manual Summary Rollup	External Tasks	Critical Split	
Summary	Inactive Summary	Manual Summary	External Milestone	Progress	

ID	WBS	Task Name	Start	Finish	Duration
96	A-2.12.2	Pipeline Reach 4 & 5 Construction	Mon 7/18/44	Mon 12/5/44	100 days
97	A-2.12.3	Construction Management	Mon 7/18/44	Sun 1/1/45	120.88 days?
98	**A-2.13**	**Tunnel 3 (MP 79.9 to MP 82.0)**	**Fri 11/18/39**	**Sun 1/1/45**	**1336 days?**
99	A-2.13.1	Tunnel Detailed Design + Bid + NTP	Fri 11/18/39	Fri 10/17/42	760 days
100	**A-2.13.5**	**Tunnel 3 Excavation and Lining (MP 79.9 to MP 82.0)**	**Fri 10/17/42**	**Mon 12/5/44**	**556 days**
104	A-2.13.18	Construction Management	Mon 3/24/42	Sun 1/1/45	725.88 days?
105	**A-2.18**	**Aqueduct Improvements**	**Mon 7/23/40**	**Sun 1/1/45**	**1160 days?**
106	A-2.18.1	Aqueduct Improvements Detailed Design + Bid + NTP	Mon 7/23/40	Mon 6/22/43	760 days
107	A-2.18.2	40 MG Day Tank Construction	Mon 6/22/43	Mon 12/5/44	380 days
108	A-2.18.3	Construction Management	Mon 6/22/43	Sun 1/1/45	400.88 days?
109	**A-2.19**	**Warehouse + Storage Yard + Vehicles + Equipment**	**Mon 9/2/41**	**Sun 1/1/45**	**870 days?**
110	A-2.19.1	Warehouse + Storage Yard + Vehicles + Equipment Design + Bid	Mon 9/2/41	Mon 8/3/43	500 days
111	A-2.19.2	Warehouse + Storage Yard + Vehicles + Equipment Construction	Mon 8/3/43	Sun 1/1/45	370 days
112	A-2.19.3	Construction Management	Mon 8/3/43	Sun 1/1/45	370.88 days?
113	**A-2.100**	**3A Programmatic Commissioning**	**Mon 8/29/44**	**Sun 1/1/45**	**90 days**
114	**A-2.100.1**	**Phase 1 - East Side**	**Mon 8/29/44**	**Mon 11/7/44**	**50 days**
115	A-2.100.1.1	IID Op Storage Cx	Mon 10/24/44	Mon 11/7/44	10 days
116	A-2.100.1.2	Canal Cx	Mon 8/29/44	Mon 9/12/44	10 days
117	A-2.100.1.3	Salinity Treatment Plant Cx	Mon 9/12/44	Mon 11/7/44	40 days
118	A-2.100.1.4	PS 1 thru PS 2 Cx	Mon 10/10/44	Mon 11/7/44	20 days
119	**A-2.100.2**	**Phase 2 - PS 2 to Lake Wohlford**	**Mon 11/7/44**	**Mon 12/5/44**	**20 days**
120	A-2.100.2.1	PS 2 thru Lake Wohlford Cx	Mon 11/7/44	Mon 12/5/44	20 days
121	**A-2.100.3**	**Phase 3 - Lake Wohlford to TOVWTP**	**Mon 12/5/44**	**Sun 1/1/45**	**20 days**
122	A-2.100.3.1	Lake Wohlford to TOVWTP Cx	Mon 12/5/44	Sun 1/1/45	20 days



Project: MS Project Scheduling
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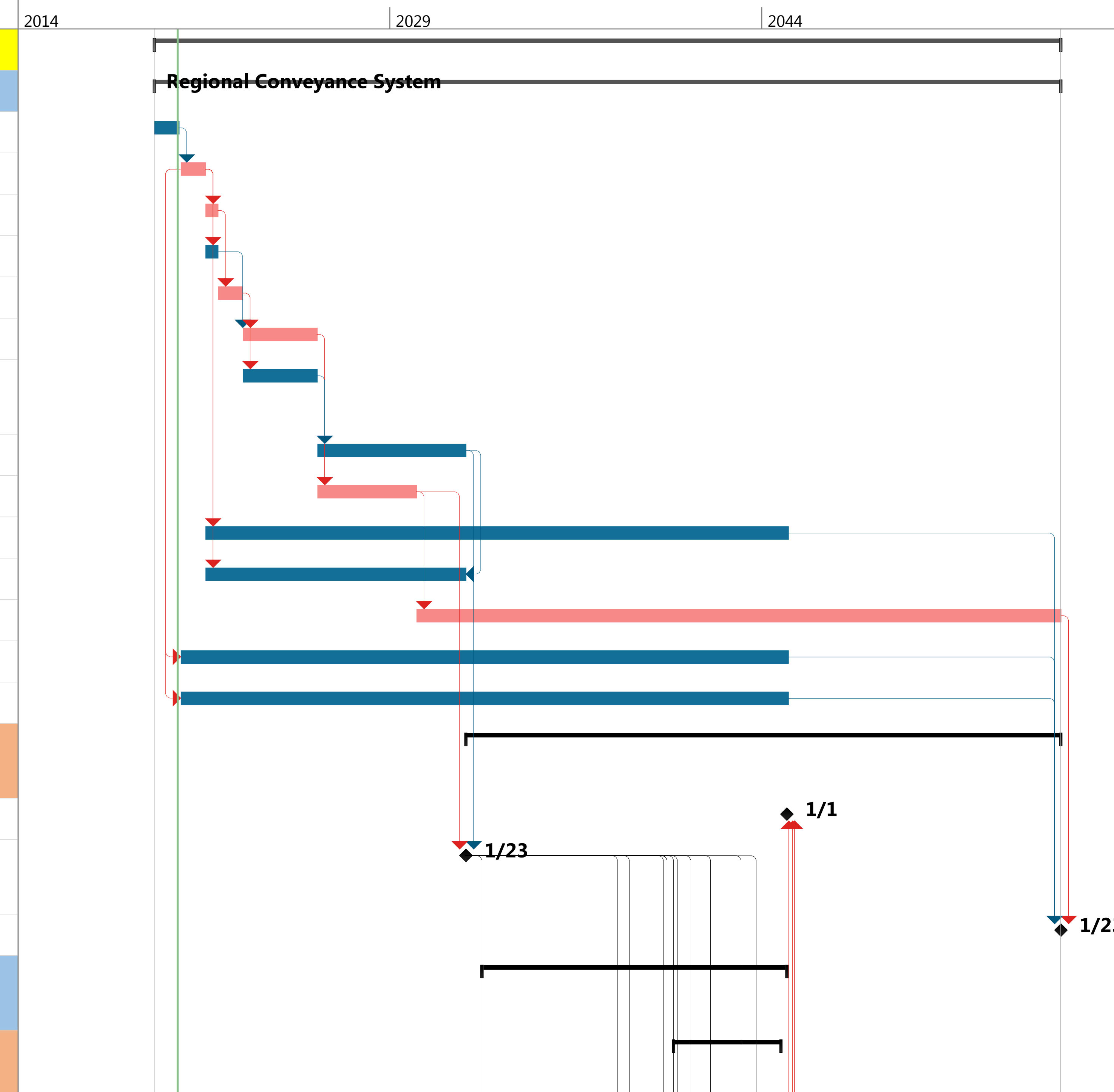
Task | Split | Milestone | Summary | Project Summary | Inactive Task | Inactive Milestone | Inactive Summary | Manual Task | Duration-only | Manual Summary Rollup | Manual Summary | Start-only | Finish-only | External Tasks | External Milestone | Deadline | Critical | Critical Split | Progress | Manual Progress

Alternative 5A- Schedule

SDCWA
RCS Study - Phase A
Alternative 5A

ID	WBS	Task Name	Start	Finish	Duration
1	**A**	**Regional Conveyance System**	**Mon 7/1/19**	**Sun 1/23/56**	**9541 days?**
2	**A-1**	**Initial Program Development**	**Mon 7/1/19**	**Sun 1/23/56**	**9541 days**
3	A-1.1	Phase A Planning Study	Mon 7/1/19	Tue 6/30/20	365 edays
4	A-1.2	Phase B Planning Study	Fri 7/24/20	Fri 7/23/21	364 edays
5	A-1.3	Procure Preliminary Designer	Sat 7/24/21	Sun 1/23/22	183 edays
6	A-1.4	Procure Env. Consultant	Sat 7/24/21	Sun 1/23/22	183 edays
7	A-1.5	Preliminary Design	Mon 1/24/22	Mon 1/23/23	364 edays
8	A-1.6	Environmental Work	Tue 1/24/23	Fri 1/23/26	1095 edays
9	A-1.7	Right-of-Way: Title, Appraisals, Phase 1, Survey, Acquisition, and Legal	Tue 1/24/23	Fri 1/23/26	1095 edays
10	A-1.8	Land Acquisition	Sat 1/24/26	Fri 1/23/32	2190 edays
11	A-1.9	CEQA / NEPA / Permits	Sat 1/24/26	Wed 1/23/30	1460 edays
12	A-1.10	Public Outreach	Sat 7/24/21	Mon 1/23/45	8584 edays
13	A-1.11	Legal Review	Sat 7/24/21	Fri 1/23/32	3835 edays
14	A-1.12	Environmental Mitigation/Monitoring	Thu 1/24/30	Sun 1/23/56	9495 edays
15	A-1.13	Owner's Representative/Programmatic Management	Fri 7/24/20	Mon 1/23/45	8949 edays
16	A-1.14	Staff Support - Programmatic	Fri 7/24/20	Mon 1/23/45	8949 edays
17	**A-101**	**Project Milestones for Multiple End Constraints**	**Fri 1/23/32**	**Sun 1/23/56**	**6262 days**
18	A-101.9	End of Construction Tasks	Sun 1/1/45	Sun 1/1/45	0 mons
19	A-101.10	Initial Program Substantially Complete to Allow Start of Projects	Fri 1/23/32	Fri 1/23/32	0 mons
20	A-101.11	Initial Program Complete	Sun 1/23/56	Sun 1/23/56	0 mons
21	**A-3**	**5A Alignment (Middle) Design, Construction, Commissioning**	**Fri 9/17/32**	**Mon 1/2/45**	**3207 days?**
22	**A-3.1**	**WMC Parallel Conveyance Reach 1 (Canal MP 0 to MP -12)**	**Tue 6/12/40**	**Mon 10/10/44**	**1130 days?**



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Task
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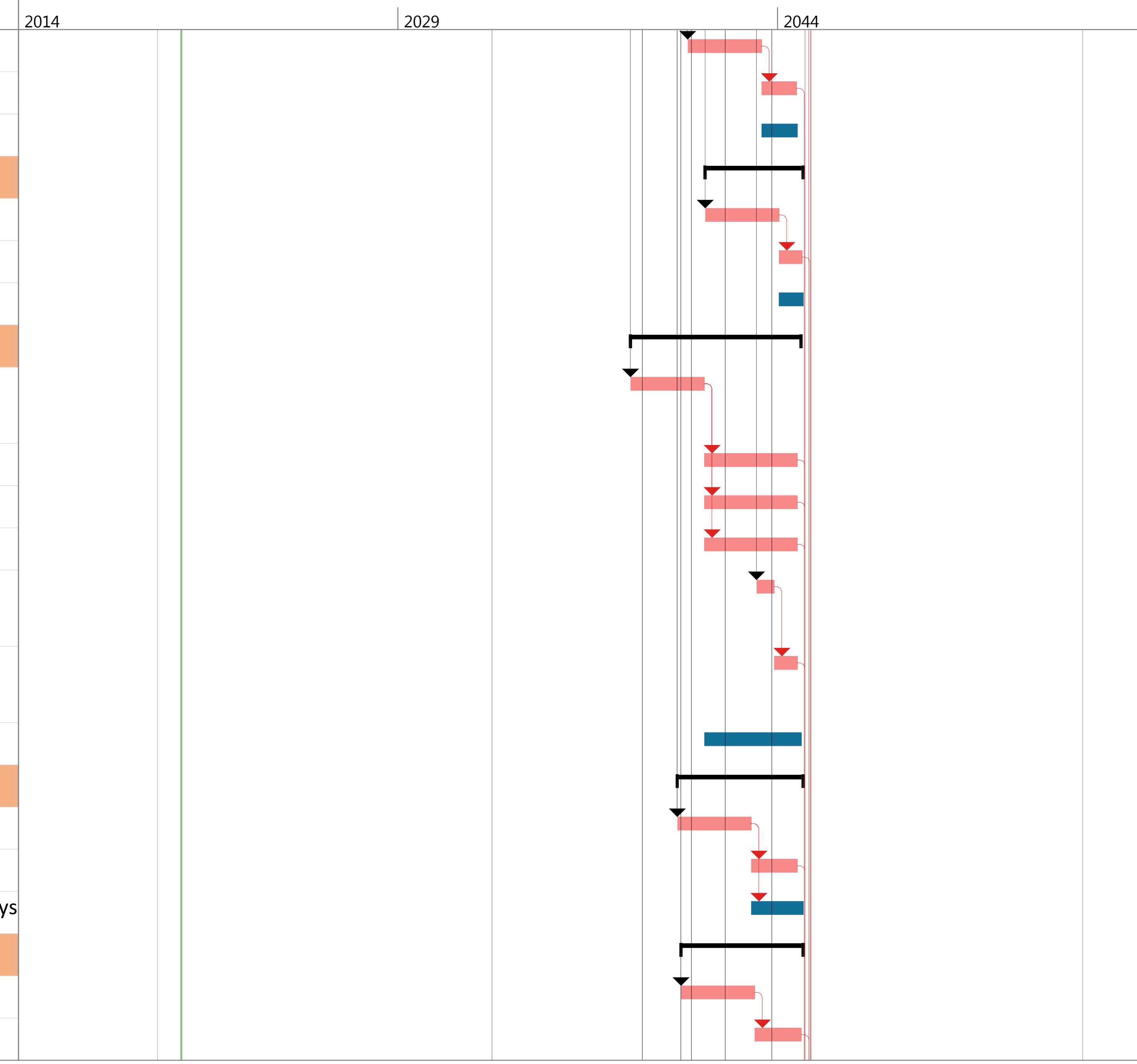
Duration-only
Manual Summary Rollup
Manual Summary
Start-only
Finish-only
External Tasks
External Milestone
Deadline
Critical
Critical Split
Progress
Manual Progress

ID	WBS	Task Name	Start	Finish	Duration
23	A-3.1.1	Parallel Conveyance Design + Bid + NTP	Tue 6/12/40	Mon 5/11/43	760 days
24	A-3.1.2	Parallel Conveyance Construction	Tue 5/12/43	Mon 9/26/44	360 days
25	A-3.1.3	Construction Management	Mon 5/11/43	Mon 10/10/44	371 days?
26	**A-3.2**	**Imperial Valley Operational Storage (900 AF)**	**Tue 2/19/41**	**Sun 1/1/45**	**1010 days?**
27	A-3.2.1	Joint Op Storage Design + Bid + NTP	Tue 2/19/41	Mon 1/18/44	760 days
28	A-3.2.2	Joint Op Storage Construction	Tue 1/19/44	Mon 12/19/44	240 days
29	A-3.2.3	Construction Management	Mon 1/18/44	Sun 1/1/45	251 days?
30	**A-3.3**	**Salinity Treatment Plant + PS 1 + Forebay**	**Tue 3/9/38**	**Mon 12/5/44**	**1760 days?**
31	A-3.3.1	Salinity Treatment + PS 1 + Forebay Design + Bid + NTP	Tue 3/9/38	Mon 2/4/41	760 days
32	A-3.3.2	Salinity Treatment Construction	Tue 2/5/41	Mon 10/10/44	960 days
33	A-3.3.3	Brine Construction	Tue 2/5/41	Mon 10/10/44	960 days
34	A-3.3.4	PS1 + Forebay Construction	Tue 2/5/41	Mon 10/10/44	960 days
35	A-3.3.5	Detailed Design of Salinity Treatment and PS1 Operating Power Facilities + NTP	Tue 3/3/43	Mon 11/9/43	180 days
36	A-3.3.6	Construct Salinity Treatment and PS 1 Operating Power Facilities	Tue 11/10/43	Mon 10/10/44	240 days
37	A-3.3.7	Construction Management	Mon 2/4/41	Mon 12/5/44	1001 days?
38	**A-3.4**	**Pipeline Reach 2 (MP 0 to MP 15.3)**	**Fri 1/13/40**	**Mon 1/2/45**	**1297 days**
39	A-3.4.1	Pipeline Reach 2 Design + Bid + NTP	Fri 1/13/40	Thu 12/11/42	760 days
40	A-3.4.2	Pipeline Reach 2 Construction	Fri 12/12/42	Tue 10/11/44	669 edays
41	A-3.4.3	Construction Management	Thu 12/11/42	Mon 1/2/45	752.38 edays
42	**A-3.5**	**Pump Station 2 (MP 15.3)**	**Tue 3/6/40**	**Sun 1/1/45**	**1260 days?**
43	A-3.5.1	PS 2 Design + Bid + NTP	Tue 3/6/40	Mon 2/2/43	760 days
44	A-3.5.3	PS 2 Construction	Tue 2/3/43	Mon 12/5/44	480 days



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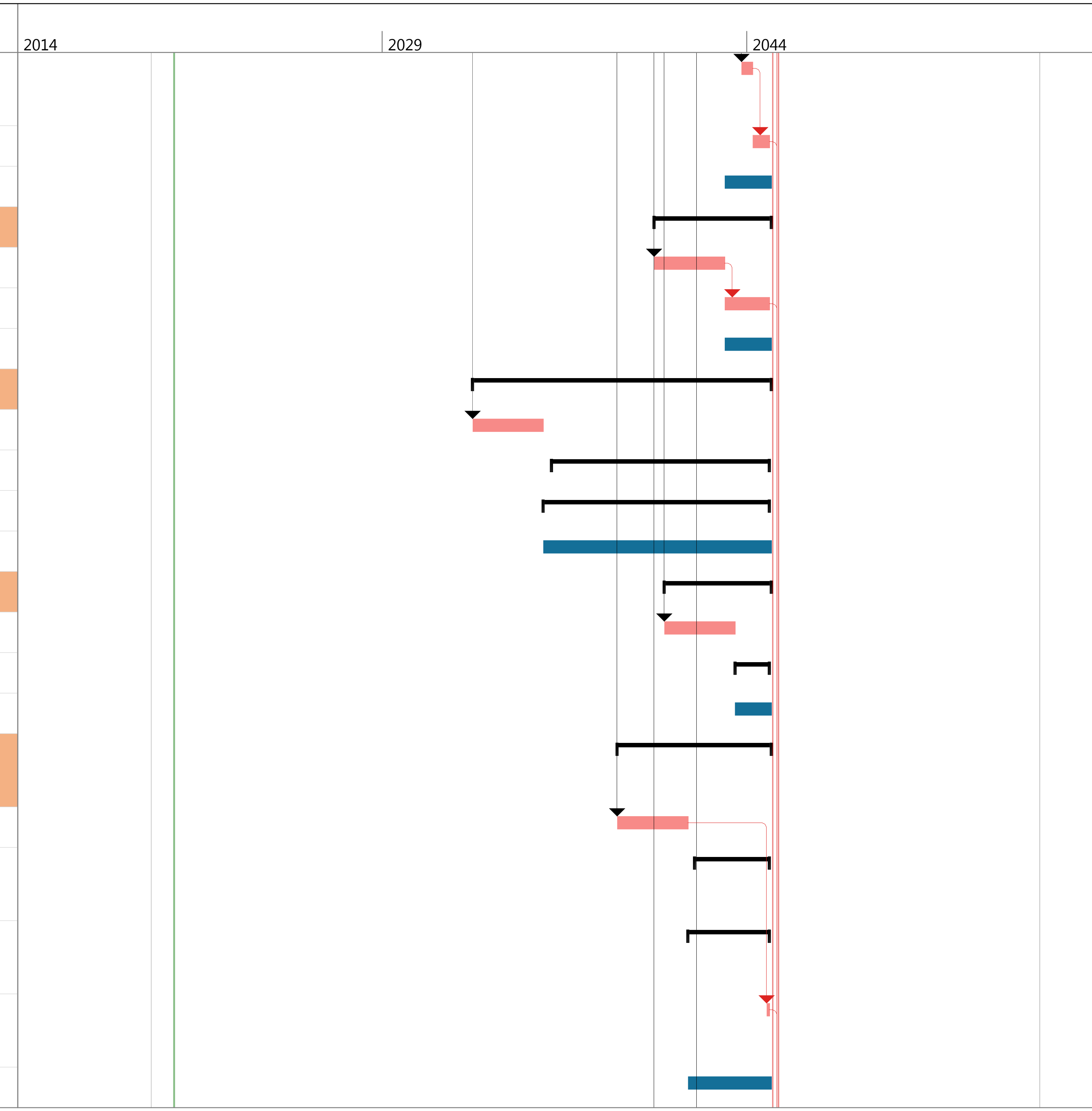
Task		Inactive Task		Manual Summary Rollup		External Milestone		Manual Progress	
Split		Inactive Milestone		Manual Summary		Deadline			
Milestone		Inactive Summary		Start-only		Critical			
Summary		Manual Task		Finish-only		Critical Split			
Project Summary		Duration-only		External Tasks		Progress			

ID	WBS	Task Name	Start	Finish	Duration
45	A-3.5.4	Detailed Design of PS 2 Operating Power Facilities + NTP	Tue 10/13/43	Mon 3/28/44	120 days
46	A-3.5.5	Construct PS 2 Operating Power Facilities	Tue 3/29/44	Mon 12/5/44	180 days
47	A-3.5.6	Construction Management	Mon 2/2/43	Sun 1/1/45	501 days?
48	**A-3.6**	**Pipeline Reach 3 (MP 15.3 to MP 29.5)**	**Tue 3/6/40**	**Sun 1/1/45**	**1260 days?**
49	A-3.6.1	Pipeline Reach 3 Design + Bid + NTP	Tue 3/6/40	Mon 2/2/43	760 days
50	A-3.6.2	Pipeline Reach 3 Construction	Tue 2/3/43	Mon 12/5/44	480 days
51	A-3.6.3	Construction Management	Mon 2/2/43	Sun 1/1/45	501 days?
52	**A-3.7**	**Tunnel 1 & 2 (MP 29.5 to MP 63.6)**	**Fri 9/17/32**	**Sun 1/1/45**	**3207 days?**
53	A-3.7.1	Tunnel 1 & 2 Design + Bid + NTP	Fri 9/17/32	Thu 8/16/35	760 days
54	**A-3.7.5**	**Tunnel 1 Excavation (MP 29.5 to MP 46.2)**	**Tue 12/18/35**	**Mon 12/5/44**	**2340 days**
58	**A-3.7.9**	**Tunnel 2 Excavation (MP 63.6 to MP 46.2)**	**Fri 8/17/35**	**Mon 12/5/44**	**2427 days**
62	A-3.7.14	Construction Management	Thu 8/16/35	Sun 1/1/45	2448 days?
63	**A-3.8**	**Vertical Shaft**	**Tue 8/7/40**	**Sun 1/1/45**	**1150 days?**
64	A-3.8.1	Vertical Shaft Design + Bid + NTP	Tue 8/7/40	Mon 7/6/43	760 days
65	**A-3.8.5**	**Vertical Shaft Excavation and Lining**	**Tue 7/7/43**	**Mon 12/5/44**	**370 days**
68	A-3.8.7	Construction Management	Mon 7/6/43	Sun 1/1/45	391 days?
69	**A-3.10**	**Tunnel 3 & 4 (MP 63.6 to MP 70.8) and Reach 5 (MP 66.8 to MP 67.2)**	**Wed 9/1/38**	**Sun 1/1/45**	**1654 days?**
70	A-3.10.1	Tunnel 3 & 4 & Reach 5 Design + Bid + NTP	Wed 9/1/38	Tue 7/30/41	760 days
71	**A-3.10.5**	**Tunnel 3 Excavation and Lining (MP 63.6 to MP 66.8)**	**Wed 11/6/41**	**Mon 12/5/44**	**804 days**
75	**A-3.10.10**	**Tunnel 4 Excavation and Lining (MP 67.2 to MP 70.8)**	**Wed 7/31/41**	**Mon 12/5/44**	**874 days**
79	A-3.10.14	Pipeline Reach 5 Construction (MP 66.8 to MP 67.2)	Tue 10/25/44	Mon 12/5/44	30 days
80	A-3.10.13	Construction Management	Tue 7/30/41	Sun 1/1/45	895 days?



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Task
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Manual Summary Rollup
Manual Summary
Start-only
Finish-only
External Tasks
External Milestone
Deadline
Critical
Critical Split
Progress
Manual Progress

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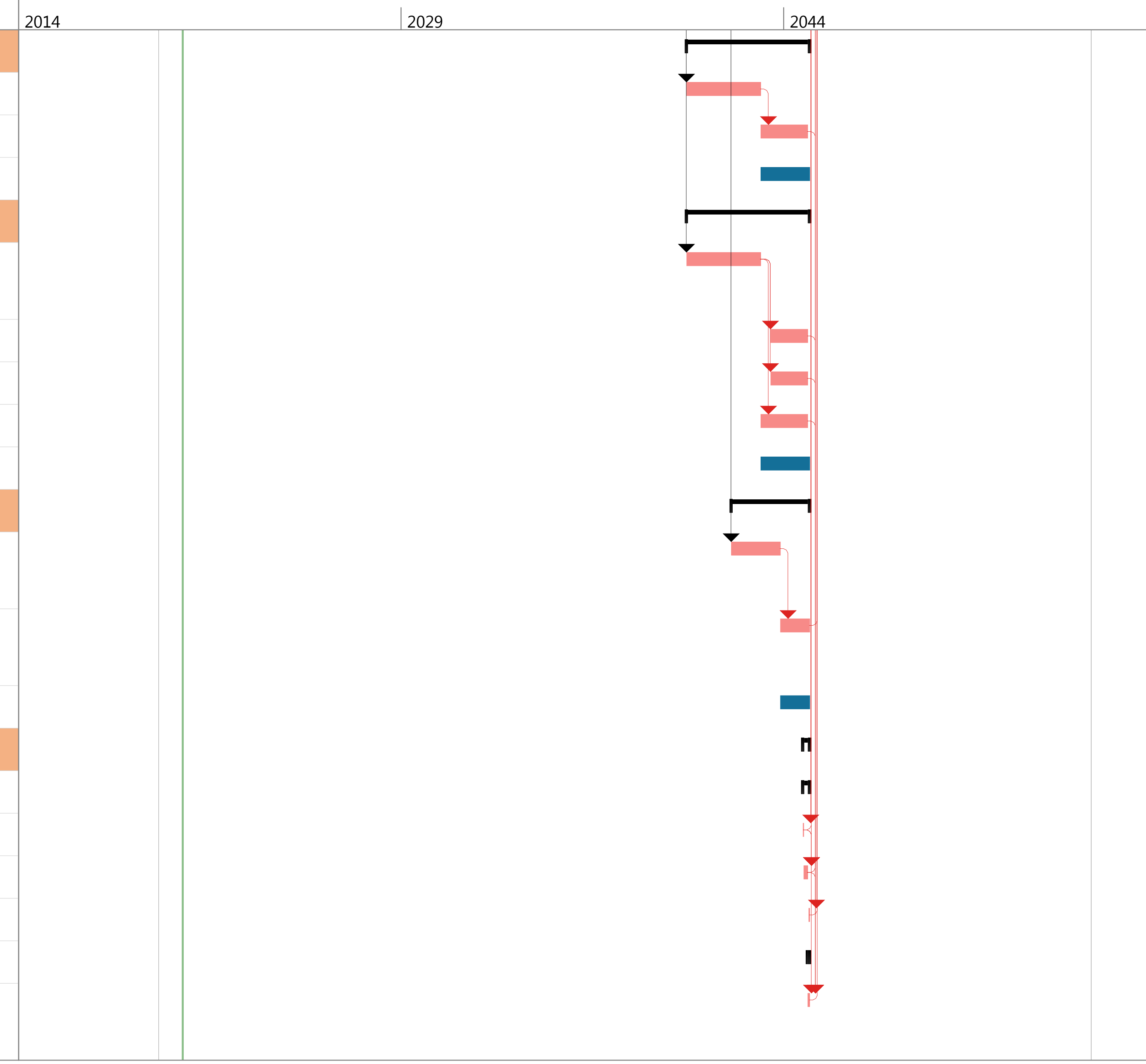
RCS Study - Phase A
Alternative 5A

ID	WBS	Task Name	Start	Finish	Duration
81	**A-3.12**	**Pressure Control Facility (MP 70.8)**	**Tue 3/6/40**	**Sun 1/1/45**	**1260 days?**
82	A-3.12.1	PCF Design + Bid + NTP	Tue 3/6/40	Mon 2/2/43	760 days
83	A-3.12.2	PCF Construction	Tue 2/3/43	Mon 12/5/44	480 days
84	A-3.12.3	Construction Management	Mon 2/2/43	Sun 1/1/45	501 days?
85	**A-3.15**	**Aqueduct Improvements**	**Tue 3/6/40**	**Sun 1/1/45**	**1260 days?**
86	A-3.15.1	Aqueduct Improvements Detailed Design + Bid + NTP	Tue 3/6/40	Mon 2/2/43	760 days
87	A-3.15.2	40 MG Day Tank Construction	Tue 6/23/43	Mon 12/5/44	380 days
88	A-3.15.3	Pipeline Construction (12.5 miles)	Tue 6/23/43	Mon 12/5/44	380 days
89	A-3.15.4	Pump Station	Tue 2/3/43	Mon 12/5/44	480 days
90	A-3.15.5	Construction Management	Mon 2/2/43	Sun 1/1/45	501 days?
91	**A-16**	**Warehouse + Storage Yard + Vehicles + Equipment**	**Tue 12/10/41**	**Sun 1/1/45**	**800 days?**
92	A-16.1	Warehouse + Storage Yard + Vehicles + Equipment Design + Bid	Tue 12/10/41	Mon 11/9/43	500 days
93	A-16.2	Warehouse + Storage Yard + Vehicles + Equipment Construction	Tue 11/10/43	Sun 1/1/45	300 days
94	A-16.3	Construction Management	Mon 11/9/43	Sun 1/1/45	301 days?
95	**A-17**	**5A Program Commissioning**	**Tue 9/27/44**	**Sun 1/1/45**	**70 days**
96	**A-17.1**	**Phase 1 - East Side IID**	**Tue 9/27/44**	**Sun 1/1/45**	**70 days**
97	A-17.1.3	Canal Cx	Tue 9/27/44	Mon 10/10/44	10 days
98	A-17.1.1	Salinity Treatment Plant Cx	Tue 10/11/44	Mon 12/5/44	40 days
99	A-17.1.2	IID Operational Storage Cx	Tue 12/20/44	Sun 1/1/45	10 days
100	**A-17.2**	**Phase 2 - Mountain to Reservoir**	**Tue 12/6/44**	**Sun 1/1/45**	**20 days**
101	A-17.2.1	Reach 2 - 5, PS2, PCF Cx, TOVWTP, Aqueduct Improvements	Tue 12/6/44	Sun 1/1/45	20 days



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Task
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Duration-only
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Start-only
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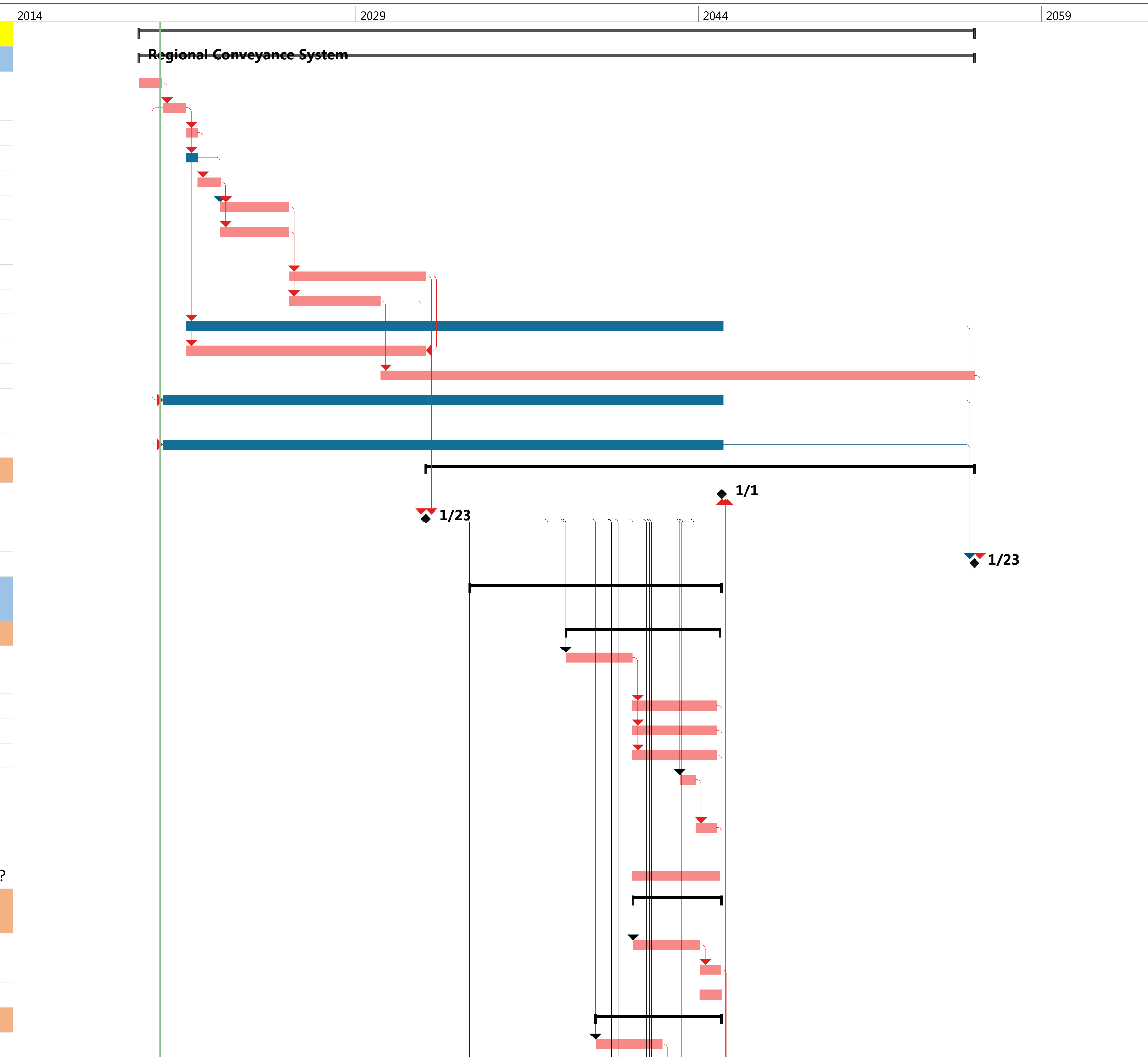
External Tasks
External Milestone
Deadline
Critical
Critical Split
Progress
Manual Progress

Alternative 5C - Schedule

SDCWA
RCS Study - Phase A
Alternative 5C

ID	WBS	Task Name	Start	Finish	Duration
1	**A**	**Regional Conveyance System**	**Mon 7/1/19**	**Sun 1/23/56**	**9540 days?**
2	**A-1**	**Initial Program Development**	**Mon 7/1/19**	**Sun 1/23/56**	**9540 days**
3	A-1.1	Phase A Planning Study	Mon 7/1/19	Tue 6/30/20	365 edays
4	A-1.2	Phase B Planning Study	Fri 7/24/20	Fri 7/23/21	364 edays
5	A-1.3	Procure Preliminary Designer	Sat 7/24/21	Sun 1/23/22	183 edays
6	A-1.4	Procure Env. Consultant	Sat 7/24/21	Sun 1/23/22	183 edays
7	A-1.5	Preliminary Design	Mon 1/24/22	Mon 1/23/23	364 edays
8	A-1.6	Environmental Work	Tue 1/24/23	Fri 1/23/26	1095 edays
9	A-1.7	Right-of-Way: Title, Appraisals, Phase 1, Survey, Acquisition, and Legal	Tue 1/24/23	Fri 1/23/26	1095 edays
10	A-1.8	Land Acquisition	Sat 1/24/26	Fri 1/23/32	2190 edays
11	A-1.9	CEQA / NEPA / Permits	Sat 1/24/26	Wed 1/23/30	1460 edays
12	A-1.10	Public Outreach	Sat 7/24/21	Mon 1/23/45	8584 edays
13	A-1.11	Legal Review	Sat 7/24/21	Fri 1/23/32	3835 edays
14	A-1.12	Environmental Mitigation/Monitoring	Thu 1/24/30	Sun 1/23/56	9495 edays
15	A-1.13	Owner's Representative/Programmatic Management	Fri 7/24/20	Mon 1/23/45	8949 edays
16	A-1.14	Staff Support - Programmatic	Fri 7/24/20	Mon 1/23/45	8949 edays
17	**A-1.100**	**Project Milestones for Multiple End Constraints**	**Fri 1/23/32**	**Sun 1/23/56**	**6261 days**
18	A-1.100.1	End of Construction Tasks	Sun 1/1/45	Sun 1/1/45	0 days
19	A-1.100.2	Initial Program Substantially Complete to Allow Start of Projects	Fri 1/23/32	Fri 1/23/32	0 days
20	A-1.100.3	Initial Program Complete	Sun 1/23/56	Sun 1/23/56	0 days
21	**A-4**	**5C Alignment (South) Design, Construction, Commissioning**	**Mon 12/26/33**	**Fri 12/30/44**	**2875 days?**
22	**A-4.1**	**Salinity Treatment Plant + PS 1 + Forebay**	**Mon 3/8/38**	**Fri 12/2/44**	**1760 days?**
23	A-4.1.1	Salinity Treatment + PS 1 + Forebay Design + Bid + NTP	Mon 3/8/38	Fri 2/1/41	760 days
24	A-4.1.2	Salinity Treatment Construction	Mon 2/4/41	Fri 10/7/44	960 days
25	A-4.1.3	Brine Construction	Mon 2/4/41	Fri 10/7/44	960 days
26	A-4.1.4	PS1 + Forebay Construction	Mon 2/4/41	Fri 10/7/44	960 days
27	A-4.1.5	Salinity Treatment + PS 1 Power Facilities Design + Bid + NTP	Mon 3/2/43	Fri 11/6/43	180 days
28	A-4.1.6	Salinity Treatment + PS 1 Power Facilities Construction	Mon 11/9/43	Fri 10/7/44	240 days
29	A-4.1.7	Construction Management	Fri 2/1/41	Fri 12/2/44	1001 days?
30	**A-4.2**	**Imperial Valley Operational Storage (900 AF)**	**Mon 2/18/41**	**Fri 12/30/44**	**1010 days?**
31	A-4.2.1	Joint Op Storage Design + Bid + NTP	Mon 2/18/41	Fri 1/15/44	760 days
32	A-4.2.2	Joint Op Storage Construction	Mon 1/18/44	Fri 12/16/44	240 days
33	A-4.2.3	Construction Management	Fri 1/15/44	Fri 12/30/44	251 days?
34	**A-4.3**	**Pipeline Reach 1 (MP 0 to MP 21.5)**	**Mon 6/27/39**	**Fri 12/30/44**	**1440 days?**
35	A-4.3.1	Pipeline Reach 1 Design + Bid + NTP	Mon 6/27/39	Fri 5/23/42	760 days

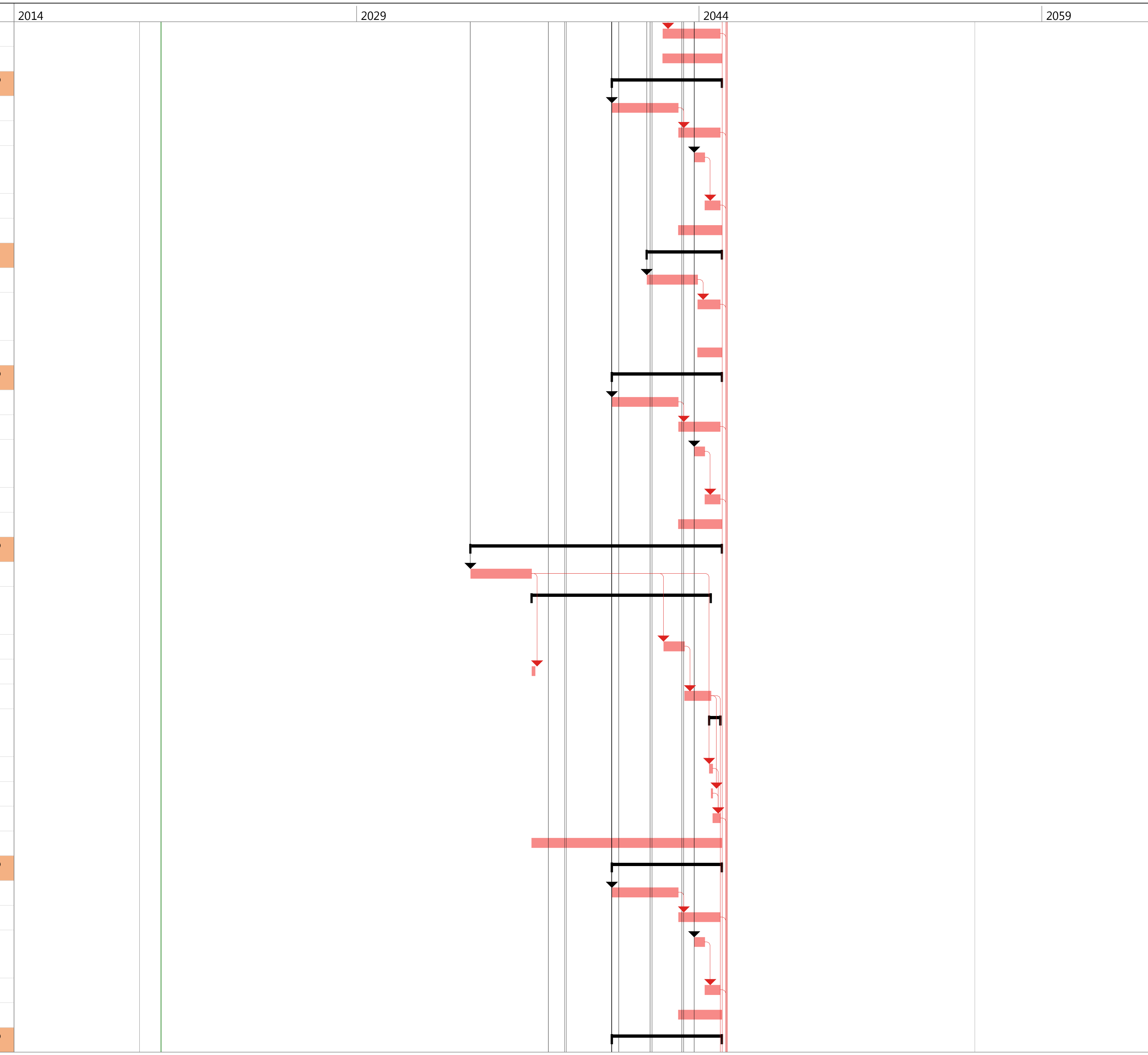


Project: MS Project Scheduling
Date: Thu 6/11/20

Task · Split · Milestone · Summary · Project Summary · Inactive Task · Inactive Milestone · Inactive Summary · Manual Task · Duration-only · Manual Summary Rollup · Manual Summary · Start-only · Finish-only · External Tasks · External Milestone · Deadline · Critical · Critical Split · Progress · Manual Progress

SDCWA
RCS Study - Phase A
Alternative 5C

ID	WBS	Task Name	Start	Finish	Duration
36	A-4.3.2	Pipeline Reach 1 Construction	Mon 5/26/42	Fri 12/2/44	660 days
37	A-4.3.3	Construction Management	Fri 5/23/42	Fri 12/30/44	681 days?
38	**A-4.4**	**Pump Station 2 (MP 21.5)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
39	A-4.4.1	PS 2 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
40	A-4.4.3	PS 2 Construction	Mon 2/2/43	Fri 12/2/44	480 days
41	A-4.4.4	PS 2 Operating Power Facilities Design + Bid + NTP	Mon 10/12/43	Fri 3/25/44	120 days
42	A-4.4.5	PS 2 Operating Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
43	A-4.4.6	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
44	**A-4.5**	**Pipeline Reach 2 (MP 21.5 to 25.3)**	**Mon 9/16/41**	**Fri 12/30/44**	**860 days?**
45	A-4.5.1	Pipeline Reach 2 Design + Bid + NTP	Mon 9/16/41	Fri 12/4/43	580 days
46	A-4.5.2	Pipeline Reach 2 PS 2 to PS 3 Construction (MP 21.5 to 25.3)	Mon 12/7/43	Fri 12/2/44	260 days
47	A-4.5.5	Construction Management	Fri 12/4/43	Fri 12/30/44	281 days?
48	**A-4.6**	**Pump Station 3 (MP 25.3)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
49	A-4.6.1	PS 3 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
50	A-4.6.3	PS 3 Construction	Mon 2/2/43	Fri 12/2/44	480 days
51	A-4.6.4	PS 3 Operating Power Facilities Design + Bid + NTP	Mon 10/12/43	Fri 3/25/44	120 days
52	A-4.6.5	PS 3 Operating Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
53	A-4.6.6	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
54	**A-4.7**	**Tunnel 1 & 2 (MP 25.1 to MP 29.6)**	**Mon 12/26/33**	**Fri 12/30/44**	**2875 days?**
55	A-4.7.1	Tunnel 1 & 2 Design + Bid + NTP	Mon 12/26/33	Fri 8/29/36	700 days
56	**A-4.7.5**	**Tunnel 1 Excavation and Lining (MP 25.1 to MP 27.7)**	**Mon 9/1/36**	**Tue 7/5/44**	**2047 days**
57	A-4.7.5.1	Procure TBM	Mon 6/9/42	Fri 5/8/43	240 days
58	A-4.7.5.2	Establish Portal Site	Mon 9/1/36	Fri 10/24/36	40 days
59	A-4.7.5.3	Tunnel 1 (eastbound) Excavation and Lining	Sun 5/10/43	Tue 7/5/44	423 days
60	**A-4.7.10**	**Tunnel 2 Excavation and Lining (MP 28.9 to MP 29.6)**	**Wed 6/8/44**	**Sun 12/4/44**	**128 days**
61	A-4.7.10.1	Establish Portal Site	Wed 6/8/44	Tue 8/2/44	40 days
62	A-4.7.10.2	Re-fit TBM	Wed 7/6/44	Tue 8/2/44	20 days
63	A-4.7.10.3	Tunnel 2 (eastbound) Excavation and Lining	Wed 8/3/44	Sun 12/4/44	124 days
64	A-4.7.13	Construction Management	Fri 8/29/36	Fri 12/30/44	2176 days?
65	**A-4.8**	**Pump Station 4 (MP 27.7)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
66	A-4.8.1	PS 4 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
67	A-4.8.3	PS 4 Construction	Mon 2/2/43	Fri 12/2/44	480 days
68	A-4.8.4	PS 4 Operating Power Facilities Design + Bid + NTP	Mon 10/12/43	Fri 3/25/44	120 days
69	A-4.8.5	PS 4 Operating Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
70	A-4.8.6	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
71	**A-4.9**	**Pump Station 5 (MP 29.6)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**



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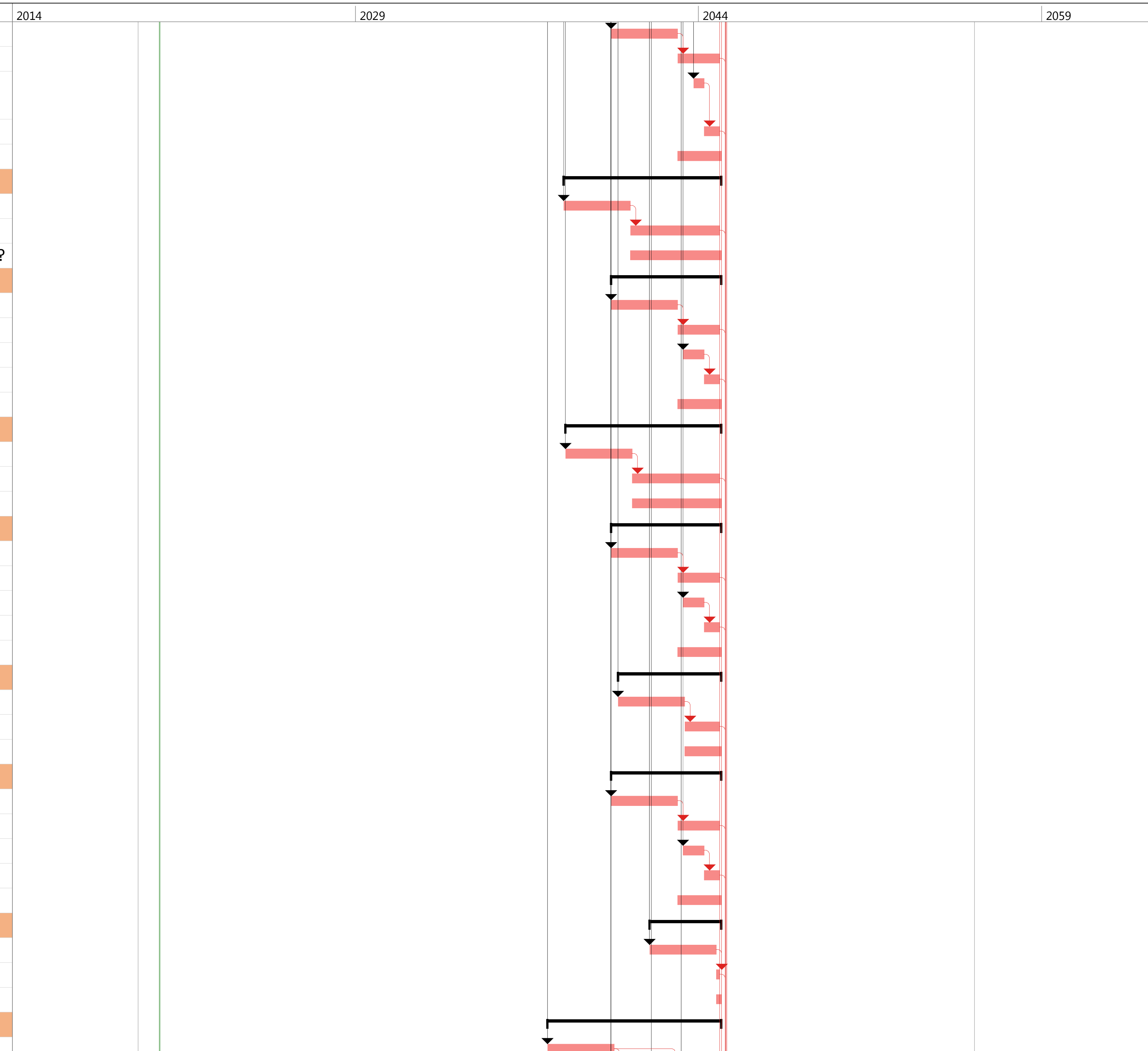
ID	WBS	Task Name	Start	Finish	Duration
72	A-4.9.1	PS 5 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
73	A-4.9.3	PS 5 Construction	Mon 2/2/43	Fri 12/2/44	480 days
74	A-4.9.4	PS 5 Operating Power Facilities Design + Bid + NTP	Mon 10/12/43	Fri 3/25/44	120 days
75	A-4.9.5	PS 5 Operating Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
76	A-4.9.6	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
77	**A-4.10**	**Pipeline Reach 3 (MP 29.6 to MP 55.0)**	**Mon 2/8/38**	**Fri 12/30/44**	**1800 days?**
78	A-4.10.1	Pipeline Reach 3 Design + Bid + NTP	Mon 2/8/38	Fri 1/4/41	760 days
79	A-4.10.2	Pipeline Reach 3 Construction	Mon 1/7/41	Fri 12/2/44	1020 days
80	A-4.10.3	Construction Management	Fri 1/4/41	Fri 12/30/44	1041 days?
81	**A-4.11**	**Power Generating Facility 1 (MP 55)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
82	A-4.11.1	PGF 1 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
83	A-4.11.2	PGF 1 Construction	Mon 2/2/43	Fri 12/2/44	480 days
84	A-4.11.3	PGF 1 Power Facilities Design + Bid + NTP	Mon 4/27/43	Fri 3/25/44	240 days
85	A-4.11.4	PGF 1 Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
86	A-4.11.5	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
87	**A-4.12**	**Pipeline Reach 4 (MP 55.0 to MP 79.8)**	**Mon 3/8/38**	**Fri 12/30/44**	**1780 days?**
88	A-4.12.1	Pipeline Reach 4 Design + Bid + NTP	Mon 3/8/38	Fri 2/1/41	760 days
89	A-4.12.2	Pipeline Reach 4 Construction	Mon 2/4/41	Fri 12/2/44	1000 days
90	A-4.12.3	Construction Management	Fri 2/1/41	Fri 12/30/44	1021 days?
91	**A-4.13**	**Power Generating Facility 2**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
92	A-4.13.1	PGF 2 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
93	A-4.13.2	PGF 2 Construction	Mon 2/2/43	Fri 12/2/44	480 days
94	A-4.13.3	PGF 2 Power Facilities Design + Bid + NTP	Mon 4/27/43	Fri 3/25/44	240 days
95	A-4.13.4	PGF 2 Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
96	A-4.13.5	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
97	**A-4.14**	**Pipeline Reach 5 (MP 79.8 to MP 85.7)**	**Mon 6/25/40**	**Fri 12/30/44**	**1180 days?**
98	A-4.14.1	Pipeline Reach 5 Design + Bid + NTP	Mon 6/25/40	Fri 5/22/43	760 days
99	A-4.14.2	Pipeline Reach 5 Construction	Mon 5/25/43	Fri 12/2/44	400 days
100	A-4.14.3	Construction Management	Fri 5/22/43	Fri 12/30/44	421 days?
101	**A-4.15**	**Power Generating Facility 3**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
102	A-4.15.1	PGF 3 Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
103	A-4.15.2	PGF 3 Construction	Mon 2/2/43	Fri 12/2/44	480 days
104	A-4.15.3	PGF 3 Power Facilities Design + Bid + NTP	Mon 4/27/43	Fri 3/25/44	240 days
105	A-4.15.4	PGF 3 Power Facilities Construction	Mon 3/28/44	Fri 12/2/44	180 days
106	A-4.15.5	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
107	**A-4.16**	**Pipeline Reach 6 (MP 85.7 to MP 93.2)**	**Mon 11/11/41**	**Fri 12/30/44**	**820 days?**
108	A-4.16.1	Pipeline Reach 6 Design + Bid + NTP	Mon 11/11/41	Fri 10/7/44	760 days
109	A-4.16.2	Pipeline Reach 6 Construction	Mon 10/10/44	Fri 12/2/44	40 days
110	A-4.16.3	Construction Management	Fri 10/7/44	Fri 12/30/44	61 days?
111	**A-4.17**	**Tunnel 3 & 4 (MP 85.7 to MP 93.2)**	**Mon 5/25/37**	**Fri 12/30/44**	**1985 days?**
112	A-4.17.1	Tunnel 3 & 4 Design + Bid + NTP	Mon 5/25/37	Fri 4/20/40	760 days



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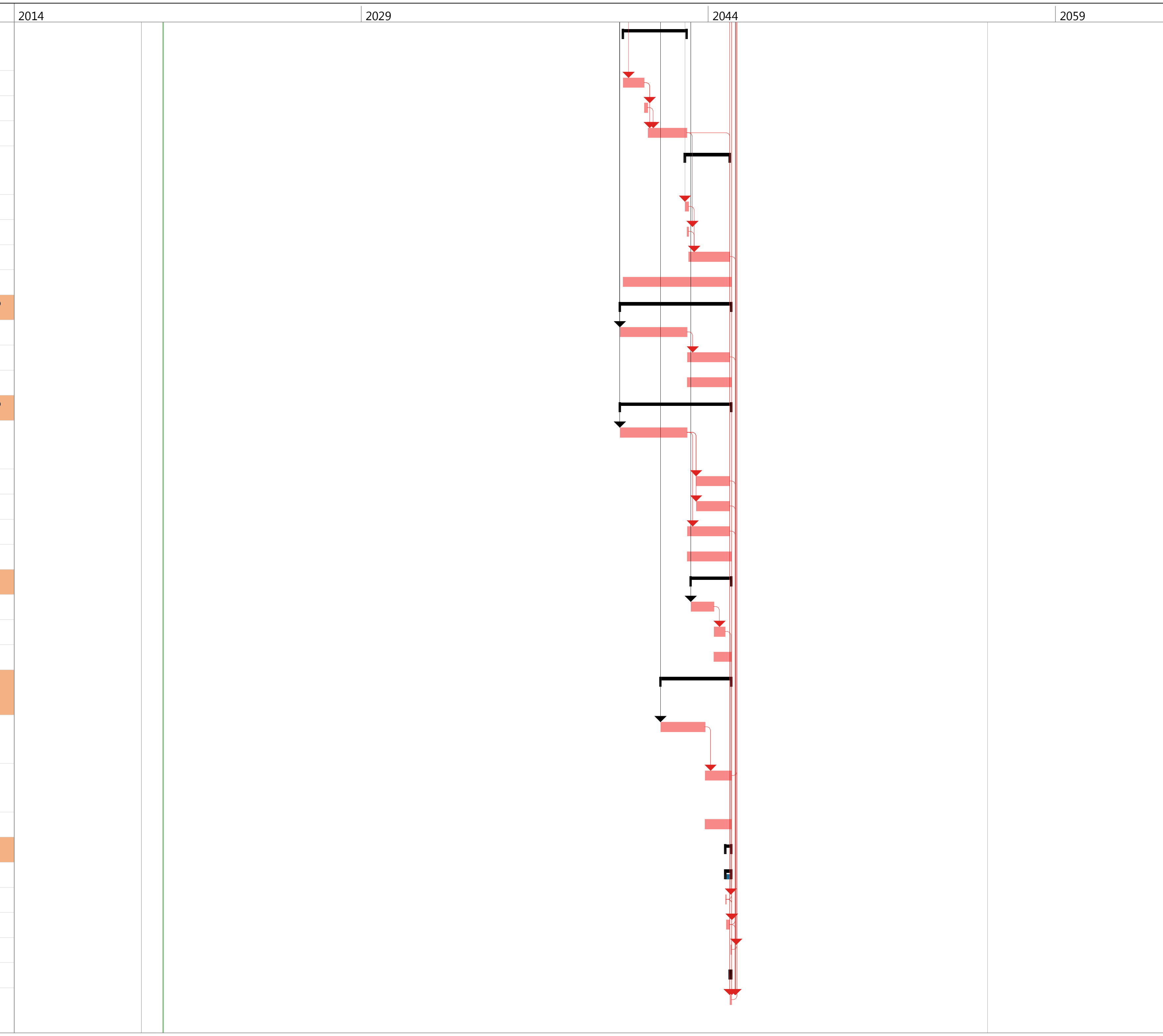
ID	WBS	Task Name	Start	Finish	Duration
113	**A-4.17.5**	**Tunnel 3 Excavation and Lining (MP 85.7 to MP 89.2)**	**Mon 4/23/40**	**Sun 1/25/43**	**720 days**
114	A-4.17.5.1	Procure TBM	Mon 4/23/40	Fri 3/22/41	240 days
115	A-4.17.5.2	Prepare Portal Site	Mon 3/25/41	Fri 5/17/41	40 days
116	A-4.17.5.3	Tunnel 3 (eastbound) Excavation and Lining	Mon 5/20/41	Sun 1/25/43	616 days
117	**A-4.17.9**	**Tunnel 4 Excavation and Lining (MP 89.5 to MP 93.2)**	**Mon 12/29/42**	**Sun 12/4/44**	**505 days**
118	A-4.17.9.1	Establish Portal Site	Mon 12/29/42	Fri 2/20/43	40 days
119	A-4.17.9.2	Re-fit TBM from Tunnel 3	Mon 1/26/43	Fri 2/20/43	20 days
120	A-4.17.9.3	Tunnel 4 (eastbound) Excavation and Lining	Sun 2/22/43	Sun 12/4/44	652 days
121	A-4.17.13	Construction Management	Fri 4/20/40	Fri 12/30/44	1226 days?
122	**A-4.18**	**Pressure Control Facility (MP 93.2)**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
123	A-4.18.1	PCF Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
124	A-4.18.2	PCF Construction	Mon 2/2/43	Fri 12/2/44	480 days
125	A-4.18.3	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
126	**A-4.21**	**Aqueduct Improvements**	**Mon 3/5/40**	**Fri 12/30/44**	**1260 days?**
127	A-4.21.1	Aqueduct Improvements Detailed Design + Bid + NTP	Mon 3/5/40	Fri 1/30/43	760 days
128	A-4.21.2	40 MG Day Tank Construction	Mon 6/22/43	Fri 12/2/44	380 days
129	A-4.21.3	Pipeline Construction (12.5 miles)	Mon 6/22/43	Fri 12/2/44	380 days
130	A-4.21.4	Pump Station	Mon 2/2/43	Fri 12/2/44	480 days
131	A-4.21.5	Construction Management	Fri 1/30/43	Fri 12/30/44	501 days?
132	**A-4.22**	**Canal Connection**	**Mon 3/30/43**	**Fri 12/30/44**	**460 days?**
133	A-4.22.1	Canal Connection Detailed Design + Bid + NTP	Mon 3/30/43	Fri 3/25/44	260 days
134	A-4.22.2	Canal Construction	Mon 3/28/44	Fri 9/23/44	130 days
135	A-4.22.3	Construction Management	Fri 3/25/44	Fri 12/30/44	201 days?
136	**A-4.23**	**Warehouse + Storage Yard + Vehicles + Equipment**	**Mon 12/9/41**	**Fri 12/30/44**	**800 days?**
137	A-4.23.1	Warehouse + Storage Yard + Vehicles + Equipment Design + Bid	Mon 12/9/41	Fri 11/6/43	500 days
138	A-4.23.2	Warehouse + Storage Yard + Vehicles + Equipment Construction	Mon 11/9/43	Fri 12/30/44	300 days
139	A-4.23.3	Construction Management	Fri 11/6/43	Fri 12/30/44	301 days?
140	**A-4.19**	**5C Program Commissioning**	**Mon 9/26/44**	**Fri 12/30/44**	**70 days**
141	**A-4.19.1**	**Phase 1 - East Side IID**	**Mon 9/26/44**	**Fri 12/30/44**	**70 days**
142	A-4.19.1.3	Canal Cx	Mon 9/26/44	Fri 10/7/44	10 days
143	A-4.19.1.1	Salinity Treatment Plant Cx	Mon 10/10/44	Fri 12/2/44	40 days
144	A-4.19.1.2	IID Operational Storage Cx	Mon 12/19/44	Fri 12/30/44	10 days
145	**A-4.19.2**	**Phase 2 - Mountain to Reservoir**	**Mon 12/5/44**	**Fri 12/30/44**	**20 days**
146	A-4.19.2.1	PS 1 - 5, Reach 1 - 6, PGF 1 - 3, PCF Cx, Aq Improvements Cx	Mon 12/5/44	Fri 12/30/44	20 days



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