SRWRS

Sacramento River Water Reliability Study

Engineering Technical Report for the ARPS-Elverta Diversion Alternative

November 2006

SACRAMENTO RIVER WATER RELIABILITY STUDY

Engineering Technical Report for the ARPS-Elverta Diversion Alternative

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LIST OF ACRONYMS AND ABBREVIATIONS

°C	degrees Celsius
ABFSHIP	American Basin Fish Screen and Habitat Improvements Project
amp	ampere
APDS	Auburn Placer Disposal Service
ASR	aquifer storage and recharge
ATPS	Auburn Tunnel Pump Station
BA	Biological Assessment
BAT	best available technology
$CaCO_3$	calcium carbonate
Cal-OSHA	California Occupational Safety and Health Administration
CDFG	California Department of Fish and Game
CDX	Central Data Exchange
CEQA	California Environmental Quality Act
cfs	cubic feet per second
CGS	California Geological Survey
CMP	Coordinated Monitoring Program
CMU	concrete masonry units
CPT	control power transformer
CSD-1	County Sanitation District – 1
CT	product of disinfection concentration and contact time
CVPIA	Central Valley Project Improvement Act
CVRWQCB	Central Valley Regional Water Quality Control Board
CWS	community water system
D/DBP	disinfectant/disinfection by-product
DBE	Design Basis Earthquake

DHS	California Department of Health Services
DOC	dissolved organic carbon
DWR	California Department of Water Resources
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
elevation	elevation in feet above mean sea level
ESWTR	Enhanced Surface Water Treatment Rule
FAA	Federal Aviation Administration
FIRM	Flood Insurance Rate Maps
fps	foot per second
g	gravity
G	velocity gradient
GAC	granular activated carbon
gpm/sf	gallons per minute per square foot
GWUDIS	groundwater under the direct influence of surface water
HAA5	five haloacetic acids
HPC	heterotropic plate count
IDSE	Initial Distribution System Evaluation
$KMnO_4$	potassium permanganate
kV	kilovolt
kVA	kilovolt-ampere
L	liter
L/mg-m	liter per milligram-meter
$lb/DS/ft^2$	pounds dry solids per square foot
LRAA	locational running annual average
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule

M&I	municipal and industrial
MCL	maximum contaminant level
MCLG	maximum containment level goal
MG	million gallons
mg/L	milligram per liter
mgd	million gallons per day
mL	milliliter
mm	millimeter
MRDL	maximum contaminant level goal
MRDLG	maximum residual disinfectant level goal
MRF	Materials Recovery Facility
MRL	minimum reporting limit
MSL	mean sea level
MTBE	methyl tertiary butyl ether
NCWS	noncommunity water system
NEDC	Natomas East Drainage Canal
NEPA	National Environmental Policy Act
NGVD 1929	National Geodetic Vertical Datum of 1929
NOAA Fisheries	National Marine Fisheries Service, National Oceanic and Atmospheric Administration
NMWC	Natomas Mutual Water Company
NOI	Notice of Intent
NPDES	National Pollutant Discharge Elimination System
NTNCWS	nontransient, noncommunity water system
NTU	nephelometric turbidity unit
OEHHA	Office of Environmental Health Hazard Assessment
oocyst/L	oocyst per liter

OSHA	Occupational Safety and Health Administration
PACl	polyaluminum chloride
PCWA	Placer County Water Agency
PG&E	Pacific Gas and Electric Company
PGA	peak ground acceleration
Phase I Report	SRWRS Initial Alternatives Report, Appendix C
PHG	Public Health Goal
POU	Place of Use
PSHA	Probabilistic Seismic Hazard Analysis
PT	potential transformer
RAA	running annual average
Reclamation	United States Department of the Interior, Bureau of Reclamation
RM	river mile
Roseville	City of Roseville
Sacramento	City of Sacramento
SAFCA	Sacramento Area Flood Control Agency
SDWA	Safe Drinking Water Act
SMUD	Sacramento Municipal Utility District
SRCSD	Sacramento Regional County Sanitation District
SRWRS	Sacramento River Water Reliability Study
SSWD	Sacramento Suburban Water District
SUVA	specific ultraviolet absorbance
SWPPP	Stormwater Pollution Prevention Plan
SWRCB	State Water Resources Control Board
SWTR	Surface Water Treatment Rule
TBM	tunnel-boring machine

TCR	Total Coliform Rule
TDS	total dissolved solids
The Reclamation Board	The Reclamation Board of the State of California
TOC	total organic carbon
TSS	total suspended solids
TT	treatment technique
TTHM	total tirhalomethanes
UBC	Uniform Building Code
UCMR	Unregulated Contaminant Monitoring Rule
UPRR	Union Pacific Railroad
USACE	United States Army Corps of Engineers
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service
USGS	Unites States Geological Survey
UV	ultraviolet
WPWMA	Western Placer Waste Management Authority
WTP	water treatment plant
µg/L	microgram per liter

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CHAPTER 1 INTRODUCTION

The Sacramento River Water Reliability Study (SRWRS) is being developed by the United States Department of the Interior, Bureau of Reclamation (Reclamation), with four local cost-sharing partners: Placer County Water Agency (PCWA), Sacramento Suburban Water District (SSWD), City of Roseville (Roseville), and City of Sacramento (Sacramento). The goal of the SRWRS is to develop a water supply plan that is consistent with the Water Forum Agreement objectives of pursuing a Sacramento River diversion to meet water supply needs of the Placer-Sacramento region and promote ecosystem preservation along the lower American River.

1.1. STUDY BACKGROUND

Five water supply alternatives were developed and presented in the SRWRS Initial Alternatives Report (March 2005).[1] The alternatives considered were the Elkhorn/Elverta Diversion Alternative (subsequently renamed SRWRS Elverta Diversion Alternative), Sankey Diversion Alternative, Feather River Diversion Alternative, American River Pump Station Alternative (subsequently renamed ARPS-Elverta Diversion Alternative), and the Folsom Dam Alternative. Of the five alternatives, the SRWRS Elverta Diversion Alternative and the ARPS-Elverta Diversion Alternative were retained for further consideration. The ARPS-Elverta Diversion Alternative is the subject of this report while the SRWRS Elverta Diversion Alternative is presented in a separate document.

This report presents the engineering refinement for the ARPS-Elverta Diversion Alternative, as described below (see **Figure 1-1**). The ARPS-Elverta Diversion Alternative and associated facility plan have been designed to accommodate the needs of the SRWRS cost-sharing partners. The Alternative assumes that PCWA would expand its American River Pump Station (ARPS) near Auburn from a capacity of 65 million gallons per day (mgd) to 130 mgd, construct a new 65 mgd Auburn Tunnel Pump Station (ATPS) Number (No.) 3, expand its planned Ophir Water Treatment Plant (WTP) from a capacity of 30 mgd to 95 mgd, and expand its transmission pipelines to serve its needs. SSWD would divert from the existing San Juan Water District (SJWD) diversion facilities (up to 15 mgd) at Folsom Dam and treat at the Peterson WTP using shoulder capacity. Roseville would increase use of groundwater (approximately 10 mgd) to satisfy its needs in this alternative, but would have no additional surface water diversions. Sacramento would divert separately at the proposed Elverta Intake Facility site on the Sacramento River with a total discharge capacity of 145 mgd. Sacramento's facilities would also include raw water conveyance pipelines, a new WTP of the same capacity, and treated water conveyance to the existing water distribution system.

This report also presents the engineering refinement for a subalternative of the ARPS-Elverta Diversion Alternative known as the ARPS-Joint Sacramento-American Basin Fish Screen and Habitat Improvements Project (ABFSHIP) Elverta Diversion Alternative (see **Figure 1-2**). ABFSHIP would consolidate five existing Sacramento River diversions of the Natomas Mutual Water Company (NMWC) and several local riparian water right holders into two diversions with positive barrier fish screens. ABFSHIP also would eliminate a dam at the mouth of the Natomas Cross Canal to benefit the environment and the Sacramento River fishery. The two diversions on the Sacramento River are located where the levee intersects Sankey Road and Elkhorn Boulevard, respectively. The development of ABFSHIP was delayed by its environmental review process, and NMWC is currently preparing an Environmental Impact Statement/Environmental Impact Report (EIS/EIR) for ABFSHIP through Reclamation (National Environmental Policy Act (NEPA) lead agency) and California Department of

[1] Reclamation. 2005. Sacramento River Water Reliability Study Initial Alternatives Report. March.

Fish and Game (CDFG, California Environmental Quality Act (CEQA) lead agency). ABFSHIP receive CALFED grant for implementation, accounting for required local share for qualifying CVPIA funding. The schedule for implementing the recommended project is subject to funding availability from the Central Valley Project Improvement Act (CVPIA) Fish Screen Program.

Under this subalternative, the proposed Elverta Intake and associated facility plan have been designed to accommodate the needs of the SRWRS cost-sharing partners, and the needs of NMWC, as provided by the Elkhorn Diversion planned in ABFSHIP. Also under this subalternative, NMWC would not construct the Elkhorn Diversion planned in ABFSHIP; instead, the proposed Elverta Intake would be expanded to include NMWC's required diversion capacity of 210 cubic feet per second (cfs) (135 mgd). The other key difference from the ARPS-Elverta Diversion Alternative is inclusion of improvements to approximately 1.6 miles of NMWC's existing Elkhorn Main Canal to allow delivery of raw water from the new Elverta Intake Facility to NMWC facilities both north and south of the intake site.

1.2. OBJECTIVE OF THE REPORT

The primary objective of this report is to refine the engineering of key elements of the ARPS-Elverta Diversion Alternative to develop an alternative that can be evaluated as part of the EIS/EIR. Engineering refinement of the ARPS-Elverta Diversion Alternative includes completing feasibility-level engineering design to generate facility type and sizing requirements, site layouts, pipeline alignments, and related facility plans for power, sewage, and storm drainage, and identify proposed operating and construction characteristics.

1.3. ORGANIZATION OF THE REPORT

This report consists of eight chapters that present various aspects of the engineering refinement. Below is a brief discussion of each chapter.

Chapter 1 – Presents an introduction to the report, including study background, report objective, and report organization.

Chapter 2 – Presents a geotechnical characterization of the project areas and highlights potential hazards. Using the characterization, construction considerations are summarized and future geotechnical investigation recommendations are made.

Chapter 3 – Discusses the new Elverta Intake Facility, the modified ARPS Intake Facility, and the ATPS. For the Elverta Intake Facility, this includes a description of design requirements, site selection, river hydrology, and configuration selection. Using this information, a preferred intake configuration is defined for both the ARPS-Elverta Diversion Alternative, and the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative. For the ARPS Intake facility and the ATPS Booster Station, a description of the PCWA proposed modifications is presented.

Chapter 4 – Discusses the raw water pipelines for the ARPS-Elverta Diversion Alternative site and modifications to the infrastructure for the ARPS diversion and pumping facilities, including a description of hydraulics, pumps, pipe alignments, pipe materials, and pipeline appurtenances.

Chapter 5 – Discusses water treatment facilities, including a new WTP (North Natomas WTP) and an expanded planned Ophir WTP. Treated water goals and objectives for the project, regulatory requirements, and water quality evaluations are presented. Using this information, the North Natomas WTP process selection and an overall facility design are presented. In addition, the proposed PCWA expansion for the Ophir WTP is presented.

Chapter 6 – Discusses the treated water pipelines for the ARPS-Elverta Diversion Alternative, including hydraulics, alignments, special crossings, pipe materials, and pipeline appurtenances.

Chapter 7 – Presents the construction cost estimate for all the water supply components of the ARPS-Elverta Diversion Alternative and the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative.

Chapter 8 – Summarizes the regulatory requirements for constructing facilities as part of the ARPS-Elverta Diversion Alternative. This includes describing the permits that must be obtained and the recommended timing of activities related to obtaining the permits.

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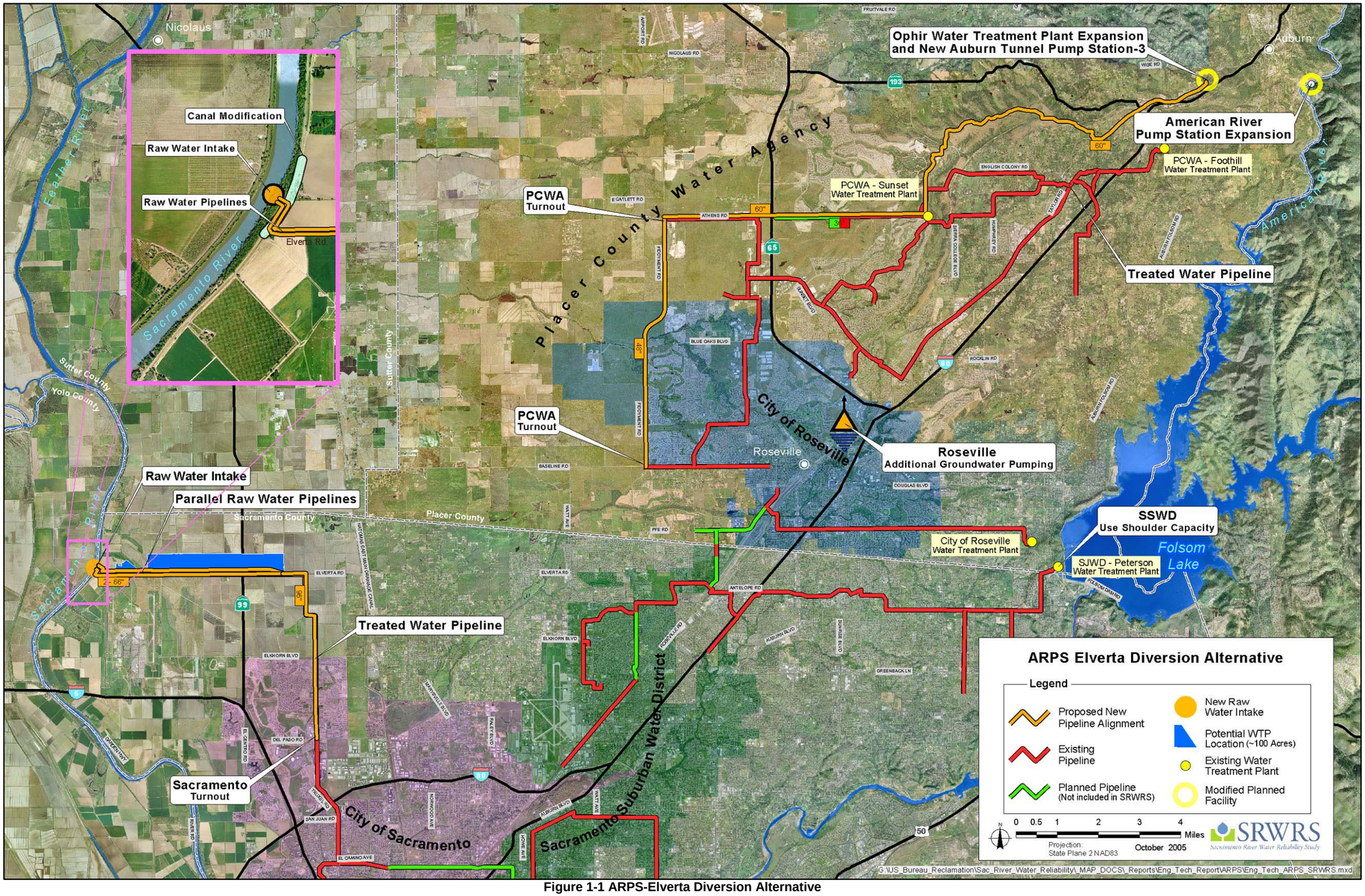

Figure 1-1 ARPS-Elverta Diversion Alternative

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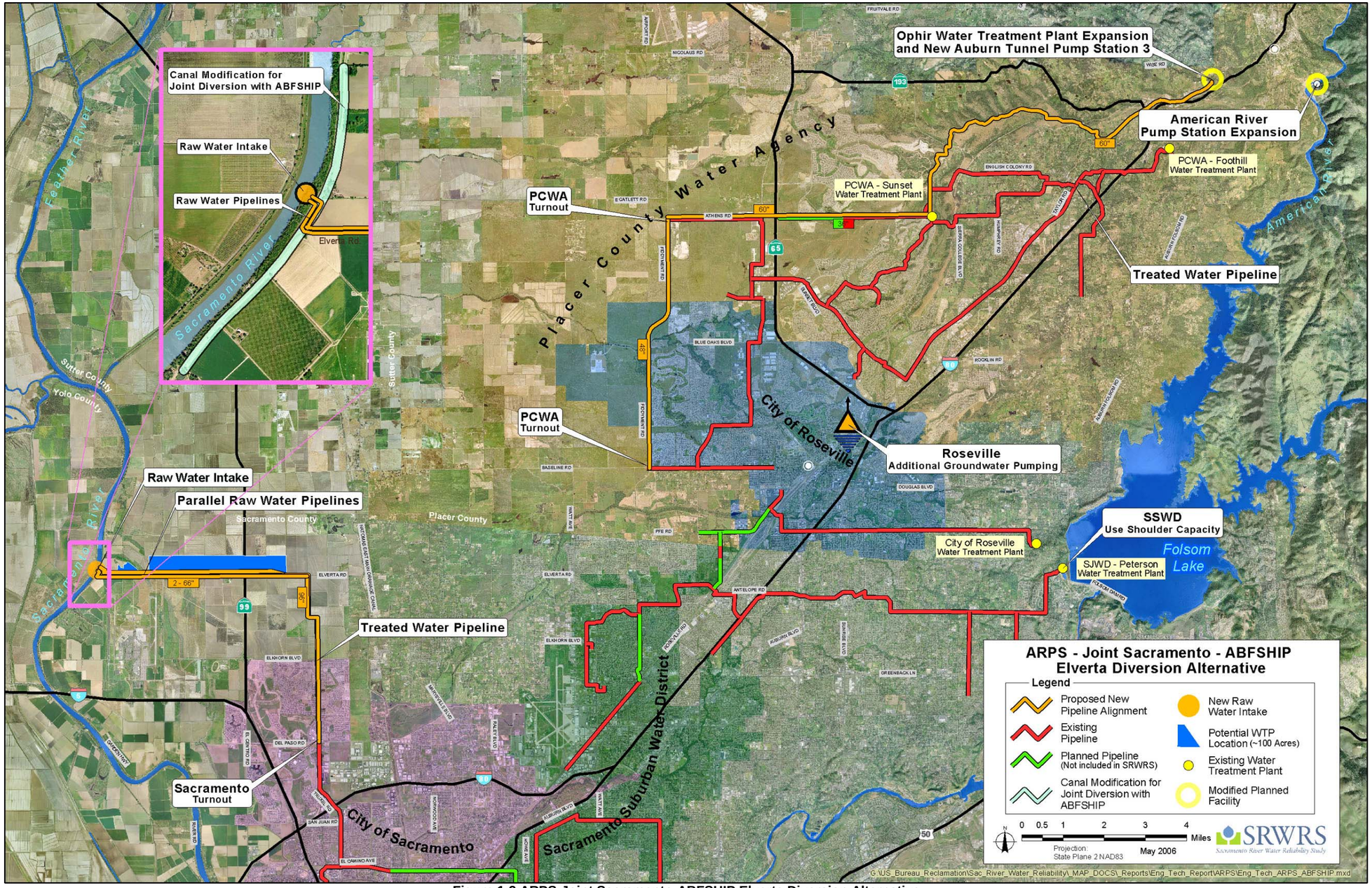

Figure 1-2 ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative

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CHAPTER 2 GEOTECHNICAL CONDITIONS

This chapter on geotechnical conditions is intended to support feasibility-level engineering design and cost estimates of the ARPS-Elverta Diversion Alternative and the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative for inclusion in the SRWRS. From a geotechnical perspective, the two alternatives are nearly identical and will be discussed in this chapter, as a single alternative, except where noted. The chapter briefly describes the alternatives, and the location and nature of each component that makes up the alternatives; presents regional geology and seismicity; details geotechnical conditions for the features associated with the alternatives; considers construction issues; discusses potential geotechnical hazards; and recommends future geotechnical investigations.

The alternative would be constructed in two distinct geomorphological provinces. The southwestern half of the study area (Sacramento portion) where the Elverta Intake Facility, North Natomas WTP, and part of the pipelines would be located, lies in the Great Valley Geomorphological Province, which was formed by low lying, flat topography, and characterized by Quaternary clay, silt, sand, and gravel deposited by flooding of the Sacramento River. Toward the east, these sediments overlap onto older alluvial fan deposits emanating from the Sierra Nevada foothills. The pipelines will cross several highways, railroads, and canals, some of which may have to be tunneled.

The new ATPS No. 3, expanded ARPS and Ophir WTP, and pipelines in the northeastern part of the study area (PCWA portion) would be located in the Sierra Nevada Geomorphological Province, which is characterized by gently undulating to hilly topography, and where trench excavations would generally be in weathered sedimentary, igneous, and metamorphic bedrock. Blasting would probably be required for major portions of these pipelines.

Geologic hazards to the projects to be evaluated in future studies include potentially liquefiable and corrosive soils in the southwestern part of the study area. However, no geotechnical conditions appear to render the planned projects infeasible.

2.1. PROJECT DESCRIPTION

This section describes the study area and components of the ARPS-Elverta Diversion Alternative and the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative.

2.1.1. Study Area

The study area includes the Sacramento facilities (and ABFSHIP facilities in the case of the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative) portion, defined by a triangle approximately 19 miles on each side, oriented in a westerly direction, with its apex at the intake on the Sacramento River. This part of the study area is characterized by the flat topography of the Sacramento Valley.

The study area also includes the PCWA facilities portion, defined by a triangle oriented in a northeasterly direction approximately 21 miles long on two sides with its apex at ARPS and its base 10 miles long, and located in southwestern Placer County. The southwestern half of the study area is characterized by the relatively flat topography of the Sacramento Valley. Towards the northeast, topography gradually becomes more pronounced until developing into the foothills of the Sierra Nevada mountains, which are dissected by southwesterly trending valleys formed by drainages to the east.

2.1.2. Sacramento Facilities

The proposed Sacramento facilities (see **Figures 1-1** and **1-2**) consist of a new raw water intake, 145 mgd pump station (281 mgd for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative), and access bridge that would be located on the left levee (east bank) of the Sacramento River near the intersection of Elverta Road and Garden Highway, which runs along the top of the levee. Twin 54-inch-diameter raw water transmission pipelines (and an additional 72-inch-diameter pipeline for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative) would run through a portion of the levee. The two 54-inch-diameter pipelines would each increase to 66-inches in diameter and continue east along Elverta Road to a new WTP to be constructed just north of Elverta Road. (The 72-inch-diameter pipeline from the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative would run through a portion of the levee and then discharge into the adjacent NMWC canal.)

From the WTP, a 96-inch-diameter pipeline would convey treated water east along Elverta Road, and parallel to the Natomas East Drainage Canal (NEDC), before following the NEDC 4.5 miles south, parallel to Natomas Boulevard/Truxel Road, to connect to the existing Sacramento distribution system at Del Paso Road.

2.1.3. PCWA Facilities

The proposed PCWA facilities include modifications to PCWA's ARPS and associated facilities. ARPS is presently under construction on the right bank of the American River just upstream of the Auburn Dam site. The ongoing construction of the ARPS will install new pump station will be equipped with six vertical lift pumps with a design capacity of 65 mgd. The ARPS-Elverta Diversion Alternative calls for installing two additional pumps, thereby increasing discharge capacity by 65 mgd to 130 mgd.

Raw water from the pump station would be pumped to the Auburn Ravine (Ophir) Tunnel where it would flow west by gravity until being pumped from the tunnel into PCWA's Ophir WTP via the new ATPS No. 3. From the expanded WTP, treated water would be pumped through a 60-inch-diameter pipeline 2.2 miles in a southwesterly direction along Ophir Road to the town of Newcastle. There, the pipeline alignment would continue in a southwesterly direction along Taylor Road for 1.25 miles to Calliston Road. The pipeline would turn west for 0.25 miles before turning northwest under the westbound UPRR tracks. The alignment would continue north 0.1 miles parallel to the west bound UPRR tracks and then turn west for 0.2 miles. The pipeline would head northwest, cross country, passing under Antelope Canal and the eastbound UPRR tracks about 0.9 miles to a future Bickford water storage tank. From the storage tank the pipeline would continue in a westerly direction for 0.3 miles and would then follow the alignment of a new road being planned as part of the development of the Bickford Ranch that gradually descends along the top of a ridge, roughly parallel to the existing Clark Tunnel Road to about elevation 535 for about 3.0 miles. It would then turn northwest parallel to Sierra College Boulevard about 0.1 miles before turning south, running cross country 0.8 miles to Whitney Reservoir, after which it would follow the southerly alignment of the existing 18-inch and 42-inch and planned 30-inch water pipelines about 1.0 miles to PCWA's Sunset WTP and storage tank.

From the Sunset WTP, the pipeline alignment would run due west, parallel to PCWA's recently constructed 24-inch-diameter water pipeline, in subdivision streets, then cross Highway 65 near Athens Avenue about 4.2 miles to intersect Athens Avenue at Industrial Avenue. From that point, the pipeline would continue west 2.2 miles along Athens Road to Fiddyment Road. The pipeline would then be reduced to 48-inches in diameter and turn south 2.8 miles to connect to PCWA's delivery system at Baseline Road.

2.2. GEOTECHNICAL CHARACTERIZATION

In this section, sources for the geotechnical information in this chapter are listed; and regional geology and seismicity, hydrogeology and groundwater, and geotechnical conditions are described.

2.2.1. Sources of Geotechnical Information

Information for preparing this chapter was obtained by reviewing geotechnical reports prepared by others in conjunction with the following projects in the general vicinity of the components of each alternative:

- Lower Northwest Interceptor project (Sacramento Regional County Sanitation District (SRCSD))
- Upper Northwest Interceptor project (SRCSD)
- Titan 1-A Missile Facility (United States Army Corps of Engineers (USACE))
- American River Pump Station Project (PCWA)

In addition, maps and reports published by the California Geological Survey (CGS), and the United States Geological Survey (USGS) were reviewed. These and other sources used to prepare this chapter are listed below:

- Bartow, J.A., and E.J. Helley. 1979. Preliminary Geologic Map of Cenozoic Deposits of the Folsom Area, California. USGS.
- Carlson, W. 1990. Auburn Dam Interim Construction Geology Report, Auburn, California. For Reclamation. November.
- Duffield, W.A., and R.V. Sharp. 1975. Geology of the Sierra Foothills Melange and Adjacent Areas, Amador County, California. USGS.
- Espana Geotechnical Consultants. 2001. Preliminary Geotechnical Report for the Lower Northwest Interceptor Project, Sacramento and Yolo County, California. For MWH. September.
- Espana Geotechnical Consultants. 2002. Final Geotechnical Report for the Upper Northwest Interceptor, Section 7 – Sacramento County/City of Citrus Heights, California. For HDR. October.
- Helley, E.J., and D.S. Harwood. 1985. Geologic Map of the Late Conozoic Deposits of the Sacramento Valley and Northern Sierra Foothills, California. USGS.
- Jennings, C.W. 1994. Fault Activity Map of California. USGS.
- Kleinfelder. 2003. Geotechnical Data Report – New Natomas Pump Station, Lower Northwest Interceptor Project, Sacramento, California. For HDR.
- Kleinfelder. 2003. Geotechnical Data Report – Natomas Force Main, Lower Northwest Interceptor Project, Sacramento, California. For Black and Veatch.
- Kleinfelder. 2003. Geotechnical Data Report – Northern Sacramento River Crossing, Lower Northwest Interceptor Project, Sacramento, California. For Hatch Mott MacDonald.
- Mark Group. 1998. Draft Report Phase 2 Geotechnical Services for Final Design, American River Pump Station Project, Placer County Water Authority, Auburn, California.
- MWH. 2000. Lower Northwest Interceptor Design Report, Sacramento and Yolo Counties, California. For Sacramento Regional County Sanitation District. September

- Wagner, D.L. 1981. Geologic Map of the Sacramento Quadrangle, California. USGS.
- Woodward Clyde. 1997. Focused Remedial Investigation Work Plan Titan 1-A Missile Facility, Lincoln, California. For USACE. May.

Field investigations to support preparation of this chapter consisted of a surficial reconnaissance of the proposed location of the facilities associated with each alternative.

2.2.2. Regional Geology

The study area is situated in the eastern portion of the Sacramento Valley which represents the northern portion of the Great Valley Geomorphological Province of California, and the western foothills of the Sierra Nevada Geomorphological Province. **Figure 2-1** is a surficial geologic map of the region with the features of the alternatives superimposed for reference.

2.2.3. Great Valley Geomorphological Province

The Great Valley of California is approximately 400 miles long and 40 miles wide, oriented along the axis of the State. Erosion of the Coast Ranges to the west and Sierra Nevada mountains to the east has generated alluvial, overbank, and localized lacustrine sediments, which have been deposited in the valley to a thickness of as much as 50,000 feet. Subsequent deformation folded these sediments into an asymmetric syncline with its axis off center toward the Coast Ranges. Along the eastern boundary of the Sacramento Valley, these alluvial deposits pinch out where they lap onto older alluvial deposits associated with western-flowing streams emanating from the foothills of the Sierra Nevada.

The portion of the project area within the Great Valley Geomorphological Province has been mapped in great detail, most recently by Helley and Harwood (1985). Map units include Holocene sediments characterized by active river channel deposits (Q) along the Sacramento and American rivers, alluvium (Qa) representing pre-levee and overbank deposits along the former meandering natural channels of the Sacramento River, and basin deposits (Qb) characterized by floodplain sediments outside former Sacramento River channels. These deposits overlay relatively older Pleistocene deposits such as the Modesto (Qm) and Riverbank (Qr) formations, which pinch out to the east against the Turlock Lake Formation (Qtl), which consists of alluvial fan material associated with western-flowing rivers and streams from the Sierra Nevada. **Table 2-1** describes the stratigraphy of this portion of the study area.

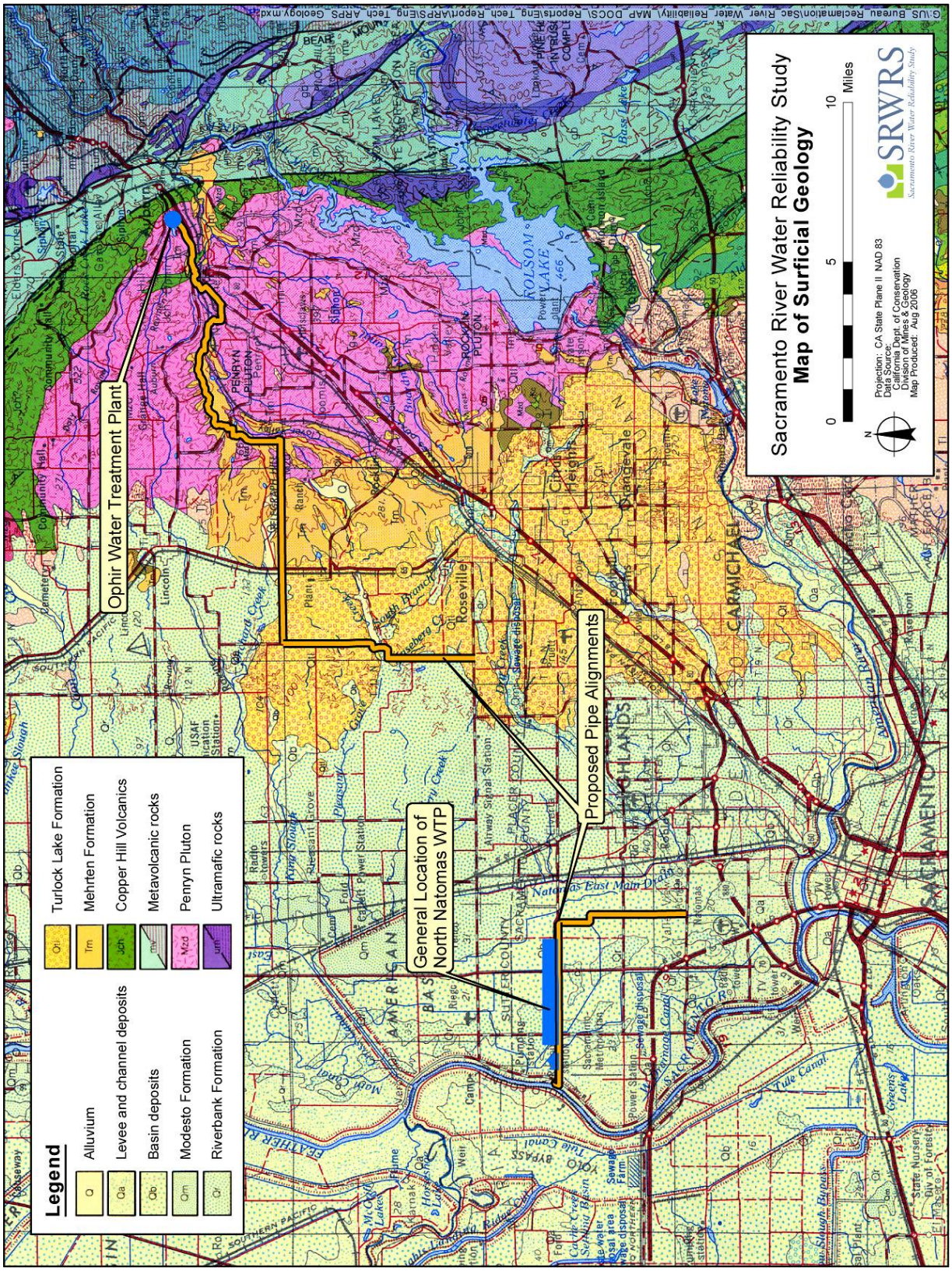

Figure 2-1 Surficial Geologic Map

Table 2-1 Stratigraphy of the Northeastern Portion of the Great Valley Geomorphological Province

AGE	FORMATION	MAP[1] SYMBOL	DESCRIPTION
Quaternary	Recent Alluvium and Levee Deposits	Q	Loose silty sand (SM), and well to poorly graded sand and gravel (SW-SP-GP) deposits in the current Sacramento River channel and against the levees.
	Basin Deposits	Qb	Layer 10 to 20 feet thick of dark, often organic, stiff to very stiff, silts (ML) and clays (CL). The basal contact of this unit is relatively uniform with the exception of apparent paleo-channels infilled with less clayey and more silty and sandy deposits. These alluvial sediments represent overbank floodplain deposits.
	Channel Deposits	Qa	Deposits 5 to 25 feet thick of loose to dense, silty sand (SM) and well to poorly graded sand (SW-SP) with localized layers or lenses of silt (ML). These sediments represent meandering channel deposits of the Sacramento and American rivers prior to levee construction.
	Modesto	Qm	From 0 (where they pinch out to the east) to as much as 60 feet thick, of dense, well to poorly graded sands and gravels (SW-SP-GP) differentiated from overlying deposits primarily on the basis of density and gravel content (i.e., Qm denser and more gravelly than Qa). Absence of Qm to the east represents pinching out against alluvial fan deposits to the east.
	Riverbank	Qr	Stiff to dense silts (ML) and clays (CL) with minor lenses of dense poorly graded sands and gravels (SP-GP). Qr outcrops east of the Sacramento River and generally underlies Qb, Qa, and Qm sediments, and is thought to represent alluvial fan deposits transported by rivers emanating from the Sierra Nevada foothills.
	Turlock Lake	Qtl	Dense, relatively hard, partially consolidated silt (ML), poorly graded sand, and gravel (SP-GP) fan material derived mainly from Sierran granitic and metamorphic rocks.

Note:
[1] Refer to Figure 2-1.

2.2.4. Sierra Nevada Geomorphological Province

In the northeastern part of the study area, the relatively flat terrain of the Great Valley Geomorphological Province gives way to the undulating and gently rolling topography of the foothills of the western edge of the Sierra Nevada Geomorphological Province. The area is characterized by the Tertiary Mehrten Formation (Tm) compressing volcanic conglomerates and tuffaceous sandstone and siltstone, which overlie Cretaceous dioritic crystalline rocks that represent the westernmost outcrops of the Sierra Nevada batholith, as characterized by the Penryn and Rocklin plutons (Mzd). East of these rocks are the Jurassic Copper Hill Volcanics (Jch), and the complex of intermixed meta-volcanic/ meta-sedimentary (Mv/Ms), gabbroic (gb), and ultramafic (um) rocks of the Foothill Melange-Ophiolite Metamorphic Belt. Geologic units within the Foothills Melange-Ophiolite Metamorphic Belt were probably formed during late Triassic to early Jurassic time as volcanic island arc basalts, mafic tuffs, and marine sedimentary rocks. During the late Jurassic to earliest Cretaceous time, these units accreted to the western margin of the North American continent, through convergent plate tectonism (subduction), during the Nevadan Orogeny.

The plutonic, volcanic, and metamorphic rocks of the Foothills Melange-Ophiolite Metamorphic Belt, as well as much younger surficial sediments exposed in the eastern half of the study area, are described in **Table 2-2**.

Table 2-2 Stratigraphy of the Western Sierra Nevada Geomorphological Province

AGE	FORMATION	MAP[1] SYMBOL	DESCRIPTION
Quaternary	Recent Alluvium	Qa	Channel and creek bed deposits generally consisting of hard, rounded metamorphic and quartzose sand, gravel (SP-GP), cobbles, and boulders with a thickness estimated to vary from a few feet to as much as 80 to 100 feet along the original alignment of the American River.
	Colluvium	Qcol	Blankets most natural ground surface, and consists generally of mixed slope wash, talus, and creep deposits composed of varying percentages and sizes of rock fragments and clay (CL). Colluvium is generally 2 to 5 feet thick ranging to 10 feet locally.
	Landslides	Qls	Landslide masses range in composition from loose unsorted rubble with minor clay matrix, to old compact masses, locally cemented with iron oxide. Masses contain variable percentages and sizes of angular rock fragments, locally including slumped rock masses. Composition depends upon source, area, and mode of transport. Landslide debris is generally greater than 10 feet thick.
Tertiary	Mehrten	Tm	Volcanic conglomerate and tuffaceous sandstone, with andesite mudflow breccia.
Cretaceous	Penryn and Rocklin Plutons	Mzd	Dioritic rocks. Medium-grained granite that constitutes the westernmost occurrence of the Sierra Nevada batholith.
Jurassic	Copper Hill Volcanics	Jch	Metamorphosed mafic pyroclastic rocks, and pillow lava with minor felsic porphyrite.
	Foothills Melange - Ophiolite Metamorphic Belt	Mv. Ms, gb, um	Chaotically intermixed rocks, including meta-volcanic rocks (Mv); meta-sedimentary rocks (Ms); gabbroic and hypaabyssal intrusive rocks (gb); and ultramafic rocks (um).

Note:
[1] Refer to Figure 2-1.

2.2.5. Regional Seismicity

Tectonically, the study area is relatively distant from major Holocene (last 10,000 years) active fault systems, as can be seen in the map of faults and historic earthquakes (**Figure 2-2**). Historic earthquake epicenters to the west of the project include the San Andreas and Hayward-Rodgers Creek fault systems, and the Coast Ranges-Sierran Block boundary system. To the east lie the Foothills Fault System, and Sierra Nevada Frontal Fault system. Since the nearest active fault systems are a considerable distance from the site, recorded ground motions have been historically low. **Figure 2-3** is a map of peak ground acceleration (PGA) contours for the region. **Table 2-3** shows the major fault systems in the region, approximate distance from the center of the study area, and magnitude of a potential earthquake in the system.

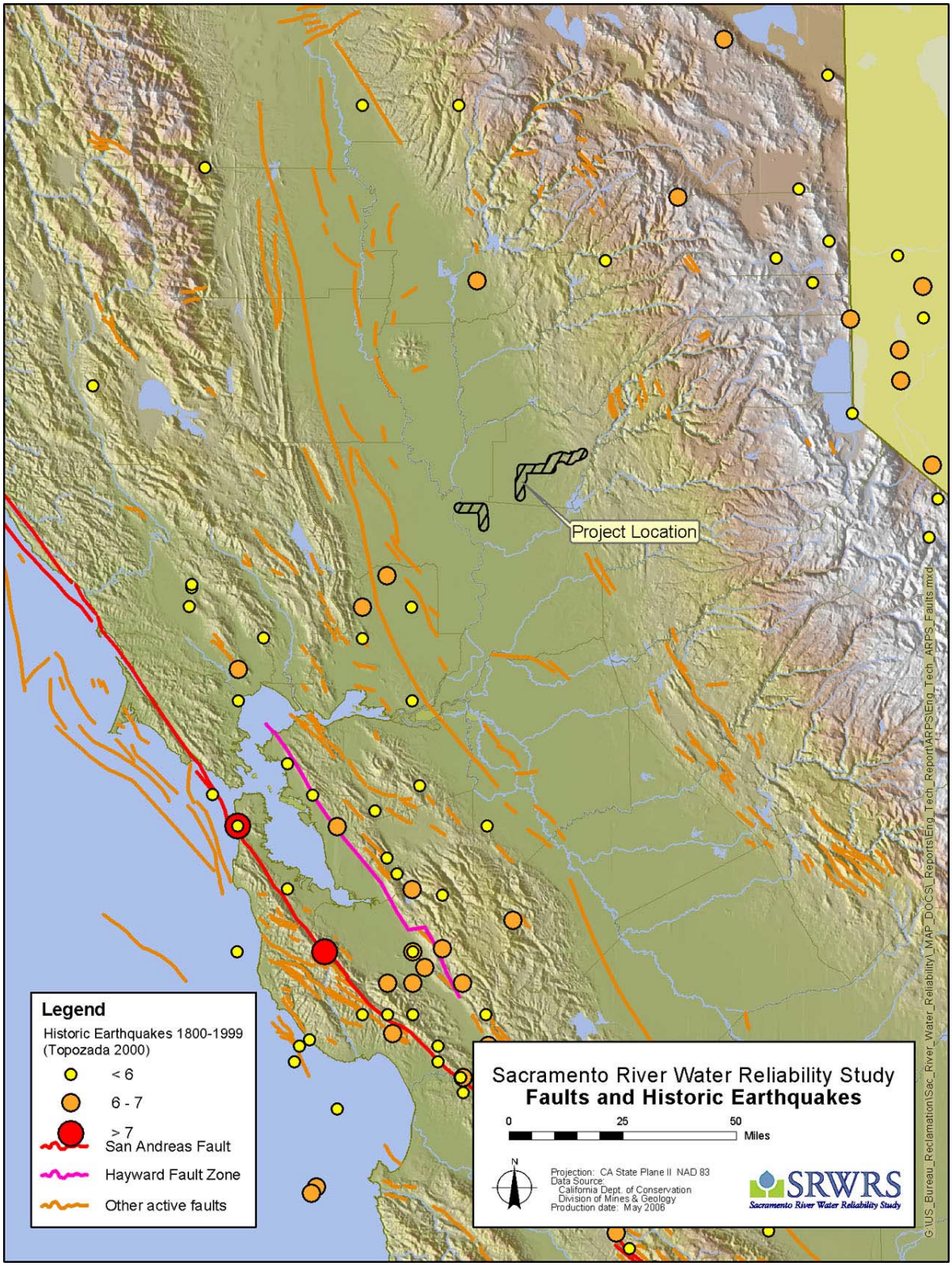

Figure 2-2 Faults and Historic Earthquakes

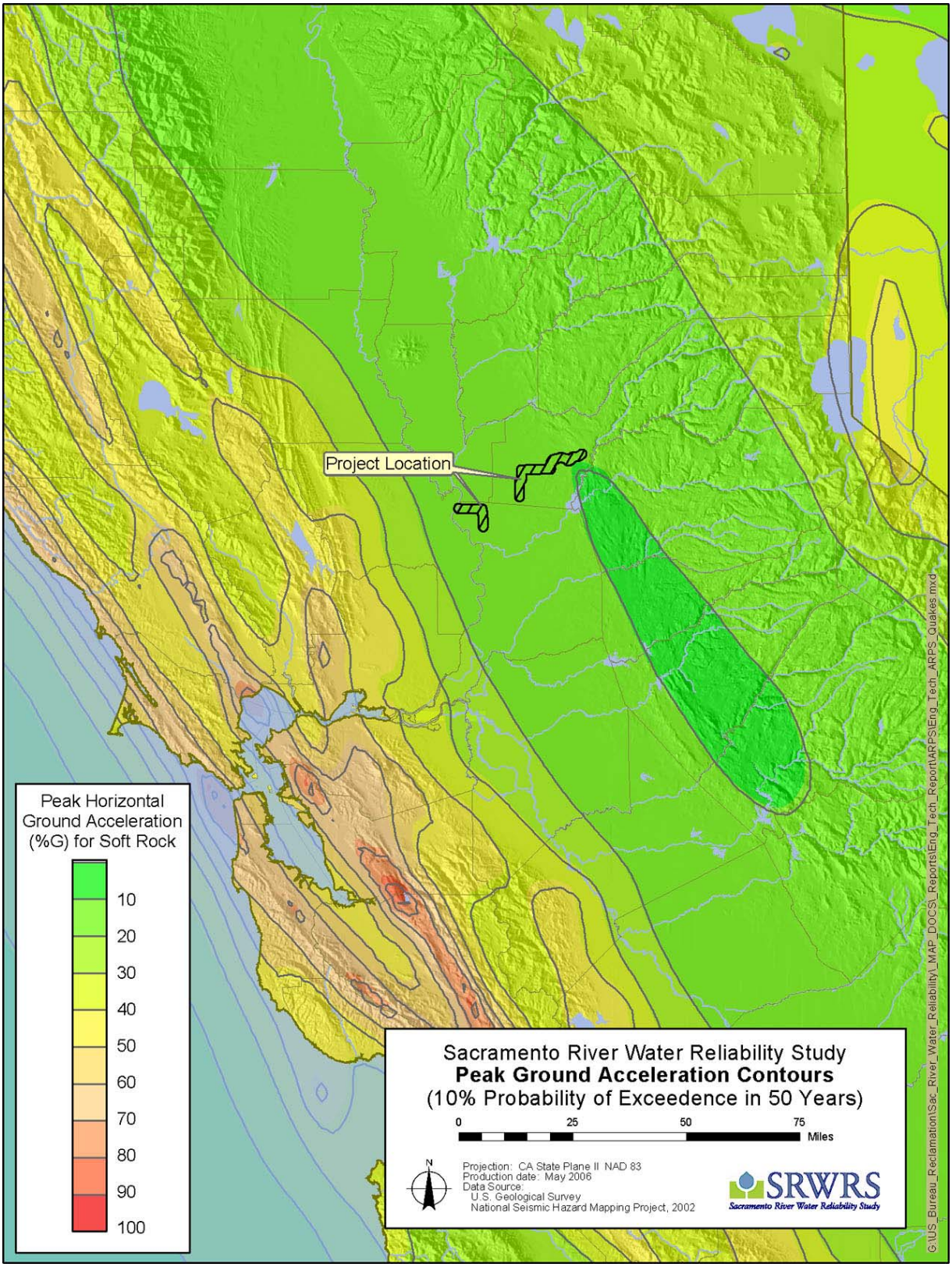

Figure 2-3 Peak Ground Acceleration Contours

Table 2-3 Regional Fault Systems

FAULT SYSTEM	DISTANCE (miles)	MAGNITUDE (M)
Foothills Fault System	15	6.5
Dunnigan Hills	25	6.5
Coast Range – Sierran Block Boundary	25	6.8
Hayward – Rodgers Creek	70	7.1
San Andreas	90	8.0

The potentially active faults nearest to the study area are associated with the Foothills Fault system immediately to the east within the Foothills Melange-Ophiolite Metamorphic Belt. This series of subparallel, northwest-trending vertical faults includes at least two major fault zones. The easternmost is the Melones Fault zone, and the westernmost is the Bear Mountains Fault zone. The Foothills Fault system is approximately 200 million years old, with the last major seismic movement occurring about 140 million years ago. Although the Willows and Dunnigan Hills faults have been mapped a relatively short distance to the west of the study area, these faults are not classified as active by CGS, and are thus not considered capable sources of potential earthquakes or ground rupture.

2.2.6. Hydrogeology and Groundwater

As described above, the surficial geology of the western portion of the study area comprises recent alluvial deposits adjacent to the Sacramento River. In general, the hydrogeologic condition of these deposits is characterized by a nearly continuous surface layer 10 to 20 feet thick of low-permeability, soft to stiff clays and silts, underlain by a layer 5 to 25 feet thick of slightly dense to dense sand conducive to relatively high storage and flow of groundwater. Beneath these two layers lie the considerably older, denser, and less permeable sand, gravel, and stiffer silts and clay of the Modesto, Riverbank, and Turlock Lake formations.

Groundwater levels in the western portion of the study area are primarily controlled by natural recharge from the American and Sacramento rivers to the south and west respectively. Groundwater levels in the area generally range from about 2 to 5 feet above msl, or about 7 to 15 feet below the ground surface. However, historic records of the California Department of Water Resources (DWR) from 1963 to 2003 indicate groundwater levels may be as high as the ground surface concurrent with high Sacramento and American River levels during major storm/flood events, such as in 1986 and 1997. Groundwater levels are expected to vary based on seasonal influences, adjacent canal or river stage, irrigation practices, runoff conditions, and other factors. Groundwater contours drawn from spring 2002 data are shown in **Figure 2-4**.

Although indications of groundwater contamination have not been encountered in the study area, pesticide, oil/petroleum hydrocarbon products, and methyl-tertiary-butyl ether (MTBE) are known to have affected groundwater in the vicinity of the Sacramento International Airport, truck stops along major highways, and Titan missile silos in Lincoln.

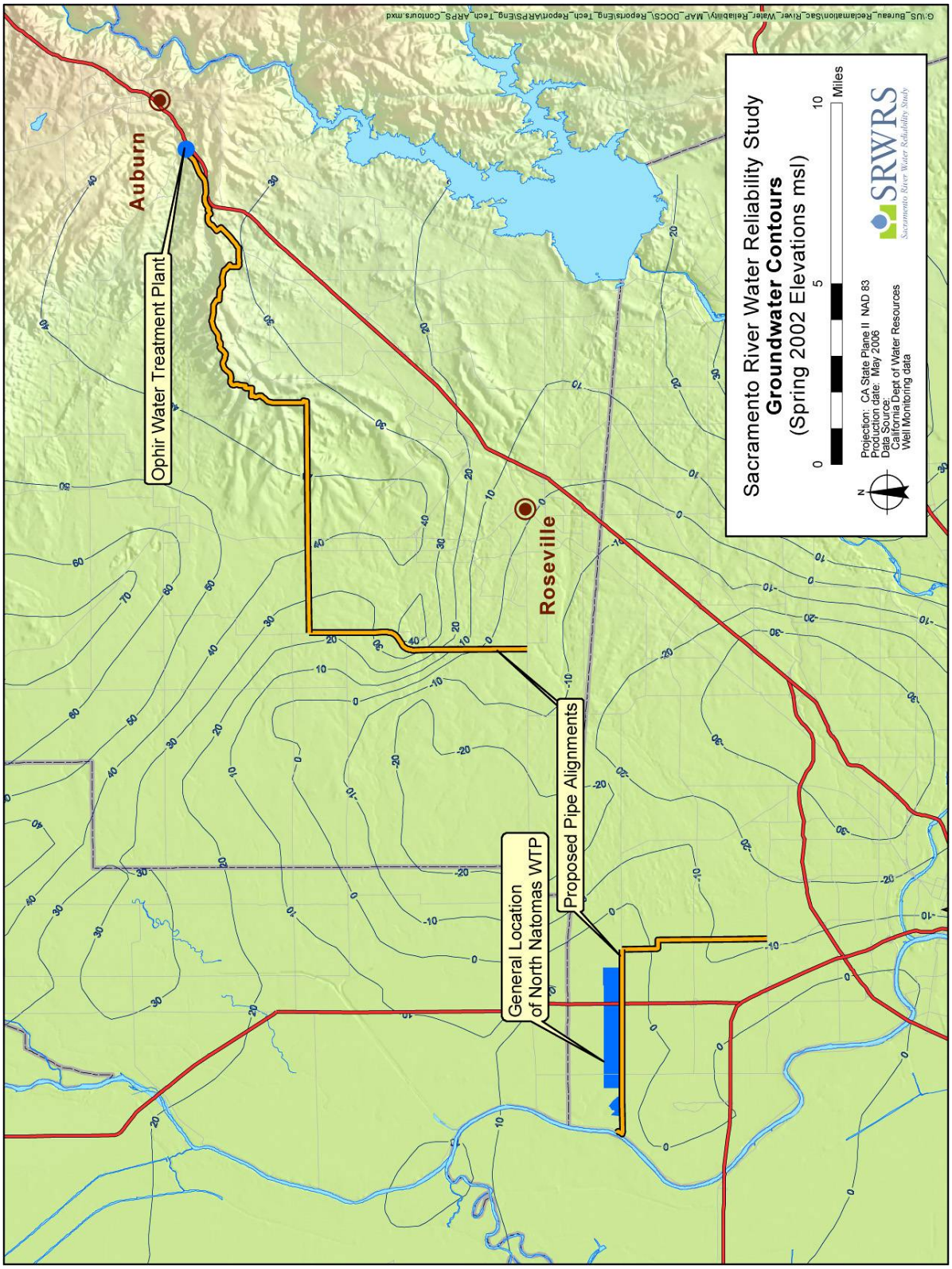

Figure 2-4 Groundwater Contours

2.2.7. Geotechnical Conditions

This section describes geotechnical conditions for Sacramento and PCWA facilities.

2.2.7.1. Conditions for Sacramento Facilities

The Elverta Intake Structure would be located in recent Quaternary alluvium of the Sacramento River channel, which consists of sand, silty sand, and gravel. The raw water pipeline would pass through the levee, composed of sand and silty sand, and then along the low-lying fields adjacent to Elverta Road. These fields are primarily basin deposits characterized by a layer of clayey, relatively impervious soils approximately 10 to 20 feet thick overlying more pervious alluvial sands.

Depending on where it is located along Elverta Road, the North Natomas WTP site could be founded in relatively soft clay, silty sand, and sandy basin deposits or firmer deposits of the same materials associated with the Riverbank Formation. The 96-inch-diameter pipeline would continue in parallel east in basin deposits or the Riverbank Formation. The 96-inch pipeline would turn south to connect to Sacramento's distribution system, and would be entirely within basin deposits.

2.2.7.2. Conditions for PCWA Facilities

The 60-inch-diameter pipeline from the Ophir WTP would be located in the shoulder or adjacent to the roads described in **Section 2.1.3**. The upstream portion of the pipeline, along Ophir and Taylor roads, under Highway 193 and the UPRR, until approximately the elevation 730 contour above the second crossing of the UPRR, would be buried in colluvium, residual soil, boulders, and sound bedrock associated with the Penryn Pluton. In most places, sound bedrock is very close to the ground surface.

As the pipeline climbs the ridge in the weathered granite of the Penryn Pluton, it would pass the contact with the overlying residual soil and moderately weathered andesitic conglomerate, sandstone, tuffaceous sandstone, and volcanic breccia of the Mehrten Formation. Along the ridge, the alignment will remain in this formation, which is characterized on the surface by large angular blocks of tuffaceous sandstone to 5-feet in diameter. As the alignment descends to Sierra College Boulevard, it would again cross into the contact into residual soil, weathered granite, and boulders of the Penryn Pluton. The alignment would cross alluvial deposits in the channel of Clover Valley Creek, before again climbing past the contact into the Mehrten Formation to the Sunset WTP at the crest of a ridge at about elevation 580.

The pipeline from the WTP would be buried in colluvium and weathered bedrock down the ridge to the west, across the alluvial filled valley formed by Grove Creek, up another low ridge composed of weathered volcanics of the Mehrten Formation, before descending to the relatively gentle topography that characterizes the partially consolidated sand, silt, and gravels of the Turlock Lake Formation, with sporadic occurrences of the Riverbank Formation described previously.

2.3. CONSTRUCTION CONSIDERATIONS

This section describes construction considerations for Sacramento and PCWA facilities.

2.3.1. Construction Considerations for Sacramento Facilities

Groundwater is expected to be the main construction consideration for the pipelines in the flat western part of the study area adjacent to the Sacramento River. Excavations for the pipelines and North Natomas WTP would have to be dewatered where the groundwater level was above the pipe trench or structure invert, and the deep excavations would have to be shored. Where the trench invert is projected to be within the upper impervious zone, care would have to be taken to ensure that enough material is left in the bottom of the trench excavation to offset uplift pressure from the underlying confined aquifer.

2.3.2. Construction Considerations for PCWA Facilities

The primary construction consideration for most of the pipelines of the ARPS-Elverta Diversion Alternative is depth to, and soundness of, bedrock. In the foothills at the upper end of the pipeline, sound bedrock of the Penryn Pluton appears to lie just below the ground surface. It is anticipated that up to 80 percent of trench excavation would require blasting. Downstream to the southwest, the crystalline rock is more deeply weathered in a spheroidal pattern that results in large subrounded outcrops and boulders surrounded by highly to moderately weathered soil generally referred to as “decomposed granite (DG)”. In this area, rock excavation by blasting is expected to be required for about 50 percent of the trench.

Trench excavation in the ridges formed by the volcanic Mehrten Formation is expected to be easier than in the granite but will still probably require blasting depending on the depth to bedrock and degree of weathering.

In the low foothills to the west, the pipeline would be in the dense, relatively hard partially consolidated silt, poorly graded sand, and gravel of the Turlock Lake and Riverbank formations.

Unsupported, vertical wall trench excavation should be possible above the water table in the excavations in weathered and sound bedrock. However, where the pipeline crosses alluvium associated with creeks, and the water table is high, dewatering and/or shoring is expected to be required.

2.4. GEOTECHNICAL HAZARDS

Geotechnical hazards discussed in this section include earthquakes, groundwater, slope stability, hazardous gases, and potentially corrosive soils.

2.4.1. Earthquakes

Aspects of earthquakes to be considered include seismic ground motions, surface rupture, and liquefaction.

2.4.1.1. Seismic Ground Motions

Ground motions are estimated by modeling the behavior of the source fault(s), the travel path to the site, and near-surface conditions beneath the site. This can be accomplished either by assuming an earthquake occurs at each source fault (i.e., deterministically) or by estimating the likelihood and understanding of an event given the fault(s) movement and seismic history (i.e., probabilistically). Most California agencies and the current Uniform Building Codes (UBC) prefer the probabilistic method. This method has been

used by CGS (1996) and USGS (1996) for the entire State for soft rock conditions. As shown in **Figure 2-3**, this Probabilistic Seismic Hazard Analysis (PSHA) modeling estimates a maximum horizontal PGA of 0.2 g for the overall project area using the Design Basis Earthquake (DBE) (10 percent probability of exceedence in 50 years) ground motion.

2.4.1.2. Surface Rupture

The potential for ground surface rupture is generally assessed on the basis of the presence of active Holocene (less than 10,000 years) faulting in the project area. Since no active faults have been mapped in the study area, and the site is not located within or near an Alquist-Priolo Earthquake Fault Zone, surface rupture is not considered a hazard for any of the planned features.

2.4.1.3. Liquefaction

Liquefaction is a condition that occurs when relatively low-density, saturated soils behave as a fluid if subjected to seismic ground motions. This condition is most prevalent in loose, granular soils within 50 feet of the ground surface. The principal effects of liquefaction on buried pipelines or structures are settlement (both total and differential), loss of foundation support, buoyancy, and lateral spreading of soils near free faces such as levees. Since low-density granular soils are known to exist beneath the western portion of the study area in conjunction with high groundwater levels, liquefaction cannot be ruled out in this area, and a liquefaction analysis should be performed during the next phase of study.

2.4.2. Groundwater

Shallow groundwater conditions are common in the western portion of the study area, especially adjacent to the Sacramento River and NEDC. Seepage from the Sacramento River and NEDC through relatively permeable sandy materials overlain by less permeable clayey soils is expected to cause locally confined aquifer conditions during periods of elevated river levels. Aquifer confinement occurs when the piezometric groundwater surface elevation is above the bottom of a confining clay layer (aquitard). Near the Sacramento River east levee, the piezometric groundwater surface elevation is expected to be above the ground surface during periods of high river levels and decrease with distance from the river. These high groundwater conditions could result in unstable excavation bottoms and side slopes unless excavations are properly dewatered or stabilized by shoring.

2.4.3. Slope Stability

Due to the flat topography of the western portion of the study area, potential for landslides and/or lateral spreading during a seismic event would be confined to existing levee slopes of the Sacramento River and NEDC. The stability of these slopes would depend on the height and steepness of the slope versus the strength of underlying materials, and should be analyzed in conjunction with proposed excavations exposing prelevee alluvium. The stability of levee slopes should be calculated considering both static stability (i.e., no seismic loading) and seismic stability considering the anticipated 0.2 g PGA for DBE ground motion. The potential for rapid drawdown conditions in the waterways should be considered and addressed as appropriate. Slope stability evaluation should also consider the potential for lateral spreading toward free faces represented by the Sacramento River and NEDC.

Slope stability is not expected to be an issue in the eastern portion of the project area located in the foothills. There, topography is more accentuated than in the western part of the project area but subsurface conditions are characterized by sound to moderately weathered crystalline and volcanic bedrock, or residual granular soils derived from weathering of that bedrock, occurring near the ground surface, and are relatively stable.

2.4.4. Hazardous Gases

Hazardous subsurface migration of gases such as methane has become a severe concern in some areas especially adjacent to landfills, and oil and natural gas fields. Auburn Placer Disposal Service (APDS) operates a landfill in conjunction with the Western Placer Waste Management Authority's (WPWMA) Materials Recovery Facility (MRF) located south of Athens Avenue near its intersection with Fiddyment Road. Future geotechnical investigations of the pipeline alignment in this area should include a Phase I Environmental Site Assessment to detect the presence of hazardous gases. Gas fields do exist within the region but none are mapped in the study area. No oil fields are located in the greater Sacramento region.

2.4.5. Potentially Corrosive Soils

Recent tests for soluble sulfates, soluble chlorides, and electrical resistivity of soils in the western part of the study area in support of SRCSD's Lower Northwest Interceptor project indicated these soils to be moderately corrosive to buried metal pipe. Mitigation measures would typically include binding of pipe joints and construction of test stations along the pipeline alignment to monitor local corrosion conditions. Cathodic protection of portions of the pipeline may be required.

2.5. RECOMMENDED FUTURE GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations for the next phase of project development should include the following:

- Detailed surficial geologic mapping
- Preliminary subsurface investigation by boreholes and test pits of pipeline alignments, including determination of which portion of the foothills trench excavation would be in sound rock requiring blasting
- Sample collection and laboratory testing
- Retention of a corrosion engineer to conduct a Soil Corrosivity Investigation and produce a report
- Comprehensive analysis of available groundwater data, and seasonal fluctuation of groundwater levels
- Phase 1 environmental assessment of groundwater quality to identify any hazardous conditions that should be avoided, and to provide baseline information for dewatering permit applications
- Phase 1 Environmental Site Assessments

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CHAPTER 3 INTAKE FACILITY AND FACILITIES REQUIRED FOR THE ARPS-JOINT SACRAMENTO-ABFSHIP ELVERTA DIVERSION ALTERNATIVE

This chapter presents an engineering analysis of the intake facilities for the ARPS-Elverta Diversion Alternative as part of the SRWRS and also describes the modified facilities required for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative. Only Sacramento's Sacramento River intake facilities are presented in this chapter. Minor modifications required for PCWA's American River raw water diversion facility are noted below, while PCWA's raw water pumping system modifications are presented in **Chapter 4**.

This chapter is a continuation and refinement of the work presented in Appendix C of the SRWRS Initial Alternatives Report (Phase I Report). The primary purpose of this chapter is to advance the engineering development of the intake facility at Elverta Road and describe it sufficiently to allow completion of the EIS/EIR. All elevations presented in this chapter are referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29).

The ARPS-Elverta Diversion Alternative, developed in the Phase I Report (Appendix C to the Initial Alternatives Report (March, 2005)) and refined here, would include construction of a new intake facility located on the Sacramento River near Elverta Road (at the same site as the SRWRS Elverta Diversion Alternative intake) as well as modifications and expanded use of PCWA's planned ARPS and its associated American River Intake. The proposed new Elverta intake on the Sacramento River would provide 145 mgd of capacity to supply Sacramento. PCWA would draw an additional 65 mgd through the ARPS intake while SSWD and Roseville would meet their water supply needs without new or modified intake facilities.

In addition to the base alternative for the Elverta Intake described above, the Phase I Report also included a brief discussion of the possibility of consolidating intake facilities with NMWC, which planned to expand its existing intake, located on the Sacramento River near the proposed Elverta Intake site, to 135 mgd (210 cfs). This chapter will refine the consolidation discussion and present this subalternative, known as the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative, which includes increased pumping capacity and canal improvements required for NMWC. The intake site evaluation and selection activities described in this section were developed using the intake facility required for the ARPS-Elverta Diversion-Alternative. Modifications and additional facilities required for the ARPS-Joint Sacramento ABFSHIP Elverta Diversion Alternative are presented in **Section 3.7**.

This chapter presents the basis of design, the site evaluation selection process, a refinement of the river hydrology, and the initial intake configuration evaluation process for the Elverta Intake Facility. Also discussed are the power, sewer, storm drainage, and special considerations for the proposed facilities. Construction and operating characteristics of the proposed facilities are also presented.

As noted, required modifications to PCWA's raw water pumping system at ARPS and the ATPS are discussed in **Chapter 4** of this report. Based on conversations with the Project Manager for PCWA's ARPS intake (George Slovinsky, Mclaughlin Engineering), it was determined that the only task associated with providing the necessary capacity from the intake would be to mobilize a crane to remove blank plates covering the fish screens, if required. The Coanda-type screens that are being constructed at two locations in the bottom of the river will have an estimated 145 mgd combined capacity after normal wearing of the screen surface. However, intake capacity will be degraded by sediment clogging and

highwater flows that increase tailwater elevations. The effects of these factors will be managed under operation and maintenance routines. No further discussion of PCWA's intake will be provided in this chapter.

3.1. BASIS OF DESIGN

Initial criteria used as the basis of design for the intake facility are based on the SRWRS cost-sharing partners' operational requirements, current published criteria for fish passage facilities by CDFG (CDFG, 1997)[2] and the National Oceanic and Atmospheric Administration (NOAA), National Marine Fisheries Service (NOAA Fisheries, 1997),[3] current industry practice, and experience at similar facilities. Criteria are presented below.

3.1.1. Project Flows and Pump Configuration

Criteria for project flows and pump configuration are shown for both the ARPS-Elverta Diversion Alternative and the ARPS-Joint City of Sacramento-ABFSHIP Elverta Diversion Alternative.

3.1.1.1. ARPS-Elverta Diversion Alternative

- Maximum water diversion at Elverta Diversion = 145 mgd (225 cfs).
- Minimum water diversion at Elverta Diversion = 44 mgd (68 cfs).
- Pump configuration could include two @ 10 mgd, two @ 20 mgd, three @ 30 mgd; some or all these may be equipped with variable-frequency drives. One additional 30-mgd pump would be provided for backup.

3.1.1.2. ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative

- Maximum overall water diversion at Elverta Diversion = 281 mgd (435 cfs).
- Additional four dedicated pumps @ 33 mgd each, with variable-frequency drives, provided for NMWC.

3.1.2. Fisheries and Fish Screens

- The project design will be based on protecting juvenile anadromous fish present in the Sacramento River at the point of diversion.
- The target species and its life stage of concern is assumed to be the winter-run Chinook salmon fry.
- River water approach velocity, normal to the screen face, will be 0.33 feet per second (fps) maximum.
- River sweeping velocities parallel to the screen face must be at least twice the approach velocity.
- The screen opening slot will be 1.75 millimeters (mm) wide (0.069 inches).
- Stainless steel wedgewire screens will be used.
- A screen cleaning mechanism designed to clean all screens within a 5-minute period will be used.

[2] CDFG. 1997. Fish Screening Criteria. April.
[3] NOAA Fisheries. 1997. Fish Screening Criteria for Anadromous Salmonids. January.

3.1.3. Debris Management

- The intake structure and intake access bridge will be designed to shed debris.
- The intake structure and intake access bridge will be designed to withstand high impacts from large, floating or submerged debris.

3.1.4. Levee Impacts

- The levee will be restored in accordance with The Reclamation Board of the State of California (Reclamation Board) levee design standards.
- The levee road (Garden Highway) will be restored and/or modified in accordance with current Sacramento County Department of Transportation design standards.
- Consultation with The Reclamation Board will take place as part of the refinement of intake alternatives to verify design and construction constraints.

3.1.5. Operation and Maintenance

- Intake facility will be unmanned.
- The project will provide means for accessing and removing fish screens and pumps for maintenance and repair.

3.1.6. Water Supply Reliability

- Intake will be designed to provide the desired flows on a continuous basis throughout the year.
- The completed project will operate at varying water levels and flow in the Sacramento River, with the range spanning the historical average low flow through the 100-year-flood flow.

3.1.7. Environmental Impacts

- Intake facility design will strive to minimize impacts to the riparian zone, aquatic habitat, and the shaded river habitat.
- Design will strive to minimize facility footprint by maximizing use of available water depth.

3.1.8. Public Safety

- The facility will be designed to minimize impacts on river traffic and recreation during construction and operation.
- Facilities will be designed with consideration of published guidelines from the United States Coast Guard (USCG) and the California Department of Boating and Waterways.

3.1.9. Security

- Design of the intake facility would consider the security of the structure and its components relative to theft and vandalism.
- Design would assume a motorized, spiked, or barbed-wire-topped gate on the bridge and alarms on the gate and doors.

3.1.10. Regulatory Requirements

Planning and design will follow published guidelines for all pertinent governmental agencies, including, but not limited to the following:

- Reclamation
- USACE
- CDFG
- UCG
- Federal Aviation and Administration (FAA)
- Sacramento Area Flood Control Agency (SAFCA)
- The Reclamation Board

3.2. ELVERTA INTAKE SITE EVALUATION AND SELECTION

The intake site selection process presented in the Phase I Report included a 3-mile reach of river in the vicinity of the location of the proposed intake site on the Sacramento River at river mile (RM) 74.6, initially identified by Sacramento. Bathymetric and topographic information developed by USACE, preliminary river flow/stage analysis, aerial photography, and field investigations were used to evaluate the river reach. It was determined that the proposed site at RM 74.6 had the best design characteristics in the 3-mile reach evaluated.

To verify that other desirable sites on the river were not being overlooked, the current study evaluated a larger portion of the Sacramento River. An approximate 16-mile reach of river, from the confluence of the Sacramento and American rivers at approximately RM 60, to the Sacramento and Sutter county line at approximately at RM 76, was evaluated. This portion of river was selected because it represents the feasible boundaries of the project, based on proximity to the proposed service area and location relative to existing intakes (Sacramento's existing Sacramento River WTP Intake is located immediately downstream from the American River confluence).

3.2.1. Site Evaluation Criteria

Potential sites in the 16-mile reach were evaluated with respect to the criteria listed in **Table 3-1**.

Table 3-1 Intake Site Evaluation Criteria

Criteria	Evaluation Approach
Available water depth	A greater water depth to river bottom is an asset as it allows the intake structure to be constructed deeper, with a smaller overall footprint in the river.
Site located on an outside bend	Since water is moving faster on the outside bend, the chance for sediment deposit and build-up is lower.
Narrow river section between defined levees	Locating the intake between defined levees reduces the chance that the river would meander away from the intake.
Proximity to existing homes	Locating the intake farther away from homes, buildings, and parks was preferred; construction noise, operating noise, and maintenance activities may cause neighbors to oppose construction, or request operational restrictions.
Proximity to turnout points	Proximity to the cost-sharing partners' turnout points reduces overall project cost.
Site located on the left (east) bank of the river	An obvious criterion; this avoids the need to tunnel conveyance piping under the Sacramento River.

3.2.2. Site Evaluation Process

The following paragraphs describe the process used to evaluate potential intake sites within the stretch of the Sacramento River from the Sacramento and Sutter county line at RM 76 to the confluence of the Sacramento and American rivers at RM 60.

Six large figures of the USACE Sacramento River bathymetry data were overlaid on color aerial photographs and printed at a scale of 1 inch to 300 feet. Each figure included a section of the river approximately 3 to 4 miles in length. The figures were evaluated for potential sites using the criteria from **Table 3-1**. Several sites that appeared to meet the evaluation criteria were identified and field-evaluated. Advantages and disadvantages of each site were summarized.

Reduced-scale copies of the original figures used in the evaluation are included as **Figures 3-1** through **3-6**. The river segments presented in each figure are evaluated in the following sections, including advantages and disadvantages of the potential sites.

3.2.2.1. RM 76 to RM 73.2

Two potential sites were identified in this reach. See **Figure 3-1**.

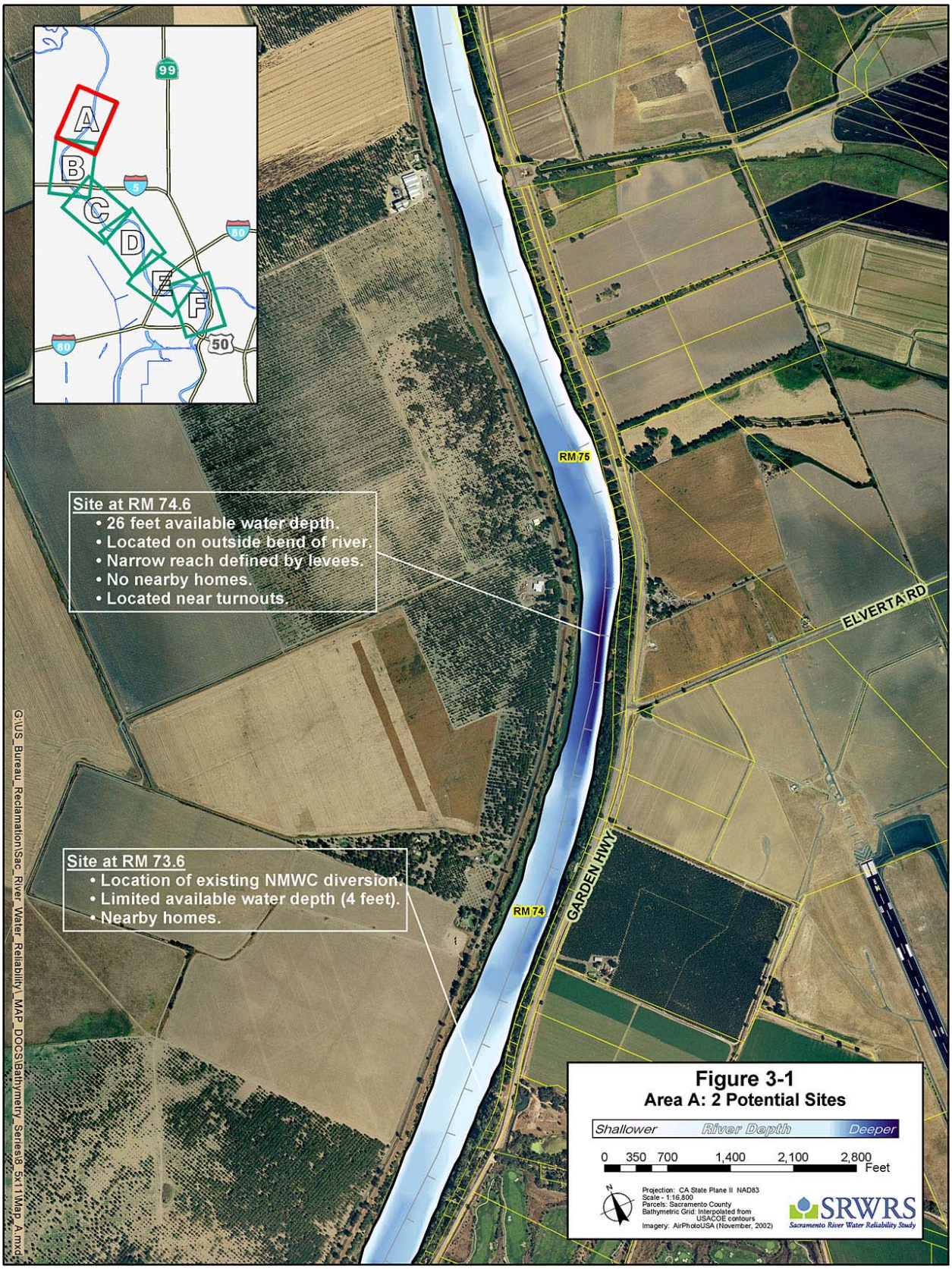


Figure 3-1 Area A: 2 Potential Sites

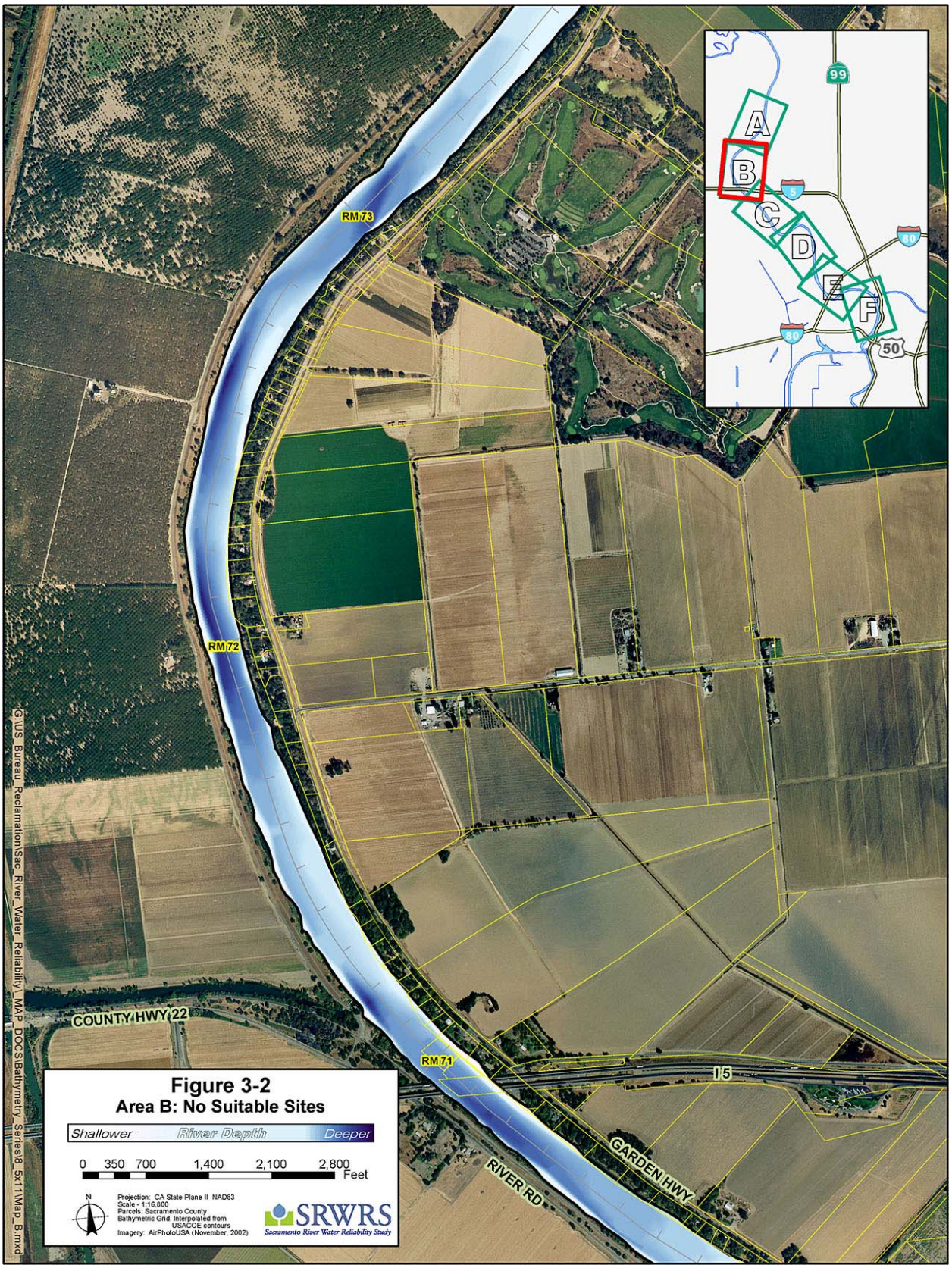


Figure 3-2 Area B: No Suitable Sites

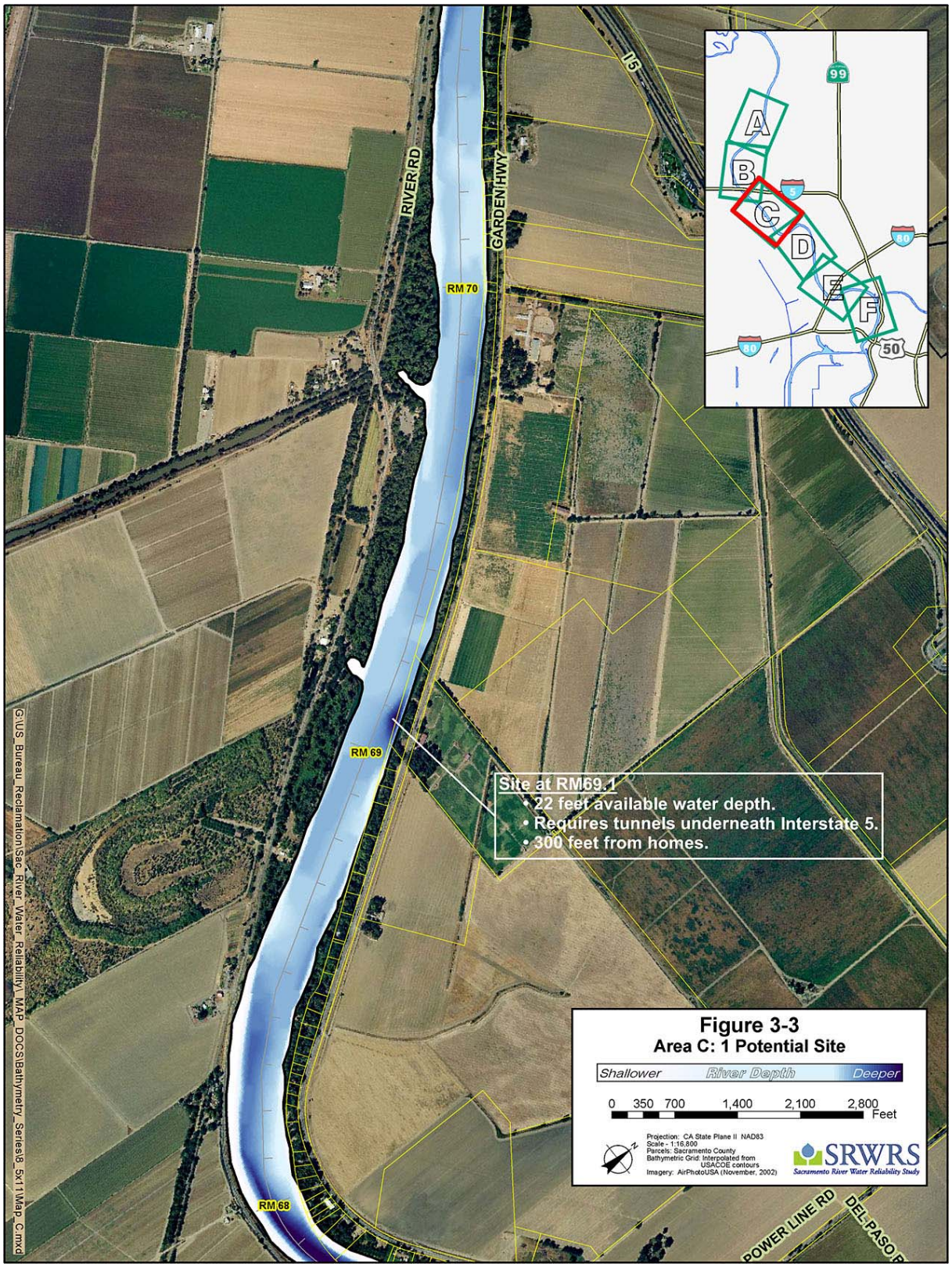


Figure 3-3 Area C: 1 Potential Site

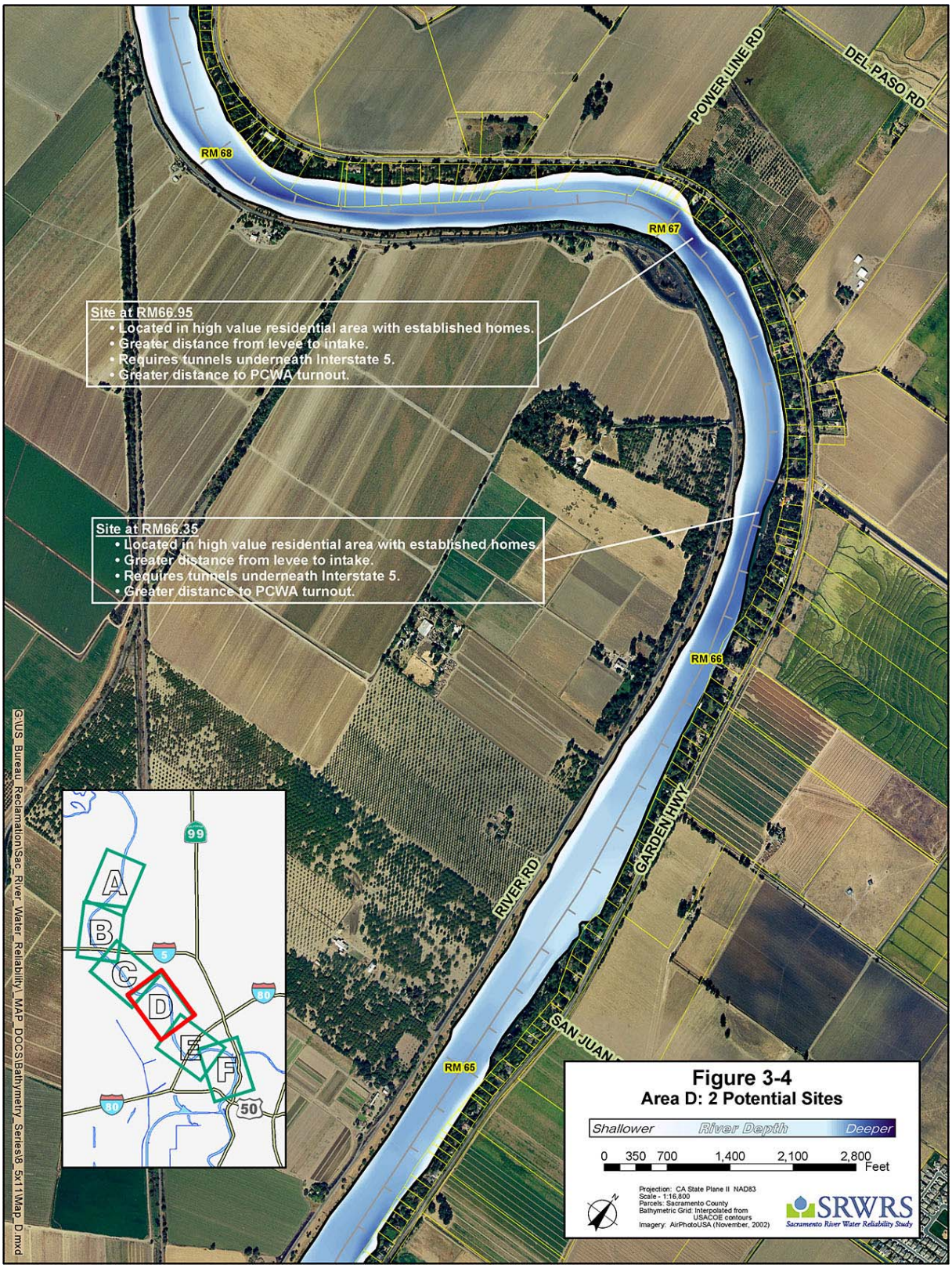


Figure 3-4 Area D: 2 Potential Sites

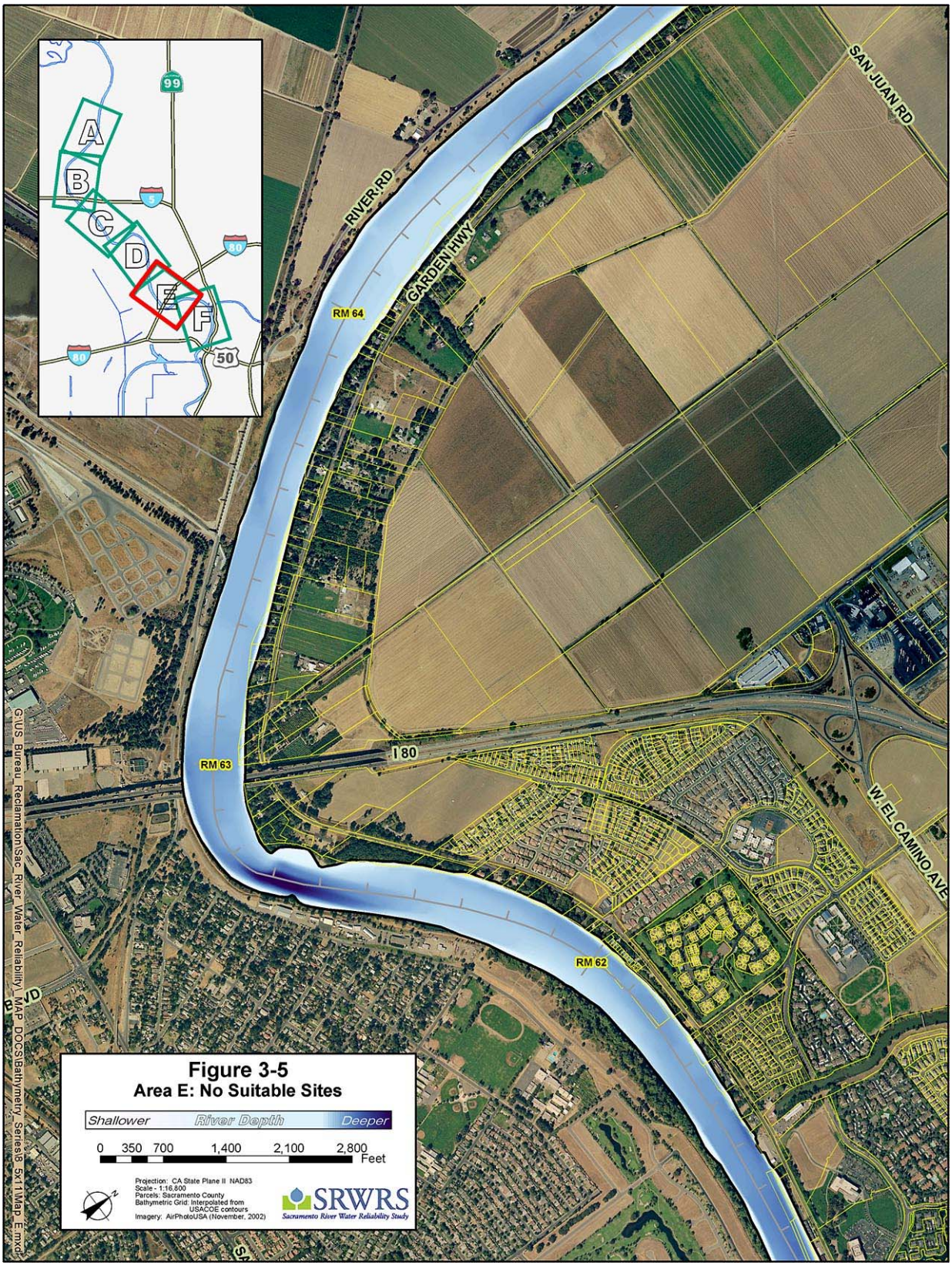


Figure 3-5 Area E: 2 No Suitable Sites

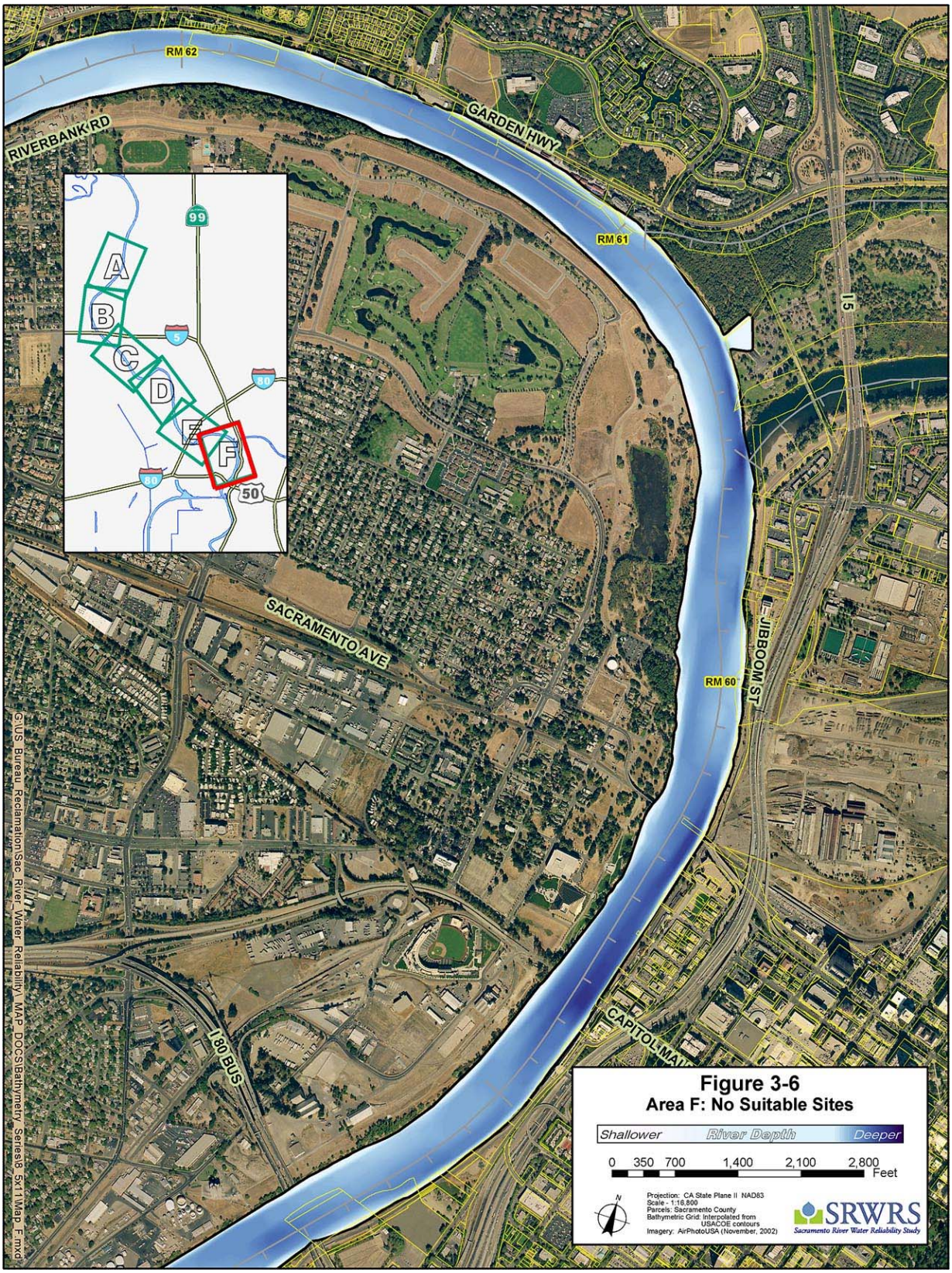


Figure 3-6 Area F: No Suitable Sites

3.2.2.1.1. Site at RM 74.6

This is the site identified in the Phase I Report, initially referred to as the Elkhorn/Elverta Diversion Alternative intake site. A photograph of this site is shown in **Figure 3-7**. Advantages of this site include the following:

- Excellent available water depth of approximately 26 feet below low water level provides significant design flexibility and potential to reduce intake footprint.
- Located on an outside bend of the river with reduced risk of sediment buildup.
- Relatively narrow river segment between defined levees limits meandering.
- Located on land owned by Sacramento County near the high-noise Sacramento International Airport and away from existing homes.
- Proximal to two SRWRS cost-sharing partners' turnout points, approximately 10 miles to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road and 22 miles to PCWA's turnout at the intersection of Athens Road and Fiddyment Road.

Figure 3-7 Potential Intake Site at RM 74.6

Disadvantages of this site include the following:

- Riparian habitat between the levee and the riverbank would be impacted by construction activities.
- Located within the Sacramento International Airport "Overflight Zone" and would require coordination with FAA.

3.2.2.1.2. Site at RM 73.6

This site is the existing NMWC intake near Elkhorn Boulevard and was initially considered in the Phase I Report because it was theorized that the cost-sharing partners could combine with NMWC to construct a new joint facility at this location. A photograph of this site is shown in **Figure 3-8**.

Figure 3-8 Existing NMWC Elkhorn Intake at RM 73.6

This site has one advantage:

- Located near cost sharing partners' turnout points; distances similar to the site identified above at RM 74.6.

Disadvantages of this site include the following:

- Site has a limited available water depth of approximately 4 feet below low water elevation, which would increase the size and complexity of the intake structure.
- Residential homes are located to the north and south of the existing NMWC diversion at an approximate distance of 1,000 feet. The new structure would be substantially larger than the existing structure and affected neighbors could object to the project and/or demand engineering, architectural, or operational restrictions to the facilities.

3.2.2.2. RM 73.2 to RM 70.3

No suitable alternatives were identified in this portion of the river (see **Figure 3-2**). Disadvantages of this river segment include the following:

- Entire segment is an inside bend of the river, with associated low velocities and sediment deposition potential.

- Significant number of homes along the bank.
- Limited water depth available.

3.2.2.3. RM 70.3 to RM 67.8

One potential site was identified in this reach (see **Figure 3-3**).

3.2.2.3.1. Site at RM 69.1

A photograph of this site is shown in **Figure 3-9**. Advantages of the site include the following:

- Water depth available below low water elevation is approximately 22 feet.
- Located on an outside river bend, with associated low sediment buildup.
- Relatively narrow river segment between defined levees limits meandering.
- Deep water is available close to the levee (approximately 150 feet), which reduces impacts to riparian habitat.

Figure 3-9 Potential Intake Site at RM 69.1

Disadvantages of this site include the following:

- A small grouping of about four homes and the Christiana Farm (a horse breeding facility) is located directly across Garden Highway from the potential intake location (approximately 300 feet). Construction and operation of the intake facility would have a significant impact on this development. In addition, as would be discussed later in this chapter, the levee road would likely need to be raised 8 to 10 feet at the intake site to accommodate an access bridge. Raising the levee would cause it to extend farther landward, further encroaching on the existing development.

- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 6 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 28 miles.

3.2.2.4. RM 67.8 to RM 64.7

Two potential locations were identified in this reach (see **Figure 3-4**).

3.2.2.4.1. Site at RM 66.95

A photograph of this site is shown in **Figure 3-10**. Advantages of the site include the following:

- Water depth available below low water elevation is approximately 29 feet.
- Located on an outside river bend, with associated low sediment buildup.
- Relatively narrow river segment between defined levees limits meandering.

Figure 3-10 Potential Intake Site at RM 66.95

Disadvantages of this site include the following:

- Located in area of high-value private property. The majority of the homes in the vicinity of the site are 1-to 2-acre parcels containing large riverfront homes.
- Relatively long distance from the levee to the intake (approximately 450 feet) increases environmental and private property impacts.

- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 5 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 27 miles.

3.2.2.4.2. Sites in the Vicinity of RM 66.35

This section of the river, approximately 1,500 feet in length, could be used for an intake facility. Photographs of this site are shown in **Figure 3-11**. Advantages of the area include the following:

- Water depth available below low water elevation is approximately 21 feet.
- Located on an outside river bend, with associated low sediment buildup.
- Relatively narrow river segment between defined levees limits meandering.

Figure 3-11 Potential Intake Sites in the Vicinity of RM 66.35

Disadvantages of this area include the following:

- Located in area of high-value private property. The majority of the homes in the vicinity of the site are 1- to 2-acre parcels containing large riverfront homes.
- Relatively long distance from the levee to the intake (approximately 450 feet) increases environmental and private property impacts.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- While the distance to Sacramento's turnout at the intersection of Del Paso Road and Truxel Road decreases to 6 miles, the distance to PCWA's turnout at the intersection of Athens Road and Fiddyment Road increases to 28 miles.

3.2.2.5. RM 64.7 to RM 61.5

No suitable alternatives were identified in this portion of the river (see **Figure 3-5**). Disadvantages of this river segment include the following:

- Majority of the segment is an inside bend of the river, with associated low velocities and sediment deposition potential.
- Significant number of homes along the bank.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Tunnel crossing of Interstate 80 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Suboptimal available water depth.
- Suboptimal distance from turn-outs.
- Challenging routing of large-diameter pipelines through highly developed areas.

3.2.2.6. RM 61.5 to RM 60

No suitable alternatives were identified in this portion of the river (see **Figure 3-6**). Disadvantages of this river segment include the following:

- Significant number of homes and businesses along the bank.
- Tunnel crossing of Interstate 5 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Tunnel crossing of Interstate 80 with both 96-inch-diameter and 72-inch-diameter pipes would be required.
- Proximity to Sacramento's existing Sacramento River WTP Intake might cause concern with regulators.
- Suboptimal distance from turn-outs.
- Challenging routing of large-diameter pipelines through highly developed area.

3.2.3. Conclusions and Site Selection

Five potential intake sites were identified on the Sacramento River between the Sutter County line and the confluence of the American River. Based on a review of the advantages and disadvantages of the sites presented above, it is clear that the site identified in the Phase I Report, located at RM 74.6, best meets the evaluation criteria presented in **Section 3.2.1**. A key advantage of this site is its location on land owned by Sacramento County that has limited uses due to its proximity to the Sacramento International Airport. The site can be developed with a manageable amount of environmental mitigation of riparian habitat. In addition, and importantly, the central location of site is with respect to the cost-sharing partners would help minimize project costs.

3.3. REFINEMENT OF RIVER HYDROLOGY

This section describes work done to determine appropriate Sacramento River water surface elevations to be used for design of the fish-screened intake at the preferred site at RM 74.6. The water surface elevation at the proposed site at any given time results from the interplay of a number of factors, including operation of the Fremont Weir and backwater affects of the American River. The methodology described below has been used successfully for design of intake structures on the Sacramento, American, and San Joaquin rivers. Additionally, initial assessments of design water surface elevations described herein should be further refined in the predesign phase of the project.

The design low water surface would define the elevation for the top of the fish screen, ensuring that it would be fully submerged and thereby ensuring also that mandated screen approach velocities would not be exceeded at the design flow rate. The 100-year flood elevation would be used to define the elevation above which in-river pumps and electrical must be located, and to define the elevation above which the underside of any access bridge must be located (with a minimum of 3 feet clearance). Intermediate water surface elevations would be developed for later use in detailed pump operation analysis. The following describes hydrologic data and modeling techniques used to determine the design water surface elevations.

River stage and flow records from USGS for the Sacramento River at Verona (Station No. 11425500, RM 78.3) were used to determine design flows for the project. An exceedence curve for Verona is presented in **Figure 3-12**. Hourly gage elevations recorded from January 1, 1990, to December 31, 2003, at Verona were averaged to obtain daily average stages. From the daily stage values, corresponding river flows were determined using Reclamation-provided rating curves. Although a larger period of record was available for flows at Verona, only the period after 1990 was used due to river system operational changes instituted for fish protection at this time as a result of the CVPIA.

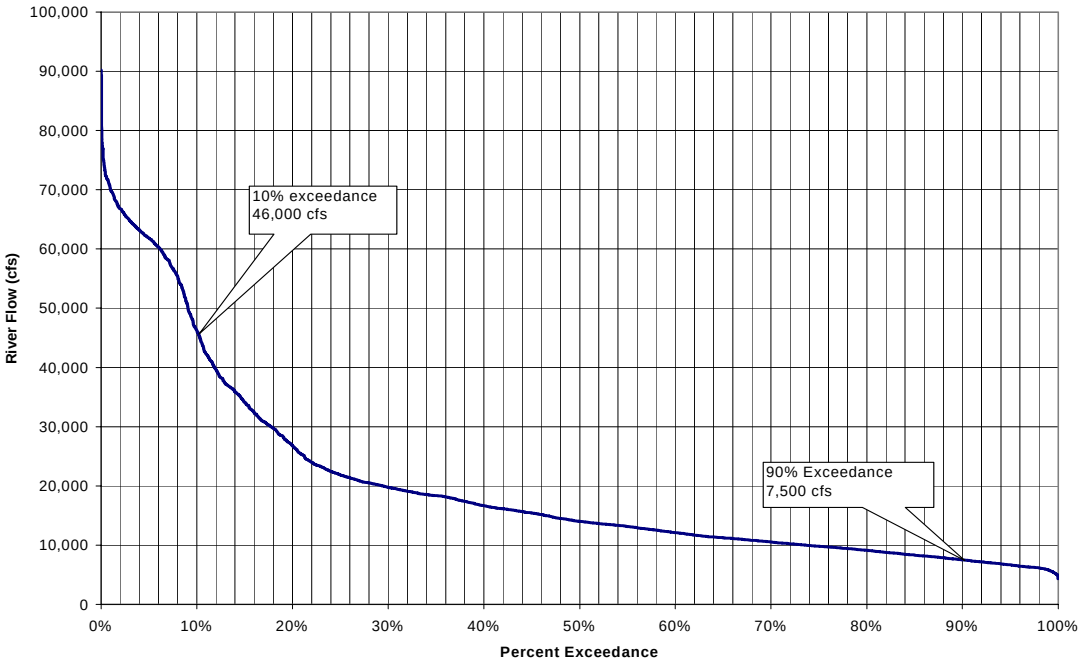


Figure 3-12 Exceedence Flow Curve for Verona (Station No. 11425500, RM 78.3)

The design low water surface elevation at the proposed site was determined by first ordering the daily stage data, in descending order, for the Verona gage station. It was noted that an elevation of 5.8 was recorded on a number of days and elevation 5.7 was recorded on several days. In addition, an elevation less than 5.7 was recorded only once in the 13-year period of record. Elevation 5.7, and its associated flow of 4,800 cfs, was selected as the low water design point.

To determine the corresponding elevation at the project site for the design low water flow determined above, a HEC-RAS computer backwater model was used. The model facilitated development of a rating curve, shown in **Figure 3-13**, for the river at the proposed Elverta Intake site. The design low water flow was then evaluated relative to the rating curve and a design low water elevation of 4.3 was established for the proposed site.

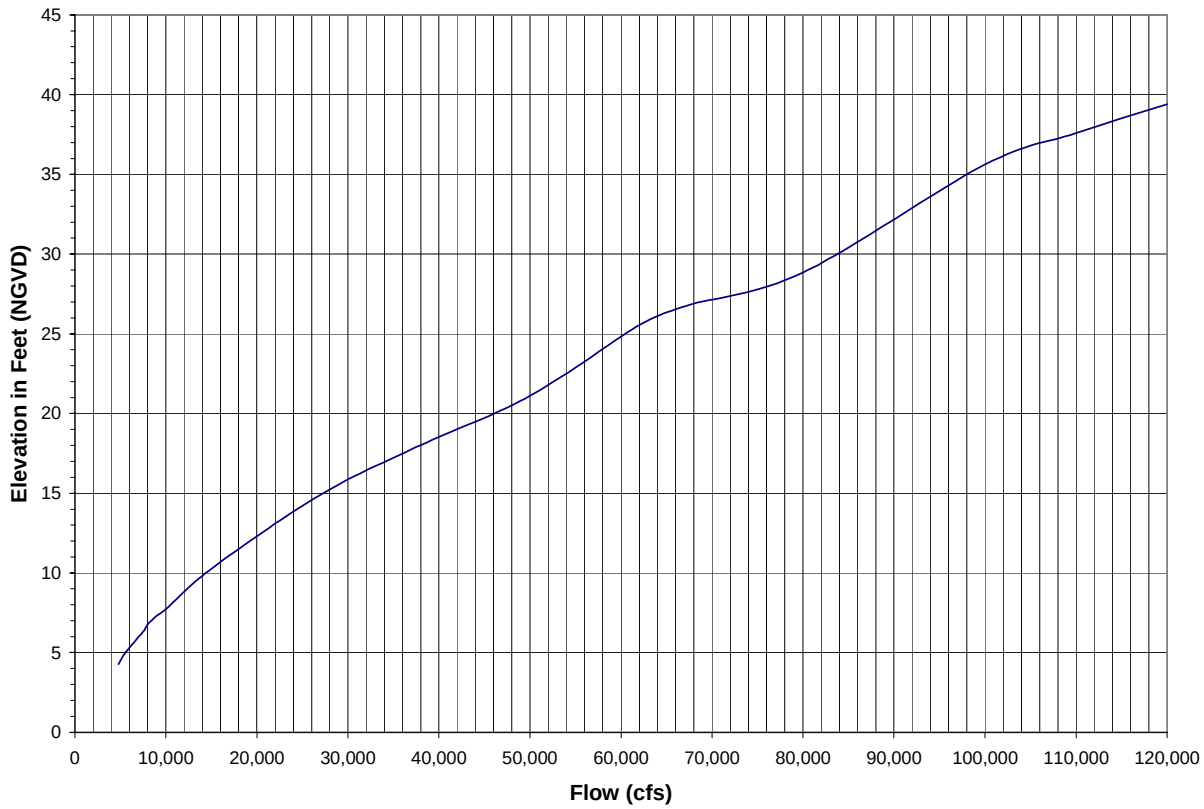


Figure 3-13 Rating Curve for the Proposed Elverta Intake Site – RM 74.6

Downstream boundary conditions used by the model were established using Sacramento River data managed at I Street (Station No. 11447500, RM 59.7). River cross section data used as the basis for the model geometry were taken from the USACE Sacramento and San Joaquin River Basins Comprehensive Study.[4] The geometry and the friction factor (Manning's n) were calibrated to match historical water surface elevations at the I Street and Verona gages.

[4] USACE. 2002. Sacramento and San Joaquin River Basins Comprehensive Study, Technical Studies Documentation. December.

The 100-year flood flow and stage, which is not impacted by CVPIA operational changes, was determined using river data for the period between 1967 and 2002. The USACE HEC-FFA (flood frequency analysis) model, based on the Guidelines for Determining Flood Flow Frequency, Bulletin No. 17B of the Hydrological Subcommittee, was used to calculate the flow. The analysis returned a 100-year flood flow of 120,000 cfs, which translated to a water surface elevation of 39.4 at the proposed Elverta site.

3.4. INTAKE CONFIGURATION EVALUATION AND SELECTION

This section presents the methodology and conclusions of the screening process for intake facility configuration alternatives for the proposed ARPS-Elverta Diversion Alternative intake site on the Sacramento River, as identified in the previous section.

3.4.1. Initial Development and Screening of Alternatives

As a first step in developing an intake configuration appropriate for the proposed project site, a design workshop was conducted and attended by MWH's leading fish-screened intake engineers. The workshop took place at the MWH offices in Bellevue, Washington, on November 17, 2003. Attendees included Dennis Dorratcague, Frank Postlewaite, and Clint Smith of the MWH Bellevue office, and Phil Salzman and M. Alejandro Salazar of the MWH Sacramento office. Workshop participants previously have been involved in design and construction of over 25 fish-screened intake facilities, ranging in size from 2 mgd to 1,600 mgd, and located in California, Washington, and Oregon.

The workshop included developing intake configuration alternatives, intake evaluation criteria, and a weighting system for the criteria, rating each alternative for its ability to meet each criterion, and scoring the alternatives based on the product of their ratings and the weightings. The methodology is summarized below.

3.4.1.1. Development of Intake Configurations Alternatives

The initial step of the evaluation process was to develop fish-screened intake structure alternatives applicable to the proposed Elverta site. Eleven conceptual alternatives were developed and are briefly described as follows:

3.4.1.1.1. Alternative 1 – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides

This alternative would incorporate an oblong-shaped in-river intake structure and pump station oriented parallel to the river, located approximately 70 feet from the river bank at average flows (see **Figure 3-14**). The structure would rise to a height of approximately 60 feet above the water surface at average flow, about 30 feet higher than the top of the levee. The pump motors and electrical equipment would be located on a deck at an elevation safely above the 100-year flood elevation. Vertical flat panels of stainless steel wedgewire fish screens would be located at the bottom of both sides of the two long walls of the structure, allowing water to flow to the pumps. Water would be pumped over the levee via pipelines located within the bridge structure required to access the intake, and would be routed to the treatment plant. As with all alternatives, the operating range of the fish screen intake structure would span from historical low flows to 100-year flood conditions.

Figure 3-14 Pier Intake and Pump Station
(Sacramento's E.A. Fairbairn Intake used as an example)

3.4.1.1.2. Alternative 2 – Pier Intake Structure with Vertical Fish Screens on Two Sides, Gravity Flow to Land-Side Pump Station

This alternative would incorporate an in-river intake structure, similar in size and orientation to Alternative 1, but with an overall height about 25 feet lower (since it would not house pumps and associated electrical gear, etc.) (see **Figure 3-15**). As with Alternative 1, vertical flat panels of stainless steel wedgewire fish screens would be located at the bottom of both sides of the two long walls of the structure. Water would flow into the structure, but unlike Alternative 1, water would flow by gravity through pipes under the levee to an underground concrete, box-shaped structure (sump), which would be about 20 feet wide by 100 feet long by 10 feet tall. Pumps located directly above the sump would then be used to direct water to the treatment plant.

Figure 3-15 Pier Intake with Land-Side Pump Station
(Sacramento's E.A. Fairbairn Intake used as an example; photo modified for illustrative purposes)

3.4.1.1.3. Alternative 3 – Pier Intake Structure and Pump Station with Vertical Fish Screens on One Side

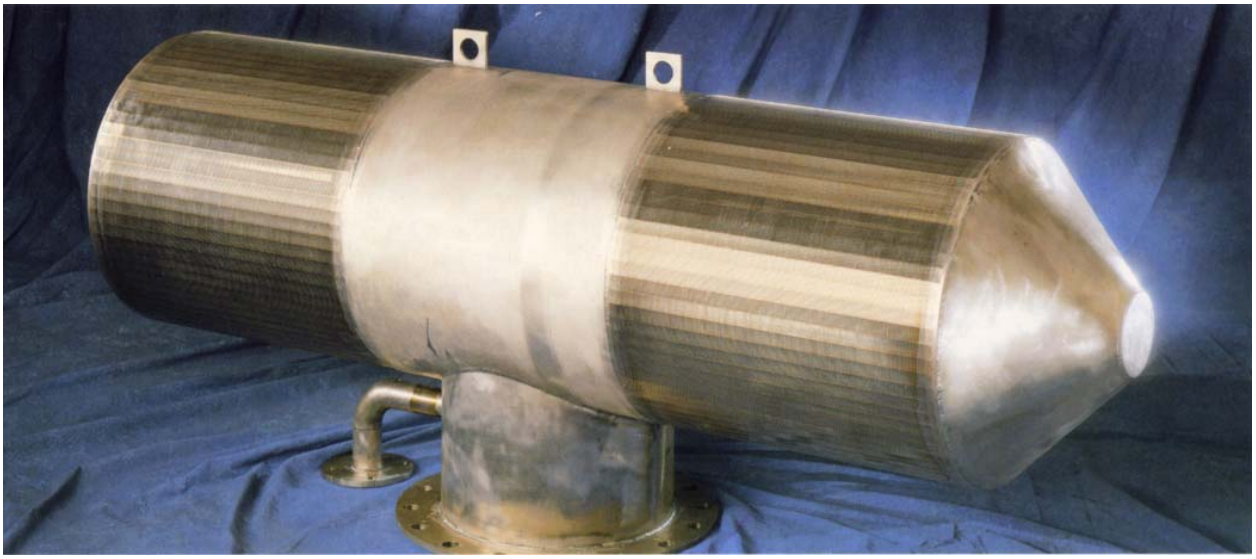
This alternative is essentially the same as Alternative 1, except that fish screens would be located at the bottom of only the river-facing long wall of the structure and not the levee-facing wall (similar to **Figure 3-14**).

3.4.1.1.4. Alternative 4 – Pier Intake Structure with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

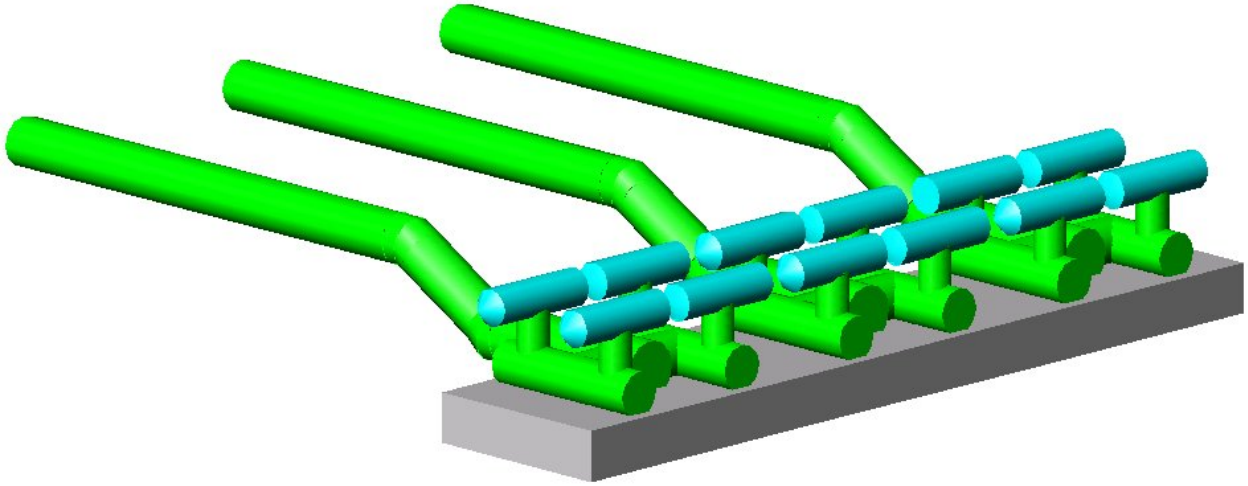
This alternative is essentially the same as Alternative 2, except that fish screens would be located at the bottom of only the river-facing long wall of the structure and not the levee-facing wall (similar to **Figure 3-15**).

3.4.1.1.5. Alternative 5 – Cylindrical Tee Screens with Land-Side Pump Station

This alternative would include a completely submerged intake structure that uses cylindrical-shaped tee screens (see **Figure 3-16**). Twelve tee screens, approximately 5 feet in diameter each and located on a concrete platform on the river bed, would be manifolded together and connected to piping routed under the levee. Water would flow by gravity through the screens and through the piping to an underground concrete sump, similar to Alternative 2. Tee screens typically use a high-energy air-burst system for cleaning, where a large volume of pressurized air would be quickly released from a land-based tank and forced through small-diameter piping into and through the screens in the reverse direction of water flow. This alternative would also include a land-side pump station similar to Alternative 2.



(a) Typical Cylindrical Tee Screen



(b) Submerged Cylindrical Tee Screen Manifold

Figure 3-16 Conceptual Plan of a Cylindrical Tee Screen

3.4.1.1.6. Alternative 6 – In-Bank Intake Structure and Pump Station with Vertical Fish Screens

This alternative is nearly the same as Alternative 3, except that the intake would be located in the bank of the river rather than out in the river (see **Figure 3-17**). However, unlike Alternative 3, this alternative would require sheet pile flow training walls upstream and downstream of the intake structure to optimize hydraulic flow conditions.

Figure 3-17 In-Bank Intake and Pump Station
(Sacramento's E.A. Fairbairn Intake used as an example; photo modified for illustrative purpose)

3.4.1.1.7. Alternative 7 – In-Bank Intake Structure and Pump Station with Inclined Fish Screens

This alternative is essentially the same as Alternative 6, except that the intake screens would be "layed back" at an angle at or near the angle of the bank rather than oriented vertically (similar to **Figure 3-17**).

3.4.1.1.8. Alternative 8 – Floating Barge with Coanda Screens and Pump Station

This alternative would incorporate a floating barge or dock, about 15 to 20 feet wide by 300 to 400 feet in length, that would adjust to the water surface elevation by sliding up and down on cylindrical steel piles set in the river. Inside the floating barge, the system would include on ogee-shaped (a flattened S-shape similar to a pool slide) coanda wedgewire fish screen. The partially submerged barge would allow water to flow over the coanda screen where any fish and solids would be screened out. This bypass flow would be pumped from the inside of the barge back to the river using a Wemco-type pump, which does not injure or kill fish. Screened water would be pumped over the levee and to the treatment plant via two to three 84- to 96-inch-diameter flexible pipes, anchored in some fashion along an access bridge provided for screen maintenance.

3.4.1.1.9. Alternative 9 - Screw Pump to Land-Side Fish Screens and Pump Station

This alternative would use three to four unscreened, inclined, 10- to 12-foot diameter, Archimedes-type pumps in the river to lift water over the levee and into a roughly 100-foot by 60-foot land-side fish screen structure and pump station. Archimedes units use a slowly rotating screw (auger) bonded within a metal cylinder to gently lift water and fish. Fish and bypass flow would be pumped or gravity-fed back into the river while screened water would be pumped to the treatment plant. An access bridge would be constructed for maintenance of the screw pumps and a required trash rack at the mouth of the screw pumps.

3.4.1.1.10. Alternative 10 - Concrete Culvert Through Levee with Land-Side Fish Screens and Pump Station

This alternative would incorporate a concrete culvert, about 12 feet by 12 feet in dimension, extending through the levee. Water would flow by gravity through the culvert to a land-side fish screen structure and pump station, similar to Alternative 9. Fish and bypass flow would be pumped back to the river using a Wemco-type pump, and screened water would be pumped to the treatment plant. An access bridge would be constructed for maintenance of a required trash rack at the mouth of the culvert.

3.4.1.1.11. Alternative 11- Ranney Collectors with In-Bank Pump Station

This alternative would include a minimum of five to six large-diameter caissons (buried vertical, concrete, pipe-shaped structures, about 24 feet in diameter) equally spaced in the bank on the river-side of the levee. Each caisson would have perforated collector piping extending, from near its base, under the riverbed in a horizontal direction. A manifold system would collect all the water into one of the caissons, which would also include a pump station. Water would be pumped over the levee and to the treatment plant via pipes integral within the pump station's access bridge.

3.4.1.2. Intake Criteria Development

Intake design criteria were first "brainstormed" and then refined. Criteria included fish protection, lower potential for damage from river debris, lower potential for levee disturbance, lower relative first cost, lower relative operation and maintenance cost, water supply reliability, lower potential for environmental impacts, technical feasibility, public safety, and security. The following paragraphs describe the criteria and key factors that caused alternatives to score well (high score) or poorly (low score) for each intake design criterion.

3.4.1.2.1. Fish Protection

Rated the potential of an alternative to ensure that fish would not be injured or killed during their separation from diverted flow. Higher (better) scores were assigned to alternatives that would not involve bypass systems that could potentially injure, disorient, or kill fish. Also, systems with shorter screen lengths scored higher due to the shorter time fish would be exposed to potentially harmful screen-induced currents.

3.4.1.2.2. Lower Potential for Damage from River Debris

Rated the potential of the fish screens, cleaning system, and structural components to avoid impacts and damage from floating or submerged objects. Higher scores were assigned to alternatives that would have fewer components in the river. Lower scores were assigned to alternatives with more, or more vulnerable, components in the river.

3.4.1.2.3. Lower Potential for Levee Disturbance

Rated the potential of the alternative to avoid physical disruption to the levee during construction of the intake structure and pump station. An alternative scored higher if minimal permanent and/or temporary levee disturbances and modifications would occur. For example, an alternative would score higher if it included pumping water over the levee rather than if it included trenching through the levee for a gravity-flow system to land-side pumps.

3.4.1.2.4. Lower Relative First Cost

Rated the potential of the alternative for lower cost of construction. An alternative scored higher if the estimated construction cost was lower relative to other alternatives.

3.4.1.2.5. Lower Relative Operation and Maintenance Cost

Rated the potential of the alternative for lower estimated cost of operation and maintenance for the intake structure, fish screens, pump station, bypass pumps, and other associated features. An alternative scored higher if the facility would be integrated into a single structure or building, and if it would involve fewer mechanical, electrical, and structural components. A higher score was also assigned to an alternative that would require fewer operators. An alternative that would require more periodic expert maintenance was assigned a lower score for this criterion.

3.4.1.2.6. Water Supply Reliability

Rated the ability of the alternative to consistently provide the desired flow. An alternative that would use proven technology received a higher score. An alternative received a lower score if it had more components that could potentially break down and/or could require a long period of time to repair.

3.4.1.2.7. Lower Potential for Environmental Impacts

Rated the potential of the alternative to avoid environmental impacts during both construction and operation and maintenance. The following subcriteria were considered in this evaluation: aesthetics, biological resources, noise, recreation, traffic, and hydrology. An alternative scored high if minimal disturbance would occur during construction and operation, and if the proposed alternative would have a relatively small size, or footprint.

3.4.1.2.8. Technical Feasibility

Rated the perceived design and construction difficulty of the alternative. An alternative similar in design to one known to have been constructed and successfully operated scored higher than an unproven design. In addition, an alternative that would require uncommon materials or equipment and/or atypical or unproven design or construction techniques scored lower.

3.4.1.2.9. Public Safety

The proposed intake would be located in an area used for sports fishing, boating, and other recreational activities. This criterion rated the degree to which the safety of the general public using this reach of the river could be impacted. An alternative scored higher if fewer elements of the alternative were in the river, thereby reducing the potential for an incident involving the public. An alternative scored lower if it had in-river facilities that were difficult to monitor and/or presented an “attractive nuisance.”

3.4.1.2.10. Security

Rated the degree to which the alternative would potentially be exposed to vandalism or terrorism. An alternative scored higher if it would have fewer components exposed or accessible, or if public access could be more easily controlled. For example, cylindrical tee screens would be completely submerged deep in the river and relatively inaccessible, so an alternative using these screens scored higher.

3.4.1.3. Weighting of Criteria

Criteria were weighted based on the consensus of workshop members regarding the relative importance of the criteria to the project. Each criterion was assigned a relative weight as a percentage, with the total for all criteria summing to 100 percent. The following bullet items summarize the rationale used to assign relative weights for each criterion. Weightings also are summarized in **Table 3-2**.

- Water supply reliability was assigned the highest relative weight of 15 percent because as a municipal and industrial (M&I) water source, it is fundamental that the alternative provide reliable water at all times.
- Lower relative first cost, technical feasibility, and fish protection were each assigned a 13 percent relative weight. The cost of the facility would obviously be a key factor for the partners, and both technical feasibility and fish protection are key aspects in successfully designing, constructing, and obtaining regulatory permits for the facility.
- Public safety and lower relative operation and maintenance cost were each assigned a 10 percent relative weight. Public safety is a key issue for all public agencies and potential liability resulting from persons accessing the facility and being injured is an important consideration. Lower operation and maintenance costs were also considered important.
- Both lower potential for environmental impacts and security were assigned a relative weight of 8 percent. While very important to the project, these criteria were considered slightly less important than public safety. It should be noted that it is the intent of this evaluation to be sensitive to environmental concerns and that a detailed, in-depth environmental assessment would be conducted subsequent to this initial evaluation.
- Lower potential for damage from river debris, assigned a relative weight of 6 percent, is an important consideration when designing an in-river structure but was considered less important than lower potential for environmental impacts and significantly less important than public safety.
- Lower potential for levee disturbance was assigned the lowest relative weight of 4 percent. Levee disturbance is an important issue relative to the difficulty of construction and the ability to obtain a Reclamation Board permit. However, this issue was considered less important than reducing potential damage to the structure from river debris and significantly less important than lower operation and maintenance costs.

Table 3-2 Criteria Weights for Intake Structure and Fish Screen Initial Screening Process

No.	Criteria	Criteria Weight (percent)
1	Fish Protection	13
2	Lower Potential for Damage from River Debris	6
3	Lower Potential for Levee Disturbance	4
4	Lower Relative First Cost	13
5	Lower Relative Operation and Maintenance Cost	10
6	Water Supply Reliability	15
7	Lower Potential for Environmental Impacts	8
8	Technical Feasibility	13
9	Public Safety	10
10	Security	8
	Total	**100**

3.4.1.4. Rating of Alternatives

Each of the eleven alternatives was scored on a scale of 1 to 5 for its ability to meet each of the 10 criteria. A score of 5 meant that an alternative had the best ability to meet the criterion, a score of 1 meant the alternative was least successful in meeting the criterion, and a score of 3 meant the alternative had an average/good ability to meet the criterion. The alternative's score for each criterion was multiplied by the criterion's weighting factor and the products were summed to obtain an overall score for each alternative. The highest (best) possible overall score was 5 and the lowest (worst) possible score was 1.

Results of the evaluation of alternatives are presented in **Table 3-3**, where alternatives are arranged from highest score to lowest score. The advantages and disadvantages of each alternative are summarized below.

Table 3-3 Initial Screening of Intake Alternatives for the Proposed Elverta Site

ALTERNATIVES	CRITERIA (Weighing Factor)										
	Fish Protection (13%)	Lower Potential for Damage from River Debris (6%)	Lower Potential for Levee Disturbance (4%)	Lower Relative First Cost (13%)	Lower Relative O&M Cost (10%)	Water Supply Reliability (15%)	Lower Potential for Environmental Impacts (8%)	Technical Feasibility (13%)	Public Safety (10%)	Security (8%)	Total Weighted Score
Alternative 6- In-bank intake structure and pump station with vertical fish screens In-bank intake structure/pump station, vertical fish screens, flow training walls upstream and downstream of intake structure, pump raw water over levee via access bridge	3	3	5	3.5	3	4	3	5	3.5	4	**3.69**
Alternative 1 - Pier intake structure and pump station with vertical fish screens on two sides Pier intake structure/pump station in river, vertical fish screens on two sides, pump raw water over levee via access bridge	4	2	5	3	3	4	4	4	3	3	**3.51**
Alternative 7 - In-bank intake structure and pump station with inclined fish screens In-bank intake structure/pump station, fish screens inclined to bank angle, flow training walls upstream and downstream of intake structure, pump raw water over levee via access bridge	3	4	5	3	3	4	3	2	4	4	**3.34**
Alternative 3 - Pier intake structure and pump station with vertical fish screens on one side Pier intake structure/pump station in river, vertical fish screens on one side, pump raw water over levee via access bridge	3	2	5	2.5	3	4	4	4	3	3	**3.32**
Alternative 5 - Cylindrical tee screens with land-side pump station Cylindrical tee screens completely submerged in-river, gravity feed raw water under levee to land-side pump station, screen access via barge	4	1	2	4	4	3	2	4	2	5	**3.31**
Alternative 10 - Concrete culvert through levee with land-side fish screens and pump station Gravity flow under levee via concrete culvert, land-side fish screen and pump station structure, gravity bypass fish back to river, access bridge for in-river trash rack required	2	5	2	2	3	4	2	4	4	4	**3.20**
Alternative 2 - Pier intake structure with vertical fish screens on two sides, gravity flow to land-side pump station Pier intake structure in river, vertical fish screens on two sides, gravity feed raw water under levee to land-side pump station, access required for maintenance	4	2	2	2	3	4	2	4	3	4	**3.18**
Alternative 11 - Ranney collectors with in-bank pump station Ranney collector caissons (minimum 4) located in bank, perforated pipes extend under river to collect raw water, gravity flow to common pumps in one caisson, pump over levee via access bridge	5	5	5	2	2	2	2	2	5	4	**3.15**
Alternative 4 - Pier intake structure with vertical fish screens on one side, gravity flow to land-side pump station Pier intake structure in river, vertical fish screen on one side, gravity feed raw water under levee to land-side pump station, access bridge required for maintenance	3	2	2	2	3	4	2	4	3	4	**3.05**
Alternative 9 - Screw pump to land-side fish screens and pump station Archimides screw pumps water and fish over levee, land-side fish screen and pump station structure, gravity bypass fish back to river, access bridge for in-river trash rack	2	5	5	3	2	2	4	2	3.5	4	**2.90**
Alternative 8 - Floating barge with Coanda screens and pump station In-river floating barge containing coanda screens and pump station, barge attached to cylindrical steel piles, fish bypass pumped directly into river, raw water pumped over levee through large-diameter flexible piping	1	1	4	1	1	1	4	1	1	1	**1.36**

Notes:
1. Alternatives were rated for each criterion on a scale of 1 to 5, with 1 being the least able to meet the criterion and 5 being best able to meet the criterion.
2. For each alternative the weighted scores for all criteria were summed to obtain a total weighted score. The highest (best) possible weighted score was 5.00 and the lowest was 1.00.
3. The evaluation table above was developed at an all-day meeting attended by several of MWH's leading fish screened intake engineers.
4. Criteria were weighted based on the group's consensus opinion of their relative importance. For example, the ability of the facility to assure a reliable water supply was weighted higher (15%) than the expected extent of levee disturbance during construction (4%).

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3.4.1.4.1. Alternative 6 (Score 3.69/5.00) – In-Bank Intake Structure and Pump Station with Vertical Fish Screens

Alternative 6 ranked well for most criteria and earned the highest overall rank in the initial evaluation. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is similar to elements of the design of Sacramento's Sacramento River WTP Intake, E.A. Fairbairn WTP Intake, and the proposed design for Reclamation District 2035's new 400-cfs intake. The large, oblong, concrete structure is durable and safe.
- Interference with navigation and recreational activities and impact on flood conveyance would be reduced due to location of the intake structure within the river bank.
- Constructability would be improved since location of the intake structure within the river bank allows direct accessibility from land, rather than from a barge or a temporary bridge.
- Levee disturbance and environmental footprint would be reduced by routing piping through the access bridge and over the levee rather than trenching through the levee.

Disadvantages:

- Upstream and downstream training walls would be required to smooth river streamlines to decrease fish swimming disruption. This would increase the environmental footprint of the structure relative to an in-river pier, and also increase impact to the shaded riverine habitat.
- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.
- Raising and regrading roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.

3.4.1.4.2. Alternative 1 (Score 3.51/5.00) – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides

Alternative 1 ranked a close second to Alternative 6. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is identical to the design of Sacramento's Sacramento River WTP Intake and E.A. Fairbairn WTP Intake. The large, oblong, concrete structure is durable and safe.
- Levee disturbance and environmental footprint would be reduced by routing piping through the access bridge and over the levee rather than trenching through the levee.

- Potential would exist for a shorter effective fish screen length, and therefore improved fish protection, since the total screen length would be split between two walls of the structure rather than a single wall.
- Disruption of the riverbank would be reduced since the structure is farther out in the river.

Disadvantages:

- This alternative would be more vulnerable to higher velocity flows and faster moving debris due to its location closer to the middle of the river.
- This alternative would be more challenging and expensive to construct since the in-river location would require work from a barge or from a temporary construction bridge and would require a more complex cofferdam design. In addition, due to the increased river depth closer to its middle, longer and/or welded segments of sheet pile would be required to construct the cofferdam.
- Longer bridge length may require in-river support piers.
- Raising and regrading of roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.

3.4.1.4.3. Alternative 7 (Score 3.34/5.00) – In-Bank Intake Structure and Pump Station with Inclined Fish Screens

Alternative 7 ranked third. The advantages and disadvantages of this alternative are essentially the same as Alternative 6 with the exceptions listed below:

Advantages:

- The angled orientation of the fish screens may improve the hydraulic flow characteristics of the structure by providing a less abrupt transition from the existing upstream bank angle to the screen "lay-back" angle.
- The angled orientation of the fish screens would provide more screen area per unit height of screen structure than a vertically oriented screen structure of the same length. This could potentially reduce the overall depth and length of the structure. However, NOAA Fisheries is considering new criteria that would only recognize the vertical projection of the slanted screen, negating this advantage.

Disadvantages:

- The inclined screen would complicate screen-cleaning system design and flow-approach velocity balancing, and may be considered somewhat less desirable by permitting agencies. In addition, the cleaning system would likely be air, which would adversely affect public safety.
- Removing the screen for maintenance or repair would require a crane and diver; design of a screen removal system may be more complex because of the inclined orientation of the screens. Although the screen removal system for Alternative 6 may ultimately require a crane and diver, the vertical orientation would allow more design flexibility and the potential that the screen might be able to slide up and out without the aid of a diver.

3.4.1.4.4. Alternative 3 (Score 3.32/5.00) – Pier Intake Structure and Pump Station with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

Alternative 3 ranked fourth. The advantages and disadvantages of this alternative are essentially the same as Alternative 1 with the exceptions listed below:

Advantage:

- Having fish screens on only one side of the structure would reduce the complexity of the screen cleaning system and potentially the screen removal system. The reduction in complexity would simplify the design and maintenance of both the cleaning system and the fish screens.

Disadvantage:

- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.

3.4.1.4.5. Alternative 5 (Score 3.31/5.00) – Cylindrical Tee screens With Land-Side Pump Station

Alternative 5 ranked fifth. The advantages and disadvantages of this alternative are listed below:

Advantages:

- Proven cylindrical tee screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Successful intake structure configuration is similar to the 150 cfs M&T/Parrott intake on the Sacramento River near Chico.
- Cylindrical tee screens would offer a significantly reduced structure on the river and associated reduction in first cost.
- Operation and maintenance difficulty and costs would be reduced since the design has no moving parts in the river.
- The submerged configuration of the screens would afford better security for the facility, as would the more easily monitored land-side pump station.

Disadvantages:

- The screens' exposure to submerged or partially submerged river debris, such as trees and logs, makes them more susceptible to damage.
- The pipeline carrying water from the intake to the land-side pump station would be trenched under and through the levee. An excavation of the depth and width required to install the pipe under the levee (up to 44 feet deep and 50 to 200 feet wide) would significantly increase the footprint of environmental disturbance for the project.
- Access to the screens would be limited. If an individual screen were damaged, divers and a barge with a crane would need to be mobilized for repairs. In addition, it would be more difficult to temporarily close off or isolate a damaged screen as compared to a flat-vertical screen system with a blank steel plate that could be positioned over the opening. These characteristics would reduce system reliability. Routine annual inspection of the screens would also require divers in the deep, fast-moving water and would be relatively expensive.

- It could be difficult to isolate boaters and jet skiers from the area above the submerged intake. Public safety issues may arise if recreational water users were above the screens when the high-energy air–burst system was activated, or if a boat anchor accidentally snagged a screen.
- Although cylindrical tee screens are a proven technology, they have typically been used on smaller diversions about half the size of the proposed diversion. New, "scale-up" problems not previously encountered could occur.
- The majority, if not all, of the tee screen river installations have been for agricultural facilities. These facilities have greater flexibility in diversion rate and can more easily schedule "down-time" if screens are damaged. This is not the case for a municipal facility.

3.4.1.4.6. Alternative 10 (Score 3.20/5.00) - Concrete Culvert through Levee with Land-Side Fish Screens and Pump Station

Alternative 10 ranked sixth. The advantages and disadvantages of this alternative are listed below:

Advantage:

- Vulnerable and maintenance-intensive facility components are moved out of the river to a protected land-side location.

Disadvantages:

- Unlike the cylindrical tee screen configuration of Alternative 5, an access bridge, with associated cost, would still be required for maintaining the trash rack located at the entrance of the culvert through the levee.
- The conduit carrying water from the intake to the land-side fish screen and pump station would be trenched under and through the levee. An excavation of the depth and width required to install the conduit under the levee would significantly increase the footprint of environmental disturbance for the project.
- Fish protection would be reduced since the fish, along with a permit-required quantity of bypass flow, would either be pumped or would gravity-flow through piping back to the river.
- The required bypass system would also increase the initial facility cost, operation and maintenance costs and complexity, and regulatory scrutiny and reporting requirements.

3.4.1.4.7. Alternative 2 (Score 3.18/5.00) – Pier Intake Structure with Vertical Fish Screens on Two Sides, Gravity Flow to Land-Side Pump Station

Alternative 2 ranked seventh. This alternative is similar to Alternative 1 with respect to intake structure shape and fish screen orientation, but is similar to Alternative 10 regarding gravity flow through the levee and pump location. The advantages and disadvantages of this alternative are as follows:

Advantages:

- Proven vertical fish screen technology is approved by the anadromous fish technical team as a safe screening system for the target fisheries.
- Intake structure configuration is similar to the successful design of Sacramento's Sacramento River WTP Intake and E.A. Fairbairn WTP Intake. The large, oblong, concrete structure is durable and safe.

- Potential exists for a shorter effective fish screen length, and therefore improved fish protection, since the total screen length is split between two walls of the structure rather than a single wall.

Disadvantages:

- This alternative is more vulnerable to higher velocity flows and faster moving debris due to its location closer to the middle of the river.
- This alternative would be more challenging and expensive to construct since the in-river location would require work from a barge or from a temporary construction bridge and would require a more complex cofferdam design. In addition, due to the increased river depth closer to its middle, longer and/or welded segments of sheet pile would be required to construct the cofferdam.
- Longer bridge length may require in-river support piers.
- Raising and regrading of roughly 1,200 feet of the levee road (Garden Highway) would be required to facilitate the transition between the road and the access bridge.
- The conduit carrying water from the intake to the land-side pump station would be trenched under and through the levee. An excavation of the depth and width required to install the conduit under the levee would significantly increase the footprint of environmental disturbance for the project.
- Maintenance of mechanical equipment at two locations, the in-river intake and the land-side pump station, would be required.
- Since a land-side pump station would be required in addition to the large, expensive in-river intake, initial facility cost would increase.

3.4.1.4.8. Alternative 11 (Score 3.15/5.00) - Ranney Collectors with In-Bank Pump Station

Alternative 11 ranked eighth. The advantages and disadvantages of this alternative are listed below:

Advantages:

- This alternative would have very few accessible or vulnerable components in either the river or on the protected side of the levee.
- Levee disturbance would be reduced by routing piping through the short access bridge and over the levee rather than trenching through the levee.

Disadvantages:

- The caissons and perforated under-river piping would be technically challenging to design at the scale required for the proposed project. The concept has been successfully used but only on facilities one-half to one-quarter of the size of the proposed facility.
- The large caissons would be challenging and expensive to construct due to the excavation required and the fabrication and positioning of the large circular sections of concrete.
- The tunneling required for the numerous under-river collector pipes would be expensive.
- The flow rate to the collector pipes would be difficult to predict and could change over time due to fine particle migration and resulting clogging.
- Construction of the caissons in the sensitive riparian environment on the river-side of the levee would have increased negative environmental impacts relative to other alternatives.

3.4.1.4.9. Alternative 4 (Score 3.05/5.00) – Pier Intake Structure with Vertical Fish Screens on One Side, Gravity Flow to Land-Side Pump Station

Alternative 4 ranked ninth. The advantages and disadvantages of this alternative are essentially the same as Alternative 2 with the exceptions listed below:

Advantage:

- Having fish screens on only one side of the structure would reduce the complexity of the screen cleaning system and potentially the screen removal system. The reduction in complexity would simplify the design and maintenance of both the cleaning system and the fish screens.

Disadvantage:

- Locating fish screens on only one side of the structure rather than both sides could cause the overall structure to be longer, potentially increasing cost and increasing the amount of time fish are exposed to the screens.

3.4.1.4.10. Alternative 9 (Score 2.90/5.00) – Screw Pump to Land-Side Fish Screen and Pump Station

Alternative 9 ranked tenth. The advantages and disadvantages of this alternative are as follows:

Advantage:

- This alternative could potentially meet minimum project criteria.

Disadvantages:

- Large, 10- to 12-foot diameter rotating cylinders could create an “attractive nuisance” on the river, negatively impacting public safety and facility security.
- An access bridge, with associated cost, would be required for maintaining the pumps and the trash racks located at the pump inlets.
- Fish protection would be reduced since fish would be pumped twice: once in the screw pump and again after screening in the land-side facility.
- Operation and maintenance costs would increase since the water would be pumped twice: once in the screw pumps to get over the levee, and again to the treatment plant using a different set of pumps.
- Operation costs would increase due to the requirement that the screw pumps would have to pump 5 to 10 percent more than the desired flow to provide water to pump fish back to the river.
- The technical feasibility of using screw pumps in this application is in question due to wide variation in river levels (head conditions) relative to the effective flow range of this type of pump.
- The screw pumps would be very susceptible to damage from floating debris.

3.4.1.4.11. Alternative 8 (Score 1.36/5.00) – Floating Barge with Coanda Screens and Pump Station

Alternative 8 ranked last. This alternative ranked well for environmental impacts, since it would not significantly disturb the bank, levee, or surrounding areas. However, the alternative rated poorly for all other criteria. The advantages and disadvantages of this alternative are listed below:

Advantage:

- Environmental footprint would be reduced in the sensitive riparian zone.

Disadvantages:

- Coanda fish screening technology is not currently approved by the governing regulatory agencies.
- Pumping fish would be required, which is less attractive to regulatory agencies.
- The large barge required would be difficult to secure from the public and could create an "attractive nuisance" on the river, negatively impacting public safety. The ease of access by recreational boaters for vandalism and accidental injury would be a significant concern.
- The barge and flexible transmission pipes would be very susceptible to damage from floating debris.
- Costs are expected to be high due to the unique "one-time" nature of the system components.
- The technical feasibility of using large-diameter flexible piping to pump continuously at the design flow rate and for this novel in-river application is in question due to material properties and availability of product in the marketplace.
- The complex pumping control system required to maintain barge depth while simultaneously diverting flow and bypass pumping fish is expected to be expensive and could jeopardize the reliability of the facility.

3.4.1.5. Conclusions of the Initial Intake Configuration Evaluation

The alternatives presented in the previous section include the full range of potential configurations for a fish-screened intake facility at the proposed site on the Sacramento River. It is clear from the ranking and by inspection of the advantages and disadvantages that certain intake configurations are obviously infeasible for the intake site and can be eliminated from further consideration.

Alternatives that can be directly eliminated include **Alternative 8 – Floating Barge with Coanda Screens and Pump Station, Alternative 9 – Screw Pump to Land-Side Fish Screens and Pump Station,** and **Alternative 11 – Ranney Collectors with In-Bank Pump Station**. These alternatives all have significant technical drawbacks. In addition, even if all of the technical difficulties could be successfully overcome, designs would be unique and unproven. Therefore, regulatory agencies would likely require extensive testing and monitoring and ultimately might not grant a permit for construction until the designs were pilot-tested at a smaller scale. While advanced design and testing of these alternatives is certainly possible, little benefit would accrue since there are no perceived cost savings, and better-suited, proven configuration alternatives are available.

Alternatives 2 and 4 – Pier Intake Structure with Vertical Fish Screens on Two Sides and One Side, respectively, **Gravity Flow to Land-Side Pump Station** can also be removed from further consideration. These alternatives have two fundamental flaws for the proposed project: greater cost and greater environmental impact than necessary. Greater costs would be incurred from construction of both the large, sturdy intake structure and a land-side pump station. It would be more cost-effective for the current project to simply add pumps to the intake structure and save the cost of the additional land-side pump station. This type of alternative is generally more attractive to agricultural facilities that require very little lift of the water (low head) and can save operating costs by not using energy to pump up over the levee. Since the proposed project requires a water elevation higher than the levee at the WTP, there would be no operational power savings. Greater environmental impacts would be caused by the excavation required to trench a gravity conduit through the levee. The trench through the levee would be up to 44 feet deep.

Excavation at this depth would require a width of 50 to 200 feet, depending on the contractor's methods, for a length of approximately 320 feet. The excavation footprint could be significantly reduced by routing the required conduit through the proposed access bridge structure above the levee and moving water via pumps relocated to the intake structure, in a configuration similar to Alternatives 1 and 6.

Alternative 10 – Concrete Culvert Through Levee with Land-Side Fish Screens and Pump Station can also be eliminated from further consideration. This "through levee" alternative has the same large environmental footprint as Alternatives 2 and 4 and is less "fish friendly." Pumping fish has significantly more impact than screening fish out as they swim by an intake structure. Governing agencies are not likely to permit a "fish-pumping" alternative if a reasonable "non-pumping" alternative is available. While this alternative does avoid the expensive, redundant in-river intake structure of Alternatives 2 and 4, it would have higher long-term costs due to maintenance of the fish pumping system and associated agency reporting requirements. In addition, an access bridge required for routine maintenance of the in-river trash rack at the mouth of the culvert would increase costs.

Alternative 5 – Cylindrical Tee Screens with Land-Side Pump Station would reduce the cost and complexity of in-river components of the facility. In addition, the land-based, air-burst screen cleaning system would require relatively little maintenance. Key reasons this alternative is not recommended for the proposed project are system reliability, liability, and environmental/levee impacts.

As noted previously, the screens are vulnerable to damage by submerged and partially submerged river debris. Although the screens can be replaced, the time required to mobilize a diver and barge with a crane could be several days. Typical manifolding of the tee screens makes isolating a defective screen from the functional screens difficult, and regulatory agencies may require a significant reduction in diversion rate during the days required to replace the screens.

Although the screens are located below the water surface, the area above the screens must be isolated from recreational river users. Screens can be damaged by boat anchors, and the air-burst cleaning system creates a large eruption of air that rises 2 to 4 feet out of the water and could potentially destabilize a boat or jet ski. While it is possible to separate this area by buoys, it would be difficult to police the area and liability from boater injury could be significant.

The environmental impacts associated with trenching through the levee for the conduit to the land-side pump station were noted in the discussion of Alternatives 2 and 4 above. As also previously stated, energy cost savings associated with a gravity flow under the levee would not apply to the proposed project and impacts of trenching activities could be avoided under other more suitable alternatives. In-river cylindrical screens could be a cost-effective solution for a flexible, agricultural-oriented application but are not recommended for the proposed project where reliability of drinking water supply limits flexibility of operation.

Alternatives 1 and 3 – Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides and **on One Side,** respectively, are relatively minor variations of the same concept. Neither alternative has a significant characteristic or component that makes it unsuitable, and either could be used for the proposed project. Typically, it would be assumed that having screens on only one side would necessitate a longer structure to achieve the required minimum screen area, making the alternative less desirable. However, due to the unusually deep water available at the proposed project site, differentiating the relative advantages and disadvantages of these two configurations would require a more detailed design layout. Therefore, both alternatives were retained for further consideration.

Alternatives 6 and 7 – In-Bank Intake Structure and Pump Station with Vertical Fish Screens and **with Inclined Fish Screens,** respectively, are also relatively minor variations of the same concept. While both configurations could be used at the proposed project site, the vertically oriented screens of Alternative 6 are superior to the inclined screens of Alternative 7. Inclined screens have a distinct advantage at locations where little water depth is available since they provide more effective screen area per unit height and can therefore reduce the overall length of screen, and structure, required. This could be a significant cost savings in low water conditions. However, ample available water depth exists at the proposed site negating this benefit and, as previously noted, proposed new NOAA design criteria may only recognize the vertical projection of the slanted screen. In addition, cleaning methods for inclined screens are limited to high pressure water jets or the relatively unproven forced air jets. Vertical screens can also be cleaned by the proven effective traveling brush cleaning system. In addition, the vertical screen orientation would more readily accommodate the flow baffle (adjustment) system required to accurately balance the flow across the entire surface of the screen to avoid "hot spots" that could negatively affect fish. For the above reasons, Alternative 7 was eliminated and Alternative 6 was retained for further consideration.

3.4.2. Comparison of Final Alternatives and Selection of Preferred Alternative

Plan and section drawings of each of the three remaining alternatives are presented in **Figures 3-18** through **3-20**. These concept-level layouts were used to compare the relative dimensions of the Intake and relative advantages and disadvantages of the alternatives. The layouts were developed using the design criteria presented in **Section 3.1** and Hydraulic Institute standards for pump intake design. Approximate pump suction bell diameters were determined based on flow rates and were used to develop facility dimensions based on Hydraulic Institute dimensional requirements. Conservative average values were used to facilitate comparison of alternatives. It was assumed that dividing walls would be placed between pumps to reduce potential disruptive flow patterns while minimizing the separation distance required between the pumps. The intake could be designed without dividing walls but separation between pumps would need to be increased on the order of 50 percent, depending on pump flow rate. The use of fewer, larger capacity pumps may also be considered during the preliminary design phase of the project. Fish screen widths and heights provide sufficient screen area such that each bay can accommodate a 33-mgd pump, and were additionally selected based on reasonable handling size for strength and constructibility, as well as adaptability to the screen cleaning system. Final selection of optimum fish screen widths and heights will be completed during the preliminary design phase of the project, in conjunction with final selection of pump size and number.

Through the project development, cost-sharing partners recognized the need to include an additional evaluation criterion of Facility Design Flexibility to account for potential changes in conditions after the current reconnaissance level of engineering work is completed. Examples of these potential changes include that FWS may consider expanding the 0.2 feet per second (fps) fish screen approach velocity criterion for delta smelt protection outside of their currently designated habitats, and that SAFCA is evaluating construction of setback levees upstream of and near the proposed Elverta Intake site.

Results of the comparison of the three final intake configuration alternatives are presented in **Table 3-4**. The three alternatives compared are all viable designs for the proposed site. Alternative 1 is a proven design, with installations on both the Sacramento and American rivers, and Alternatives 3 and 6 are slight variations of that design. However, Alternative 1 appears to be the best choice for the proposed site. This alternative is slightly more complex than Alternative 3 from a mechanical perspective, but provides the greatest flexibility to accommodate potential regulatory or physical site changes, as described in the preceding paragraph. Of the three alternatives, the design of the two-sided structure could most easily accommodate either an increase in total screen area or a reduction in screen depth. The one-sided

structures of the other two options, however, would require an excessively tall screen or increased facility footprint to accommodate screen area or depth changes.

In addition, Alternative 1 has a smaller footprint relative to Alternative 6, thereby causing less environmental disturbance. Also, although Alternative 6 is believed to have the lowest construction cost due to the potential for lower-risk, land-based construction, future structural analysis may reduce this benefit if a greater number of piles are required to balance seismic forces generated by the adjacent stream bank. Although Alternatives 3 and 6 would have a slightly less complex screen cleaning system since it would only be located on one side of the structure, this advantage is offset by the proven design record and greater design flexibility of Alternative 1.

Based on the advantages discussed above and shown in **Table 3-4**, Alternative 1- Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides will be retained as the preferred intake configuration for the Sacramento River portion of the ARPS-Elverta Diversion Alternative.

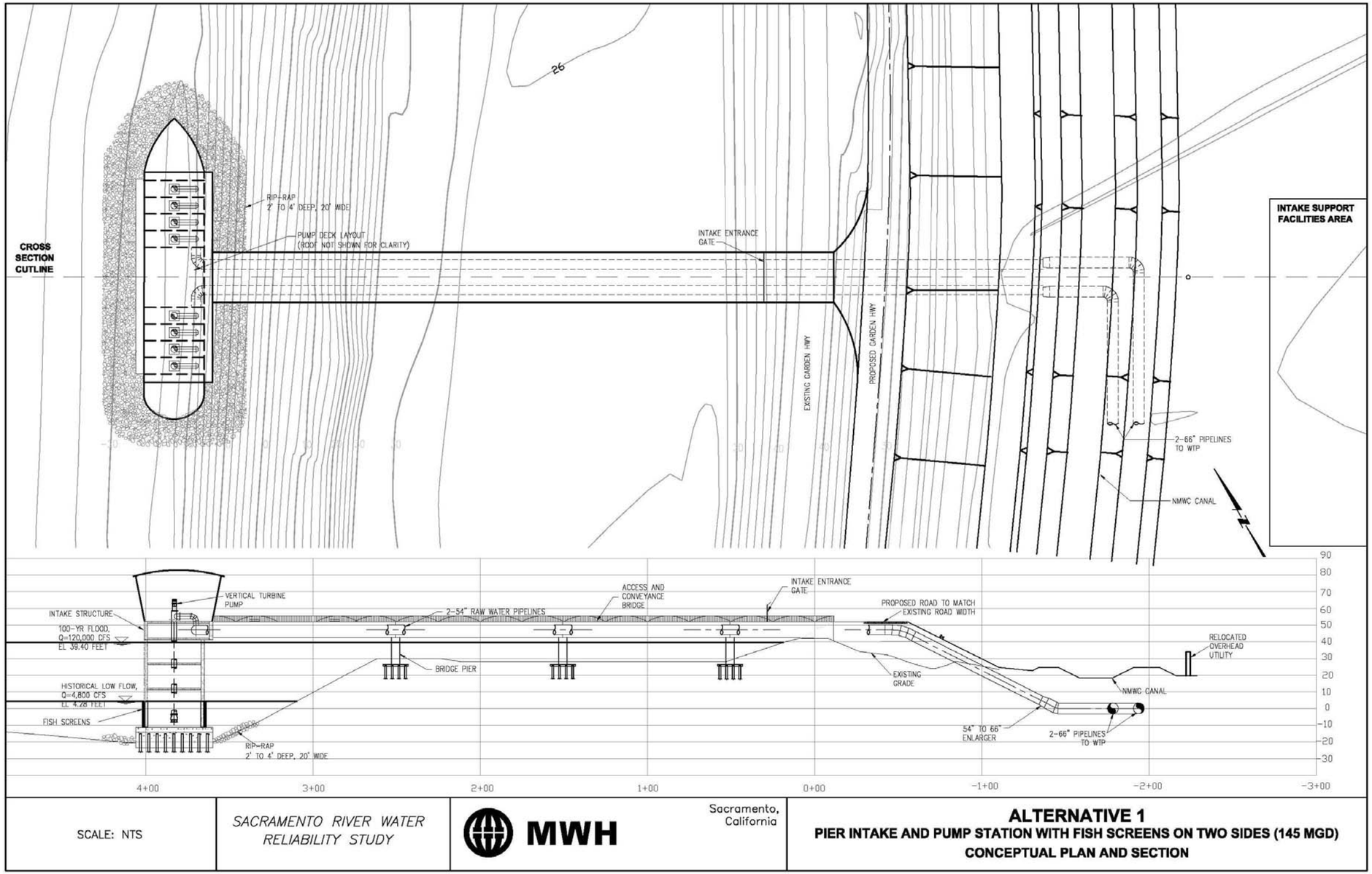

Figure 3-18 Alternative 1 Pier Intake Structure and Pump Station with Vertical Fish Screens on Two Sides (145 mgd) Conceptual Plan and Section

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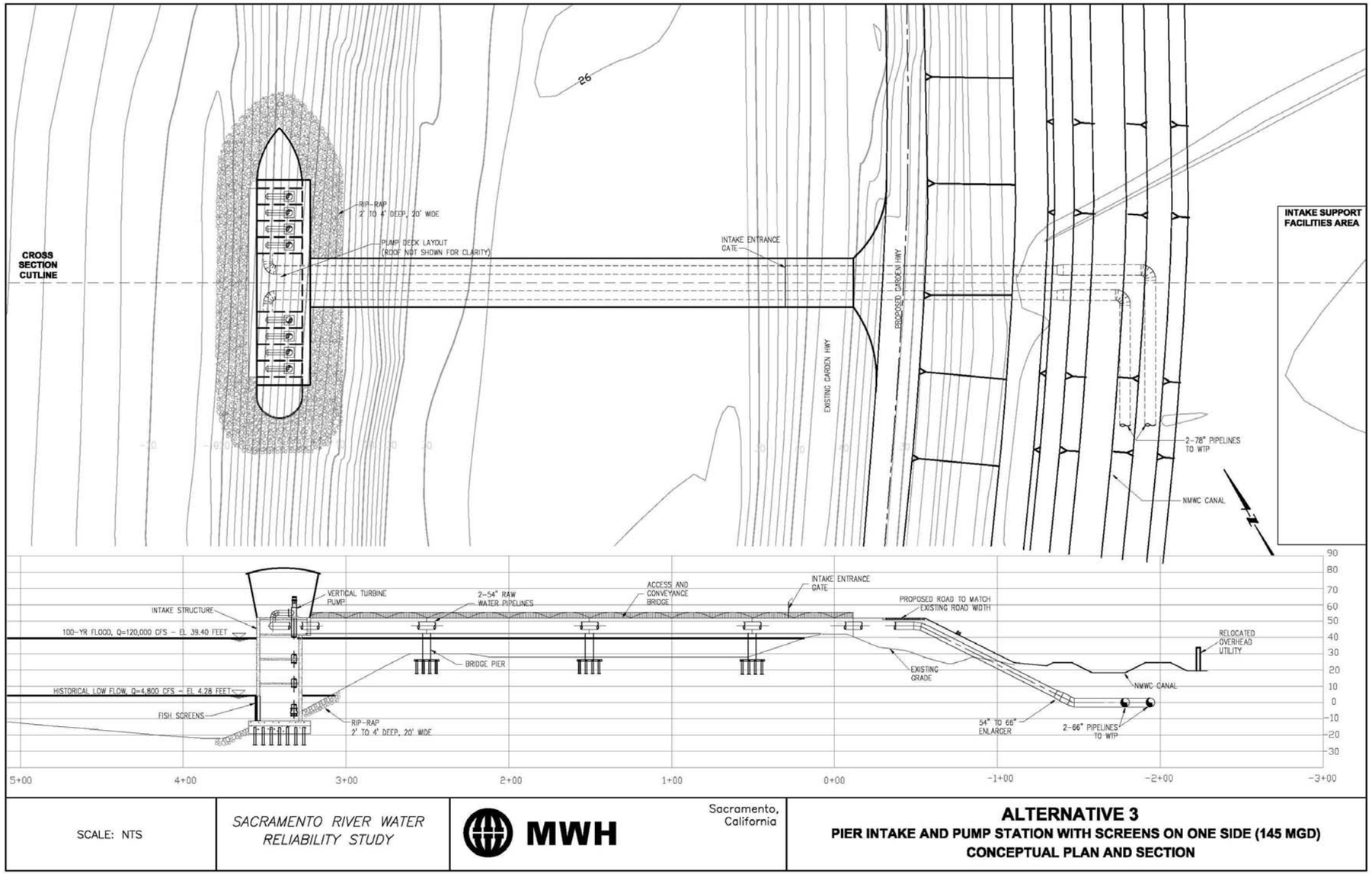

Figure 3-19 Alternative 3 Pier Intake Structure and Pump Station with Vertical Screens on One Side (145 mgd) Conceptual Plan and Section

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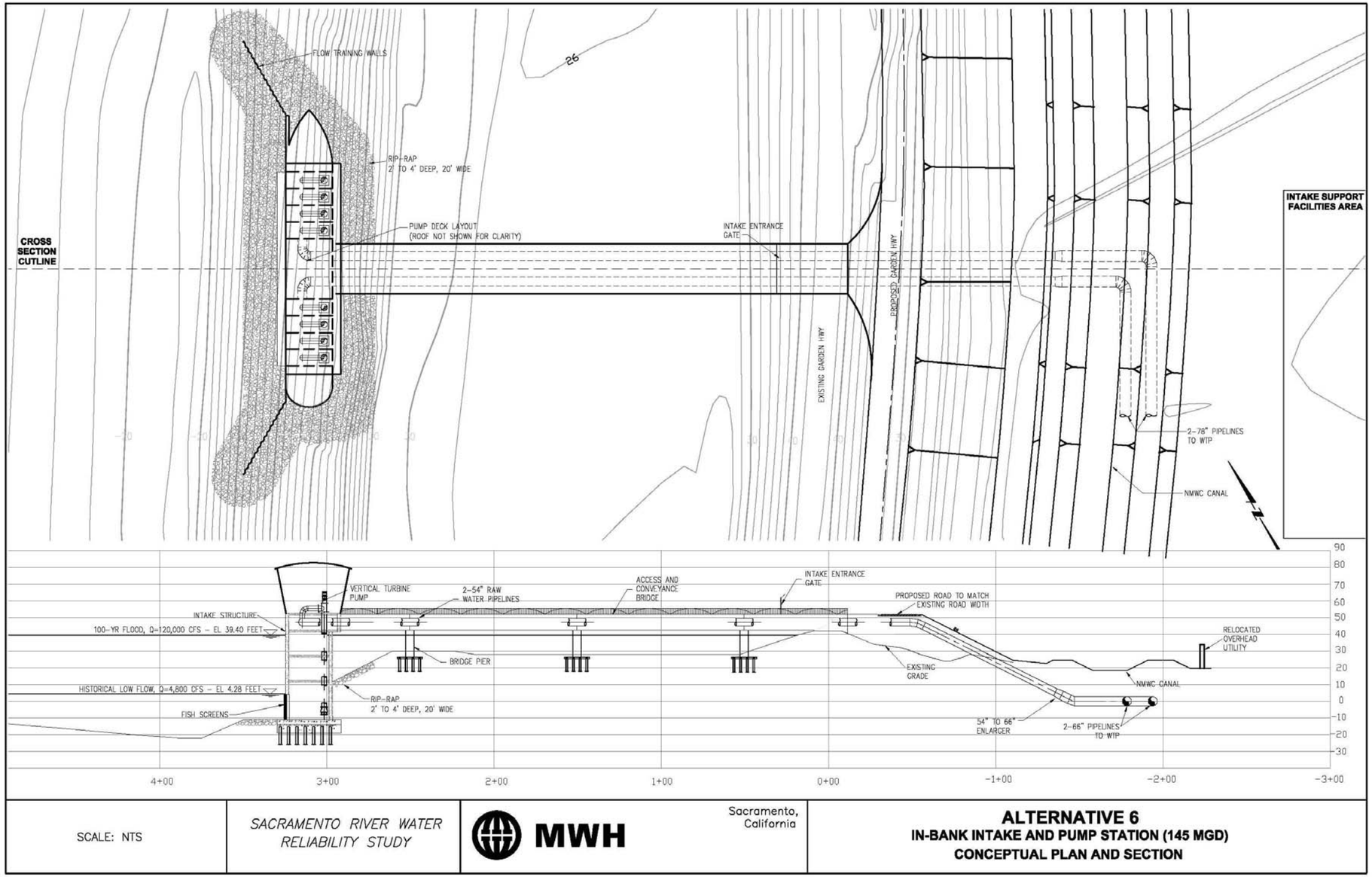

Figure 3-20 Alternative 6 In-Bank Intake Structure and Pump Station with Vertical Fish Screens (145 mgd) Conceptual Plan and Section

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Table 3-4 Comparison of Final Intake Configuration Alternatives

Comparison Item	Alternative 1: Two-Sided Pier	Alternative 3: One-Sided Pier	Alternative 6: In-Bank
Intake Structure Dimensions (feet)	Length (at base) = 180 Width (at base) = 36	Length (at base) = 180 Width (at base) = 28	Length (at base) = 180 Width (at base) = 28
Fish Screen Quantity and Average Dimensions (feet)	16 Screens Width = 8 Height = 10	8 Screens Width = 8 Height = 20	8 Screens Width = 8 Height = 20
Length of Bridge (feet)	3,375	325	305
Overall Intake Environmental Footprint (acres)	Area (at base, including riprap) = 0.43	Area (at base, including riprap) = 0.39	Area (at base, including riprap) = 0.48
Distance from Top of Bank to River-Side of Intake (feet)	105	65	40
Advantages	• Design and construction similar to two existing Sacramento facilities • Offers most flexible design to accommodate regulatory or physical site changes	• Reduced bridge length • Screens on one side reduce complexity of cleaning system and improve screen accessibility	• Intake location may allow simpler, less expensive construction from land • Reduced bridge length • Screens on one side reduce complexity of cleaning system and improve screen accessibility
Disadvantages	• Longest bridge • More screens and screen cleaners to maintain • More difficult construction relative to Alternative 6	• More difficult construction relative to Alternative 6 • Footprint would increase significantly to accommodate regulatory or physical site changes	• Larger environmental footprint in sensitive area • More in-river excavation required • More complicated structural design and more piles required due to unbalanced soil loading on intake • Footprint would increase significantly to accommodate regulatory or physical site changes
Estimated Relative Construction Cost	Highest	Lower	Lowest

3.5. POWER, SEWER, STORM DRAINAGE, AND SPECIAL CONSIDERATIONS AT THE PROPOSED INTAKE FACILITY

In this section, details are discussed of power feed and supply for both the intake facility required for the ARPS-Elverta Diversion Alternative. Wastewater facilities, stormwater management, and coordination with the FAA and Sacramento County Airport Service are also described. Additional or modified requirements for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative are presented in **Section 3.7**.

3.5.1. Power Feed and Supply at 145 mgd Facility with Cost-Sharing Partners Only

Aspects of power feed and supply for this facility would include power requirements, availability, and reliability, motor starter requirements, backup options, and dual feed options.

3.5.1.1. Primary Power Requirement and Availability

The maximum power requirement for the 145 mgd Elverta Intake Facility has been estimated to be 2,300 kilovolt-amperes (kVA). **Table 3-5** generally summarizes power requirements.

Table 3-5 Power Requirement Summary for 145 mgd Facility

Intake Pump Station	Peak Flow (mgd)	Pump Load[(1)] (hp)	Misc. Loads (kVA)	Power (kVA)	Amps @ 4,160 Volts	1/2 Load (kVA)
Intake Facility	145	2,050	250	2,300	319	1,150

Notes:
[(1)] Includes a spare pump.
Key:
hp – horsepower kVA – kilovolt-ampere mgd – million gallons per day

Power would enter the site and go directly to transformers to reduce voltage from 69 kilovolts (kV) to 4.16 kV. The secondary transformers would then go to two main breakers at the Elverta Intake power distribution switchgear. The transformer area is expected to be approximately 130 feet by 130 feet per Sacramento Municipal Utility District (SMUD) requirements. The intake medium voltage switchgear building is expected to be approximately 40 feet by 40 feet. The building would house the two mains, a tie breaker, potential transformers (PTs) and control power transformers (CPTs) and each of the two buses at 4.16 kV with two 1,200-amp main breakers. The distribution switchgear would have breakers to feed all of the 4.16 kV loads at the intake. Exhaust fans and heaters would be minimum building requirements.

It is anticipated that the 480-volt loads would be distributed from one switchboard to serve the plant's 480-volt motor control center loads.

SMUD is the governing power utility for the proposed WTP sites as per Article 11, Section 9, of the California Constitution. Power for this load is available from existing SMUD lines routed westward along Elverta Road up to Power Line Road. Two 69 kV power lines (in parallel) are currently in place and SMUD is currently upgrading these lines due to increased commercial and residential development in the North Natomas area. The loads presented here can be considered as part of SMUD's upgrade.

At Power Line Road, the overhead lines turn south. It is expected that SMUD would continue the feed west with 69 kV, beyond Power Line Road, using underground lines due to the runways at Sacramento International Airport.

Underground 69 kV lines have a budget cost of $175.00/foot, excluding trenching. The 69 kV service from existing upgraded overhead lines has a budget cost of $30.00/foot, excluding poles. The owner would incur the cost of poles or trenching in addition to the charge for the lines.

3.5.1.2. Utility Reliability

SMUD can provide a design that would incorporate the level of redundancy the owner would require. SMUD can design its connection points and multiple switching configurations for the redundancy that would meet the needs and satisfaction of the owner.

3.5.1.3. Motor Starter Requirements

SMUD requires all large medium voltage and low voltage motors to be reduced voltage solid state starters to reduce the impact of the starting currents on the SMUD system.

3.5.1.4. Primary Backup Power Supply

The proposed primary means for backup power supply is installing two primary feeds in the Elverta Intake site. The reliability of power supply at the site would increase greatly with installation of these separate primary feeds into the two transformers that provide the 4.16 kV at the Elverta Intake power distribution substation. The proposed plan for the power feeds at the site is to receive one feed from each of the two existing upgraded parallel 69 kV lines into the site.

Each transformer secondary would be connected to a main circuit breaker. The two mains would be connected by a tiebreaker. Upon loss of power detected in one of the two main breakers, that main would open and after a time delay (selected by the owner), the tiebreaker would close, resuming power to the side of the bus that lost power.

3.5.1.5. Alternative Backup Power Supply Option

An alternative backup power supply option would be use of a diesel generator at the Elverta Intake site. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the 145 mgd site would require a 1,150 kVA generator. A day tank (300 gallons) and fuel storage tank are required for each generator. The generator uses 80 gallons of fuel per hour at full load and would require a total of 640 gallons of fuel for an 8-hour time period (full load). The output power for each generator would vary with load requirements; therefore, if the load was less than 1,,150 kVA, the fuel consumption would be less.

The space required for the low voltage controls, day tank, and generator would be approximately 700 square feet in a building with integral automatic air flow louvers and fire alarm system design. Additional space outside would be required for the fuel storage tank.

A more detailed evaluation of backup power requirements and specific loads that would be deemed critical if both main breakers into the plant were lost is strongly recommended during the next phase of analysis to optimize sizing of these generators and associated facilities.

3.5.1.6. Dual Feeds from SMUD and Another Power Utility

SMUD does not allow another utility to serve within the SMUD service area.

3.5.2. Sewer System

It is assumed that an incinerator-type toilet would be provided at the intake site.

3.5.3. Stormwater Management

Currently, no storm drainage services are located in the northwest corner of Sacramento County near the project area. It has been assumed that all stormwater on the river-side of the levee would drain naturally to the river. For the approximately 0.5 acres required for the electrical substation, and standby power building on the land-side of the levee, it is assumed that the stormwater would need to be captured and managed on-site. The site would be constructed and graded to collect stormwater runoff and channel it to an on-site detention basin. This basin has been sized to meet the capacity of a 10-year storm over 5 days. The Sacramento City/County Drainage Manual indicates that the water depth of such a storm would be 5.76 inches. It has been estimated that approximately 10,500 cubic feet of water would need to be planned for in the detention basin design. It was assumed that the detention basins would be 3 feet deep to allow for evaporative drying. Therefore, a detention basin approximately 60 feet by 60 feet would be required on-site. An overall area of 1 acre has been reserved for the intake support facilities area.

3.5.4. Special Considerations

The intake facility is located within the overflight zone of the Sacramento International Airport. For this reason, the design of this facility must be developed to account for safety issues identified by the Sacramento County Airport Service and the FAA. Although no direct objections to the facility have been expressed in preliminary discussions with these agencies, items for continuing coordination would be overall height of the structure, design of lighting, and verification that electrical equipment at the site would not interfere with airport equipment.

3.6. CONSTRUCTION AND OPERATION OF THE PROPOSED INTAKE FACILITY

Construction characteristics and operating characteristics of the intake facility required for the ARPS-Elverta Diversion Alternative are discussed in this section. Additional or modified requirements for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative are presented in **Section 3.7**.

3.6.1. Construction Characteristics

Construction of the intake structure would require erection of a sheet pile cofferdam in the river. The cofferdam would be approximately 200 feet long and 60 feet wide. Construction of the cofferdam would require placement of sheet piles, excavation within the cofferdam area, and stabilization of the cofferdam. Steel H-piles would then be driven in the cofferdam to provide structural support for the intake structure. Next, tremie concrete seal would be poured and the work area dewatered. It is estimated that construction of the cofferdam and structural piles would take approximately 18 weeks. The contractor would likely drive the piles using a floating barge as a platform. In addition, the contractor may construct a temporary H-pile bridge from the bank to the cofferdam to facilitate construction. A discharge permit would be obtained for these construction activities.

Riprap would be placed for a distance of approximately 20 feet around the intake structure to prevent scour. Prior to placement of the stones, excavation of 2 to 4 feet of native material would be required.

Excavation would be required at each of the bridge piers and clearing would be required along the full length and width of the bridge. Steel H-piles also would be driven at each pier. A concrete pile cap would also be constructed at each pier.

Due to the required elevation of the intake access bridge, the levee road (Garden Highway) would need to be raised 8 to 10 feet. This elevation change would require regrading the road for a distance of approximately 600 feet in both the north and south directions from the access bridge. Raising the levee would result in an expansion of the levee extents to a maximum of 50 feet on its landward side for the 1,200-foot regrading length. See the "section" view of **Figure 3-18** for maximum roadway cross section. The overall time for construction is estimated to take 21 to 24 months.

Limited site grading would be required on approximately 1.0 acres of land adjacent to the levee at the intake site. This site would be used for intake support facilities, including an electrical substation and standby power equipment, as required.

Construction-related traffic (e.g., materials delivery trips, workers, etc.) would access the site from Elverta Road and Garden Highway. Disposal of excavated materials and installation of concrete would require numerous truck trips to and from the site. A traffic control plan would be prepared by the contractor and reviewed by Sacramento County to make sure traffic is safely routed by the work site.

Safety on the construction-site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. Federal Occupational Safety and Health Administration (OSHA) and California (Cal)-OSHA standards would apply for all work.

The construction contract documents would include a general stormwater pollution prevention plan (SWPPP). The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at the work site exit

3.6.2. Operating Characteristics

The Elverta Intake would operate continuously, 24 hours per day, 7 days per week, at various flow rates throughout the year. Ongoing operations and maintenance would occur. The facility is unmanned; however, it is expected that maintenance personnel would visit the site at least twice per day to confirm operation and perform minor maintenance. More advanced maintenance of the pumps and motors would be required periodically.

Routine vehicle traffic would comprise mainly full-size pick-ups driven by maintenance staff. Specialty requirements for scheduled and emergency maintenance would include heavier load trucks.

3.7. ARPS-JOINT SACRAMENTO-ABFSHIP ELVERTA DIVERSION ALTERNATIVE

This section describes the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative (see **Figures 1-2** and **3-21**), a subalternative for the ARPS-Elverta Diversion Alternative, includes the participation of NMWC in the project intake facility. The majority of the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative is identical to the ARPS-Elverta Diversion Alternative, with the exception of modifications to the size of the intake facility, modifications to NMWC's existing canals, and additional facility modifications, as described in this section.

Under the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative, the NMWC Elkhorn Diversion addressed in the CALFED-supported ABFSHIP Sankey/Elkhorn diversion alternative would be consolidated with the SRWRS Elverta Intake in a joint diversion (see **Figure 3-22**). The Elverta Intake Facility capacity would be increased from 145 mgd (225 cfs) to 281 mgd (435 cfs) to accommodate the capacity of 136 mgd (210 cfs) required by NMWC. In addition, the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative would include improvements to approximately 1.6 miles of NMWC's existing Elkhorn Main Canal as well as associated modifications required to enable delivery of the water pumped from the new intake. The remainder of the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative is identical to the ARPS-Elverta Diversion Alternative. It is anticipated that all facilities, with the exception of NMWC's Elkhorn Main Canal, would be constructed, owned, and operated by the SRWRS cost-sharing Partners. The proposed Elverta intake structure, raw water pipelines, and North Natomas WTP would be owned and operated by Sacramento. NMWC would continue to own and operate the Elkhorn Main Canal. The treated water pipelines delivering water to PCWA, SSWD, Roseville, and Sacramento would be owned and operated by individual purveyors. Project facilities that differ from ARPS-Elverta Diversion Alternative are described in the subsections that follow; all other facilities would be constructed as described for the ARPS-Elverta Diversion Alternative.

3.7.1. Intake Facilities

The ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative includes modifications to the Elverta Intake as described for the ARPS-Elverta Diversion Alternative. The modifications are described below.

3.7.1.1. Pumps, Discharge Piping, and Energy Dissipation

As previously noted for the ARPS-Elverta Diversion Alternative, water would be drawn into the intake structure by vertical turbine pumps with varying capacities. Pump configuration could include two 11-mgd, two 22-mgd, and four 33-mgd pumps, with one of the four 33-mgd pumps a redundant or backup pump. To supply water for NMWC, an additional four dedicated 33-mgd pumps (approximate capacity) would be required. The pumps could discharge to a manifold and be routed across the access bridge and over the Sacramento River levee in a dedicated 72-inch pipe, as shown in **Figure 3-22**.

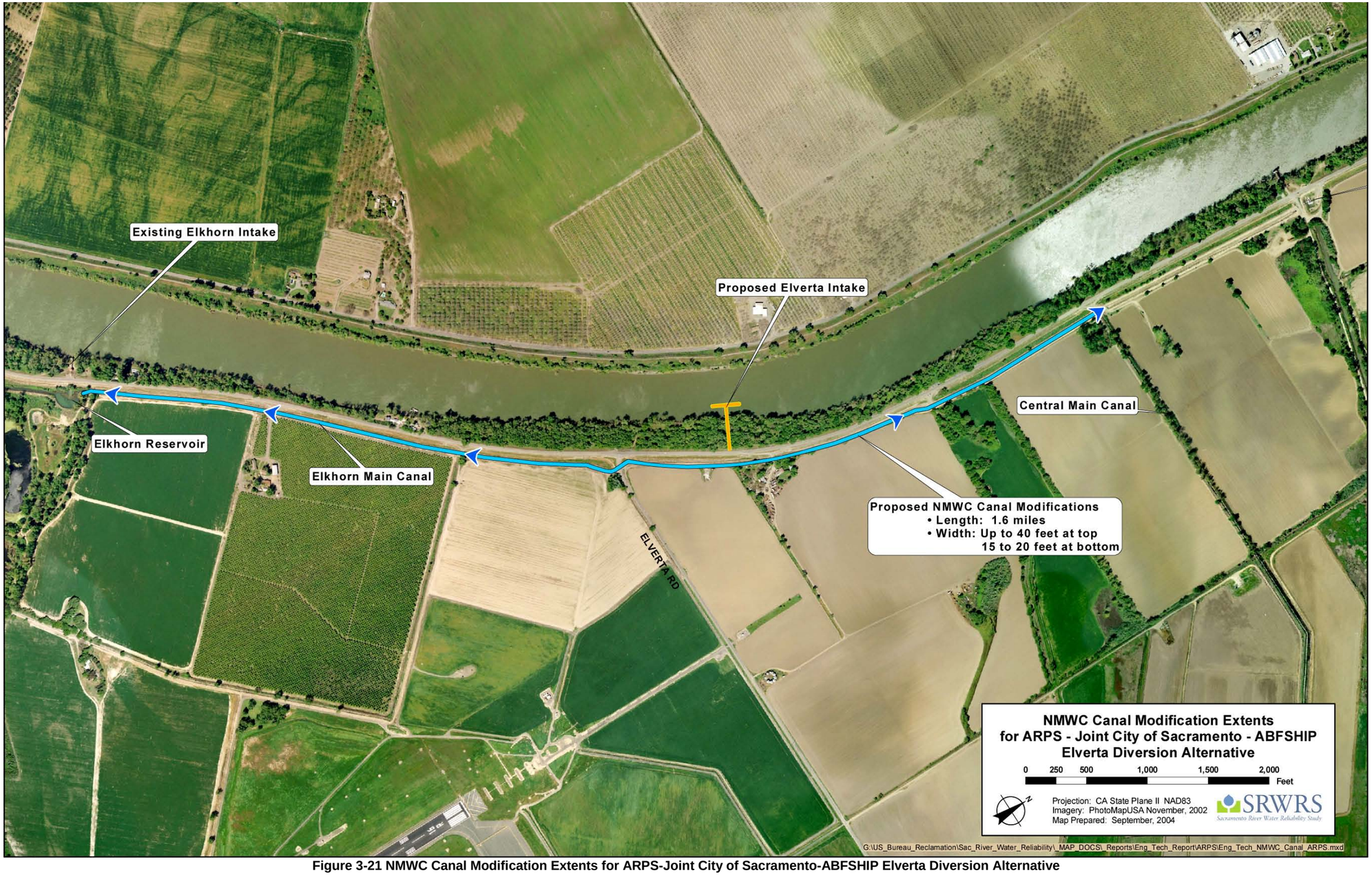

Figure 3-21 NMWC Canal Modification Extents for ARPS-Joint City of Sacramento-ABFSHIP Elverta Diversion Alternative

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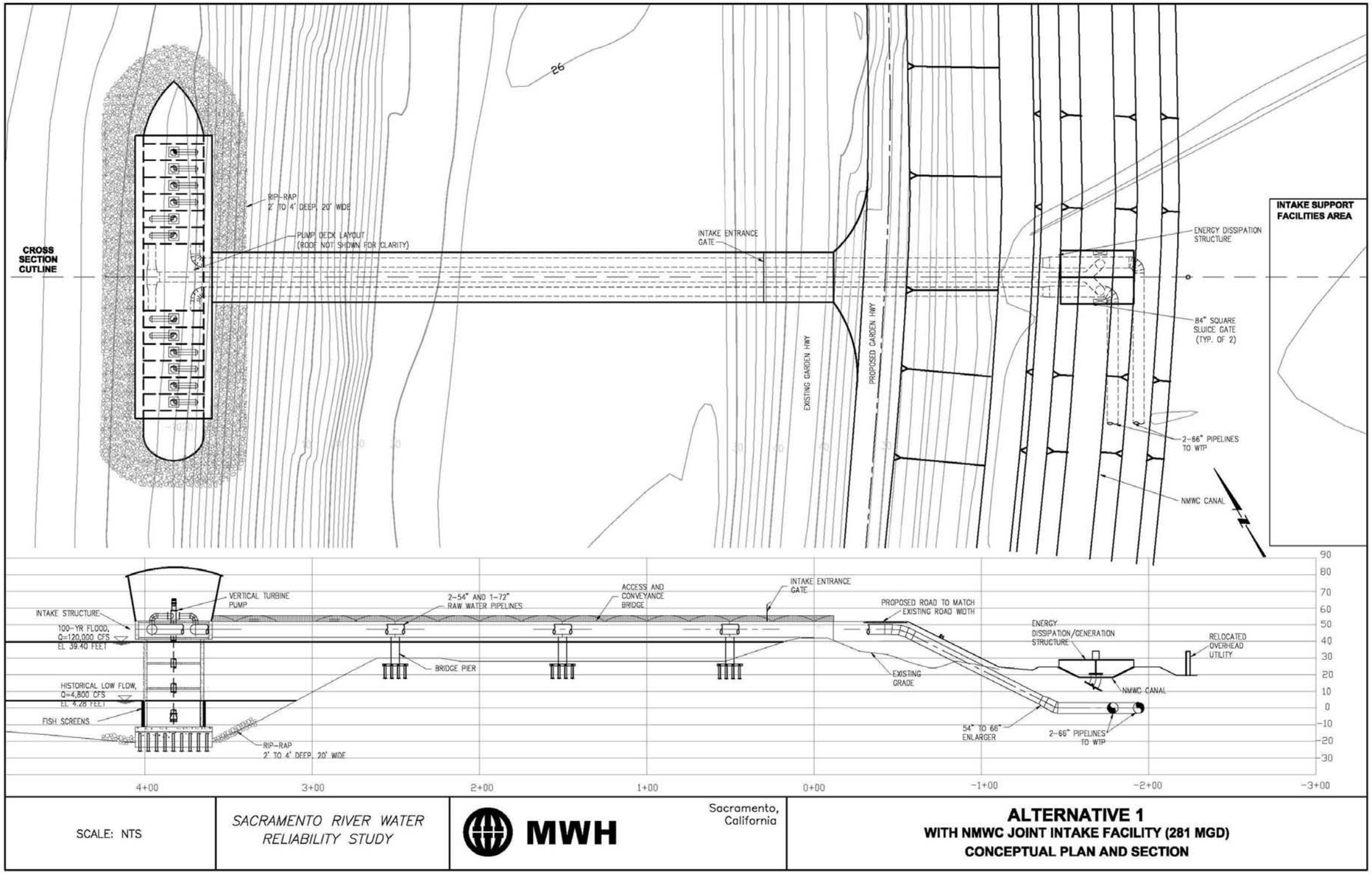

Figure 3-22 Alternative 1 with NMWC Joint Intake Facility (281 mgd) Conceptual Plan and Section

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The NMWC water pumped from the intake is to be routed to the Elkhorn Main Canal, which is routed parallel with the levee, at varying distance from its land-side toe. Due to the elevation difference between the top of the levee and the Elkhorn Main Canal, the pumped water would have a significant amount of excess energy that must be dissipated in a controlled fashion. It is assumed that water from the 72-inch discharge pipe would enter the canal via a concrete structure designed to dissipate its excess energy. The concrete structure is assumed to be approximately 30 feet long by 40 feet wide and approximately 12 feet in overall height. The structure is further assumed to have an 84-inch by 84-inch sluice gate at both its north and south outlets to the canal, providing a variable release rate of water in the northward or southward directions. Specific design characteristics of the energy dissipation structure will be established during the preliminary design phase of the project.

3.7.1.2. Fish Screens

As previously noted for the ARPS-Elverta Diversion Alternative, the 145 mgd diversion would require 16 fish screens, each an average of 8 feet wide and 10 feet high. The four additional pumps required for NMWC's additional 136 mgd would require 4 additional pump bays, each requiring approximately 155 square feet of screen area divided between the two sides of each bay. If it is assumed that the additional screens will be 8 feet wide, their required height would then be approximately 10 feet each.

3.7.1.3. Intake Structure and Conveyance Bridge

The additional four pump bays required for NMWC dedicated flow will add 40 feet to the length of the joint intake structure, increasing the structure length to 220 feet. While the structure foundation width should not change from the ARPS-Elverta Diversion Alternative, the upper portion of the intake structure will increase slightly to accommodate the additional dedicated NMWC manifold piping. The bridge for the ARPS-Joint City of Sacramento-ABFSHIP Elverta Diversion Alternative will need to be approximately 10 feet wider, approximately 40 feet total width, to accommodate the new dedicated 72-inch-diameter pipe for NMWC. The additional width required for the bridge will make the intake structure a grand total of 230 feet long. The footprint area of the bridge support piers would increase proportionately.

3.7.1.4. Intake Support Facilities

Additional power will be required for the additional dedicated NMWC pumps but this will not change the approximate footprint of required facilities. In addition, backup power will not be provided for the NMWC pumps.

3.7.1.4.1. <u>Primary Power Requirements, Availability, and Reliability</u>

The maximum power requirement for the 281 mgd Intake Structure and the NMWC requirements have been estimated to be 3,900 kVA. **Table 3-6** provides a general summary of the power requirements. Power routing and other requirements would be similar to those outlined for the 145 mgd facility.

Table 3-6 Power Requirement Summary for 281 mgd Facility

Intake Pump Station	Peak Flow (mgd)	Pump Load[(1)] (hp)	Misc. Loads (kVA)	Power (kVA)	Amps @ 4,160 Volts	1/2 Load (kVA)
Intake Facility with Natomas Mutual Water Company Capacity	281	3,650	250	3,900	541	1,950

Notes:
[(1)] Includes a spare pump.
Key:
hp – horsepower kVA – kilovolt-ampere mgd – million gallons per day

3.7.1.4.2. Alternative Backup Power Supply Option

Since agricultural water operations typically have greater flexibility and are not required for emergency services, backup power would be provided only for the municipal portion of the intake. The required power facilities would be as previously described in **Section 3.5**.

3.7.2. Elkhorn Main Canal Modifications

As noted in **Chapter 1**, the capacity of the Elkhorn Diversion in the CALFED-supported ABFSHIP Sankey/Elkhorn diversion alternative would be consolidated with the ARPS-Elverta Diversion Alternative to form ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative (refer to **Figure 3-21**). Based on information provided by NMWC and its engineering consultant, Mead and Hunt, the canal modifications portion of the ABFSHIP Sankey/Elkhorn diversion alternative currently includes upgrading and realigning approximately 1.6 miles of the NMWC's Elkhorn Main Canal. This reach of the canal runs from the Elkhorn Reservoir in the south, where the Elkhorn Diversion with a capacity of 120 cfs was to be located, to the Central Main Canal in the north. The existing Elkhorn Main Canal is a 15- to 20-foot-wide earthen trough, 4 to 6 feet deep. The modified portions of the earthen canal would be uniformly trapezoidal in shape, with a 15- to 20-foot bottom approximately 5 feet deep, with 2:1 side slopes, and would be up to 40 feet wide at the top. The canal improvements would allow up to 150 cfs of water to be directed northward from the intake near the Elkhorn Reservoir to the Central Main Canal. Under ABFSHIP Sankey/Elkhorn diversion alternative, the canal would also be relocated up to 30 feet to the east to reduce levee seepage concerns of Reclamation District 1000, the agency that maintains the Sacramento River levee in this reach. The canal modification work described in the ABFSHIP Sankey/Elkhorn diversion alternative also includes demolition, relocation, and modification of existing turnouts, drain sump pumps, concrete headwalls and culverts, piping, and utilities.

The planned canal modification portion of the ABFSHIP Sankey/Elkhorn diversion alternative would be modified under the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative. The Elkhorn Main Canal would be required to slope both north and south from the Elverta Intake Facility, rather than sloping only northward from the Elkhorn diversion, as previously planned. Additional check structures or turnout modifications may also be required to accommodate changes in water surface elevation due to this canal slope modification. An energy dissipation structure would be required to direct water into the canal from the Elverta Intake, as discussed in **Section 3.7.1.1**. Additional modifications would also be required at the canal entrance structure at the Elkhorn Reservoir to provide a means for maintaining appropriate water surface elevations for wildlife habitat. The ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative, similar to the ARPS-Elverta Diversion Alternative, would require the realignment of the Elkhorn Main Canal a maximum of 35 additional feet eastward beyond the planned realignment to accommodate the required levee raise and associated realignment of Garden Highway.

3.7.3. Construction and Operation of Facilities

Construction activities for the Elverta Intake would be similar to those described for the ARPS-Elverta Diversion Alternative. The extents of construction would be slightly larger due to the larger footprint of the joint intake. The width of the cofferdam would not change but the length would increase approximately 50 feet to create an overall length of 250 feet.

Extensive regrading along the entire existing approximately 1.6-mile Elkhorn Main Canal would be required. It is assumed that the canal would be constructed such that excavated material would be used to form the canal berms. More detailed analysis would be required to verify the cut and fill balance. Construction activities for the Elkhorn Main canal would include soil excavation, backfilling, and compaction. Excavated material would be used to form canal berms in areas where the canal would be realigned and/or widened. Small concrete structures containing weirs or gates would be constructed to control the water level at various locations in the canal. Drainage would be collected and piped to existing drainage ditches, or recirculated into the canal. Utilities would be relocated as required. Construction areas would be accessed directly off Elverta Road, and the staging area for the work would be coordinated with the staging area used for the proposed Elverta Intake structure. Construction would be scheduled to avoid impacts on NMWC irrigation deliveries. Construction activities related to intake improvements and canal widening are anticipated to occur concurrently with construction of the other facilities.

NMWC would own the Elkhorn Main Canal and manage all operations and maintenance (O&M) activities, including control of hydrologic and hydraulic regimes during seasonal operations. Sacramento would own and operate the Elverta Intake Facility.

3.7.4. North Natomas Water Treatment Plant

The North Natomas WTP location, facilities, construction activities and schedule, and O&M for this Alternative would be the same as those described for the ARPS-Elverta Diversion Alternative.

3.7.5. Raw and Treated Water Pipelines

Raw and treated water pipeline routes, materials, support structures, construction activities and schedules, and O&M for this alternative would be the same as those described for the ARPS-Elverta Diversion Alternative.

3.7.6. Decommissioning Elkhorn Pump Station

Under this alternative, NMWC would discontinue use of the Elkhorn pump station and the pump station would be decommissioned. Decommissioning would be performed in accordance with the standards of The Reclamation Board. Discharge pipes through the levee would be removed or abandoned in place by filling with concrete. The outfall, rubble and debris, and pumps would be removed. Wooden pilings in the river would be removed or cut off at the base. The historic pump house and pumping plant would be left. Pipes would be removed, along with walkways, for river pump platform access. Revegetation would be performed in accordance with permit conditions.

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CHAPTER 4 RAW WATER PIPELINES

The raw water pipeline conveys water from the intake facility to the WTP. This chapter describes the hydraulics (design flow, velocity, head loss, and pipe size) and the alignment the pipeline would follow, and provides characteristics about the pipeline and its construction that would be important for environmental documents. Raw water systems for the Sacramento River source are very different from those for the American River source. These two raw water systems are discussed separately in this chapter, starting with the raw water system fed from the Sacramento River, known as the Elverta raw water system. This discussion is followed by the raw water system fed from the American River, known as the ARPS raw water system. All elevations presented in this chapter are referenced to NGVD 29.

4.1. ELVERTA RAW WATER SYSTEM

The Elverta raw water system takes water from the intake on the Sacramento River to the WTP along Elverta Road. The system consists of a pipeline from the pumps on the intake to the grit basin at the WTP. Alternative WTP sites currently under consideration are described in **Chapter 5**, see **Figure 5-1**. This discussion covers pipelines routed to each of three WTP sites.

4.1.1. Hydraulics

The peak flow in the raw water pipeline would be 145 mgd. Two pipelines would convey the flow to provide redundancy should one pipe require maintenance, and also to make it possible to maintain higher velocities during low flow periods by using only one of the pipes. Pipeline length would be between 1 and 4 miles depending on the location selected for the WTP site. A WTP site located approximately 2.6 miles from the intake was assumed for illustrative purposes. The head loss that would occur was calculated for pipelines of varying diameters. It was found that head loss, and therefore power cost, would increase more than pipe cost decreased if the two pipes were each any smaller than 66 inches in diameter. Two parallel pipes 66 inches in diameter each are recommended for the raw water pipeline. At peak flow, the velocity in these pipes would be 4.7 fps. The head loss between the intake and the WTP would be between 4.2 and 19 feet, depending on the location selected for the WTP site. (Alternative WTP sites currently under consideration are described in **Chapter 5**; see **Figure 5-1**). Typically, water elevations in the river vary from a low water level of elevation 4.28 to a high water elevation of 19.98 (the 10 percent exceedence value). The water would discharge into a grit removal chamber at the WTP. The exact elevation of this grit removal chamber would not be known until the plant design is complete. For planning purposes, it is estimated that the water elevation in the grit removal chamber would be 45 at a site located at the western end of potential WTP sites and 40 at a site located at the eastern end of potential WTP sites. Combining the lift and the head losses gives the range of total heads that would have to be pumped. Pumping heads would be highest when pipeline flows are highest and the river level is low, and heads would be lowest when pipeline flows are low and the river level is high. Predicted pumping heads for the range of conditions are given in **Table 4-1**.

Table 4-1 Range of Pumping Heads from Intake to Water Treatment Plant

Pipeline Discharge Point	Total Dynamic Pumping Head per Condition (feet)	
	Low River Level, Peak Intake Flow	High River Level, Low Intake Flow
To extreme western Water Treatment Plant site	45.0	25
To Water Treatment Plant Site assumed at 2.6 miles from intake for illustrative purposes	52.5	25
To extreme eastern Water Treatment Plant site	54.7	20

4.1.2. Alignment Selection and Description

From the intake, the raw water pipeline would follow an alignment as close to the projected toe of the existing levee as permitted by Reclamation District 1000. For planning purposes, the pipelines are estimated to be a minimum of 60 feet from the projected toe of the existing levee. The pipelines would continue south to Elverta Road and then turn east and follow Elverta Road to the WTP. An alternative route crossing diagonally through the field between the intake and Elverta Road was rejected because it would have required the purchase of additional right-of-way and would have impacted more sensitive habitat. Along Elverta Road, the pipes would be placed in the westbound lane, as close to the right-of-way line as possible. In areas where pipe construction would impact sensitive habitats on the roadside, the pipe would jog into or across the roadway. An overview of the pipeline alignment is shown in **Figure 1-1**. A more detailed view of the pipeline alignment and profile is presented in **Figure 4-1**. As noted previously, a WTP site located 2.6 miles from the intake was assumed for illustrative purposes.

4.2. AMERICAN RIVER PUMP STATION RAW WATER SYSTEM

This section describes the pump station, pipeline additions, and modifications needed to move the 65 mgd of raw water planned for PCWA as part of the SRWRS project from the American River to the planned Ophir WTP (see **Figure 4-2**). A pump station is under construction to pump water from the American River as a raw water source for PCWA. This pump station, known as the American River Pump Station or ARPS, would discharge into the Auburn Tunnel, which conveys water to a point close to where the Ophir WTP will be built (see **Figure 1-1**). The existing ATPS No. 1 pumps water from the Auburn Tunnel into the South Canal, where it is conveyed to an existing PCWA treatment plant, the Foothill WTP. Another pump station, ATPS No. 2, is under design. The ATPS No. 2 would have the capability to pump into the south canal, but would also be able to pump water into pipelines to either the existing Foothill WTP or the planned Ophir WTP. The current construction of the ARPS (initial capacity 65 mgd) and the planned initial construction of the Ophir WTP (initial capacity 30 mgd) are not part of this project. This project would involve expanding the ARPS to pump an additional 65 mgd, using capacity already available in the Auburn Tunnel, and then constructing a new ATPS No. 3 to pump 65 mgd out of the Auburn Tunnel and into an expanded Ophir WTP. The capacity of the Auburn Tunnel has been assumed adequate, although an evaluation of pump discharge under low pressure should be conducted to verify.

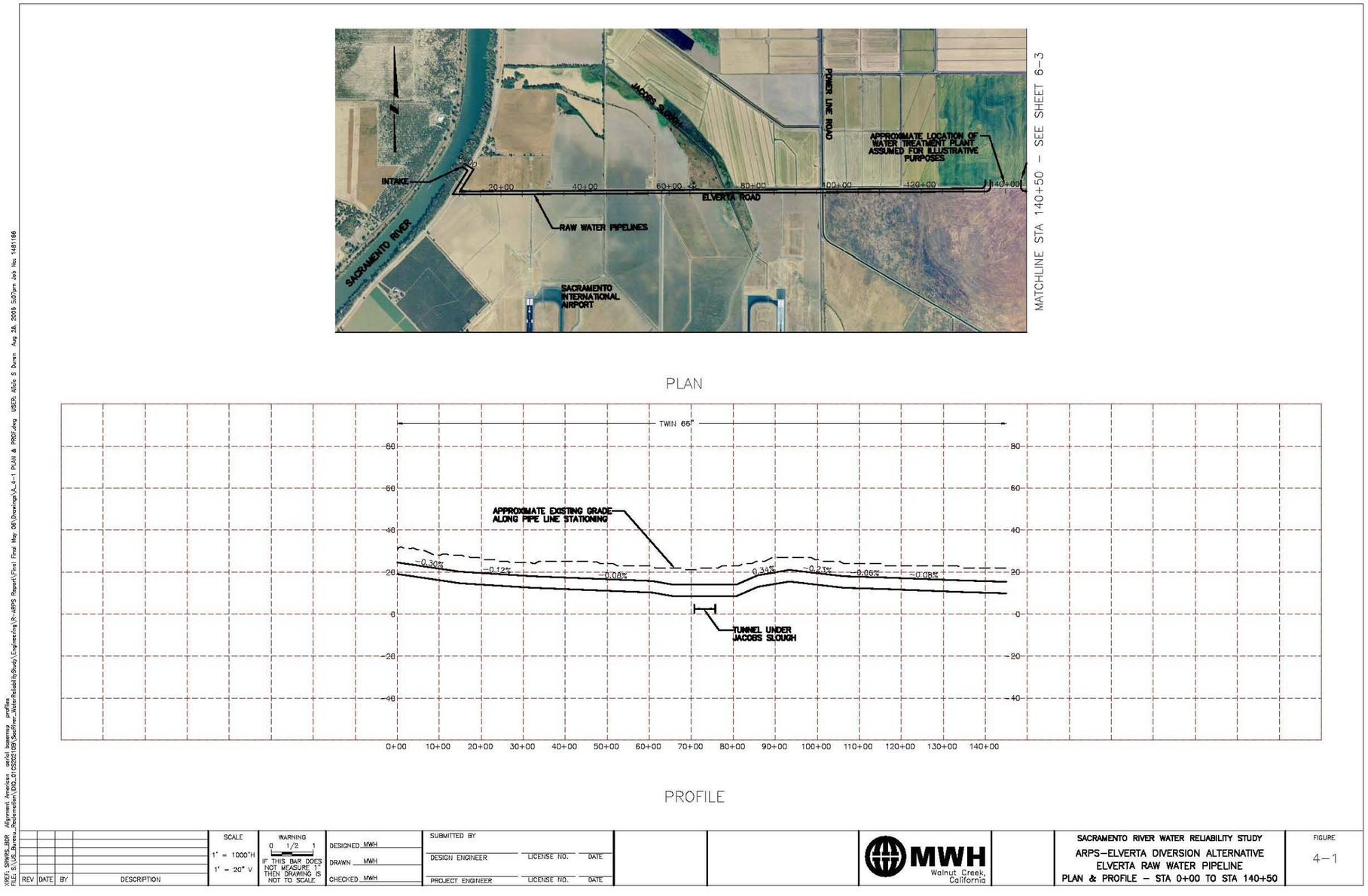

Figure 4-1 ARPS-Elverta Diversion Alternative Elverta Raw Water Pipeline Plan & Profile – STA 0+00 to STA 140+50

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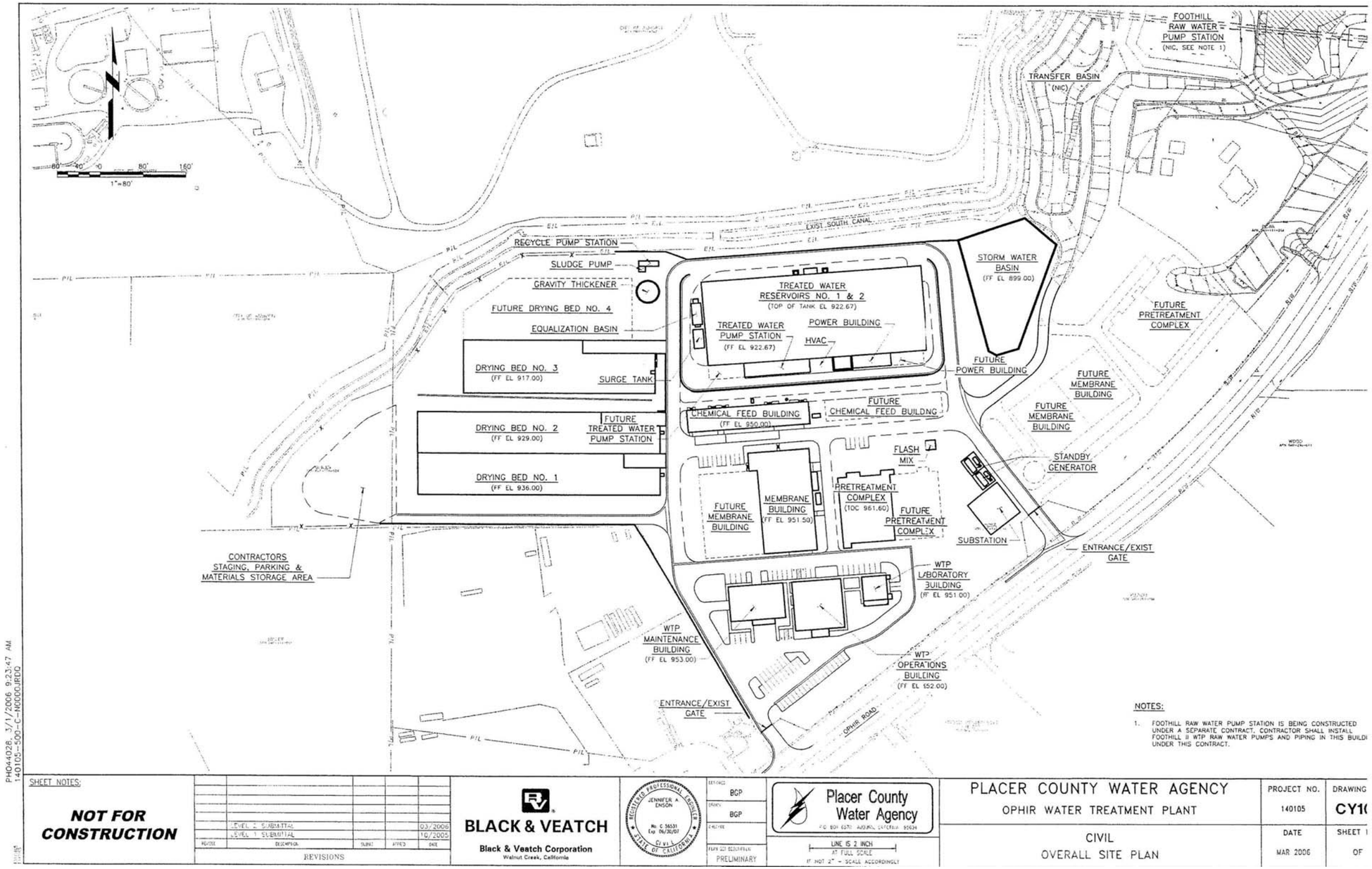

Figure 4-2 Ophir WTP Overall Site Plan

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4.2.1. Facility Modifications

The capacity of the ARPS would be increased by 65 mgd by placing one new 32.3-mgd pump in a spare bay, upgrading three 24.5-mgd pumps to pump 32.3 mgd each, and upgrading one 11-mgd pump to pump 20.3 mgd. These improvements would provide a pump station with 130 mgd capacity with the largest pump out of service. The pump station was designed for potential expansion. The structure, piping, and electrical system at the ARPS have the capacity to handle the additional pump, and the upgrade of the three 24.5-mgd pumps to 32.3 mgd. The fuse feeding each pump would need to be upsized and a fan would need to be added to the transformer. The upgrading of the 11-mgd pump to 20.3 mgd was not envisioned in the original design and therefore may require more significant modifications to the electrical system and piping than would be needed for the other changes. Upsizing of the PG&E feed to the pump station would probably be necessary.

The ATPS No. 3 would be an entirely new pump station constructed just to the east of the ATPS No. 2. The ATPS No. 3 would draw up to 65 mgd from the Auburn Tunnel and pump it in a dedicated 54-inch pipeline approximately 1,000 feet to the inlet of the Ophir WTP. The pump station would consist of five vertical turbine pumps, one of which would be a backup. The pump motors, discharge heads, valves, and discharge header would be placed above grade. The pumps would be located in the Auburn Tunnel, 200 feet below. The columns would be placed in holes drilled through the rock to the Auburn Tunnel. Two of the pumps would be fitted with a variable-frequency drive to permit fine tuning of flows.

The maximum power requirement for ARPS No. 3 has been estimated to be 4,872 kVA. **Table 4-2** summarizes power requirements.

Table 4-2 Power Requirement Summary

Facility	Peak Flow (mgd)	Pump Load[(1)] hp	Misc. Load (kVA)	Power (kVA)	Amps @ 4,160 Volts	1/2 Load (kVA)
Auburn Tunnel Pump Station No. 3	65	4,372	500	4,872	676	2,436

Notes:
(1) Includes a spare pump.
Key:
hp – horsepower kVA – kilovolt-ampere mgd – million gallons per day

Power would enter the site and go directly to a transformer to reduce voltage from 12 kV to 4.16 kV. The secondary transformer would go to a main breaker inside a switchgear lineup at the pump station. The switchgear would include 4,160-volt starters for the pumps. The starters would be reduced voltage solid state if constant speed.

Backup power supply for the ATPS No. 3 site would be a diesel generator. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the ATPS No. 3 site would require a 2,436 millivolt-ampere (mVA) generator. The generator requires a day tank (300 gallons) and a fuel storage tank. The generator uses 150 gallons of fuel per hour at full load and would require a total of 1,200 gallons of fuel for an 8-hour time period (full load). Output power for the generator would vary with load requirements; therefore, if the load demand is less than 2,436 kVA, fuel consumption would be less.

The ATPS No. 3 footprint would include a pad approximately 60 feet by 25 feet for the pumps. The space required for the generator, low voltage controls, and day tank would be approximately 900 square feet in a building with integral automatic air flow louvers. Additional space outside would be required for the fuel storage tank.

4.3. PIPE MATERIAL

Several materials would be suitable for this pipeline. The most common pipe types for this function and size are welded steel, ductile iron, and pretensioned concrete cylinder pipe. Final project specifications would be written for one or more of these three pipe types.

Should the pipe be steel, it would be coated and lined. The lining is usually cement mortar, although epoxy linings are occasionally used. The coating could be cement mortar, epoxy, or polyethylene tape. Cathodic protection may be used to protect the pipe from corrosion, depending on the corrosiveness of local soils. This would be determined during predesign investigations.

Should the pipe be ductile iron, it would have a cement mortar lining. The pipe would not have bonded coating, but would have polyethylene sleeves over the pipe for corrosion protection. Cathodic protection might be used, as with steel pipe.

No additional lining or coating is used with pretensioned concrete cylinder pipe. Cathodic protection may be used, as with steel pipe.

4.4. PIPELINE APPURTENANCES

The piping system would include valves at strategic locations. Isolation valves would be used to isolate reaches of the pipe for maintenance and would be installed approximately every 1,000 feet along the pipe. The system would also include an air release valve at each high point and a blowoff at each low point. The air release valve assembly would be housed in a small aboveground enclosure located along the side of the road. The blowoff assembly would be entirely below ground. The system would also include access ports into the pipeline at intervals of approximately 1,000 feet.

4.5. CONSTRUCTION CHARACTERISTICS

The pipe trench would be typically 16 to 18 feet wide and 10 to 15 feet deep. Shoring would be used to maintain a narrow vertical side-wall trench and to protect workers. **Figure 4-3** presents a typical trench cross section. A work area at least 5 feet wide on one side of the trench and at least 15 feet wide on the other side of the trench would be needed for construction. Where available, a larger work area of up to 40 feet on one side of the trench would be provided to facilitate construction and reduce cost. Some of the work area could be achieved through temporary lane closures during work hours.

Groundwater is high year-round in the Sacramento River area; therefore, extensive dewatering would be needed during construction, from before the trench is opened until after the trench is backfilled. The need for dewatering is not expected to be as severe for the PCWA pipelines and pump station. Water removed from the construction area would be treated to remove sediment, and would be discharged to the closest drainage way. A discharge permit would be needed. The dewatering method most likely to be used is a network of well points along the pipeline alignment. The wells would be drilled to several feet below the trench invert, which would be 10 to 12 feet below grade. Well spacing could vary widely. Commonly, wells would be about 100 feet apart.

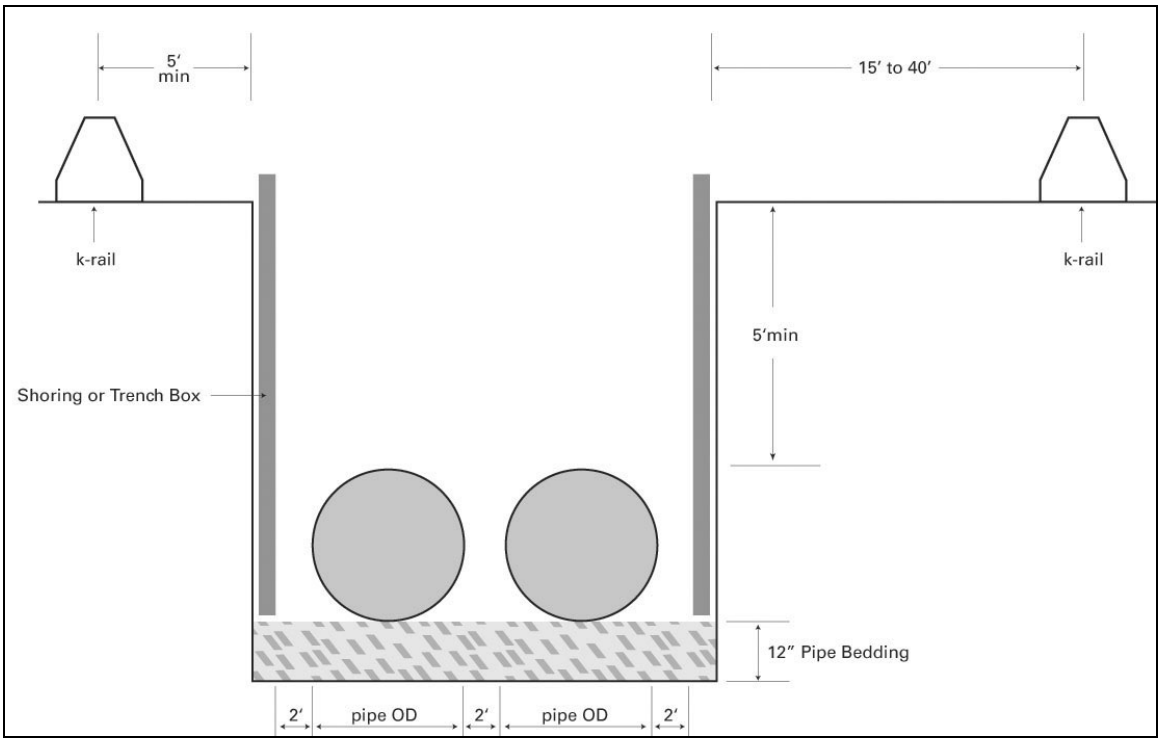

Figure 4-3 Typical Trench Section for Pipe Installation

Pipe bedding would be crushed rock or sand. Pipe zone backfill would be sand or crushed rock or controlled density fill (very low strength concrete). Trench zone backfill would be native material. Any native materials unsuitable for trench backfill would be hauled away to a disposal site selected by the project sponsors.

Crews should be able to install pipe of this size and depth at production rates of 100 feet of trench per day during dry weather if no problems occur. However, to account for possible delays, average production rates would probably be about 40 feet of trench per day. **Table 4-3** presents estimated pipe lengths and construction durations for alternative raw water pipelines for the Elverta raw water system. A contract period 40 work days longer than the construction period would be needed to allow for mobilization, demobilization, and punchlist work. Typical work days would be from 7:00 a.m. to 3:30 p.m. Monday through Friday, with work occasionally continuing as late as 7:00 p.m. and/or on Saturday.

Table 4-3 Estimated Construction Duration for the Elverta Raw Water System Pipeline

Pipeline Discharge Point	Pipe Length (feet)	Trench Length (feet)	Construction Duration (work Days)	Contract Period (work days)	Contract Period (calendar days)
To Water Treatment Plant site at extreme western end of possible sites	9,400	4,700	120	160	260
To Water Treatment Plant site at 2.6 miles from intake	27,400	13,700	340	380	600
To Water Treatment Plant site at extreme eastern end of possible sites	47,000	23,500	590	630	1000

Constructing the upgrade of ARPS and ATPS No. 3 would take approximately 300 calendar days. Long lead times required for the pumps and electrical gear would control construction duration. Days of actual work in the field would be about 100 work days for both pump stations. The raw water pipeline from ATPS No. 3 to the Ophir WTP is less than 2,500 feet and should not be a schedule constraint.

The pipeline construction operation could use a number of different combinations of equipment. A possible scenario would include one or two excavators to excavate the trench, place pipe bedding and pipe zone backfill, and set the pipe; a front-end loader to move soil around the work site and load trucks; a dozer or tractor to move trench backfill into place; a large compactor and smaller walk-behind compactors; two to six end dump trucks to haul soil to and from the work site; and miscellaneous trucks to deliver materials and imported fill. Crew size would be 6 to 10 people, not including truck drivers. Laborers and operators in the crew would be hired locally. The crew superintendent and the contractor's project manager and field engineer may be local staff or, if the contractor is not local, may be brought in from outside the area.

The number of truck trips to and from the construction-site each day would vary depending on how much of the native soil can be used for backfill. If all the backfill can be native material taken from the trench and stored at the work area, only about 11 truck trips would be needed to haul away excess material, and 11 more truck trips to haul in imported material on an average day. Should the native material be unsuitable for backfill, or inadequate space exist at the work site to store the material until the trench is ready for backfill, the number of truck trips would increase to about 23 trips each to bring in material and haul away material.

Trucks hauling materials to and from the construction-site would have loads with weights below highway load limits. Trucks hauling soil, rock, or sand to and from the job site would haul from 5 to 10 cubic yards of material in each load. Loads for other trucks would vary depending on what is being hauled, but would always be below H-20 load limits.

Safety on the construction-site would be the responsibility of the construction contractor. The construction contractor would have a company safety program and a job-specific safety program administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. A traffic control plan would be prepared by the construction contractor and reviewed by Sacramento County to ensure traffic is safely routed around the work site. OSHA and Cal-OSHA standards would apply for all work.

No particularly noisy equipment would be anticipated for the construction work (e.g., no pile driving). Typical noise would include trucks and diesel-powered equipment. The work would comply with all county noise ordinances.

The construction contractor would have a staging area for field offices and to temporarily park equipment and supplies. This area would be 1 to 5 acres in size. A site has not been selected yet for this staging area. A 2- to 10-acre site also would be used for disposing excess material removed from the trench. Some material would be stockpiled only temporarily at the disposal site and then used later for backfill. Other material would be permanently placed at the disposal site. A grading permit would be obtained for the disposal site. Work at the disposal site would comply with all county requirements, including grading ordinance and sedimentation and erosion control requirements. A site has not yet been selected for this disposal site.

The Elverta raw water pipeline crosses one stream, Jacobs Slough. A stream alteration permit would be requested from the CDFG. It is not likely that the permit would allow using open-cut trenching to install

the pipe across the stream; instead, tunneling would be used. A pressure balance tunneling technology would be used because since the tunnel would be below groundwater levels. Tunneling would involve a jacking pit approximately 15 feet wide by 30 feet long by 25 feet deep on one side of the stream and a smaller receiving pit on the other side.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at each work site exit
- Placing waddles or straw bales around the open trench work area

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CHAPTER 5 WATER TREATMENT FACILITIES

This chapter presents refined engineering for the water treatment facilities for the ARPS-Elverta Diversion Alternative. This alternative includes two water treatment facilities: the North Natomas WTP, which would provide treated water to Sacramento, and an expansion of the Ophir WTP, which would provide treated water to PCWA.

The North Natomas WTP would be designed for a maximum flow of 145 mgd to provide Sacramento with a base supply and peaking supply for the summer months with an operating range between 20 and 145 mgd. The WTP would be located in the north Sacramento County area near Elverta Road, where the major transmission pipelines would be placed, but a final site has not yet been selected. The potential WTP location area is shown in **Figure 5-1**. The WTP would require a 90- to 100-acre site within the area shown. Sites located in the western portion of the potential WTP area would be in or near the approach to the two existing runways for Sacramento International Airport.

The expanded Ophir WTP would be designed for an ultimate capacity of 95 mgd to provide PCWA with a base supply and peaking supply for the summer months with an operating range of 0 to 65 mgd. PCWA and its consultant are conducting environmental analysis and designing the first phase (30 mgd) of this future facility. It would be located on Ophir Road near Auburn, as shown in **Figure 5-2**. All design information provided in this chapter for this expanded facility has been taken from the Preliminary Water Treatment Plant Design and the Initial Planning and Siting Study developed by Black and Veatch[5, 6] the Draft Environmental Impact Report developed by ESA[7], and revised information provided by PCWA.

Roseville would obtain its water by increasing the use of groundwater resources (up to 10 mgd). This would require the installation of four new groundwater wells with pump stations and chemical feed facilities. Preliminary design of these facilities has been developed using Roseville's Diamond Creek Well as a template. The locations of these wells have not been finalized yet, and would be determined by Roseville during future land-use planning.

SSWD would obtain its water supply by diverting water (up to 15 mgd) at Folsom Dam and treating it at SJWD's Peterson WTP using shoulder capacity. It is assumed that no facility expansion would be required to treat and deliver this water to SSWD.

5.1. TREATED WATER GOALS AND OBJECTIVES

When planning water treatment facilities, it is necessary to identify goals and objectives for the treated water to guide in process selection, design of facilities, and development of an operations plan. The following are general goals and objectives for the treated water:

1. Treated water shall be potable and at a minimum meet all Federal and State drinking water standards.
2. Treated water shall be aesthetically pleasing to the consumer.
3. Treated water shall be provided to each SRWRS cost-sharing partner to blend with individual systems without creating distribution system water quality problems.

[5] Black &Veatch, 2004. Preliminary Water Treatment Plant Design Memorandum. January.
[6] Black & Veatch, 2004. Foothill Phase II Water Treatment Plant Initial Planning and Siting Study. July.
[7] ESA, 2005. Foothill Phase II Water Treatment Plant and Pipeline-Draft Environmental Impact Report. April.

4. Treated water shall be provided reliably and as cost-effectively as possible.
5. Treated water shall have a sufficient disinfectant residual to provide delivery with detectable residual concentrations to customers.
6. Treated water shall be non-corrosive to the SRWRS partners' distribution systems.

In addition to the above general goals and objectives, several more specific criteria have been set that directly impact design and operation of the water treatment facilities, including the following:

1. Water treatment facilities shall be designed to achieve appropriate microbial treatment, including a minimum of 3-log reduction of *Giardia*, 4-log reduction of viruses, and 2-log reduction of *Cryptosporidium* (subject to water quality monitoring data) through physical removal and chemical inactivation.
2. Filters shall be designed for filter-to-waste operation after a backwash.
3. Filters shall be designed with an auxiliary backwash system, using either air scour or surface wash.
4. Combined filter effluent turbidity shall be less than 0.1 nephelometric turbidity units (NTU) at all times.
5. Individual filter effluent turbidity shall be less than 0.1 NTU within 1 hour of bringing the filter online or after a backwash until the end of the filter run.
6. Facilities shall be designed for recycle of waste washwater decant. This recycle shall occur as necessary and be limited to less than 10 percent of WTP flow. All recycle streams shall be equalized prior to return. Sludge decant shall be managed through an alternative management strategy, such as disposal to the sewer or treatment and discharge.

5.2. REGULATORY BACKGROUND

This section reviews current and anticipated drinking water regulations as promulgated by the United States Environmental Protection Agency (USEPA) and the California Department of Health Services (DHS). Under the provisions of the Safe Drinking Water Act (SDWA), DHS has the primary enforcement responsibility (referred to as "primacy"). The Health and Safety Code of the California Administrative Code establishes DHS authority and stipulates drinking water quality and monitoring standards. To maintain primacy, a State's drinking water regulations can be no less stringent than the Federal standards (a State's regulations can be more stringent).

USEPA and DHS establish primary regulations for controlling contaminants that affect public health, and secondary regulations for compounds that affect the taste or aesthetics of drinking water. For each contaminant that is regulated, USEPA is required to establish a maximum contaminant level (MCL) or a treatment technique (TT) to limit the level of these compounds in drinking waters. USEPA is also required to recommend a Best Available Technology (BAT) for removing each contaminant during treatment.

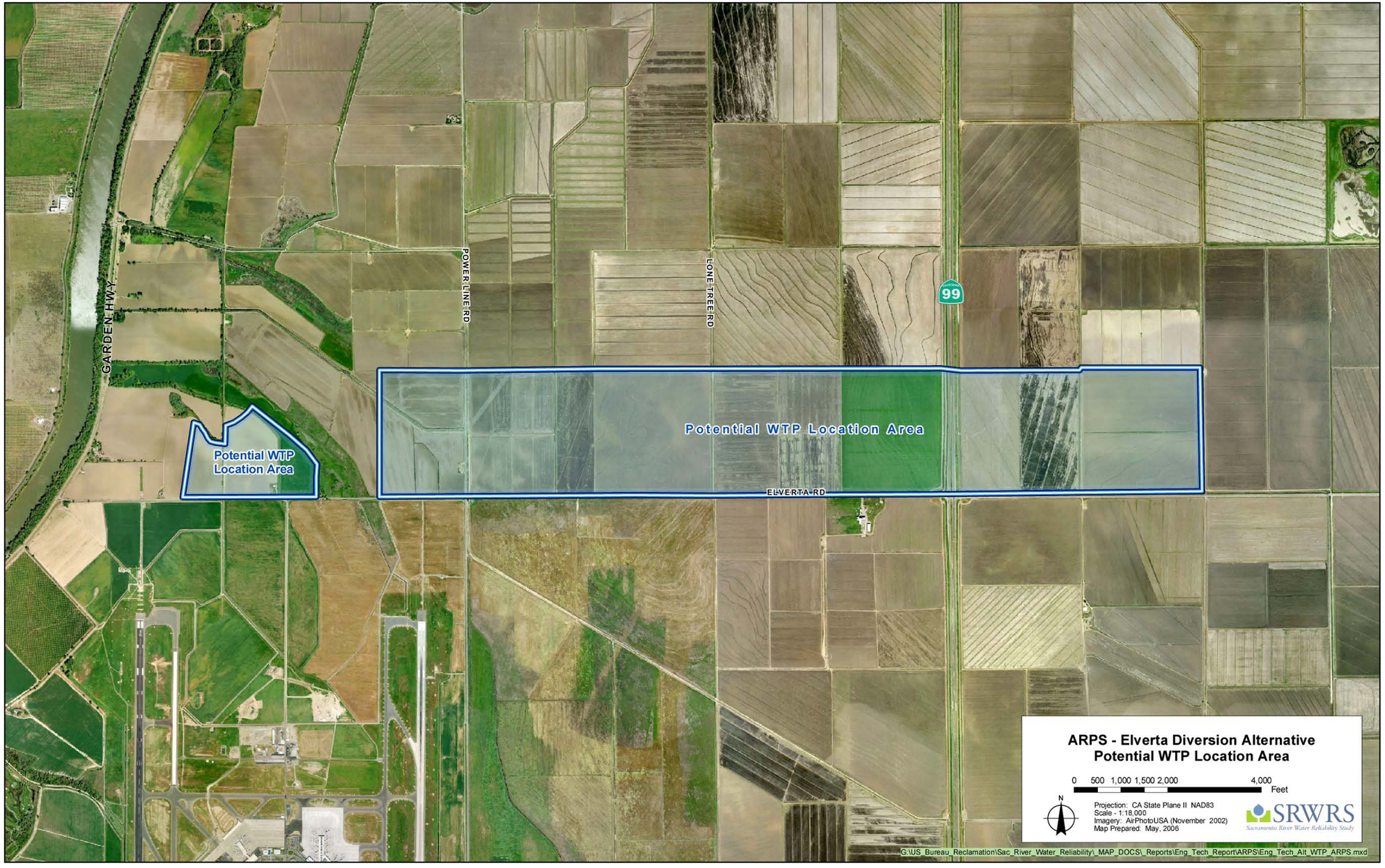

Figure 5-1 ARPS-Elverta Diversion Alternative Potential WTP Location Area

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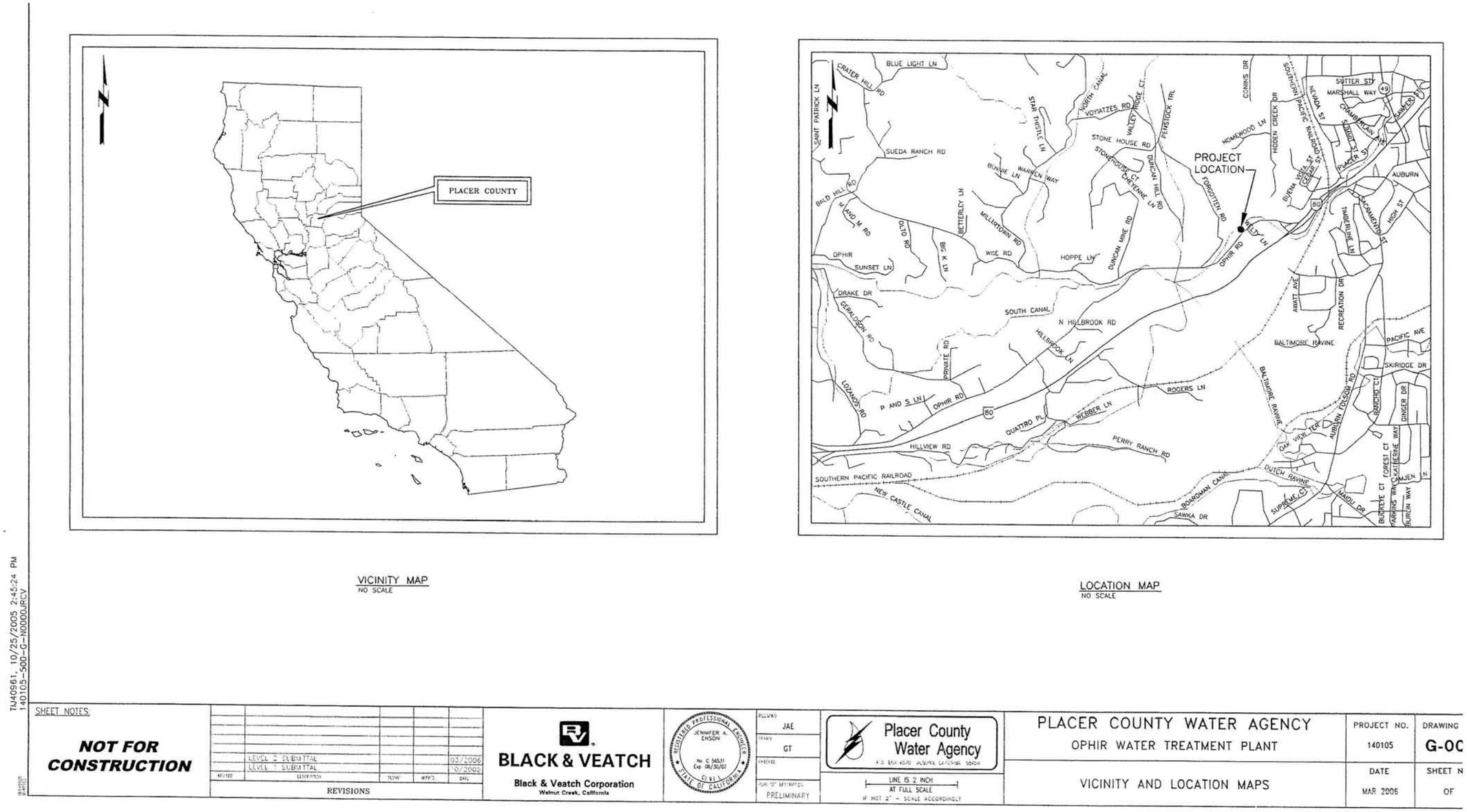

Figure 5-2 PCWA Ophir Preferred Water Treatment Plant Site

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5.2.1. Current Regulations

The most significant drinking water quality regulations are shown in **Table 5-1**. **Appendix A** summarizes each contaminant in drinking water currently regulated by both USEPA and DHS. The table identifies the regulation and the MCL or the TT associated with each contaminant listed. The following is a general discussion of the requirements of selected regulations.

Table 5-1 Summary of Major Federal and State Drinking Water Quality Regulations

Regulation	Year of Promulgation	Number of Contaminants	Targeted Contaminants
National Interim Primary Drinking Water Regulations (NIPDWR)	1975-1981	7	Trihalomethanes, Arsenic, Radiologicals
Phase I Standards	1987	8	VOCs
Phase II Standards	1991	36	VOCs, SOCs, and IOCs
Phase V Standards	1992	23	VOCs, SOCs, and IOCs
Surface Water Treatment Rule (SWTR)	1989	5	Microbiological and Turbidity
Total Coliform Rule (TCR)	1989	2	Microbiological
Lead and Copper Rule (LCR)	1991/2003[(1)]	2	Lead and Copper
Drinking Water Source Assessment and Protection Program	1996	-	Source Water Protection
Information Collection Rule (ICR)	1996	-	Microbiological and D/DBPs
Stage 1 Disinfectants/Disinfection By-Products (D/DBP) Rule	1998	14	D/DBPs and Precursors
Stage 2 D/DBP Rule	2006	9	DBPs
Interim Enhanced Surface Water Treatment Rule (ESWTR)	1998	2	Microbiological and Turbidity, Systems >10,000 people
Long-Term 2 ESWTR	2006	1	*Cryptosporidium*
Unregulated Contaminant Monitoring Rule	1999	36	Organics and Microbiological
Radionuclides Rule	2000	4	Radionuclides
Arsenic Rule	2001	1	Arsenic
Filter Backwash Rule	2001	-	Microbiological and Turbidity
Long-Term 1 ESWTR	2002	2	Microbiological and Turbidity, Systems <10,000 people
Drinking Water Candidate Contaminant List	2003	9	Various

Notes:
[(1)] California Adoption of Federal Rule Minor Revisions.
Key:
D/DBP – Disinfectants/Disinfection by-products
ESWTR – Enhanced Surface Water Treatment Rule
ICR – Information Collection Rule
IOC – Inorganic compounds
NIPDWR – National Interim Primary Drinking Water Regulations
VOC – Volatile organic compound
SOC – Synthetic organic compound
SWTR – Surface Water Treatment Rule
TCR – Total Coliform Rule

5.2.1.1. Surface Water Treatment Rule

The Surface Water Treatment Rule (SWTR) was promulgated to control the levels of turbidity, *Giardia lamblia*, viruses, *Legionella*, and heterotrophic plate count bacteria in United States drinking waters. Many of the detailed requirements of this regulation would be enhanced or superceded by the Interim and Long-Term 2 Enhanced Surface Water Treatment Rules (LT2ESWTR), described later.

The California SWTR requires all utilities using a surface water supply or a groundwater supply under the influence of a surface water supply, to provide adequate disinfection, and under most conditions, to provide filtration. Exemptions from filtration of surface water supplies are provided in rare occasions when the source water supply meets extremely rigid requirements for water quality and the utility possesses control of the watershed.

5.2.1.1.1. General Requirements

The SWTR includes the following general requirements to minimize human exposure to microbial contaminants in drinking water:

- Utilities are required to achieve at least 99.9 percent removal and/or inactivation of *Giardia lamblia* cysts (3-log removal) and a minimum 99.99 percent removal and/or inactivation of viruses (4-log removal). The required level of removal/inactivation must occur between the point where the raw water ceases to be influenced by surface water runoff to the point at which the first customer is served.
- The disinfectant residual entering the distribution system must not fall below 0.2 milligrams per liter (mg/L) for more than 4 hours during any 24-hour period.
- A disinfectant residual must be detectable in 95 percent of distribution system samples. A heterotrophic plate count (HPC) concentration of less than 500 colonies/milliliter (mL) can serve as a detectable residual if no residual is measured.
- Each utility must perform a watershed sanitary survey at least every 5 years.

5.2.1.1.2. Removal Credit

The level of removal credit given a utility for both *Giardia lamblia* and viruses is determined by the type of treatment process used. For a conventional WTP, the SWTR provides a 2.5-log removal credit for *Giardia lamblia* and a 2.0-log removal credit for viruses.

5.2.1.1.3. Disinfection Credit

Disinfection during conventional treatment (assuming all operational criteria and performance standards are met, and the plant receives 2.5-log credit for physical removal of *Giardia* and 2-log credit for physical removal of viruses) must achieve 0.5-log inactivation of *Giardia lamblia* and 2.0-log inactivation of viruses. To determine the inactivation of *Giardia lamblia* and viruses achieved at a WTP, the SWTR established the concept of CT. CT is the product of the concentration of disinfectant remaining at the end of a treatment process (“C” in mg/L) and the contact time in which 10 percent of the water passes through the treatment process (“T” or “T10” in minutes). The contact time in which 10 percent of the water travels through a unit process can be conservatively estimated from DHS guidelines or more accurately determined by conducting a tracer study. The USEPA Guidance Manual for the SWTR includes tables that identify the log removal of both *Giardia lamblia* and viruses achieved for a calculated CT value based on the type of disinfectant, the water temperature, and pH.

5.2.1.2. Stage 1 Disinfectants and Disinfection By-Products Rule

The purpose of the Stage 1 Disinfectants/Disinfection By-Product (D/DBP) Rule is "…to minimize risks from disinfection by-products and still maintain adequate control over microbial contamination."

5.2.1.2.1. Maximum Residual Disinfectant Level Goals

The USEPA has set maximum residual disinfectant level goals (MRDLG) for chlorine, chloramines, and chlorine dioxide, as shown in **Table 5-2**.

Table 5-2 Maximum Residual Disinfection Level Goals

Disinfectant	Goal
Chlorine	4 mg/L as Cl_2
Chloramines	4 mg/L as Cl_2
Chlorine Dioxide	0.8 mg/L as ClO_2

Key: mg/L – milligrams per liter

MRDLGs are set at levels for which no known or anticipated adverse health effects occur. These goals are non-enforceable health goals based only on health effects and exposure information.

5.2.1.2.2. Maximum Residual Disinfectant Levels

The Stage 1 D/DBP Rule established maximum residual disinfectant levels (MRDL) for chlorine, chloramines, and chlorine dioxide, as shown in **Table 5-3**.

Table 5-3 Maximum Residual Disinfectant Levels

Disinfectant	Level
Chlorine	4.0 mg/L as Cl_2
Chloramines	4.0 mg/L as Cl_2
Chlorine Dioxide	0.8 mg/L as ClO_2

Key:
mg/L – milligrams per liter

Chlorine. The residual disinfectant level must be monitored at the same points in the distribution system and at the same time as when sampling for total coliforms. Compliance with the MRDL would be based on the running annual average of the monthly average of all samples, computed quarterly. Plant operators could increase the residual chlorine level in the distribution system above the MRDL if necessary to protect public health from acute microbiological contamination problems, including distribution line breaks, storm runoff events, source water contamination, or cross-connections.

5.2.1.2.3. Maximum Contaminant Level Goals for TTHMs, HAA5, Chlorite, and Bromate

The USEPA has set maximum contaminant level goals (MCLGs) for four trihalomethanes, two haloacetic acids, chlorite, and bromate, as shown in **Table 5-4**. (The MCLGs for chloroform was removed by the USEPA on May 30, 2000.)

The MCLGs are set at levels for which no known or anticipated adverse health effects occur. These goals are non-enforceable health goals based only on health effects and exposure information.

Table 5-4 Maximum Contaminant Level Goals

Disinfection By-Product	Goal
Bromodichloromethane	0 mg/L
Dibromochloromethane	0.06 mg/L
Bromoform	0 mg/L
Dichloroacetic Acid	0 mg/L
Trichloroacetic Acid	0.3 mg/L
Chlorite	0.8 mg/L
Bromate	0 mg/L

Key:
mg/L – milligrams per liter

5.2.1.2.4. Maximum Contaminant Levels for TTHMs, HAA5, Chlorite, and Bromate

The Stage 1 D/DBP Rule set MCLs for total trihalomethane (TTHM), five haloacetic acids (HAA5), chlorite, and bromate, as shown in **Table 5-5**.

Table 5-5 Maximum Contaminant Levels

Contaminant	Level
TTHM[(1)]	0.080 mg/L
HAA5[(2)]	0.060 mg/L
Chlorite	1.0 mg/L
Bromate	0.010 mg/L

Notes:
[(1)]TTHM includes chloroform, bromodichloromethane, dibromochloromethane, bromoform.
[(2)] HAA5 includes mono-, di- and tri-chloroacetic acids and mono and di-bromoacetic acids.
Key:
mg/L – milligrams per liter

Total Trihalomethanes and Haloacetic Acids. TTHMs and HAA5 are formed when disinfectants react with naturally occurring organic matter in water. All systems must monitor the distribution system for TTHMs and HAA5. Compliance for surface water, groundwater under the direct influence of surface water (GWUDIS), and groundwater systems with a population greater than 10,000 is based on the running annual average of quarterly averages of all samples taken in the distribution system, computed quarterly.

5.2.1.2.5. Treatment Technique for Disinfection By-Product Precursors

The USEPA requires systems that have surface water or GWUDIS as a supply and use conventional filtration treatment, to remove specific amounts of organic material by implementing a treatment technique, either by enhanced coagulation or enhanced softening. The percent of removal required depends on source water total organic carbon (TOC) and alkalinity. **Table 5-6** summarizes removal requirements.

Compliance with this treatment technique must be calculated on a quarterly basis after 12 months of data are available. Each month the system must calculate percent actual TOC removal, determine the percent required TOC removal (from above), and calculate the removal ratio (must be greater than 1.0).

Table 5-6 TOC Removal Requirements

TOC Level, mg/L	Removal Percentage by Alkalinity Level		
	0 – 60 (mg/L)	> 60 – 120 (mg/L)	> 120 (mg/L)
> 2.0 - 4.0	35	25	15
> 4.0 - 8.0	45	35	25
> 8.0	50	40	30

Key:
mg/L – milligrams per liter TOC – total organic carbon

Systems can be granted a 1.0 ratio for the monthly removal ratio under the four following conditions (regardless of the calculated removal ratio):

- Remove greater than or equal to 10 mg/L of magnesium hardness (as calcium carbonate ($CaCO_3$)
- Raw water TOC is less than 2.0 mg/L
- Raw water or treated water specific ultraviolet absorbance (SUVA) is less than or equal to 2.0 liters per milligram-meter (L/mg-m)
- Treated water alkalinity is less than 60 mg/L (only for systems practicing enhanced softening)

The USEPA has also provided alternative compliance criteria from the treatment technique requirements. Utilities would not be required to achieve specified TOC removals provided one of the following conditions is met:

- Source water TOC is less than 2.0 mg/L
- Treated water TOC is less than 2.0 mg/L
- Source water TOC is less than 4.0 mg/L, source water alkalinity is greater than 60 mg/L, and distribution system TTHM is less than 0.04 mg/L and HAA5 is less than 0.03 mg/L
- Distribution system TTHM is less than 0.04 mg/L and HAA5 is less than 0.03 mg/L, and only chlorine is used for primary disinfection and distribution system residual
- Source water SUVA, prior to any treatment, is less than or equal to 2.0 L/mg-m
- Treated water SUVA is less than or equal to 2.0 L/mg-m

5.2.1.3. Stage 2 Disinfectants and Disinfection By-Products Rule

The Stage 2 D/DBP Rule was published in January 2006. It applies to public water systems, community water systems (CWS) or nontransient noncommunity water systems (NTNCW), that add a primary or residual disinfectant other than ultraviolet light or deliver water treated with a primary or residual disinfectant other than ultraviolet light.

The key provision in this rule is the change in calculating the MCL. Currently, compliance with the MCL is calculated using a running annual average (RAA) to average compliance samples from all distribution system sampling locations. Under the Stage 2 D/DBP Rule, the MCL will be calculated using locational running annual averages (LRAA). Public water systems must maintain the LRAA for each compliance sampling location at or below 0.080 mg/L total TTHM and 0.060 mg/L HAA5. All systems, including consecutive systems, must comply with the MCLs for TTHM and HAA5 LRAA using compliance sampling locations identified from the Initial Distribution System Evaluation (IDSE) Final Report.

5.2.1.3.1. Initial Distribution System Evaluation

An IDSE will be performed to identify locations with representative high TTHM and HAA5 concentrations throughout a system's retail distribution system. The IDSE results will be used in conjunction with the Stage 1 D/DBP Rule compliance monitoring to identify and select Stage 2 D/DBP Rule routine compliance monitoring locations. There are four IDSE options:

- Standard monitoring program
- System specific study (based on TTHM and HAA5 monitoring) and modeling requirements
- Obtainment of a 40/30 waiver
- Obtainment of a very small system waiver

Both the timing and number of IDSE and routine compliance monitoring are based on the retail population, served by the individual public water system(s). For example, the timing of when the IDSE must be completed is based on either an individual system's retail population, or in the case of a combined distribution system, the retail population served by the largest system in that combined system. The numbers of IDSE samples in the standard monitoring option are based on each individual system's retail population.

5.2.1.3.2. Compliance Monitoring

Compliance with the Stage 2 D/DBP Rule will be based on calculating a LRAA, where compliance means maintaining the annual average at each compliance sampling location in the distribution system at or below 0.080 mg/L and 0.060 mg/L for TTHM and HAA5, respectively. This is in lieu of the RAA MCL calculation under the Stage 1 D/DBP Rule that averaged observed values across distribution system compliance sampling locations. Monitoring for the LRAA will occur at compliance sampling locations identified in the IDSE Final Report at specific frequencies based on system population.

If a water system is required to conduct quarterly monitoring, it must make compliance calculations at the end of the fourth calendar quarter that follows the compliance date and at the end of each subsequent quarter (or earlier if the LRAA calculated based on fewer than four quarters of data would cause the MCL to be exceeded regardless of the monitoring results of subsequent quarters). If a system is required to conduct monitoring at a frequency that is less than quarterly, it must make compliance calculations beginning with the first compliance sample taken after the compliance date.

5.2.1.3.3. Operational Evaluation Levels

The Stage 2 D/DBP Rule includes the concept of operational evaluation levels. Operational evaluation levels trigger a system to evaluate system operational practices and identify opportunities to reduce DBP concentrations in the distribution system in order to reduce the potential the system will exceed the MCL. The Stage 2 DBP operational evaluation levels are identified using the system's Stage 2 D/DBP Rule compliance monitoring results.

Operational evaluation levels are calculated as follows:

If $(Q1 + Q2 + 2Q3)/4 > MCL$, then the system must conduct an operational evaluation

Where:

Q3 = current quarter measurement
Q2 = previous quarter measurement
Q1 = quarter before previous quarter measurement.
MCL = Stage 2 MCL for TTHM (0.080 mg/l) or Stage 2 MCL for HAA5 (0.060 mg/L).

The operational evaluation includes an examination of system treatment and distribution operational practices, including changes in sources or source water quality, storage tank operations, and excess storage capacity, which may contribute to high TTHM and HAA5 formation. Systems must also identify what steps could be considered to minimize future operational evaluation level exceedences.

5.2.1.3.4. Minimum Reporting Levels for Disinfection By-Products

The rule establishes regulatory minimum reporting limits (MRL) for compliance reporting of DBPs by public water systems. These regulatory MRLs also define the minimum concentrations that must be reported as part of the Consumer Confidence Reports. Beginning April 1, 2007, quantitative data must be completed for concentrations at least as low as those listed for all DBP samples analyzed for compliance.

5.2.1.3.5. Maintain TOC < 4 mg/L for Reduced TTHM and HAA5 Monitoring

To qualify for reduced routine compliance monitoring for TTHM and HAA5, subpart H systems (i.e., systems that use surface water supplies or GWUDIS) that are not monitoring to demonstrate compliance with TOC removal requirements of Stage 1 D/DBP Rule (i.e., plants that are not conventional filtration designs) must take TOC samples every 30 days at a location prior to any treatment, beginning April 1, 2008 or earlier, if specified by the State. The source water TOC RAA must be <4.0 mg/L (based on the most recent four quarters of monitoring) on a continuing basis at each treatment plant to reduce or remain on reduced monitoring for TTHM and HAA5. After demonstration of the TOC level compliance, the system may reduce monitoring to every 90 days.

Systems on a reduced monitoring schedule may remain on that reduced schedule as long as the average of all samples taken in the year (for systems that must monitor quarterly), or the results of the sample (for systems that must monitor no more frequently than annually) are no more than 0.060 mg/L and 0.045 mg/L for TTHMs and HAA5, respectively.

5.2.1.4. Interim Enhanced Surface Water Treatment Rule

The Interim Enhanced Surface Water Treatment Rule (ESWTR) applies to public water systems that use surface water or GWUDIS and serve a population greater than 10,000. The purpose of this regulation is "...to improve control of microbial pathogens, including specifically *Cryptosporidium*, in drinking water; and address risk trade-offs with disinfection by-products."

5.2.1.4.1. Cryptosporidium

The Interim ESWTR set an MCLG of zero (0) for the protozoan genus *Cryptosporidium*. Since no reliable means exists for monitoring this constituent in the drinking water at the time of promulgation, a treatment technique requirement was established in lieu of setting an MCLG. The treatment technique requires a 2-log (99 percent) *Cryptosporidium* removal or control for public water systems that are currently required to filter under the existing ESWTR. This removal must be achieved between the raw water intake and the first customer.

The rule provides that systems with conventional or direct filtration WTPs would be granted the 2-log removal credit if turbidity requirements are met for the existing SWTR (1.0/5.0 NTU) and the combined filter effluent requirements for this rule (0.3/1.0 NTU).

The rule also provides that systems with slow sand or diatomaceous earth filtration WTPS would be granted the 2-log removal credit if turbidity requirements are met for the existing SWTR (1.0/5.0 NTUs).

5.2.1.4.2. Turbidity

For surface water and GWUDIS systems that are required to filter their source water under the existing SWTR, and that employ conventional or direct filtration for treatment, the combined filter effluent turbidity requirements have been tightened. For alternative filtration technologies, the State would set turbidity performance requirements at a level that, in combination with disinfection, would consistently achieve 99.9 percent removal/inactivation of *Giardia*, 99.99 percent removal/inactivation of viruses, and 99 percent removal of *Cryptosporidium*.

The combined filter effluent turbidity must be less than 0.3 NTU in 95 percent of measurements and may never exceed 1 NTU (based on 4-hour measurements). The combined filter effluent turbidity shall not exceed 1.0 NTU for more than 8 hours (based on 15-minute measurements). Combined filter effluent and individual filter effluent continuous turbidity monitoring shall be recorded every 15 minutes. Monthly reports must show the total number of measurements taken, and have two options for value reporting:

- Report 15-minute measurements and show the 50th, 90th, 95th, 98th, and 99th percentiles, and report all measurements greater than 1 NTU
- Report 4-hour measurements and show all results greater than 0.3 NTUs (based on 15-minute measurements), and percent of measurements less than or equal to 0.3 NTUs (based on 15-minute measurements).

The rule requires continuous, online measurement of turbidity for each individual filter. These data must be recorded every 15 minutes. Systems with two or fewer filters may conduct continuous monitoring of the combined filter effluent turbidity in lieu of individual monitoring. Individual filter effluent turbidity monitoring shall be less than 0.3 NTUs within 60 minutes after return to service.

DHS is expected to add several other requirements to the rule, including the following:

- All filters shall be visually inspected once per year as part of the operations plan based on DHS guidance.
- Raw water shall be sampled for total coliform and either fecal coliform or *E. Coli* at least once per month.
- Chlorine residual shall be confirmed in 95 percent of distribution samples every month.
- Online turbidimeters shall be manually verified once per week for combined filter effluent and once per month for individual filter effluent.
- Turbidity shall be recorded and reported for sedimentation effluent at least once per day.
- Flow rate and turbidity shall be recorded and reported for recycled backwash water at least once per day.
- System must report turbidity data to the State within 10 days after the end of each month.

5.2.1.4.3. Disinfection Profiling and Benchmarking

The purpose of disinfection profiling and benchmarking is to develop a process to assure no significant reduction in microbial protection occurs as a result of significant disinfection process modifications to meet the new MCLs for TTHMs and HAA5 from the Stage 1 D/DBP Rule.

Profiling would be required for surface water systems that have either TTHM levels greater than or equal to 80 percent of the new MCL (0.064 mg/L) or HAA5 levels greater than or equal to 80 percent of the new MCL (0.048 mg/L).

The disinfection profile is developed using a minimum of 1 year of weekly *Giardia lamblia* log inactivation. The month with the lowest average log inactivation shall be identified as the critical period or benchmark.

After profiling and benchmarking is complete, a utility must submit this information to the State as part of the sanitary survey. If a utility decides to make changes to disinfection practices, the utility must consult with the State to ensure that microbial protection is not compromised.

5.2.1.4.4. Finished Water Reservoirs

Under this rule, surface water and GWUDIS systems must cover all new treated water reservoirs, holding tanks, and other storage facilities.

5.2.1.4.5. Sanitary Surveys

Primacy states, such as California, must now conduct sanitary surveys for all surface water and GWUDIS systems, regardless of size. These surveys must be conducted every 3 years for CWSs and every 5 years for noncommunity water systems (NCWS). DHS may grant a waiver to water utilities to perform the sanitary survey every 5 years if the system has outstanding performance based on previous sanitary surveys. DHS must determine how outstanding performance would be evaluated to allow for the reduced frequency of the sanitary survey.

Sanitary surveys must meet the eight components of the 1995 USEPA/State Guidance. These components include source assessment, treatment, distribution system, finished water storage, pumps, pumping facilities and controls, monitoring and reporting, data verification, system management and operation, operator compliance with State requirements, and disinfection profiling (if required).

5.2.1.5. Long Term 2 Enhanced Surface Water Treatment Rule

The LT2ESWTR was published by the USEPA in early January 2006 in the Federal Register. This regulation will apply to all public water systems that use surface water or GWUDIs.

The LT2ESWTR includes deadlines that directly affect drinking water utilities of all sizes, and many will have to meet deadlines later this year. Some systems serving more than 100,000 people will have to submit detailed monitoring plans under the LT2ESWTR by July 1, 2006. The Major Milestone Schedule for Stage 2 D/DBP Rule and LT2ESWTR Implementation provides an overview of key monitoring, reporting, and compliance milestones under both rules.

The requirements for filtered and unfiltered systems are different. This section summarizes only the requirements for filtered systems.

5.2.1.5.1. Source Water Monitoring

Filtered systems are not required to conduct source water monitoring if the system will provide a total of at least 5.5-log of treatment for *Cryptosporidium*. Otherwise, public water systems using surface water or GWUDI are required to monitor their source water (i.e., the influent water entering the treatment plant) monthly for 24 months to determine an average *Cryptosporidium* level. As described in the next section, monitoring results determine the extent of *Cryptosporidium* action requirements under the LT2ESWTR. Large systems must also monitor for *E. coli* and turbidity at the same time in source water.

Systems must adhere to the sampling plan and report results no later than 10 days after the end of the first month following the month when the sample is collected. All systems serving at least 10,000 people must report the results from the initial source water monitoring to USEPA electronically using the Central Data Exchange (CDX). Submission of historical (grandfathered) data is allowed when it meets the quality assurance and quality control requirements specified in the rule.

Systems serving less than 10,000 persons may use *E. coli* as a surrogate indicator for *Cryptosporidium.* However, if the *E. coli* levels are sufficiently high, these systems must then undertake *Cryptosporidium* monitoring.

The rule also includes a second round of *Cryptosporidium* sampling for all systems. This second round of sampling will take place 6 years following bin classification for the source water.

5.2.1.5.2. Analytical Method

Systems must analyze for *Cryptosporidium* using either USEPA Method 1623 or Method 1622. Systems must analyze at least a 10-liter (L) sample or a packed pellet volume of at least 2 mL. The rule contains specific quality assurance and quality control requirements. Only EPA-approved laboratories can perform the *Cryptosporidium* sample analysis. Specific analytical methods are also specified for turbidity and *E. coli* measurements required by the rule.

5.2.1.5.3. Sampling

Filtered systems serving at least 10,000 people must sample their source water for *Cryptosporidium*, *E. coli*, and turbidity at least monthly for 24 months. Filtered systems serving fewer than 10,000 people must sample their source water for *E. coli* at least once every 2 weeks for 12 months. Filtered systems serving fewer than 10,000 people must sample their source water for *Cryptosporidium* at least twice per month for 12 months or at least monthly for 24 months if the system does not conduct *E. coli* monitoring, or if the initial E. coli sample exceed the following criteria:

- For systems using lake/reservoir sources, the annual mean E. coli concentration is greater than 10 *E. coli*/100 mL.
- For systems using flowing stream sources, the annual mean E. coli concentration is greater than 50 *E. coli*/100 mL.

Systems must collect samples within a 5-day period around the schedule date. If an extreme condition or situation exists that may pose danger to the sample collector, or that cannot be avoided and causes the system to be unable to sample, the system must sample as close to the scheduled date as is feasible unless the State approves an alternative sampling date. The system must submit an explanation for the delayed sampling date to the State concurrent with the shipment of the sample to the laboratory. If a system is unable to report a valid analytical result for a scheduled sampling date due to equipment failure, loss of or damage to the sample, failure to comply with the analytical method requirements, including the quality

control requirements, or the failure of an approved laboratory to analyze the sample, then the system must collect a replacement sample.

Replacement samples should be collected not later than 21 days after receiving information that an analytical result cannot be reported for the scheduled date unless the system demonstrates that collecting a replacement sample within this time frame is not feasible or the State approves an alternative re-sampling date. The system must submit an explanation for the delayed sampling date to the State concurrent with the shipment of the sample to the laboratory. Systems that fail to meet these criteria for any source water sample must revise their sampling schedules to add dates for collecting all missed samples. Systems must submit the revised schedule to the State for approval prior to when the system begins collecting the missed samples.

5.2.1.5.4. Monitoring Location

Systems must collect samples for each plant that treats a surface water or GWUDI source. Where multiple plants draw water from the same influent, such as the same pipe or intake, the State may approve one set of monitoring results to be used for all plants. Systems must collect source water samples prior to chemical treatment, such as coagulants, oxidants, and disinfectants. The State may approve a system to collect a source water sample after chemical treatment. To grant this approval, the State must determine that collecting a sample prior to chemical treatment is not feasible for the system and that the chemical treatment is unlikely to have a significant adverse effect on the analysis of the sample. Systems that recycle filter backwash water must collect source water samples prior to the point of filter backwash water addition. Specific requirements are included from bank filtration and other special cases.

A system that begins using a new source of surface water or GWUDI after the system is required to begin monitoring under paragraph (c), the monitoring section of the LT2ESWTR, and must monitor the new source on a schedule the State approves.

5.2.1.5.5. Monitoring and Treatment Compliance Dates

Starting dates for monitoring are staggered by system size, with smaller systems beginning monitoring after larger systems. Milestones for monitoring, reporting, and compliance occur first for very large systems (>100,000 persons), then systems serving 50,000 to 99,999 persons, followed by systems serving 10,000 to 49,999 persons, and finally systems serving fewer than 10,000. Populations are based on retail population.

5.2.1.5.6. Bin Classification Table for Filtered Systems

Filtered water systems will be classified in one of four categories or bins based on their monitoring results. The rule specifies several calculation procedures depending on how many samples were collected or if the sample frequency was not consistent.

Bin placement may be calculated as follows:

- Total of at least 48 samples; the bin concentration is equal to the arithmetic mean of all sample concentrations.
- Total of at least 24 samples, but not more than 47 samples; the bin concentration is equal to the highest arithmetic mean of all sample concentrations in any 12 consecutive months during which *Cryptosporidium* samples were collected.

- For systems that serve fewer than 10,000 people and monitor for *Cryptosporidium* for only one year (i.e., collect 24 samples in 12 months), the bin concentration is equal to the arithmetic mean of all sample concentrations.
- For systems with plants operating only part of the year that monitor fewer than 12 months per year under § 141.701(e) of the LT2ESWTR, the bin concentration is equal to the highest arithmetic mean of all sample concentrations during any year of *Cryptosporidium* monitoring.

Additional action for *Cryptosporidium* (beyond 3.0-log reduction awarded for conventional filtration) will be based on source water concentrations of the protozoa and the type of treatment implemented at the plant. If the maximum running annual average (MRAA) is less than 0.075 oocysts/L, the source is assigned Bin 1 classification and no additional action is required. Assuming conventional filtration credit, if the MRAA is between 0.075 and 1.0 oocysts/L, the source is assigned to Bin 2 and 1-log action is required; if the MRAA is between 1.0 and 3.0 oocysts/L, the source is assigned to Bin 3 and 2-log action required; and if the MRAA is greater than 3.0 oocysts/L, the source is assigned to Bin 4 and 2.5-log action required.

Systems classified in Bins 2, 3, and 4 must provide 1.0- to 2.5-log additional action for *Cryptosporidium*. Systems will select from a wide range of treatment and management strategies in the "microbial toolbox" to meet their additional action requirements. Systems classified Bin 3 and Bin 4 must achieve at least 1 log of additional treatment using either one or a combination of the following: bag filters, bank filtration, cartridge filters, chlorine dioxide, membranes, ozone, or ultraviolet (UV) light.

5.2.1.5.7. Microbial Toolbox

Public water systems can achieve additional *Cryptosporidium* treatment credit through implementing pretreatment processes, such as presedimentation or bank filtration, by developing a watershed control program, and by applying additional treatment steps like ozone, chlorine dioxide, UV, and membranes. In addition, public water systems can receive a higher level of credit for existing treatment processes through achieving superior filter effluent turbidity or through a demonstration of performance. Taken as a whole, this list of control options is termed the "microbial toolbox." public water systems may use one or more tools to accumulate the needed treatment credits to meet the treatment requirement associated with their bin classification.

5.2.1.5.8. UV Dose Table

Systems receive *Cryptosporidium*, *Giardia lamblia*, and virus treatment credits for UV light reactors by achieving the UV dose values described in the rules. Systems must validate and monitor UV reactors to demonstrate that they are achieving a particular UV dose value for treatment credit. UV reactor validation must occur at full-scale using a test microbe with quantified dose-response characteristics using low-pressure mercury lamps. Validation must include operating conditions of flow rate, UV intensity as measured by a UV sensor, and UV lamp status, as well as other considerations, including lamp fouling and inlet/outlet hydraulics. To receive treatment credit for UV light, systems must treat at least 95 percent of the water delivered to the public during each month by UV reactors operating within validated conditions for the required UV dose.

5.2.1.5.9. CT Tables

CT is the product of the disinfectant contact time (T, in minutes) and disinfectant concentration (C, in mg/L). Systems with treatment credit for chlorine dioxide or ozone must calculate CT at least once each day, with both C and T measured during peak hourly flow. Systems with several disinfection segments in

sequence may calculate and sum the CT for each segment, where a disinfection segment is defined as a treatment unit process with a measurable disinfectant residual level and a liquid volume. Systems receive the *Cryptosporidium* treatment credit by meeting the corresponding CT value for the applicable water temperature specified in CT tables specified in the rule.

5.2.1.5.10. Open Finished Water Reservoirs

Until now, regulations required public water systems to cover all new storage facilities for finished water but did not address existing uncovered finished water storage facilities. Under the LT2ESWTR, public water systems using uncovered finished water storage facilities must either cover the storage facility or treat the storage facility discharge to achieve inactivation and/or removal of 4-log virus, 3-log *Giardia lamblia*, and 2-log *Cryptosporidium* on a State-approved schedule.

5.2.1.5.11. Microbial Profiling and Benchmarking

Following the completion of initial source water monitoring (date varies by system size), a system that plans to make a significant change to its disinfection practice must develop disinfection profiles and calculate disinfection benchmarks for *Giardia lamblia* and viruses. Significant changes to disinfection practice are defined as follows:

- Changes to the point of disinfection
- Changes to the disinfectant(s) used in the treatment plant
- Changes to the disinfection process
- Any other modification identified by the State as a significant change to disinfection practice

5.2.1.6. Arsenic Rule

The Final Arsenic Rule was promulgated by the USEPA on January 22, 2001. The rule sets an MCLG of 0 mg/L and an MCL of 0.010 mg/L (10 µg/L) for arsenic. DHS has not yet adopted this regulation and the State version may be more stringent (see later discussion).

5.2.1.7. Filter Backwash Recycling Rule

The Final Filter Backwash Recycling Rule applies to all public water systems that use surface water and employ conventional or direct filtration and recycle water within the WTP.

This rule requires all recycle streams to pass through all treatment processes; therefore, all streams need to be returned prior to chemical addition and coagulation. Also, each system must notify DHS in writing that it practice recycling. This notification must include a plant schematic that shows the type and location of recycle streams, typical recycle flow data, highest plant flow in the previous year, design flow of the plant, and DHS-approved operating capacity.

Each system must collect and maintain the following information: copy of recycle notice to DHS, list of all recycle flows and frequency, average and maximum backwash flow rate and duration, typical filter run length and how determined, type of recycle treatment, and data on recycle treatment facilities.

5.2.2. Anticipated Regulations

The USEPA and DHS are developing new regulations. Major anticipated regulations that would impact surface water supplies are shown in **Table 5-7**, and selected regulations are discussed below.

Table 5-7 Summary of Anticipated Major Federal and State Drinking Water Quality Regulations for Surface Water Supplies

Regulation	Year Final Expected	Number of Contaminants	Targeted Contaminants
Perchlorate[(1)]	2004	1	Perchlorate
Arsenic[(2)]	2004	1	Arsenic
Hexavalent Chromium[(1)]	2004	1	Hexavalent Chromium
Drinking Water Candidate Contaminant List/ Unregulated Contaminant Monitoring Rule	2007	-	Microbiological and Chemical
Distribution System Rule/Revised Total Coliform Rule	2008	-	Microbiological

Notes:
[(1)]California rule only.
[(2)]California adoption of Federal rule expected to be more stringent.

5.2.2.1. California Arsenic Regulation

DHS is required to develop a revised arsenic standard for drinking water in California by June 30, 2004. This may be delayed due to change in the governor's administration. The Office of Environmental Health Hazard Assessment (OEHHA) has developed a Public Health Goal (PHG) for arsenic of 4 mg/L. This is well below the current MCL of 10 µ/L. DHS is currently developing a revised MCL using this information.

5.2.2.2. California Hexavalent Chromium Regulation

DHS was required to develop a new hexavalent chromium standard for drinking water in California by January 1, 2004. This has been delayed due change in the governor's administration. OEHHA repealed the PHG of 0.2 µg/L and OEHHA was to final a PHG in 2003. DHS plans to develop an MCL for hexavalent chromium shortly after publication of the PHG.

5.2.2.3. Drinking Water Candidate Contaminant List/ Unregulated Contaminant Monitoring Rule

The 1996 SDWA Amendments provided a list of chemical and microbial contaminants for possible future regulation. Every 5 years, USEPA selects at least five contaminants from the list and determine whether to continue to regulate them. The regulations would be determined based on risk assessment and cost-benefit considerations and on minimizing overall risk. USEPA developed a draft second list for determination in April 2004.

The Unregulated Contaminant Monitoring Rule (UCMR) requires CWSs to conduct "treated" water monitoring of specified unregulated constituents. The purpose is to assist USEPA in collecting information about contaminants present in drinking water supplies that are currently unregulated. In agreement with the Contaminant Candidate List, the next UCMR, expected in 2004 or 2005, would be revised to reflect current constituents of concern.

5.2.2.4. Distribution System Rule/Revised Total Coliform Rule

USEPA conducted a review of 69 existing drinking water regulations in April 2002. USEPA determined only the Total Coliform Rule (TCR) was a candidate for revision. USEPA conducted two meetings with

experts to identify major distribution system issues. From these meetings, nine white papers were developed on the most critical subjects, including the following:

- Cross connection control
- Aging infrastructure and corrosion
- Permeation and leaching
- Nitrification
- Biofilms/growths
- Covered storage
- Decay in water quality over time
- New/repaired water mains

USEPA plans to publish a revised TCR by 2006 and a final rule by 2008.

5.3. WATER QUALITY EVALUATION

Below is a summary of water quality at two of the locations identified as parts of the ARPS-Elverta Diversion Alternative. Several monitoring programs were queried to obtain available water quality data between 1992 and 2002. **Table 5-8** summarizes those programs and the data acquired.

5.3.1. Description of Sampling Locations

Several sampling locations are identified in the monitoring programs used as resources for water quality data. On the Sacramento River, one monitoring site is located upstream at Verona, one just downstream at Veteran's Bridge, and one farther downstream at Bryte Bend. These sites have available data for developing a characterization of the water for treatment purposes. No major discharges occur into the Sacramento River between Verona and Bryte Bend, but in-river activities and minor discharges occur that would cause some deterioration of the water quality downstream. Below is a brief description of each sampling location, including its location relative to the Elverta Intake and ARPS Intake locations.

- Sacramento River at Verona: This sample site is located on the Sacramento River, just downstream from the confluence with the Feather River. At this location, both major agricultural drains (Colusa Basin Drain and Sacramento Slough) have discharged to the Sacramento River. This site is located upstream from the Elverta Intake site.
- Sacramento River at Veteran's Bridge: This sample site is located on the Sacramento River at Veteran's Bridge, the Interstate 5 crossing, just downstream from the Elverta Intake site.
- Sacramento River at Bryte Bend WTP: This sample site is located on the Sacramento River, near the Interstate 80 crossing ,downstream from the Elverta Intake site.
- North Fork of the American River at PCWA Pump Station: This sample site is located on the North Fork of the American River, where PCWA currently diverts water. This is the same site as the PCWA ARPS Intake site.

Table 5-8 Monitoring Program Summary

Program	Monitoring Period	Parameters	Location of Sample Site (s)
USGS National Ambient Water Quality Assessment Program	February 1996 through April 1998	• Total dissolved solids • Dissolved organic carbon • General water quality: iron, temperature, conductivity, pH, alkalinity, hardness, suspended solids • Rice herbicides: molinate, thiobencarb, carbofuran	• Feather River near Nicolaus • Sacramento River at Verona
Sacramento River Watershed Program	June 1998 through May 2002	• Total dissolved solids • Coliforms, protozoa • Dissolved organic carbon at UV254 • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, suspended solids, turbidity • Organics: diazinon, molinate, thiobencarb	• Feather River at Nicolaus • Sacramento River at Veteran's Bridge
SRCSD Coordinated Monitoring Program	December 1992 through June 2002	• Total dissolved solids • Coliforms, protozoa • Total and dissolved organic carbon at UV254 • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, suspended solids, turbidity • Organics	• Sacramento River at Veteran's Bridge
DWR Municipal Water Quality Investigation	April 1994 through February 1998	• Total dissolved solids • Trihalomethane formation potential • Dissolved bromide	• Sacramento River at Bryte Bend WTP
City of West Sacramento	January 1995 through December 1999	• Total dissolved solids • Coliforms • UV254 • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, turbidity	• Sacramento River at Bryte Bend WTP
PCWA	January 1993 through December 1997	• Total dissolved solids • Coliforms • General water quality: nutrients, metals, minerals, temperature, conductivity, pH, alkalinity, hardness, turbidity	• North Fork of the American River at the PCWA Pump Station

Key:
DWR – Department of Water Resources
PCWA – Placer County Water Agency
SRCSD – Sacramento Regional County Sanitation District
USGS – United States Geological Survey
UV254 – ultraviolet 254
WTP – Water Treatment Plant

5.3.2. Discussion of Water Quality

Both sources of water are high quality surface waters. Downstream locations generally are more susceptible to contaminating activities and therefore may have a less preferable water quality. The American River is softer, with less buffering capacity and lower solids loading. Both rivers would provide a supply that could be treated to meet all existing State and Federal drinking water standards. Metals are not a concern, at levels for human health, on either river.

On the Sacramento River, a substantial agricultural input into the river occurs downstream from its confluence with the Feather River. The primary agricultural use is rice farming. Pesticides associated with rice farming, including molinate, thiobencarb and carbofuran, have primary and secondary drinking water standards. Monitoring data are available at several locations, and all data show that both primary and drinking water standards can be met. The secondary standard for thiobencarb is approached closely (0.7 µg/L versus 1.0 µg/L MCL) at several locations during the spring months. This may be a taste concern in the distribution system if not pre-treated. The cities of West Sacramento and Sacramento have facilities in place to conduct oxidation with potassium permanganate, as necessary.

5.3.3. Sacramento River at Elverta Intake Site

Table 5-9 summarizes the available general water quality of the Sacramento River near the Elverta Intake site, specific to drinking water purposes.

Table 5-9 General Water Quality of the Sacramento River near Elverta Intake Site

Constituent	Minimum	Maximum	Average	Median
Alkalinity, mg/L				
Verona	24	73	54	55
Veteran's Bridge	16	77	60	61
Bryte Bend WTP	25	92	58	58
Bromide, mg/L	<0.01	0.07	0.01	0.01
Bryte Bend WTP				
DOC, mg/L				
Verona	1.3	3.6	1.9	1.6
Veteran's Bridge	0.7	10	3.0	3.0
Hardness, mg/L				
Verona	24	69	52	54
Veteran's Bridge	28	97	59	59
Iron, mg/L				
Veteran's Bridge	0.356	2.0	0.86	0.61
Manganese, mg/L				
Veteran's Bridge	0.028	0.107	0.057	0.047
pH, units				
Verona	7.5	8.1	7.8	7.8
Veteran's Bridge	6.2	8.9	7.7	7.7
Bryte Bend WTP	6.7	8.4	7.6	7.6
Suspended Sediment, mg/L	24	117	59	49
Verona				
Specific Conductance, µS/cm				
Verona	62	186	131	135
Veteran's Bridge	21	316	155	155
Temperature, Celsius				
Verona	8.7	22.5	15.4	14.3
Veteran's Bridge	7.5	25	15.1	14.8
Bryte Bend WTP	7.2	25.2	16.4	16.1
TDS, mg/L				
Veteran's Bridge	11.5	165	105	104
Bryte Bend WTP	<50	135	89	88
TOC, mg/L	<0.2	6.6	2.9	3.0
Veteran's Bridge				
TSS, mg/L	4	200	41	32
Veteran's Bridge				
Turbidity, NTU				
Veteran's Bridge	3.75	81.2	26.6	24.6
Bryte Bend WTP	7	387	34	25
UV254, cm-1	0.0606	0.14	0.103	0.105
Veteran's Bridge				

Key:
µS/cm – microSiemens per centimeter
CM-1 – absorbance per centimeter
DOC – dissolved organic carbon
mg/L – milligrams per liter
NTU – nephelometric turbidity units
TDS – total dissolved solids
TOC – total organic carbon
TSS – total suspended solids
WTP – water treatment plant

Graphs of temperature, pH, total suspended solids (TSS), and organic carbon have been generated to look at the seasonal variability of each constituent (see **Figure 5-3** through **Figure 5-6**).

These graphs show that temperature is greatly affected by season, and is very predictable, with the lowest levels in the winter months (7 degrees Celsius (°C) and the highest levels in the late summer months (over 22 °C). The average temperature is 15 °C.

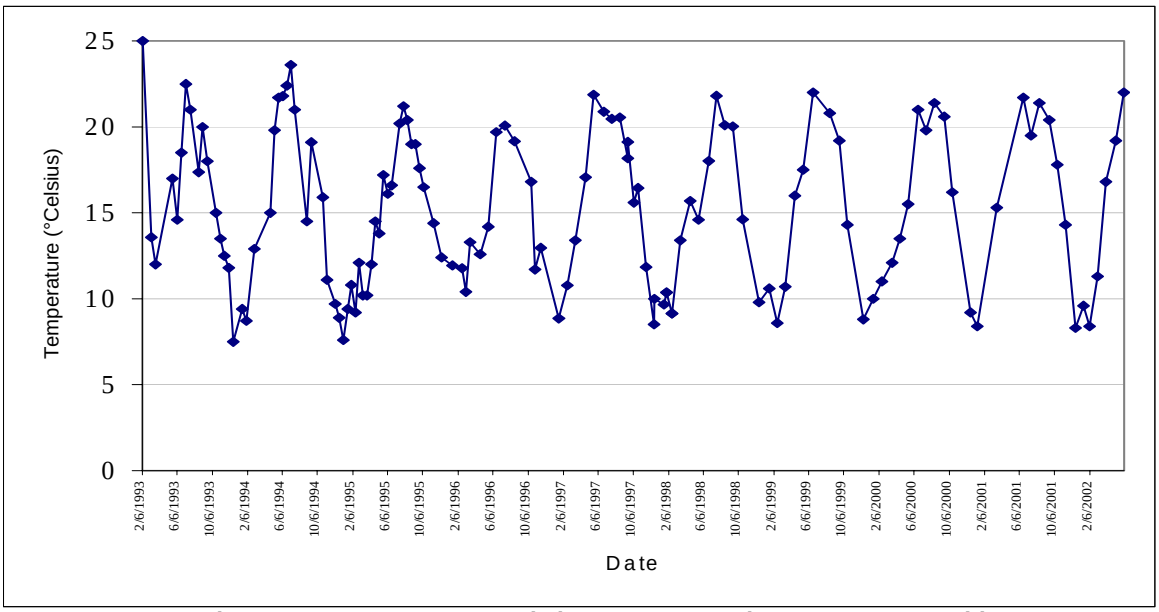

Figure 5-3 Temperature Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

Values for pH are also variable, but have less predictability than temperature. Extreme lows occur near 6.5 pH units and extreme highs near 9 pH units, but generally pH varies between 7 and 8 pH units.

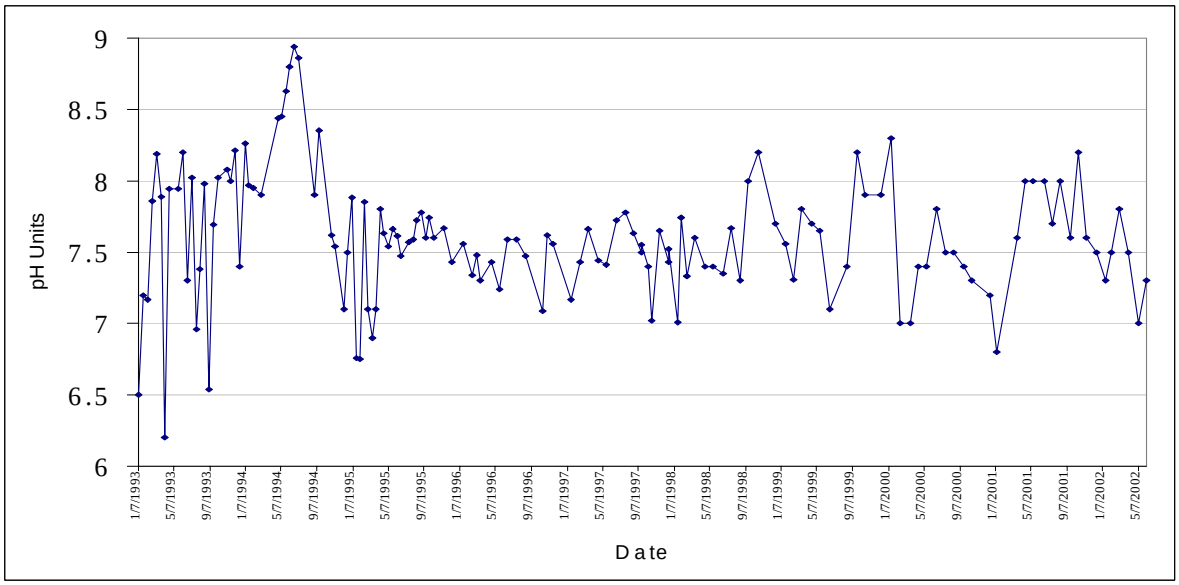

Figure 5-4 pH Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

TSS is highest during the winter months. This measurement is typically around 1.2 times higher than turbidity measurements. This shows that TSS can range from 4 to 200 mg/L, resulting in probable turbidity levels of 3 to 165 NTU, with an average of 28 to 34 NTU. This solids load is likely associated with wet weather events and releases from upstream reservoirs.

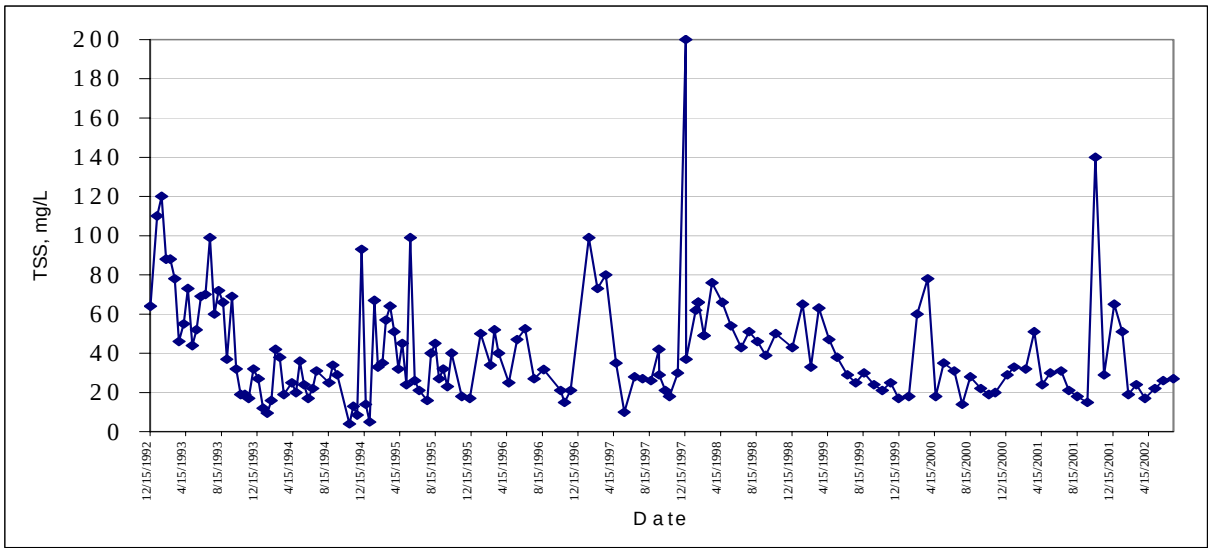

Figure 5-5 Total Suspended Solids at Sacramento River at Veteran's Bridge Coordinated Monitoring Program

TOC is a measure of the organic carbon in water and is recognized as a general indicator of the occurrence of DBP precursor material. The Coordinated Monitoring Program (CMP) monitors ambient river levels for TOC and dissolved organic carbon (DOC) (small particulate carbon). These data show that levels can range from 0.2 to 5.2 mg/L, with an average value just over 2 mg/L. The highest levels are seen in the late fall and winter months and the lowest levels are seen through the summer and early fall months. Intake data collected by the City of West Sacramento at the Bryte Bend WTP show only sporadic winter detects of TOC greater than 2 mg/L.

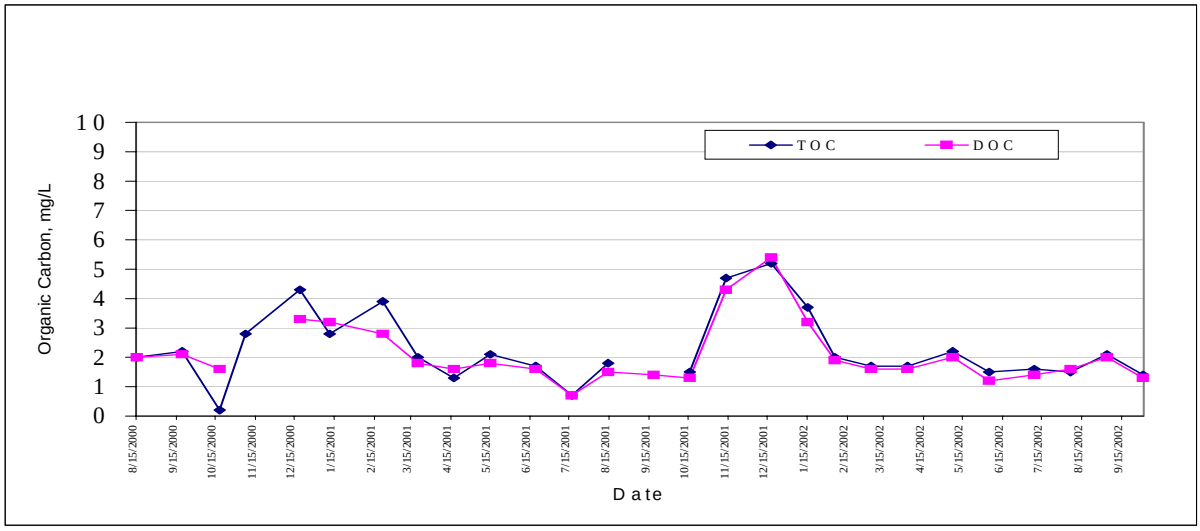

Figure 5-6 Organic Carbon Levels in Sacramento River at Veteran's Bridge Coordinated Monitoring Program

Data also are available for microbial constituents. Coliforms and protozoa have been monitored since 1998 at this site. **Table 5-10** summarizes the available data.

Table 5-10 Microbial Water Quality of the Sacramento River near Elverta Intake

Constituent	Minimum	Maximum	Geometric Mean
Total Coliform, MPN/100 mL			
Veteran's Bridge	17	16,000	480
Bryte Bend WTP	<2	>16,000	460
Fecal Coliform, MPN/100 mL			
Veteran's Bridge	2	2400	30
Bryte Bend WTP	<2	1300	30
E. Coli, MPN/100 mL			
Veteran's Bridge	<2	300	20
Bryte Bend WTP	<2	3000	20
Constituent	**No. of Samples**	**No. of Samples Positive**	**12-Month Running Annual Average**
Giardia, cysts/L	38	6	0.058
Cryptosporidium, oocysts/L	38	2	0.033

Key:
L – liter
mL – milliliter
MPN/100ml – most probable number per 100 milliliters
WTP – water treatment plant

Giardia and *Cryptosporidium* detects were primarily found in the late fall and early winter months. Average levels are low enough to ascertain that 3/4-log reduction of *Giardia* and viruses and 2-log reduction of *Cryptosporidium* are expected to be appropriate for the North Natomas WTP.

A graph of coliform over the sampling period is provided in **Figure 5-7**. It can be seen that coliform can peak throughout the year, but are mostly associated with wet weather events.

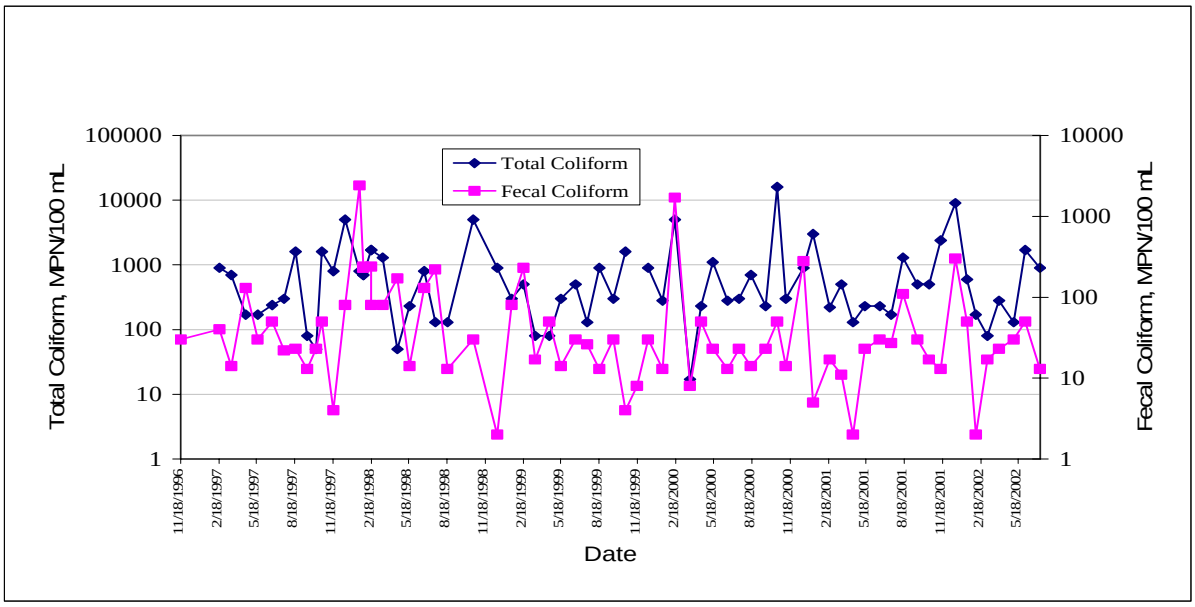

Figure 5-7 Coliform Levels at Sacramento River at Veteran's Bridge Coordinated Monitoring Program

Based on water quality data collected from other monitoring programs in the vicinity of the proposed Elverta Intake site, and presented herein, it appears that water quality is expected to be very good. It is recommended that a monitoring program be implemented at the proposed intake site to collect further data for turbidity, pH, alkalinity, TOC, *E. Coli*, *Giardia*, and *Cryptosporidium*. The SRWRS partners may also want to petition the California Department of Pesticide Regulation to monitor spring herbicide levels at the proposed diversion-site during the 2004 or 2005 monitoring program to compare with the Bryte Bend and Sacramento River WTP intakes.

5.3.4. American River at ARPS Intake Site

PCWA currently diverts water from its ARPS Intake site on a temporary basis and has water quality data to characterize the water for treatment purposes. **Table 5-11** summarizes the available general water quality, specific to drinking water purposes.

Table 5-11 General Water Quality of the American River at ARPS Intake Site

Constituent	Minimum	Maximum	Average	Median
Alkalinity, mg/L	16	34	23	-
Hardness, mg/L	12	20	16	16
pH, units	6.8	7.7	7.2	7.1
Specific Conductance, µmhos/cm	42	46	44	43
Temperature, Celsius	7.78	26.67	14.1	14.4
TDS, mg/L	36	53	42	38
TOC, mg/L	1.2	1.6	1.4	1.4
Turbidity, NTU	0.19	401.1	24.1	1.4

Key:
ARPS – American River Pump Station
mg/L – milligrams per liter
NTU – nephelometric turbidity unit
TDS – total dissolved solids
TOC – total organic carbon
µmhos/cm – micro mhos per centimeter

Graphs of temperature, pH, and turbidity have been generated to review the seasonal variability of each constituent (see **Figures 5-8** through **5-10**). These graphs show that temperature is greatly affected by season, and is very predictable, with the highest levels in the summer months and the lowest levels in the winter months. Turbidity is highest during the winter months, likely associated with wet weather events.

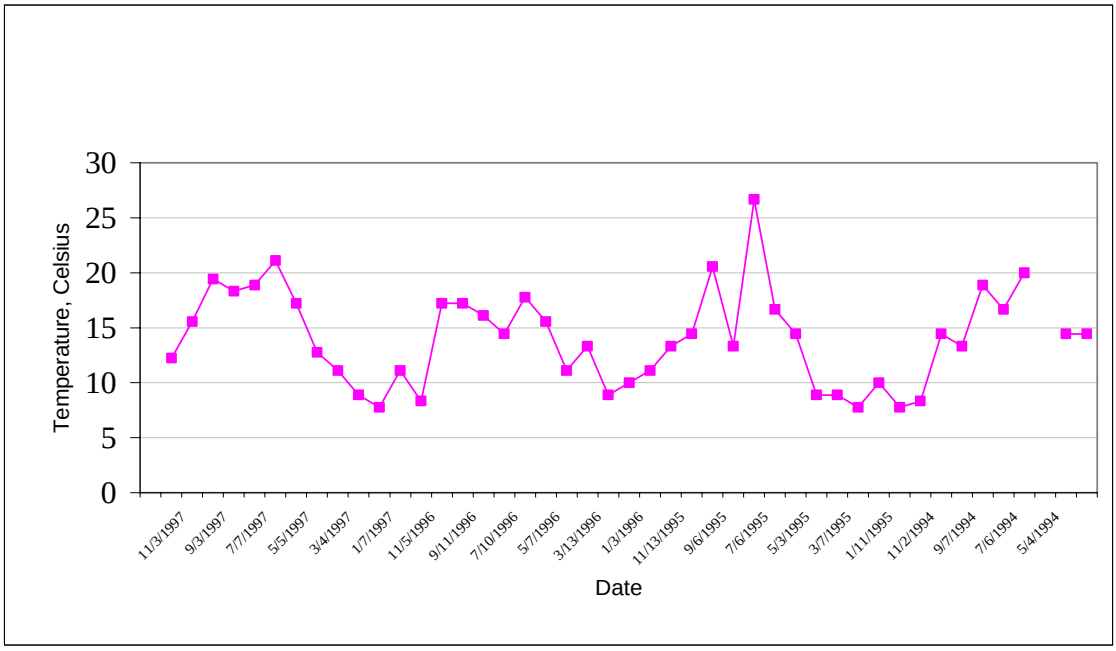

Figure 5-8 Temperature at American River at ARPS Intake Site

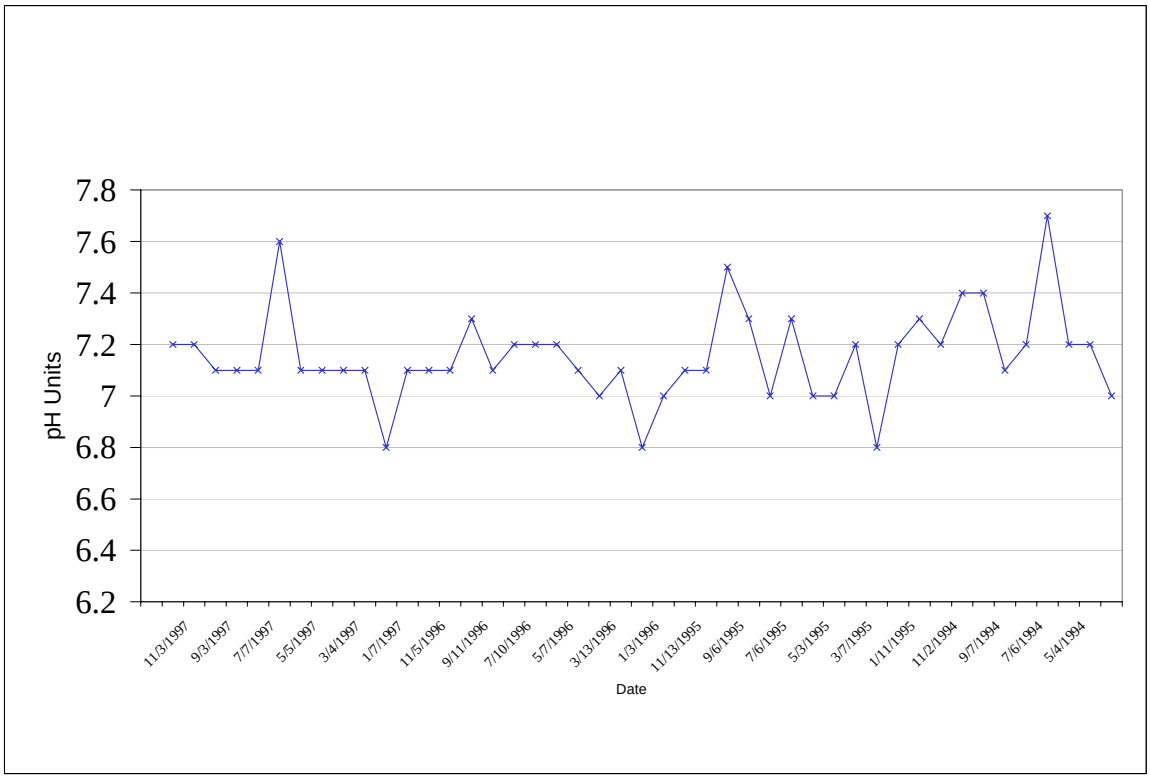

Figure 5-9 pH at American River at ARPS Intake Site

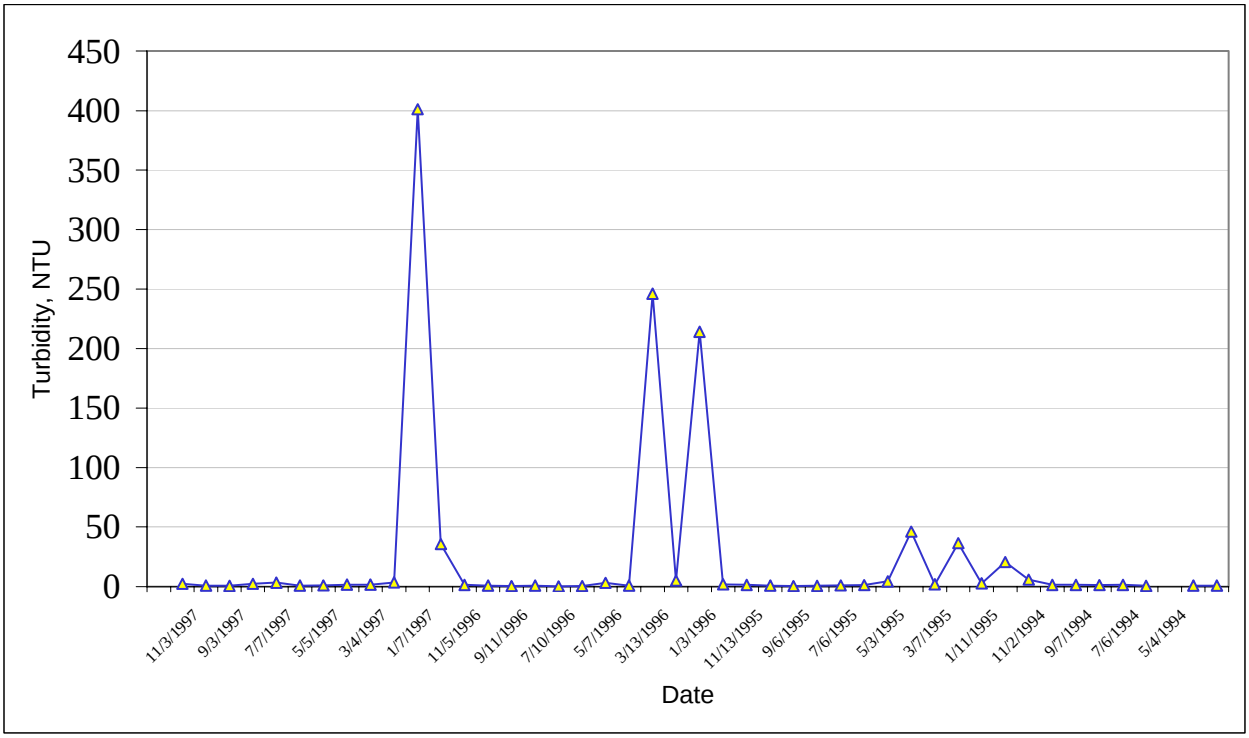

Figure 5-10 Turbidity at American River at ARPS Intake Site

Data are also available for microbial constituents. Coliforms have been monitored at this site. **Table 5-12** summarizes the available data.

Table 5-12 Microbial Water Quality of the American River at ARPS Intake Site

Constituent	Minimum	Maximum	Geometric Mean
Total Coliform, MPN/100 mL	2	>1,600	60
Fecal Coliform, MPN/100 mL	<2	130	9

Key:
ARPS – American River Pump Station
MPN/100 mL – most probable number per 100 milliliters

A graph of coliform over the sampling period is provided in **Figure 5-11**. it can be seen that coliforms peak mostly during the fall through spring, and are likely associated with wet weather events.

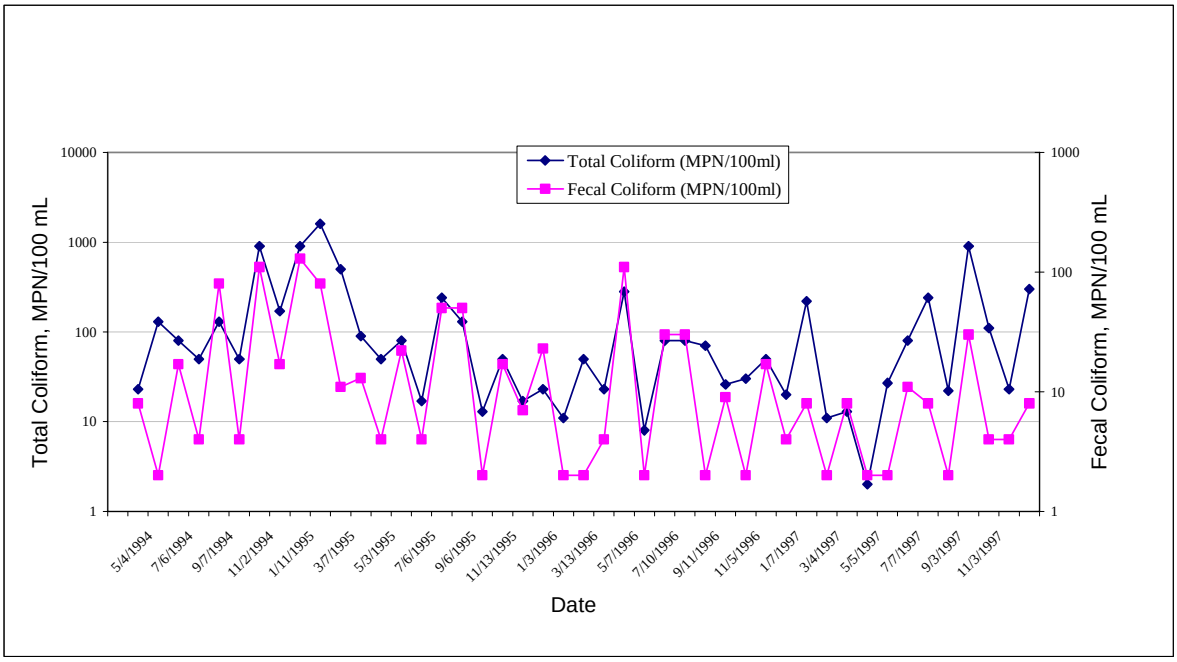

Figure 5-11 Coliform Levels in the American River at ARPS Intake Site

Based on the water quality data collected from the existing ARPS Diversion-site, and presented herein, it appears that the water quality is expected to be excellent.

5.4. NORTH NATOMAS WTP

In this section, the water treatment process for the North Natomas WTP is identified, and design criteria are discussed for filtration facilities, solids handling facilities, chemical feed and supply systems, and electrical. In addition, sewer and stormwater management, site configuration and layout, special considerations, and construction characteristics are discussed.

5.4.1. Process Identification and Design Criteria

Using the water quality data obtained and summarized above, 3/4/2-log reduction of *Giardia*/viruses/*Cryptosporidium* has been identified as the likely level of treatment required under the SWTR and LT2ESWTR. In addition to treating microbial constituents, removing of rice herbicides may be desired seasonally. Therefore, oxidation facilities using potassium permanganate (KMnO4) would be provided. In the event that additional log reduction of *Cryptosporidium* is required in the future, both physical and hydraulic space would be reserved on-site to allow for future installation of UV light facilities.

Conventional filtration with chlorine disinfection is proposed for the North Natomas WTP because it is employed widely, is reliable for treating water with seasonal variability in its quality, and because it's long hydraulic detention time allows plant performance to be less sensitive to abrupt hydraulic or raw water quality changes. Long sedimentation detention times improve the removal of solids and TOC and

assist in controlling taste and odors during treatment. Conventional filtration also involves high capital costs and a large facility footprint due to the need to construct large facilities.

The North Natomas WTP would be designed for a plant capacity flow rate of 145 mgd. **Table 5-13** below summarizes the preliminary design values for each treatment process. A process flow diagram has been developed and is shown in **Figure 5-12**, at end of chapter. The treatment process would include the following facilities:

- Grit basin
- Flash mix
- Flocculation/sedimentation basin
- Dual media gravity filtration
- Future UV light
- Chlorine contact tank (to achieve 0.5 log *Giardia* inactivation)
- Clearwell (for WTP operational storage only at 10 percent of plant capacity)

Table 5-13 Preliminary Design Values for Water Treatment Processes

Description	Units	Preliminary Design Value
PLANT CAPACITY		
Design Flow	mgd	145
	cfs	225
	gpm	100,630
GRIT BASINS		
Type	-	Horizontal Flow
Grit Basin Flow Rate	mgd	145
	gpm	100,630
Number of Basins	no.	2
Width of Basins	ft	40
Length of Basins	ft	190
Water Depth	ft	12
Volume, total	cu ft	182,400
Detention Time	min	13.5
Surface Loading	gpm/sf	6.6
Grit Collection Type	-	Chain & Flight

Table 5-13 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
FLASH MIX SYSTEM		
Type	-	Pump Diffusion
Number of Systems	no.	2
Mixing Energy	sec^{-1}	750 - 1,000
FLOCCULATION BASINS		
Type	-	Vertical Turbine
Number of Basins	no.	4
Basin Width	ft	80
Basin Length	ft	100
Number of Compartments per Basin	no.	16
Compartment Width	ft	19.5
Compartment Length	ft	19.5
Water Depth	ft	16
Volume, each basin	cu ft	128,000
Volume, total	cu ft	512,000
Flocculation Detention Time	min	28.8
Basin Detention Time	min	37.9
Number of Stages	no.	4
Flocculators per Stage	no.	4
Total Number of Flocculators	no.	64
Mixing Energies		
Stage 1	sec^{-1}	60/30
Stage 2	sec^{-1}	40/20
Stage 3	sec^{-1}	30/15
Stage 4	sec^{-1}	20/10
SEDIMENTATION BASINS		
Type	-	Rectangular, Horizontal Flow
Number of Basins	no.	4
Basin Width	ft	80
Basin Length	ft	360
Water Depth	ft	16
Width: Length Ratio	-	0.22
Volume, each	cu ft	460,800
Volume, total	cu ft	1,843,200
Detention Time	min	136.5
Surface Loading	gpm/sq ft	0.87
Sludge Removal Type	-	Chain & Flight

Table 5-13 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
FILTERS		
Type	-	Constant Level,
Filter Flow Rate	mgd	145
	gpm	100,630
Number of Filters	no.	16
Bays per Filter	no.	2
Width of Bay	ft	18
Length of Bay	ft	38
Media Area per Filter	sq ft	1,368
Total Filter Media Area	sq ft	21,888
Filtration Rate		
All Filters in Service	gpm/sq ft	4.6
One Filter Not in Service	gpm/sq ft	4.9
Filter Media		
Anthracite Coal		
Depth	inch	30
Effective Size	mm	1.0
Sand		
Depth	inch	12
Effective Size	mm	0.5
Total Depth	inch	42
Total L/d Ratio	-	1,372
Gravel Depth	inch	None
Filter Backwash System		
Underdrain Type	Concrete Plenum with Nozzles	
Backwash Rate	gpm/sq ft	18
Backwash Duration	mins	10
Filter Auxiliary Wash System		
Type		Air Scour
Wash Rate	gpm/sq ft	-
	scfm/sq ft	3.5
	min	3
CT Tank (Post-Chlorine)		
Type	Baffled, Buried Concrete	
Plant Flow Rate	mgd	145
Number of Basins	no.	1
Volume Basin	mg	2.5
Max Water Depth	ft	16
Area Basin	acres	0.48

Table 5-13 Preliminary Design Values for Water Treatment Processes (cont.)

Description	Units	Preliminary Design Value
CLEARWELL		
Type	Buried Concrete, Rectangular	
Number	no.	1
Volume Clearwell	mg	16.5
Max Water Depth	ft	16.0
Area Clearwell	acres	3.2

Key:
cfs – cubic feet per second
CT – chlorine contact time
cu ft – cubic feet
ft – feet
gpm – gallons per minute
gpm/sf – gallons per minute per square feet
mg – milligram
mgd – million gallons per day
min – minute
mm – millimeter
scfm/sq ft – standard cubic feet per minute per square foot
sec^{-1} – per second

5.4.2. Description of Conventional Filtration Facilities

All of the conventional filtration facilities would be constructed of concrete and painted. The grit basin, flocculation and sedimentation basins, and filters would be open-water areas. Clearwells would be covered and buried. The operations and maintenance building, chemical building, electrical building, and treated water pump station all would be enclosed buildings. These may be constructed of concrete masonry units (CMU) or steel. CMU buildings may be faced with materials such as stucco or split-face block. Steel structures would be painted.

Results of the geotechnical characterization, presented in **Chapter 2**, found that the area where the North Natomas WTP may be constructed likely has low-density granular soils and a high groundwater table. Geotechnical conditions will require more detailed evaluation in the preliminary design phase of the project to determine if the major structures would require pile foundations to prevent settling, loss of foundation support, buoyancy, or lateral spreading of soils. Also, it is expected that large amounts of dewatering would be required during construction, especially related to the buried clearwells.

5.4.2.1. Grit Basin

The purpose of a grit basin is to remove grit, such as silt and sand, to protect mechanical equipment and as prevent the accumulation of grit in the flow split, flash mix, and pretreatment processes. The grit basin is a simple sedimentation tank that removes solids via gravity settling. The grit basin would be located at the influent to the water treatment facility. Multiple basins would allow for draining, cleaning, or repair while maintaining plant operations. The basins would be rectangular, with similar configuration to the horizontal-flow sedimentation basins for improved flow characteristics. The basins would have a length-to-width ratio of nearly 5:1, with a length-to-depth ratio of greater than 15:1 to ensure good settling characteristics. At maximum plant flow the detention time would be approximately 13.5 minutes and the surface loading rate would be less than 7 gallons per minute per square foot (gpm/sf). Grit would be collected using chain and flight. A preliminary plan and section are shown in **Figure 5-13** (end of chapter).

5.4.2.2. Flash Mix

The purpose of flash mixing is to introduce and disperse the primary coagulant chemical in the raw water quickly and evenly. Complete and instantaneous dispersion of chemical coagulants is necessary to achieve optimum coagulation and flocculation, and to maximize the use of the coagulant. Aluminum

sulfate (alum) requires a mixing time of less than 1 second. The amount of energy required to achieve mixing is described by the velocity gradient (G). A G value of 750 to 1,000 sec^{-1} is typically required to achieve proper initial mixing. Energy can be input to the water either mechanically or hydraulically.

A pumped diffusion injection mixing system is a hydraulic method for flash mixing that is recommended for the North Natomas WTP. This mixing system achieves dispersion of the coagulant by diverting a portion of the mainstream flow through a flash mix pump, and then injecting the chemical on the discharge side of the pump in the immediate vicinity of the counter-current injection nozzle. Velocities in the injection nozzle are designed to be in the range of 25 to 30 fps. This velocity provides nearly instantaneous dispersion of the coagulant. This system is advantageous because it requires minimal energy input, provides efficient use of coagulant, causes little headloss, and has low operation and maintenance costs. Also, it is effective over a wide range of plant flows.

5.4.2.3. Flocculation and Sedimentation

The objective of flocculation is to induce contacts between coagulated particles formed in the flash mix process by providing gentle and prolonged agitation, the particles collide, forming larger and more easily settling floc. The sedimentation process removes suspended particles heavier than water by gravity settling. Flocculation and sedimentation basins vary in configuration, mixer type, baffling design, and sludge removal equipment. The width and depth of the flocculation basins should match the sedimentation basins.

Flocculation basins are sized by the required detention time. Typical detention times range from 20 to 40 minutes depending on the source water, coagulant used, and downstream treatment provided. A detention time of at least 25 minutes is recommended for the North Natomas WTP. Flocculators can be configured either horizontally or vertically. Vertical shaft flocculators are recommended for the North Natomas WTP to minimize the impact of a failed drive unit and allow for easier inspection and preventative maintenance of motors and gearboxes. The fundamental design parameter for mechanical flocculators is the velocity gradient, G. Typical values of G range from 15 to 60 sec^{-1}. Normal practice is to taper the flocculation; that is, reduce the G value as the flow proceeds through the flocculation basin.

To transition floc particles smoothly into the sedimentation basin, there would be a diffuser wall between the flocculation and sedimentation basin. This would allow for a smooth hydraulic transition that prevents floc breakup.

Sedimentation basins have several design criteria, including detention time, surface loading rate, and effective water depth. Sedimentation basins can be configured as horizontal-flow basins, circular clarifiers, or solids contact basins. Horizontal-flow basins are recommended for the North Natomas because they are the most flexible for the highly variable source water quality that could be seen at this facility. Sedimentation detention time can range from 90 to 180 minutes; a minimum of 120 minutes is recommended for the North Natomas WTP. Surface loading rates should be 1 gpm/sf or less. Water depth is typically 12 to 18 feet; 16 feet is recommended for the North Natomas WTP. Sludge collection for horizontal-flow basins can be either chain and flight with cross collection or a traveling bridge mechanism; chain and flight is recommended for the North Natomas WTP. A preliminary plan and section is shown in **Figures 5-14** and **5-15** (end of chapter).

5.4.2.4. Filtration

Filtration is a physical and chemical separation process to remove suspended and colloidal materials from water by passing the water through a porous medium. Filters have several design criteria, including

filtration rate, size and number of filters, media selection, backwash system, and underdrain type. Gravity filters with constant level and constant rate are proposed for the North Natomas WTP.

It is recommended that a maximum filtration rate of 5.6 gpm/sf, with one filter out of service, be used for the North Natomas WTP. This would require 18,000 square feet of filter area at 145 mgd. Each filter area should be less than 1,600 square feet; therefore, 16 filters, with an area of 1,368 square feet each, are recommended. Each filter would consist of two bays, with each bay being 18 feet wide and 38 feet long.

Filters can have single-, dual- or tri-media. Given the expected source water quality and proposed pre-treatment processes, dual media filters are recommended for the North Natomas WTP. These would comprise of 30 inches of anthracite coal and 12 inches of sand.

Numerous types of filter underdrains are available. A false filter bottom with nozzles is recommended for the North Natomas WTP to allow for uniform distribution of backwash flow.

The North Natomas WTP would be equipped with an auxiliary backwash system, consisting of air scour wash. The filters would also be equipped with piping to allow for filter-to-waste after backwashing. This would allow lower quality water produced during filter maturation to be sent to the equalization basins prior to recycling to the headworks. A preliminary plan and section are shown in **Figure 5-16** (end of this chapter).

5.4.2.5. Future Ultraviolet Light

Comply with potential future requirements the LT2ESWTR, water treatment design for the North Natomas WTP includes adequate footprint and hydraulic head for future UV light installation. USEPA recommends that chlorination occur upstream from UV treatment, but it can occur downstream as proposed for the North Natomas WTP. The footprint for the 145 mgd facility is approximately 7,200 square feet plus access area, assuming the use of four low pressure reactors with one standby unit. Maximum overall head loss for a UV system is estimated at 8 feet. However, most literature cites 3 feet as a required hydraulic standard. Initial design layout would include a head loss of 6 feet for future UV installation.

Several design parameters must be considered during the preliminary design process for UV installation. Water quality, UV lamp fouling/aging, chemical considerations and application points, flow rate, and power quality are the primary constituents that drive validation of the UV system. These parameters would be investigated during the Refinement of the Engineering Analysis to ensure that the final design was compatible with all UV operational requirements.

5.4.2.6. Chlorine Contact Tank and Clearwell

The North Natomas WTP would have a treated water clearwell set at approximately 10 percent of the WTP capacity (16.5 million gallons (MG)). This volume is intended to provide operational flexibility at the North Natomas WTP, but would not accommodate peaking flows to the distribution system.

Since conventional filtration would be implemented for the North Natomas WTP, disinfection would likely be required to achieve 0.5-log inactivation of *Giardia* and 2-log inactivation of viruses to meet the 3/4-log reduction for *Giardia*/viruses. Inactivation requirements for *Giardia* are significantly higher than for viruses, so meeting the 0.5-log inactivation for *Giardia* would govern.

Inactivation must be completed prior to distribution to the first customer. Sacramento could have future customers located near the North Natomas WTP; therefore, all disinfection requirements must be met

prior to distribution. For this reason, an additional CT tank was added to the treated water train with a volume of 2.5 MG.

The WTP would use free chlorine as the primary and secondary disinfectant. Chlorine contact time (CT) would be calculated to determine the required inactivation. The amount of CT required can be estimated using the following equation:

$CT_{req'd}$ = 0.2828 x $pH^{2.69}$ x Residual Chlorine Concentration $(mg/L)^{0.15}$ x (log reduction required) x $0.933^{(Temperature\ [in\ C]-5)}$

From this equation, it can be seen that increases in pH and residual chlorine concentration cause an increase in the CT required, while an increase in temperature causes a reduction in required CT. **Table 5-14** summarizes a range of potential CT requirements for the North Natomas WTP.

Table 5-14 Potential CT Requirements for the North Natomas WTP

Temperature (C)	Chlorine Residual (mg/L)	Log Inactivation Required	CT Required (mg/L-min) at pH = 6	CT Required (mg/L-min) at pH = 6.5	CT Required (mg/L-min) at pH = 7
7	0.5	0.5	14	17	21
7	0.5	1.0	27	34	42
7	1.0	0.5	15	19	23
7	1.0	1.0	31	38	46
7	1.5	0.5	16	20	25
7	1.5	1.0	32	40	49
22	0.5	0.5	5	6	7
22	0.5	1.0	10	12	15
22	1.0	0.5	5	7	9
22	1.0	1.0	10	14	16
22	1.5	0.5	6	8	9
22	1.5	1.0	12	14	17

Key:
°C – degrees Celsius
CT – chlorine contact time
mg/L – milligrams per liter
mg/L-m – milligrams per liter per minute
WTP – water treatment plant

Since predisinfection would occur at the headworks, some attributable CT would be achieved in the grit basins, flocculation/sedimentation basins, filters, and miscellaneous piping. The majority of CT would be achieved in the CT tank and clearwell. The amount of CT achieved is calculated using the following equation:

CT_{ach} = Residual Chlorine Concentration, mg/L x (T_{10}/T x [Basin Volume, gallons / Plant Flow Rate, gpm])

The T_{10}/T ratio compares the disinfection contact time to the theoretical detention time in a basin. This ratio can be determined by the baffling classification in a basin. The CT tank and clearwell would be designed to provide T_{10}/T ratios of 0.7. The design would include perforated inlet baffles, serpentine, or perforated intra-basin baffles, and either an outlet weir or perforated launders. **Table 5-15** summarizes the CT achieved at the North Natomas WTP under a difficult case scenario; low temperature (7°C), high pH (7 units) water with 1-log inactivation required.

Table 5-15 CT Achievements for 1-log Inactivation Requirements

Residual Chlorine (mg/L)	T_{10}/T	$CT_{req'd}$ (mg/L-min)	Sacramento Flow (mgd)	Sacramento CT Tank/ Clearwell Volume (mg)	Sacramento CT_{ach} (mg/L-min)	Minimum Clearwell Volume to Achieve CT On-Site (%)
0.5	0.7	42	145	19	66	64
1.0	0.7	46	145	19	132	35
1.5	0.7	49	145	19	198	25

Key:
CT – chlorine contact time
CT_{ach} – chlorine contact time achieved
MG – million gallons
mg/L – milligrams per liter
mg/L-min – milligrams per liter per minute
mgd – million gallons per day

This table shows that at maximum plant flow, CT requirements can be met for a difficult case scenario such as low temperature and high pH, and at a variety of residual chlorine concentrations. The size of the clearwell also provides some flexibility for operating level, allowing the clearwell to be at varying levels while still meeting CT on-site. Worst case conditions occur under low chlorine residual concentrations, less than 0.5 mg/L, when CT achieved upstream of the clearwell and in the treated water piping could have a significant impact on the overall CT achieved. A preliminary plan drawing is shown in **Figure 5-17** (end of chapter).

5.4.2.7. Operations and Administration Building

Efficient management of any WTP is highly dependent on the design and layout of the operations and administration building. This building serves as the major human interface between the WTP and its operators. To design and, ultimately, build an effective building, the overall building design must address the functionality and architectural and structural integrity of the structure.

Functional design components can be divided into four spatial categories: (1) administrative zone, (2) operational management zone, (3) product quality control zone (laboratory), and (4) the mechanical/workshop zone. The administrative section is critical for day-to-day operations of the plant and may consist of a reception area, storage room for records and office supplies, toilet facilities, a conference room, and offices for plant managers. The operational management zone serves as an interface between staff and the process operations of the treatment plant. A control room, laboratory, and a lunchroom are essential components of the management area. If sampling for water quality control is done on-site, the laboratory should consist of four discrete areas: general chemistry lab, instrumentation lab, bacteriology lab, and a management office. Each laboratory area would have specific design requirements that would help achieve successful water quality management. The fourth spatial area is the mechanical/workshop zone. This area should house the building's mechanical equipment and provide adequate working space for computer or electronic repair.

The proposed operations and administration building for the North Natomas WTP is a two-story structure, with a footprint of 10,000 square feet. A preliminary plan and elevation are shown in **Figures 5-18** and **5-19** (end of chapter). Pursuant to client request and efficient plant management, the proposed building parking lot is located directly ahead of the entrance to the plant. In addition, plant access driveways direct all public traffic to the building. The entrance design and building locale discourage unauthorized vehicles from entering the site and allow the plant staff to control visitor traffic.

The architectural and structural aspects of the building are highly dependent on owner preference and Federal, State, and local laws. Coordination between the owner and engineering design team would prove to be invaluable for the successful architectural and structural design of the building. It is anticipated that the building would be constructed of CMU with a stucco facing. Colors would be selected to blend with the expected urban development in the North Natomas area.

5.4.2.8. Treated Water Pump Station

The treated water pump station would be located on the clearwell, and would pump directly into the transmission main for Sacramento. The pump station total design flow would be 145 mgd with a design total dynamic head of 161 feet. The pump station would likely consist of six pumps: two at approximately 18 mgd and four at approximately 36 mgd capacities at the design head, with some equipped with variable-frequency drives. One 36-mgd pump would be a standby pump. The total connected horsepower, including the backup pump, would be approximately 6,250 hp. One switchgear and control building would be built for the pump station. The building would be constructed on grade and adjacent to the clearwell and would measure at least 65 feet by 25.

5.4.2.9. Plant Hydraulics

Raw water would be delivered to the grit basin by the raw water intake pump station. Water would then flow by gravity through the flocculation/sedimentation basins and filters to the treated water clearwells.

The facilities within the site would be positioned to maximize process flow efficiency and eliminate the need for booster pumps between the treatment processes. Although the topographic layout of the site is generally level, material excavated for the stormwater detention basins, equalization basins, and sludge settling basins may be used as fill to raise other facilities. This would increase the water surface elevation and improve plant hydraulics, and reduce or eliminate the need to haul excavated soil off site. **Tables 5-16** through **5-18** summarize the estimated hydraulics through the plant and **Figures 5-20** and **5-21** (end of chapter) provide hydraulic profiles of the facilities at three potential WTP sites. **Table 5-16** and **Figure 5-20** represent conditions at a site located at the western end of potential WTP sites and assumed to have an area of approximately 90 acres. **Table 5-18** and **Figure 5-21** represent conditions at a site, located at the eastern end of potential WTP sites, assumed to have an area of approximately 100 acres. **Table 5-17** represents conditions at a site located near the middle of potential WTP sites and assumed to have an area of approximately 100 acres. The sites located at the western end and near the middle of potential WTP sites will have the same finished grade and water surfaces; therefore, **Figure 5-20** represents the hydraulic profile for both sites. It should be noted that the existing grade at western and middle site is not located within the 100-year floodplain. However, the site at the eastern end of potential sites would be located within the 100-year floodplain, as per Flood Insurance Rate Map (FIRM) dated July 1998. Raising the grade may minimize or eliminate flooding. This should be considered during the engineering analysis as part of the WTP site selection.

Table 5-16 North Natomas WTP Site at Western End of Potential Sites-Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	22.5	33.0	45.0	12	2.0
Flash Mix	23.2	29.5	-	-	1.7
Flocculation/Sedimentation	23.7	29.5	41.3	16	1.5
Filters	23.6	28.5	39.8	14	9.2
UV Disinfection (future)	24.2	24.2	-	-	8.0
CT/Clearwell	23.5	23.5	22.6	16	2.6
Equalization Basins	23.3	23.3	20.0	15	-
Sludge Settling Basins	22.5	22.5	20.0	4	-

Key:
CT – chlorine contact time
ft – feet
UV – ultraviolet
WTP – water treatment plant

Table 5-17 North Natomas WTP Site Near Middle of Potential Sites-Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	20.2	33.0	45.0	12	2.0
Flash Mix	20.0	29.5	-	-	1.7
Flocculation/Sedimentation	20.0	29.5	41.3	16	1.5
Filters	20.0	28.5	39.8	14	9.2
UV Disinfection (future)	19.9	24.0	-	-	8.0
CT/Clearwell	19.8	23.5	22.6	16	2.6
Equalization Basins	19.9	23.0	20.0	15	-
Sludge Settling Basins	20.0	22.5	20.0	4	-

Key:
CT – chlorine contact time
ft – feet
UV – ultraviolet
WTP – water treatment plant

Table 5-18 North Natomas WTP Site at Eastern End of Potential Sites-Hydraulic Summary

Facility	Existing Grade (ft)	Finished Grade (ft)	Water Surface Elevation (ft)	Max. Depth (ft)	Head Loss (ft)
Grit Basin	15.0	28.0	40.0	12	2.0
Flash Mix	15.0	24.5	-	-	1.7
Flocculation/Sedimentation	15.0	24.5	36.3	16	1.5
Filters	15.0	23.5	34.8	14	9.2
UV Disinfection (future)	15.0	19.0	-	-	8.0
CT/Clearwell	15.0	18.0	17.6	16	2.6
Equalization Basins	15.0	15.0	15.0	15	-
Sludge Settling Basins	15.0	15.0	15.0	4	-

Key:
CT – chlorine contact time
ft – feet
UV – ultraviolet
WTP – water treatment plant

5.4.3. Description of Solids Handling Facilities

Waste streams generated at the North Natomas WTP include grit from the grit basin, sludge removed from the sedimentation basins, filter backwash water, filter-to-waste water, and sampling water.

Filter backwash water, filter-to-waste water, sampling water, and sludge settling basin decant would be treated with a polymer and then stored in an equalization basin. The basin would be sized to accommodate four filter backwashes and filter-to-waste cycles plus 10 percent. Two basins would be provided to allow for cycling. Decant would be recycled to the headworks while the solids would be sent to the sludge settling basins. **Table 5-19** summarizes of the design of the equalization settling basins.

Table 5-19 Design Criteria for the Equalization Basins

Description	Units	Preliminary Design Value
Backwash Volume Required	cu ft	144,500
Number of Basins	no.	2
Water Depth	ft	15
Basin Width	ft	40
Basin Length	ft	120
Volume, each	cu ft	72,000
Volume, total	cu ft	144,000

Key:
cu ft – cubic feet ft – feet no. – number

Solids from the grit basin and the equalization basin and sludge from the sedimentation basin would be sent to sludge settling basins for drying. The facilities would be located to allow for cycling of drying beds on 4-month cycles. Solids generation was estimated for the 4 month winter period from December through March and the remaining 8 months of the year, hereafter called summer. It has been assumed that 10 pounds of solids can be applied per square foot for evaporative drying during the winter months and 15 pounds per square foot during the summer. Four settling basins would be provided to allow for cycling and settling periods. Two of these settling basins would be dedicated as winter settling basins and 2 would be used during the remainder of the year. The quantity of solids generated by these waste streams was estimated using the equation below based on the coagulant dose, the polymer dose, treatment flow, and total solids in the raw water. **Table 5-20** summarizes the estimated solids generation.

Solids Production Rate (lbs/day) =

[(Alum Dose in mg/L x 0.26) + (Turbidity in NTU x 1.2) + Polymer Dose in mg/L](8.34)(Flow in mgd)

Table 5-20 Estimated Solids Generation for the North Natomas WTP

Average Parameter	Winter Conditions (December – March)	Summer Conditions (April – November)
Average Flow (mgd)	70	125
Raw Water Turbidity (NTU)	47	25
Alum Dose (mg/L)	40	20
Polymer Dose (mg/L)	0.7	0.7
Average Solids Production Rate (lbs/day)	39,400	37,400

Key:
lbs/day – pounds per day
mg/L – milligrams per liter
mgd – million gallons per day
NTU – nephelometric turbidity units
WTP – water treatment plant

To size the sludge settling basins, several criteria were evaluated. First, settling basin loading rates described above were used to calculate the area required for each bed. Then, a depth of sludge was determined. Typically, settling basins should be less than 6 feet in depth. The total volume of solids produced per period was calculated assuming 4 percent solids sludge when drying, which is expected to have a density of approximately 64 pounds per cubic foot. The total area of the settling basins was then used to identify the required sludge depth. **Table 5-21** lists the design criteria for the sludge settling basins.

Table 5-21 Design Criteria for Sludge Settling Basins

Description	Units	Preliminary Design Value
Target Sludge Settling Basin Loading (winter/summer)	lb DS/ft^2	10/15
Number of Settling basins (winter/summer)	no.	2/2
Settling Basin Width	ft	250
Settling Basin Length	ft	1100
Settling Basin Depth (winter/summer)	ft	4/6
Total Settling Basin Volume	ft^3	5,500,000

Key:
ft – feet
ft^3 – cubic feet
lb DS/ft^2 – pounds dry solids per square foot
no. – number

The settling basins would be designed with a downward slope of 0.5 to 1.0 percent toward the outlet, with a vehicle access ramp. A multilevel decant facility would be built that would operate continuously during the drying cycle. The decant water would be continuously returned to the equalization basins. Consideration could be given to the use of polymers or aeration in the sludge settling basins during preliminary design phase of the project.

Dried sludge would be transported to a landfill for ultimate disposal. It is expected that settling basins would be routinely cleaned and dried sludge removed approximately three times per year.

5.4.4. Description of Chemical Feed and Storage Systems

The North Natomas WTP would include chemical feed and storage systems for the chemicals shown in **Table 5-22**. Chemical application points are shown in **Figure 5-22** (end of chapter).

Table 5-22 Summary of Chemicals Selected and Purpose

Chemical	Purpose	Injection Point
Chlorine (Cl_2)	CT Disinfection Credit	Raw Water Line, Downstream from Recycle Line
Potassium Permanganate (KMnO4)	Taste and Odor – Rice Herbicides	Raw Water Line, Downstream from Recycle Line
Powdered Activated Carbon (PAC)	Taste and Odor – Geosmin/MIB	Raw Water Line, Downstream of Recycle Line
Aluminum Sulfate (Alum)	Coagulation	Flash Mix Pump Discharge
Polyaluminum Chloride (PACl)	Coagulation	Flash Mix Pump Discharge
Cationic Polymer	Coagulation Aid	Flash Mix Pump Discharge
Sodium Hydroxide (NaOH – Caustic Soda)	pH Adjustment	Flash Mix Pump Discharge
Anionic Polymer	Flocculation Aid	Flocculation Basin Influent Channel
Chlorine (Cl_2)	CT Disinfection Credit	Filter Influent Channel
Non-Ionic Polymer	Filter Aid	Filter Influent Channel
Chlorine (Cl_2)	CT Disinfection Credit	Filter Effluent Weir
Hydrofluosilicic Acid (H_2SiF)	Fluoridation	Clearwell Influent Box (Sacramento only)
Quicklime (CaO)	pH Adjustment	Treated Water Pump Intake Channel

Key:
CT – chlorine contact time
Geosmin/MIB – typical taste and odor causing agents
SSWD – Sacramento Suburban Water District

5.4.4.1. Chlorine Gas

Chlorine is obtained as a pressurized gas in 1-ton cylinders. The chemical would be fed into the raw water line, filter influent channel, and filter effluent weir. Six chlorinators would be provided, one for each feed location and one as a spare. The chlorine system would include cylinder-mounted vacuum regulators, scales, automatic switchover system, chlorinators, injectors, leak detectors, and associated piping, valves, and controls. Feed and storage equipment would be located in a chemical building. Adequate space would be provided for moving the cylinders with an overhead hoist and trolley system. Ventilation would be provided in both the storage and feed areas. Storage would be provided for 30 days.

5.4.4.2. Potassium Permanganate

Potassium permanganate is obtained in dry, granular form in pails or drums. The chemical would be fed into the raw water line. The feed system would include a volumetric feeder with hopper for loading chemicals. The permanganate would be fed into solution tanks that would use raw water. The solution would then be injected into the raw water line. It is recommended that feed equipment be located in an adjacent building adjacent, with a ventilation system, since the chemicals are very heavy and difficult to handle. Since this system would not be used often, only 7 days of storage would be provided.

5.4.4.3. Aluminum Sulfate

Aluminum sulfate (alum) is obtained as a liquid (49 percent solution) in bulk delivery. This chemical would be fed into the flash mix. Diaphragm metering pumps would deliver the coagulant to the flash mix area. Three metering pumps would be provided for each of the three process trains, two for feed and one for standby. A magnetic flow meter would be used on the discharge delivery piping as near to the point of application as possible for feedback control of the metering pumps. The storage tanks would be located on-site and the feed equipment would be located in a chemical building, in a sealed room with a ventilation system. Thirty days of storage would be provided.

5.4.4.4. Polyaluminum Chloride

Polyaluminum chloride (PACl) is an alternate primary coagulant that would likely be used during the winter months when the raw water turbidity is higher and pH range can vary more widely. Liquid can be obtained as a 50 percent solution. Two metering pumps would be provided for each of the three process trains; one duty pump and one standby pump. A magnetic flow meter would be used on the discharge delivery piping as near to the point of application as possible for feedback control of the metering pumps. The storage tanks would be located on-site and the feed equipment would be located in a chemical building in a sealed room with a ventilation system. Seven days of storage would be provided.

5.4.4.5. Cationic Polymer

Since the dosing for cationic polymer can be variable, it is not desirable to store large volumes. Cationic polymer is obtained in liquid form (100 percent active) in 300-gallon bins. Two metering pumps would be provided for each of the three process trains. The feed equipment would be located in a chemical building, in a sealed room with ventilation system. Fourteen days of storage would be provided.

5.4.4.6. Caustic Soda

Caustic soda is obtained as a liquid (25 percent solution) in bulk delivery. The chemical would be fed into the flocculation influent channel as needed for pH adjustment. Diaphragm metering pumps would deliver the caustic soda to the flocculation influent channel. The feed equipment would be located in a chemical building in a sealed room with a ventilation system. Since this system would not be used often; only 7 days of storage would be provided.

5.4.4.7. Anionic Polymer/Nonionic Polymer

Many types of these polymers are available, which can be provided in dry or liquid form. These polymers are usually added at very low doses, making storage and feed systems relatively small. A package polymer feed system would be planned for feeding either dry or liquid form that includes dry feeder, mixing tank, aging tank, and metering pumps. Space would be provided for 14 days of storage for either barrels or pallets.

5.4.4.8. Hydrofluosilicic Acid

Hydrofluosilicic acid is obtained as a liquid (23 percent solution) in bulk delivery. This chemical would be fed only into the water delivered to Sacramento. For this reason, it would be fed into the Sacramento CT tank influent channel. This chemical is highly corrosive, even when diluted, and therefore needs to be located near its point of application. Storage and feed equipment need to be constructed of specific materials to resist corrosion. Storage needs to be 100 percent contained for maximum acid volume. Space would be provided for 30 days of storage.

5.4.4.9. Lime

Lime is added to treated water as a corrosion control measure to elevate pH and add alkalinity. The pH adjustment alone is not sufficient for a low buffering capacity water, such as in the Sacramento River. Lime is added to obtain a positive Langelier Index (to maintain excess calcium carbonate in the treated water). Lime adds calcium to the water, unlike caustic soda or soda ash, which then can precipitate and be deposited on pipe walls to enhance corrosion control. Lime is available in two forms: quicklime and hydrated lime. Hydrated lime is more expensive than quicklime and more needs to be added to provide the same corrosion control as quicklime. Storage facilities for hydrated lime also need to be larger, but the feed equipment is easier to operate and maintain. Quicklime is recommended due to space and cost efficiency, and requires storage facilities and a slaker to create the lime slurry for feeding. Since lime slurry is difficult to pump, storage and feed facilities should be as close to the point of application as possible. Space would be provided for 30 days of storage of quicklime.

5.4.4.10. Chemical Buildings

Lime and fluoride should be housed together, located adjacent to the Sacramento CT tank and clearwells. **Figure 5-23** (end of chapter) presents a floor plan and elevation of the building.

Chlorine gas and caustic soda should be housed together since caustic soda is used to scrub chlorine leaks. All other chemical storage and feed systems could be stored in this building. This building should be centrally located to reach all chemical application points, but also be located near the operations building to allow for frequent visits by operation and maintenance staff. See **Figure 5-24** (end of chapter) for a plan and elevation of this building.

All feed equipment, and storage facilities should be enclosed in buildings. **Table 5-23** summarizes the chemical feed and storage requirements.

Table 5-23 Summary of Chemical Feed and Storage Requirements

Chemical	Storage Criteria	Storage Weight or Volume	Type of Container	Number of Containers
Chlorine Gas	30 days @ 2.5 mg/L and 145 mgd	9,000 pounds	2,000-pound cylinders	52 cylinders
Potassium Permanganate	7 days @ 1.5 mg/L and 120 mgd	113 cubic feet	Vertical steel hopper	1 – 150 cubic feet
Aluminum Sulfate	30 days @ 15 mg/L and 145 mgd	99,000 gallons	Vertical steel rubber-lined	4 – 28,000 gallons
Polyaluminum Chloride	7 days @ 0.4 mg/L as Al and 100 mgd	4,320 gallons	Vertical steel rubber-lined	1 – 5,000 gallons
Cationic Polymer	14 days @ 0.5 mg/L and 145 mgd	940 gallons	300-gallon bins	4 bins
Caustic Soda	7 days @ 5 mg/L and 60 mgd	6,490 gallons	Horizontal steel	1 – 7,000 gallons
Anionic/ Nonionic Polymer	14 days @ 0.2 mg/L and 145 mgd	3,390 pounds	50-pound bags or 55-gallon drums	68 bags or 7 drums
Hydrofluosilicic Acid	30 days @ 0.8 mg/L as F and 145 mgd	16,000 gallons	Fiberglass	2 – 9,000 gallons
Lime	30 days @ 5 mg/L and 145 mgd	3,600 cubic feet	Vertical steel silos	1 – 4,000 cubic feet

Key:
al – aluminum F – fluoride mgd – million gallons per day mg/L – milligrams per liter

5.4.5. Electrical Feed and Supply Considerations

This section presents the electrical planning issues for the proposed 145 mgd North Natomas WTP that would serve Sacramento. Three proposed WTP sites have been evaluated, all of which are located in Sacramento County north of Sacramento International Airport.

5.4.5.1. Primary Power Requirements, Availability, and Reliability

The maximum power requirement for the 145 mgd North Natomas WTP has been estimated to be 7,400 kVA. **Table 5-24** generally summarizes how the power requirements were estimated.

Table 5-24 Power Requirement Summary

Facility	Peak Flow (mgd)	Pump (hp[1])	Misc. Load (kVA)	Apparent Power (kVA)	Amps @ 4,160 Volts	½ Load (kVA)
City of Sacramento	145	6,400	1,000	7,400	1,027	3,700

Notes:
[1]Includes spare pump.
Key:
hp – horsepower kVA – kilovolt amperes mgd – million gallons per day

SMUD is the governing power utility for the potential WTP sites. Power for this load would be available from existing SMUD lines routed along Elverta Road up to Power Line Road. There are two 69 kV power lines (in parallel) are currently in place and SMUD is in the process of upgrading these lines due to increased commercial and residential development in the North Natomas area. The loads presented here can be considered in the SMUD upgrade. The various potential WTP sites would be supplied with power as follows (see **Figure 5-25**).

- WTP site located at western end of potential sites: This site would be located approximately 1 mile east of the Garden Highway. Power feed would be supplied from existing lines on Elverta Road up to Power Line Road. The power feed would continue underground to the WTP location. It is expected that this line may have to be routed underground due to safety concerns with aboveground facilities located in the approach and departure zone for Sacramento International Airport. These lines would be located within the Elverta Road right-of-way. SMUD indicated that underground 69 kV lines have a budget cost of $175.00 per foot (not including trenching). SMUD requires the owner to incur the cost of poles and trenching for underground lines.
- WTP site located in middle of potential sites: This site would be located approximately 1 mile east of Power Line Rd. This 69kV service is available from upgraded existing overhead lines along Elverta Road. SMUD indicated that overhead 69kV feed from its line to a transformer has a budget cost of $30.00 per foot.
- WTP site located at the eastern end of potential sites: This site would be located just east of Highway 99. This 69kV service is available from upgraded existing overhead lines along Elverta Road. SMUD indicated that overhead 69kV feed from its line to a transformer has a budget cost of $30.00 per foot.

SMUD can provide a design that would incorporate the level of redundancy the owner would require. SMUD can design its connection points and multiple switching configurations for the redundancy that would meet the needs and satisfaction of the owner.

5.4.5.2. Motor Starter Requirements

SMUD requires all large medium-voltage, and all large low-voltage motors to have reduced-voltage solid state starters.

5.4.5.3. Primary Backup Power Supply

The proposed primary means for backup power supply is the installing two primary feeds at the North Natomas WTP site. The reliability of power supply at the North Natomas WTP site would be increased greatly with the installation of these two separate primary feeds into the two transformers at the North Natomas WTP to feed 4.16kV into the power distribution substation. The proposed plan for the power feeds at the North Natomas WTP is to receive one feed from each of the two existing upgraded parallel 69kV lines into the site.

Each secondary transformer would be connected to a main circuit breaker. The two mains would be connected by a tiebreaker. Upon loss of power detected in one of the two main breakers, that main would open and after a specified time delay (selected by the owner), the tiebreaker would close, resuming power to the side of the bus that lost power.

5.4.5.4. Alternative Backup Power Supply Option

SMUD does not allow another utility to serve within the SMUD service area. An alternative backup power supply option would be the use of diesel generators at the North Natomas WTP site. The SRWRS partners selected a 50 percent backup generation capacity for evaluation. The required 50 percent backup generation for the 145 mgd North Natomas WTP site would require 3,700 kVA of paralleled generators. The parallel generators would require a switchgear for control, a day tank (300 gallons) for each generator, and a fuel storage tank. The generators use 150 gallons of fuel per hour at full load and would require a total of 2,400 gallons of fuel for an 8 hour time period (full load). The paralleling switchgear would control the output power for each generator; therefore, if the load was less than 3,700 kVA, fuel consumption would be less.

The space required for the paralleling switchgear, low voltage controls, fuel tank, and generators would be approximately 5,000 square feet in a building with integral automatic air flow louvers and fire alarm system design.

A more detailed evaluation of backup power requirements and specific loads that would be deemed critical if both main breakers into the plant were lost is strongly recommended during the preliminary design phase of the project to optimize sizing of these generators and associated facilities.

5.4.5.5. Electrical Building

Power would enter the site and go directly to transformers to reduce voltage from 69kV to 4.16kV. The secondary of the transformers would then go to two main breakers at the North Natomas WTP power distribution substation. The transformer area is expected to be 130 feet by 130 feet, per SMUD requirements. The plant substation, medium-voltage switchgear building is expected to be 50 feet by 50 feet. The building would house the two mains, tie breaker, PTs and CPTs and utility metering section. Each of the two buses would have a capacity of eight 4.16 kV 1,200-amp breakers. The distribution substation would have breakers to feed all of the 4.16 kV loads on the plant. Minimum building requirements would include exhaust fans and heaters.

It is anticipated that the 480-volt loads would be distributed from one or two large power centers centrally located to serve the plant's 480-volt motor control center loads.

5.4.6. Sewer and Stormwater Management

Sewer from the operations and administration building would be conveyed to the County Sanitation District-1 (CSD-1) collection system. CSD-1 plans to extend its system into the Sacramento Metropolitan Airpark. A 12-inch trunk, the NN Metro Air Trunk, is planned to be installed up to Elverta Road, approximately 3,600 feet east of Power Line Road. CSD-1 was contacted and confirmed that the NN Metro Air Trunk would be able to accept the discharge of wastewater from the North Natomas WTP. It is expected that a sewer line, not to exceed 6 inches, would be installed from the WTP site to the connection with the NN Metro Air Trunk.

Currently, no storm drainage services are located in the northwest corner of Sacramento County near the project area. It has been assumed that all stormwater would need to be captured and managed on-site. The site would be constructed and graded to collect stormwater runoff and channel it to on-site detention basins. These basins would be sized to meet the capacity of a 10-year storm over 5 days. The Sacramento City/County Drainage Manual indicates that the water depth of such a storm would be 5.76 inches. Overall site areas are approximately 100 acres, but open-water facilities, such as sedimentation basins and sludge settling basins, would not contribute to stormwater runoff. These areas account for approximately 30 acres. The remaining 70 acres could contribute to stormwater run off at varying rates. It was assumed that overall, 70 percent of rainfall would run off the site. It has been estimated that just over 1 million cubic feet of water would need to be planned for in the detention basin design. It was assumed that the detention basins would be 3 feet deep to allow for evaporative drying. Therefore, approximately 7 acres of detention basins would be required on-site.

5.4.7. Site Configuration and Layout

Preliminary site configurations and layouts have been prepared for three representative sites in the potential WTP area (refer to **Figure 5-1**). Each of the sites includes full 8-foot fencing with victory arms. A gate would be placed at the main entry, which would be set back from Elverta Road to allow trucks to exit the roadway while waiting for entry to the site. Landscaping at all three sites would include native or Xeriscape™ type plants to the extent possible. Landscaping would be laid out to improve the view from neighboring facilities.

5.4.7.1. Representative Site Located at Western End of Potential Sites

The site area is irregularly shaped with a total land area of approximately 90 acres, including roadways, as shown in **Figure 5-26** (end of chapter). The site's facilities are arranged in the design to maximize process flow efficiency and to address operational, security, and FAA safety concerns.

The grit basin would be located at the southwest corner of the site since this location would provide the closest connection to the incoming raw water pipeline. The flow split and flash mix area and flocculation/sedimentation basins would be located northeast of the grit basin. Raw water would travel north through the flocculation/sedimentation basins to the northern half of the facility. The water would turn east to the filter building to provide shortened piping connections between the two facilities and keep facilities centrally located near the operations and administration building. Just southeast of the filter building space has been allocated for a future UV facility. The water would then continue south to the clearwells located along the front side of the site with the treated water pump stations. The pump stations would be located in north corners of the clearwells to be near the electrical power source. The treated water piping would run south between the clearwells and continue east into Elverta Road for distribution.

The operations and administration building would be located near the middle of the plant, on a direct path from the main entrance road for improved security. Vehicles entering the property would be directed to the operations and administration building, thereby decreasing the potential for unauthorized entry. By being able to view the front entrance, plant staff would also be able to monitor incoming and outgoing traffic to and from the plant, respectively.

The main chemical building would be located to the west of the main entrance road, increasing the safety of chemical deliveries by confining the chance of a spill from chlorine or polymer delivery trucks to this area. The chemical building would also be just south of the flocculation/sedimentation basins to reduce polymer piping length, and to be in close proximity to the clearwells to reduce chlorine piping length. The lime/fluoride building would be located east of the filter building to be in close proximity to the points of application to the treated water.

The SMUD substation and electrical building would be located to the east of the lime/fluoride building, north of the clearwells. This arrangement keeps the electrical building close to the highest power requirement, the treated water pump stations at the clearwells.

The equalization basins would be located northeast of the filter building to receive the backwash water. The remaining land at the site would be used for drying settling basins and stormwater detention basins.

5.4.7.2. Representative Sites Located at Middle and at Eastern End of Potential Sites

These site areas are rectangular shaped with a total land area of approximately 100 acres, including roadways, as shown in **Figures 5-27** and **5-28** (end of chapter). Since these sites have the same configuration, the layouts are very similar and for purposes of discussion, are referred to together. The sites' facilities are arranged to maximize process flow efficiency and to address operational, security, and FAA safety concerns.

The grit basin would be located at the southwest corner of the sites since this location would provide the closest connection to the incoming raw water pipeline. The flow split and flash mix area and flocculation/sedimentation basins would be located northeast of the grit basin. Raw water would travel north through the flocculation/sedimentation basins to the northern half of the facility. The water would turn east to the filter building to provide shortened piping connections between the two facilities and keep facilities centrally located near the operations and administration building. Just southeast of the filter building space has been allocated for a future UV facility. Water would then continue south to the clearwells located along the front side of the site with the treated water pump stations. The pump stations would be located in north corners of the clearwells to be adjacent to the electrical power source. The treated water piping would run south between the clearwells and continue east into Elverta Road for distribution.

The operations and administration building would be located near the middle of the plant, on a direct path from the main entrance road for improved security. Vehicles entering the property would be directed to the operations and administration building, thereby decreasing the potential for unauthorized entry. By being able to view the front entrance, plant staff would also be able to monitor incoming and outgoing traffic to and from the plant, respectively.

The main chemical building would be located to the west of the main entrance road, increasing the safety of chemical deliveries by confining the chance of a spill from chlorine or polymer delivery trucks to this area. The chemical building would also be just south of the flocculation/sedimentation basins to reduce polymer piping length, and to be in close proximity to the clearwell to reduce chlorine piping length as

well. The lime/fluoride building would be located east of the filter building to be in close proximity to the points of application to the treated water.

The SMUD substation and electrical building would be located to the east of the lime/fluoride building, northeast of the clearwells. This arrangement keeps the electrical building close to the highest power requirement, the treated water pump stations at the clearwells.

The equalization basins would be located north of the filter building to receive the backwash water. The remaining land on the sites would be used for solids drying basins and stormwater detention basins.

5.4.8. Special Considerations

The representative sites located at the western end and at the middle of potential sites are located within the overflight zone of Sacramento International Airport. For this reason, the design of these facilities must be developed to account for safety issues identified by the Sacramento County Airport Service and the FAA. Preliminary discussions with these agencies indicate that the primary area of concern is the potential for open-water areas to serve as a bird attractant, which would be undesirable for the airport.

Currently, most of the land use near the three potential sites is agricultural or rural with its primary use as rice cropping. This land use leads to an increased presence of birds in the vicinity of potential WTP sites.

For the two reasons above, it has been recognized that the water treatment detailed design would need to incorporate methods for bird detraction. Preliminary information indicates that numerous options for bird detraction exist, including the following:

- Selected design details in the buildings and facilities
- Installation of proprietary detraction devices
- Installation of false predatory birds
- Installation of a predatory bird call sound system
- Covering of open water basins

More recent discussions with the Sacramento County Airport Service and the FAA have indicated greater reluctance to accommodate facilities that may be perceived as bird attractants. Close coordination with these two agencies will be required in the time leading up to the preliminary design phase of the project.

5.4.9. Operating Characteristics

The North Natomas WTP would operate continuously, 24 hours per day, 7 days per week, at various flow rates throughout the year. At a water treatment facility of this size, operations and maintenance would be ongoing. Several types of staff would be expected on-site at varying levels throughout the day, including WTP operators (16), laboratory technicians (6), electrician (1), mechanic (1), machinist (1), instrument technician (1), administrator (1), and other miscellaneous support staff (3). Most staff would be on-site during the daytime hours, from approximately 7:00 a.m. to 5:00 p.m. It is expected that WTP operators (approximately four per shift) would be on-site during all hours of the day. DHS would require the North Natomas WTP to have a Treatment Grade 5 operator to supervise the operation and maintenance and Treatment Grades 2, 3, and 4 for various plant operation shifts.

Daily traffic would comprise mainly operations and maintenance staff. Specialty requirements for scheduled and emergency maintenance also would occur that may include heavier load trucks and chemical deliveries.

Numerous chemicals, as discussed previously, would be stored and used on-site for water treatment operations. Primary chemicals used, including chlorine gas, aluminum sulfate, hydrofluosilicic acid, and lime, would have 30 days of storage at maximum plant daily flow. These chemicals would be delivered by large bulk transport trucks a maximum of once per month. Polymers used daily for treatment would have 14 days of storage at maximum plant daily flow. Because use of this chemical is significantly lower than the other major chemicals, delivery of polymers would occur using a smaller transport truck, a maximum of twice monthly. Other chemicals are only used seasonally, including potassium permanganate, caustic soda, and polyaluminum chloride. Therefore, delivery of these chemicals would occur only during their specific period of use, which is expected to be short.

In addition to water treatment chemicals, minor amounts of other chemicals would be used for equipment operation and operation of facilities (i.e., lubricants, oils, cleaning solvents, laboratory solutions). These would likely be stored in the operations and administration building. Diesel storage for the backup generators, if used, also would be located on the site. It has been estimated that storage would be 2,400 gallons. All chemical and fuel storage would be contained and safety procedures and best management practices would be implemented at this facility similar to other water treatment facilities of the SRWRS partners.

The treated water pump station and backup generators are proposed to be constructed inside buildings, which would reduce their noise emissions. Minor noise would occur associated with low power equipment such as sludge collectors, flocculators, and pumps, in addition to water flow noises.

5.4.10. Construction Characteristics

Construction activities would involve grading the site and erecting the new facilities described in this chapter. Because of the flat topography of the site, grading would likely occur over a large portion of the project site. This would include excavation for the clearwells and chlorine contact tank, equalization basins, sludge settling basins, and stormwater detention basins. The grit basin, flocculation/sedimentation basin, and filters would need to be raised to allow for gravity flow through the facility. It is intended that the excavated materials would be used, if acceptable from an engineering perspective, as fill on-site.

Standard construction methods are proposed, but pile drivers may be used to construct footings for new water-holding structures (i.e., grit basin, flocculation/sedimentation basin, filters, etc.) if the geotechnical investigation determines they would be required. Groundwater levels are expected to be high in this area and therefore large amounts of dewatering may be required during construction. The water removed would be settled and then discharged to a drainage way. A discharge permit would be obtained for these construction activities.

Construction-related traffic (e.g., materials delivery trips, workers, etc.) would access the site from Elverta Road. Materials trips would depend on geotechnical findings regarding the usability of the soil for foundations and the scheduling of construction activities. A traffic control plan would be prepared by the contractor and reviewed by Sacramento County to ensure traffic is safely routed past the work site. No off-site facilities are proposed for this project.

Safety on the construction-site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses

prepared before the beginning of each new operation. OSHA and Cal-OSHA standards would apply for all work.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at the work site exit

5.5. OPHIR WTP

In this section, the water treatment process is identified, and design criteria are discussed for filtration facilities, solids handling facilities, chemical feed and supply systems, and electrical feed and supply. In addition, sewer and stormwater management, site configuration and layout, special considerations, and operating and construction characteristics are discussed.

5.5.1. Process Identification and Design Criteria

Using the water quality data obtained and summarized above, 3/4/2-log reduction of *Giardia*/viruses/*Cryptosporidium* has been identified as the likely level of treatment required under the SWTR and LT2ESWTR. If additional log reduction of *Cryptosporidium* is required in the future, both physical and hydraulic space would be reserved on-site to allow for future installation of UV light facilities.

Low pressure membrane submerged filtration with Actiflo® or plate settler pretreatment and chlorine disinfection has been selected by PCWA for this facility and final design is ongoing by Black and Veatch. The Ophir WTP would be designed for an ultimate plant capacity flow rate of 120 mgd. PCWA plans to construct the first phase of this facility, up to 30 mgd, independent of the SRWRS. Many aspects of the initial phase would include options to upgrade or easily expand those facilities. The SRWRS project would involve expanding and upgrading the future facility from 30 to 95 mgd.

This document reflects the "Foothill Phase II Water Treatment Plant Initial Planning and Siting Study" developed by Black and Veatch in July 2004, the "Foothill Phase II Water Treatment Plant and Pipeline – Draft Environmental Impact Report" developed by ESA in April 2005, and additional information and drawings provided by PCWA. Two key elements of the Ophir WTP are the flash mix and Actiflo® or plate settler pretreatment, and the submerged membrane filters. It is assumed that pretreatment would only be required when raw water turbidity exceeds 100 NTU. **Table 5-25** summarizes the preliminary design values for the treatment processes required as part of the expansion.

Table 5-25 Preliminary Design Values for Water Treatment Processes

Description	Units	Expansion Preliminary Design Value (30 – 95 mgd)
PLANT CAPACITY		
Design Flow	mgd	65
	cfs	100
	gpm	45,139
NEW FLASH MIX SYSTEM		
Type	-	Jet Mixer
Number of Systems	no.	1
NEW FLOCCULATION BASIN		
Flow	mgd	20
Number of Trains	no.	4
Detention Time at 20 mgd	min	15
Mixer Type	-	Vertical Hydrofoil
Stages	no.	2
NEW SEDIMENTATION BASINS		
Number of Basins	no.	4
Basin Loading Rate	gpm/sf	5
High Rate Settling Equipment Type	-	55-degree Lamella Plates
Solids Collection Type	-	Circular
NEW MEMBRANE FILTRATION		
Type	-	Low Pressure Submerged
Flow Rate	mgd	65
	gpm	45,139
Approximate Number of Membrane Cells	no.	14
Redundancy	-	N + 1
Design Flux	gfd	40 at 20 degrees C
Backwash Interval	min	30
Chemical Wash Interval	hours	24
CIP Interval	days	45
Recovery	percent	95

Key:
cfs – cubic feet per second
cfs/sq ft – cubic feet per minute per square foot
ft – feet
gpm – gallons per minute
gpm/sq ft – gallons per minute per square foot
mgd – million galls per day
min – minute
mm – millimeter
no. – number
sq ft – square feet

5.5.2. Description of WTP Facilities

The preliminary WTP design developed by Black and Veatch provides a site layout for the ultimate capacity of the Ophir WTP, shown in **Figure 5-29** (end of chapter). Facilities indicated as future would be required for the proposed project expansion. A process flow diagram has also been developed that depicts the flow of treated water, waste streams, and chemical application points (see **Figure 5-30** at the end of this chapter).

5.5.2.1. Flash Mix

The purpose of flash mixing is to introduce and disperse the primary coagulant chemical in raw water quickly and evenly. Complete and instantaneous dispersion of chemical coagulants is necessary to achieve optimum coagulation and flocculation, and to maximize use of the coagulant. Aluminum sulfate (alum) requires a mixing time of less than one second. The amount of energy required to achieve mixing is described by the velocity gradient (G). A G value of 750 to 1,000 sec-1 is typically required to achieve proper initial mixing. Energy can be input to the water either mechanically or hydraulically. Horizontal end suction centrifugal pump dispersion has been selected for the Ophir WTP.

5.5.2.2. Flocculation and Sedimentation

As noted previously in this chapter, the objective of flocculation is to induce contacts between coagulated particles formed in the flash mix process by providing gentle and prolonged agitation, the particles collide forming larger and more easily settling floc. The sedimentation process removes suspended particles heavier than water by gravity settling. Flocculation and sedimentation basins vary in configuration, mixer type, baffling design, and sludge removal equipment. The width and depth of the flocculation basins should match the sedimentation basins.

Flocculation basins are sized by the required detention time. The detention time has been determined by Black and Veatch to be approximately 15 minutes. Vertical shaft flocculators have been recommended for the Ophir WTP.

Sedimentation basins typically have several design criteria, including detention time, surface loading rate, and effective water depth. Sedimentation basins can be configured as horizontal-flow basins, circular clarifiers, or solids contact basins. Lamellar plates have been recommended for the Ophir WTP to increase solids settling rate and reduce sedimentation basin footprint. The surface loading rate has been estimated at 5 gpm/sf.

5.5.2.3. Membrane Filtration

Membrane Filtration is a physical separation process to remove suspended and colloidal materials from water by passing the water through a porous medium. Low pressure submerged membranes have been selected for the Ophir WTP. Membrane filters have several design criteria, including flux, number of membrane cells, backwash interval, chemical wash interval, CIP interval, and percent recovery.

5.5.2.4. High Service Pump Station

The high service pump station is expected to be designed during the initial phase, up to 30 mgd, to serve initial demands and allow for expansion to meet future demands.

It is not clear, but likely, that the initial phase of construction would include a pump station wet well with space available for the new pumps needed for this expansion project. The initial phase is expected to

include stub-outs from the pump discharge header to permit its extension to connect to the new pumps. Expansion of the pump station would include adding five new pumps (one of which would be standby), each with a capacity of approximately 16 mgd with a discharge head of 85 to 100 feet to lift water to the high point on the transmission pipeline.

5.5.2.5. Plant Hydraulics

Raw water would be delivered from ATPS No. 3 to the pretreatment facilities. Water would then flow by gravity through pretreatment and membrane filtration to the treated water reservoirs. The facilities within the site are positioned to maximize process flow efficiency and eliminate the need for booster pumps between the treatment processes. The topographic layout of the site is naturally sloped and facilities have been sited to use the natural elevation as an advantage. **Table 5-26** summarizes the water levels through the plant, and **Figure 5-31** (end of chapter) provides a hydraulic profile of the facilities.

Table 5-26 Ophir WTP Water Surface Elevations

Facility	Water Surface Elevation (ft)	Head Loss (ft)
Pretreatment	960	10
Filters	950	32
Treated Water Storage Tanks	918	-

Key:
ft – feet WTP – water treatment plant

5.5.3. Description of Solids Handling Facilities

Waste streams generated at the Ophir WTP would include sludge removed from the pretreatment basins, membrane filter backwash water, filter-to-waste water, sampling drainage, and decant from the sludge thickener and drying beds. The quantity of solids generated by these waste streams has been estimated to average over 3,790 pounds per day (lb/d) for the proposed expansion.

The sludge from the pretreatment basins would be sent to a sludge equalization tank and then sent on to a sludge thickener. The solids then would be sent to drying beds, while decant would be discharged to a continuously mixed backwash equalization tank. The membrane filter backwash water would be sent to the backwash equalization basin also, and these streams would be treated to remove solids prior to discharging decant to a plant recycle pump station and the solids to the sludge thickener. The water in the plant recycle pump station would be recycled to the head of the plant. Sampling drainage would be sent directly to the plant recycle pump station and then recycled to the head of the plant. Decant waters from the sludge thickener and drying beds would be sent to the backwash equalization tank for treatment.

Expansion of the Ophir WTP to 95 mgd would need to include some additional solids handling facilities, including a new drying bed, as shown in **Figure 5-29**.

5.5.4. Description of Chemical Feed and Storage Systems

The Ophir WTP would include expansion of the existing chemical feed systems for the chemicals shown in **Table 5-27** and **Figure 5-29**. A summary of the feed and storage requirements is provided in **Table 5-28**.

Table 5-27 Chemical Feed and Storage Systems for Ophir WTP

Chemical	Purpose	Injection Point
Powdered Activated Carbon (PAC)	Taste and Odor – Geosmin/MIB	Raw Water Line, Upstream from Recycle Line
Sodium Hydroxide NaOH – Caustic Soda)	pH and Alkalinity Adjustment	Raw Water Line Downstream from Recycle Line, High Service Pump Station
Aluminum Sulfate (Alum)	Coagulation	Flash Mix Pump Suction
Polyaluminum Chloride (PACl)	Coagulation	Flash Mix Pump Suction
Sodium Hypochlorite (NaOCl)	CT Disinfection Credit	Flash Mix Pump Discharge and Filter Influent and Effluent Weir
Non-Ionic Polymer	Coagulant and Filter Aid	Actiflo® and Filter Influent Channel

Key:
CT – chlorine contact time Geosmin/MB – typical taste and odor causing agents WTP – water treatment plant

Table 5-28 Expanded Chemical Feed and Storage Requirements for Ophir WTP

Chemical	Storage Criteria	Additional Storage Weight or Volume	Type of Container	Additional Number of Containers
Powdered Activated Carbon	30 days @ 5 mg/L and 95 mgd	80,000 pounds	4,000 pound supersack	20 super sacks
Sodium Hydroxide (Caustic Soda)	21 days @ 10 mg/L and 95 mgd	60,000 gallons	Vertical steel, rubber-lined	4 – 15,000 gallon
Aluminum Sulfate	14 days @ 15 mg/L and 95 mgd	40,000 gallons	Vertical steel, rubber-lined	4 – 10,000 gallon
Polyaluminum Chloride	10 days @ 10 mg/L and 95 mgd	10,000 gallons	Vertical steel, rubber-lined	2– 5,000 gallon
Sodium Hypochlorite	21 days @ 4 mg/L and 95 mgd	34,000 gallons	Horizontal steel	4 – 8,500 gallon
Nonionic Polymer – Dry	10 days @ 0.1 mg/L and 95 mgd	50 pounds	50 pound bags	1 bag
Nonionic Polymer – Liquid	10 days @ 0.1 mg/L and 95 mgd	160 gallons	55 gallon drums	3 drums

Key:
mgd – million gallons per day mg/L – milligram per liter WTP – water treatment plant

5.5.4.1. Powdered Activated Carbon

Powdered activated carbon (PAC) would be used for seasonal taste and odor control. PAC would be obtained in 4,000-pound supersacks. This system would include a bag unloader, hopper, volumetric feeder, slurry mixing tank, and metering pumps. Feed equipment would be located in a chemical building. Thirty days of storage would be provided.

5.5.4.2. Sodium Hydroxide

Sodium hydroxide, or caustic soda, would be obtained as a liquid (25 percent solution) in bulk delivery. Metering pumps would deliver the caustic soda to the raw waterline and the high service pump station influent channel. Feed equipment would be located in a chemical building, in a sealed room with a ventilation system. Twenty-one days of storage would be provided.

5.5.4.3. Aluminum Sulfate

Aluminum sulfate (alum) is obtained as a liquid (48.5 percent solution) in bulk delivery. The chemical would be fed into the flash mix. Metering pumps would deliver the coagulant to the flash mix area. Three metering pumps would be provided. The storage tanks would be located on-site and the feed equipment would be located in a chemical building, in a sealed room with a ventilation system. Fourteen days of storage would be provided.

5.5.4.4. Polyaluminum Chloride

Polyaluminum Chloride (PACl) is an alternate primary coagulant that would likely be used during the winter months when raw water turbidity is higher and pH range can vary more widely. Liquid can be obtained as a 50 percent solution. Two metering pumps would be provided for each of the three process trains: one duty and one standby pump. Storage tanks would be located on-site and feed equipment would be located in a chemical building, in a sealed room with a ventilation system. Ten days of storage would be provided.

5.5.4.5. Sodium Hypochlorite

Sodium hypochlorite would be used as the primary disinfectant. It would be obtained as a liquid, delivered at a 12.5 percent solution. Eight metering pumps would be provided as part of the expansion: four duty and four standby. Four storage tanks located on-site and the feed equipment would be located in a chemical building. Twenty-one days of storage would be provided.

5.5.4.6. Nonionic Polymer

Many types of these polymers are available, and can be provided in dry or liquid form. These polymers are usually added at very low doses, making storage and feed systems relatively small. A package polymer feed system has been planned for feeding both dry and liquid form that includes dry feeder, mixing tank, aging tank, and metering pumps. Space would be provided for 10 days of storage for drums and pallets.

5.5.5. Electrical Feed and Supply Considerations

This section presents electrical planning issues for the proposed 65 mgd Ophir WTP expansion for PCWA. The proposed WTP site is located on Ophir Road near Interstate 80.

5.5.5.1. Primary Power Requirements, Availability, and Reliability

The maximum power requirement for the Ophir WTP expansion of 65 mgd has been estimated to be 1.6 kVA. **Table 5-29** summarizes how power requirements were estimated.

Table 5-29 Power Requirement Summary

Facility	Expanded Peak Flow (mgd)	Spare Pump (percent)	Pump (hp)	Misc. Load (kVA)	Apparent Power (kVA)	Amps @ 480 Volts
Expanded Ophir WTP	65	20	1,114	500	1,614	1,942

Key:
mgd – million gallons per day hp – horsepower kVA – kilovolt-ampere WTP – water treatment plant

PG&E is the governing power utility for the future site. Construction of the initial phase of the WTP would bring electrical feed onto the site. The preliminary design does not include future load requirements from the Ophir WTP expansion to 95 mgd. When PCWA expands the Ophir WTP, it is anticipated that the PG&E transformer at the WTP would need to be replaced with a larger unit or a separate service drop to accommodate the additional load for the plant. Also, it is a strong possibility that the 12 kV service line may have to be upgraded as part of this expansion.

5.5.5.2. Primary Backup Power Supply

The Ophir WTP has been designed by Black and Veatch with a preliminary load of 2,700 kVA and a 1,250 kilowatt (kW) backup generator. It is assumed that additional backup power generation would not be required for the Ophir WTP expansion.

5.5.5.3. Electrical Building

The Ophir WTP initial phase, up to 30 mgd, would include an electrical building that would be designed to allow for expansion up to 120 mgd. Modifications to this building would be required as part of this expansion.

5.5.6. Sewer and Stormwater Management

The initial phase of the Ophir WTP would include connection to the sanitary sewer for the City of Auburn. Continued use of this connection is expected for expansion of the WTP.

The site configuration and layout prepared by Black and Veatch indicates that site stormwater would be managed on-site using infiltration trenches, grassy swales, and a stormwater detention basin. Stormwater management would be designed to meet a 10-year, 48-hour storm event, which is estimated at 5 inches.

5.5.7. Operating Characteristics

The Ophir WTP would operate continuously, 24 hours per day, 7 days per week, at various flow rates through the year. At a water treatment facility of this size, operations and maintenance would be ongoing. Staffing recommendations at the site include WTP operators (7), an electrician (1), and mechanics (2). Most staff would be on-site during the daytime hours, from approximately 7:00 a.m. to 5:00 p.m. DHS would require the Ophir WTP to have a Treatment Grade 5 (T5) operator to supervise operation and maintenance and Treatment Grades 2, 3, and 4 staff for various plant operation shifts.

Daily traffic would comprise mainly operations and maintenance staff. Also, specialty requirements for scheduled and emergency maintenance may include heavier load trucks and chemical deliveries.

Numerous chemicals, as discussed previously, would be stored and used on-site for water treatment operations. The chemicals to be used would be provided with 10 to 21 days of storage at maximum plant

daily flow. These chemicals would be delivered using large bulk transport trucks a maximum of twice monthly.

In addition to water treatment chemicals, minor amounts of other chemicals would be used for equipment operation and operation of facilities (e.g., lubricants, oils, cleaning solvents). These chemicals likely would be stored in the operations and administration building. All chemical storage would be contained and safety procedures and best management practices would be implemented at this facility similar to other water treatment facilities of the SRWRS cost-sharing partners.

The treated water pump station is proposed to be constructed inside a building, which would reduce noise emissions. Minor noise would be associated with low power equipment such as the pretreatment basin and pumps, in addition to water flow noise.

5.5.8. Construction Characteristics

Construction activities would involve grading the site and erecting the new facilities described in this chapter. Because of the steep topography of the site, grading would be extensive on the portions of the project site where construction would occur. This would include excavation for the new pretreatment basins and membrane filters. It is unclear if excavated materials would be used on-site or would require disposal. Standard construction methods are proposed, but if hardrock is encountered, blasting may be required.

Construction-related traffic (e.g., materials delivery trips, workers) would access the site from Ophir Road. Materials trips would depend on the geotechnical findings regarding soil types and the schedule of construction activities. A traffic control plan would be prepared by the contractor and reviewed by Sacramento County to ensure traffic is safely routed past the work site. No off-site facilities are proposed for this project.

Safety on the construction-site would be the responsibility of the contractor. The contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. OSHA and Cal-OSHA standards would apply for all work.

The construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at the work site exit

5.6. ROSEVILLE GROUNDWATER WELLS

Roseville would obtain its water supply by installing four new groundwater wells and associated facilities. It is expected that each well would provide a capacity of 1,750 gpm, (to a total of 10 mgd). Wells would be located throughout Roseville's distribution system and are each expected to pump to 350 feet of total dynamic head. This would require pump stations of approximately 225 hp. The wells would also need to have chemical feed facilities for disinfection and fluoridation. It is proposed to provide on-site sodium hypochlorite generators and sodium fluoride saturators, similar to Roseville's Diamond Creek Well installed in 2003/2004. If all facilities are enclosed, including pumps, chemical feed and storage, and electrical facilities, the building is expected to be approximately 1,000 to 1,200 square feet. The building would likely be a CMU or prefabricated steel building with limited architectural and landscaping elements.

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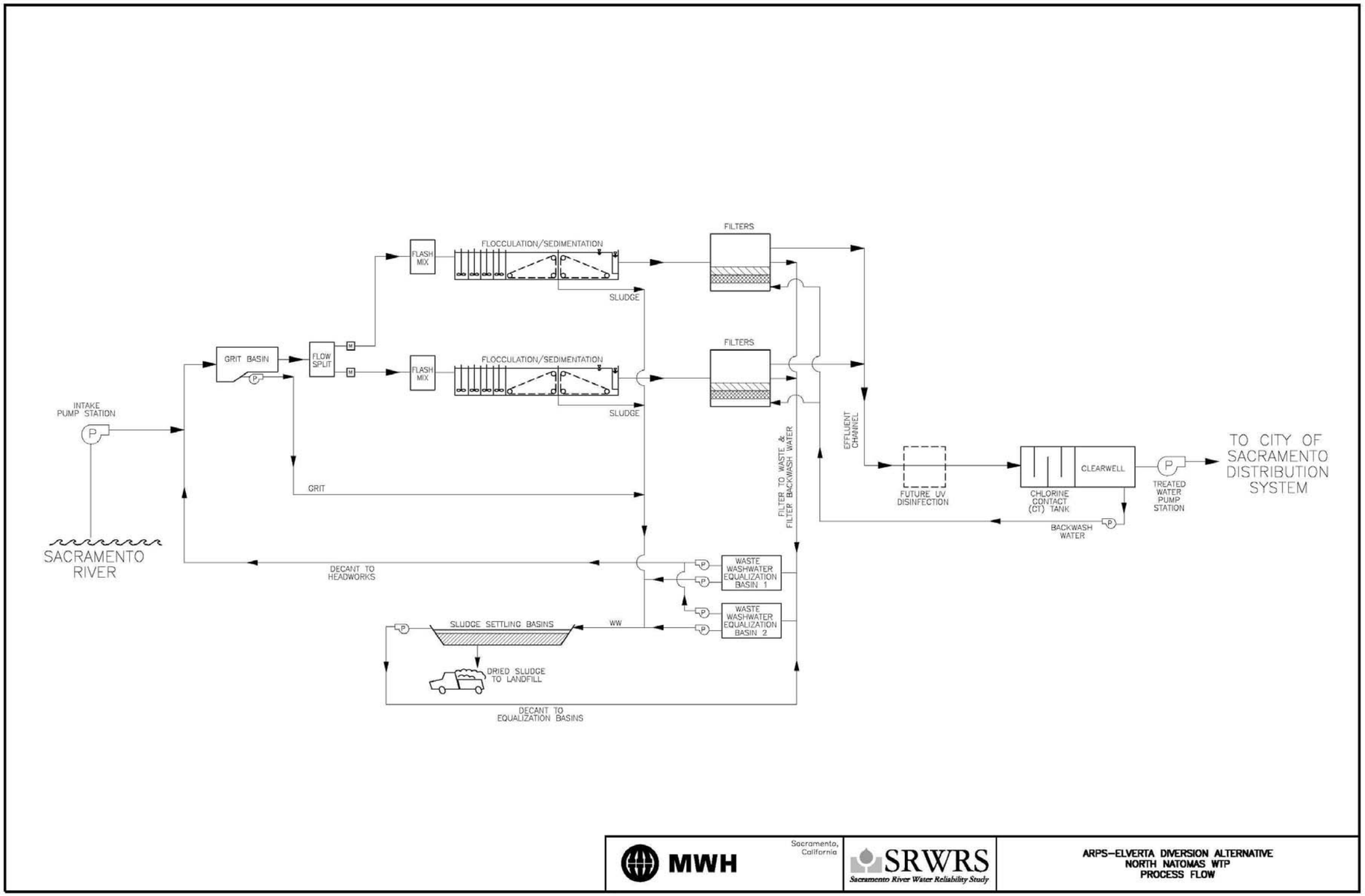

Figure 5-12 ARPS-Elverta Diversion Alternative North Natomas WTP Process Flow

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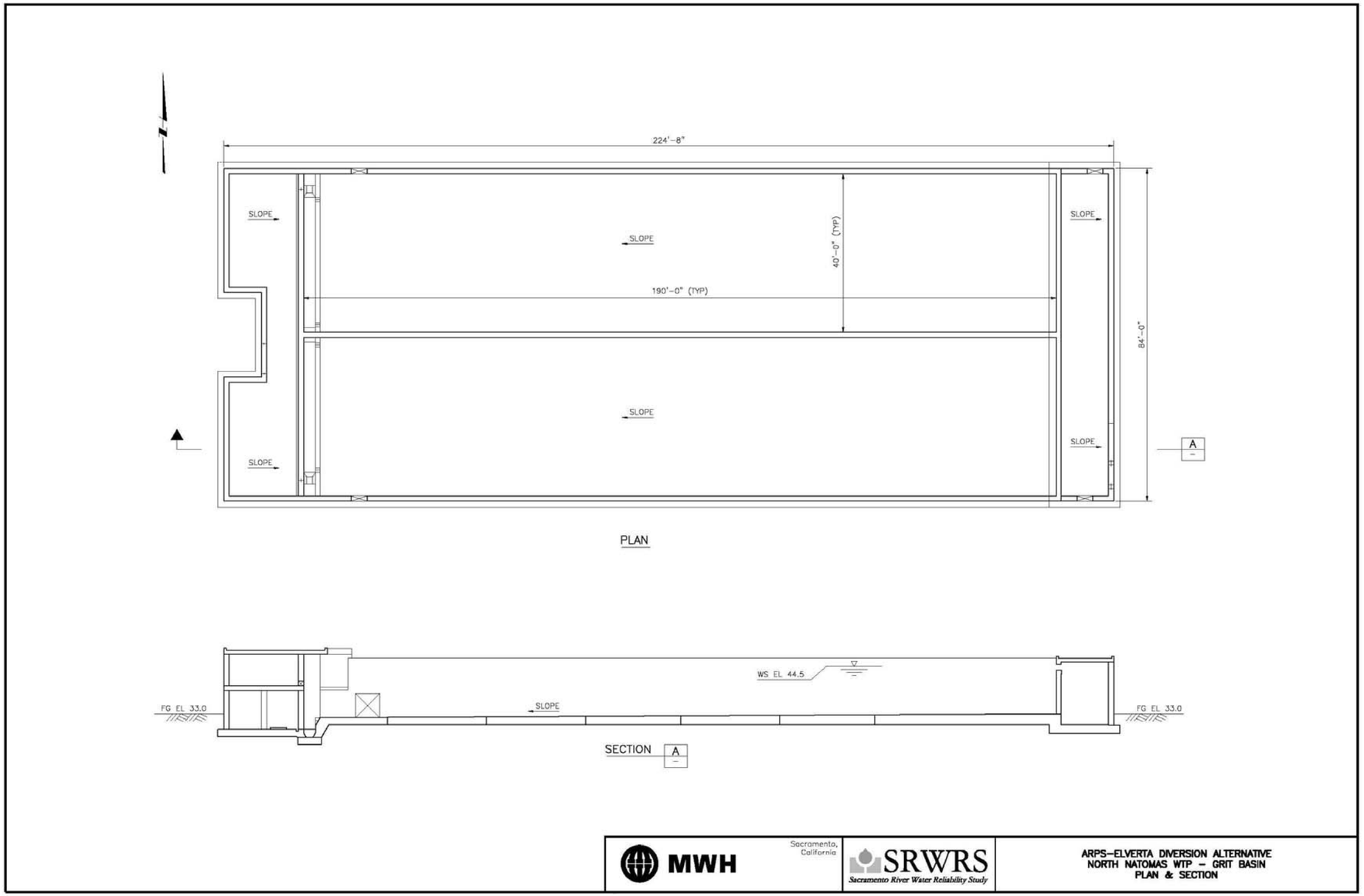

Figure 5-13 ARPS-Elverta Diversion Alternative North Natomas WTP – Grit Basin Plan & Section

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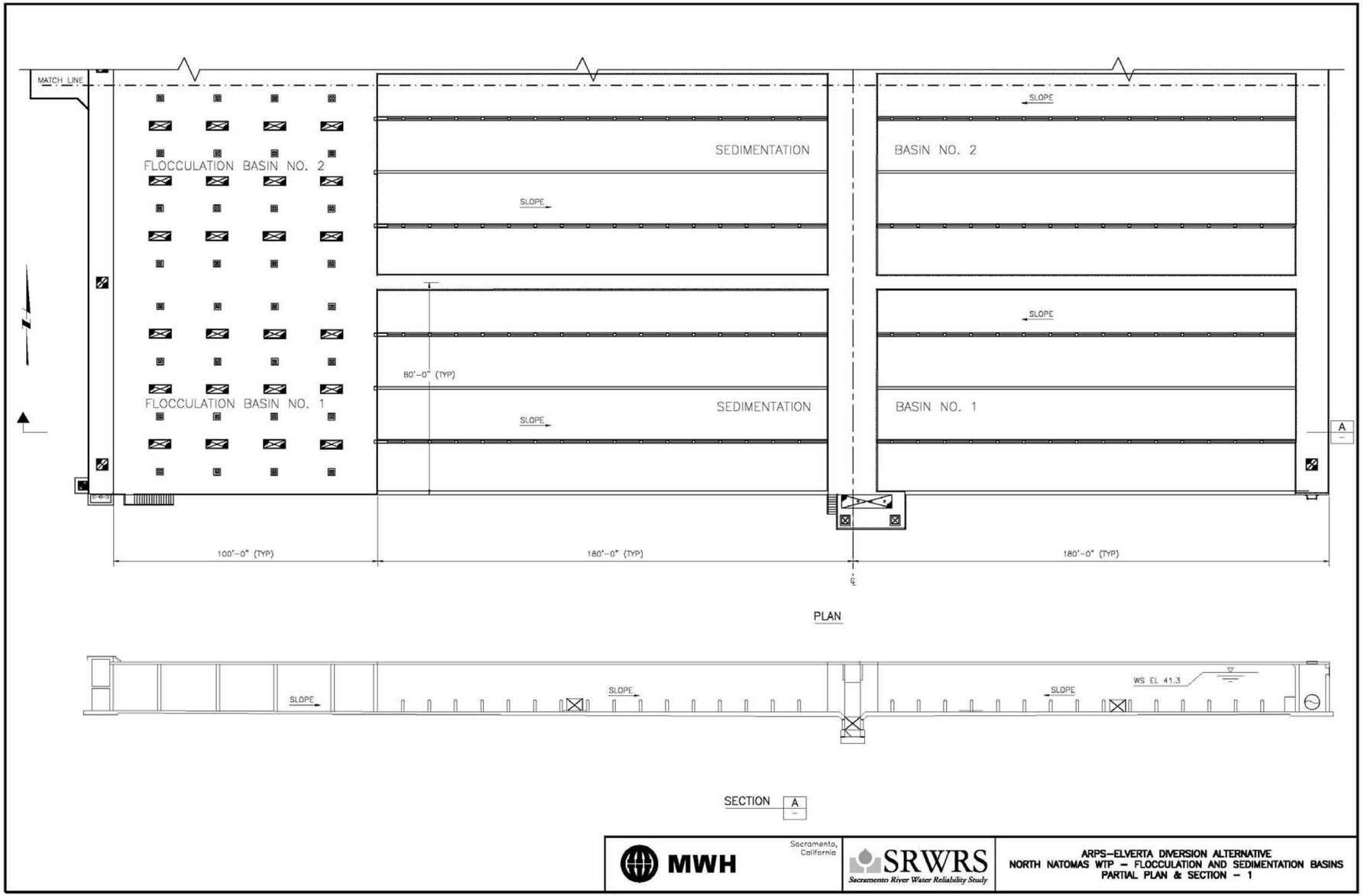

Figure 5-14 ARPS-Elverta Diversion Alternative North Natomas WTP – Flocculation and Sedimentation Basins Partial Plan & Section – 1

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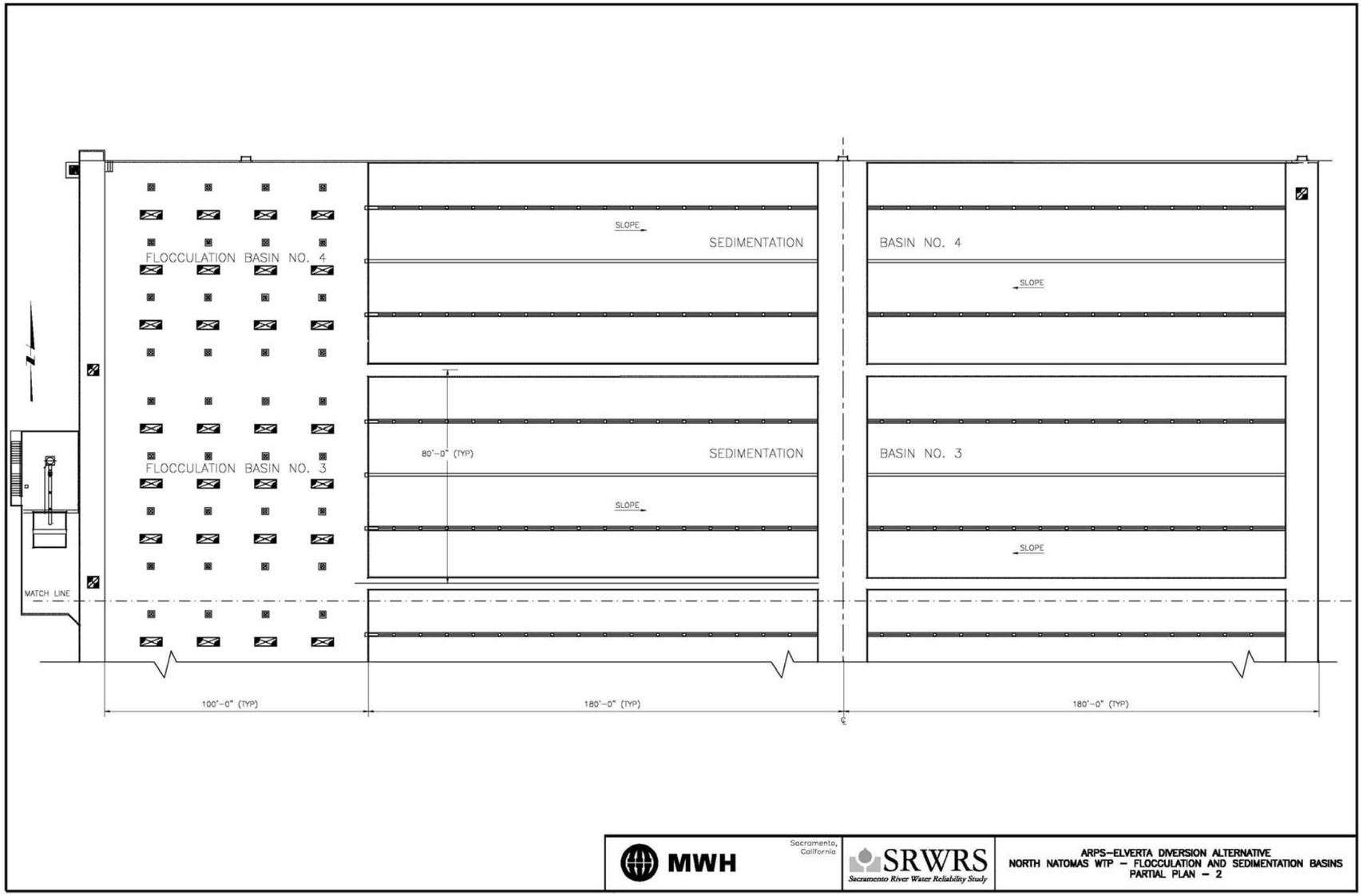

Figure 5-15 ARPS-Elverta Diversion Alternative North Natomas WTP – Flocculation and Sedimentation Basins Partial Plan – 2

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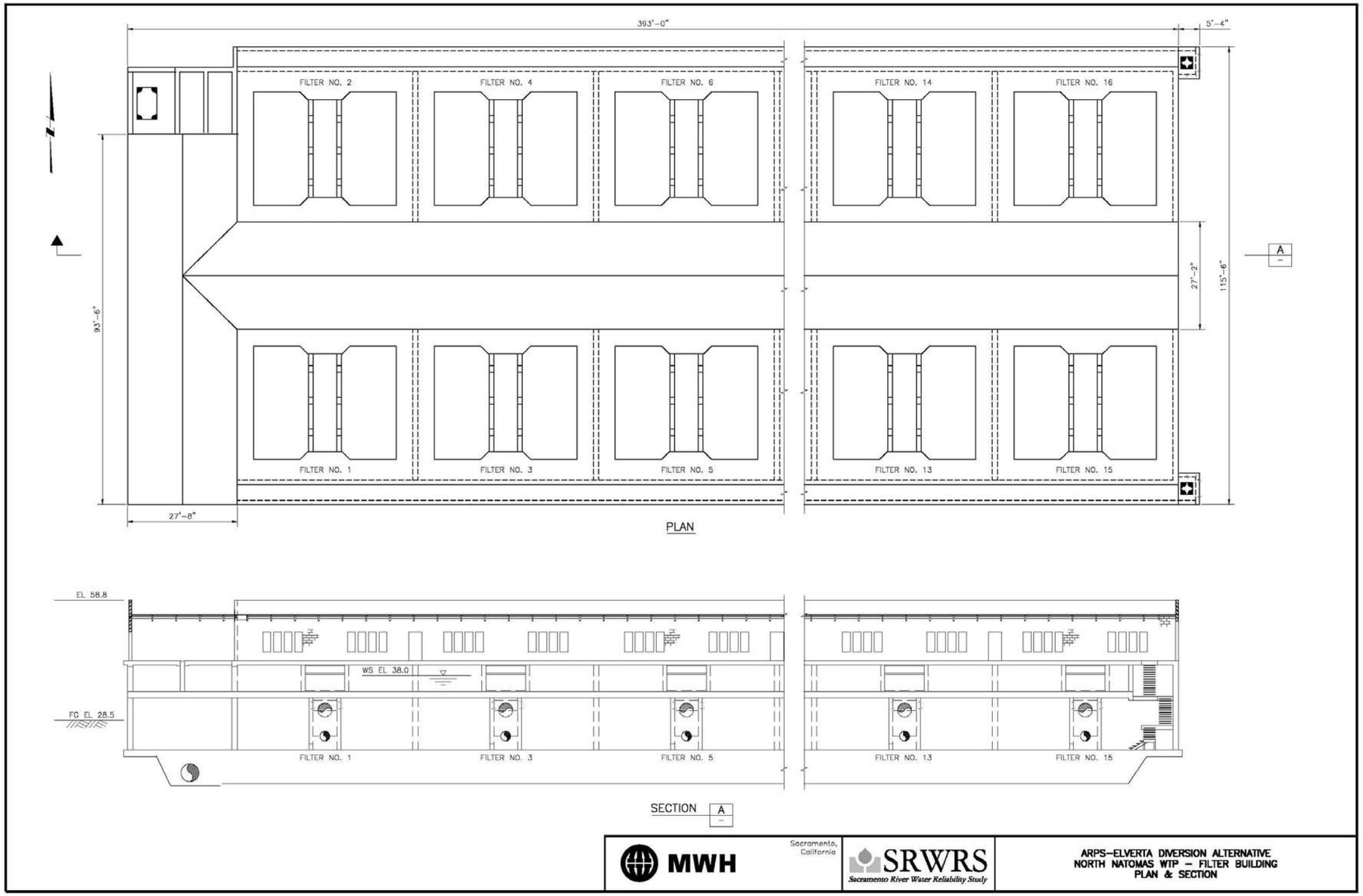

Figure 5-16 ARPS-Elverta Diversion Alternative North Natomas WTP – Filter Building Plan & Section

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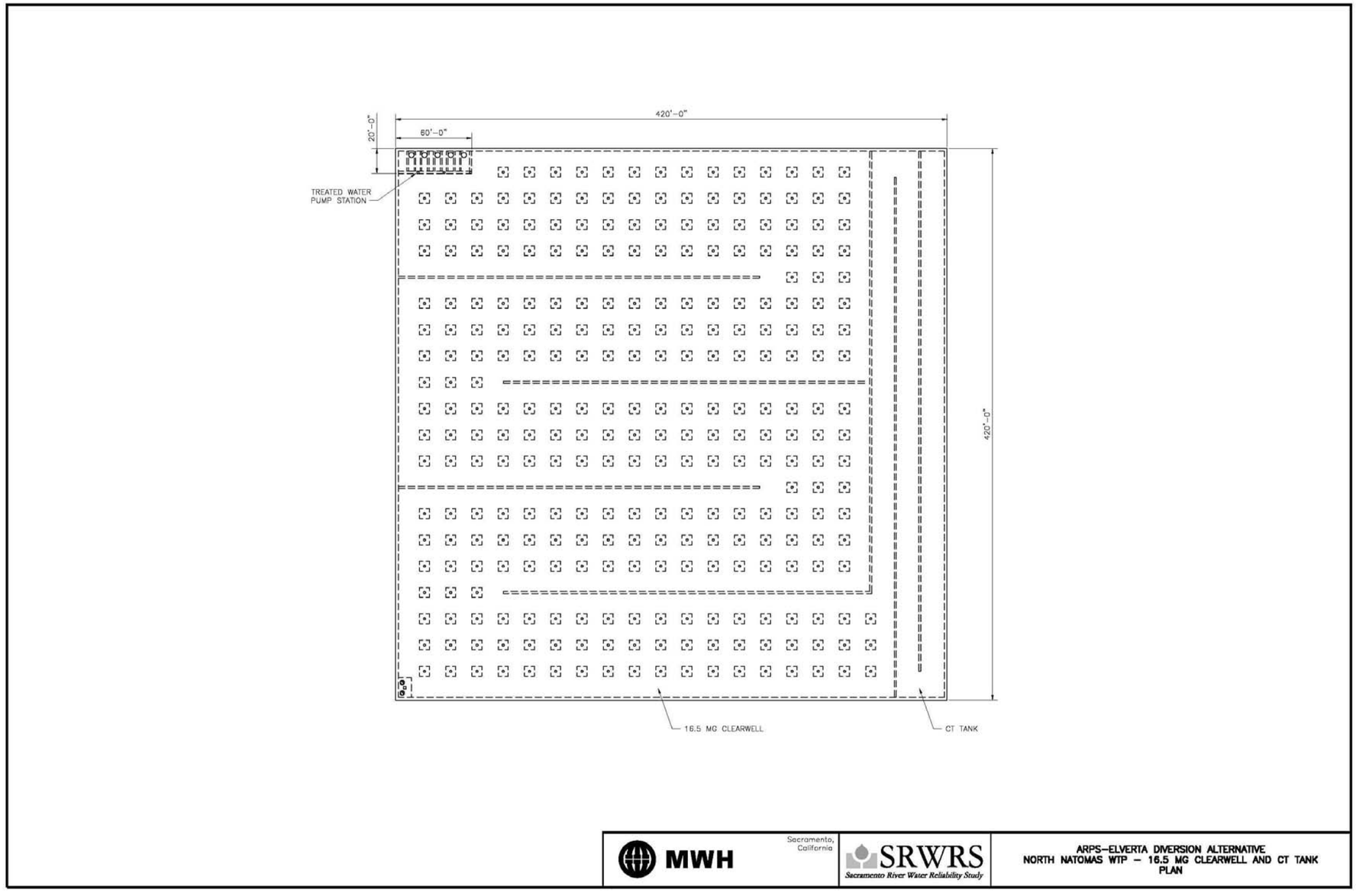

Figure 5-17 ARPS-Elverta Diversion Alternative North Natomas WTP – 16.5 MG Clearwell and CT Tank Plan

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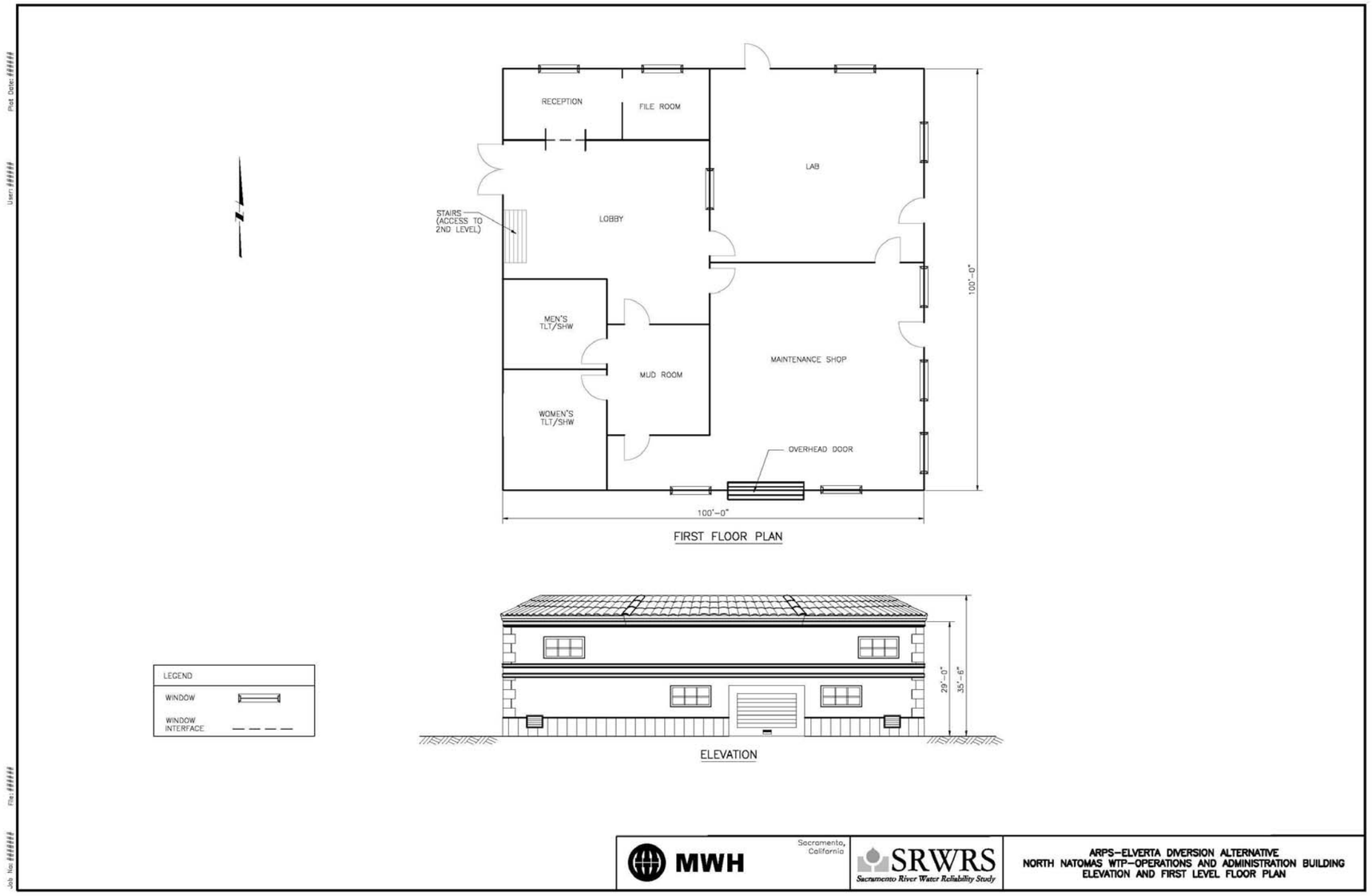

Figure 5-18 ARPS-Elverta Diversion Alternative North Natomas WTP – Operations and Administration Building Elevation and First Level Floor Plan

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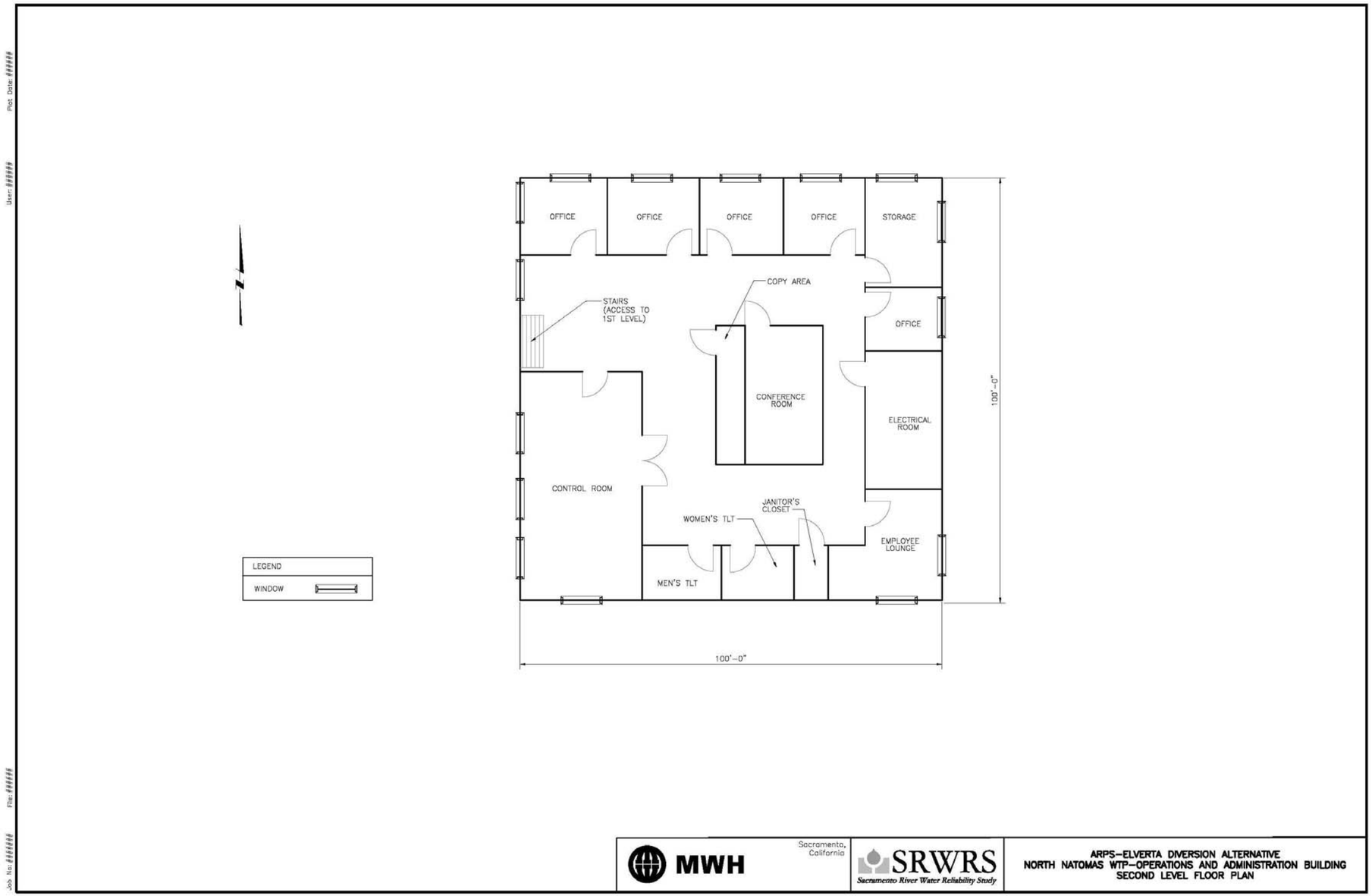

Figure 5-19 ARPS-Elverta Diversion Alternative North Natomas WTP – Operations and Administration Building Second Level Floor Plan

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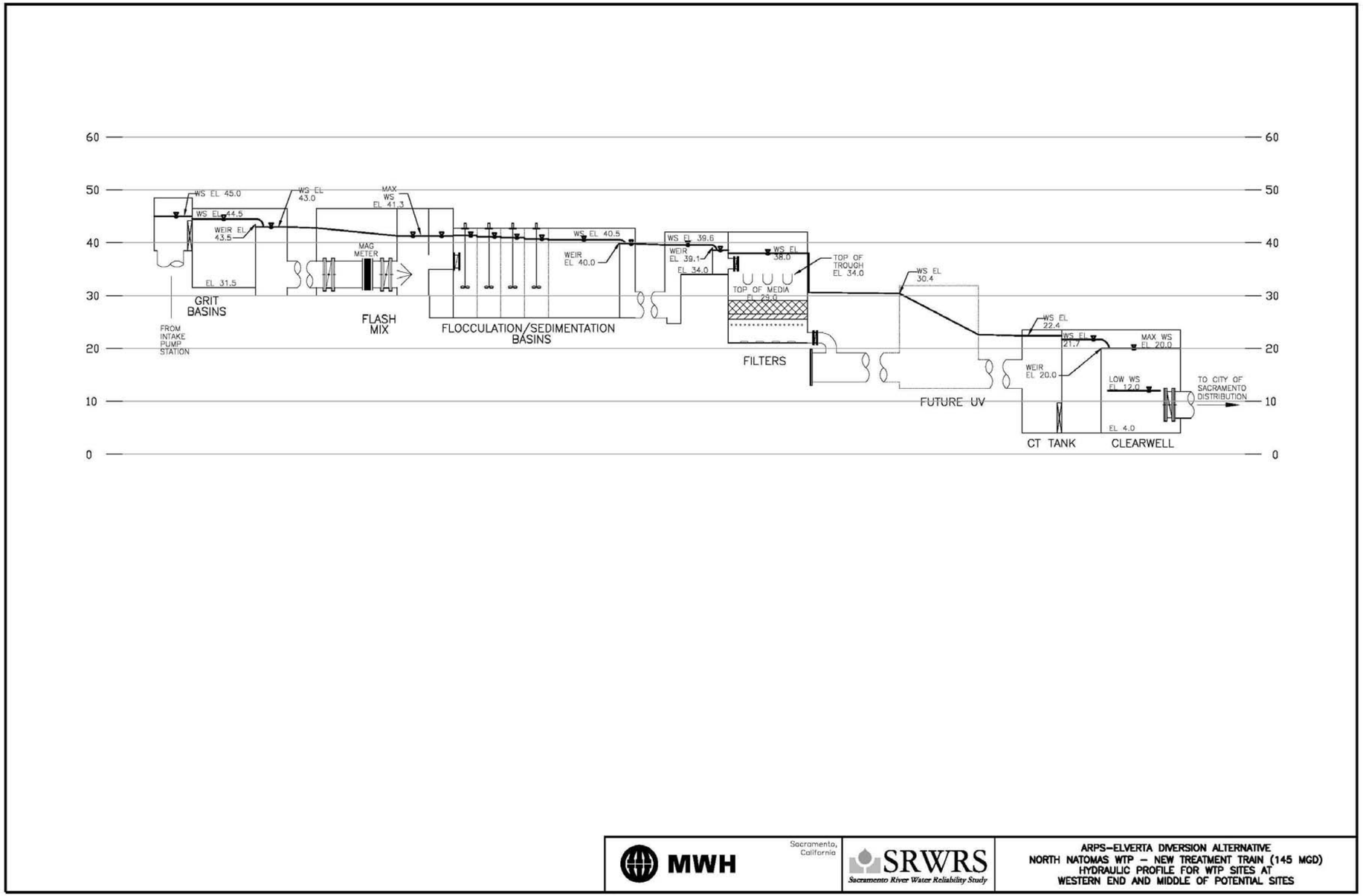

Figure 5-20 ARPS-Elverta Diversion Alternative North Natomas WTP – New Treatment Train (145 mgd) Hydraulic Profile for WTP Sites Located at Western End and Middle of Potential Sites

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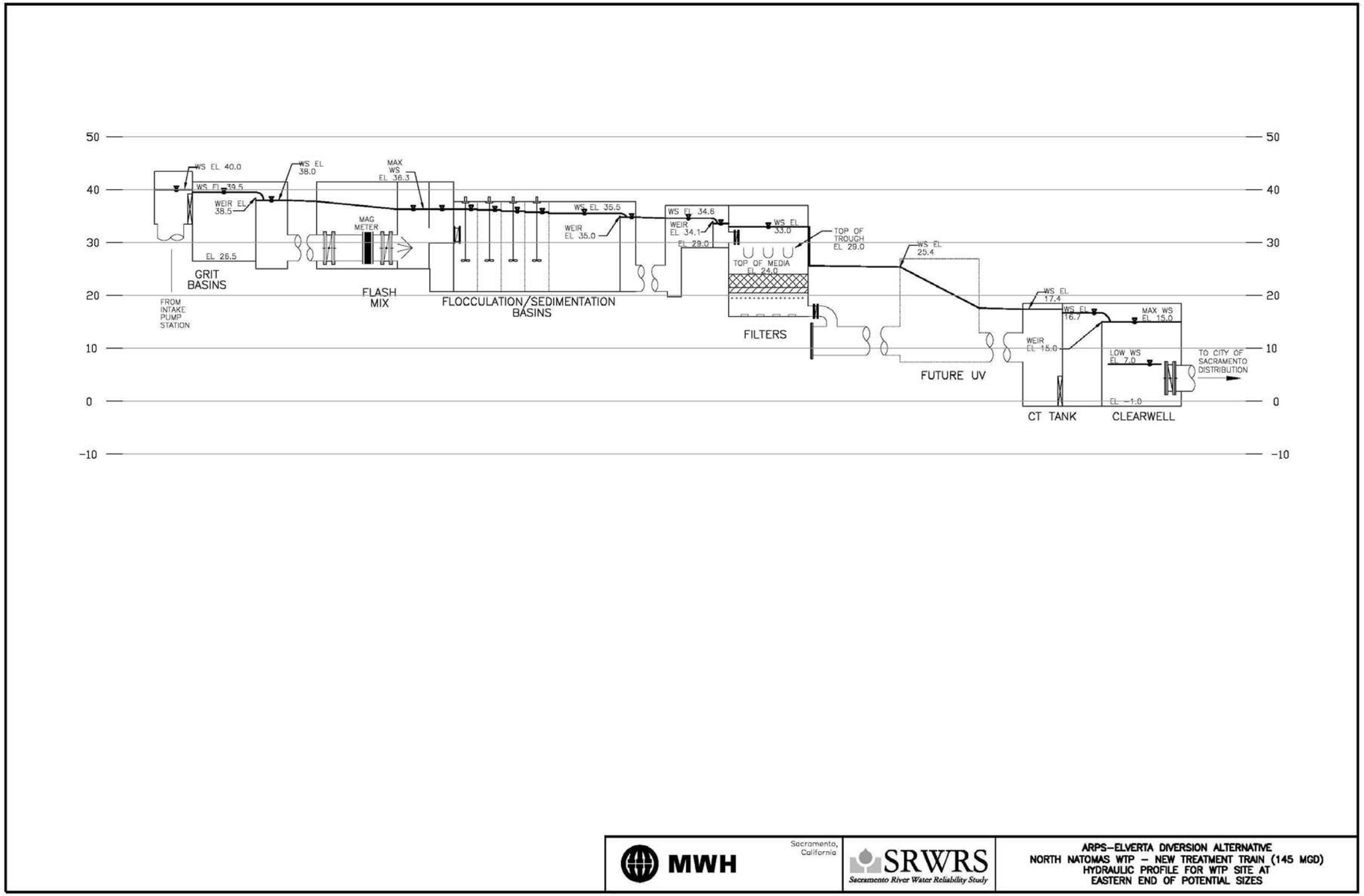

Figure 5-21 ARPS-Elverta Diversion Alternative North Natomas WTP – New Treatment Train (145 mgd) Hydraulic Profile for WTP Site Located at Eastern End of Potential Sites

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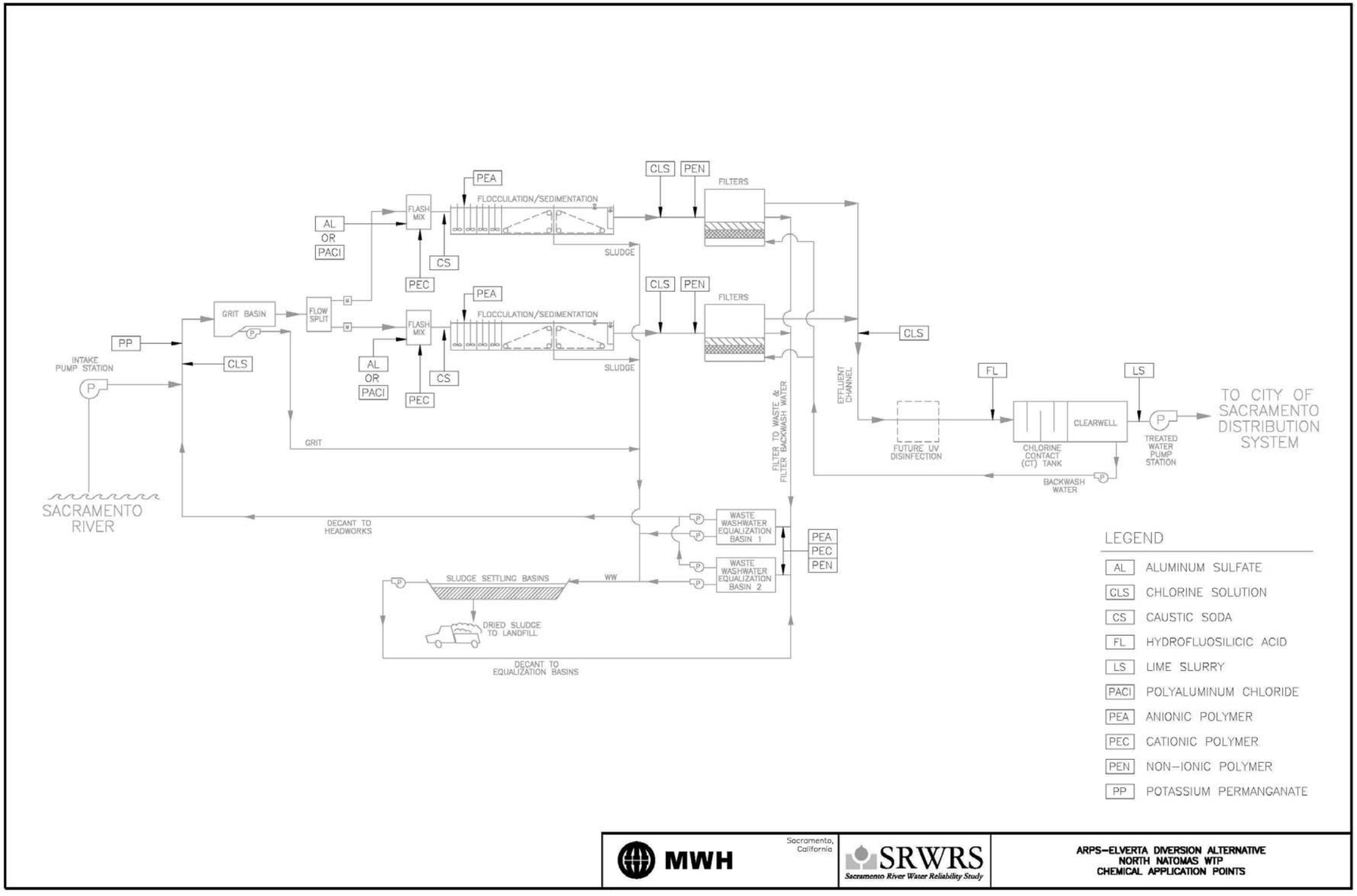

Figure 5-22 ARPS-Elverta Diversion Alternative North Natomas WTP Chemical Application Points

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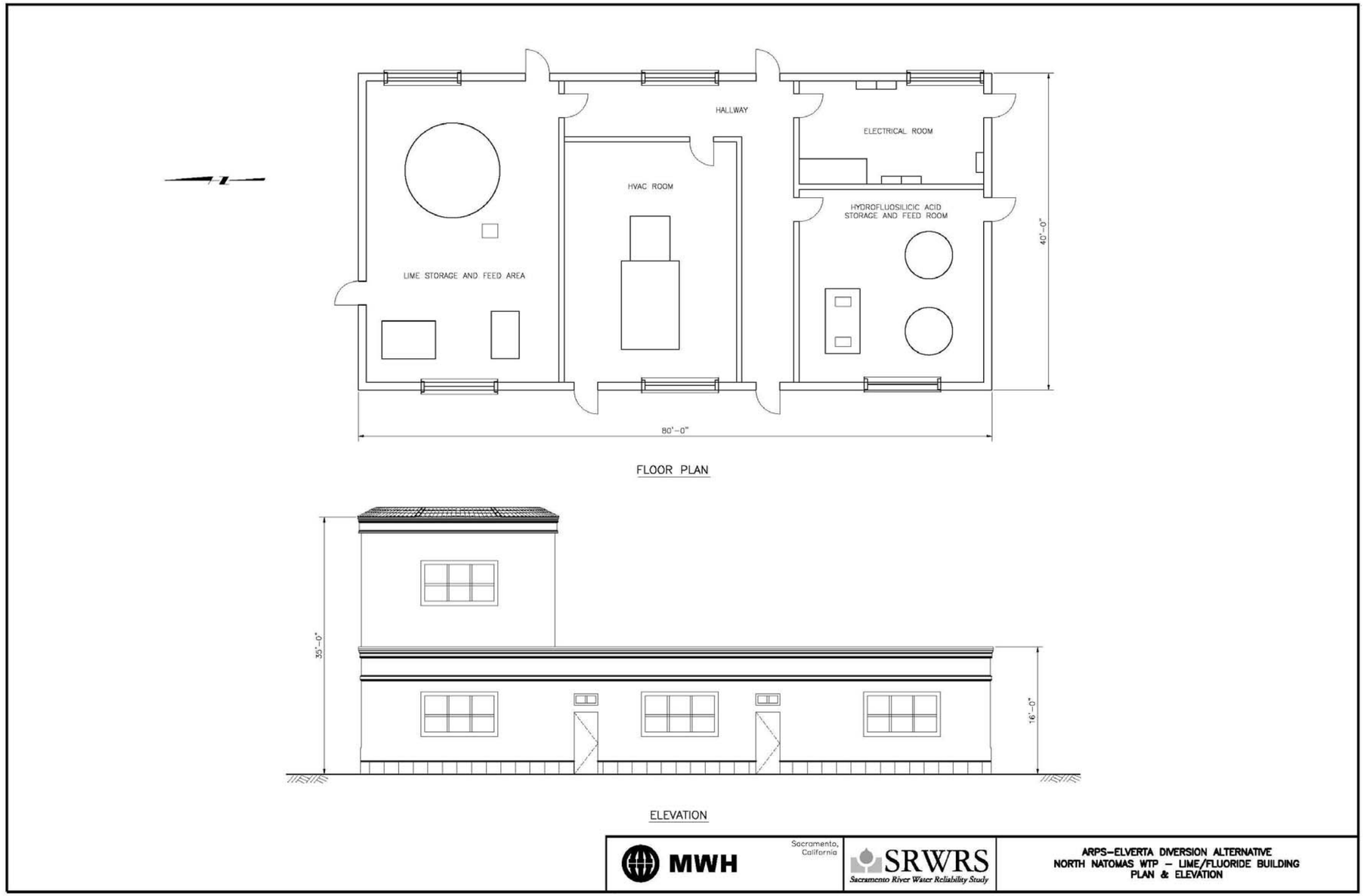

Figure 5-23 ARPS-Elverta Diversion Alternative North Natomas WTP – Lime/Fluoride Building Plan & Elevation

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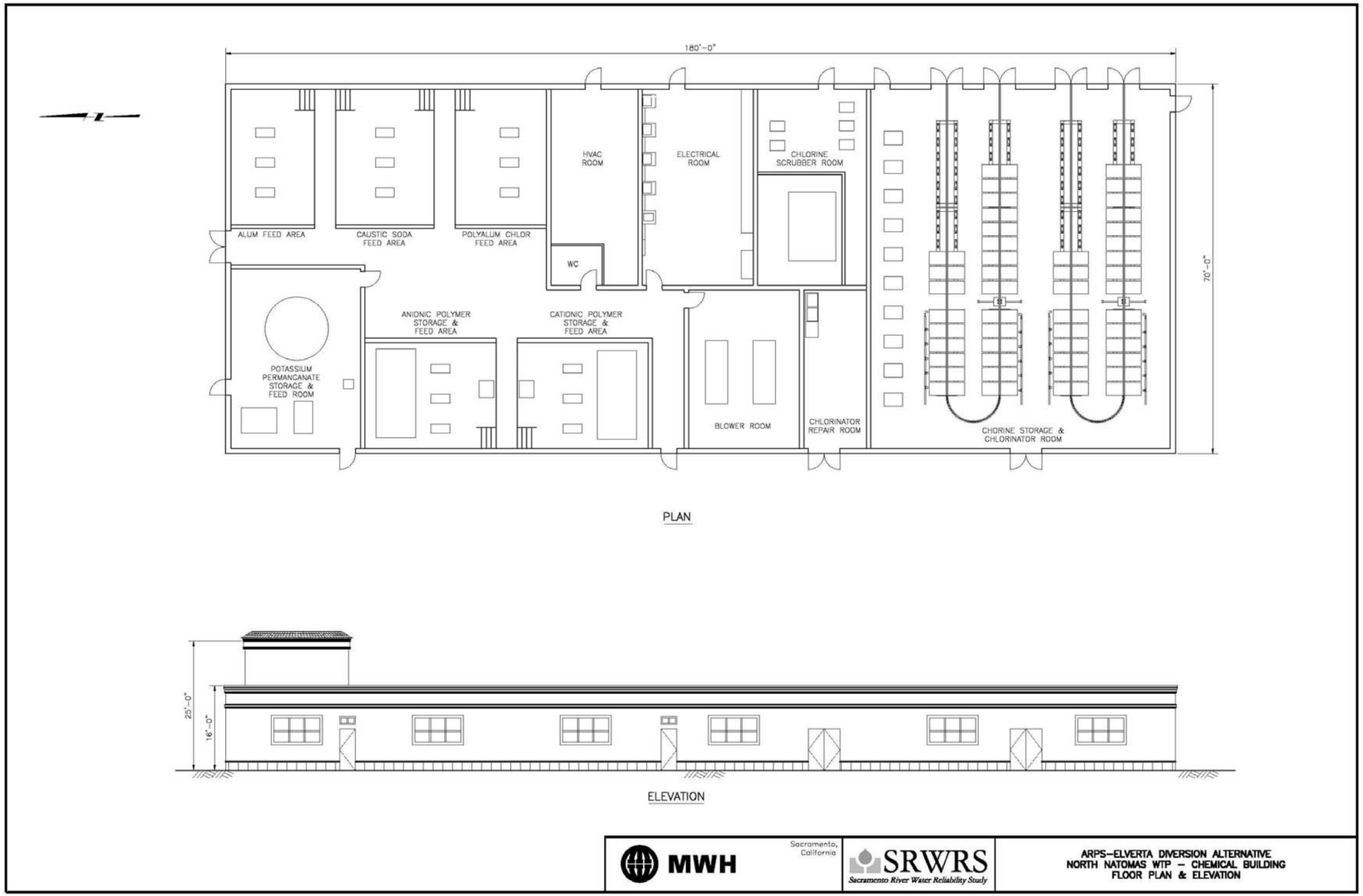

Figure 5-24 ARPS-Elverta Diversion Alternative North Natomas WTP – Chemical Building Floor Plan & Elevation

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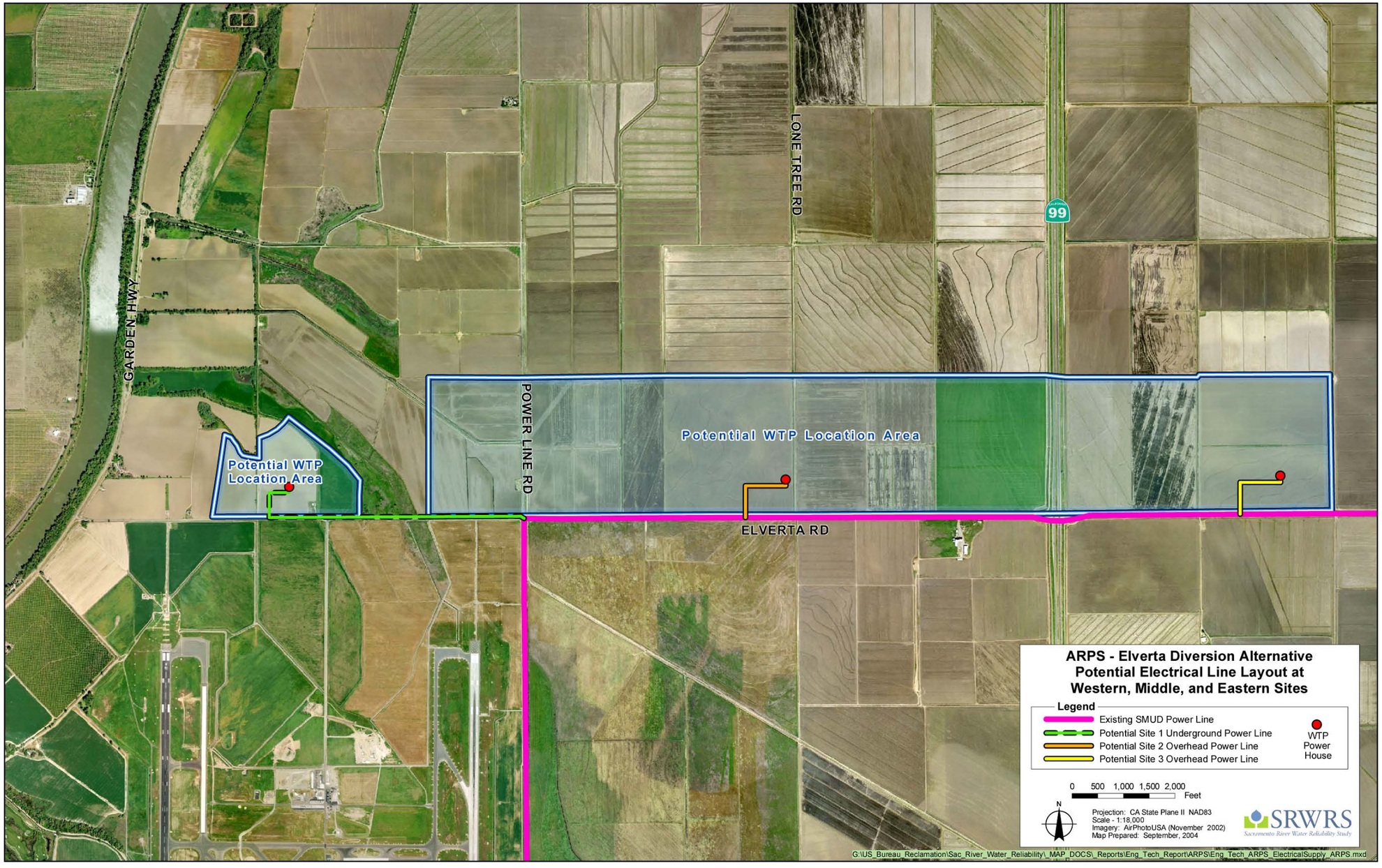

Figure 5-25 ARPS-Elverta Diversion Alternative Potential Electrical Line Layout at Western, Middle, and Eastern Sites

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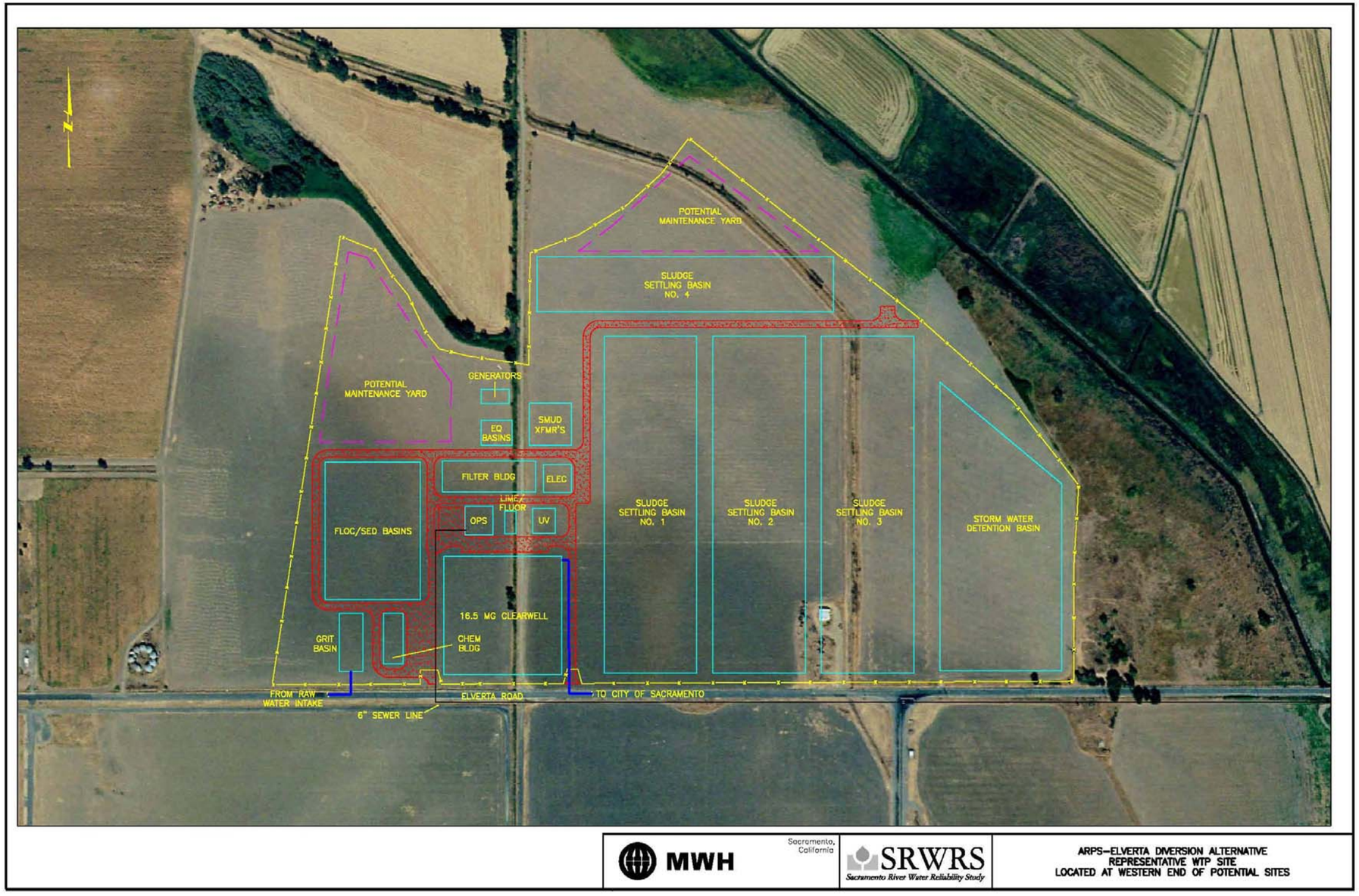

Figure 5-26 ARPS-Elverta Diversion Alternative Representative WTP Site Located at Western End of Potential Sites

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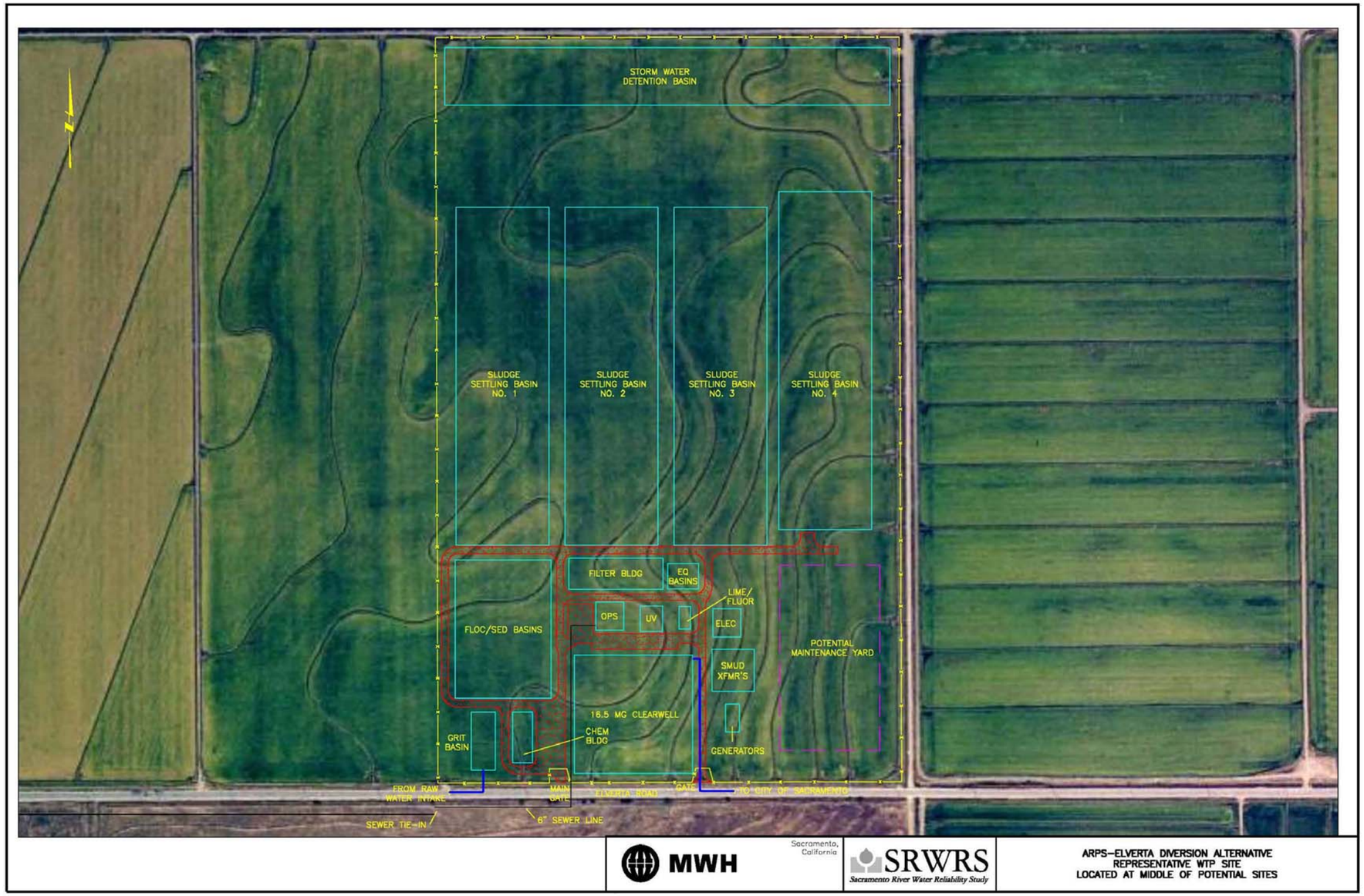

Figure 5-27 ARPS-Elverta Diversion Alternative Representative WTP Site Located at Middle of Potential Sites

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Figure 5-28 ARPS-Elverta Diversion Alternative Representative WTP Site Located at Eastern End of Potential Sites

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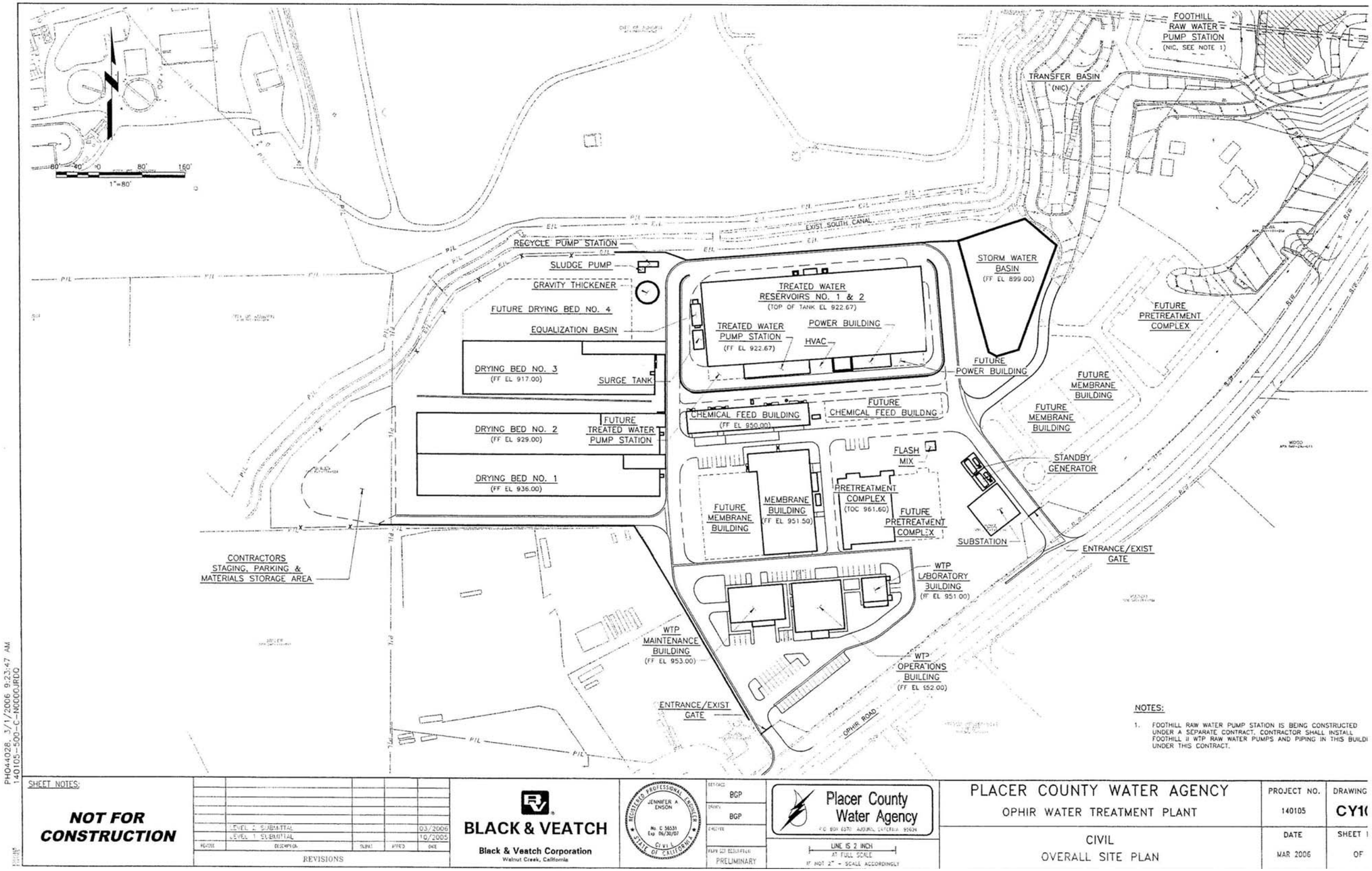

Figure 5-29 Ophir WTP Overall Site Plan

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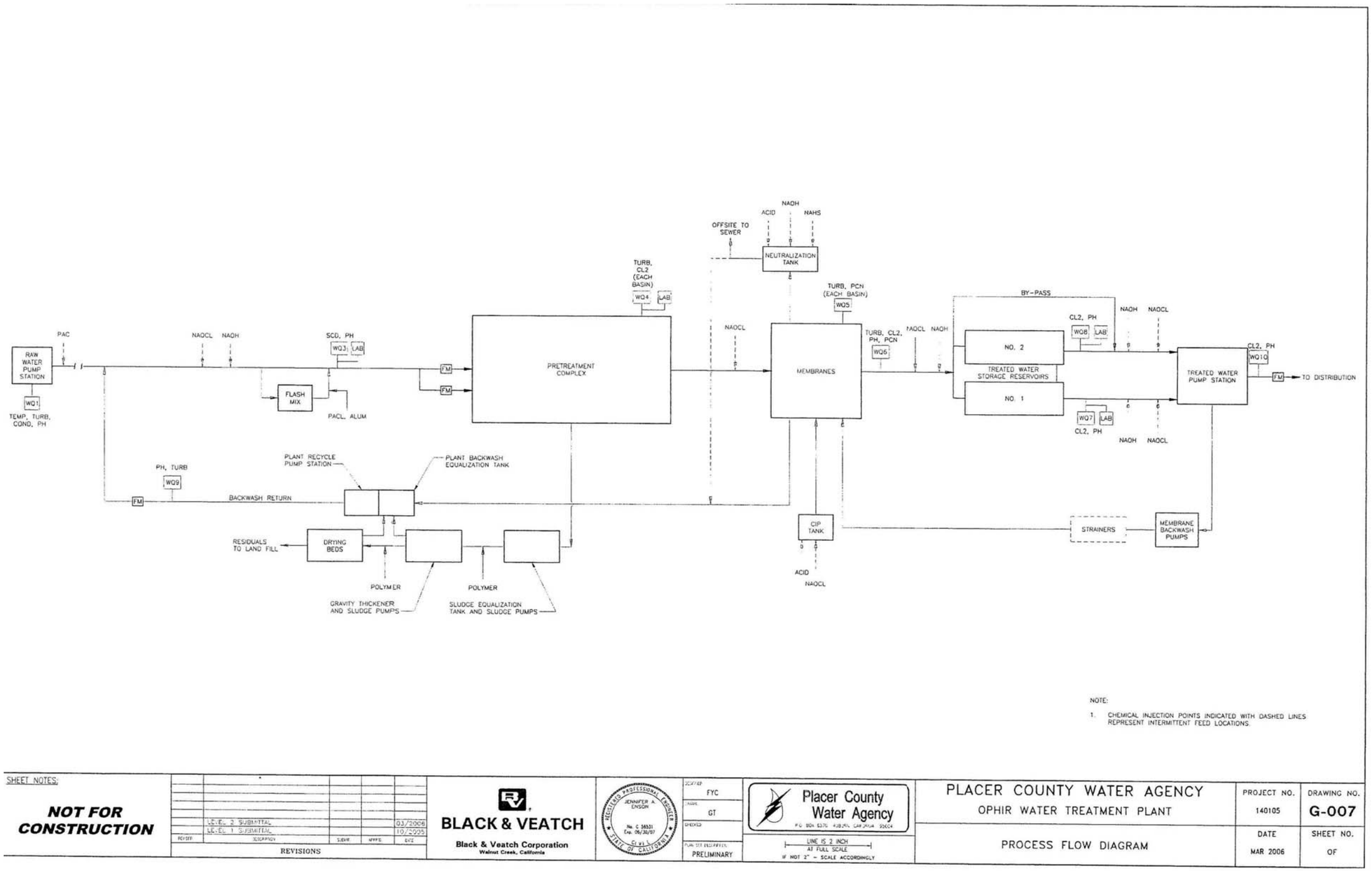

Figure 5-30 PCWA Ophir WTP Process Flow Diagram

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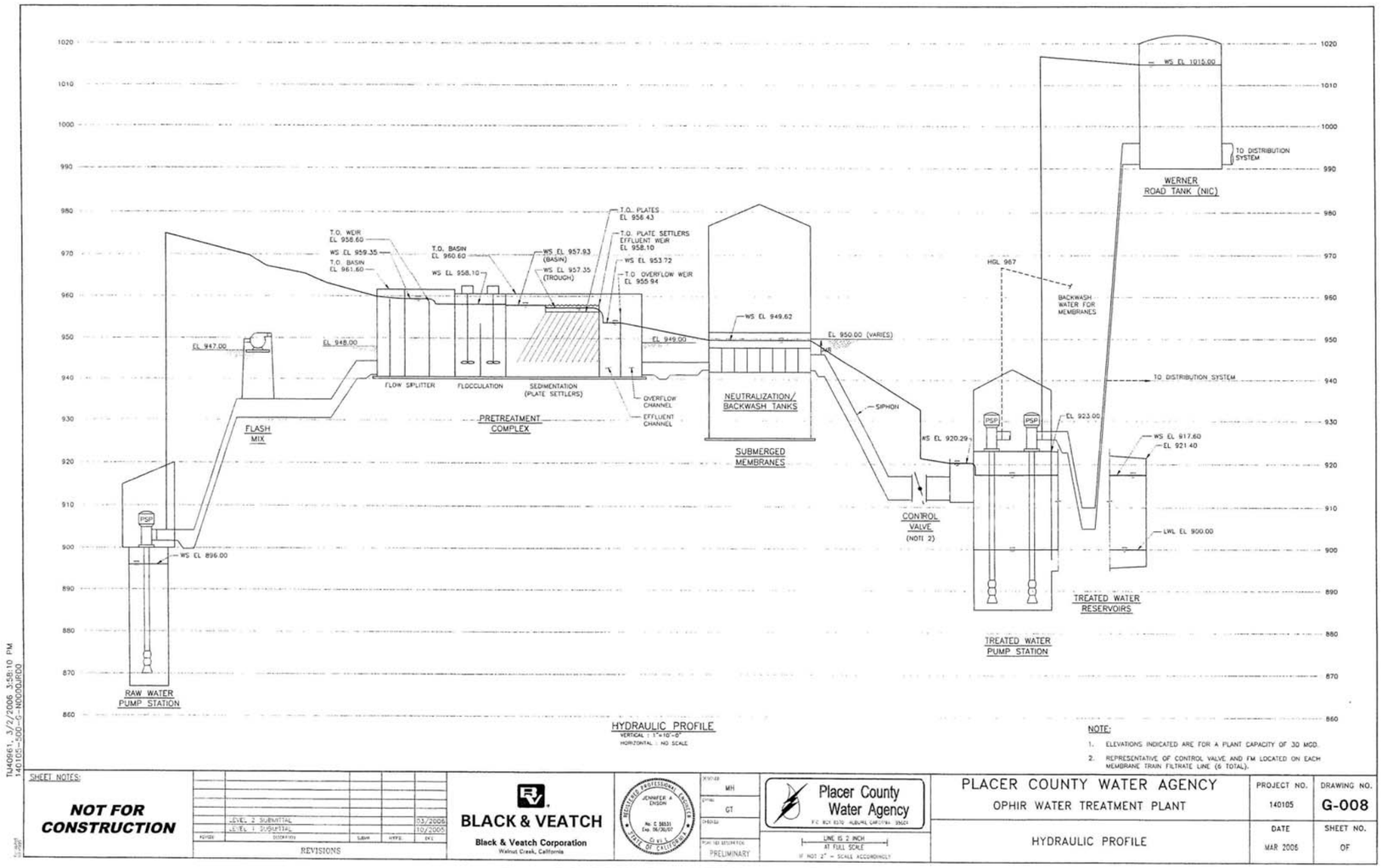

Figure 5-31 PCWA Ophir WTP Hydraulic Profile

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CHAPTER 6 TREATED WATER PIPELINES

This chapter describes the pumps and pipes that would be used to convey water from the WTP to the four project partners. The systems discussed here are the major water transmission pumps and pipelines only. Storage of the treated water and distribution of that water to the ultimate users is the responsibility of each individual partner and is not part of this project.

Treated water is to be conveyed to two of the four partners: PCWA and Sacramento. As previously mentioned, under this alternative, Roseville would increase use of groundwater, and SSWD would use existing facilities to divert additional water via use of shoulder capacity. Therefore, no additional treated water pipelines are required.

The pump station at the Ophir WTP on Ophir Road would pump into a transmission main that would deliver water to PCWA. PCWA would receive water at two points: one at the intersection of Athens Road and Fiddyment Road, and the other at the intersection of Baseline Road and Fiddyment Road. The total flow for PCWA would be 65 mgd. The predicted maximum day demand at the first turnout along the transmission main, the turnout at Athens Road and Fiddyment Road, would be 38.8 mgd. The predicted maximum day demand at the second PCWA turnout, the turnout at Baseline Road and Fiddyment Road, would be 26.2 mgd. To provide greater operating flexibility, the pipeline between the first and second PCWA turnout would be sized to convey 26.2 mgd plus half of the 38.8 mgd to the farthest turnout.

A treated water pump station would be built at each of the two WTPs. The pump station at the North Natomas WTP on Elverta Road would pump into a transmission main that would deliver water to Sacramento. Multiple turnouts from this pipeline would probably connect it to Sacramento's distribution system, but to provide maximum flexibility, it has been assumed that all water would be introduced into Sacramento's system at the farthest point from the WTP, the intersection of the East Drainage Canal with Del Paso Road.

6.1. HYDRAULICS

MWH used a water network model, H2ONet, to determine pipeline sizes, flow velocities, and heads. **Figure 6-1** presents the pipeline sizes recommended based on that computer analysis. **Figure 6-1** also presents the flow velocity in each pipe reach at peak flow. **Table 6-1** presents the design criteria used in the network analysis.

Five pump stations would be required in the system. One pump station would be located at the Sacramento River intake and would include pumps for the SRWRS partners, and additionally pumps for NMWC in the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion Alternative. The second pump station would be the treated water pump station for Sacramento. Two pump stations would be in the raw water system supplying raw water from the American River. (The raw water pumping system is described in **Chapter 4** of this report). The fifth pump station would be a treated water pump station for the Ophir WTP. **Table 6-2** presents basic statistics for each pump station.

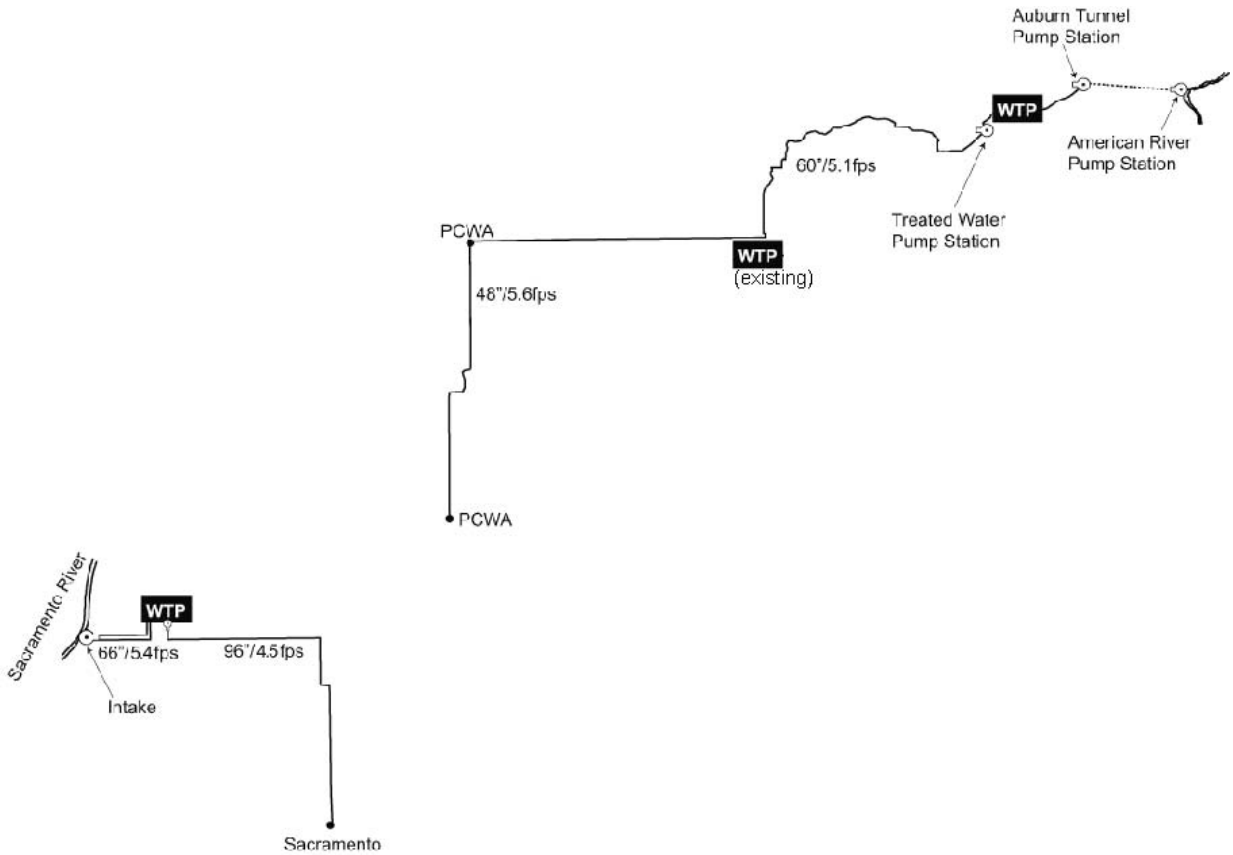

Figure 6-1 Pipe Sizes and Flow Velocities in Feet per Second at Design Flow for the ARPS-Elverta Diversion Alternative

Analyses were conducted to test some of the assumed design criteria. Testing was performed to see how sensitive the results were to the assumed pipe friction "C" value. If a "C" value of 120 (higher friction losses) is used instead of 140, the head losses in the system increase less than 6 percent. It was concluded that the system is not very sensitive to the assumed "C" values and that it is safe to remain with the initially assumed "C" value of 140 for all calculations.

The pipes shown in **Figure 6-1** were all sized to have a peak pipe velocity of about 5 fps. Testing was performed to determine how sensitive the system is to pipe velocity. Hydraulic analyses were conducted with pipe sizes that gave peak velocities of 8 fps second and 10 fps, respectively. **Table 6-3** summarizes the total dynamic head that the Sacramento treated water pump station would have to pump under each of the three pipeline velocity scenarios.

Table 6-1 Design Criteria for Pipeline Hydraulic Analysis

Description	Planning Assumption
Turnout Flows	
Sacramento (one turnout at Del Paso Road)	145 mgd
PCWA Dry Creek-West Placer Turnout at Baseline Rd near Fiddyment Road)	45.6 mgd
PCWA Turnout at Fiddyment Road and Athens Avenue	38.8 mgd
Turnout Delivery Pressure	
Sacramento (one turnout at Del Paso Road)	50 psi
PCWA Dry Creek-West Placer Turnout at Baseline Rd near Fiddyment Road	290 feet above msl
PCWA Turnout at Fiddyment Road and Athens Avenue	350 feet above msl
Storage Reservoirs	
Locate storage at the WTP equal to the operational storage requirements (i.e., capacity to handle differences between plant flow and pumped flow, estimated as about 0.1 x maximum daily flow). No other storage to be provided by this system.	
Storage at Elverta WTP	16.5 MG
Storage at Ophir WTP	6.5 MG
Miscellaneous Modeling Criteria	
Pipe Velocity	5 fps
Pipe Friction Hazen Williams factor	140
Pipe Incidental Losses (fittings per 1,000 feet of pipe)	6

Key:
fps – feet per second
mgd – million gallons per day
MG – million gallons
msl – mean sea level
PCWA – Placer County Water Agency
psi – pounds per square inch
SSWD – Sacramento Suburban Water District
WTP – water treatment plant

Table 6-2 Pump Station Hydraulic Characteristics

Pump Station	Peak Flow (mgd)	Discharge W.S. El. (ft)	Supply W.S. El. (ft)	Friction Head Loss (ft)	TDH (ft)	Efficiency (%)	Connected Horsepower[(1)]
Intake at Sac River	145	45	4	5	46	70	2,050
Intake at Sac River for Natomas	136	40	4	1	37	70	1,600
Sacramento TWPS	145	130	15	46	161	80	6,400
American River PS Modifications	65	726	485	5	246	80	3,515
ATPS No. 3	65	950	700	5	255	80	4,372
Ophir WTP TWPS	65	1020	935	30	65	80	1,114

Note:
(1) Connected horsepower includes an allowance for a backup pump.
Key:
ATPS – Auburn Tunnel Pump Station
ft – feet
mgd – million gallons per day
PS – pump station
TDH – total dynamic head
TWPS – treated water pump station
W.S. El. – water surface elevation
WTP – Water Treatment Plant

Table 6-3 Effects of Pipeline Velocity on Pump Station Dynamic Head for the Sacramento Treated Water Pump Station

Pipe Size and Peak Flow Velocity	Pump Station Total Dynamic Head (ft)	Pump Station Connected Horsepower
Velocity = 5 ft/s; Pipe Diameter = 96"	183	7,007
Velocity = 8 ft/s; Pipe Diameter = 78"	270	11,750
Velocity = 10 ft/s; Pipe Diameter = 72"	331	14,405

Key:
ft – feet
ft/s – feet per second
Flow rate – 165 mgd

As shown in **Table 6-3**, head loss in the system is quite sensitive to pipe velocity. Maintaining design pipe velocities at 5 fps is recommended.

A significant elevation drop occurs from the high point on the transmission main serving water to PCWA to its lowest turnout at Fiddyment Road and Baseline Road. To avoid excessive pressures in the pipelines, pressure-reducing valves would be needed at selected points. Two pressure-reducing valves would be placed on the pipeline between the Ophir WTP and the Sunset WTP. A third pressure-reducing station would be placed between the Sunset WTP and the turnout at Athens Road and Fiddyment Road; a fourth pressure-reducing valve would be located between the turnout at Athens Road and Fiddyment Road and the turnout at Baseline Road and Fiddyment Road.

6.2. PREFERRED ALIGNMENT

The selected treated water pipeline alignments are presented in **Figure 1-1**. More detailed plans and profiles for the pipeline are presented in **Figures 6-2** through **6-12** (end of chapter). The plans and profiles in **Figures 6-2** through **6-8** were developed specifically for this report. The plans and profiles in **Figures 6-9** through **6-12** were initially developed (in the PCWA Water Conveyance Alternative Evaluation[8]) prepared by Steve Yaeger and Loren Bottorff for PCWA. The report identified the preferred

[8] PCWA. 2003. Water Conveyance Alternatives Evaluation for Foothill Phase 2 Water Treatment Plant. November.

route for a pipeline from the Ophir WTP to the Sunset WTP. Revised horizontal alignments have been recently developed for portions of **Figures 6-9** through **6-12**. The approximate locations of these revised alignments are indicated on the drawings.

The treated water pipeline feeding Sacramento would leave the pump station at the WTP and follow Elverta Road east to the East Drainage Canal where it would turn south. The pipeline would follow just outside the levee on the east side of the East Drainage Canal and would end at Del Paso Road where it would connect to the Sacramento water distribution system. Along Elverta Road, the pipe would be constructed approximately at the fog line (white line on the edge of the outside lane of the road) on the north side of the road, although it would shift north or south occasionally to avoid obstacles (structural or environmental).

The treated water pipeline for PCWA would originate at a pump station at the Ophir WTP. It would follow a complex route to the Sunset WTP as described further below. Proceeding downstream from the Sunset WTP, the pipeline would travel due west to Fiddyment Road via Athens Road. The pipeline would follow Fiddyment Road south to the turnout at the corner of Baseline Road and Fiddyment Road.

The route for the transmission main between the Ophir WTP and the Sunset WTP would be parallel to and in the same right-of-way as PCWA's planned 48-inch treated water pipeline. The route for the transmission main would be as follows (starting at the Sunset WTP and working upstream):

- The pipeline would begin (station 0+00) at the City of Lincoln metering point north of the Sunset WTP. A pressure reducing station would be required to match the energy level (about elevation 840) in the pipes at the metering point. The line would be in, or near, the existing dirt maintenance road to Whitney Reservoir. Two existing pipelines also follow the road.
- Geotechnical field investigations would be required to determine the best alignment (approximately 1,000 feet in length) past Whitney Reservoir during design of the pipeline. The pipeline could be encased in concrete to counteract buoyancy if located within the reservoir area. An alignment within the dam area is possible, but would likely require complete reconstruction of the dam. An alignment downstream of the dam would need to traverse very rough terrain.
- The pipeline would continue north of Whitney Reservoir within, or near, the existing maintenance road adjacent to the Caperton Canal to approximately station 42+50. A 30-inch-diameter stub to the City of Lincoln would be provided at this point.
- The pipeline would traverse cross-country near existing property lines to Caperton Court and cross Sierra College Boulevard; this segment would also cross the Caperton Canal.
- For approximately the next 18,000 feet, the pipeline would be located within the alignment of the proposed Bickford Ranch Road and adjoining streets to the proposed 3 MG treated water tank near the existing communications tower. (Note: Placing the pipeline within the proposed road alignments avoids encroaching into the conservation easement that is part of the Bickford Ranch permits.) The pipeline within the proposed Bickford Ranch Road would be parallel to the planned 48-inch treated water pipeline and the water distribution line for the development.
- From the proposed tank, the pipeline would continue easterly for about 3,500 feet.
- The pipeline would pass under (bore and jack) the existing eastbound line of the UPRR. About 200 feet of the Antelope Canal adjacent to the railroad would be placed in pipe
- The pipeline would head easterly for about 1400 feet cross-country, near the edge of a pasture.

- The pipeline would turn south for about 1200 feet, parallel to the westbound line of the UPRR, and would then pass under the westbound line of the UPRR. (Note: Three optical cables and high-pressure petroleum pipeline parallel to the railroad on the south side.)
- The pipeline would head easterly and parallel Callison Road for about 1000 feet to the intersection with Taylor Road.
- The pipeline would continue northeast, parallel to Taylor Road within the paved shoulder. An existing telephone cable is in one shoulder of Taylor Road and a gas line is in the other shoulder. Portions of this segment may be located adjacent to the paved area or within private property adjacent to the road right-of-way.
- The pipeline would pass under (bore and jack) the railroad and Highway 193. (Note: Three optical cables and high-pressure petroleum pipeline parallel to the railroad on the south side.)
- The pipeline would be located within the Taylor Road paved area to an open cut crossing of the Dutch Ravine Canal near Ridge Road.
- The pipeline would be located within, or adjacent to, Ophir Road to the Ophir WTP. Most of this segment would share the alignment with the proposed raw pipeline to PWCA's Foothill WTP.

6.3. ALIGNMENT EVALUATION SELECTION

Alternative treated water pipeline alignments were analyzed in the November 2003 report prepared by Steve Yaeger and Loren Bottorff, who analyzed three different alignments for the pipeline between the Ophir WTP and the Sunset WTP. The preferred alignment described in this report is the same as the recommended alignment in the Yaeger/Bottorff report with the exception of the revised horizontal alignment portions indicated on the drawings.

6.4. TUNNEL SECTIONS AND OTHER SPECIAL CROSSINGS

The treated water pipeline would cross several drainage ways, railroads, and highways where the pipe may need to be constructed using trenchless technologies (tunneling). The locations listed in **Table 6-4** have been identified as places where trenchless technology would need to be used for pipeline construction. It is possible that biological surveys yet to be conducted would identify additional locations where trenchless technology might be the preferred construction method.

Trenchless technology, or tunneling, involves digging a jacking pit on one side of the drainage way and a receiving pit on the other side, then using a tunneling machine to bore between the two pits, and a jacking machine to push pipe through the hole. Another trenchless technology, known as directional drilling, may be preferred for pipes less than about 48 inches in diameter. If directional drilling is used, the jacking and receiving pits are eliminated and the pipe when installed forms an inverted arc under the drainage way.

Table 6-4 Locations Where Trenchless Technology May Be Used for Pipeline Construction

Description of Location	Approximate Crossing Length (feet)	Approximate Pipeline Station[(1)]
Along the Pipelines from the Sacramento River		
Jacobs Slough crossing Elverta Road just north of the airport (two parallel pipes)	100	70+25
Highway 99 at Elverta Road	300	200+00
East Drainage Canal at Elverta Road	200	280+00
Along the Pipeline from the American River		
Highway 193 and Union Pacific Railroad at Taylor Road	1,700	B (revised alignment)
Union Pacific Railroad near Callison Road	150	(revised alignment)
Union Pacific Railroad near Antelope Canal	150	B280+00
Sierra College Boulevard	150	B70+00
Caperton Canal	100	B63+00
Highway 65	200	940+55
Industrial Avenue and Union Pacific Railroad	200	910+20

Note:
(1)"B" before the station number indicates the stationing is from the Yaeger/Bottorff report. Other station numbers are from the plan and profile sheets developed for this report.

6.5. PIPE MATERIAL

Several materials would be suitable for the treated water pipelines. The most common pipe types for this function and size are welded steel, ductile iron, pretensioned concrete cylinder, and high-density polyethylene pipe. Final project specifications would be written for one or more of these four pipe types.

Should the pipe be steel, it would be coated and lined. The lining is usually cement mortar, although epoxy linings are occasionally used. The coating can be cement mortar, epoxy, or polyethylene tape. Cathodic protection may be used to protect the pipe from corrosion, depending on the corrosiveness of local soils. This would be determined during predesign investigations.

Should the pipe be ductile iron, it would have a cement mortar lining. The pipe would have no bonded coating, but would have polyethylene sleeves over the pipe for corrosion protection. Cathodic protection may be used as with steel pipe.

No additional lining or coating is used with pretensioned concrete cylinder pipe. Cathodic protection may be used as with steel pipe.

No coating or lining is necessary for high-density polyethylene pipe. The pipe itself requires no cathodic protection, but the valves and some of the other appurtenances would include ferrous metals and may require cathodic protection.

6.6. PIPELINE APPURTENANCES

The piping system would include valves at strategic locations. A valve would be provided on the two downstream sides of each "tee" in the pipeline to isolate reaches of the pipe for maintenance. Isolation valves would be installed approximately every 1,000 feet along the pipe where there are no "tees." The system would also include an air release valve at each high point and a blowoff at each low point. The air release valve assembly would be housed in a small aboveground enclosure located along the side of the

road. The blowoff assembly would be entirely below ground. The system also would include access ports into the pipeline at intervals of approximately 1,000 feet.

6.7. CONSTRUCTION CHARACTERISTICS

The pipe trench would be typically about 12 feet wide for the Sacramento pipe and 8 to 10 feet wide for the PCWA pipe. The trench would be 10 to 15 feet deep. Shoring would be used to maintain a narrow vertical side-wall trench and to protect workers. See **Figure 4-3** for a typical trench cross section. A work area at least 5 feet wide on one side of the trench and at least 15 feet wide on the other side of the trench would be needed for construction. Where available, a larger work area of up to 40 feet on one side of the trench would be provided to facilitate construction and reduce cost. Some of the work area could be achieved through temporary lane closures during work hours.

Groundwater is high year-round along the alignment west of the NEMDC. Extensive dewatering would be needed during construction in those areas with high groundwater, from before the trench is opened until after the trench is backfilled. Water removed from the construction area would be treated to remove sediment and discharged to the closest drainage way. A discharge permit would be needed. The dewatering method most likely to be used is a network of well points along the pipeline alignment. Wells would be drilled to several feet below the trench invert, which would be 10 to 12 feet below grade. Well spacing could vary widely. Commonly, wells would be about 100 feet apart.

Trenches in the area east of Fiddyment Road may be into rock. Blasting and/or drilling may be needed to excavate these trenches in certain areas.

Pipe bedding would be crushed rock or sand. Pipe zone backfill would be crushed rock or sand or controlled density fill (very low strength concrete). Trench zone backfill would be native material. Any native materials unsuitable for trench backfill would be hauled away to a disposal site selected by the project sponsors.

Crews would be able to install pipe of this size and depth at production rates of 100 feet of trench per day during dry weather if no problems were encountered. However, to account for possible delays, average production rates would probably be about 40 feet of trench per day. **Table 6-5** presents the estimated pipe lengths and construction durations for the raw water pipeline to each of the WTP sites. For long pipe reaches, it is assumed that the contractor would work on multiple headings at once, reducing the construction duration. A contract period 40 to 60 workdays longer than the construction period would be needed to allow for mobilization, demobilization, punchlist work, and weather delays. Typical workdays would be from 7:00 a.m. to 3:30 p.m. Monday through Friday, with occasional work continuing as late as 7:00 p.m. and/or Saturday.

Table 6-5 Estimated Construction Duration for the Treated Water Pipelines

	Pipe Length (feet)	Trench Length (feet)	Construction Duration (work days)	Contract Period (work days)	Contract Period (calendar days)
From a WTP site near the middle of potential sites (about 2.6 miles from the intake) to Sacramento turnout	36,000	36,000	450 (2 headings)	500	800
From Ophir WTP to Sunset WTP	53,000	53,000	442 (3 headings)	500	800
From Sunset WTP to Fiddyment Road and Athens Road	34,000	34,000	425 (2 headings)	475	760
From Fiddyment Road and Athens Road to Baseline Road and Fiddyment Road	36,000	36,000	450 (2 headings)	500	800

Key:
PCWA – Placer County Water Agency
WTP – water treatment plant

The construction operation could use a number of different combinations of equipment. One possible scenario would include at each heading one or two excavators to excavate the trench, place pipe bedding and pipe zone backfill and set the pipe; a front-end loader to move soil around the work site and load trucks; a dozer or tractor to move trench backfill into place; a large compactor and smaller walk-behind compactors; two to six end dump trucks to haul soil to and from the work site; and miscellaneous trucks to deliver materials and imported fill. Crew size would be 6 to 10 people, not including truck drivers. The crew superintendent and the contractor's project manager and field engineer may be local staff or, if the contractor is not a local, may be brought in from outside the local area.

The number of truck trips to and from each construction heading each day would vary depending on how much of the native soil can be used for backfill. If all the backfill can be native material taken from the trench and stored at the work area, about 11 truck trips would be needed to haul away excess material and 11 more truck trips to haul in imported material on an average day. Should the native material be unsuitable for backfill, or should there be inadequate space at the work site to store the material until the trench is ready for backfill, the number of truck trips would increase to about 23 truck trips to bring in material and about 23 truck trips to haul away material.

Trucks hauling materials to and from the construction-site would have loads below highway load weight limits. Trucks hauling soil, rock, or sand to and from the job site would haul from 5 to 10 cubic yards of material in each load. Loads for other trucks would vary depending on what is being hauled, but would always be below H-20 load limits.

Safety on the construction-site would be the responsibility of the construction contractor. The construction contractor would have a company safety program and a job-specific safety program, administered by a project safety officer. Typical procedures would include weekly safety meetings with the construction crew and hazard analyses prepared before the beginning of each new operation. A traffic control plan would be prepared by the construction contractor and reviewed by Sacramento County to ensure traffic is safely routed around the work site. OSHA and Cal-OSHA standards would apply for all work.

For most operations, no particularly noisy equipment is anticipated for the construction work (e.g., no pile driving). The contractor may elect to drive soldier piles and/or sheet piles for shoring of the trench or the jacking and receiving pits. These pile-driving operations would be short term. Typical noise of trucks and diesel-powered equipment would occur. Work would comply with all county noise ordinances. Noise levels would be significant where rock is encountered in the trench excavations.

The construction contractor would have a staging area for field offices and to temporarily park equipment and supplies. This area would be 1 to 5 acres in size. A site has not been selected for this staging area. A 2- to 10-acre site would be chosen for disposal of excess material removed from the trench. Some material would be stockpiled only temporarily at the disposal site and then used later for backfill. Other material would be permanently placed at the disposal site. A grading permit would be obtained for the disposal site. Work at the disposal site would comply with all county requirements, including the grading ordinance and sedimentation and erosion control requirements. An area has not been selected for this disposal site.

The treated water pipeline crosses drainage ways. A stream alteration permit would be requested from CDFG for each crossing. The permit may not allow using open-cut trenching for installing the pipe across the stream; instead, tunneling may be required. A pressure balance tunneling technology would be used if the tunnel was below groundwater levels. Tunneling would involve an approximately 15-foot-wide by 30-foot-long by 25-foot-deep jacking pit on one side of the stream and a smaller receiving pit on the other side.

Construction contract documents would include a general SWPPP. The construction contractor would be required to submit a specific, more detailed SWPPP. The general plan would outline minimum requirements that must be met to minimize erosion and control sediments. The general and specific SWPPPs would comply with the county sediment and erosion control ordinances. Typical best management practices that would be used include the following:

- Covering all exposed slopes and stockpiles with plastic, straw, or hydroseed
- Placing silt fences at the downstream side of all work areas
- Placing a sediment filter in each drop inlet
- Sweeping all work areas frequently
- Constructing sediment ponds in key locations
- Placing waddles or hay bales across steep, disrupted slopes
- Constructing gravel driveways at each work site exit
- Placing waddles or straw bales around the open trench work area

6.8. OPERATING CHARACTERISTICS

Pipelines require very little operations or maintenance. Cathodic protection systems, if used, must be checked once a year. Valves should be exercised every few years. When the pipeline gets older (e.g., say 50 years old or more), occasional pipeline breaks may occur depending on how corrosive the soils are and how well the cathodic protection system is maintained. These breaks would necessitate pipe repairs. The turnouts would include flow monitoring and a flow control valve. Regular operation, monitoring, and repair of the turnouts would be needed. Five to ten people would be involved in pipeline maintenance but none would work full time on this pipeline.

Regular maintenance of the pump stations would be needed, including exercising the standby generator, monitoring the status of equipment at the pump stations, and repairing any damaged parts. It is estimated that maintenance, repair, and operation of the pump stations would take the equivalent of about 20 percent of one person's time at each of the two treated water pump stations.

No hazardous materials would be used for operating the pipelines and pump stations beyond small amounts of lubricants for the pumps and valve operators and fuel for the standby generator. No regular large truck traffic would occur associated with operations and maintenance of the pipelines and pump stations.

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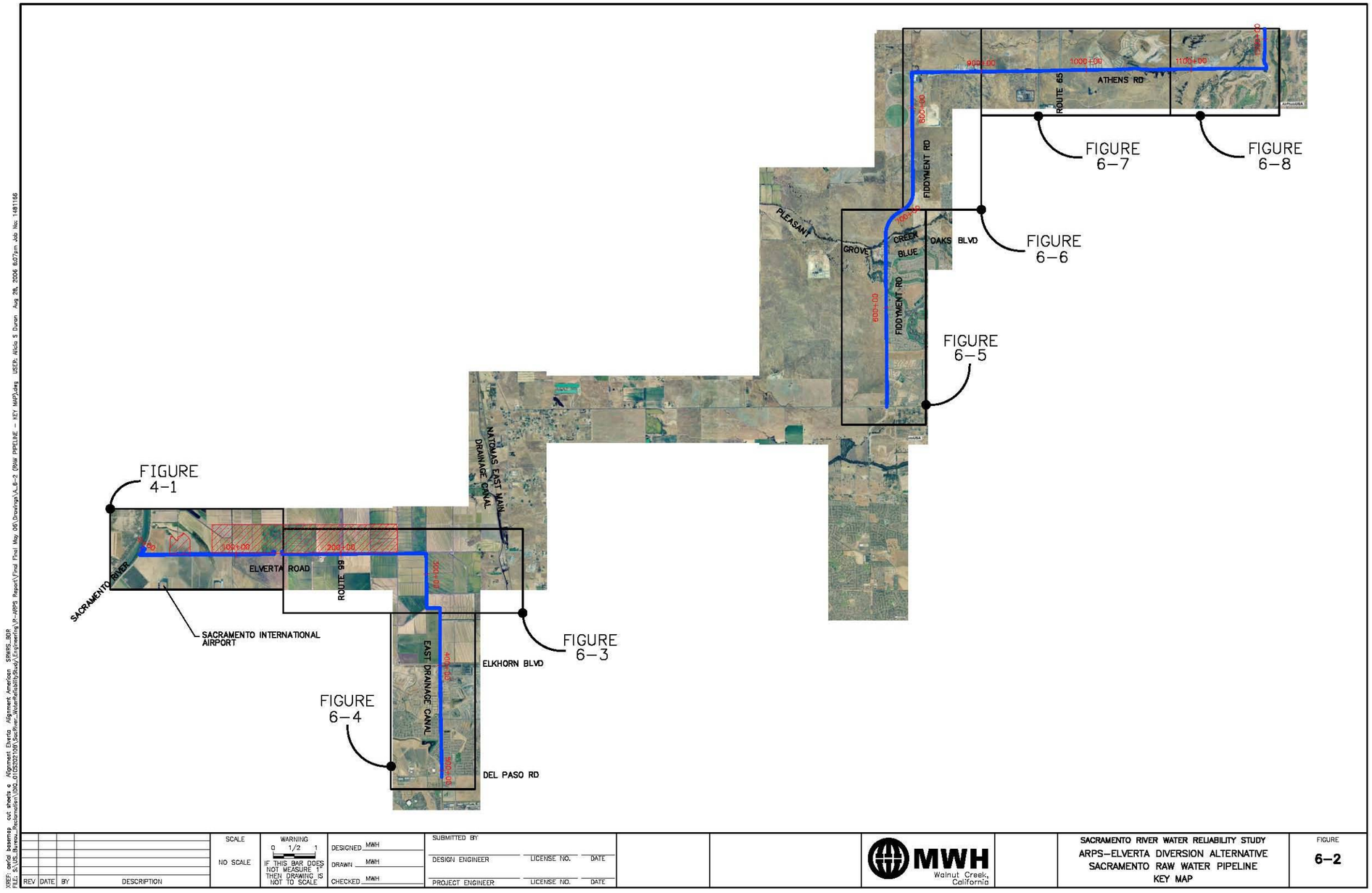

Figure 6-2 ARPS-Elverta Diversion Alternative Sacramento Raw Water Pipeline Key Map

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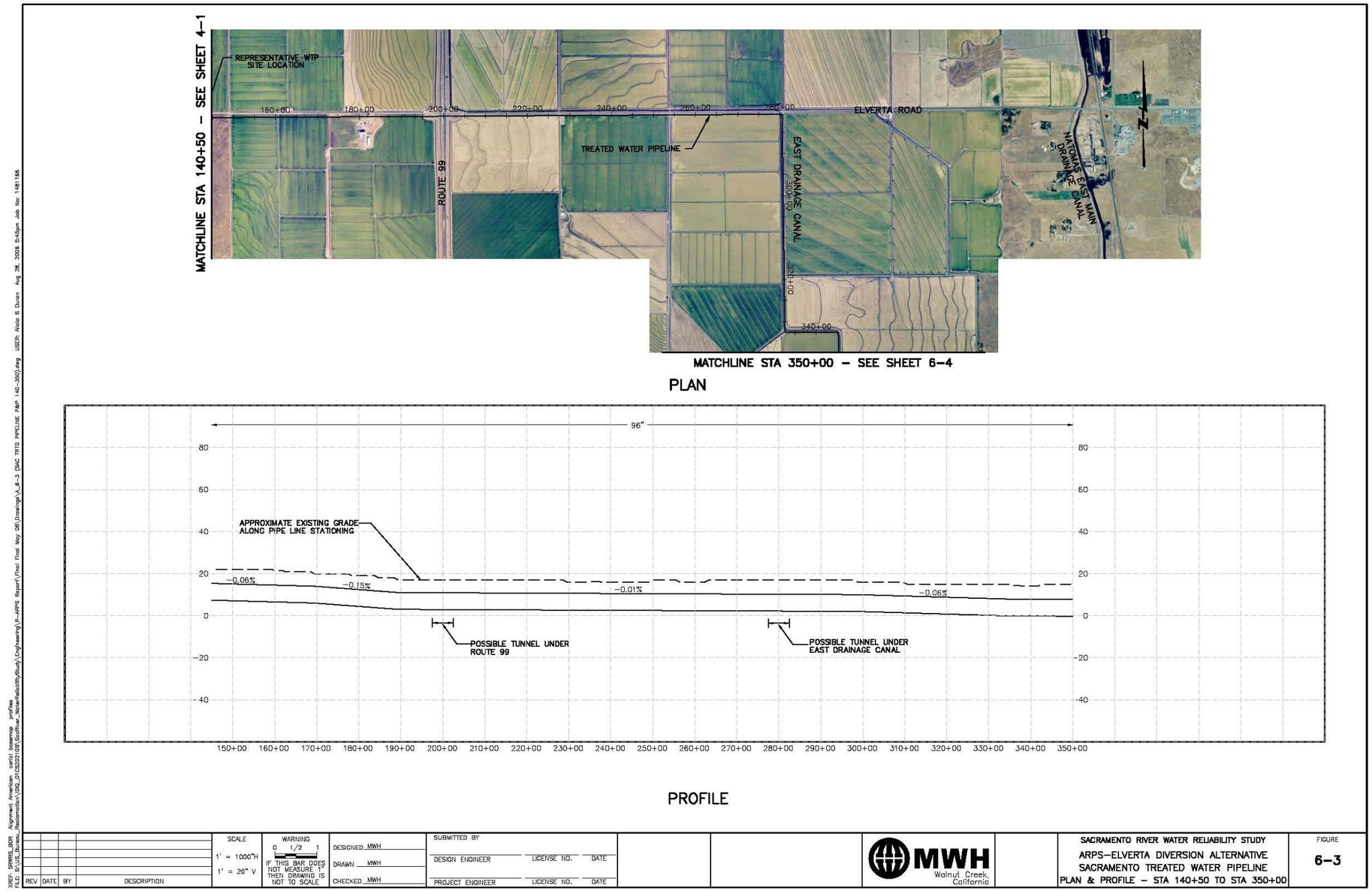

Figure 6-3 ARPS-Elverta Diversion Alternative Sacramento Treated Water Pipeline Plan & Profile – STA 140+50 to STA 350+00

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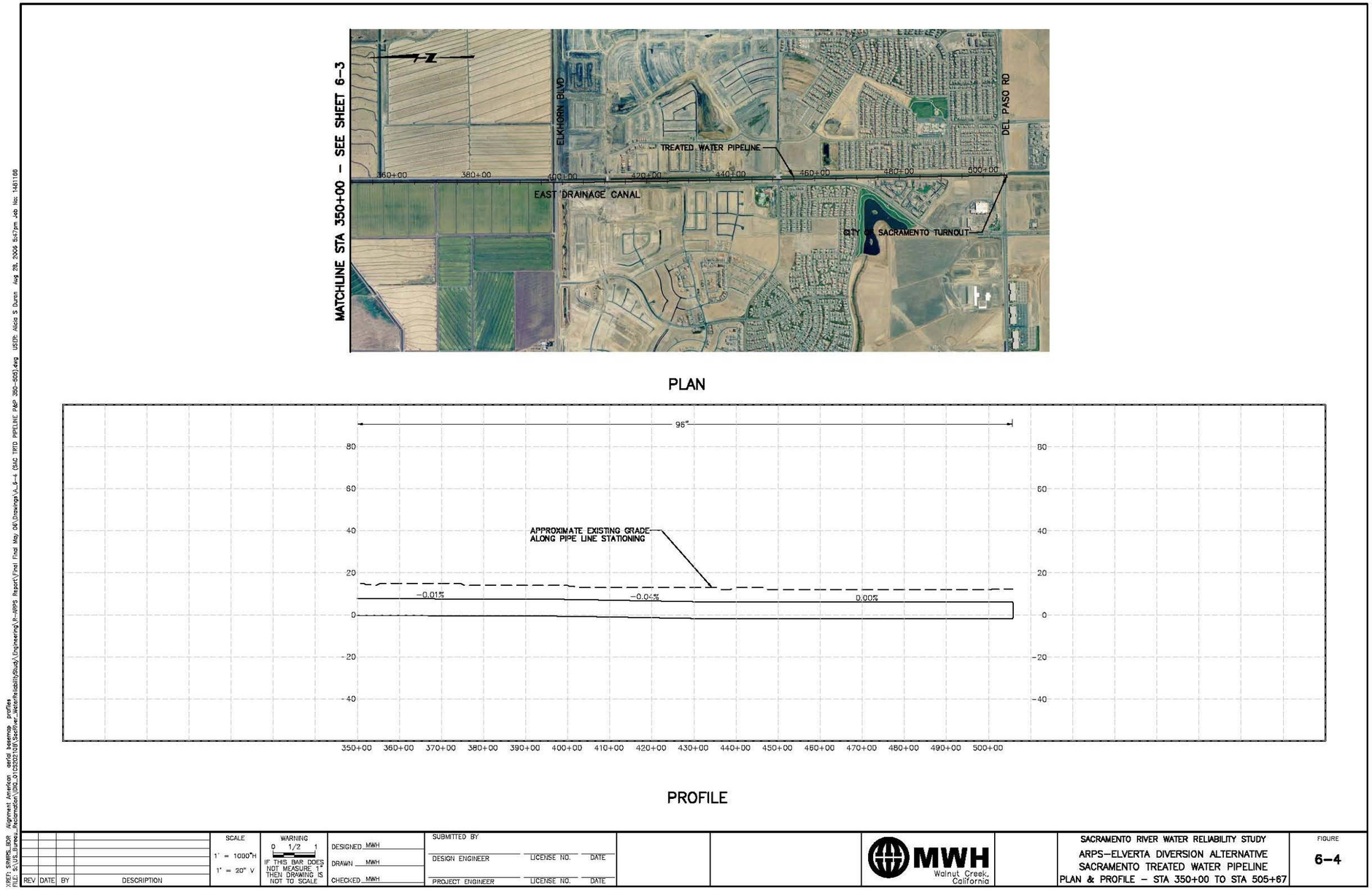

Figure 6-4 ARPS-Elverta Diversion Alternative Sacramento Treated Water Pipeline Plan & Profile – STA 350+00 to STA 505+67

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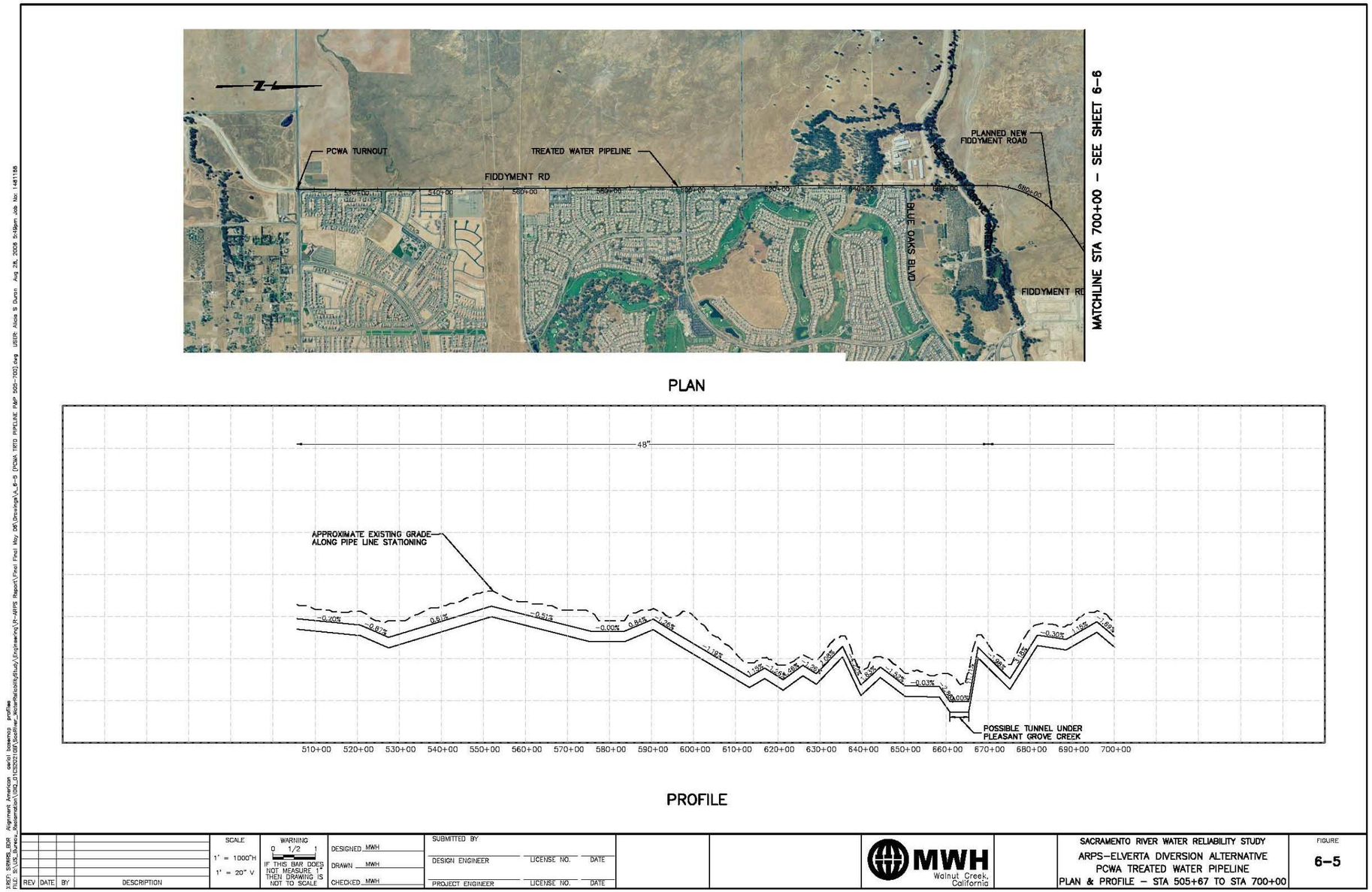

Figure 6-5 ARPS-Elverta Diversion Alternative PCWA Treated Water Pipeline Plan & Profile – STA 505+67 to STA 700+00

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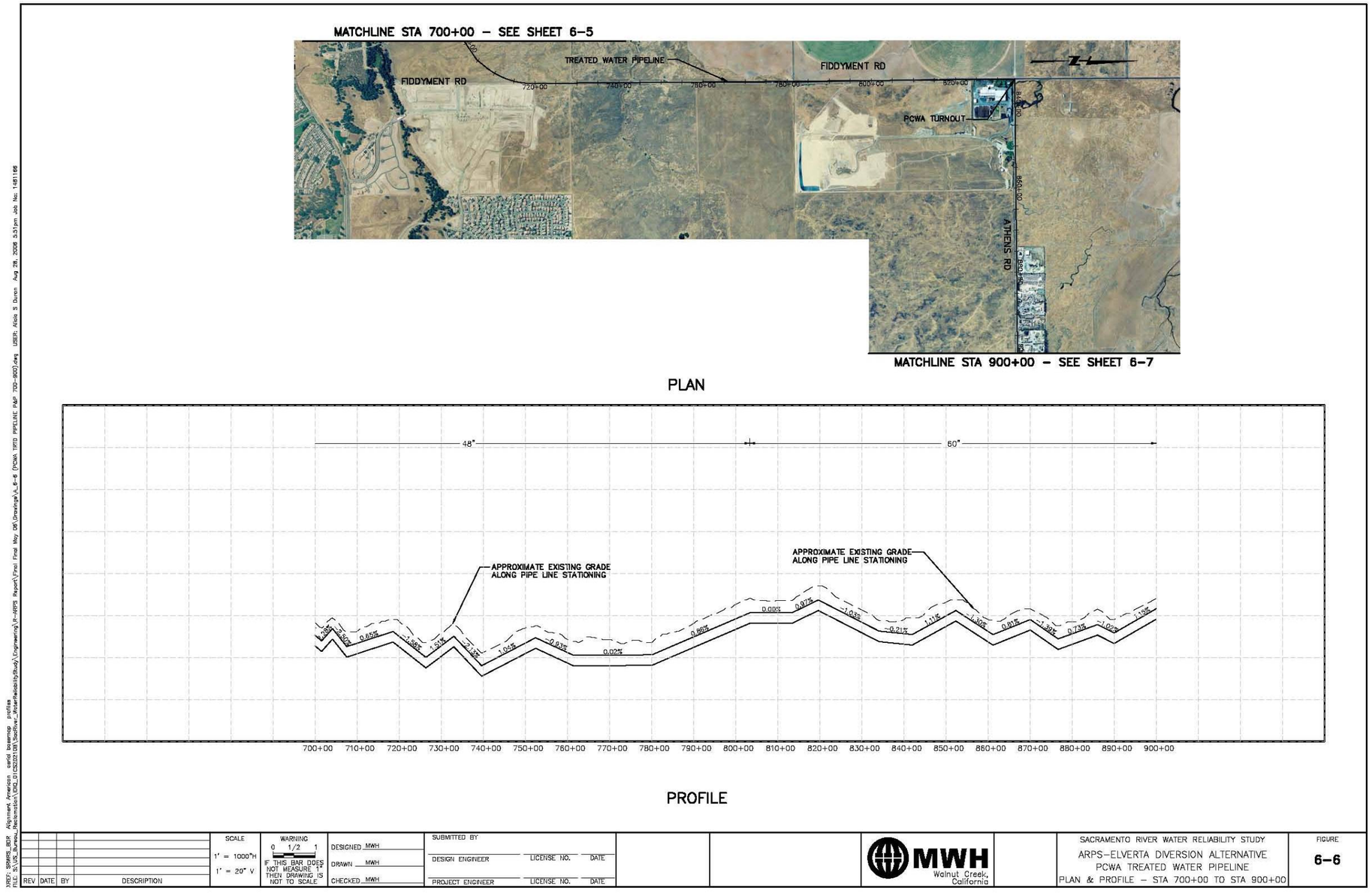

Figure 6-6 ARPS-Elverta Diversion Alternative PCWA Treated Water Pipeline Plan & Profile – STA 700+00 to STA 900+00

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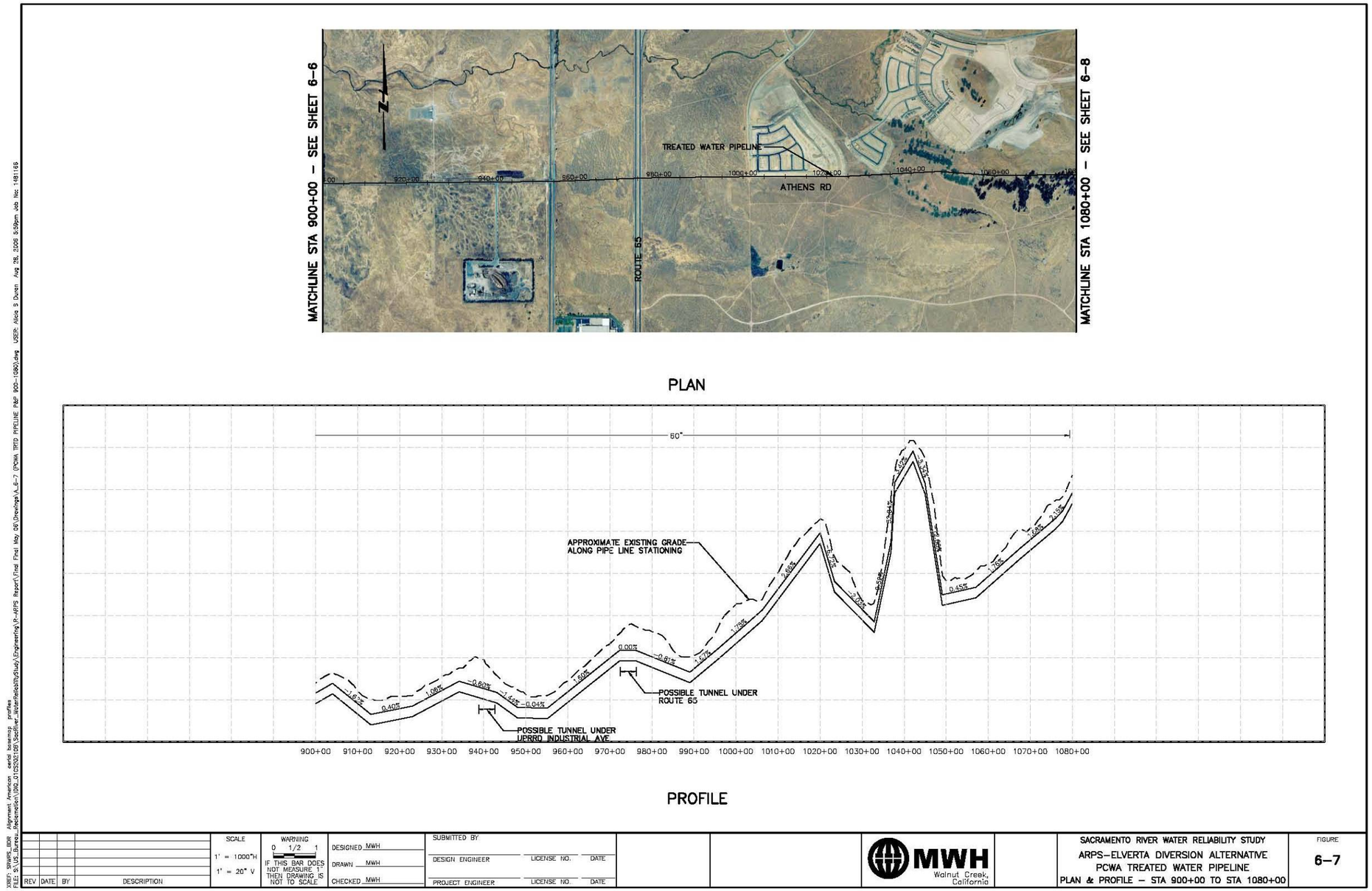

Figure 6-7 ARPS-Elverta Diversion Alternative PCWA Treated Water Pipeline Plan & Profile – STA 900+00 to STA 1080+00

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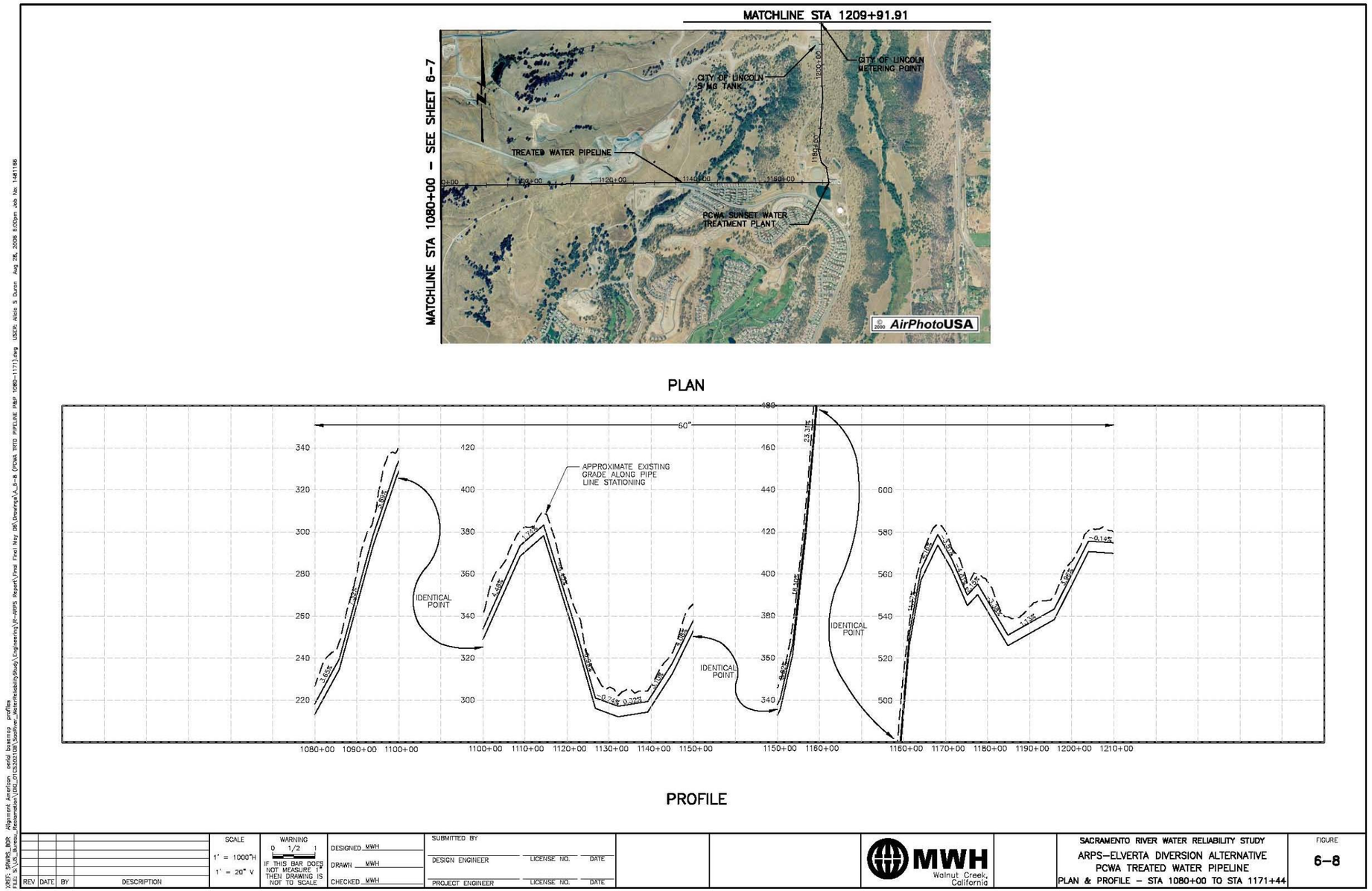

Figure 6-8 ARPS-Elverta Diversion Alternative PCWA Treated Water Pipeline Plan & Profile – STA 1080+00 to STA 1171+44

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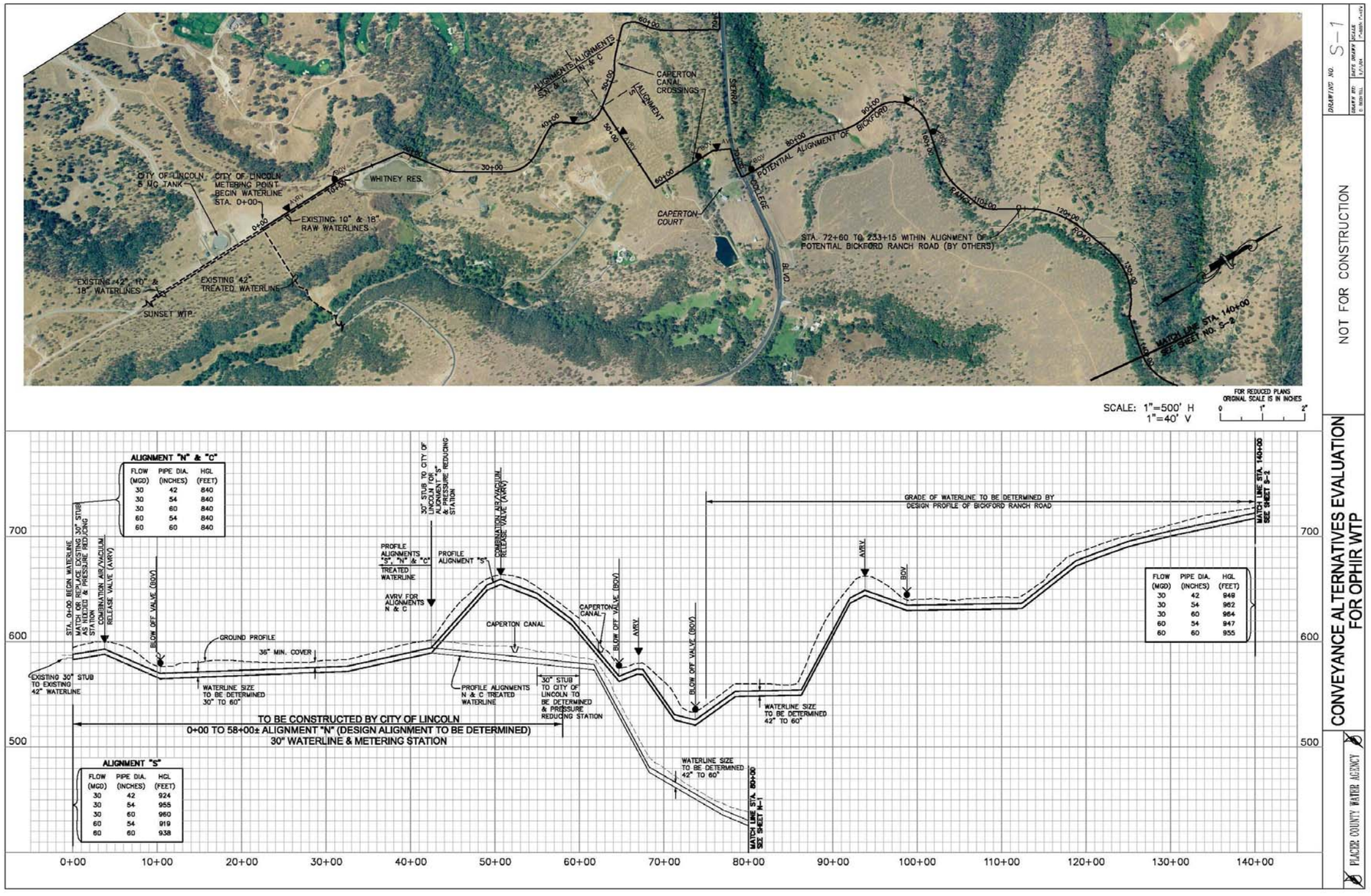

Figure 6-9 Conveyance Alternatives Evaluation for Ophir WTP STA 0+00 to STA 140+00

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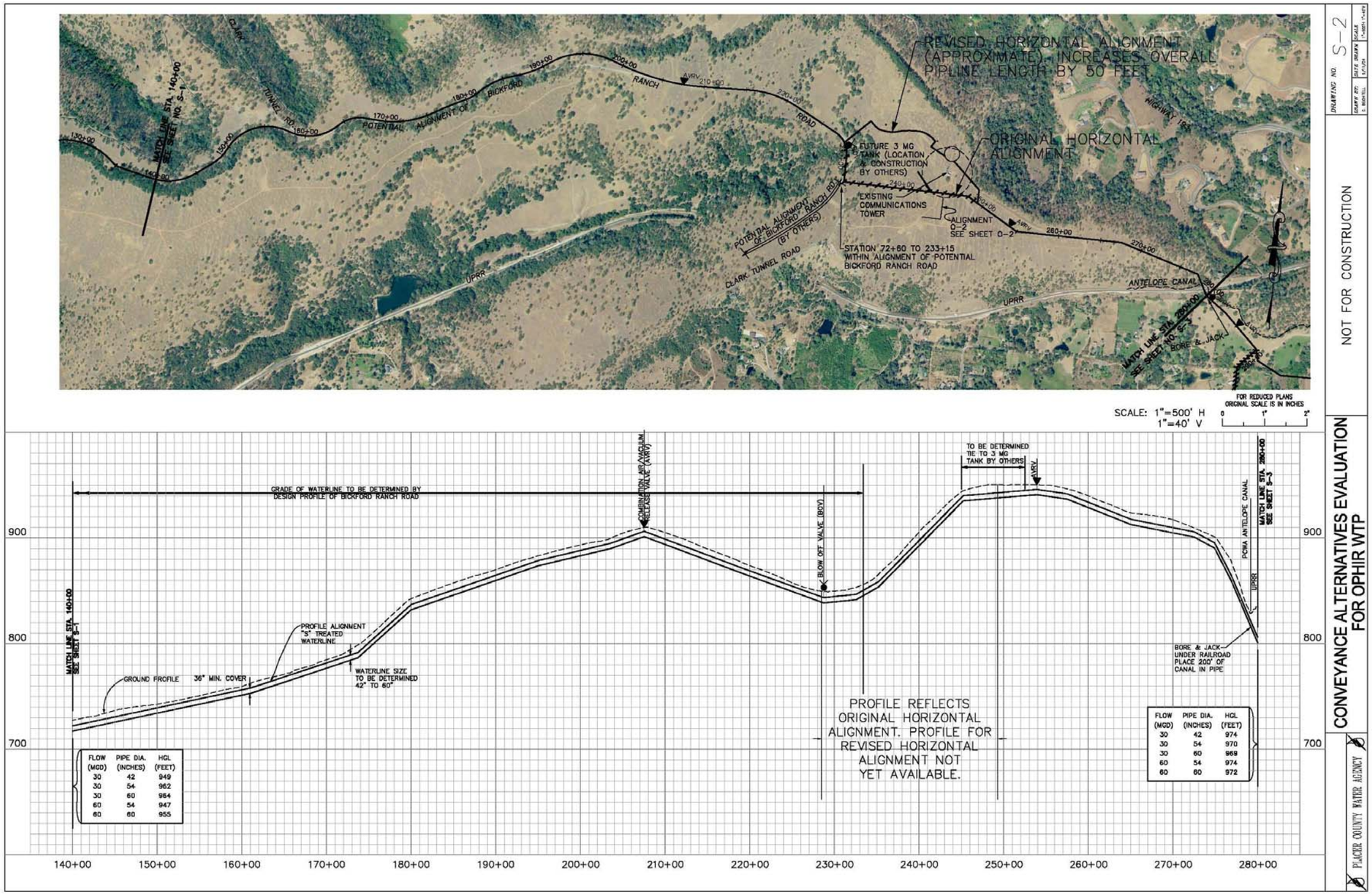

Figure 6-10 Conveyance Alternatives Evaluation for Ophir WTP STA 140+00 to STA 280+00

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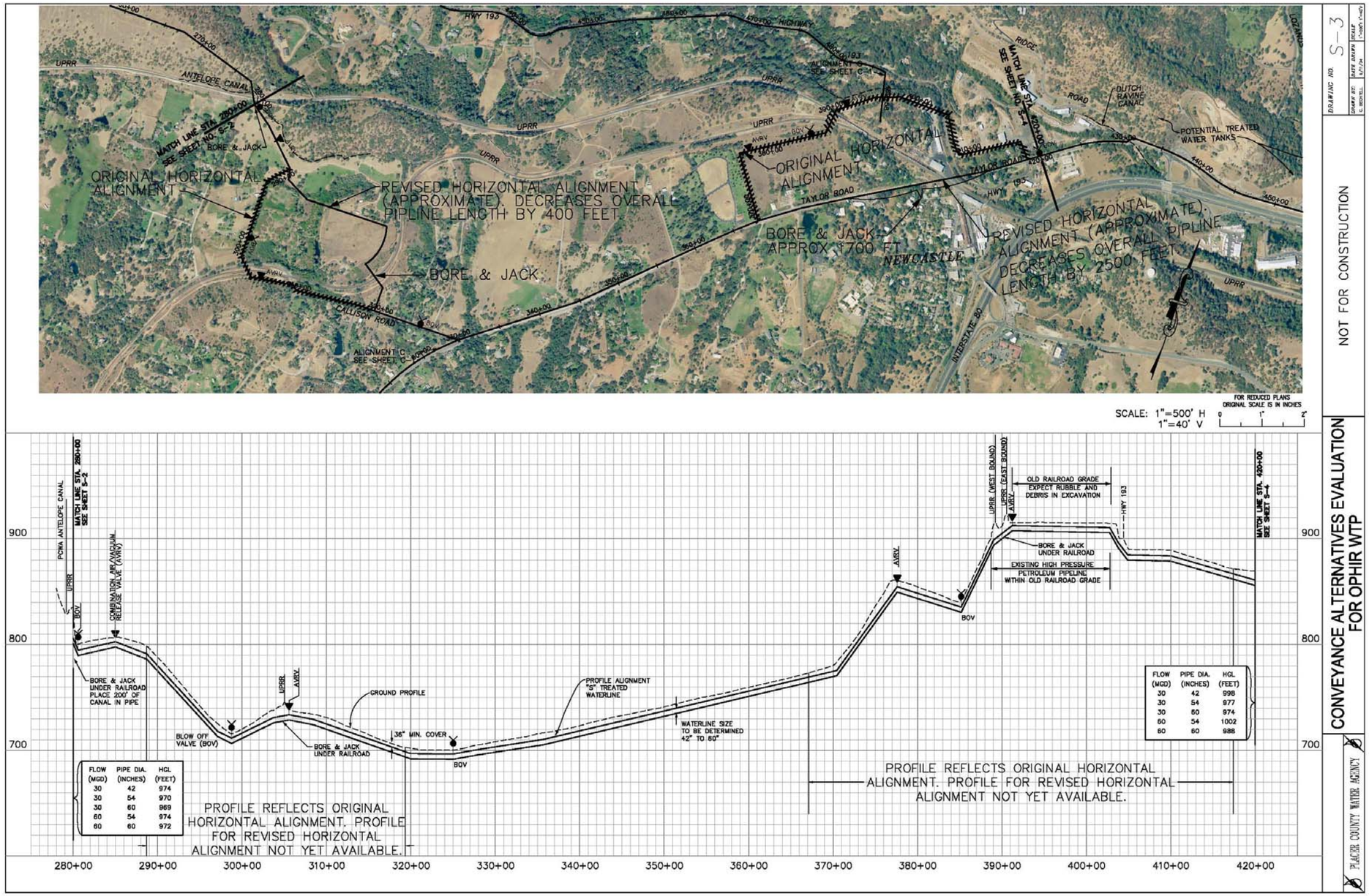

Figure 6-11 Conveyance Alternatives Evaluation for Ophir WTP STA 280+00 to STA 420+00

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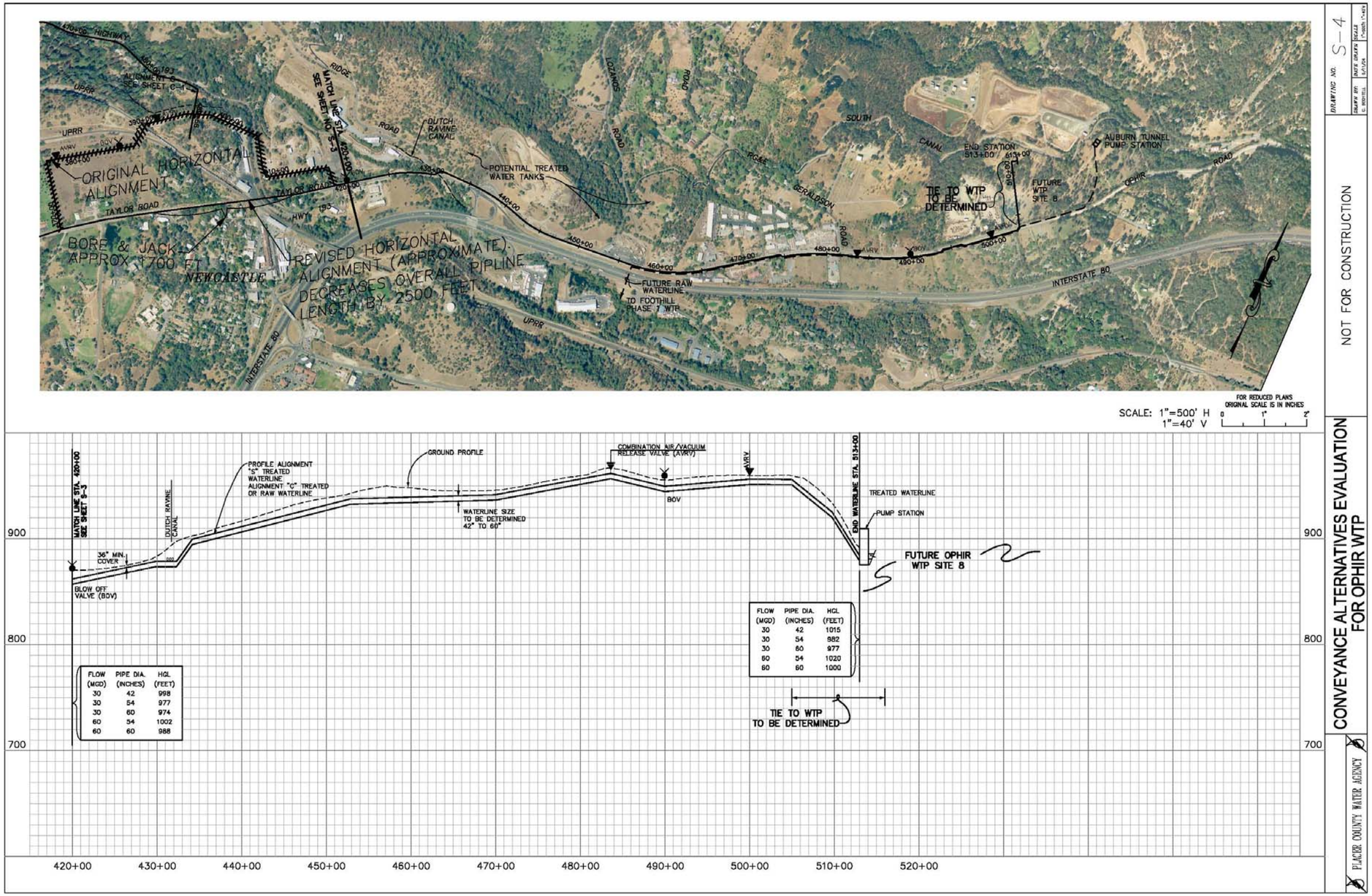

Figure 6-12 Conveyance Alternatives Evaluation for Ophir WTP STA 420+00 to STA 520+00

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CHAPTER 7 COST ESTIMATE

A feasibility-level cost estimate has been developed for the ARPS-Elverta Diversion Alternative. The cost estimate has been separated into five main sections: Intake Improvements, Raw Water Conveyance, Water Treatment, Treated Water Conveyance, and Groundwater Usage. An additional section for the ARPS-Joint Sacramento-ABFSHIP Elverta Diversion subalternative has been provided. To simplify evaluation and comparison, quantities used in the cost estimate assume that the WTP would be constructed on Elverta Road at a site located near the middle of the potential sites (approximately 2.6 miles from the intake). Final selection of the preferred WTP site would not occur until the next phase of the project; however, cost differences between the three potential sites are not expected to vary significantly given this report's scope and level of detail. Costs for easements and land purchases, as well as future advanced oxidation processes, have not been included in the estimate.

Costs have been developed for a current cost basis and then escalated to the estimated midpoint of construction in 2012. Costs for the proposed intake have been determined by starting with actual costs for the recently constructed Sacramento River WTP intake and modifying these costs to reflect an increased flow rate, a slightly longer bridge, and a reduced architectural effort. Raw and treated water pipeline costs have been developed using current pipeline construction pricing and incorporating additional project-specific costs for such elements as tunnel crossings, high groundwater conditions, and rock conditions. Water treatment costs are based on current and historical WTP construction costs using a cost-per-mgd basis and including additional site-specific costs such as foundation piles. The cost for groundwater usage by Roseville has been based on similar groundwater extraction and pumping facilities designed for Roseville.

Total costs for engineering, environmental, administration, and legal services have been estimated at 30 percent of construction costs. In addition, a 20 percent estimating contingency has been included.

Cost estimate detail is included in **Table 7-1**. The total escalated project cost, excluding NMWC, has been estimated at $1.104 billion. Including NMWC adds $43 million, for a total project cost of $1.147 billion. A distribution of costs per cost-sharing partner, based only on percentage of flow capacity, has also been included for reference.

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Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Base Scope								
1		**INTAKE IMPROVEMENTS**						
	A	***Sacramento River Intake***						
	1)	Sacramento River Intake Structure - CIP	MGD	145	$ 164,000	$ 23,780		Factored from Sac River WTP SOV - Balfour Beatty
	2)	Mobe/Demobe & Contr General Conds	MGD	145	$ 18,000	$ 2,610		Factored from Sac River WTP SOV - Balfour Beatty
	3)	Allowances	LS	1	$ -	$ -		Included
	4)	Temporary Controls	LS	1	$ -	$ -		Included
	5)	Demonstration & Training	LS	1	$ -	$ -		Included
						Item Total:	**$ 26,400**	Cost Basis 7/06
	B	***American River Intake***						
	1)	PCWA Intake Mods	LS	1	$ 121,000	$ 121		Remove Blank Plates at (E) Fish Screens as required
	2)	Increase Pumping Capacity at ARPS	HP	3515	$ 610	$ 2,144		Pump Upgrades (5) / Mech Piping / Electrical
	3)	Aurburn Tunnel Pump Station No. 3	HP	4372	$ 1,800	$ 7,870		Historical Pricing
	4)	VT Shaft Development Costs	LF	1000	$ 4,200	$ 4,200		5-200' Deep Shafts in Bedrock - 10' Dia
	5)	Mobe/Demobe & Contr General Conds	LS	1	$ 1,150,000	$ 1,150		at 10%
	6)	Allowances	LS	1	$ 1,212,000	$ 1,212		Environmental or Geotechnical Related
	7)	Temporary Controls	LS	1	$ 606,000	$ 606		Environmental or Geotechnical Related
	8)	Demonstration & Training	LS	1	$ 121,000	$ 121		Startup/Testing/Training
						Item Total:	**$ 17,400**	Cost Basis 7/06
						Total:	**$ 43,800**	

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
2		**RAW WATER CONVEYANCE**						
	A	***Sacramento River RW Pipeline to N. Natomoas WTP***						
	1)	C200 - Dual 66" Pipelines at Levee Rd	LF	1500	$ 1,400	$ 2,100		Next to Levee, Placed In Fill, 4' Cover
	2)	Garden Hwy/Leeve Improvements	LS	1	$ 606,000	$ 606		Raise Grade/New Paving Section
	2)	Garden Hwy/Levee/Canal Microtunnel	LF	600	$ 1,900	$ 1,140		Bore & Jack As Alternative - Dual 84"
	3)	Jacobs Slough Microtunnel - PBT	LF	500	$ 1,900	$ 950		Stream Crossing
	4)	C200 - Dual 66" Pipelines at Elverta Rd.	LF	13400	$ 1,400	$ 18,760		North Side of Elverta Rd., In ROW, High GW
	5)	Cathodic Protection	LF	13400	$ 20	$ 268		Allowance
	6)	Isolation Valves	EA	27	$ 36,400	$ 983		@ 1,000'
	7)	Air Release / Blowoff Valves	EA	5	$ 12,100	$ 61		@ High/Low Points
	8)	Access Manways	EA	27	$ 12,100	$ 327		@ 1,000'
	9)	Increased Dewatering Allowance	LF	13400	$ 60	$ 804		Well Point System, See LNWI Porgram
	10)	Increased Shoring Allowance - Sheet Piles	LF	13400	$ 600	$ 8,040		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	11)	Increased Traffic Controls - Elverta Rd.	LF	13400	$ 20	$ 268		One Lane w/ Flagmen Control
	12)	Pavement Restoration - Elverta Rd.	LF	6700	$ 60	$ 402		Assume 50% of alingment on Elverta Rd.
	13)	Mobe/Demobe & Contr General Conds	LS	1	$ 2,430,000	$ 2,430		Included at 8% of Direct Costs
	14)	Allowances	LS	1	$ 1,212,000	$ 1,212		Unknown Geology/Utilities
	15)	Temporary Controls	LS	1	$ 606,000	$ 606		Unknown Flood Controls/Envir Protections
	16)	Demonstration/Training/Cleanup	LS	1	$ 182,000	$ 182		Includes Startup & Testing Program
						Item Total:	**$ 39,100**	Cost Basis 7/06
	B	***American River RW Pipeline To Ophir WTP***						
	1)	C200 - Sgl 54" Pipeline	LF	1000	$ 650	$ 650		From ARPS No. 3 To Ophir WTP
	2)	Increase for Rock Conditions	LF	1000	$ 120	$ 120		Blast Trench - Granite
	3)	Cathodic Protection	LF	1000	$ 20	$ 20		Allowance
	4)	Isolation Valves	EA	2	$ 36,400	$ 73		@ 1,000'
	5)	Air Release / Blowoff Valves	EA	1	$ 12,100	$ 12		@ High/Low Points
	6)	Access Manways	EA	1	$ 12,100	$ 12		@ 1,000'
	7)	Mobe/Demobe & Contr General Conds	LS	1	$ 71,000	$ 71		Dewatering/Environmental
	8)	Allowances	LS	1	$ 606,000	$ 606		Unknown Geology/Utilities
	9)	Temporary Controls	LS	1	$ 303,000	$ 303		Unknown Flood Controls/Envir Protections
	10)	Demonstration/Training/Cleanup	LS	1	$ 121,200	$ 121		Includes Startup & Testing Program
						Item Total:	**$ 2,000**	Cost Basis 7/06
						Total:	**$ 41,100**	

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
3		**WATER TREATMENT**						
	A	***N. Natomas Water Treatment Plant***						Conventional Filtration w/ Chlorine Disinfection
	1)	Plant Construction - CIP	MGD	145	$ 970,000	$ 140,650		Historical Pricing
	2)	Booster Pump Station	HP	6400	$ 1,600	$ 10,240		Historical Pricing
	3)	Increase for Foundation System	LS	1	$ 9,700,000	$ 9,700		Dirven Conc Piles at 10' spacing 60' Deep,
	3)	Mobe/Demobe & Contr General Conds	LS	1	$ -	$ -		Included
	4)	Allowances	LS	1	$ 1,212,000	$ 1,212		Dewatering/Environmental
	5)	Temporary Controls	LS	1	$ 1,212,000	$ 1,212		Environmental
	6)	Demonstration/Training/Cleanup	LS	1	$ -	$ -		Included
						Item Total:	**$ 163,000**	Cost Basis 7/06
	B	***Expansion of Ophir WTP***						Conventional Filtration w/ Chlorine Disinfection
	1)	Plant Construction - CIP	MGD	65	$ 909,000	$ 59,085		Historical Pricing
	2)	Booster Pump Station	HP	1114	$ 1,600	$ 1,782		Historical Pricing
	3)	Mobe/Demobe & Contr General Conds	LS	1				Included
	4)	Allowances	LS	1	$ 1,212,000	$ 1,212		Dewatering/Environmental
	5)	Temporary Controls	LS	1	$ 606,000	$ 606		Environmental
	6)	Demonstration/Training/Cleanup	LS	1	$ 121,000	$ 121		Included
						Item Total:	**$ 62,800**	Cost Basis 7/06
						Total:	**$ 225,800**	
4		**TREATED WATER CONVEYANCE**						
	A	***City of Sacramento Pipeline***						
	1)	C200 96" From WTP to COS T/O	LF	36000	$ 1,000	$ 36,000		8 Miles, Elverta Rd to EDC.
	2)	Microtunnel - Major (2 Ea)	LF	600	$ 1,500	$ 900		EDC, Elverta Rd
	3)	Microtunnel - Average (4 Ea)	LF	800	$ 1,000	$ 800		Surface Streets at 500'
	4)	Cathodic Protection	LF	36000	$ 20	$ 720		Allowance
	5)	Isolation Valves	EA	42	$ 60,600	$ 2,545		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	8	$ 12,100	$ 97		@ High/Low Points
	7)	Access Manways	EA	42	$ 12,100	$ 508		@ 1,000'
	8)	Increased Dewatering Allowance	LF	36000	$ 50	$ 1,800		Well Point System, See LNWI Porgram
	9)	Increased Shoring Allowance - Sheet Piles	LF	36000	$ 600	$ 21,600		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls - Elverta Rd.	LF	20000	$ 20	$ 400		One Lane w/ Flagmen Control
	11)	Pavement Restoration - Elverta Rd.	LF	10000	$ 60	$ 600		Assume 50% of alingment on Elverta Rd.
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 5,300,000	$ 5,300		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 1,212,000	$ 1,212		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 606,000	$ 606		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 182,000	$ 182		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	1	$ 303,000	$ 303		End Point Allowance
						Item Total:	**$ 73,600**	Cost Basis 7/06

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
	B	***PCWA Pipeline From Ophir WTP To Sunset Res.***						
	1)	60" Waterline (Minimal Rock)	LF	11300	$ 500	$ 5,650		See Botorff Report and revisions
	2)	60" Waterline (Lava Cap Trenching)	LF	25000	$ 600	$ 15,000		"
	3)	60" Waterline (100% Rock)	LF	3000	$ 800	$ 2,400		"
	4)	60" Waterline (35% Rock)	LF	13000	$ 600	$ 7,800		"
	5)	Clearing	LS	1	$ 242,400	$ 242		"
	6)	Shoring	LS	1	$ 133,300	$ 133		"
	7)	Butterfly Valves + Couplings	EA	28	$ 92,100	$ 2,579		"
	8)	Fittings (Angles)	LS	1	$ 30	$ 0		"
	9)	Hydrants	EA	20	$ 9,100	$ 182		"
	10)	Air/Vaccum Release	EA	28	$ 9,700	$ 272		"
	110	Blow Off Valves	EA	28	$ 6,100	$ 171		"
	12)	Manholes	EA	28	$ 7,000	$ 196		"
	13)	Pressure Reducing Station	EA	1	$ 139,400	$ 139		"
	14)	30" Stub to Lincoln	LS	1	$ 36,400	$ 36		"
	15)	Trench Dewatering	LF	26000	$ 10	$ 260		"
	16)	Dutch Ravine Canal Crossing	EA	1	$ 48,500	$ 49		"
	17)	Antelope Canal Pipe	LF	200	$ 200	$ 40		"
	18)	Corrosion Monitoring	LS	1	$ 242,400	$ 242		"
	19)	Testing/Disinfection	LS	1	$ 109,100	$ 109		"
	20)	Erosion Control	LS	1	$ 93,340	$ 93		"
	21)	Taylor Road/Ophir Road Concrete Repair	LF	6000	$ 100	$ 600		"
	22)	Other Pavement Replacement	LF	5000	$ 30	$ 150		"
	23)	Misc. Driveway & Fence Repair	LS	1	$ 72,700	$ 73		"
	24)	Surveying	LS	1	$ 97,000	$ 97		"
	25)	Traffic Controls	LS	1	$ 84,900	$ 85		"
	26)	Bore Pits	EA	3	$ 80,000	$ 240		"
	27)	Sta 280+00 RR Bore & Jack	LS	1	$ 327,300	$ 327		"
	28)	Sta 305+00 RR Bore & Jack (+/- rev location)	LS	1	$ 169,700	$ 170		"
	29)	Sta 390+00 RR Bore & Jack (+/- rev location)	LS	1	$ 1,090,900	$ 1,091		"
	30)	Mobe/Demobe & Contr General Conds	LS	1	$ 2,660,000	$ 2,660		"
						Item Total:	**$ 41,100**	Cost Basis 7/06

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
	C)	***PCWA Pipeline From Sunset Res To Fiddyment/Athens***						
	1)	C200 60"	LF	34000	$ 700	$ 23,800		3.7 Miles, Baseline Rd.
	2)	Microtunnel - Major (2 Ea)	LF	1000	$ 1,500	$ 1,500		Hwy 99, RR
	3)	Microtunnel - Average (4 Ea)	LF	1000	$ 1,000	$ 1,000		Minor Surface Streets
	4)	Cathodic Protection	LF	34000	$ 20	$ 680		Allowance
	5)	Isolation Valves	EA	34	$ 60,600	$ 2,060		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	6	$ 12,100	$ 73		@ High/Low Points
	7)	Access Manways	EA	34	$ 12,100	$ 411		@ 1,000'
	8)	Increase For Rock Conditions	LF	20000	$ 40	$ 800		Well Point System, See LNWI Porgram
	9)	Increased Shoring Allowance - Sheet Piles	LF	14000	$ -	$ -		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increase For Trench Dewatering	LF	14000	$ -	$ -		Not Required
	11)	Increased Traffic Controls	LF	34000	$ -	$ -		One Lane w/ Flagmen Control
	12)	Pavement Restoration - Athens/Fiddyment	LF	34000	$ 100	$ 3,400		Assume 50% of alingment on Elverta Rd.
	13)	Mobe/Demobe & Contr General Conds	LS	1	$ 2,698,000	$ 2,698		Included at 8% of Direct Costs
	14)	Allowances	LS	1	$ 1,212,000	$ 1,212		Unknown Geology/Utilities
	15)	Temporary Controls	LS	1	$ 606,000	$ 606		Unknown Flood Controls/Envir Protections
	16)	Demonstration/Training/Cleanup	LS	1	$ 121,000	$ 121		Includes Startup & Testing Program
	17)	Turnout/Distribution Facilities	LS	1	$ 242,000	$ 242		End Point Allowance
						Item Total:	**$ 38,600**	Cost Basis 7/06
	D	***PCWA Pipeline Along Fiddyment From Athens Intersection***						
	1)	C200 48"	LF	35000	$ 500	$ 17,500		3.7 Miles, Baseline Rd.
	2)	Microtunnel - Major (2 Ea)	LF	600	$ 1,500	$ 900		Hwy 65, Caperton Canal
	3)	Microtunnel - Average (4 Ea)	LF	1000	$ 1,000	$ 1,000		Minor Surface Streets
	4)	Cathodic Protection	LF	35000	$ 20	$ 700		Allowance
	5)	Isolation Valves	EA	20	$ 60,600	$ 1,212		@ 1,000'
	6)	Air Release / Blowoff Valves	EA	4	$ 12,100	$ 48		@ High/Low Points
	7)	Access Manways	EA	20	$ 12,100	$ 242		@ 1,000'
	8)	Increased Dewatering Allowance	LF	35000	$ -	$ -		Well Point System, See LNWI Porgram
	9)	Increased Shoring Allowance - Sheet Piles	LF	35000	$ -	$ -		Drive/Pull Sheet Piles 25' Driving Depth, 75% Reuse
	10)	Increased Traffic Controls	LF	35000	$ -	$ -		One Lane w/ Flagmen Control
	11)	Pavement Restoration - Fiddyment Rd.	LF	35000	$ 100	$ 3,500		Assume 50% of alingment on Elverta Rd.
	12)	Mobe/Demobe & Contr General Conds	LS	1	$ 2,008,000	$ 2,008		Included at 8% of Direct Costs
	13)	Allowances	LS	1	$ 1,212,000	$ 1,212		Unknown Geology/Utilities
	14)	Temporary Controls	LS	1	$ 606,000	$ 606		Unknown Flood Controls/Envir Protections
	15)	Demonstration/Training/Cleanup	LS	1	$ 121,000	$ 121		Includes Startup & Testing Program
	16)	Turnout/Distribution Facilities	LS	1	$ 242,400	$ 242		End Point Allowance
						Item Total:	**$ 29,300**	Cost Basis 7/06
						Total:	**$ 182,600**	

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
5		**GROUND WATER RECOVERY**						
	A	***City Of Roseville Production Wells***						
	1)	Scope TBD	EA	4	$ 1,400,000	$ 5,600		Historical Pricing
						Item Total:	**$ 5,600**	Cost Basis 7/06
Base Scope Cost Summary								
	a)	Item/Task Direct Costs					**$ 498,900**	Items 1-5
	b)	Escalation to Mid Point of Constructon					**$ 208,800**	n=6 or 2012 at 6.0%/Year
		Sub Total (Direct Costs):					**$ 707,700**	
	c)	Engineering/Environmental/Administration	30%				**212,300**	Allowance
		Legal Services Allowance						
		Sub Total (Direct/Indirect Costs):					**$ 920,000**	
	d)	Program Contingency	20%				**184,000**	
		Engineers' Opinion of ARPS-Elverta Diversion Alternative Total Capital Costs:					**$ 1,104,000**	Cost Escalated to 7/12, Midpoint of Construction

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Additive Alternative No. 1			MGD	136				
		AARS - Joint Sacramento ABFSHIP Elveta Diversion Alternative						
		Scope: Provide Additional Raw Water Intake Pumping Capacity and Canal Improvements						
	1)	Additional Intake Capacity at Sac River	MGD	136	$ 121,000	$ 16,456		Use 2/3's of Item's 1 Costs on a MGD Basis
	2)	Energy Dissipation Structure	LS	1	$ 420,000	$ 420		Includes Valves/Slide Gates
	3)	Relovate Canal	MI	1.6	$ 680,000	$ 1,088		Widen/Grade Existing Canal, Factored at $2/SY
	5)	Mobe/Demobe & Contr General Conds	LS	1	$ 1,437,000	$ 1,437		Factored at 5% of Totals
	6)	Allowances	LS	1	$ -	$ -		Included
	7)	Temporary Controls	LS	1	$ -	$ -		Included
	8)	Demonstration & Training	LS	1	$ 60,610	$ 61		Allowance
							$ 19,500	Cost Basis 7/06
Alternative Summary								
	a)	Item/Task Direct Costs					**$ 19,500**	
	b)	Escalation to Mid Point of Constructon					**$ 8,200**	n=6 or 2012 at 6.0%/Year
		Sub Total (Direct Costs):					**$ 27,700**	
	c)	Engineering/Environmental/Administration Legal Services Allowance	30%				**8,300**	
		Sub Total (Direct/Indirect Costs):					**$ 36,000**	
	d)	Program Contingency	20%				**7,200**	
	Engineers' Opinion of ARPS-Joint Sacramnto-ABFSHIP Elverta Diversion Alternative Additive Capital Costs:						**$ 43,000**	Cost Escalated to 7/12, Midpoint of Construction

Table 7-1 American River Pump Station Alternative
Engineer's Opinion of Probable Capital Costs – Feasibility-Level (cont.)

Item	Task	Description	UOM	Quantity	Unit Price	Total Cost (x $1,000)	Total (x $1,000)	Comments
Partner Share Summary								
	a)	City of Sacramento (COS)					$ 668,500	
	b)	Placer County Water Agency (PCWA)					$ 423,100	
	c)	Sacramento Suburban Water District (SSWD)					$ -	
	d)	City of Roseville (COR)					$ 12,400	
							$ 1,104,000	Cost Escalated to 7/12, Midpoint of Construction
	e)	Natomas Mutual Water Company (NMWC)					$ 43,000	
		Engineers' Opinion of ARPS-Joint Sacramnto-ABFSHIP Elverta Diversion Alternative Total Capital Costs:					$ 1,147,000	Cost Escalated to 7/12, Midpoint of Construction

Notes/Comments

1) Cost Basis = 7/06
2) Costs for easements, land purchase, or future advanced oxidation processess have been excluded.
3) Includes COS Fluiridation Dosing Station
4) Pricing predicated on competitive market conditions (+4 bidders/trade).
5) Pricing excludes any labor premium for overtime conditions.
6) Pricing excludes a change order contingency.
7) Pricing excludes an allowance for design/CM oversight or field inspections services.
8) Pricing excludes any financing or cost-of-money expenses.
9) Pricing excludes costs for obtaining required bldg permits or fees.

Key:

CIP – cast-in-place concrete
CM – Construction Management
COR – City of Roseville
COS – City of Sacramento
Demobe – demobilization
GW – groundwater
hp – horsepower
lf – linear foot
LNWI – Lower Northwest Interceptor
ls – lump sum
mgd – million gallons per day
mi – mile
Mobe – mobilization
NMWC – Natomas Mutual Water Company
PCWA – Placer County Water Agency
RM – River Mile
ROW – right-of-way
RR – railraod
SOV – Schedule of Values
SSWD – Sacramento Suburban Water District
UOM – unit of measurement
WTP – Water Treatment Plant

CHAPTER 8 PERMIT REQUIREMENTS

To comply with Federal, State, and local laws, ordinances, and regulations, the SRWRS would be required to conduct or obtain numerous investigations, consultations, and permits. A Permit Acquisition Plan has been developed, and submitted separately, which discusses the permits that would need to be obtained after certification and approval of the EIS/EIR for the project. Discussed herein are some highlights from this plan.

8.1. REGULATORY REQUIREMENTS

The specific regulatory requirements have been organized by the five facilities of the ARPS-Elverta Diversion Alternative: Elverta Intake, North Natomas WTP, ARPS, ATPS, Ophir WTP Expansion, and pipelines. **Tables 8-1** through **8-5** summarize the permits or consultations required for each facility. These tables identify the regulation, the permit or consultation required, the permitting agency and contact information, and include some general notes.

Each consultation and permit has specific submittal requirements, as identified herein, and therefore would have different timing requirements for initiation with the permitting agency as well as final application submittal. **Table 8-6** provides an overview of the type of documentation typically submitted with the major permit applications.

8.2. RECOMMENDED TIMING OF PERMIT ACQUISITION

Many Federal and State permitting agencies have mandated periods for responding to permit applications. Using the documentation from **Table 8-6**, mandated response periods, and historical experience in obtaining permits, the timing for permit initiation and application periods has been estimated and is presented below.

8.2.1. Work to Be Completed During Preliminary Design Phase of the Project

As part of the preliminary design phase of the work, consultation would be initiated with numerous permitting agencies to begin discussion of project-specific conditions and design criteria that would need to be included in the design of ultimate facilities in order to obtain permits from these agencies. These contacts would not result in permits, but rather would identify the conditions and requirements for permit applications to be submitted as part of the final design when more detailed engineering design is available. This would include coordination with the following agencies:

- USACE (Section 404/10 Permit)
- DHS (Water Supply Permit)
- California Department of Transportation (Encroachment Permit)
- The Reclamation Board (Encroachment Permit)
- CVRWQCB (National Pollutant Discharge Elimination System (NPDES) Permit)
- Sacramento and Placer Counties (Encroachment Permits)
- Cities of Sacramento and Roseville (Encroachment Permits)

In addition to these consultations, several other permits and consultations can be completed or obtained during this phase of work, including the following:

- USCG (Aid to Navigation)
- FAA/Sacramento County Airport Service (Form 7460-1)
- UPRR (Encroachment Permit)
- Cal-OSHA (Gas Classifications)
- SAFCA (Flood Impact Consult)
- Reclamation District 1000 (Flood Impact Consult)
- CSD-1/Sacramento County Department of Water Resources (Sewer/Storm Drain Connection)
- Sacramento County (General Use and Building Permits)

8.2.2. Work to Be Completed During Final Design

As part of the final design, permit applications would be prepared for the agencies that were only consulted during the enhanced engineering analysis. This would include coordination with the following:

- USACE (Section 404/10 Permit)
- DHS (Water Supply Permit)
- California Department of Transportation (Encroachment Permit)
- The Reclamation Board (Encroachment Permit)
- CVRWQCB (NPDES Permit)
- Sacramento and Placer Counties (Encroachment Permits)
- Cities of Sacramento and Roseville (Encroachment Permits)

In addition to the permits above, several other permits and consultations would be ready to be completed or obtained during the final design, including the following:

- CDFG (Streambed Alteration Agreement)
- California State Lands Commission (Letter for Avoid Land Use Lease)
- CVRWQCB (Section 401 Water Quality Certification)
- SWRCB (Notice of Intent (NOI) for Stormwater and Low Threat Discharges)
- Sacramento Metropolitan Air Quality Control District (Generator Permit)
- PCACD (Generator Permit)
- Sacramento County (Tree Removal Permit)
- Placer County (Tree Removal Permit)

Table 8-1 Elverta Intake Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
Federal Clean Water Act	Section 404 Individual Permit	U.S. Army Corps of Engineers	Mike Finnan	1325 J Street Sacramento, CA 95814-2922	(916) 557-5324	Need to conduct pre-application consultation and then complete and submit an Application for a Department of the Army Permit.
Rivers and Harbors Act	Section 10 Individual Permit	U.S. Army Corps of Engineers		1325 J Street Sacramento, CA 95814-2922		Need to conduct pre-application consultation and then complete and submit an Application for a Department of the Army Permit.
	Private Aid to Navigation	U.S. Coast Guard	Brian Aldridge	MSO San Francisco Bay Waterways Management Bldg. 14 Coast Guard Island Alameda, CA 94501-5100	(510) 437-2983	Need to submit application for temporary and permanent aids to navigation and provide notice in Local Notice to Mariners during construction.
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office-Margie Drilling	Air-traffic Division AWP 520 15000 Aviation Blvd. Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 -General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service.
STATE						
	Aids to Navigation	California Department of Boating and Waterways	Mike Sotelo	2000 Evergreen Street, Ste. 100 Sacramento, CA 95815-3888	(916) 263-0787	Need to ensure that USCG private aids to navigation also meet State standards.
Fish and Game Code	Streambed Alteration Agreement	California Department of Fish and Game		Region 2 1701 Nimbus Rd, Ste. A Rancho Cordova, CA 95670	(916) 358-2900/ (916) 445-0045	Need to submit application for streambed alteration.
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on design criteria, then amend water supply permits.
California Code of Regulations and Public Resources Code	Land Use Lease	California State Lands Commission	Lorna Burkes	Land Mgmt. Division 100 Howe Avenue, Ste. 100S Sacramento, CA 95825-8202	(916) 574-1900	Need to obtain land use lease for intake located within riverbed. Not required if obtaining permit from USACE or The Reclamation Board.
Clean Water Act	Section 401 Water Quality Certificate	Central Valley Regional Water Quality Control Board	Patrick Gillum	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709	Need to obtain The Water Quality Certification Waiver for USACE Permit.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No.200 Rancho Cordova, CA 95670-6114	(916) 464-4662	Need to obtain dewater permit for low-threat discharges for dewatering cofferdam at intake structure.
Clean Water Act	NPDES Permit	Central Valley Regional Water Quality Control Board	Jacque Kelley	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4764	May need to obtain permit to discharge stormwater to groundwater via detention basin or to surface water via discharge.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.
California Streets and Highways Code	Encroachment Easement	California Department of Transportation	Rich Jones	703 B Street P.O. Box 911 Marysville, CA 95901	(530) 741-5374	Need permit to cross and potentially redesign/realign the Garden Highway.
	Tunneling Permit – Gas Classification	California Occupational Safety and Health Administration	Gerald Fulhgrum	Cal-OSHA - Division of Mining and Tunneling 2211 Park Towne Circle Sacramento, CA 95825-0414	(916) 574-2540	Only required if tunnel through levee.
California Water Code	Encroachment Permit	The Reclamation Board	Stephen Bradley	Floodway Prot. Section 1416 9th Street, Rm 1623 Sacramento, CA 95814	(916) 574-0608/ (916) 574-0682	Need to submit application to encroach on floodway of the Central Valley.

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Table 8-1 Elverta Intake Permit Requirements (Cont.)

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
LOCAL						
	Review Impacts to Levees	Sacramento Area Flood Control Agency	Pete Ghelfi	1007 7th Street, 7th Floor Sacramento, CA 95814	(916) 874-7606/ (916) 874-8289	Need endorsement of Project by The Reclamation Board and USACE as well as coordinate with local projects for District 2.
	Review Impact to Levees	Reclamation District 1000	Jim Clifton	1633 Garden Highway Sacramento, CA 95833	(916) 922-1449 (916) 922-9173	Need Endorsement of Project by The Reclamation Board.
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Sacramento Metropolitan Air Quality Management District	Brian Krebbs	777 12th Street, 3rd Floor Sacramento, CA 95814	(916) 874-4800	Need to submit application for intake pumps and motors as well as standby generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Need to meet all safety requirements for future Approach/Departure Zone of SMF.
County Zoning Ordinance	Use Permit	Sacramento County Department of Planning and Community Development	Charlie Dyer	827 7th Street, Rm. 230 Sacramento, CA 95814	(916) 874-6221 (information) (916) 874-6141/ (916) 874-6400	Conduct pre-application consultation and then submit application.
	Review Impact to Garden Highway	Sacramento County Department of Transportation	LDSIR-Tech Resources: Norm Novak	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-6544 (Tech. Res.) (916) 874-6873	Need to coordinate with CalTrans.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section-Landscape Design and Tree Section: Henry Yasui	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-5278 Yasui Direct: (916) 874-8114/ (916) 874-1677	Need to submit application to removal riparian trees on riverside of levee.

Key:
Cal-OSHA – California Occupational Safety and Health Administration
DHS – Department of Health Services
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
SMF – Sacramento International Airport
USACE – United States Army Corps of Engineers
USG – United States coast Guard

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Table 8-2 North Natomas WTP Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office Margie Drilling	Air-traffic Division AWP 520 15000 Aviation Boulevard Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service if located within Overflight Zone of SMF.
STATE						
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on process selection and design criteria, then amend water supply permits.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709 (916) 464-4662	May need to obtain dewater permit for low-threat discharges for construction-related dewatering.
Clean Water Act	NPDES Permit	Central Valley Regional Water Quality Control Board	Jacque Kelley	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4764	May need to obtain permit to discharge stormwater to groundwater via detention basin or to surface water via discharge.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Sacramento Metropolitan Air Quality Management District		777 12th Street, 3rd Floor Sacramento, CA 95814	(916) 874-4800	Need to submit application for standby generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Coordinate with Sacramento County Airport Service to submit information to FAA if located within Overflight Zone of SMF.
County Zoning Ordinance	Use Permit	Sacramento County Department of Planning and Community Development		827 7th Street, Rm. 230 Sacramento, CA 95814	(916) 874-6221	Conduct pre-application consultation and then submit application.
	Building Permit	Sacramento County Department of Engineering and Administration	Bill Durkee	827 7th Street, Rm 304 Sacramento, CA 95814	(916) 874-1691/ (916) 874-7100 Durkee Direct: (916) 874-6521/ (916) 874-5919	Need to determine if Building Permit required for Operations Building.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-5278	Need to obtain permit to remove trees if necessary.
	Storm Drain System Connection	Sacramento County Department of Water Resources	Kerry Schmitz	827 7th Street, Rm. 301 Sacramento, CA 95814	(916) 874-6851	Consultation to determine if storm drain system can be extended for connection.
	Collection System Connection	County Sanitation District 1/ Sacramento Regional County Sanitation District		10545 Armstrong Avenue Mather, CA 95655	(916) 876-6000	Consultation to Expand Services to New WTP for Wastewater.
	Septic System Permit	Sacramento County Environmental Management Department	Steve Kalvelage	8475 Jackson Road, Ste. 240 Sacramento, CA	(916) 875-8484 Kalvelage Direct: (916) 875-8416/ (916) 875-8513	Consultation to install septic system if not able to extend SRCSD collection system.

Key:
DHS – Department of Health Services
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
SMF – Sacramento International Airport
SRCSD – Sacramento Regional County Sanitation District
WTP – Water Treatment Plant

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Table 8-3 American River Pump Station Intake Modification Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
STATE						
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus and then amend water supply permit.
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Placer County Air Pollution Control District	Zach Lee	11464 B Avenue DeWitt Center Auburn, CA 95603	(530) 889-7127 (530) 889-7107	Need to submit application for standby generator if used.

Key:
DHS – Department of Health Services

Table 8-4 Ophir WTP Expansion Project Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
STATE						
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus and then amend water supply permit.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4700 (916) 464-4662	May need to obtain dewater permit for low-threat discharges for construction-related dewatering.
Clean Water Act	NPDES Permit	Central Valley Regional Water Quality Control Board	Jacque Kelley	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4764	May need to amend permit to discharge stormwater to groundwater via detention basin or to surface water via discharge.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Placer County Air Pollution Control District	Zach Lee	11464 B Avenue DeWitt Center Auburn, CA 95603	(530) 889-7127 (530) 889-7107	Need to submit application for standby generator if used.
	Tree Permit	Placer County Planning Department		11464 B Avenue DeWitt Center Auburn, CA 95603	(530) 886-3000	

Key:
DHS – Department of Health Services
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
WTP – Water Treatment Plant

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Table 8-5 Pipeline Permit Requirements

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
FEDERAL						
Federal Clean Water Act	Section 404 Individual Permit	U.S. Army Corps of Engineers	Mike Finnan	1325 J Street Sacramento, CA 95814-2922	(916) 557-5324	Need to conduct pre-application consultation and then complete and submit an Application of a Department of the Army Permit for Dry Creek and Pleasant Grove Creek crossings.
	Consultation for Airport Impacts	Federal Aviation Administration	Western Pacific Regional Office Margie Drilling	Air-traffic Division AWP 520 15000 Aviation Boulevard Hawthorne, CA 90260	(310) 725-3618 or (310) 725-3608 General line	Need to submit Form 7460-1 to FAA in conjunction with Sacramento County Airport Service.
	Permit to Cross or Encroach	Union Pacific Railroad	Jon Devish	1800 Farnam Omaha, NE 68102	(402) 997-3563 (402) 997-3601	Need to submit application and Exhibit A for each crossing or encroachment.
STATE						
Fish and Game Code Section 1601	Streambed Alteration Agreement	California Department of Fish and Game		Region 2 1701 Nimbus Road, Ste. A Rancho Cordova, CA 95670	(916) 358-2900/ (916) 445-0045	Need to submit application for streambed alteration of Dry Creek and Pleasant Grove Creek.
California Health and Safety Code	Public Water System Permit	California Department of Health Services	Brian Kinney	DDWEM, Sacramento District P.O. Box 942732 Sacramento, CA 94234-7320	(916) 449-5688 (916) 449-5656	Need to meet with DHS to present design and obtain consensus on design criteria, then amend water supply permits.
California Streets and Highways Code	Encroachment Easement	California Department of Transportation	Rich Jones	703 B Street P.O. Box 911 Marysville, CA 95901	(530) 741-5374	Need permit to cross Highway 99.
	Tunneling Permit – Gas Classification	California Occupational Safety and Health Administration	Gerald Fulhgrum	Cal-OSHA Division of Mining and Tunneling 2211 Park Towne Circle Sacramento, CA 95825-0414	(916) 574-2540	Need Gas Classifications for potential tunnel/boring under roads and creeks.
Clean Water Act	Section 401 Water Quality Certificate	Central Valley Regional Water Quality Control Board	Patrick Gillum	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4709	Need to obtain Water Quality Certification Waiver for USACE Permit.
Clean Water Act	Dewatering Permit - General Order No. 5-00-175	Central Valley Regional Water Quality Control Board	Michael Negrette	11020 Sun Center Drive No. 200 Rancho Cordova, CA 95670-6114	(916) 464-4662	May need to obtain dewater permit for low threat discharges for construction dewatering.
Clean Water Act	Stormwater Permit for Construction Activities	State Water Resources Control Board		P.O. Box 1977 Sacramento, CA 95812-1977	(916) 341-5537	Need to submit NOI for General Permit for Construction Activities.

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Table 8-5 Pipeline Permit Requirements (contd).

Regulation	Permit Required	Permitting Agency	Agency Contact	Agency Address	Agency Phone/Fax	Permit Notes
LOCAL						
Clean Air Act and California HSC Section 42300	Permit to Construct and Operate Stationary Generators and Motorized Equipment	Placer County Air Control District	Zach Lee	11464 B Avenue DeWitt Center Auburn, CA 95603	(530) 889-7127/ (530) 889-7107	Need to submit application for Booster Pump Station generator if used.
FAA Coordination	Consultation with Sacramento County Airport Service to Design Facilities to Meet Safety Standards and Presentation to FAA	Sacramento County Airport Service	Leonard Takayama/ Greg Rowe		(916) 874-0619/ (916) 874-0698	Need to meet all safety requirements for current Overflight Zone and future Approach/Departure Zone at SMF.
	Tree Pruning and Removal Permit	Sacramento County Public Works Agency	Technical Resources Section		(916) 874-5278	Need to submit application to remove riparian trees on river-side of Sacramento River levee.
	Encroachment Permit	Sacramento County Department of Transportation (point of contact according to Web site)	Dennis Nakagawa	827 7th Street, Rm. 102 Sacramento, CA 95814	(916) 874-5823 LDSIR: (916) 874-6544	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	Placer County Department of Public Works, Road Maintenance Division	Bob Vrooman	11444 B Avenue DeWitt Center Auburn, CA 95602	(530) 889-7565/ (530) 889-3528	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	City of Sacramento Department of Public Works	George Wilson	660 J. Street, Suite 250 Sacramento, CA 95814	(916) 808-1981/ (916) 448-8450	Need permit to construct pipeline in road right-of-way.
	Encroachment Permit	City of Roseville Public Works	Chris Kraft	311 Vernon Street Roseville, CA 95678	(916) 746-1300/ (916) 774-5379	Need permit to construct pipeline in road right-of-way.
	Tree Permit	Placer County Planning Department		11444 B Avenue DeWitt Center Auburn, CA 95602	(530) 886-3000	

Key:
Cal-OSHA – California Occupational Safety and Health Administration
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FAA – Federal Aviation Administration
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Table 8-6 Overview of Permit Documentation Needs

Permitting Agency/Permit	Documentation Type									
	Project Location/Map	Project Purpose	Project Description	Detailed Engineering	General Engineering	Environmental Assessment	Geotechnical Review	Drawings	Adjacent Property Holders	Other Permits/ Certifications
USACE – Section 404/10	√	√	√	√		√		√	√	√
USCG – Private Aid to Navigation	√		√	√				√		
FAA/Sac Co. Airport Service – Form 7460-1	√	√	√	√				√		
Union Pacific RR – Permit to Cross or Encroach	√		√	√			√	√		
CDFG – SAA	√		√	√		√		√		√
DHS – Water Supply Permit	√	√	√	√		√		√		√
CVRWQCB – NOI Dewater	√		√		√	√	√			√
CVRWQCB – Section 401 WQ	√	√	√		√	√		√	√	√
CVRWQCB – NPDES for Stormwater	√	√	√		√	√	√	√	√	√
SWRCB – NOI Construction Stormwater	√		√	√		√				√
CalTrans – Encroachment Permit	√		√	√			√	√		
Cal-OSHA – Gas Classification	√		√		√		√			
The Reclamation Board – Encroachment Permit	√	√	√	√		√	√	√		√
SMAQMD – Generator Permit	√			√						
PCACD – Generator Permit	√			√						
Sac Co. Planning Dept. – Use Permit	√	√	√		√	√		√	√	√
Sac Co. PW – Tree Removal Permit	√			√		√		√		
Sac Co. Eng./Admin. – Building Permit	√			√				√		
Sac Co. Water Resources – Storm Drain Connection	√		√		√	√	√			
CSD-1/SRCSD – Sewer Connection	√		√		√	√	√			
Sac Co. Env. Mgmt. Dept. – Septic System Permit	√			√		√	√	√		
Sac Co. Dept. of Transportation – Encroachment Permit	√			√				√		
Placer County PW – Encroachment Permit	√			√				√		
City of Sac PW – Encroachment Permit	√			√			√	√		
City of Roseville PW – Encroachment Permit	√			√				√		
Placer County Planning Department – Tree Permit	√		√	√		√		√		

Key:
Cal-OSHA – California Occupational Safety and Health Administration
CDFG – California Department of Fish and Game
CDHS – California Department of Health Services
CSD-1 – County Sanitation District 1
CVRWQCG – Central Valley Regional Water Quality Control Board
FAA – Federal Aviation Administration
NOI – Notice of Intent
NPDES – National Pollutant Discharge Elimination System
PCACD – Placer County Air Control District
PW – Public Works
Rec. Board – State of California Water Resources Agency – The Reclamation Board
RR – railroad
SAA – Streambed Alteration Agreement
Sac. Co. – Sacramento County
SMAQMD – Sacramento Metropolitan Air Quality Management District
SMF – Sacramento International Airport
SRCSD – Sacramento Regional County Sanitation District
USACE – United States Army Corps of Engineers
USCG – United States Coast Guard
WQ – Water Quality

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APPENDIX A

SUMMARY OF CONTAMINANTS

APPENDIX A
ARPS-ELVERTA DIVERSION ALTERNATIVE

Summary of Contaminants Currently Regulated by USEPA and DHS

Contaminant		Regulation	MCL (mg/L)
Inorganics (Section 64432)			
	Aluminum	DHS	1
	Antimony	Phase V	0.006
	Arsenic	NPDWR	0.010
	Barium	DHS/Phase II	1.0/2.0
	Beryllium	Phase V	0.004
	Cadmium	Phase II	0.005
	Chromium	DHS/Phase II	0.05/0.1
	Copper	LCR	1.3 [1,2]
	Cyanide	Phase V	0.15
	Fluoride	DHS/NPDWR	2.0/4.0
	Lead	LCR	0.015 [1,2]
	Mercury	Phase II	0.002
	Nickel	Phase V	0.1 [3]
	Selenium	Phase II	0.05
	Thallium	Phase V	0.002
Nitrate, Nitrite (Section 64432.1)			
	Nitrate	Phase II	10 as N (45 as NO3)
	Nitrite	Phase II	1 as N
	Nitrate + Nitrite	Phase II	10 (sum as N)
Asbestos (Section 64432.2)			
	Asbestos	Phase II	7 MFL (>10um)
Secondary Standards (Section 64449, Table 64449-A)			
	Aluminum	DHS	0.2
	Color	DHS	15 Units
	Copper	LCR	1
	Corrosivity	DHS	non-corrosive
	Foaming Agents	DHS	0.5
	Iron	DHS	0.3
	Manganese	DHS	0.05 (0.5 [1])
	Methyl-tert-butyl-ether (MTBE)	DHS	0.005
	Odor-Threshold	DHS	3 Units
	Silver	DHS	0.1
	Thiobencarb	DHS	0.001
	Turbidity	DHS	5 NTU
	Zinc	DHS	5

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

Contaminant		Regulation	MCL (mg/L)
Secondary Standards (Section 64449, Table 64449-B)			
	Total Dissolved Solids	DHS	500/1,000/1,500 [4]
	Specific Conductance	DHS	900/1,600/2,200 [4]
	Chloride	DHS	250/500/600 [4]
	Sulfate	DHS	250/500/600 [4]
General Mineral (Section 64449 (c) (2))			
	Bicarbonate	DHS	MO
	Carbonate	DHS	MO
	Hydroxide	DHS	MO
	Alkalinity	DHS	MO
	pH	DHS	MO
	Calcium	DHS	MO
	Magnesium	DHS	MO
	Sodium	DHS	MO
	Hardness	DHS	MO
(Volatile) Organic Chemicals (Section 64444, Table 64444-A (a))			
	Benzene	DHS/Phase I	0.001/0.005
	Carbon Tetrachloride	DHS/Phase I	0.0005/0.005
	o-Dichlorobenzene	Phase II	0.6
	p-Dichlorobenzene	DHS/Phase I	0.005/0.0785
	1,1-Dichloroethane	DHS	0.005
	1,2-Dichloroethane	DHS/Phase I	0.0005/0.005
	1,1-Dichloroethylene	DHS/Phase I	0.006/0.007
	cis-1,2-Dichloroethylene	DHS/Phase II	0.006/0.1
	trans-1,2-Dichloroethylene	DHS/Phase II	0.010/0.1
	Dichloromethane (Methylene chloride)	Phase V	0.005
	1,2-Dichloropropane	Phase II	0.005
	1,3-Dichloropropene	DHS	0.0005
	Ethylbenzene	Phase II	0.3
	Methyl-tert-butyl ether (MTBE)	DHS	0.013
	Monochlorobenzene	DHS/Phase II	0.07/0.1
	Styrene	Phase II	0.1
	1,1,2,2-Tetrachloroethane	DHS	0.001
	Tetrachloroethylene	Phase II	0.005
	Toluene	DHS/Phase II	0.15/1.0
	1,2,4-Trichlorobenzene	Phase V	0.005
	1,1,1-Trichloroethane	Phase I	0.2
	1,1,2-Trichloroethane	Phase V	0.005
	Trichloroethylene	Phase I	0.005

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

Contaminant	Regulation	MCL (mg/L)
Trichlorofluoromethane	DHS	0.15
1,1,2-Trichloro-1,2,2-Triflouroethane	DHS	1.2
Vinyl Chloride	DHS/Phase I	0.0005/0.002
Xylenes (total)	DHS/Phase II	1.75/10
(Non-Volatile Synthetic) Organic Chemicals (Section 64444, Table 64444-A (b))		
Acrylamide	Phase II	TT (PAP)
Alachlor	Phase II	0.002
Atrazine	Phase II	0.001
Bentazon	DHS	0.018
Benzo(a)pyrene	Phase V	0.0002
Carbofuran	DHS/Phase II	0.018/0.04
Chlordane	DHS/Phase II	0.0001/0.002
2,4,-D	Phase II	0.07
Dalapon	Phase V	0.2
Dibromochloropropane	Phase II	0.0002
Di (2-ethylhexyl) Adipate	Phase V	0.4
Di (2-ethylhexyl) Phthalate	DHS/Phase V	0.004/0.006
Dinoseb	Phase V	0.007
Diquat	Phase V	0.02
Endothall	Phase V	0.1
Endrin	Phase V	0.002
Epichlorohydrin	Phase II	TT (PAP)
Ethylene Dibromide	Phase II	0.00005
Glyphosate	Phase V	0.7
Heptachlor	DHS/Phase II	0.00001/0.0004
Heptachlor Epoxide	DHS/Phase II	0.00001/0.0002
Hexachlorobenzene	Phase V	0.001
Hexachlorocyclopentadiene	Phase V	0.05
Lindane	Phase II	0.0002
Methoxychlor	Phase II	0.03
Molinate	DHS	0.02
Oxamyl (vydate)	Phase V	0.05
Pentachlorophenol	Phase II	0.001
Picloram	Phase V	0.5
PCBs	Phase II	0.0005
Simazine	Phase V	0.004
Thiobencarb	DHS	0.07
Toxaphene	Phase II	0.003
2,3,7,8-TCDD (Dioxin)	Phase V	3.00E-08
2,4,5-TP (Silvex)	Phase II	0.05

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

Contaminant		Regulation	MCL (mg/L)
Unregulated (Volatile) Organic Chemicals (Section 64450, Table 64450-A)			
	Dichlorodifluoromethane	DHS	1.0 [1]
	1,2,3-Trichloropropane	DHS	0.000005 [1]
	Ethyl-tert-butyl-ether (ETBE)	DHS	MO (if vulnerable)
	tert-Amyl-methyl ether (TAME)	DHS	MO (if vulnerable)
	Perchlorate	DHS	0.004 [1]
	Boron	DHS	1.0 [1]
	Hexavalent Chromium	DHS	MO (if vulnerable)
	tert-Butyl alcohol	DHS	0.012 [1]
	Vanadium	DHS	0.05 [1]
Natural Radioactivity (Section 64441)			
	Gross Alpha Particle Activity	NPDWR	15 pCi/L
	Combined Radium 226 & 228	NPDWR	5 pCi/L
	Uranium	DHS	20 pCi/L
Man-Made Radioactivity (Section 64443)			
	Tritium	DHS	20,000 pCi/L
	Strontium-90	DHS	8 pCi/L
	Gross Beta Particle Activity	NPDWR	50 pCi/L
Disinfection By-Products			
	Total Trihalomethanes (Chloroform, Bromoform, Chlorodibromomethane, Bromodichloromethane)	Stage 1 D/DBP Rule	0.08
	Haloacetic Acids 5 (Mono, di, and tri-chloroacetic acid, mono and di-bromoacetic acid)	Stage 1 D/DBP Rule	0.06
	Chlorite	Stage 1 D/DBP Rule	1
	Bromate	Stage 1 D/DBP Rule	0.01
Disinfection By-Product Precursors			
	Total Organic Carbon	Stage 1 D/DBP Rule	TT (percent Removal)
Disinfectants			
	Chlorine (as Cl2)	Stage 1 D/DBP Rule	4 [5]
	Chloramines (as Cl2)	Stage 1 D/DBP Rule	4 [5]
	Chlorine Dioxide (as ClO2)	Stage 1 D/DBP Rule	0.8 [5]
Microbial			
	Giardia Lamblia	SWTR	TT(3-log Reduction)
	Legionella	SWTR	TT
	Viruses	SWTR	TT(4-Log Reduction)
	Disinfectant Residual	SWTR	TT(detectable)
	Total Coliform	TCR	TT(<5percent mo. samples pos., if >40 samples per month)
	Fecal Coliform	TCR	TT (positive sample)
	E. Coli	TCR	TT (positive sample)

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

	Contaminant	Regulation	MCL (mg/L)
	Turbidity	IESWTR	TT (<0.3 in 95percent CFE samples, <1 in 100percent CFE)
	Cryptosporidium	IESWTR	TT(2-log Reduction)
Additional Organics with Action Levels			**Action Levels**
	Aldicarb	DHS	0.007
	Aldrin	DHS	0.000002
	Baygon	DHS	0.03
	a-Benzenehexachloride	DHS	0.000015
	b-Benzenehexachloride	DHS	0.000025
	n-butylbenzene	DHS	0.26
	sec-butylbenzene	DHS	0.26
	tert-butylbenzene	DHS	0.26
	Captan	DHS	0.0015
	Carbaryl	DHS	0.7
	Carbon disulfide	DHS	0.16
	Chlorate	DHS	0.8
	Chloropicrin	DHS	0.056
	2-chlorotoluene	DHS	0.14
	4-chlorotoluene	DHS	0.14
	Chlorpropham	DHS	1.2
	1,3-Dichlorobenzene	DHS	0.6
	2,4-Dimethylphenol	DHS	0.1
	1,4-Dioxane	DHS	0.003
	Diazinon	DHS	0.006
	Dieldrin	DHS	0.000002
	Diphenamide	DHS	0.2
	Ethion	DHS	0.004
	Ethylene glycol	DHS	14
	Formaldehyde	DHS	0.1
	Isopropylbenzene	DHS	0.77
	Malathion	DHS	0.16
	Metam sodium	DHS	0.02
	Methyl isobutyl ketone (MIBK)	DHS	0.12
	Methylisothiocyanate	DHS	0.05
	Methyl parathion	DHS	0.002
	Napthalene	DHS	0.17
	N-Nitrosodimethylamine (NDMA)	DHS	0.00001
	Parathion	DHS	0.04
	Pentachloronitrobenzene	DHS	0.02
	Phenol	DHS	4.2
	n-propylbenzene	DHS	0.26
	Trithion	DHS	0.007

Summary of Contaminants Currently Regulated by USEPA and DHS (contd.)

Contaminant	Regulation	MCL (mg/L)
2,3,5,6-tetrachloroterephthalate	DHS	3.5
1,2,4-Trimethylbenzene	DHS	0.33
1,3,5-Trimethylbenzene	DHS	0.33

Notes:
[1] - Action Level
[2] - Based on 90th Percentile of Tap Water Samples
[3] - DHS MCL lower than EPA, EPA remanded in 1995
[4] - Recommended/Upper/Short Term MCLs
[5] - Maximum Residual Disinfectant Level (MRDL)

Key:
CFE – Combined Filter Effluent
D/DBP – Disinfectants and Disinfection By-Products
DHS – California Department of Health Services
IESWTR – Interim Enhanced Surface Water Treatment Rule
LCR – Lead and Copper Rule
MCL – Maximum Contaminant Level
MO – Monitored Only
NPDWR – National Primary Drinking Water Regulation
PAP – Polymer Addition Practices
SWTR – Surface Water Treatment Rule
TCR – Total Coliform Rule
TT – Treatment Technology
USEPA – United States Environmental Protection Agency