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Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program

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Delta Habitat Conservation and Conveyance Program (DHCCP)

November 30, 2009

Conceptual Engineering Report
Through-Delta Facility Conveyance Option

DHCCP: Program
Preliminary Not for Construction

CONCEPTUAL ENGINEERING REPORT

Through Delta Option

November 30, 2009

Prepared by:
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DHCCP – Program
Preliminary – Not for Construction
Support Document for BDCP EIR/S Administrative Draft, January 2013

LIMITATIONS

This report was prepared in accordance with the contract, Conceptual-Level Engineering & Design / Conveyance Options Analysis. The report required use of information that was readily available from Lead Agencies and from site visits during the time the analysis was performed. New information obtained following the distribution of this report could change the details and conclusions provided herein.

The purpose of this report is to provide conceptual engineering of facilities required for the Delta Habitat Conservation and Conveyance Program (DHCCP) in order to assist the Lead Agencies in their decision making process. It is anticipated that this document would be used to support engineering and design and the environmental impact statement (EIS) and environmental impact report (EIR) for the Bay Delta Conservation Plan (BDCP). It is also expected that the Lead Agencies, the EIR/EIS consulting team, and other stakeholders in the BDCP process would recommend modifications to these facilities over the coming months, and such changes would be evaluated as needed to support the EIR/EIS.

This document is part of an iterative process of developing options that can be used as alternatives in the EIR/EIS. Therefore, all locations, dimensions, quantities, design concepts, construction techniques, and so on presented herein are subject to change as more information becomes available. The alignment and alignment features presented in this document are preliminary and subject to change.

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ACRONYM AND ABBREVIATION LIST

AASHTO	American Association of State Highway and Transportation Officials
ADT	average daily traffic
AF	acre-feet
AN	Above Normal
Banks	Harvey O. Banks
BBID	Byron Bethany Irrigation District
BDCP	Bay Delta Conservation Plan
BMP	best management practice
BN	Below Normal
BNSF	Burlington Northern and Santa Fe [Railroad]
C	Critical
CAISO	California Independent System Operator
CALFED	CALFED Bay-Delta Program
Cal/OSHA	California Occupational Safety and Health Administration
CALSIM	California Water Resources Simulation Model
Caltrans	California Department of Transportation
CCF	Clifton Court Forebay
CER	conceptual engineering report
cfs	cubic feet per second
CHTR	collection, handling, transport, and release
CIP	cast-in-place
CRSB	Coast Ranges-Sierran Block
CVP	Central Valley Project
cy	cubic yard
D	Dry
DC	direct current
DCC	Delta Cross Channel
DHCCP	Delta Habitat Conservation and Conveyance Program
DMC	Delta-Mendota Canal
DRMS	Delta Risk Management Strategy
DSM2	Delta Simulation Model II
DWR	California Department of Water Resources
EBMUD	East Bay Municipal Utility District
EIR	environmental impact report
EIS	environmental impact statement
EL	elevation

EPB	earth pressure balance
ESA	Endangered Species Act of 1973
FB	freeboard
FFTT	Fish Facilities Technical Team
fps	feet per second
g	acceleration due to gravity
GIS	geographical information system
GPS	global positioning satellite
H:V	horizontal to vertical ratio
HEC-RAS	Hydrologic Engineering Center-River Analysis System (floodplain management modeling software)
HGL	hydraulic grade line
HPS	high pressure sodium vapor
HPU	hydraulic power unit
HVAC	heating, ventilation, and air conditioning
ICF	Isolated Conveyance Facility
JOC	Joint Operations Center
Jones	C.W. “Bill” Jones
km	kilometers
kV	kilovolt
lb/d	pounds per day
MCC	motor control center
MHHW	mean higher high water
mph	miles per hour
msl	mean sea level
MVA	megavolt amperes
MW	megawatt
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NPDES	National Pollutant Discharge Elimination System
O&M	operation and maintenance

PCS	plant control system
PGA	peak ground acceleration
PG&E	Pacific Gas and Electric
PLC	programmable logic controller
psi	pounds per square inch
RCCP	reinforced concrete cylinder pressure pipe
RCP	reinforced concrete pipe
Reclamation	United States Bureau of Reclamation
ROW	right-of-way
RSP	rock slope protection
RWQCB	Regional Water Quality Control Board
SAFCA	Sacramento Area Flood Control Agency
SCADA	supervisory control and data acquisition
SDIP	South Delta Improvement Program
SJRTPP	San Joaquin River Tunnel Pumping Plant
Skinner Facility	Skinner Delta Fish Protection Facility
SLDMWA	San Luis and Delta-Mendota Water Authority
SLR	sea level rise
SMUD	Sacramento Municipal Utility District
SR	State Route
SWP	State Water Project
SWPPP	Stormwater Pollution Prevention Plan
TBD	to be determined
TBM	tunnel-boring machine
TDF	Through-Delta Facility
TSS	total suspended solids
UPRR	Union Pacific Railroad
URS	URS Corporation
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VCPP	Victoria Canal Fish Salvage Facility Pumping Plant
VFD	variable frequency drive
W	Wet
WAPA	Western Area Power Administration

WGCEP	Working Group on Northern California Earthquake Probabilities
WSE	water surface elevation
%	percent

EXECUTIVE SUMMARY

This report describes the Through-Delta Facility (TDF) Option, a conveyance system designed to bring water from the Sacramento River to the water export pumping plants in the south Sacramento River and San Joaquin River Delta (Delta). The TDF Option is one of four conveyance options requested by Governor Schwarzenegger to be investigated as potential solutions to help reduce the deteriorating conditions in the Delta. These conceptual engineering reports (CERs) have been prepared by the Delta Habitat Conservation and Conveyance Program (DHCCP).

Since 2000, numerous studies have investigated various approaches to improve the existing system for conveying water through the Delta. DWR has a Web page on TDF investigations (http://bayDeltaoffice.water.ca.gov/nDelta/summaryreport/index.cfm). One of the most complete overviews of TDFs in the north and south Delta is provided in Alternatives for Delta Water Transfer (DWR, 1983). More recently, DWR completed the Draft Through-Delta Facility Prefeasibility Study (DWR, 2007b) (Prefeasibility study, or report) and Through-Delta Facility Value Planning Study Final Report (DWR, 2007c). Both of these reports focus primarily on facilities that would divert water into the northern interior Delta. Upgrades to strengthen Delta channel levees were considered during the Delta Risk Management Strategy (DRMS) investigations, as documented in Evaluation of Delta Risk Management Strategies Preliminary Report (DWR, 2007d).

Another distinct approach to through-Delta conveyance is the Delta Corridors Proposal (Jones and Stokes, 2007). This approach includes a series of measures to separate the San Joaquin River Corridor from the Sacramento River to better isolate fish from the water supply corridor. This approach uses channel closures, improved levees, divided channels, and siphons to reconfigure the Delta.

For the purposes of DHCCP, two of the more recent TDF concepts are being considered. This report looks at the improved armored corridor approach. The armored corridor approach was previously considered by DRMS and cited in Governor Schwarzenegger’s letter to Senators Perata, Steinberg, and Machado as warranting further analysis (Governor Schwarzenegger, 2008). Armoring levees for flood protection has occurred throughout the Delta. Armoring schemes vary from island to island but typically consist of installing large rocks (rip-rap) on the waterside of the levee. The Delta Corridors Proposal is being separately evaluated by DHCCP. This proposal is currently undergoing modifications, but will be considered in the EIR/EIS.

The TDF Option is depicted on Figure ES-1 and modifies the existing water conveyance system utilizing existing Delta channels to improve the protection of endangered species and reduce the vulnerability of this system to flood, earthquake, and sea level rise (SLR). Water collected from the Delta Cross Channel and potential new intakes would flow to the south through existing river and slough channels to new fish salvage facilities near Clifton Court Forebay (CCF). After passing through the fish screens, water would be conveyed to the pumping plants serving the State Water Project (SWP) and Central Valley Project (CVP).

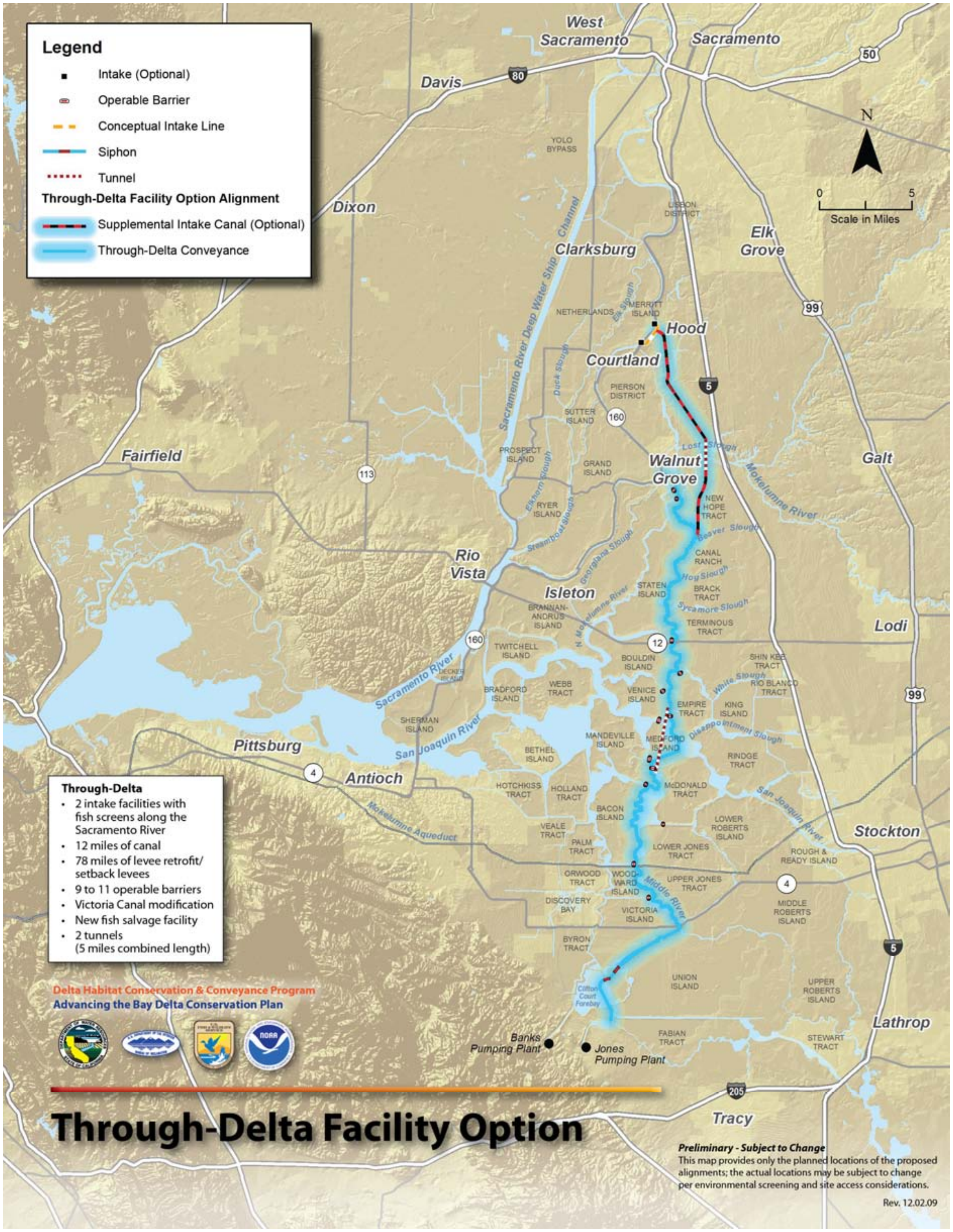


Figure ES-1 – Overview

The system includes:

- New intake facilities equipped with fish screens
- A potential new canal to convey flows from new intake locations
- Operable barriers that can be raised to better protect the natural conveyance channels if a catastrophic event occurs
- Setback levees and armoring of existing levees to reduce the system vulnerability
- Modern fish salvage facilities to reduce the impacts of predation at CCF

Other facilities to support the function of the proposed TDF Option include pumping plants, culvert siphons and tunnels for crossing existing channels, roadway and bridge modifications, and other utility improvements.

The facilities are summarized in Table ES-1.

Table ES-1: Summary of TDF Conveyance Option Physical Characteristics

	Feature Description / Acreage*	Characteristic
Overall Project / 11,500		
	Export Capacity (cfs)	15,000
	Overall Length (miles)	49.5
Intake Facilities / 200		
Sacramento River		
	Number of Fish-Screened Intakes	2
	Flow Capacity at Each Intake (cfs)	2,000
	Each Plant 4 Pumps, Capacity Per Unit (cfs)	500
	Total Dynamic Head (ft)	8
	Each Plant Total Electric Load (MW)	1.75
Delta Cross Channel		
	Maximum Flow Capacity (cfs)	9,000
Rivers and Sloughs		
	Net Flow Entering Through-Delta Corridor (cfs)	Variable
Intermediate Pumping Plants / 500		
Optional San Joaquin River Tunnel Pumping Plant		
	Number of Chevron-Type Fish-Screened Intakes	5
	Flow Capacity at Each Intake (cfs) (two pumps per intake)	3,000
	10 Pumps, Capacity Per Unit (cfs)	1,000
	Total Dynamic Head (ft)	20
	Total Electric Load (MW)	33

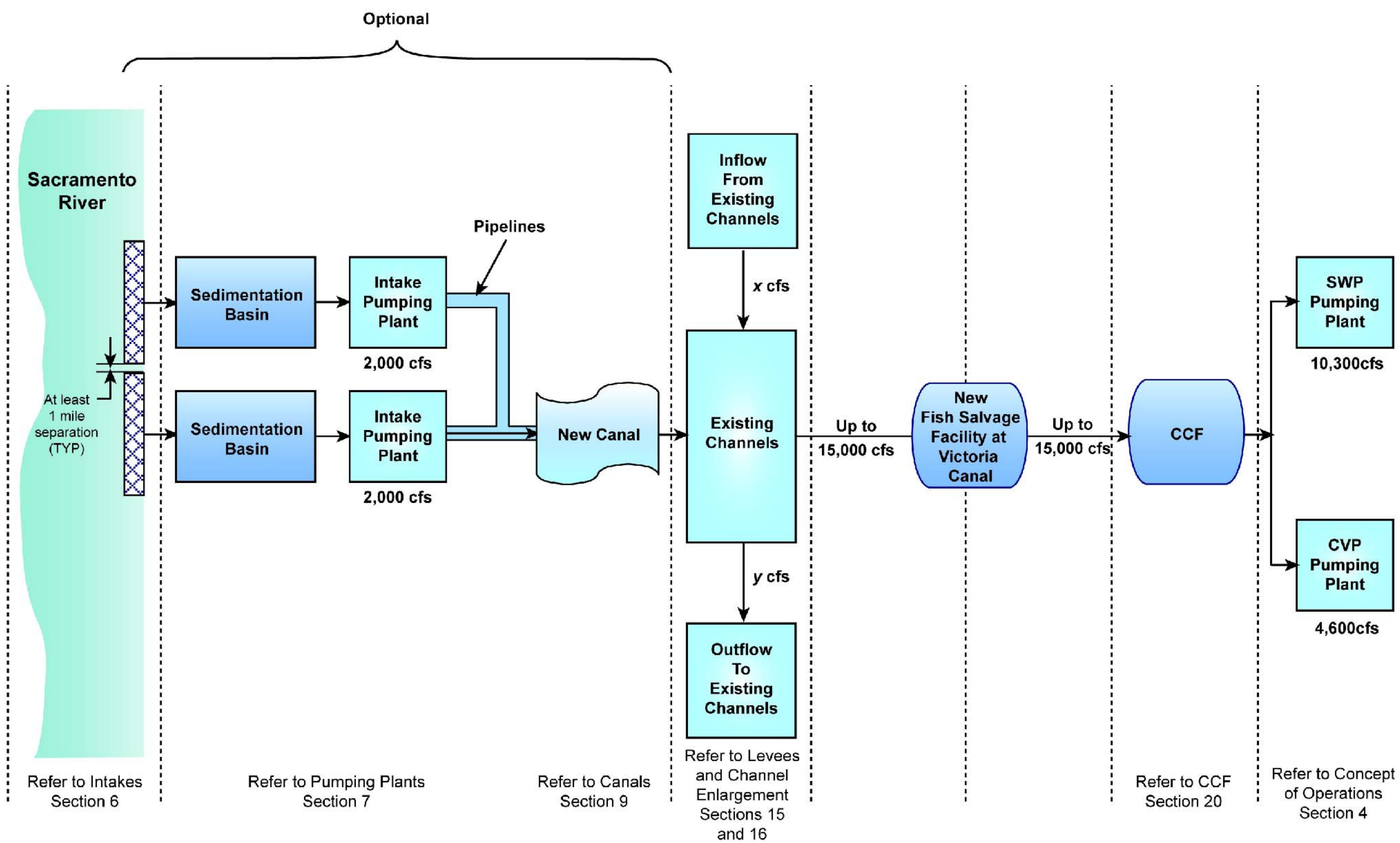
	Feature Description / Acreage*	Characteristic
Victoria Canal		
	Number of Chevron-Type Fish-Screened Intakes	5
	Flow Capacity at Each Screened Intake (cfs)	3,000
	10 Pumps, Capacity Per Unit (cfs)	1,000
	Total Dynamic Head (ft)	10
	Total Electric Load (MW)	33
Optional Supplemental Intake Conveyance Canal / **2,000**		
	Type	Unlined
	Approximate Maximum Top Width (ft) (Outside to outside at top of embankment)	378
	Bottom Width (ft)	65
	Depth	23.5
	Side Slopes	3H:1V, 8H:1V
	Average Permanent Right-of-Way Width (ft)	1,500
	Canal Length (miles)	12.1
	Permanent Right-of-Way (acres)	1,613
	Excavation Volume (million cy)	10
	Compacted Embankment Volume (million cy)	23
	Stone Lake Drain Siphon (2 barrel, 19 ft by 19 ft) length (ft)	1,736
Culvert Siphons / **167**		
	Old River, Siphon (5 barrel, 23 ft by 23 ft) length (ft)	1,255
	West Canal, Siphon (4 barrel, 26 ft by 26 ft) length (ft)	1,557
Tunnel Siphons / **450**		
Lost Slough/Mokelumne River Tunnel		
	Tunnel Length (ft)	9,350
	Number of Tunnel Shafts	2
	Number of Tunnel Bores	1
	Tunnel Diameter (ft)	27
Optional San Joaquin River Tunnel		
	Tunnel Length (ft)	18,340
	Number of Tunnel Shafts	4
	Number of Tunnel Bores	2
	Tunnel Diameter each (ft)	33
Levees and Operable Barriers / **8,000**		
	Levee Length of Corridor (miles)	38.4
	Total Length of Levees Constructed/Modified (miles)	78.0
	Length of Adjacent Setback Levees (miles)	38.6
	Length of Offset Setback Levees with No Alteration of Existing Levee (miles)	6.7
	Length of Offset Setback Levees with Notched Existing Levee (miles)	10.5
	Length of On-Channel Setback Levees of Widened Existing Channel (miles)	13.8
	Length of On-Channel Levees at New Channel (miles)	2.7

Feature Description / Acreage*		Characteristic
	Length of Facilities Protection Levee (miles)	1.2
	Length of Strengthened Existing Levees (miles)	4.5
	Compacted Setback and Strengthened Levee Volume (million cy)	127
	Number of Operable Barriers	9 to 11

cfs = cubic feet per second
cy = cubic yard
ft = feet
H:V = horizontal to vertical ratio
MW = megawatt
TDF = Through-Delta Facility

*Acreage is for permanent facilities and associated temporary disturbance.
Overall Project Acreage includes facilities not listed, such as bridge abutments.

The report describes the facilities, site plans, and construction methodologies proposed. A simplified flow diagram is provided on Figure ES-2.



California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program

DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Conveyance Schematic

FIGURE
ES-2

DHCCP Engineering
WORKING DRAFT - Subject to Change

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1.0 INTRODUCTION

1.1 Program Overview

This report is in partial fulfillment of Task 4, Subtask 4.5, of the Delta Habitat Conservation and Conveyance Program (DHCCP). This conceptual engineering report (CER) describes the facilities that make up the Through-Delta Facility Conveyance (TDF) Option. Conceptual engineering information is needed to support the development of the environmental impact statement (EIS) and environmental impact report (EIR) for the Bay Delta Conservation Plan (BDCP).

This document continues the DHCCP iterative process of developing the TDF Option so that it can be considered an alternative for the EIR/EIS. The material herein describes the conceptual engineering of facilities currently proposed to convey water through the Delta. Other solutions may be considered in the future as needed to support the EIR/EIS.

1.2 Purpose and Scope of Conveyance Option

The preliminary concept for the TDF Option is to improve the existing conveyance system, which relies primarily on natural channels to convey water from the Sacramento River into Clifton Court Forebay (CCF) and to the pumping plant for the State Water Project (SWP). An inlet channel from Old River provides water to the Central Valley Project (CVP). Sacramento River water currently enters the northern end of the system through the Delta Cross Channel (DCC) and Georgiana Slough, and flows through a series of rivers and sloughs to CCF and the inlet channel to the CVP. Pumping from the SWP at CCF and the CVP draws the flow through the central Delta. Narrow channels in the north and south Delta and inefficient transfer of water in the north Delta limit the capacity through the existing system and impact water quality.

The TDF Option includes the following improvements to enhance the performance of the existing system.

- New conveyance and channel modifications address capacity restrictions in the north Delta and improve water quality, thereby reducing the need for water to be released from upstream reservoirs to maintain compliance with salinity standards.
- Levees along the TDF rivers and sloughs replaced or strengthened to reduce their vulnerability to flood, earthquake, and subsidence.
- Operable barriers installed to improve water quality while benefiting migrating fish.
- Channels in the south Delta enlarged to reduce the likelihood of scour.
- New fish facilities added at the inlet to CCF to reduce the impacts of predation.

The facilities included in the TDF Option were engineered based on the following assumptions:

- The TDF Option could be capable of delivering 15,000 cfs to CCF.
- The north end of the TDF does not need to independently provide 15,000 cfs from the Sacramento River. The TDF Option is not an isolated facility and is expected to pick up

additional water as flow proceeds to the south (e.g., from the Mokelumne and San Joaquin Rivers).

- The through-Delta corridor of natural channels conveying water to CCF should be protected against a 200-year flood event and a 200-year return period earthquake. In the event one of these catastrophic events occurs, the conveyance system needs to be restored within a matter of weeks.
- The impacts to endangered fish species from the facilities included in the TDF Option should be less than the impacts from the existing system (e.g., increasing impacts to smelt and salmon by adding intakes south of the DCC is not a viable option).

These assumptions affect the facilities that have been included in the TDF Option. Changing any of these assumptions fundamentally alters the conceptual engineering approach.

The TDF Option includes some optional facilities that may or may not be included in the final design if this alignment moves forward in the planning process. These are described below.

- **Supplemental Intake Canal Option.** Under low-flow conditions and high export pumping rates, Delta salinity increases when the DCC gates are closed to protect emigrating juvenile Sacramento Basin salmon. Multiple options have been considered to enable system operation during low-flow conditions with a better balance of fishery, water quality, and water supply objectives. One option is to add new intakes that can be operated when the DCC gates are closed. A new canal connected to supplemental intakes located in the vicinity of the town of Hood and discharging to the South Fork of the Mokelumne River was considered in this CER (see Section 6.0 and Appendix B for further details on intake locations).
- **San Joaquin River Operable Barrier and Tunnel Options.** If a major seismic event occurs, the resulting loss of levees in the Delta could significantly impact operation of the TDF Option. The greatest vulnerability to the channel conveying water to the SWP and CVP pumping facilities is where the channel crosses the San Joaquin River. A failure of levees protecting the islands bordering the San Joaquin River could result in the formation of a large source of high salinity water that would compromise the water quality delivered south of the San Joaquin River. Two options are explored in this report to address this concern. The first is placing an operable barrier in the San Joaquin River that can be operated under an emergency to isolate the TDF Option from salt water intrusion to the system. Constructing a barrier or lock in the San Joaquin River requires a major construction effort and would impact shipping in the Stockton Deep Water Ship Channel. One advantage of constructing a barrier is that it would reduce the extent of new setback levees on the east side of the TDF Option. Alternatively, a tunnel could be constructed to convey water from the through-Delta corridor on the north side of the San Joaquin River to the south side of the river. This is also a major construction feature. It would minimize the impacts to shipping, but require more extensive levees on the east side of the TDF Option and increase the complexity of operations and maintenance.

1.3 Report Organization

The following sections are included in this report:

- Section 2.0 provides background information.
- Section 3.0 provides an overview of the alignment.
- Section 4.0 describes the operation of the existing facilities and the TDF Option.
- Section 5.0 describes the hydraulics of the entire alignment.
- Sections 6.0 through 20.0 describe the individual facilities included in the TDF Option.
- Sections 21.0 through 24.0 describe temporary construction operations and facilities.
- Section 25.0 is reserved for the Dual Conveyance Facility and is not used in this CER.
- Section 26.0 describes the potential permits and environmental reviews for implementation of a project.
- Section 27.0 provides reference citations in this CER.

The following appendices included in this report provide additional information:

- Appendix A includes geology and seismic information.
- Appendix B includes information on intakes.
- Appendix C includes information on sedimentation basins.
- Appendix D includes information on pumps.
- Appendix E includes information on pipeline materials.
- Appendix F includes information on pipe floatation.

2.0 BACKGROUND

2.1 General

The SWP and federal CVP divert water primarily from the Sacramento and San Joaquin Rivers for use by cities and farms in the Central Valley, San Francisco Bay Area, and southern California. Delta water diversions are made in the southern Delta.

The current method for conveying water to the aqueduct systems of the SWP and the CVP is based solely upon through-delta conveyance. SWP and CVP facilities include upstream reservoirs on the Sacramento and the San Joaquin Rivers. These rivers flow into the Delta where the network of rivers and sloughs are effectively used as conveyance channels to convey water to the pumping facilities near Tracy. The DCC, near Walnut Grove, controls the flow of Sacramento River water into the eastern Delta. Internal Delta channels are used to convey the water from the DCC through the central Delta to the pumping and fish salvage facilities of the SWP Harvey O. Banks (Banks) Pumping Plant and the CVP C.W. "Bill" Jones (Jones) Pumping Plant in the south Delta, near the town of Tracy. The installed maximum pumping capacity of the SWP and CVP facilities is 10,300 cfs and 4,600 cfs, respectively, for a combined pumping capacity into both the SWP and CVP aqueducts of approximately 15,000 cfs.

2.2 History of Conveyance Option

Initial investigations of an improved TDF Option to address Delta water transfer problems began following the defeat of Proposition 9 in June 1982. Interest in modifying the existing conveyance increased in December 1999 when, under low-flow conditions and high export pumping rates, Delta salinity increased when the DCC gates were closed to protect emigrating juvenile Sacramento Basin salmon (California Department of Water Resources [DWR], 2007a). This consequence revealed the need for facilities that would enable system operation with a better balance to meet fishery, water quality, and water supply objectives.

The CALFED Bay-Delta Program (CALFED) considered how to preserve both the fish benefits of closing the DCC gates and the water quality benefits of diverting Sacramento River water into the northern interior Delta, particularly during low-flow periods. Options ranged from the tide-related operations of the DCC gates (allowing a large fraction of the normal flow of Sacramento River water to pass through the DCC with the gates open only a portion of the time) to providing a new canal to convey Sacramento River water to the northern interior Delta when the DCC gates are closed. The evaluation of the TDF Option considered the possibility of a single channel, originating at a variety of locations, or the possibility of using several smaller channels. Various combinations of screening the DCC and new channel(s) were evaluated by CALFED. An approach for through-Delta conveyance was proposed in the CALFED Record of Decision (CALFED, 2000b).

Since 2000, numerous studies have investigated various approaches to improve the existing system for conveying water through the Delta. DWR has a Web page on TDF investigations (http://bayDeltaoffice.water.ca.gov/nDelta/summaryreport/index.cfm). One of the most complete overviews of TDFs in the north and south Delta is provided in Alternatives for Delta Water Transfer (DWR, 1983). More recently, DWR completed the Draft Through-Delta Facility

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For the purposes of DHCCP, two of the more recent TDF concepts are being considered. This report looks at the improved armored corridor approach. The armored corridor approach was previously considered by DRMS and cited in Governor Schwarzenegger’s letter to Senators Perata, Steinberg, and Machado as warranting further analysis (Governor Schwarzenegger, 2008). The Delta Corridors Proposal is being separately evaluated by DHCCP. This proposal is currently undergoing modifications, but will be considered in the EIR/EIS.

2.3 Existing Levee Conditions

Levee failures have flooded islands and tracts 166 times since 1900, and some flooded lands were never recovered (DWR, 2007e). Many Delta islands and tracts have flooded multiple times. High flood flows into the Delta during major storms in the upstream watershed have caused most levee failures. Some levees have failed during the summer when river flows were relatively low (sunny day failures). Water overtopping levees during high water, erosion, seepage through the levee embankment, seepage through the levee foundation, burrowing animals, and high tides have contributed to levee failures.

Most of these levees were built before modern engineering techniques, and many rest on peat soil foundations that have settled with the added weight. Levees were originally constructed of sands, silts, clays and organic soils, often by mounding up nearby excavated or dredged material. Some sandy areas within the levees and their foundations are particularly vulnerable to damage during an earthquake. Levees originally built in the 1860s through the 1920s to allow draining of swamp land for agriculture now protect a wide variety of valuable uses (DWR, 2007e). The levees have been periodically widened and raised to keep pace with subsidence on Delta islands.

There are multiple consequences of levee failures in the Delta, including the disruption of in-Delta and export water supplies. Salt water from San Francisco Bay can flow upstream to fill islands after a levee failure making Delta water too salty for use. This inferior water quality can require stopping in-Delta diversions for agriculture and exports for agriculture and urban uses.

3.0 OVERVIEW OF CONVEYANCE OPTION

3.1 Alignment and Key Components

The TDF Option would convey water from the Sacramento River in the north to CCF using the existing channels to the extent possible. Water currently enters the northern portion of the corridor of existing channels from the DCC and flows south into CCF. The TDF Option can be operated with the DCC alone; however, the Supplemental Intake Canal is also described in this report as an optional facility that would supplement water from the DCC with a new intake at the northern end of the alignment (additional intake concepts are discussed in Section 6.0). Water would enter a strengthened through-Delta corridor that would deliver water south through the South Fork of the Mokelumne River, Little Potato Slough, Little Connection Slough, Columbia Cut, Middle River, and Victoria Canal before entering CCF. This alignment and its suboptions, discussed below, are shown on Figure ES-1.

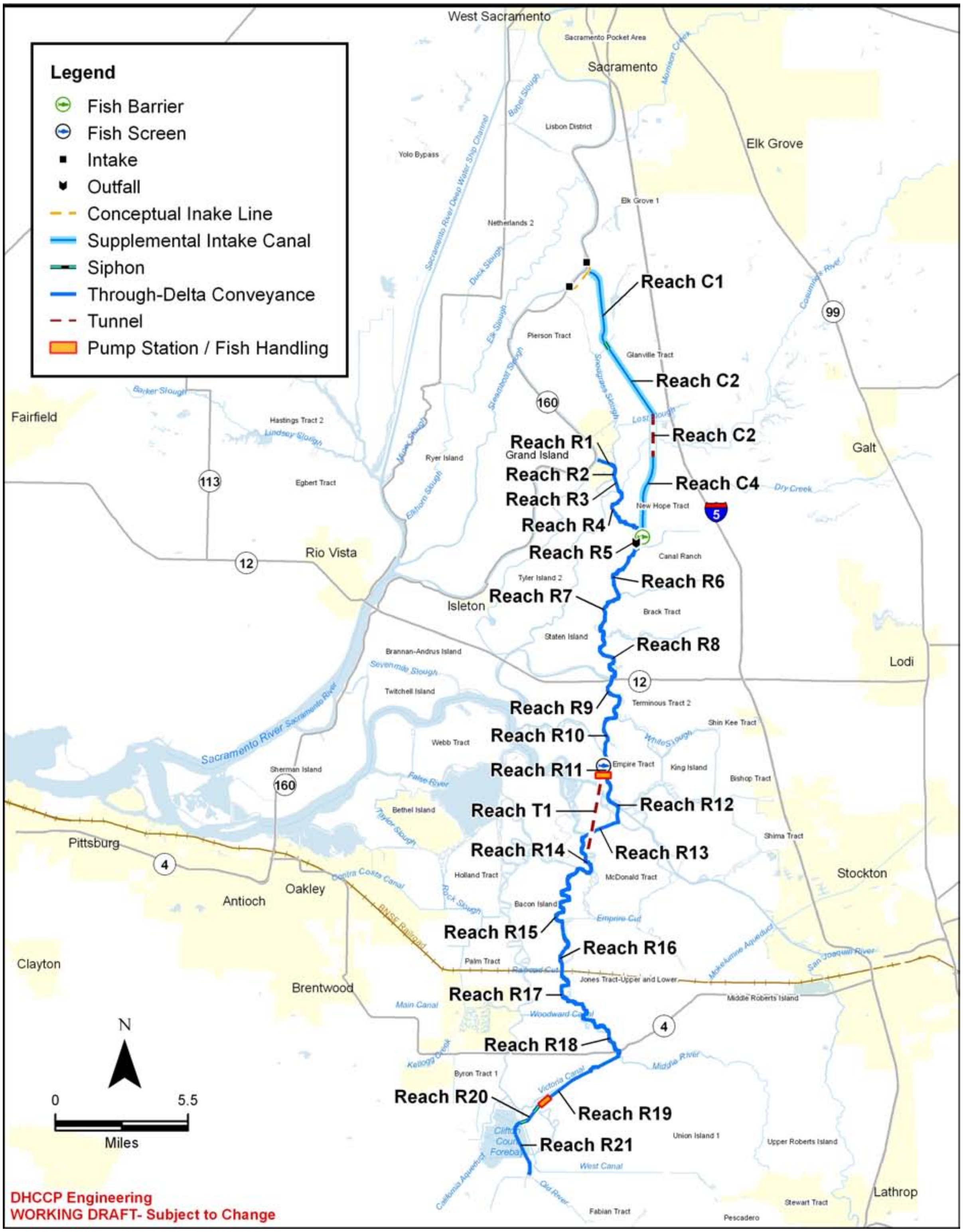
The new intake concept option facilities would consist of local pumping at two points on the Sacramento River. Water would be transported from the intake pumping plants via an open canal to the improved Delta corridor. The conveyance portions of the Supplemental Intake Canal alignment are divided into four reaches.

Experience and studies have shown that much of the existing levee system in the Delta is vulnerable to damage and potentially catastrophic failure. Analyses have shown that a significant seismic event could destabilize and initiate the failure of multiple levees concurrently, causing flooding of multiple levee-protected areas (DWR, 2007e). Most of these areas are situated below sea level. Consequently, significant inundation would occur, promoting a flush of bay water (of higher salinity content) upstream into the Delta. Such a flush of higher salinity water could compromise water quality substantially within a matter of hours, and higher salinity conditions could last for many months.

To protect against such outcomes, the TDF Option incorporates upgrades and replacement measures for levees along the corridor. A combination of approaches would be needed, including:

- Armoring of existing levees (complementing any existing armoring).
- Construction of new setback levees.
- Creation of habitat areas along levees and between existing and new setback levees.
- Notching of selected levees where new setbacks are constructed.
- Partial or complete lowering of existing levees where new setback levees are constructed.

Armoring levees for flood protection has occurred throughout the Delta. Armoring schemes vary from island to island but typically consist of installing large rocks (rip-rap) on the waterside of the levee. Large vegetation is removed from the levee to prevent damage from root masses and so inspections can be conducted efficiently.



Through-Delta Facility Reaches

Setback levees are preferred over armoring, where feasible, to improve the reliability of the through-Delta corridor. This portion of the alignment is divided into 20 reaches from north to south.

At the south end of the strengthened through-Delta corridor, the existing Victoria Canal would be widened to address current scouring conditions. The TDF Option also includes a new fish salvage facility to reduce predation of endangered species.

The primary facilities associated with this conveyance option are summarized in Table ES-1. Reaches for the TDF were developed from north to south along the alignment with a separate series of reaches established for supplemental intakes. For canals, the alignment was divided into reaches using major hydraulic structures associated with river or slough crossings. For the improved corridor within existing channels, the alignment was divided into reaches at the confluence of each major slough or river.

The Supplemental Intake Canal in the north includes four reaches (Reaches C1 through C4) from the intake structures on the Sacramento River to the discharge (or outlet) into the South Fork of the Mokelumne River. The corridor of existing channels through the Delta has a total of 21 reaches (Reaches R1 through R21), beginning at the DCC in the north and ending at a new intake canal to the CVP Jones Pumping Plant in the south. A final reach occurs on the south end of CCF where additional work would be needed to modify the inlet to the SWP Pumping Plant.

3.2 Reach Descriptions

The features of each reach are summarized in Table 3-1. A brief summary of the geography and alignment of the reaches are provided below.

Table 3-1: Summary of TDF Component Reaches

Reach	Beginning Feature	Ending Feature	Reach Description
Supplemental Intake Canal Intake Concept			
C1	Intake pumping plant.	Siphon outlet at Stone Lake Drain.	Includes pumping plant discharge pipelines to an open unlined canal.
C2	Siphon outlet at Stone Lake Drain.	North of Lost Slough at tunnel inlet.	Open unlined channel.
C3	North of Lost Slough at tunnel inlet.	Tunnel outlet south of Mokelumne River.	Single barrel, 27-foot-diameter tunnel.
C4	Tunnel outlet south of Mokelumne River.	Canal outlet at South Fork of the Mokelumne River.	Open, unlined channel.
San Joaquin River Tunnel Concept Reach			
RT1	Intake structure on Little Connection Slough north of the San Joaquin River.	Discharge to Columbia Cut south of the San Joaquin River.	San Joaquin River tunnel option (alternatively, a barrier can be constructed across the San Joaquin River). Tunnel conveys water from Little Connection Slough to Columbia Cut. Includes chevron fish screens and pumping plant with discharge to a tunnel shaft with two 33-foot-diameter bores. Mostly standard setback levees. Some offset setback levees with no alterations to existing levee to the west.

Reach	Beginning Feature	Ending Feature	Reach Description
Improved Through-Delta Corridor Reaches			
R1	Inlet structure at the Sacramento River.	Confluence of the Delta Cross Channel and Snodgrass Slough.	Strengthened existing levees.
R2	Confluence of the Delta Cross Channel and Snodgrass Slough.	Confluence of the Snodgrass Slough and Deadhorse Island Slough.	Strengthened existing levee to the west side. No new levee or levee alteration to the east side.
R3	Confluence of the Snodgrass Slough and Deadhorse Island Slough.	Confluence of the Deadhorse Island Slough and Mokelumne River.	Offset setback levee with widened existing channel to the west. No new levee or levee alteration to the east side.
R4	Confluence of the Deadhorse Island Slough and Mokelumne River.	Supplemental Intake Canal outlet.	Offset setback levee with widened existing channel to the west. Offset setback levee with no alterations to existing levee to the east.
R5	Supplemental Intake Canal outlet.	Confluence of the South Fork of the Mokelumne River and Beaver Slough.	Confluence of the Supplemental Intake Canal with protected through-Delta corridor. Offset setback levee with widened existing channel to the west. Offset setback levee with notched existing levee to the east.
R6	Confluence of the South Fork of the Mokelumne River and Beaver Slough.	Confluence of the South Fork of the Mokelumne River and Hog Slough.	Offset setback levee with widened existing channel to the west. Standard setback levee to the east.
R7	Confluence of the South Fork of the Mokelumne River and Hog Slough.	Confluence of the South Fork of the Mokelumne River and Sycamore Slough.	Offset setback levee with widened existing channel to the west. Standard setback levee to the east.
R8	Confluence of the South Fork of the Mokelumne River and Sycamore Slough.	Confluence of the South Fork of the Mokelumne River and Little Potato Slough.	Standard setback levees and offset setback levees with notched existing levees on both sides.
R9	Confluence of the South Fork of the Mokelumne River and Little Potato Slough.	Confluence of the Little Potato Slough and White Slough.	Offset setback levee with widened existing channel to the west. Strengthened existing levee and standard setback levee to the east.
R10	Confluence of the Little Potato Slough and White Slough.	Confluence of the Little Potato Slough and both the Little Connection Slough and Potato Slough.	Mostly standard setback levees with some strengthened levees.
R11	Confluence of the Little Potato Slough and both the Little Connection Slough and Potato Slough.	Confluence of the Little Connection Slough and the San Joaquin River.	Offset setback levee with no alterations to existing levee to the west. Standard setback levee to the east.
R12	Confluence of the Little Connection Slough and the San Joaquin River.	Confluence of the San Joaquin River and the Columbia Cut.	Offset setback levees with no alterations to existing levees and standard setback levees on both sides.

Reach	Beginning Feature	Ending Feature	Reach Description
R13	Confluence of the San Joaquin River and the Columbia Cut.	Confluence of the Columbia Cut and Middle River.	Offset setback levee with no alterations to existing levee to the west. Offset setback levee with widened existing channel to the east.
R14	Confluence of the Columbia Cut and Middle River.	Confluence of the Middle River and Connection Slough.	Offset setback levees with no alterations to existing levees and standard setback levees.
R15	Confluence of the Middle River and Connection Slough.	Confluence of the Latham Slough and the Empire Cut.	Mostly standard setback levees surrounding the inundated Mildred Island.
R16	Confluence of the Latham Slough and the Empire Cut.	Confluence of the Middle River and the channel cut along the north side of Woodward Island.	Standard setback levees.
R17	Confluence of the Middle River and the channel cut along the north side of Woodward Island.	Confluence of the Middle River and the Woodward Canal.	Mostly standard setback levees.
R18	Confluence of the Middle River and the Woodward Canal.	Confluence of the Middle River, Trapper Slough and the Victoria Canal.	Along this reach there are a combination of four different levee types: standard setback levees, offset setback levees with no alterations to existing levees, offset setback levees with widened existing channel and offset setback levees with notched existing levees.
R19	Confluence of the Middle River, Trapper Slough and the Victoria Canal.	Old River Siphon.	Widened Victoria Canal with new fish salvage facility and pumping plant. Discharge to siphon under Old River.
R20	Old River Siphon.	CCF.	Open, unlined channel from siphon under Old River to culvert crossing West Canal and discharging into CCF.
R21	CCF.	End of inlet channel to Tracy Fish Screen Facility.	Open unlined channel from CCF to Tracy Fish Screen Facility. Includes two new gate structures. One in the new canal and one to isolate Old River from the channel to the W.R. Jones Pumping Plant.

CCF = Clifton Court Forebay
CHTR = collection, handling, transport, and release
TDF = Through-Delta Facility

3.2.1 Supplemental Intake Canal Intake Concept Reaches

Reach C1 – Intake Pumping Plant to Stone Lake Drain Siphon

Reach C1 starts at the connection to the pumped intakes on the Sacramento River near Hood and ends at the siphon structure located at the south side of the Stone Lake Drain. The intakes are located at two separate points on the Sacramento River. The intake headers carry the water under State Route (SR) 160 to the intake pumping plants. The intake pumping plants discharge into an unlined canal. From the intake headers, the alignment crosses through farmland. The canal travels southeast towards the Union Pacific Railroad (UPRR) tracks. The canal then runs south, parallel and adjacent to the abandoned UPRR tracks. The canal centerline is offset approximately 600 feet west of the tracks. The canal crosses beneath 115-kilovolt (kV) overhead power lines and also crosses under Lambert Road. Reach C1 terminates at the downstream end of the Stone Lake Drain siphon. This siphon passes below the abandoned UPRR tracks.

Throughout Reach C1, the top of the canal embankment would be at an approximate elevation (EL) of 32 feet. Most of the natural topography of the farmland through which the canal passes varies in elevation from 5 to 13 feet above sea level. The canal does pass through one local depression where the elevation is near or slightly below sea level. Locating the canal along the west side of the railroad tracks avoids potential seasonal wetlands that may lie along the east side of the tracks.

Reach C2 – Stone Lake Drain Siphon to Lost Slough/Mokelumne River Tunnel

The second reach begins at the siphon outlet south of the Stone Lake drain and turns southeast. The canal passes under Twin Cities Road and then ends 500 feet north of Lost Slough at the first of two tunnel shafts.

Reach C3 – Lost Slough/Mokelumne River Tunnel.

Reach 3 is a single barrel, 27-foot-diameter tunnel to avoid interference with the existing floodway. It begins at the upstream tunnel shaft located 1,300 feet north of Lost Slough. The tunnel runs approximately 9,400 feet south beneath Lost Slough, the McCormack Williamson Tract and the Mokelumne River. The tunnel and Reach 3 end 3,800 feet south of the Mokelumne River at the downstream tunnel shaft. This alignment provides the shortest stretch of tunnel under Lost Slough, the McCormack Williamson Tract, and the Mokelumne River.

Reach C4 – Lost Slough/Mokelumne River Tunnel to South Fork of the Mokelumne River Canal Outlet.

The fourth reach starts at the tunnel shaft south of the Mokelumne River and terminates where it joins with the South Fork of the Mokelumne River near Beaver Slough. From the tunnel shaft, the canal continues south crossing West Lauffer Road, then bends west to avoid developments in the outskirts of the City of Thornton. The canal continues south until joining with the South Fork of the Mokelumne River approximately 3,000 feet north of the confluence of Beaver Slough. The canal crosses a gas pipeline 2,500 feet south of the tunnel shaft. Prior to joining with the South Fork of the Mokelumne River the canal runs beneath West Walnut Grove Road.

Throughout Reach 4, the top of the canal embankment would be at an approximate elevation of 32 feet. The natural topography throughout Reach 4 is relatively flat, varying from 1 foot below sea level to 4 feet above sea level. The canal passes through various parcels of farmland and, when possible, the alignment avoids splitting parcels by traveling along property boundaries. Reach 4 lies above existing natural gas fields. There are four abandoned gas wells within the proposed footprint of the canal. The alignment minimizes the interference with these gas wells.

3.2.2 Improved Through-Delta Corridor Reaches North of the San Joaquin River

Reach R1 – Sacramento River Inlet to DCC/Snodgrass Slough

Reach R1 is the first reach that consists of an existing river channel along which the levees would be fortified, as opposed to the reach consisting of a constructed conveyance feature such as a canal (thus, the reach designation of “R,” instead of “C”). Reach R1 begins at the existing radial gate inlet structure that allows water to pass from the Sacramento River into the DCC. The reach extends the length of the DCC to its confluence with Snodgrass Slough. Continued operation of the DCC is an integral feature of the TDF Option.

The DCC is situated just northeast of the city of Walnut Grove and just south of Delta Meadows State River Park. Situated in close proximity to the channel on both sides are communication towers, each with cable stays that extend down to anchorages near the channel levees.

Due to the proximity of existing infrastructure near Walnut Grove, as well as concerns for potential impacts to cultural resource features, no new setback levees would be constructed along Reach R1. Rather, the existing levees on both sides of the channel would be strengthened and their crest elevations raised, resulting in strengthened existing levees (e.g., armoring). At the radial gates dividing the channel from the Sacramento River, the new levee crest elevations would tie in to the existing levee elevations. New levee crest elevations would increase moving downstream along the length of the reach.

Reach R2 – DCC/Snodgrass Slough to Snodgrass Slough/Deadhorse Island Slough

Reach R2 begins at the confluence of the DCC with Snodgrass Slough and extends along Snodgrass Slough to its confluence with Deadhorse Island Slough. This part of Snodgrass Slough is bounded to the west by the same levee (an extension thereof) that bounds the southern side of the DCC. Similar concerns exist for this part of the levee as for the part along the DCC, notably the proximity to the communication tower and anchorages. Therefore, the existing levee would be strengthened and its crest elevations raised to create a strengthened existing levee.

On the opposite side of Snodgrass Slough, along the southwestern margin of the McCormack Williamson Tract, no new levee or alteration to the existing levee is sited. The existing levee may be susceptible to failure or overtopping, but such an occurrence would not cause a significant loss of water from the conveyance. A short-term flooding condition would give way to a relatively static inundation condition, and water would not continue to be lost from the conveyance. Due to the surrounding river system, floodwaters on the McCormack Williamson Tract would be expected to return to Snodgrass Slough, Deadhorse Island Slough, or the Mokelumne River, each of which is part of the TDF Option. Additionally, much of the McCormack Williamson Tract has been identified as an environmental habitat area. Because levee failure or overtopping would not result in significant loss of water from the conveyance and environmental areas of the McCormack Williamson Tract would not result in a significant loss, no new levee strengthening is planned for the levee along the eastern side of Snodgrass Slough.

Reach R3 – Snodgrass Slough/Deadhorse Island Slough to Deadhorse Island Slough/Mokelumne River

Reach R3 begins at the confluence of Snodgrass Slough with Deadhorse Island Slough and extends the length of Deadhorse Island Slough to its confluence with the Mokelumne River. Deadhorse Island Slough is bounded to the east by the same levee (an extension thereof) that bounds the eastern side of Snodgrass Slough through Reach 2. Similar considerations exist for this part of the levee as for the part along Snodgrass Slough. Therefore, no new levee or alteration to the existing levee is sited along the eastern side of Deadhorse Island Slough.

On the opposite side of Deadhorse Island Slough, along the eastern part of Deadhorse Island, a new setback levee would be constructed. Deadhorse Island Slough currently does not have sufficient capacity to reliably carry design flows; therefore, the new setback levee would be an offset setback levee with a widened existing channel. The new offset setback levee with a widened existing channel runs the full length of Deadhorse Island Slough.

The potential setback levee alignment locations shown in this report correspond to the widest width that may be required. Actual channel width required would likely be somewhat less than that corresponding to the levee alignment location shown. In such case, the setback levee alignment location would be closer to the existing levee and river than is shown in this report.

At the upstream end of the reach, at the confluence of Snodgrass Slough and Deadhorse Island Slough, an operable barrier would be located crossing Snodgrass Slough. An operable barrier is needed at this location (use of operable barriers is described elsewhere in this report; refer to Section 17.0).

Reach R4 – Deadhorse Island Slough/Mokelumne River to Supplemental Intake Canal

Reach R4 begins at the confluence of Deadhorse Island Slough with the Mokelumne River and extends downstream into the South Fork of the Mokelumne River and along the river to the location of the outlet of the Supplemental Intake Canal. At the confluence of Deadhorse Island Slough with the Mokelumne River, the Mokelumne River splits into the North Fork and South Fork. An operable barrier is located crossing the North Fork of the Mokelumne River. An operable barrier is needed at this location for potential emergency conditions when river flows could be maintained within the conveyance and directed along the South Fork of the Mokelumne River, as opposed to allowing flows to split at the conveyance and much of the flow to run down the North Fork of the Mokelumne River.

The stretch of South Fork of the Mokelumne River that extends through Reach R4 does not have sufficient capacity to reliably carry design flows. Therefore, a new offset setback levee with a widened existing channel would be constructed along most of the length of Reach R4. For most of this length (the downstream portions), the new offset setback levee with a widened existing channel is situated along the western side of the river, along the northeastern end of Staten Island.

The setback levee alignment location shown on Figure 15-1 corresponds to the widest width that may be required. Actual channel width required would likely be somewhat less than that corresponding to the levee alignment location shown. In such case, the setback levee alignment location would be closer to the existing levee and river than is shown in this report.

The opposite side of the river, which forms the southwestern margin of New Hope Tract, would not be widened along most of the length through Reach R4. Rather, adjacent setback levees would run along most of the length of this side of the river. Relatively short lengths of offset setback levees also would be situated along this side of the river.

At the upstream end of the reach, at the confluence of Deadhorse Island Slough and the Mokelumne River, the typical configuration for an offset setback levee with a widened existing channel is modified to avoid a habitat area. Where Walnut Grove Road crosses the Mokelumne River, immediately southwest of New Hope Landing, the existing levee contains habitat that would be destroyed using the typical configuration for an offset setback levee with a widened existing channel. Instead of removing this levee and the habitat, a new levee would be constructed adjacent to the current landside of the existing levee for the length of the habitat. Upstream and downstream of this area, the existing levee would be removed (per the typical configuration), resulting in an island at this location. The levees were situated to minimize the affects to New Hope Landing.

An additional consideration in selecting this configuration was potential impacts to New Hope Landing. If the existing South Fork of the Mokelumne River channel was to be relied upon as the primary route of conveyance, the eastern levee of the river would need to be strengthened or a new setback levee constructed. If the existing levee were strengthened or if an adjacent setback levee was constructed, construction would necessarily result in destruction of much of New Hope Landing. If an offset setback levee with no alteration of existing levee was constructed, risk of flooding at New Hope Landing would likely be increased.

In this area, Walnut Grove Road crosses the river on an existing fixed bridge. Widening of the river channel and construction of new, taller levees would require a new bridge to be constructed for Walnut Grove Road. Refer to Section 12.0 of this report for further discussion of new bridge construction.

Reach R5 – Supplemental Intake Canal to South Fork of the Mokelumne River/Beaver Slough

Reach R5 begins at the location of the outlet of the Supplemental Intake Canal along the South Fork of the Mokelumne River and extends down the South Fork of the Mokelumne River to its confluence with Beaver Slough. As for Reach R4, the stretch of South Fork of the Mokelumne River that extends through Reach R5 does not have sufficient capacity to reliably carry design flows. Therefore, the offset setback levee with a widened existing channel along the western side of the river extends downstream throughout the length of Reach R5.

The setback levee alignment shown in this report is conservative. Actual channel width required would likely be somewhat less and closer to the existing levee and river than is shown in this report.

The existing levee along the opposite side of the river, at the southwestern corner of New Hope Tract, is winding and follows sharp bends in the river. Along this stretch of river, a new offset setback levee with notching of existing levee would be constructed, and the area between the new levee and existing levee would be used for habitat restoration. The ground surface elevation in the area between the new levee and existing levee (the intra-levee area), which is currently below the river elevation, would be raised using native soils generated from excavations along the levee alignment. The existing levee would be notched to allow water to flow in and out of the intra-levee area.

Reach R6 – South Fork of the Mokelumne River/Beaver Slough to South Fork of the Mokelumne River/Hog Slough.

Reach R6 begins at the confluence of the South Fork of the Mokelumne River with Beaver Slough and extends down the South Fork of the Mokelumne River to its confluence with Hog Slough. An offset setback levee with a widened existing channel along the western side of the river extends downstream throughout the length of Reach R6.

The setback levee alignment location shown in this report corresponds to the widest width that may be required. Actual channel width required would likely be somewhat less than that corresponding to the levee alignment location shown. In such case, the setback levee alignment location would be closer to the existing levee and river than is shown in this report.

Like for the upstream end of Reach R4, it is desirable to protect the habitat area situated along the western existing levee just downstream of the confluence at Beaver Slough. Similar to the approach used at Reach R4, instead of removing the existing levee at this location, a new levee would be constructed adjacent to the current landside of the existing levee for the length of the habitat. Upstream and downstream of this area, the existing levee would be removed (per the typical configuration), resulting in a long island at this location.

Along most of the eastern side of the river through Reach R6 there would be an adjacent setback levee. A relatively short length of offset setback levee with notching of existing levee would be constructed at a sharp bend in the river, providing opportunity for on-site disposal of excavated native soils and for habitat restoration.

Reach R7 – South Fork of the Mokelumne River/Hog Slough to South Fork of the Mokelumne River/Sycamore Slough.

Reach R7 begins at the confluence of the South Fork of the Mokelumne River with Hog Slough and extends down the South Fork of the Mokelumne River to its confluence with Sycamore Slough. An offset setback levee with a widened existing channel along the western side of the river extends downstream throughout most of the length of Reach R7. The existing channel capacity is generally greater along this reach than the upstream reaches of the South Fork of the Mokelumne River, and so the setback levee alignment location is not as far offset from the existing levee as for upstream reaches.

Along most of the eastern side of the river through Reach R7 there would be an adjacent setback levee. A relatively short length of offset setback levee with notching of existing levee would be constructed at a sharp bend in the river, providing opportunity for on-site disposal of excavated native soils and for habitat restoration.

At the upstream end of Reach R7, however, is an existing habitat area that is situated immediately adjacent to the eastern existing levee, just downstream of the confluence at Hog Slough along the northwestern margin of Brack Tract. To minimize the area of impact along this habitat area, the existing levee would be strengthened and raised instead of constructing a new setback levee along this portion of the alignment.

Reach R8 – South Fork of the Mokelumne River/Sycamore Slough to South Fork of the Mokelumne River/Little Potato Slough.

Reach R8 begins at the confluence of the South Fork of the Mokelumne River with Sycamore Slough and extends down the South Fork of the Mokelumne River to its confluence with Little Potato Slough. The river channel generally has greater flow carrying capacity along this reach than along upstream reaches, and widening of the channel is not required.

This stretch of the South Fork of the Mokelumne River has some relatively sharp bends and correspondingly winding levees. Therefore, the new setback levees along the reach, on both sides, are mostly a combination of interspersed adjacent setback levees and offset setback levees with notching of existing levees.

Protection from future inundation risks for this area is provided by the existing levee, which would not be strengthened as part of this conveyance project. At one location along the eastern side of the river, along the western margin of Terminous Tract, is an existing park that is situated along a sharp curvature of the existing levee. If the existing levee were strengthened or if an adjacent setback levee was constructed, construction would necessarily result in destruction of much of the park. Therefore, an offset setback levee with no alteration of existing levee was selected for this location. It should be noted that use of this type of levee leaves the intra-levee area and the undisturbed feature (i.e., the park) on the waterside of the new setback levee.

Reach R9 – South Fork of the Mokelumne River/Little Potato Slough to Little Potato Slough/White Slough

Reach R9 begins at the confluence of the South Fork of the Mokelumne River with Little Potato Slough and extends down Little Potato Slough to its confluence with White Slough. At the upstream end of the reach, at the confluence of the South Fork of the Mokelumne River and Little Potato Slough, an operable barrier is located crossing the South Fork of the Mokelumne River. An operable barrier is needed at this location for design conditions when river flows could be maintained within the conveyance and directed along Little Potato Slough, as opposed to allowing flows to split at the conveyance and much of the flow to continue downstream along the South Fork of the Mokelumne River. Additionally, the barrier is needed to separate the conveyance flows from adjacent waters downstream in the South Fork of the Mokelumne River during an emergency condition, should it occur, wherein nearby existing levees fail and high salinity content water intrudes into this portion of the Delta.

An offset setback levee with a widened existing channel is located along the western side of the river for much of the reach.

Downstream, the offset setback levee transitions to an adjacent setback levee, such that river flows are directed along the existing river channel. The adjacent setback levee on the western side of the river transitions to an offset setback levee with notching of existing levee at the downstream end of the reach. The intra-levee area bounded by this offset setback area is relatively large. Also, at this location, at the eastern end of Bouldin Island, the existing ground surface elevation is below the river elevation. It is not yet known to what thickness native soils would be placed within the intra-levee area. The type of habitat restoration environment created in this area may differ somewhat from those created further north on the alignment.

On the opposite side of the river, a smaller intra-levee area is formed just north of the confluence at White Slough by an offset setback levee with notching of existing levee. North of this area, a new adjacent setback levee runs along approximately half the length of the reach.

Along the upstream portion of the reach, though, a strengthened existing levee is planned along the eastern side of the river. The existing levee here runs adjacent to the developed area known as Terminous (also referred to as Tower Park, which is the name of the marina that runs along the levee). To the landside of the levee is residential development, and on the levee itself are commercial buildings mostly associated with the marina. Marina facilities run along the waterside of the existing levee. Foundation improvement and earthwork to raise the levee would be key elements of the design.

Consideration was given to avoiding disturbing this area by routing the alignment eastward of Terminous, creating an offset setback levee with no alteration of existing levee. The alignment selected was based on considerations of improved flood protection for the residential portions of Terminous. If it is decided, however, to change the levee alignment to the offset setback levee with no alteration of existing levee option, the alignment would be located immediately east of Terminous. The offset setback levee would tie in to the adjacent setback levee at the first river bend southeast of Terminous, near the former Westgate Landing site. The offset setback levee would be aligned parallel to and along the agricultural boundary/dirt road that extends northward from this river bend, then cross State Highway 12 and angle northwestward to tie in to the adjacent setback levee at the southeastern-most bend of the South Fork of the Mokelumne River.

Also in this area, at the northern side of Terminous, State Highway 12 crosses Little Potato Slough on an existing bridge. Widening of the river channel and construction of new, taller levees would require major modifications and partial reconstruction of this bridge.

Reach R10 – Little Potato Slough/White Slough to Little Potato Slough/Little Connection Slough/Potato Slough.

Reach R10 begins at the confluence of Little Potato Slough with White Slough and extends down Little Potato Slough to its confluence with Little Connection Slough and Potato Slough. Most of the length of this reach has adjacent setback levees on both sides. Toward the downstream end of the reach, at the southeastern-most tip of Bouldin Island, an offset setback levee with notching of existing levee is planned.

On the opposite side of the river, a wooded area is situated adjacent to the landside of the existing levee. At this location, along the western margin of Empire Tract, the wooded riparian area surrounds a pond formed by a scour hole. The scour hole formed during a breach of the adjacent levee by the scouring action of water passing through the breach. The area of the feature is roughly 1,000 feet by 2,000 feet in plan dimensions. To protect the wooded area, a strengthened existing levee is planned for this location instead of an adjacent setback levee to reduce the area of impact. A special consideration for strengthening the existing levee at the scour hole feature is that construction operations—notably foundation improvements—would likely reduce levee underseepage.

Consideration was given to avoiding disturbing this area by routing the alignment easterly around it, creating an offset setback levee with no alteration of existing levee. Such an alignment would extend parallel to the northeasterly side of the scour hole feature, bend around the eastern end of the feature, then angle southwesterly back toward the river, to just south of the confluence of Little Potato Slough and Little Connection Slough. This alternative alignment was not selected due to its notably greater length, overall footprint area, and volume of import materials needed for construction, and also considering that the offset levee would not provide protection for the feature from inundation due to failure or overtopping of the existing levee (whereas strengthening the existing levee does provide such protection).

Reach R11 – Little Potato Slough/Little Connection Slough/Potato Slough to Little Connection Slough/San Joaquin River.

Reach R11 begins at the confluence of Little Potato Slough with Little Connection Slough and Potato Slough and extends down Little Connection Slough to its confluence with the San Joaquin River. At the upstream end of the reach, at the confluence of Little Potato Slough, Little Connection Slough, and Potato Slough, an operable barrier is located crossing Potato Slough. An operable barrier is needed at this location (see Section 17.0).

Just south of the operable barrier, adjacent to the existing levee along the eastern margin of Venice Island, is a pond area formed by a scour hole of slightly smaller dimensions than the scour hole feature on the eastern side of Reach R10. Another slightly larger scour hole pond surrounded by riparian wooded area is located near the southern end of Reach R11, near the southeastern tip of Venice Island. To protect the wooded area, levee alignments were selected to avoid these features. Consequently, an offset setback levee with no alteration of existing levee was selected for the entire western side of this reach of the alignment. Use of a setback levee avoids direct disturbance of these two areas and also avoids the risk of cutting off underseepage below the existing levees that may be providing continuing water supply to the features. The setback levee alignment selected is approximately the same length as the alignment would otherwise be if it ran adjacent to the existing levees.

Along the eastern side of the river along Reach R11, adjacent setback levees extend the length of the reach.

Reach R12 – Little Connection Slough/San Joaquin River to San Joaquin River/Columbia Cut

Reach R12 begins at the confluence of Little Connection Slough with the San Joaquin River and extends upriver along the San Joaquin River to its confluence with Columbia Cut. Along each side of this reach are interspersed adjacent setback levees and offset setback levees with no alteration of existing levees. A relatively large intra-levee area is created by the offset setback levee along the western side of the reach, on Medford Island. Medford Island has been identified as having multiple types of habitats. Considering the various types of levee configurations and lengths of alignments and corresponding footprint areas, an offset setback levee alignment location extending across the southeastern part of the island was selected as being the most likely to have the least impact.

At the northern end of Medford Island, at the beginning of Reach R12, the San Joaquin River crosses the western boundary of the reach and of the conveyance. At the current stage of design, it is not yet known whether the San Joaquin River may be left hydraulically connected to the rest of the conveyance system and have the system function as intended. High salinity content water may intrude into the conveyance system via the San Joaquin River.

An alternative to leaving the conveyance hydraulically connected to the central part of the Delta via the San Joaquin River is to construct an operable barrier crossing the San Joaquin River. The San Joaquin River is not only a major river, it is also the route of a deep water ship channel serving the Port of Stockton. An operable barrier across the San Joaquin River and deep water ship channel would present significant challenges and impacts.

Another alternative to leaving the conveyance hydraulically connected to the central part of the Delta via the San Joaquin River is to construct a diversion off of Little Connection Slough, route the diverted flows into a tunnel that passes underneath the San Joaquin River, and return the flows to the conveyance into Middle River south of Medford Island. This San Joaquin River Tunnel sub-option is discussed further in subsequent sections, as Reach T1.

Reach R13 – San Joaquin River/Columbia Cut to Columbia Cut/Middle River

Reach R13 begins at the confluence of the San Joaquin River and Columbia Cut and extends the length of Columbia Cut to its confluence with Middle River. Along the northwestern side of Columbia Cut, running along the southern part of Medford Island, a series of offset setback levee with no alteration of existing levee, adjacent setback levee, and strengthened existing levee was selected as being the alignment and configuration system most likely to have the least impact on habitats on Medford Island and along the waterway.

Columbia Cut generally does not have sufficient capacity to reliably carry design flows. Therefore, an offset setback levee with widened existing channel is located along the southeastern side of the channel for the length of the reach.

Reach R14 – Columbia Cut/Middle River to Middle River/Connection Slough

Reach R14 begins at the confluence of Columbia Cut and Middle River and extends upstream along Middle River to its confluence with Connection Slough. At the beginning of the reach, at the confluence of Columbia Cut and Middle River, an operable barrier is located crossing Middle River. An operable barrier is needed at this location to separate the conveyance flows from adjacent waters in Middle River during an emergency condition, should it occur, wherein nearby existing levees fail and high salinity content water intrudes into this portion of the Delta.

Along the length of the reach, on both sides of the river, is a combination of adjacent setback levees and offset setback levees with no alteration of existing levees. This combination of levees was selected primarily to minimize length, overall footprint area, and volume of import materials needed for construction. The intra-levee areas associated with the offset setback levees collectively are relatively large. It is unlikely that the volume of native soils that would be generated from construction excavation in adjacent areas would be sufficient to fill the intra-levee areas up to the levee of the river, and so these intra-levee areas would be inundated if the existing levees were notched. The existing habitat in the intra-levee areas associated with the offset setback levees was considered likely to be of higher value than habitat formed by inundation of these areas, especially considering the close proximity of Mildred Island, which is already inundated.

3.2.3 San Joaquin River Tunnel Option Reach

Reach T1

Reach T1 is the San Joaquin River Tunnel sub-option and its alignment and configurations are an alternative to those described in Reach 11 and 12, as explained above.

Reach T1 begins at the connection to the pumped intakes on Little Connection Slough and ends at the exit from the tunnel shaft structure located to the southwest, near the eastern margin of Venice Island. The intakes are located at five separate points along Little Connection Slough. The intake headers carry the water from the intake pumping plant to nearby sedimentation basins and associated facilities. From these facilities, the water enters the tunnel shaft. The facilities between the intakes and the tunnel shaft, and including the tunnel shaft, would be surrounded by levees along the western and southern sides of the facilities; these levees would tie in to the adjacent setback levees along Little Connection Slough.

The tunnel shaft consists of two tunnel barrels, each one being 33 feet in diameter (for further description of tunnel characteristics, refer to Section 11.0 of this report). The tunnel begins at the upstream tunnel shaft located near the southeastern margin of Venice Island west of Little Connection Slough and north of the San Joaquin River. The tunnel runs approximately 3.5 miles south beneath the San Joaquin River, Medford Island, and Columbia Cut. The tunnel ends approximately south of Columbia Cut and east and north of Middle River at the downstream tunnel shaft. From the downstream tunnel shaft, water is conveyed in a short canal to an outlet into Middle River. The facilities at the downstream tunnel shaft would be surrounded by levees along the northern and eastern sides of the facilities; these levees would tie in to the adjacent setback levees along Middle River.

Two locations of ventilation shafts extend up from the tunnel barrels to the ground surface on Medford Island. One such location is adjacent to the existing levee along the northeastern margin of the island; the other location is adjacent to the existing levee along the southern margin of the island.

An operable barrier is needed to separate the conveyance flows from adjacent waters in Middle River during an emergency condition, should it occur, wherein nearby existing levees fail and high salinity content water intrudes into this portion of the Delta.

Adjacent setback levees are situated along the length of the reach on the east side of the river, except for where the outlet facilities are situated and the setback levees tie in to these facilities. Along the west side of the river, an offset setback levee with no alteration of existing levee extends most of the length of the reach. The offset setback levee was selected primarily to minimize length, overall footprint area, and volume of import materials needed for construction. The intra-levee area associated with the offset setback levee is relatively large. It is unlikely that the volume of native soils that would be generated from construction excavation in adjacent areas would be sufficient to fill the intra-levee area up to the levee of the river, and so this intra-levee area would be inundated if the existing levees were notched. The existing habitat in the intra-levee area associated with the offset setback levee was considered likely to be of higher value than habitat formed by inundation of this area, especially considering the close proximity of Mildred Island, which is already inundated.

3.2.4 Improved Through-Delta Corridor Reaches South of the San Joaquin River

Reach R15 – Middle River/Connection Slough to Latham Slough/Empire Cut

Reach R15 begins at the confluence of Middle River and Connection Slough and extends upstream along Middle River, across Mildred Island (which is inundated), and along Latham Slough to the confluence with Empire Cut. At the beginning of the reach, at the confluence of Middle River and Connection Slough, an operable barrier is located crossing Connection Slough. An operable barrier is needed at this location to separate the conveyance flows from adjacent waters in Connection Slough during an emergency condition, should it occur, wherein nearby existing levees fail and high salinity content water intrudes into this portion of the Delta.

Along most of the length of the reach, on both sides of the river, are adjacent setback levees. At a sharp bend on each side of the river, a new offset setback levee with notching of existing levee would be constructed, and the area between the new levee and existing levee would be used for habitat restoration. The ground surface elevation in the intra-levee area would be raised using native soils generated from excavations along the levee alignments. The existing levee would be notched to allow water to flow in and out of the intra-levee areas, as described in preceding sections of this report.

Reach R16 – Latham Slough/Empire Cut to Middle River/Channel Cut

Reach R16 begins at the confluence of Middle River, Mildred Island (which is inundated), and Latham Slough with Empire Cut and extends up Middle River to its confluence with the channel cut along the north side of Woodward Island. Adjacent setback levees are situated along the entire reach along both sides of the river.

In the middle of this reach, Bacon Island Road crosses Middle River on an existing bridge that is fixed across its west span and is a swing bridge across its east span. New, taller setback levees at either end of the bridge would require modifications to the bridge approaches. The Burlington Northern and Santa Fe Railroad (BNSF) tracks also cross the levees to the south of West Lower Jones Road.

Reach R17 – Middle River/Channel Cut to Middle River/Woodward Canal

Reach R17 begins at the confluence of Middle River and the channel cut along the north side of Woodward Island and extends upstream along Middle River to the confluence with Woodward Canal. At the beginning of the reach, at the confluence of Middle River and the channel cut along the north side of Woodward Island, an operable barrier is located crossing the channel cut. An operable barrier is needed at this location.

At the location of the operable barrier, an existing railroad alignment is located along the middle of the channel cut, with the railroad tracks elevated above the waterway.

Along most of the length of the reach, on both sides of the river, are adjacent setback levees. At two sharp bends of the river along the western side, new offset setback levees are sited. One of these levees would be an offset setback levee with notching of existing levee, for native soil placement and habitat restoration. The other offset levee would be an offset setback levee with no alteration of existing levee. This offset levee was selected primarily to minimize length, overall footprint area, and volume of import materials needed for construction.

Reach R18 – Middle River/Woodward Canal to Middle River/Trapper Slough/Victoria Canal

Reach R18 begins at the confluence of Middle River and Woodward Canal and extends upstream along Middle River and its confluence with Trapper Slough and the New Victoria Canal. At the beginning of the reach, at the confluence of Middle River and Woodward Canal, an operable barrier is located crossing Woodward Canal. An operable barrier is needed at this location to separate the conveyance flows from adjacent waters in the channel cut during an emergency condition, should it occur, wherein nearby existing levees fail and high salinity content water intrudes into this portion of the Delta.

A combination of adjacent setback levees and offset setback levees with no alteration of existing levees are situated along the east side of Middle River through this reach. This combination of levees was selected primarily to minimize length, overall footprint area, and volume of import materials needed for construction, as well as to minimize impact to habitat areas located adjacent to existing levees.

Along the western side of the river, a portion of the reach is bounded by a new adjacent setback levee. An adjacent setback levee is not used along the remainder of the reach, however, because the capacity of the channel needs to be increased. Middle River sometimes experiences scour along the channel during high pumping rates at the South Delta pumping facilities. To reduce this occurrence, offset setback levees would be used to widen the existing channel. Offset setback levees would be constructed along the southwestern side of the channel along this stretch of the reach.

3.2.5 Victoria Canal and Clifton Court Forebay Reaches

Reach R19 – Middle River/Trapper Slough/Victoria Canal to Old River Siphon

This reach begins just before the confluence of Trapper Slough and Middle River. The existing Victoria Canal would be widened to increase its capacity. Facilities to be constructed for the Victoria Canal include a new fish salvage facility to reduce predation in CCF and a siphon under Old River. See Section 20.0 for a more detailed discussion.

Reach R20 – Old River Siphon to CCF

This reach includes a short canal section from the end of the siphon under Old River to a culvert crossing the West Canal with an outfall into CCF.

Reach R21 – CCF to Tracy Fish Screening Facility

This reach consists of a new inlet canal constructed from CCF to the upstream end of the existing Tracy Fish Screening Facility associated with the W.R. Jones Pumping Plant. The reach includes two new gate structures. One would be installed in the new canal and another one would be installed to isolate Old River from the channel to the W.R. Jones Pumping Plant.

3.3 Geology

3.3.1 Regional Geology

The Delta represents the arm of the San Francisco Bay estuary that extends into the Central Valley geomorphic province of California. The Central Valley province is a sedimentary basin (Figure 3-1), approximately 700 kilometers (km) long and up to 100 km wide, which lies between the primarily granitic mountain ranges of the Sierra Nevada province to the east and the accretionary Franciscan Complex rocks of the Coast Ranges province to the west. The Central Valley province is characterized by a large northwest trending asymmetrical synclinal trough filled with a prism of upper Mesozoic-age (approximately 135 Ma) through recent sediments up to 9 km thick (Bartow, 1991).

The geomorphology and surficial geology of the Delta have been shaped by the landward spread of tidal environments resulting from SLR after the last glacial period. During the last glacial period, approximately 15,000 years ago, the Pacific coast was at least 6 miles west of its present position, and the relative sea level was approximately 300 feet lower than today. At this time, the location of the present day Delta formed part of the arid alluvial floodplain. As a consequence, alluvial and eolian sand deposits underlie most of the late Holocene Delta soils. Between 10,000 and 5,000 years ago, relative sea-level rise was rapid, out-stripping the rate of deposition of flood-borne sediments supplied by the river systems (Byrne et al., 2001). This resulted in the landward transgression of the ocean through the Carquinez Strait and into the Central Valley, forming the Suisun Bay and the Delta. This period of time saw the widespread deposition of organic silt and clay across the alluvial floodplain surface. Approximately 5,000 years ago, relative SLR slowed, halting landward transgression of the tidal wetlands. At this time, the deltaic environment remained in approximately its present position, with slow relative sea-level rise balanced by vertical marsh growth through biomass accumulation and sediment deposition (Atwater and Belknap, 1980).

3.3.2 Project Area Geologic Units

Geologic units exposed within the study area consist predominantly of Holocene deposits of alluvial and tidal environments. These deltaic deposits are underlain by alluvial fan and eolian deposits of Holocene and Pleistocene age, derived from the drainage basins in the Sierran and Coastal ranges to the east and west. As indicated above, these surficial geologic units are underlain by a massive thickness of upper Mesozoic and Cenozoic sediments. These sediments form a broad syncline with progressively older units being exposed at greater depth and at higher elevations in the mountain ranges to the east and west.

3.3.2.1 Artificial Levee Fill (Historical)

This material includes constructed levees bordering rivers, streams, sloughs, and Delta islands for the purpose of containing flood or tidal waters. The project area includes extensive levee and drainage systems constructed between the 1860s and 1930s as part of the sustained agricultural development of the Delta. These structures have been modified and raised to keep up with settlement of levees and subsidence of the interior island soils. In general, levee construction prior to 1965 (enactment of the Uniform Building Code) was conducted in a non-engineered fashion (without select materials or the use of compaction), and levee materials were generally derived from excavation and dredging of the channels and waterways. As a result, levee materials are highly variable and typically consist of mixtures of soft silts, clays, peat, and loose sands.

3.3.2.2 Alluvial Channel and Natural-levee Deposits (Holocene)

Alluvial channel and natural-levee deposits are characterized by loose, poorly graded, sandy to clayey silt and silty sands. These deposits are associated with active, historic, and prehistoric non-tidal channels. This unit is mapped only on broad natural levees and crevasse splays of the Sacramento River and its distributaries (Atwater, 1982), but is present also in the immediate vicinity of historic and prehistoric non-tidal channels in areas of undivided flood-plain alluvium. The contact with adjacent basin and tidal deposits commonly grades across tens of thousands of feet; the levees likely formed the interface between rapidly flowing and nearly standing water (Brice, 1977).

3.3.2.3 Floodplain Deposits (Holocene)

Atwater (1982) mapped this unit in the western San Joaquin and Sacramento valleys to indicate a time-transgressive floodplain of the San Joaquin River. Most, if not all, of this area has been inundated historically during large floods. Part of this area was covered historically with tidal-wetland peat, but underlying deposits have been since exhumed by wind erosion. This unit generally slopes downstream at low gradients parallel to the San Joaquin River. These deposits consist mainly of firm silty clay, micaceous silt, and micaceous sand with low organic content.

3.3.2.4 Flood-basin Deposits (Holocene)

This unit consists of sediments that accumulated from standing or slow moving water in topographic basins. Within the project area, this unit formed the supratidal reaches of basins flanking the Sacramento River and in interdistributary basins cut off from tidal waters (Atwater, 1982). Flood-basin deposits typically consist of firm to stiff silty clay, clayey silt, and silt, commonly with carbonate, and locally with oxide nodules. These deposits grade laterally into peaty mud and mud of tidal wetlands.

3.3.2.5 Peat and Peaty Mud of Tidal Wetlands (Holocene)

This unit includes sediments deposited in tidal marsh at, or near, sea level. Delta peat and mud typically have low bulk density and include silt, clay and peat with minor sand (Atwater, 1982). Organic content is highest in the central and south-central Delta, and lower in the southern-most and northern areas, where peaty mud is typically intercalated with mud in layers 1 to 10 centimeters thick (Atwater, 1982). This unit generally occupies historical lowlands (tidal wetlands and waterways) that are now dry because of the construction of dikes and levees. Many of these areas are now below sea level due to historical subsidence and deflation.

3.3.2.6 Dune Sands (Pleistocene to Holocene)

These deposits consist of very well sorted fine to medium grained eolian sand. Holocene sand may discontinuously overlie latest Pleistocene sand, both of which may form a mantle of varying thickness over older materials. Most of these deposits are thought to be associated with latest

Pleistocene to early Holocene periods of low sea level, during which large volumes of fluvial and glacially-derived sediments were blown into dunes. These materials are mapped within the project area by Atwater (1982) as eolian deposits of the upper member of the Modesto Formation and include the Oakley-Antioch dunes field.

3.3.2.7 Older Alluvium (Pleistocene)

This general description of the older alluvium applies to the Pleistocene Modesto, Riverbank, Montezuma, Turlock Lake, and Red Bluff Formations. These deposits form low hills, fans and terraces, with distal ends that grade to low plains and basins and proximal ends that grade to colluvium along the foothills surrounding the valley. Typically, these units consist of tan, brown, gray, black, and red gravels, sands, silts, and clays. Lithologically, they reflect the source area, being typically lithic and non-micaceous along the flanks of the Coast Ranges, and arkosic, commonly micaceous, and including rock-flour-like silt and very fine sand derived from Pleistocene glaciation along the Sierran Range. The youngest of these deposits are unconsolidated and show minimum weathering, while the oldest display maximal weathering and are semi-consolidated.

3.3.2.8 Bedrock (Tertiary and Upper Cretaceous)

The above-described relatively poorly-consolidated to unconsolidated Quaternary deposits overlie Cretaceous- to Tertiary-age sedimentary bedrock, which is generally deeper than 1,000 feet within the project area (Brocher, 2005). For the most part, these sedimentary rocks consist of interbedded marine sandstone, shale, and conglomerate. However, deposition of shallow marine, terrestrial, and volcanoclastic sediments was predominant by the late Tertiary.

3.4 Seismic Hazards

Active faulting and earthquakes in central California result from transpressional (region of oblique shear) deformation related to movement of the North American plate to the southeast relative to the Pacific plate. Most of this movement is accommodated along the major strike-slip fault systems of the San Andreas and Hayward-Calaveras fault systems, which lie to the west of the Delta. Other strike-slip faults nearer the Delta also accommodate the motion between the tectonic plates, and some plate motion is taken up on reverse and thrust faults like those in the Coast Ranges-Sierran Block (CRSB) boundary zone.

3.4.1 Seismic Sources

A model of the active and potentially active seismogenic faults in the greater San Francisco Bay region was developed as part of the DRMS study (DWR, 2007a) (Figure 3-2). Each seismic source was characterized using the latest geologic, seismological, and paleoseismic data and the currently accepted models of fault behavior. A major study by the Working Group on California Earthquake Probabilities (WGCEP) (2003) entitled Earthquake Probabilities in the San Francisco Bay Region: 2002-2031 describes and summarizes the current understanding of the major faults in the San Francisco Bay area. The DRMS study adopted the WGCEP (2003) seismic source model for the San Andreas, Hayward/Rodgers Creek, Concord/Green Valley, San Gregorio, Greenville, and Mt. Diablo thrust faults. The characterization of the Calaveras was slightly modified by William Lettis and Associates and URS Corporation (URS) for DRMS (DWR, 2007a).

“Blind” faults beneath the Delta and the Western Tracy and Vernalis faults, part of the CRSB (Wong et al., 1988), are of particular significance to the assessment of seismic hazards in the Delta. The Delta sources include the Northern Midland zone, the Southern Midland fault, the Thornton Arch zone, and the Montezuma Hills source zone (Figure 3-2). As is the case for many “blind” faults, the characterization of the Delta seismic sources is highly uncertain because of the very limited amount of available data. What is known about these sources primarily has come from subsurface seismic data. Descriptions of the Delta faults (or fault zones) and four faults in the CRSB are shown on Figure 3-2.

3.4.1.1 Ground Motions

In the DRMS study, URS performed a Probabilistic Seismic Hazard Analysis study in which the annual probabilities of occurrence at selected times over the next 200 years (e.g., 2005, 2050, etc.) for plausible earthquake events were defined for all seismic sources that could impact the Delta (DWR, 2007a). Time-dependent seismic hazard results were computed at six sites in the Delta for the years of 2005, 2050, 2100, and 2200. A time-dependent probabilistic ground-shaking hazard map for 500-year return periods were developed for the Delta area as shown on Figure 3-3. The map is for peak ground acceleration (PGA) and a stiff soil site condition. An important point is that these maps are for a uniform site condition so site response effects are not apparent on this map.

At all return periods, the ground motions decrease from west to east due to increasing distance from the San Andreas fault system. At 100 years, the PGA values, in unit of g, range from 0.12 g in Sacramento, which is the most eastern site on the edge of the Delta faults to 0.27 g at Montezuma Slough. The latter site is located adjacent to the Pittsburg-Kirby Hills fault. The controlling seismic source varies from site to site but the Southern Midland fault and Northern Midland zone are major contributors to several sites.

In the 2002 version of the United States Geological Survey (USGS) National Hazard Maps, which are the basis for the International Building Code, Frankel et al. (2002) estimated probabilistic ground motions for the United States for the exceedance probabilities of 2 percent (%), 5%, and 10% in 50 years (return periods of 2475, 975, and 475 years, respectively). The maps are for a firm rock site condition (National Earthquake Hazards Reduction Program site class B/C) so a direct comparison with the firm soil results of the DRMS study is not possible. The USGS values for a 500-year return period range from approximately 0.14 g to 0.40 g. The firm soil values in the DRMS study range from approximately 0.20 g to 0.50 g (DWR, 2007a). The difference can be attributed to site amplification of the soil versus the USGS firm rock ground motions. The DRMS earthquake ground motions also were compared to an earlier Department of Water Resources (DWR) study and to a 2000 CALFED study (CALFED, 2000a). The results for the 200-year return period event were found to be very comparable.

For more detailed description of Delta ground motions, see Appendix A and the DRMS Seismology report (DWR, 2007a).

3.4.1.2 Surface Fault Rupture Hazard

None of the faults or fault sources in the Delta are known to have produced surface rupture in the Holocene (approximately the last 12,000 years). Of the four faults described in Appendix A, the Southern Midland fault is perhaps the most likely to rupture to the ground surface during a future earthquake. Recent research described in the DRMS Seismology report (DWR, 2007a) indicates that the Southern Midland fault may offset the contact between Holocene peat deposits and the underlying sandy deposits approximately 2 to 4 meters (m). However, this relationship is not well constrained; and it is possible that the apparent offset may result from landscape features existing prior to encroachment of sea level and formation of peat in the Delta. The above-described potentially fault-related offset of a geologic horizon thought to be 6,000 to 7,000 years old is the strongest evidence for potential surface rupture in the Delta. We judge the surface rupture hazard in the Delta to be low.

3.4.1.3 Liquefaction

Minimum penetration resistance values of levee foundation materials have been compiled from thousands of borings during the DRMS study (DWR, 2008a). A large fraction of the boring contains loose sands with blow count values less than 15. When saturated, these foundation material loose sands, which are most common in the west central part of the Delta, are highly susceptible to liquefaction. In addition, levee fills in many places are composed of silty sands that also are susceptible to liquefaction. The levees in the Delta that are composed primarily of loose saturated sandy soils, or have these soils in their foundations, may liquefy during future moderate to strong shaking, resulting in levee failure (DWR, 2008b).

3.5 Flood Protection Considerations

The conveyance options have been engineered to withstand water level rise resulting from the following potential factors or sources:

- 200-year return flood event in the Sacramento, Mokelumne, and San Joaquin rivers;
- Inundation of floodplain from a 200-year return flood event with levee breach;
- Wind-induced waves;
- Mean higher high water (MHHW) tides; and
- SLR due to climate change over the next 100 years.

Flood water levels resulting from these factors vary across the Delta depending on location and source. Water level rise would affect each conveyance option from both outside of the conveyance canal and, in TDF, within the existing channel. The specific amount of water level rise to be used in the conceptual design for each conveyance option would be included in design criteria currently being developed.

The conveyance option description in this report is based on input from DWR, the United States Army Corps of Engineers (USACE), and other sources as obtained and evaluated by the DHCCP and considers six potential flooding scenarios:

- River flooding assuming no levee failures.
- Floodplain flooding assuming multiple river levee failures or overflows.
- Island flooding limited by levee heights.
- Island flooding limited by river stage.
- Island flooding limited by flood volume.
- Tidal flooding due to SLR and assuming a levee breach without a storm flood event.

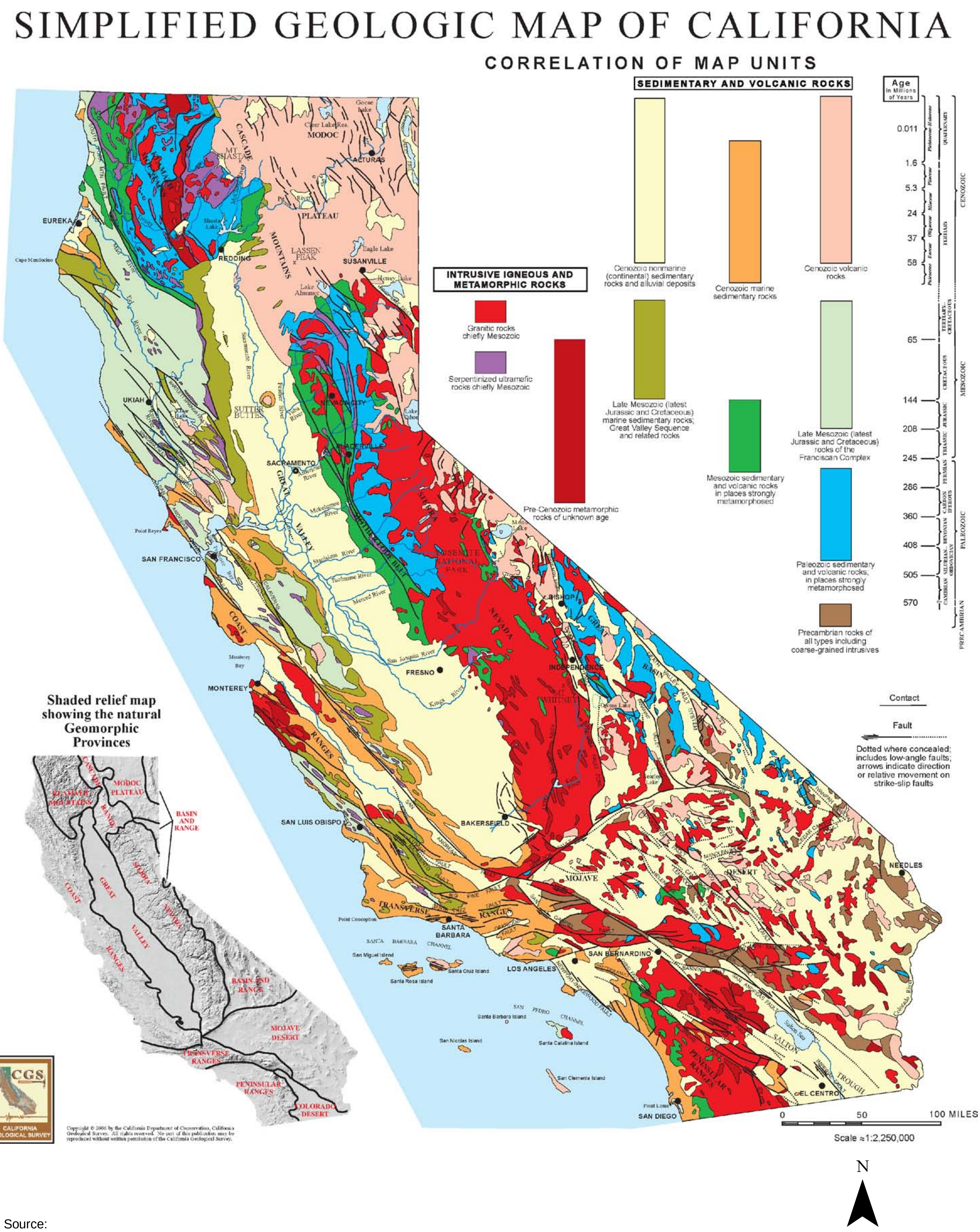
The flood levels estimated in this report are consistent with DWR's ***Proposed Interim Levee Design for Urban and Urbanizing Area State-Federal Project Levees, third edition***, published in 2009 (DWR, 2009), which states that the physical top of levee would need to be at least 3 feet higher than the 200-year flood event water surface elevation (WSE), with an additional freeboard (FB) allowance for wind wave run-up.

A value of 55 inches of SLR at the Golden Gate Bridge was used based on the recommendation of Delta Vision Blue Ribbon Task Force to Governor Schwarzenegger in September to set SLR planning standards for critical state investments. The SLR impact decreases farther inland and is estimated by a derived hydraulic relationship referenced to the SLR at the Golden Gate Bridge tide gauge location.

The DHCCP conveyance facility is considered to be a critical lifeline facility for the State of California. The proposed ICFs run through low and flat terrain in the western and eastern section of the Delta. The proposed Through-Delta facility runs along several natural channels of Delta sloughs and rivers. It is understood that these facilities must be protected from flooding, and the level of protection to be provided must be consistent with the interim guidance for urban levees (DWR, 2009) mentioned above.

The flood levels, SLR and wind wave run-up determined in the conceptual engineering phase will be further refined in the upcoming engineering phases, which will provide more accurate WSE information. A composite FB protection that considers climate change and wind wave run-up will be developed.

The recommended flood protection criteria for the conceptual and preliminary engineering phases is the 200-year flood event WSE, including SLR, with an additional FB allowance that is the higher of the 3-foot standard FB or the computed wind wave run-up.



Source:
California Geological Survey, 2006, Simplified Geologic Map of California, Map Sheet 57, California Department of Conservation

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT THROUGH-DELTA FACILITY OPTION **Regional Geologic Setting**	FIGURE **3-1**

Source: DWR, 2007a, DRMS Seismology TM

Legal Delta Boundary V. 2002-4

Surficial faults used in the hazard analysis

Blind faults used in the hazard analysis

Bounds of delta islands

CRSB - Coast Range Sierran Block

N

0 5 10

Miles

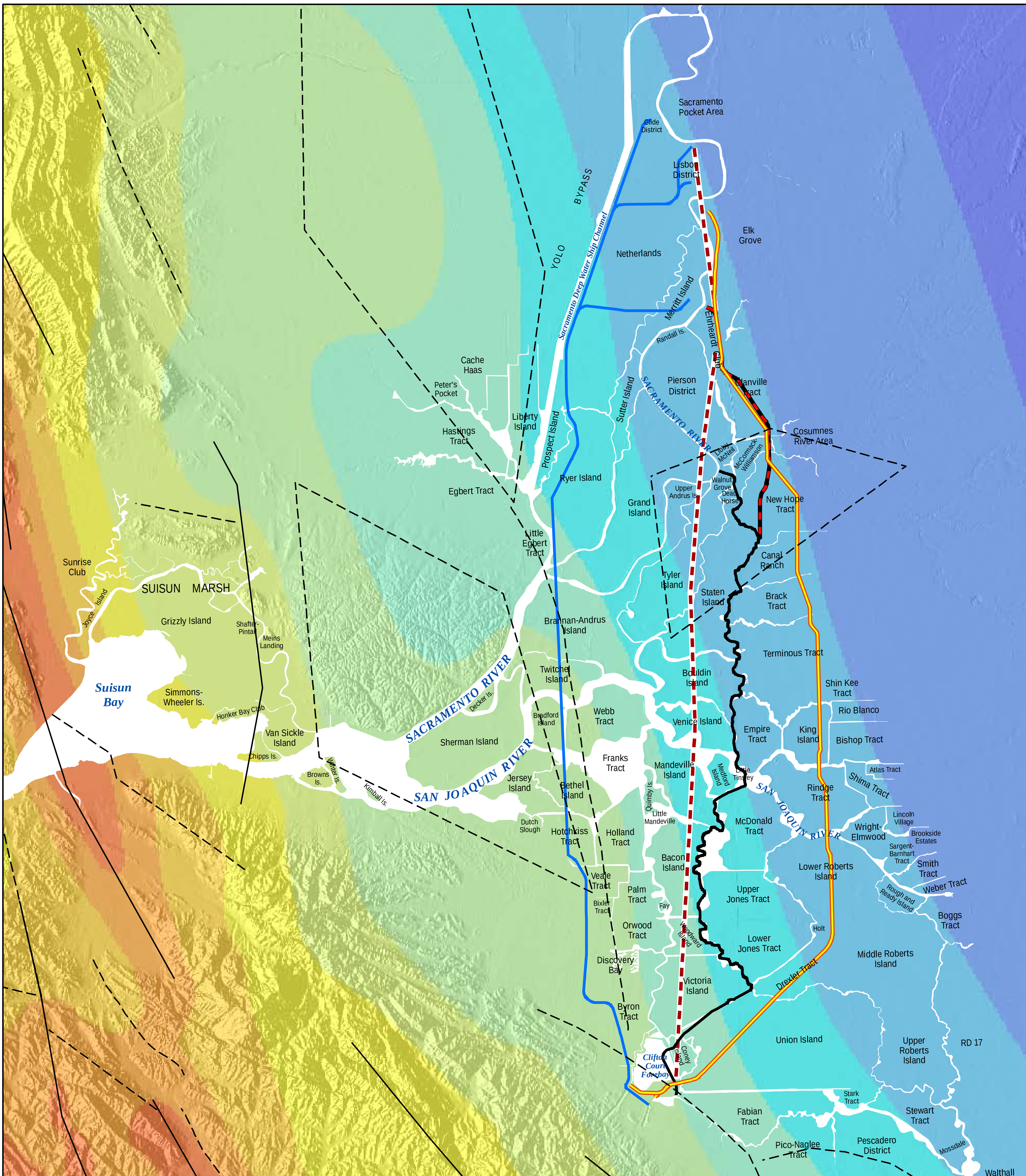
California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Active Faults in the Delta Region

FIGURE
3-2

URS Oakland - C.Raumann \\S021emc2\DRM\GIS\DHCCP\Maps\Geotechnical\DHCCP_CE_figures\Through-Delta\Figure_3-3_500yr_PGA_Hazard_TDF.mxd - 11/12/2009 @ 4:44:41 PM



Faults

- Surficial fault used in the hazard analysis
- Blind fault used in the hazard analysis

BDCP Initial Alignment Options

- East Alignment
- Through Delta Alignment, Existing Waterway
- Through Delta Alignment, Hood-New Hope Outlet
- West Alignment
- All Tunnel

PGA (g), 500-year return period

- 0.00 - 0.10
- 0.11 - 0.15
- 0.16 - 0.20
- 0.21 - 0.25
- 0.26 - 0.30
- 0.31 - 0.35
- 0.36 - 0.40
- 0.41 - 0.45
- 0.46 - 0.50
- 0.51 - 0.55
- 0.56 - 0.60
- 0.61 - 0.65
- 0.66 - 0.70

Source: DWR, 2007a, DRMS Seismology TM

N

0 4 8

Miles

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Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

PGA Hazard for a 500-year Return Period

FIGURE
3-3

4.0 CONVEYANCE SYSTEM OPERATIONS

This section describes the operation of the existing SWP and CVP Delta export facilities and operational considerations and concepts for the TDF Option.

4.1 Existing Systems and Operations

The current method for conveying water to the SWP and the CVP Delta export pumping plants is solely by through-Delta conveyance. The headwaters for the existing SWP and CVP Delta export pumps originate, predominantly, from Shasta (United States Bureau of Reclamation [Reclamation]), Oroville (Department of Water Resources [DWR)], and Folsom (Reclamation) Dams. Releases from these reservoirs into the Sacramento River are managed through the terms of the Coordinated Operations Agreement between DWR and Reclamation.

The daily allocation of water available in the Delta for export via the SWP and CVP Delta export facilities is jointly determined by DWR and Reclamation taking into account the various environmental and water quality standards, inflows from other tributaries entering the Delta, and other constraints and opportunities.

The SWP and CVP Delta export facilities each consist of a complete and separately dedicated facility with fish screening and collection, inlet channel, pumping plant, operations staffing, control philosophy and pump operating regime. The SWP and CVP Delta export pumping plants are the Harvey O. Banks (Banks) Pumping Plant and the Jones Pumping Plant, respectively.

4.1.1 SWP Delta Export Facilities

The SWP Delta export facilities comprise CCF, Skinner Delta Fish Protection Facility (Skinner Facility), and the Banks Pumping Plant.

DWR uses the National Geodetic Vertical Datum of 1929 (NGVD29) to reference water elevations at the SWP Delta export facilities. The DHCCP uses the North American Vertical Datum of 1988 (NAVD88). As part of this initial study effort, a DWR survey crew performed a closed loop survey between the Jones and Banks Pumping Plants and tied back to NAVD88 datum. The conversion between NGVD29 and NAVD88 is +2.36 feet at Banks Pumping Plant. However, the DWR survey results indicate a conversion of approximately +3.10 feet from Banks Pumping Plant elevations to NAVD88 datum. Field survey work is ongoing to determine whether CCF and Skinner Facility are also on the same relative datum as the Banks Pumping Plant. For clarity, in the remainder of this section, reported elevations at the SWP Delta export facilities are listed as “Banks site datum.”

4.1.1.1 Clifton Court Forebay

DWR’s portion of available daily water for export is moved into CCF by opening the CCF intake radial gates. Operation of these gates is scheduled to take advantage of the tidal cycle to reduce approach velocities, prevent scour in adjacent channels, and minimize water elevation fluctuation in the south Delta. The intake gates enable incoming flow into CCF to be measured and water to be stored in CCF and moved into the SWP facilities downstream at a later time, thereby enabling the SWP to maximize pumping during off-peak hours. Diversions into CCF are restricted to a peak instantaneous flow of 12,000 cfs, a daily maximum of 13,870 acre-feet (AF), and a maximum 13,250 AF per day average over any three-day period. The CCF operation is linked to the Banks Pumping Plant operation. The gates are not designed for reverse flow back into Old River.

The CCF intake gate operation and number of pumps to be operated at a given time are generally determined by DWR several days in advance. The period within the tidal cycle in which the CCF intake gates are opened is based on minimizing impacts to South Delta water users.

DWR reports that the CCF water level varies throughout the day, typically between elevation -2 feet and EL +0 to +2 feet [Banks site datum] depending on tidal conditions and predetermined CCF gate opening priority. Typical operation is targeted to restore the CCF water level to EL -1 foot [Banks site datum] each day at midnight. This creates the required head differential between the available water in the Delta and CCF to allow water to flow from the Delta into CCF to provide sufficient water for the SWP's Delta Export Allocation for the following day. The CCF gates are closed once DWR's daily allocation has been reached. If tidal or other conditions prevent DWR's daily allocation from being reached, the schedule for the following day's operation is adjusted to minimize the potential for future export shortages.

The CCF maximum design operating storage is 28,653 AF at the maximum design operating WSE of +5 feet [Banks site datum]. The minimum design operating storage is 13,965 AF at the minimum design operating WSE of -2 feet [Banks site datum] (DWR, 1974a). DWR has indicated that for future operations, unless engineering improvements are made to the perimeter embankment around CCF, the maximum operating WSE should be limited to +4 feet [Banks site datum]. Table 4-1 summarizes CCF water elevation information.

Table 4-1: CCF Operational Water Elevations (measured at CCF Inlet Gates)

Criteria	Data Used by Operators	Data in NAVD88
Vertical Datum	NGVD29[1]	Banks site datum +3.10 feet[2] = NAVD88
Minimum Operating Surface Elevation[3]	-2 feet	+1.1 feet
Typical Range in CCF water elevation under Priority 1 Gate operations[4]	Typically -2 to +0 feet	+1.1 feet to +3.1 feet
Historical Typical Range in CCF water elevation[1,5]	Typically -2 to +2 feet	+1.1 feet to +5.1 feet
Maximum Operating Surface Elevation for Future Operations[4]	+4 feet	+7.1 feet
Maximum Design Operating Surface Elevation[1, 3, 4]	+5 feet	+8.1 feet

1. According to pumping plant operators.
2. Sacramento-San Joaquin Delta (DWR) survey results at Banks Pumping Plant, February 2009, assumed to also apply at CCF.
3. Source: DWR, 1974a (Bulletin No, 200, November 1974, DWR California State Water Project).
4. As reported by DWR.
5. As reported by DWR.

CCF = Clifton Court Forebay
DWR = Department of Water Resources
feet^2 = square feet
NAVD88 = North American Vertical Datum of 1988
NGVD29 = National Geodetic Vertical Datum of 1929

4.1.1.2 Skinner Facility

The Skinner Facility is located on the California Aqueduct Approach Channel between CCF and the Banks Pumping Plant. Under peak pumping operations, head loss across the screens is reported to be 1.5 feet. The Skinner Facility has a maximum operating EL of +10.6 before overtopping.

4.1.1.3 Banks Pumping Plant

The Banks Pumping Plant has an installed capacity of 10,670 cfs and presently takes water from CCF via the California Aqueduct Approach Channel downstream of the Skinner Facility. The pump plant comprises 11 pumps: 9 large pumps (5 at 1,130 cfs each, 4 at 1,067 cfs each) and 2 smaller pumps with 375 cfs capacity each. The pumps are configured with four groups of two large pumps each which discharge into a single 15-foot-diameter discharge pipeline conveying flow up to a short section of canal (approximately 1 mile) and then into the Bethany Reservoir. The output of each large pump group is 2,260/2,134 cfs. In addition, one 1,130-cfs large pump and two 375-cfs pumps discharge to a single 13.5-foot-diameter discharge line. There are flow meters on each of the five discharge lines.

The existing Banks Pumping Plant is designed to operate with a WSE just upstream of the pumping plant ranging from -0.9 to +8.1 feet (NAVD88) (DWR, 1999). Per DWR operating personnel, the Banks Pumping Plant could operate to a maximum of +13.1 feet but is constrained to a maximum of +10.6 feet, the overtopping limit at the Skinner Facility.

The Banks Pumping Plant is generally operated on an on-peak/off-peak schedule. The pumping schedule maximizes pumping of the SWP's export allocation from CCF into the Bethany Reservoir and SWP conveyance system downstream during the off-peak hours and spreads additional pumping at a reduced rate during on-peak hours if conditions dictate. The off-peak hours are typically 10 p.m. to 7 a.m. Monday through Saturday and all day Sunday as well as some holidays throughout the year.

The magnitude of the Banks Pumping Plant electrical power demand has the following consequences on existing operations (for additional information regarding power supply, see Section 19.0):

- DWR coordinates closely with both California Independent System Operator (CAISO) (the wholesale power grid operator) and DWR's in-house power generation operations and provides pump operation schedules several days in advance.
- The number of pumps in operation is often varied on an hourly basis during periods of operation to assist in the optimization of power usage in the local grid. Typically, the Banks Pumping Plant has been operated to maximize pump operation during the period midnight to 6 a.m. with the number of pumps stepped up to peak output of 9,790 cfs. Per DWR operating personnel, typical operation is to initially ramp up to a maximum of 5,650 cfs (five units) in the initial hour and the remaining required units on the next step up. Depending on the flow demand, it is more efficient to run one unit per discharge line. The individual pumps are started at 5-minute intervals both to reduce the instantaneous power demand increase on the system and also to mitigate the effects of unsteady flow within the SWP canal system

downstream. The number of hours at which the peak output is maintained is varied according to the daily export allocation and water level within CCF. The pumps are taken off line in a similar phasing, with the pumps brought off line individually, typically also at 5-minute intervals and with no more than 5,650 cfs reduction during a one-hour period.

- The pumping schedule at the Banks Pumping Plant is also adjusted frequently to accommodate variations in “active storage” (i.e., the available water within CCF), while still delivering the targeted allocation. DWR reports that, under recent operating restrictions where the range in water elevation within CCF has been tidally limited to approximately 2 feet, the resulting 4,200 AF storage is insufficient to operate purely on off-peak pumping and limited pumping during the day is required to reach the daily export allocation while maintaining the CCF water level within the range required for the following day’s export.
- The SWP has limited contractor turnouts upstream of the O’Neill Forebay. The SWP is thus able to use the volume of water in the O’Neill Forebay together with the San Luis Reservoir for flow regulation with the varying daily SWP demand further downstream (south) while operating primarily on the on-peak/off-peak schedule at the Banks Pumping Plant. Table 4-2 summarizes Banks Pumping Plant operational water elevation information.

Table 4-2: Operational Water Elevations at Banks Pumping Plant

Criteria	Data as Reported by DWR	Data in NAVD88
Vertical Datum	NGVD29[1]	Banks site datum +3.10 feet[2] = NAVD88
California Aqueduct Approach Channel Immediately Upstream of Pump Trash Racks		
Lowest Water Elevation for Pump Operation [1, 5]	-4 feet (-3.5 feet)[6]	-0.9 feet (-0.4 feet)
Typical Range in Water Elevation under Priority 1 Gate Operations[4]	Typically -2 to +0 feet	+1.1 o +3.1 feet
Historical Typical Range in Water Elevation[1]	Typically -2 to +2 feet	+1.1 to +5.1 feet
Maximum Design Surface Elevation under Normal Operations[1, 3]	+5 feet	+8.1 feet
Highest Water Elevation for Overall Existing System Operation[1, 7]	+7.5 feet	+10.6 feet
Highest Water Elevation for Pump Operation [1]	+10 feet	+13.1 feet

1. According to pumping plant operators.
2. DWR survey results, February 2008.
3. Source: DWR, 1974a (Bulletin No, 200, November 1974, DWR California State Water Project).
4. As reported by DWR.
5. Dictated by pump submergence requirements to prevent cavitation.
6. Byron-Bethany Irrigation District pumps in approach canal limited to -3.5 feet prior to cavitation.
7. Dictated by top elevation of fish screens at Skinner Delta Fish Protection Facility.

DWR = Department of Water Resources
NAVD88 = North American Vertical Datum of 1988
NGVD29 = National Geodetic Vertical Datum of 1929

4.1.2 CVP Delta Export Facilities

The CVP Delta export facilities comprise the Tracy Fish Collection Facility and the Jones Pumping Plant.

Reclamation uses a site-specific datum at the Jones Pumping Plant, with EL. 0.00 established as the centerline of the outlet pipeline from each individual pump set (Reclamation, 1951). The DHCCP Program uses the NAVD88. As part of this initial study effort, a DWR survey crew performed a closed loop survey between the Jones and Banks Pumping Plants and tied back to NAVD88. This determined that the conversion between the Jones Pumping Plant site specific datum and NAVD88 to be approximately -0.43 foot. For clarity, in the remainder of this section, reported elevations at the CVP Delta export facilities are listed as “Jones site datum.”

4.1.2.1 Tracy Fish Collection Facility

The Jones Pumping Plant is located at the end of a 2.5-mile-long unlined section of the Delta-Mendota Canal (DMC) off Old River. The Tracy Fish Collection Facility is located at the head of this section of canal. The facility intercepts fish using louver screens, which are then collected into tanker trucks and relocated away from pump intakes. There are trash racks immediately upstream of the louver screens.

4.1.2.2 Jones Pumping Plant

In contrast to the SWP facilities, the Jones Pumping Plant has no forebay and is served directly from an inlet channel off Old River, a tidally-influenced body. The Jones Pumping Plant is owned by Reclamation, but operated by the San Luis and Delta-Mendota Water Authority (SLDMWA). The Jones Pumping Plant discharges into the DMC.

The Jones Pumping Plant has an original capacity of 5,100 cfs and a refurbished maximum of 5,630 cfs, but is constrained by downstream canal configuration to a maximum of 4,600 cfs. The CVP pumping plant is comprised of six pumps: four refurbished units (one at 1,000 cfs, two at 990 cfs, and one at 950 cfs) and two original units at 850 cfs each. The pumps are configured in groups of two pumps discharging into a single 15-foot-diameter discharge pipeline conveying flow up to the DMC. Output into the DMC for the west line, central line, and east line is 1,790 cfs, 1,790 cfs, and 1,900 cfs, respectively, with two pumps discharging into each line. There are flow meters on each of the three discharge lines.

The capacity of the DMC immediately downstream is limited to 4,600 cfs with a further hydraulic constraint of approximately 4,300 cfs immediately upstream of the O’Neill Forebay. There are approximately 100 turnouts along the DMC between the Jones Pumping Plant and the O’Neill Forebay. When these turnouts are not in operation, the Jones Pumping Plant peak output is limited to 4,300 cfs.

Generally, the Jones Pumping Plant is in continuous operation and operates irrespective of tidal water elevation at the plant inlet because of hydraulic limitations of the delivery system downstream and limited storage. In addition, because the plant is federally owned and obtains power from Western Area Power Administration (WAPA), the plant’s power tariff, which is different from Banks, provides no economic advantage for Jones to operate according to an on-peak/off-peak schedule. Typically, between two and five pumps are operated at one time at Jones, with changes in numbers of pumps in operation generally scheduled several days in advance.

The design operating conditions for the Jones Pumping Plant are EL -1 foot to EL +10 feet (Jones site datum) (Reclamation, 1951). The typical high water elevation is EL +6 (Jones site datum), and the pumps cavitate when the water level drops below EL -2 feet (Jones site datum). Table 4-3 summarizes water elevation information at Jones Pumping Plant.

Table 4-3: Operational Water Elevations at Jones Pumping Plant

Criteria	Data as Reported by Reclamation/SLDMWA	Data in NAVD88
Vertical Datum	Site specific, (centerline of pump discharge set at Elevation 0.00)	Site -0.43 feet[1] = NAVD88
Design Operating Criteria[2]	-1 to +10 feet	-1.43 to +9.57 feet
Lowest Water Level for Operation[3]	-2 feet	-2.43 feet
Typical Range in Upstream Water Level Measured Immediately Upstream of Trash Racks at Inlet to Pumping Plant[4]	Typical range 0 to +4 feet (generally +1 to +2 feet)	-0.43 to +3.57 feet
Highest Water Level under Typical Operation[4]	+6 feet	+5.57 feet
Highest Water Elevation for Pump Operation[2]	+10 feet	+9.57 feet

1. The site-specific datum translation value at the Jones Pumping Plant of -0.43 ft to NAVD88 is derived from DWR survey results undertaken for the DHCCP team (February 2009).
2. Source: Reclamation, 1951.
3. Dictated by pump submergence requirements to prevent cavitation.
4. As reported by SLDMWA.

DHCCP = Delta Habitat Conservation and Conveyance Program
NAVD88 = North American Vertical Datum of 1988
SLDMWA = San Luis and Delta-Mendota Water Authority
Reclamation = United States Bureau of Reclamation

4.1.3 Comparison between SWP and CVP Delta Export Delivery Systems

The primary differences between the SWP and CVP Delta export facilities are summarized in Table 4-4. The water elevations presented are converted to NAVD88.

Table 4-4: Comparison between Delta Export Delivery Systems for the SWP and CVP

Factor	SWP	CVP
Owner	DWR	Reclamation
Operator	DWR	SLDMWA
Pumping Plant	Banks Pumping Plant	Jones Pumping Plant
Installed Capacity	Nominal 10,670 cfs (11 units)	Nominal 4,600 cfs (six units)
Pump Sizing	Five at 1,130 cfs; four at 1,067 cfs; two at 375 cfs.	One at 1,000 cfs; two at 990 cfs; one at 950 cfs; two at 850 cfs[1] (Original capacity: six at 767 cfs)

Factor	SWP	CVP
Pumping Regime	Operated on an on-peak/off-peak schedule. Typically, periods of non operation during on-peak hours.	24/7 at constant rate when water is available for export.
Flow Variation	Output varies throughout the day, flow changes as often as once per hour. Output scheduled several days in advance.	Limited changes in number of pumps in operation, often constant throughout 24-hour period, any changes scheduled several days in advance.
Fish Screens	Skinner Delta Fish Protection Facility	Tracy Fish Collection Facility
Forebay	CCF, 28,653 AF	None, tidal channel
"Forebay" Water Level Control	Tidally influenced, partial control using radial gates at CCF inlet.	None, tidal
	Elevations Converted to NAVD88	
Typical Range in Upstream Water Level	+1.1 to +3.1 feet	-0.43 to +3,57 feet
Lowest Water Level for Pump Operation[2]	-0.9 feet (Note BBID pumps limited to -0.4 ft)	-2.43 feet
Highest Water Level under Typical Operation	+5.1 feet	+5.57 feet
Highest Water Level for Overall Existing Operation	+10.6 feet	+9.57 feet
Highest Water Elevation for Pump Operation	+13.1 feet	+9.57 feet

1. As reported by SLDMWA.
2. Dictated by pump submergence requirements to prevent cavitation.

AF = acre-feet
BBID = Byron Bethany Irrigation District
CCF = Clifton Court Forebay
cfs = cubic feet per second
CVP = Central Valley Project
DWR = Department of Water Resources
NAVD88 = North American Vertical Datum of 1988
SLDMWA = San Luis and Delta-Mendota Water Authority
SWP = State Water Project
Reclamation = United States Bureau of Reclamation

Figure 4-1, at the end of this section, graphically illustrates and compares the WSEs of the two pumping plants to NAVD88.

4.1.4 Existing Pumping Plants Operating Limits

A key feature of the TDF is that the Banks and Jones Pumping Plants would have a common forebay, CCF. The operating range in WSEs within CCF is required to be compatible with the operating ranges of the existing pump sets at both Banks and Jones Pumping Plants. Based on the pumping plant operating levels tabulated in Tables 4-2 and 4-3, an initial CCF operating range of 6 feet is proposed between EL +0.5 foot to +6.5 feet NAVD88, as shown on Figure 4-2.

4.2 Concept of Operations

The concept of operation for the TDF Option is essentially the same as existing operation of the SWP and CVP, with the exception that both projects would draw from the CCF.

4.2.1 Operating Assumptions

The preliminary concept of operation for the TDF Option has the following assumptions:

- Operate safely and reliably complying with all applicable regulations including all long-term Delta operating rules developed by the BDCP for the protection of fish and conservation measures.
- Maintain the water level within CCF within the efficient operating bands of all pump sets at both Banks and Jones Pumping Plants.
- Move the SWP and CVPs combined daily water allocation (i.e., "X" AF of water per day), with the flow rate a function of the operating duration.
- Minimize impacts to the established operational methodology and control philosophy of both the SWP and CVP downstream of their respective existing Delta export pumping plants.
- Minimize impacts to water quality during a catastrophic levee failure through use of operable barriers.

4.2.2 Overall Operation of System Components

The primary components of the TDF Option are:

- Existing Delta Cross Channel (DCC)
- Optional Supplemental Intake Canal
- Existing and Improved Natural Channels and New Operable Barriers
- New Intermediate pumping plants – Victoria Canal Fish Salvage Facility Pumping Plant (VCPP), optional San Joaquin River Tunnel Pumping Plant (SJRTPP)
- Existing CCF

The Operator selects the target flow rate to be withdrawn from the Delta and the pumps to be in service at any given time and at which location.

The daily schedule is developed taking into account the factors listed in Table 4-5.

Table 4-5: Daily Operational Considerations for Diversion into the TDF System

Factor	Comments
Target Export from Delta	SWP and CVP Delta allocations. Ongoing coordination between DWR and Reclamation. Ongoing coordination with electrical utility provider.
Biological	Possible or confirmed presence of species listed under the Federal Endangered Species Act and the California Endangered Species Act local to a either of the two optional supplemental intakes. Influence of biological factors on Delta.

Factor	Comments
Hydrological	Limitations on volume available for export based on flow rate within Sacramento River. Limitations on permissible time during which withdrawal is allowable based on flow rate within Sacramento River (possible ebb tide pumping). Ongoing coordination with releases from Shasta, Oroville, and Folsom Dams. High flood levels in Sacramento River.
System Mode of Operation	System limitations (e.g., maintenance). Decision D-1641 and subsequent amendments.
Integration with Other Scheme Components	Intake pumping schedule coordinated with pumping schedules for intermediate pumping plants, Banks Pumping Plant, and Jones Pumping Plant.
Energy Usage	Consideration of power tariff structures.
Seasonal	Compliance with seasonal restrictions for withdrawal from the Delta.
Water Quality	Water quality monitoring (turbidity, chemicals) local to given intake. Water quality concerns elsewhere in Delta (such as salinity).
Maintenance	Maintenance schedules for intake facilities. Local sediment buildup. Consideration of rotation between intake facilities during prolonged periods of lower flows.

CVP = Central Valley Project
DWR = Department of Water Resources
Reclamation = United States Bureau of Reclamation
SWP = State Water Project

4.2.3 Delta Cross Channel

The TDF Option draws water through a number of existing natural or manmade channels, including the existing 120-foot-wide gate structure at the diversion from the Sacramento River into the Delta Cross Channel at Walnut Grove. Current operational philosophy and restrictions on the operation of the DCC gates, as set out in State Water Resources Control Board Decision 1641, would be maintained. These constraints are summarized in Table 4-6.

Table 4-6: Constraints on Operation of Delta Cross Channel Gates

Period	Restrictions
November 1 through January 31	Gates closed for a total of up to 45 days for fisheries protection. Gates may be closed on very short notice.
February 1 through May 20	Gates closed.
May 21 through June 15	Gates closed for a total of up to 14 days for fisheries protection. Gates may be closed on very short notice.
June 16 through October 31	Gates generally open.

High flows on the Sacramento River, unforeseen fishery protection actions, or water quality compliance in the Delta may necessitate a short-term closure of the gates. Reclamation's standing operation procedures require gate closure when flow in the Sacramento River exceeds 20,000 to 25,000 cfs.

4.2.4 Optional Supplemental Intake Canal

The preliminary concept for the Optional Supplemental Intake Canal includes two 2,000-cfs intake facilities along the Sacramento River, with a combined total capacity of 4,000 cfs. Each intake facility comprises an intake, sedimentation basin, and intake pumping plant.

Figure 4-3 presents the historical average, California Water Resources Simulation Model (CALSIM)-adjusted, daily Sacramento River-level measured at Freeport Gage over the 53-year period 1956 to 2008, grouped using the Sacramento Valley Water Year Hydrologic Classification into the five categories: Wet (W), Above Normal (AN), Below Normal (BN), Dry (D) and Critical (C).

The BDCP is expected to include long-term water operating rules for the Delta, including north Delta diversion bypass rules representing the minimum flow required to be maintained in the Sacramento River downstream of any diversion (intake) location. The difference between the flow in the Sacramento River upstream of any diversion point and the required diversion bypass flow represents the available flow for diversion into the Supplemental Intake Canal at any time. The water available for diversion is expected to vary from month to month based on several environmental considerations and from year to year based on wet or dry water year conditions. At times, the maximum capacity of 2,000 cfs at each of the two intake diversions would be diverted, and at other times much less or no water would be diverted. The BDCP long-term water operating rules are also expected to include the concept of intermittent diversion linked to the tidal cycle when the flow within the Sacramento River is below a threshold value. The hours of operation of intermittent pumping at a given intake location is understood to be specific to the tidal state at that intake location.

When the available flow for withdrawal is significantly less than the maximum 4,000 cfs for a significant period of time, one or both of the intake facilities and associated pipelines may be dormant for prolonged periods. Periodic use of these intakes would be required to turn over the equipment and maintain operational functionality. It is possible that the northern-most intake would be used for diversion in preference to the more southern intakes for various reasons. Such preferential selection of the upstream intakes would lead to a marginal increase in energy usage because the upstream TDF pumping plants have a higher pumping head than the intake pumping plants downstream.

The concept of intermittent diversion limits withdrawal to two six-hour periods centered on peak ebb tides when Sacramento River flows are below a given threshold, potentially 20,000 cfs. During intermittent pumping, it would be necessary to progressively bring the intake pumps on line in a phased manner so as not to cause excessive surges or waves within the canal system downstream. The intake pipelines discharge directly into the open canal, without benefit of a forebay to help attenuate the impact of the rapid change in incoming flow. During the months of July through November, it is possible that withdrawal might be subject to intermittent pumping. Based on DWR operational experience with the SWP canal system several hours may be required to first progressively increase the flow from zero to the maximum permitted under the particular flow regime and a further similar period to ramp the flow back down to zero. Intermittent pumping operations could reduce the total volume that can be withdrawn from the river during a given 24-hour period. DHCCP hydraulic modeling would be performed in the preliminary engineering phase to determine the maximum rate of change at which flow could be safely introduced into the TDF Option canal system.

The variable frequency drives (VFDs) on each individual intake pump are used to regulate the delivery from each pump to achieve consistent individual pump output under varying Sacramento River water stages and for ease of control of the overall system. Flow is measured using flow meters on each intake delivery main and totalized for the system.

The TDF Option includes two sections of canal:

- Supplemental Intake Canal (Sacramento River intakes to South Fork of the Mokelumne River, capacity 4,000 cfs)
- Victoria Canal, (existing) capacity 15,000 cfs

The operation of the 4,000-cfs canal is similar to the operation of the existing SWP. The canal is divided into a series of reaches by control structures with radial gates located at the entrances to the inverted siphon and the tunnel structure to control flow and regulate the water elevation within the canal.

When a change in flow is required, intake pumping would be increased or decreased and the radial gates would be raised or lowered to achieve the required flow while minimizing changes to WSE. By this methodology, changes in flow do not lead to significant variations in water elevation within the canal, and it is anticipated that water elevation fluctuations during normal operation would be limited to 1 foot during the day. Each flow control structure has water level sensors on each side of the gate connected via the supervisory control and data acquisition (SCADA) system to provide real time information to the Operator. The Operator coordinates the operation of individual control structures throughout the system. Individual radial gates would typically be operated in unison at each control structure location.

The conveyance approach is for control structures to be provided at the inlet to the tunnel crossing the Mokelumne River and at the Stone Lake Drain siphon. It may be possible to reduce the number of control structures operational simplicity while still maintaining sufficient control on water elevation fluctuations within the canal under various flow conditions. Further hydraulic modeling is required to validate this approach.

4.2.5 Natural Channels and Operable Barriers

Natural channels do not include any means of WSE control. Water elevation would normally vary depending on tidal cycle and incoming flow into the Delta.

The most challenging aspect of the TDF Option is reliably delivering water from the DCC and supplemental intakes through existing natural channels into CCF. Even under normal conditions, water brought into an improved through-Delta corridor is prone to escape the corridor through natural channels that intersect the corridor and flow to the west. A catastrophic failure of levees on islands adjacent to the improved corridor, or SLR due to climate change, have even more dramatic effects on the reliability of delivering water of suitable quality to CCF. In an effort to compensate for these effects, extensive use of new operable barriers would be made to maintain the corridor under emergency conditions (see Section 1.0).

Operable barriers are proposed for many of the channels intersecting the through-Delta corridor. However, barriers would be extremely difficult to implement in the San Joaquin River because of impacts to the deep water ship channel to the Port of Stockton. During a catastrophic levee failure or, as a result of SLR, a large body of brackish water could form in the vicinity of the San Joaquin River that would be preferentially captured by the SWP and CVP pumps. Two options to address the San Joaquin River:

- Constructing a barrier system in the San Joaquin River that could accommodate ship traffic, but would be closed in the event of a catastrophic levee failure.
- Constructing a tunnel to convey water from the Delta north of the San Joaquin River to the Delta south of the San Joaquin River.

Operable barriers are not intended to be routinely operated. Their operation would be scheduled in advance by the Operator and gate opening position recorded in the SCADA system. Additional details on operable barriers are provided in Section 17.0.

4.2.6 Intermediate Pumping Plants

The Operator develops the daily schedule for the SJRTPP operation in advance based on the daily export allocation. The Operator selects the target flow rate and the number of pumps to be in service at any time. Pump operation is rotated to produce even wear on the pumps. Individual pumps are started/stopped no more frequently than once per hour. Control structures at the inlet and outlet of the tunnel are used to isolate one or more of the parallel tunnels during periods of sustained low flow.

Victoria Canal Fish Salvage Facility Pumping Plant. VCPP comprises a centralized intake facility on the New Victoria canal with a capacity of 15,000 cfs. The intake facility includes a screened intake, sedimentation basins, and a pumping plant. The Operator develops the daily schedule for the VCPP operation in advance based on the daily export allocation. The Operator selects the target flow rate and the number of pumps to be in service at any time. Pump operation is rotated to produce even wear on the pumps. Individual pumps are started/ stopped no more frequently than once per hour. Control structures at the inlet and outlet of the tunnel are used to isolate one or more of the parallel tunnels during periods of sustained low flow. Flow is measured using flowmeters on each intake delivery main and totalized for the system.

4.2.7 Clifton Court Forebay

The TDF Option continues to use CCF, which provides a degree of flow regulation within the conveyance system and operational flexibility. Water is pumped into CCF from VCPP. The Operator would maintain the water elevation in the CCF system to within the defined water level operating range. Water levels would be monitored and conveyance capacity would be adjusted accordingly.

4.3 Modes of Operation

The TDF Option would operate under two primary modes of operation: normal and maintenance.

4.3.1 Normal Operations

The TDF Option provides operational flexibility to optimize water quality and environmental in-Delta concerns while continuing Delta export operations. The overall system operation is based on moving the daily water allocation ("X" AF of water per day) for each of the SWP and CVP export demands rather than maintaining a set flow rate.

To the extent possible, the system operates to minimize the use of pumps to convey the water through to Delta export facilities. Where pumping is required, the system operates to maximize the use of off-peak pumping with reduced or no flow during periods of higher energy cost. Operation of the supplemental intake facilities, intermediate pumping plant(s), Banks Pumping Plant, and Jones Pumping Plant would be synchronized by Operations to convey the required flow rate through the TDF Option system into the SWP and CVP Delta export facilities. Minor adjustments in pumping duration would be made to restore the water elevation in CCF to the agreed-upon level at the start of each daily cycle.

4.3.2 Maintenance Operations

The TDF Option has a number of features to improve operational redundancy and reliability to enable conveyance capacity to be maximized during maintenance operations. These features are summarized in Table 4-7.

During periods when more than one intermediate pumping sets are out of service for maintenance, the remaining available pumps would pump for longer periods to transfer the required daily allocation to CCF.

Table 4-7: System Reliability and Redundancy

Element	Project Feature
Overall	Multiple flow paths to convey flow through the Delta. Critical elements located above flood elevation, including allowance for rise in sea level due to climate change impacts.
Intake pumping plant	Two intake pumping plants with total diversion capacity of 4,000 cfs. Multiple pumps at each intake location with fully redundant pump set provided at each intake even when intake at full capacity at 2,000 cfs (4 duty, 1 standby). Variable frequency drive motor for additional flexibility.
Intake pipelines	Multiple parallel intake pumping plants pipelines, each with flow measurement from each pumping plant. Radial gates provided at connection to canal so that, in the event of a pipe breakage, gates would automatically close to prevent canal from draining back into pipeline.
New canal systems	Canal divided into reaches, separated by control structures. Multiple control gates at each control structure location, sized to pass full system capacity with one control gate out of service at any given location. Standby generators provided at each control structure. Limit velocity and water level variation within a given canal reach to minimize possibility of scour and local levee failure.

Element	Project Feature
Intermediate pumping plants (SJRTPP, VCPP)	Multiple pump sets to match delivery flowrates, all pumps constant speed. One redundant pump set at full capacity.
Siphons/tunnels	Multiple parallel siphons/tunnels each with means of isolation upstream and downstream and trash racks upstream. Ability to reduce number of siphons/tunnels barrels in operation during periods of lower flow to maintain velocity in active siphons/tunnels.
CCF	Allowance for buildup of sediment below minimum operating level.
Communication system	Redundant communication paths and equipment.

CCF = Clifton Court Forebay
cfs = cubic feet per second
SJRTPP = San Joaquin River Tunnel Pumping Plant
VCPP = Victoria Canal Fish Salvage Facility Pumping Plant

4.3.3 Implications of Option on Current SWP and CVP Operations

The TDF Option enhances the existing means of conveyance through the Delta but retains the same overall operational concerns. Two supplemental intake locations further upstream may mitigate environmental constraints that dictate when water can be exported. The principal proposed changes are:

- New supplemental intake locations, total capacity 4,000 cfs, with integral fish screens
- Conveyance system intermediate pumping plant(s) and fish screening facilities to the start of the SWP and CVP export facilities
- CCF would serve both the SWP and CVP pumping plants
- Removal of tidal influence from water level upstream of SWP and CVP pump sets

A greater level of daily routine operational coordination would be required between DWR and Reclamation. In addition to existing coordination for water releases and available water for export, the scheduling of operation of individual pumps at each plant would need to be coordinated to manage the conveyance system upstream and the water level within the common CCF.

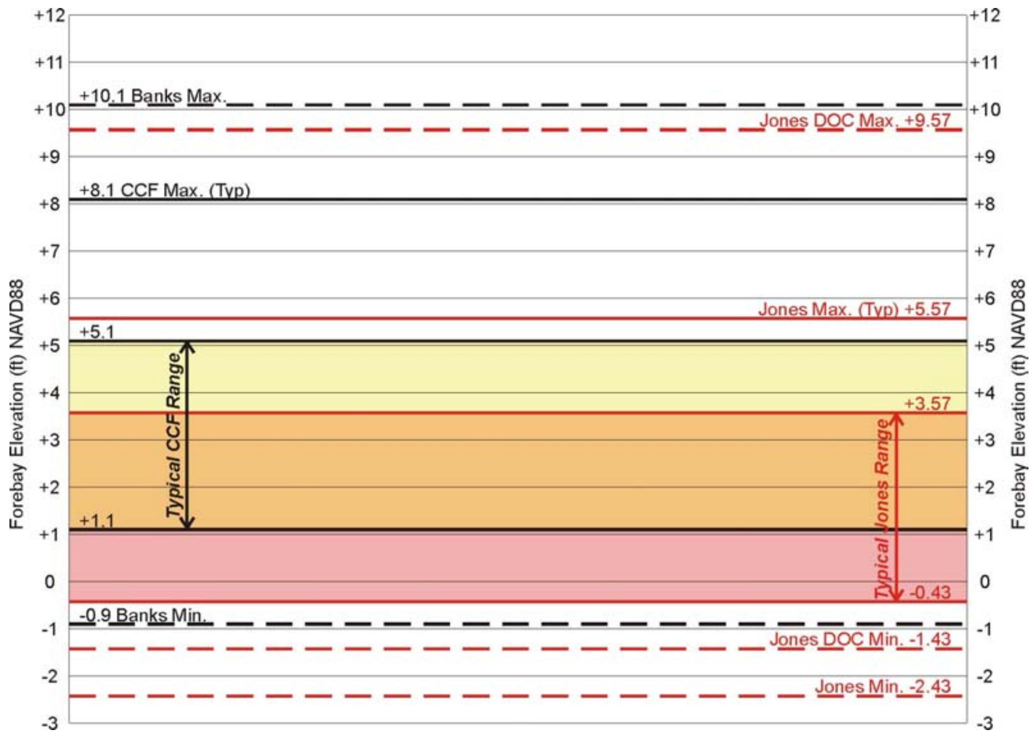


Figure 4-1 – Banks Pumping Plant and Jones Pumping Plant Operating Water Surface Elevations (NAVD88)

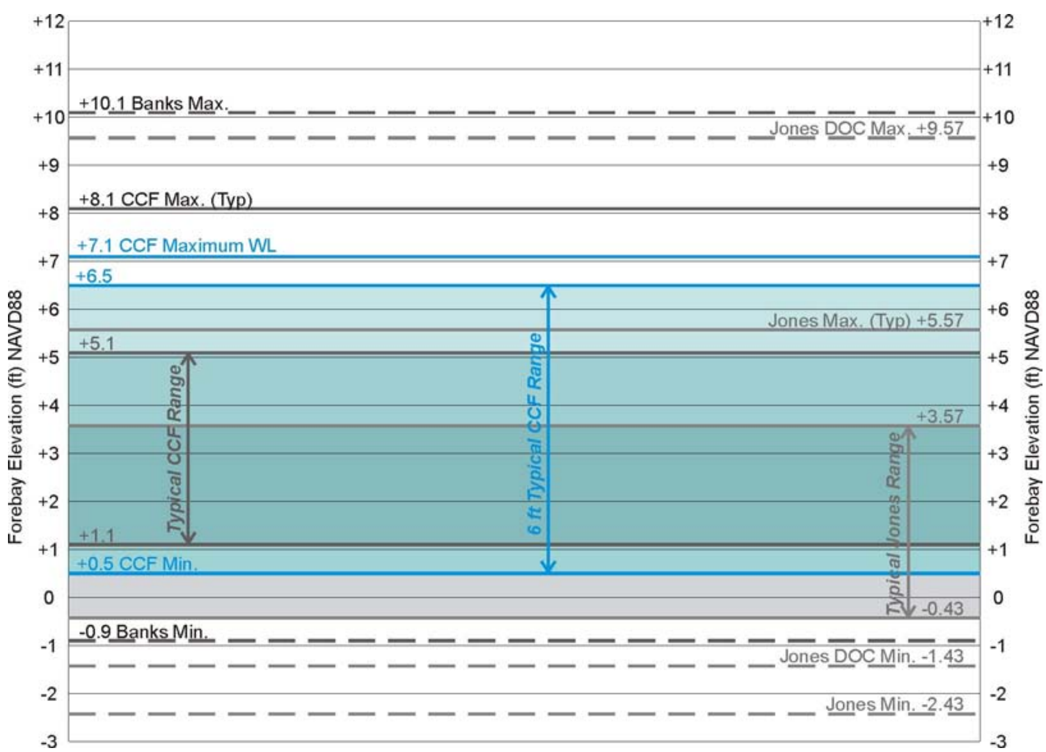


Figure 4-2 – Banks Pumping Plant, Jones Pumping Plant, and Proposed CCF Operating Water Surface Elevations (NAVD88)

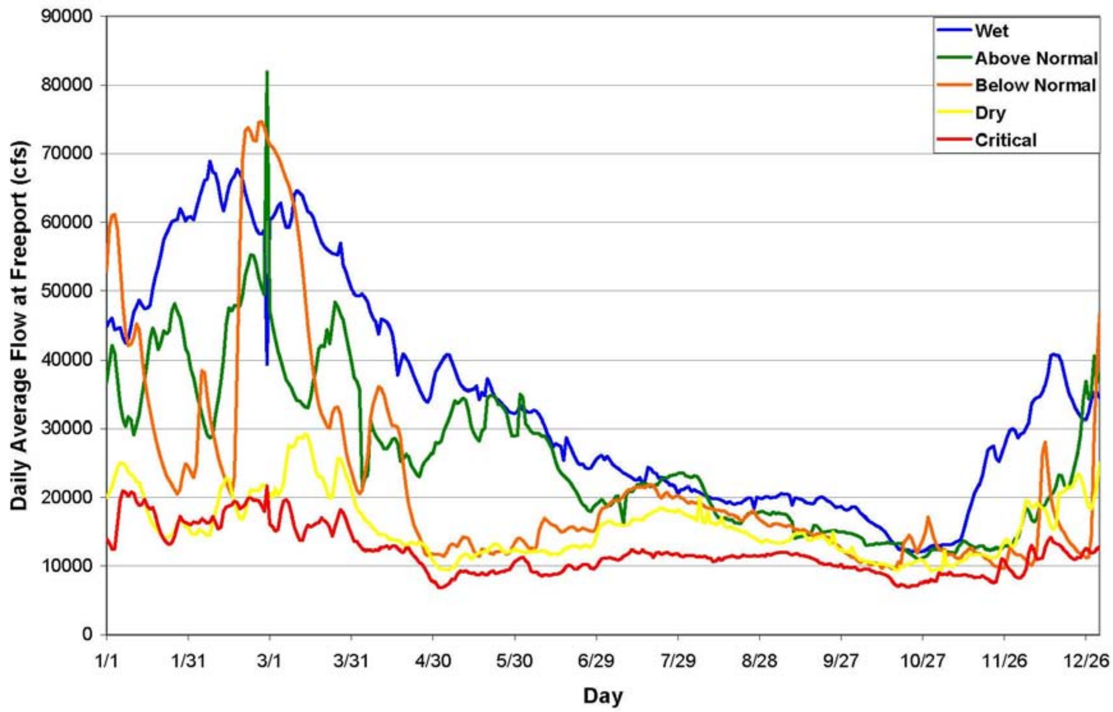


Figure 4-3 – Sacramento River Flow at Freeport Gage (1956-2008) using Sacramento Valley Water Year Hydrological Classifications

5.0 CONVEYANCE SYSTEM HYDRAULICS

The system hydraulics are depicted on Figure 5-1 at the end of this section.

5.1 Facility Capacity

The capacity of the TDF is 15,000 cfs; individual channel capacities are listed in Table 5-1.

Table 5-1: Channel Capacities of the TDF

Channel	Design Flow
Supplemental Intake Canal	4,000 cfs
Through-Delta corridor north of San Joaquin River	8,000 to 12,000 cfs
Through-Delta corridor south of San Joaquin River	15,000 cfs
Victoria Canal[1]	15,000 cfs
CCF[2]	15,000 cfs
Banks Pumping Facility[3]	10,300 cfs
Jones Pumping Facility[4]	4,600 cfs

[1] Including fish screen and pumping plant
[2] Includes storage
[3] Approach channel
[4] Approach channel and intertie

CCF = Clifton Court Forebay
cfs = cubic feet per second

5.2 Preliminary Hydraulic Analysis

A preliminary hydraulic analysis of the Delta and the TDF alignment were completed to assess channel capacities, system reliability, and function. System reliability and function are discussed in this section. Channel capacities are discussed in Section 16.0. This analysis also contributed to the preliminary design of the Supplemental Intake Canal, natural channel modifications, CCF and various pumping facilities associated with the alignment. In addition, a hydraulic profile was developed for design reference and is presented on Figure 5-1.

5.2.1 Supplemental Intake Canal

Water delivered via the Supplemental Intake Canal would provide additional fresh water to the central Delta under circumstances when the DCC is closed. This is the primary function for the Supplemental Intake Canal.

Estimations for hydraulic losses throughout the Supplemental Intake Canal have been refined and improved. Previously, hydraulic gradients in canal sections were estimated assuming normal depth calculations using a spreadsheet model. Consideration was not given to potential backwater effects that may arise in the downstream reaches of canal sections due to transitions for siphons (tunnels or box culverts) and control structures. The refined head loss calculations have been estimated using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center-River Analysis (HEC-RAS) 4.0 Software. HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. Both the Steady Flow Water Surface Profile component and the Unsteady Flow Simulation component were utilized for analysis of the proposed Supplemental Intake Canal. The Steady Flow component was used to estimate losses, and backwater effects, and to determine an appropriate channel invert. The Unsteady Flow Simulation component was used to evaluate operational considerations, including control gate requirements.

The HEC-RAS model of the Supplemental Intake Canal includes a more accurate representation of transitions into and out of siphons and control structures than a spreadsheet model. Transition lengths between trapezoidal and rectangular cross sections have been estimated as approximately 3 times the rectangular channel width. The rectangular channel width is determined by the total opening width of siphons/control structures and associated side walls. The model included the proposed bridge crossings, assuming two piers in the water and a full size canal width. The impact of bridge crossings on water depth was minimal (less than 0.05 foot). Other design parameters included in the HEC-RAS model are:

- Canal slope = 0.000045 feet per foot
- Canal design flow depth = 23.5 feet
- Typical canal cross section = trapezoidal with bottom width of 40 feet; 3 horizontal to 1 vertical ratio (3H:1V) side slopes up to 0.75 of design flow depth; 8H:1V side slopes above 0.75 of design flow depth
- Manning's roughness in canal sections = 0.03
- Manning's roughness in closed conduits (box culverts and tunnels) = 0.013

The proposed Supplemental Intake Canal is an unlined earth canal, where average flow velocity would be limited to 1.6 feet per second (fps). Estimation of inverts has been developed by assuming a conservatively high downstream WSE with a flow of 4,000 cfs in the Supplemental Intake Canal. A downstream boundary condition of 11.5 feet in the South Fork of the Mokelumne River was chosen. This WSE accounts for high flows in the South Fork of the Mokelumne River coinciding with high tide. Inverts for the canal were established to maintain a flow depth of 23.5 feet in each of the canal reaches.

Other hydraulic considerations for the proposed Supplemental Intake Canal are:

- The TDF Supplemental Intake Canal has been modified such that it intersects the Stone Lake Drain through a single siphon crossing (open cut construction).
- A control structure has been included at the downstream end of the Supplemental Intake Canal. This control structure would enable water depths of approximately 23.5 feet to be sustained throughout the canal regardless of inflows into the Supplemental Intake Canal or WSEs in the South Fork of the Mokelumne River.
- The calculation of friction losses through closed conduits (box culverts and tunnels) assumed pre-cast concrete internal surface with no lining.

The proposed canal invert would average approximately 10 feet below the existing ground elevation. The calculated hydraulic grade line (HGL) would remain between 11 and 14 feet below the proposed flood protection levees (see Figure 5-2).

The HEC-RAS model indicates that the majority of head losses in the Supplemental Intake Canal occur at the critical structures, the siphon and control structures, especially at the tunnel. The total HGL is a function of the critical structures, control structures, transitions and the slope gradient of the canal sections.

The head loss at 4,000 cfs is approximately 9 feet over the 63,600 feet of length of the Supplemental Intake Canal. Critical structures and associated head losses, velocities, and structure dimensions are presented below.

- Stone Lake Drain siphon: Head loss = 1.6 feet; velocity = 5.5 fps; 2 barrel, 19 feet by 19 feet concrete culvert boxes
- Lost Slough-Mokelumne River Tunnel siphon: Head loss = 3.7 feet; velocity = 7.0 fps; single tube, 27 feet inside diameter
- End of canal control structure: Head loss = 0.5 feet, velocity = 4.9 fps: 2 barrel, 19 feet wide rectangular channels with radial gates

At a design flow of 4,000 cfs, head losses through these structures account for approximately 70% of the total head losses associated with the Supplemental Intake Canal. Although the impact of these structures upon head loss is significant, their sizing seems adequate with regard to minimizing the risk of sediment deposition in closed conduits. The hydraulic gradient required to balance these head losses would be provided by lowering the canal invert downstream of the structure.

An additional steady-state simulation for a low flow scenario (1,000 cfs) was modeled. The findings indicate that gate operation (closure) to maintain canal depth of 23.5 feet is achievable. Three gate control structures have been included in the Supplemental Intake Canal model. These structures are located:

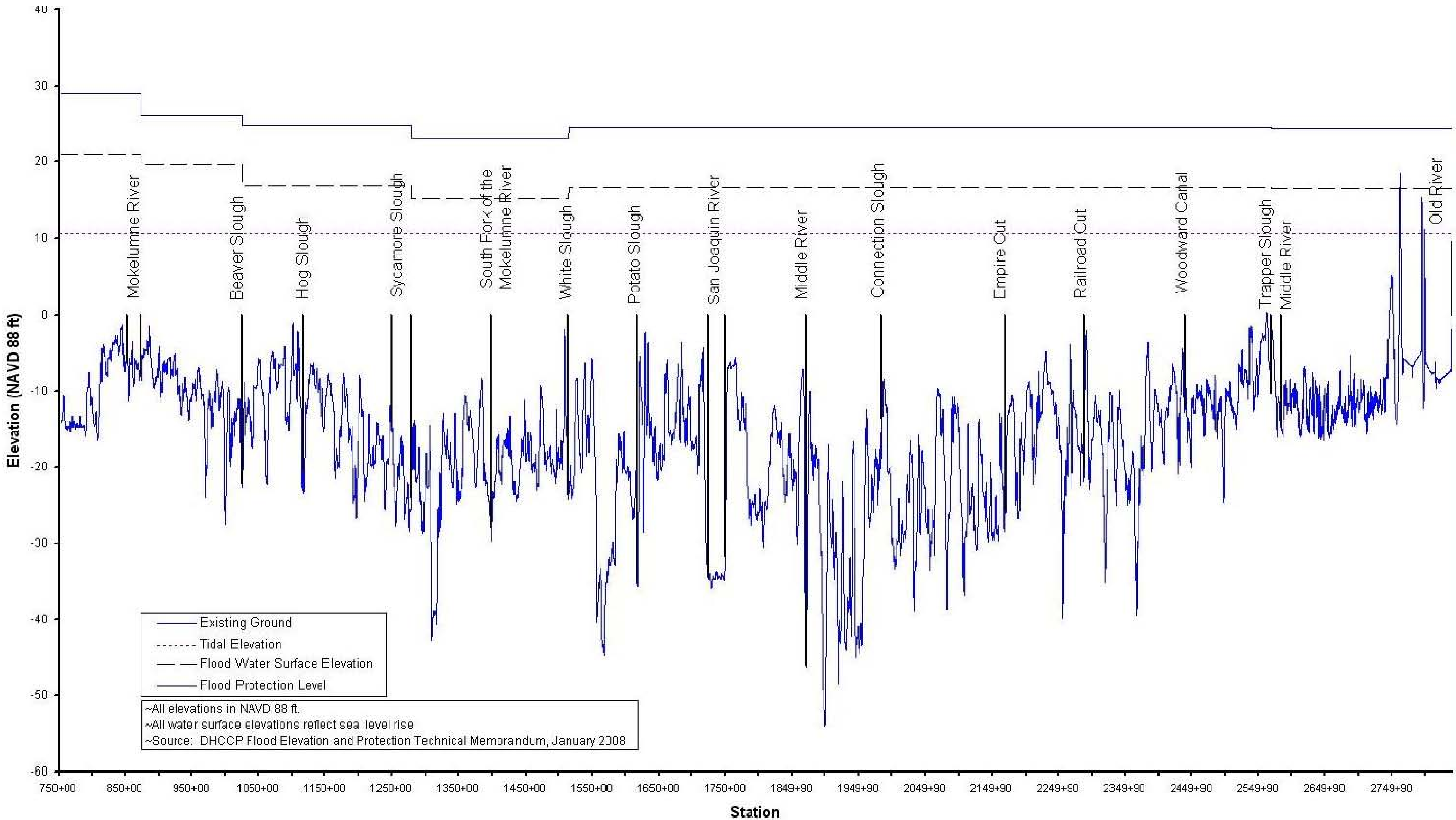
- Immediately upstream of the Stone Lake Drain siphon
- Immediately upstream of the Lost Slough-Mokelumne River tunnel
- Immediately upstream of the confluence of the Supplemental Intake Canal with the South Fork of the Mokelumne River

Representative unsteady simulations were modeled. These simulations modeled the gradual change in the flow rate diverted at the intake pumping plants. Some observations of these simulations are that the canals contain adequate storage to dampen the effects of flow imbalances that may occur and provide operators with sufficient time to react to flow imbalances by operating either the gates or pumps as necessary. These observations do not include impacts of transient waves. Additional findings and observations of these simulations include the following:

- The introduction of a change in flow initiated at the upstream end of the Supplemental Intake Canal does not result in an immediate change in flow at the downstream end of the canal.

Due to routing effects, it takes approximately two hours for any change at the intake pumping plant to propagate to the downstream end of the Supplemental Intake Canal.

- The proposed 5-mile maximum distance between each control structure provides adequate means of controlling the water depth within a reasonable range of the proposed normal depth of 23.5 feet in the Supplemental Intake Canal.
- When considering a gradual change of flow at the intake pumping station, the rate of closure/ opening of the radial gates seemed slow enough to prevent the formation of significant surge waves in the canal.
- An emergency shutdown where flows quickly reduce from 4,000 cfs to 0 cfs at the intakes was also evaluated. If gates were not operated to maintain depths of 23.5 feet in Supplemental Intake Canal, the water level would drop by 1 foot within the first 25 minutes, 2 feet within the first 45 minutes, and 5 feet in approximately 2 hours in the vicinity of the shutdown. For this simulation, flows dropped from 4,000 cfs to 0 cfs over 30 minutes.



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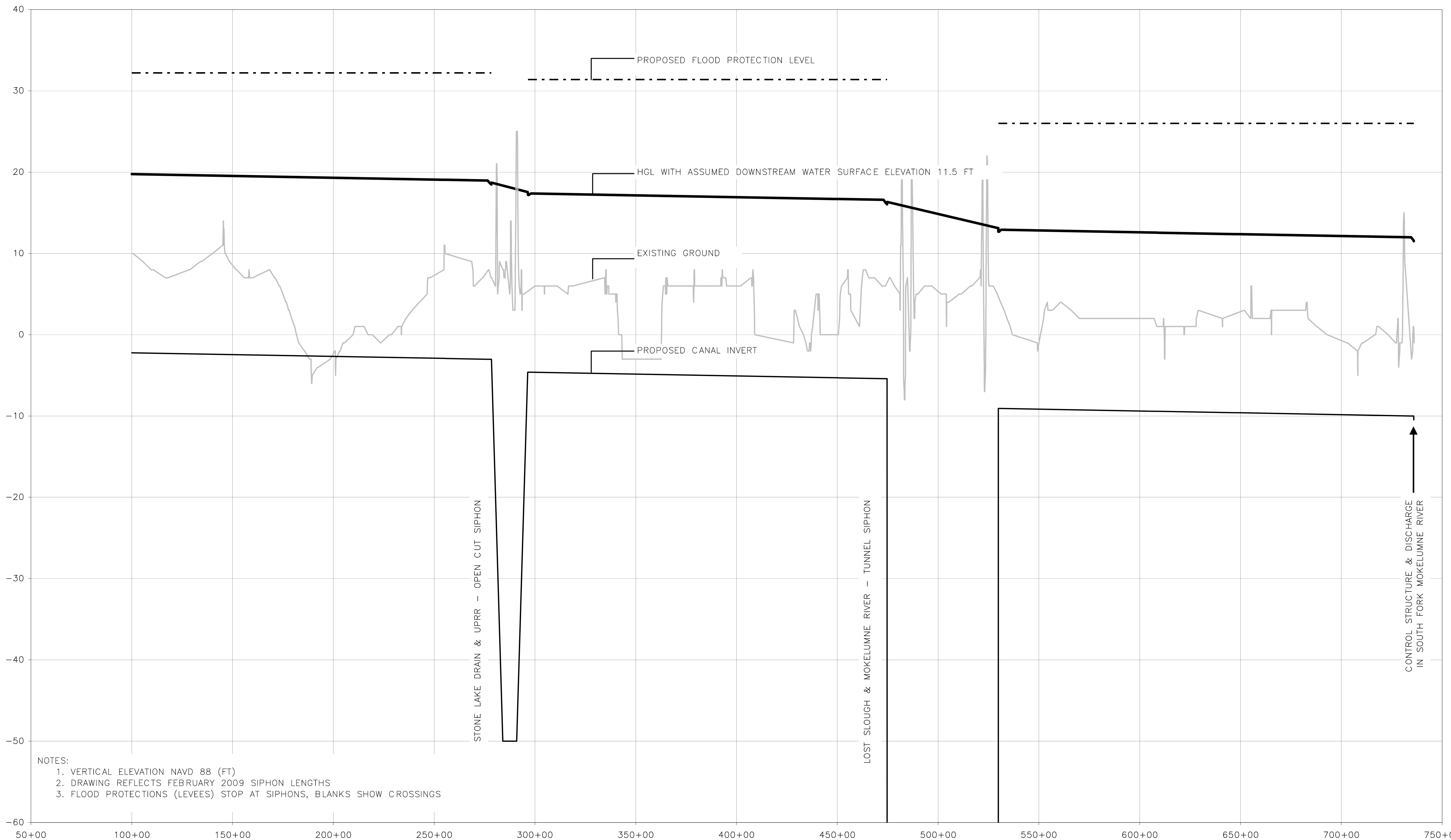
California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Hydraulics, TDF Option
Hydraulic Profile

FIGURE
5-1



NOTES:
1. VERTICAL ELEVATION NAVD 88 (FT)
2. DRAWING REFLECTS FEBRUARY 2009 SIPHON LENGTHS
3. FLOOD PROTECTIONS (LEVEES) STOP AT SIPHONS, BLANKS SHOW CROSSINGS

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DHCCP Engineering
WORKING DRAFT – Subject to Change

California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Hydraulics, Supplemental Intake Canal Hydraulic Profile

FIGURE
5-2

6.0 INTAKES

6.1 Description and Site Plans

Three discrete intake elements or projects are potentially required to achieve performance objectives for the TDF:

- North Delta water transfer (Supplemental Intake Canal)
- New intake and fish salvage facility at CCF off the Victoria Canal (Section 20.0)
- New intake and fish salvage facility at San Joaquin River Tunnel Option

These intake elements are discussed below.

6.1.1 Recommended Intake Locations

6.1.1.1 North Delta Water Transfer Option

The network of waterways within the Delta restricts conveyance of high quality water to the SWP and CVP export facilities in the south Delta. In its Record of Decision, the CALFED program included "studying and evaluating a screened diversion facility on the Sacramento River with a range of diversion capacities up to 4,000 cfs" (CALFED, 2000b).

A variety of diversion and conveyance options have been considered (see Figure 6-1 and Table 6-1). The options examined included a potential new intake at Georgianna and Snodgrass Sloughs, modifying the DCC, and implementing a new diversion from the Sacramento River near Hood. Preliminary intake locations are shown on Figure 6-2. Recent evaluations of potential intake locations for all Delta conveyance alignments have concluded that intakes should be located north of Walnut Grove to minimize impacts to endangered species. New intakes located further south of Walnut Grove would potentially have a greater impact on Delta smelt and be subject to greater tidal influence. Consistent with the ***Delta Conveyance Improvement Studies Summary Report*** (EDAW, 2007), the ***Final TDF Prefeasibility Study*** (DWR, 2007a) and the ***Through-Delta Facility Value Planning Study*** (DWR, 2007c), a new river intake on the east side of the Sacramento River at Hood was evaluated in detail for this report.

Following the Prefeasibility Study (DWR, 2007b), the BDCP organized a Fish Facilities Technical Team (FFTT) to study best available technologies and potential locations for fish-screened diversions on the Sacramento River between the City of Sacramento and Walnut Grove. Although taken to a conceptual level, the findings of the FFTT are also applicable to the North Delta Water Transfer concept. For additional information regarding FFTT recommendations, refer to ***Conceptual Proposal for Screening Water Diversion Facilities along the Sacramento River*** (DWR, 2008b). The criteria and configurations proposed by the FFTT significantly alter the approach to fish-screened diversion for a North Delta Water Transfer concept.

Additionally, a Value Planning Study was conducted by the DHCCP to review the data and conclusions of the FFTT study and to further develop river intake options that best suit the objectives of the BDCP and DHCCP. The Value Planning Study (DWR-DHCCP, 2008c) provided additional intake options for consideration and tools by which to compare features and performance. The outcome from these two studies provides the basis by which to propose a reasonable intake solution for the North Delta Water Transfer concept. These key findings supersede the original intake arrangements for this option. A detailed summary about the concept development for river intakes is provided in Appendix B.

Based on the results and conclusions of the subject studies, the following North Delta Water Transfer concepts were considered:

- Upgrading the DCC inlet to include a new fish screening and sediment removal facility. Embankments on both sides of the DCC would be improved to meet design criteria for seismic stability and flood protection.
- Adding a new intake and canal system north of the DCC. This new 12-mile canal (Supplemental Intake Canal) would extend from the vicinity of Hood to the South Fork of the Mokelumne River.
- Adding a new intake and shorter canal system south of Walnut Grove in the vicinity of Georgiana Slough (e.g., Option CF-15 from the ***Through-Delta Facility Value Planning Study*** [DWR, 2007c]).

In reviewing these options, several key findings have been made which aid in selecting the best possible arrangement for the North Delta Diversion concept. Upon review of river hydrology during the supposed period in which the North Delta Water Transfer would operate (December of 1999 given as the example low flow design case), zero to reverse flow conditions can occur downstream of Freeport bridge. This bi-directional flow effect argues in favor of intake placement further north than the DCC and Georgiana Slough (Walnut Grove). Due to the opportunity for greater tidal influence, the probability of higher smelt abundance, and potential for inducing reverse flows in Steamboat and Sutter Sloughs, it is recommended that intakes not be considered in the vicinity of the DCC and Georgiana Slough. This is additionally reinforced by planning efforts advocating placement of river intakes for the isolated conveyance facilities upstream of Courtland.

Table 6-1: Intake Options for TDF Option (Source: DWR, 2007b)

Option	Description	Advantages	Disadvantages
TDF	1. A new intake structure at Hood draws 4,000 cfs from the Sacramento River. 2. New intake has a trashrack structure and "V-shaped" or saw-tooth-shaped fish-screen and fish-bypass feature. The design approach velocity is 0.2 fps. 3. A pumping plant is constructed as part of the intake structure to withdraw the required 4,000 cfs. 4. The water is conveyed by a new canal from Hood to the South Fork of the Mokelumne River near the confluence with Snodgrass Slough. 5. At the downstream end of the canal is a gated structure outlet to discourage fish from swimming upstream to the pumping station.	• Provides additional flow through the Delta, improving water quality and reducing the need for carriage water release from State reservoirs. • Fish screen associated with the intake structure reduces the impact on endangered species.	• High capital cost. • Impact to surrounding environment and private lands.
CF-04	The option reduces the conveyance footprint and shortens the channel. Similar electrical current reductions are expected at the pumps at substantially less cost. Feasible measures would be implemented to reduce the channel cross section (minimize ROW acquisition and environmental, social, and economic costs).	• Reduces cost by eliminating 40% of channel length, two siphons, and one bridge. • Reduces environmental impact by eliminating the above infrastructure and reducing width. • Eliminates severance of agricultural facilities and operations on New Hope Tract. • Eliminates need to address severe constraints on McCormack-Williamson Tract (towers, flood flows, habitat). • "Vests" infrastructure improvements along Peripheral Canal corridor.	• Reduced water quality benefit • Delivers flows to area with hydrologic constraints. • Uses old canal alignment. • Would cause backwater at DCC and reduce its capacity to divert Sacramento River flows (may not be significant).

Option	Description	Advantages	Disadvantages
CF-10a CF-10b CF-10c	This option provides for an intake structure north of Walnut Grove/Locke. The concept includes a fish screen on the river, reinforced box culvert through the levee section, forebay, trashrack, low-head pumping plant (except Option CF-10b), floodgate, multiple pipes to the Mokelumne River, and an outlet structure. The length of the conveyance system would vary depending on the discharge point selected. Three option alignments, Options CF-10a, CF-10b, and CF-10c, discharge to Mokelumne River, Snodgrass Slough, and the South Fork of the Mokelumne River, respectively.	• Each option results in a project cost reduction. • Reduces required ROW and associated environmental and economic impacts. • Does not use the site at Hood, which may be needed in the future for the old canal. • Leaves 4,000 cfs in the Sacramento River from Hood to the town of Locke.	• Possible reduction in benefit to export water quality. • Possible greater public/environmental impacts. • Possible impacts to DCC hydraulics.
CF-15	This option concept, Georgiana Slough at Walnut Grove, diverts 4,000 cfs of Sacramento River water for transfer to the South Fork of the Mokelumne River just north of its confluence with Beaver Slough. The size of the diversion is the same as the original concept. Major features include an intake structure with on-river fish screens, radial floodgates, low head pumps, service bridge, south levee road bridge, approximately 20,000 feet of canal, two siphons, and an outlet structure at the river. Required ROW is approximately 500 feet wide and 4.2 miles long.	• 4.2 miles long vs. 12.0 miles long. • No fish handling downstream from screens. • Two siphons instead of three. • One public road bridge at intake. • Diverted water stays in stream for additional 12 miles. • Pumps have lower head. • No trashrack required.	• Water quality reduced slightly. • ROW more costly. • Slightly outside Record of Decision limits. • Siphon construction more difficult. • On-river screen more costly. • Screen construction more difficult.
CF-19	This option provides diversions at multiple locations that have a combined diversion capacity of up to 4,000 cfs. The location, size (capacity), and configuration of each diversion are developed based on the capacity of the existing downstream receiving channel(s), distance to the receiving channel, environmental constraints, local conditions, and ability to meet water quality improvement goals: • A diversion into Snodgrass Slough between Meadows Slough and the community of Locke. • A diversion into Meadows Slough north of the DCC channel. • A diversion into Snodgrass Slough along north side of Twin Cities Road. • An independent and isolated diversion at DCC. • An independent and isolated diversion at Georgiana Slough. • A diversion into the upper end of Snodgrass Slough.	• Shorter and smaller conveyance channels. • Sized for downstream receiving waters. • Reduce or eliminate pumping. • Greater flexibility and reliability in operation (only partial shutdowns). • Allow variations in Delta circulation/flow patterns. • Greater selection of diversion sites. • Standard proven designs. • Probable lower overall cost. • Maintenance of existing landscape (e.g., agricultural use). • Preserve Hood site for possible future use. • Leave more flow in upper portions of Sacramento River. • Phased construction.	• Marginally greater O&M costs. • Less “economy of scale.” • Possible enlargement of existing channels. • Possibly less water quality improvement compared to DWR alternative. • Possibly more backwater on existing diversions (DCC and Georgiana Slough).

Option	Description	Advantages	Disadvantages
CF-38	This option involves the following features: • Refurbishing the existing DCC gates. • Increasing the diversion capacity of the DCC by adding an additional gate on the northern side of the existing facility. • Widening the existing DCC diversion channel. • Provision of V screens in the widened channel to capture fish passed through the new gates. The fish screens could be expanded to collect fish passing through the existing gates. • Provision of a fish collection and pump back system. It is assumed that additional salinity reductions at Delta export facilities may be realized by these improvements.	• Refurbishment of existing DCC gates, adding reliability of diversions toward Delta export facilities. • Increasing diversion capacity of DCC, adding flexibility to facility operation by allowing one, two, or three diversion gates to be open for control of salinity at export facilities. • Does not impact boat traffic in the area and, therefore, does not require mitigation. • Potential to stage screening of entire DCC and realize significant fish benefits and greater flexibility in operating DCC during periods when it would be closed due to fish concerns. • Keep a higher flow rate in the Sacramento River for a longer distance.	• Extent of downstream channel improvements required to accommodate increased DCC diversions is not known and may incur additional costs. • May result in frequent and rapid changes in downstream WSEs that may be detrimental to both downstream channels and water users. • Increased diversions may encourage additional withdrawals by downstream users.

cfs = cubic feet per second
DCC = Delta Cross Channel
DWR = Department of Water Resources
O&M = operation and maintenance
ROD = Record of Decision
ROW = right-of-way
TDF = Through-Delta Facility

According to the above findings and recommendations, it is proposed the North Delta Water Transfer concept consist of two screened intake structures each coupled with a sediment management basin and a low-head pumping plant with both discharging into a 12-mile canal extended to the South Fork of the Mokelumne River near Beaver Slough. A fish barrier is included at the proposed canal discharge point to prevent entry of in-migrant salmonids.

This Supplemental Intake Canal intake arrangement consists of two individual intake locations with a diversion capacity of 2,000 cfs each. The intakes would be single in-river type sized to accommodate the design flow while meeting fish protection criteria for anadromous salmonid and Delta Smelt species. Locations on the east side of the Sacramento River, one at Hood and one approximately 1 mile downstream are proposed. Preliminary illustrations of the intake type are provided in Appendix B.

In general, the intakes would be reinforced concrete structures subdivided into individual bays that can be isolated and individually managed. Each bay would be fitted with vertical flat-plate screen panels, flow control baffles, and bulkhead provisions for isolation. Diverted water would be routed from each intake bay through respective under-levee piping to a receiving sedimentation basin with a chain-and-flight sediment removal mechanism. After removal of abrasives and heavy fines, diverted water would be directed into corresponding bays of the pumping plant lifting the water and discharging it into the canal. A more detailed explanation of intake, sediment management and pumping facilities is provided in Appendices C and D.

6.1.1.2 Victoria Canal Fish Salvage Facility Intake

In 2001, DWR issued a request for qualifications to provide engineering and design services for the South Delta Improvement Project (SDIP). This project involved the replacement of fish protection and water quality control provisions within and in the vicinity of CCF. The project was cancelled but included consideration of facilities (e.g., channel realignments, intakes, fish salvage facilities, associated site improvements) appropriate for the TDF Option.

Accordingly, a new intake upstream of CCF is needed to improve fish protection. Anecdotal reports indicated that greater than 80% of juvenile salmonids and juvenile/adult smelt entering CCF do not survive. Fish mortality is said to be primarily attributed to predation occurring in the forebay. A smaller component of fish loss is due to export pumping and salvage operations.

A logical approach to improving fish survival is to prevent their entry into the forebay using more modern fish screen and salvage facilities. Locating a positive exclusion barrier upstream of CCF at the confluence of Old River, Italian Slough, and the North Victoria Canal or at the downstream terminus of the Victoria Canal serves to eliminate entrainment into the forebay.

The original SDIP plan for improving the Skinner (DWR) and Tracy (Reclamation) intake facilities involved a new combined intake facility at the north end of CCF to replace the existing state and federal facilities.

The CCF intake concept considered a new diversion facility situated at the north end of Clifton Court. The concept required realignment of Italian Slough to accommodate the requisite intake and fish salvage facilities. A new isolated connection between Old River and CCF was required to function as the new path of diversion into CCF. The existing Banks Pumping Plant (SWP) would continue to operate as it has historically drafting water directly from CCF, whereas the inlet channel to the Jones Pumping Plant (CVP) would have to be linked to CCF. The existing Jones and Banks Pumping Plants would move water from CCF to their respective canals. The existing diversion structure at the southeast corner of CCF would no longer be needed and would be isolated accordingly.

The SDIP diversion concept originally consisted of five individual, modular, fish-screen structures designed for 2,500 cfs each. The fish screen intakes are arranged in a chevron configuration, with self-cleaning trashracks at the throat, radial gate structures downstream for isolation, vertical flat-plan screen panels converging to a fish-bypass channel, flow-control baffles and traveling brush systems at the screen banks, and a collection and salvage facility similar to the aforementioned existing collection, handling, transport, and release (CHTR) facilities.

The SDIP diversion facilities consisted of five parallel intake modules, each coupled with an individual low-head pumping facility featuring integral low-head tide gates. The pumping equipment would improve diversion flexibility and capability whereas the low-head tide gates would provide for reduced operating costs when gravity conveyance was possible. The intakes would feature bypass or collection piping at their downstream limits to channel entrained fish to collection and handling facilities.

Although the original diversion concept proposed by the SDIP is relevant to the TDF Option, some modifications are appropriate to suit the performance objectives of the DHCCP. For conveyance reasons, it is recommended the diversion facilities be located at the downstream terminus of the Victoria Canal on the east side of Old River. This would prevent influencing the Old River during operations and would ensure positive flow past new diversions under construction on the Victoria Canal. The modified CCF diversion concept is further described below.

Diversion Facilities. The required CCF diversion improvements involve a unique mixture of facilities given the location in the watershed. Because the point of diversion for the SWP and CVP export facilities is at a terminus in the Delta network, sufficient carriage flows are not available to sweep fish past a typical screened intake. This “dead end” condition requires an intake configuration with a means of escape from excessive exposure to the screen bank and subsequent impingement. Therefore an “off-channel” intake design is needed which includes a bypass mechanism for accomplishing this function. The transport or sweeping velocity component is a fundamental requirement of an off-channel fish screen design. Accordingly, a chevron screen configuration is required as was proposed in the SDIP per the above.

For the purpose of the DHCCP two fish protection concepts exist: a screened intake with a return channel, or a screened intake with a salvage facility. The function of a new diversion concept at CCF is to screen fish from water diverted from Old River into the forebay while simultaneously directing those lacking the swimming ability to compete against the transport current into a collection facility or return channel. There are two options for returning fish back to the Delta from this point: using a CHTR salvage facility similar to the Tracy and Skinner Facilities which rely on truck-and-transport methods, or a return bypass channel which relies on water conveyance.

Under the return channel concept, water routed from the intakes to a bypass channel requires an Archimedes (screw) pump or Wemco Hidrastal recessed vortex impeller unit. These pumps protect fish and produce the hydraulic gradient for gravity conveyance into a common bypass return channel. A bypass channel can be constructed following an alignment somewhat parallel to the Old River and can discharge into Franks Tract approximately 12 miles to the north. Potentially, an operable barrier is needed to minimize the opportunity for return to CCF. Additionally, a means of distributing bypassed at Franks Tract is appropriate to avoid discharging concentrated fish at a single location.

The bypass and CHTR approaches are both continuous operations. The CHTR approach relies on gravity for the transport of fish to collection facilities, whereas the bypass channel requires constant pumping. CHTR requires daily manpower, whereas the bypass approach can be fully automated. Based on unreleased information from the fish mortality studies being conducted by DWR in CCF, the CHTR facilities have been found to produce little injury and death of collected fish. Conversely, it is the routine release of fish at the same location that has been found to produce significant loss due to learned behavior of predators at the release site.

According to preliminary results from the forthcoming mortality study, a concept which involves concentrated release of fish at a single location is not considered biologically protective. Furthermore, these findings are deemed contrary to the BDCP and DHCCP objectives. Therefore, a return channel is dismissed from further consideration. Instead, a CHTR facility is included in the makeup of the diversion concept with a variable release plan.

Consistent with these findings, the original SDIP diversion concept is considered a valid solution. However, the SDIP intake concept was originally comprised of five individual, modular fish screen structures designed for 2,500 cfs each. To account for the full 15,000-cfs diversion rate, the individual capacities of each intake module or train are increased to 3,000 cfs herein.

The proposed diversion facilities consist of the following:

- Inlet channel connected at the downstream end to Victoria Canal (channel modifications required to accommodate a CHTR facility on Victoria Canal are described in Section 20.0)
- Self-cleaning trash rack facility at the throat of the inlet channel including a debris conveyor system and a spoil containment/handling area
- Series of pneumatic spillway gates for individual train isolation
- Sedimentation basin with subdivided chambers corresponding to intake modules
- Dual chain-and-flight raking systems and desanding pump systems at each basin train
- Five chevron configuration fish screened intake modules with vertical flat plate screen panels, flow control baffles, and traveling brush cleaning systems
- Five bypass channels or collection pipelines
- Low-head pumping plant to lift water from the inlet channel to CCF during lower tides
- Discharge canal to CCF

The details of fish protection facilities are further described below.

Screened Intake. The intake component of the CCF diversion option consists of a single-source intake made up of several chevron screen intake modules coupled with a common CHTR facility similar to the existing Skinner Facility. This intake option would have to be sized and configured to suit fish protection criteria applicable to Delta fisheries (0.20 fps approach velocity) because the presence of smelt in the vicinity of the intake is greater than in the Sacramento River.

As explained above, a chevron screen configuration is the appropriate screen technology for the application. A chevron intake involves opposing screen banks within a common diversion structure. Like the in-river intake concept, the chevron screen configuration involves dual screens banks within a single intake, thereby reducing structure magnitude and footprint for a given flow rate. The opposing screen banks converge in the downstream direction toward a central bypass channel or pipe inlet. As flow is “peeled off” of the diverted water column through the screens, the transport velocity in between the screen banks is maintained and gradually reduced as the bypass inlet is approached. This provides a constant downstream sweeping velocity within the structure to carry fish into a return channel or fish collection piping.

To suit a total diversion rate of 15,000 cfs, multiple chevron intake modules are required. The facilities are aligned in parallel and butted up to one another to provide separate diversion trains creating manageable compartments that can be individually isolated and accessed.

A series of five side-by-side chevron intakes are proposed, each with an individual capacity of 3,000 cfs each. The chevron intakes involve similar construction approaches and elements as the river intakes described for the North Delta Water Transfer option. In general, the chevron intake concept consists of the following:

- Foundation support system comprised of a cofferdam cell, pile support matrix, mat slab, and reinforced subgrade as needed
- Reinforced concrete flume with access decking and structural steel support framing
- Converging banks of vertical plate, wedge-wire fish screen panels
- Adjustable baffle assemblies behind the screen panels
- Traveling brush cleaning mechanisms at each screen bank
- Bridge or gantry crane systems for screen and baffle removal/insertion
- Intermediate bypasses to prevent excessive fish exposure
- Collection or return piping/channel at downstream limit of fish screen banks
- Fish salvage facility (CHTR)

Fish Salvage Facilities. The collection of entrained fish is performed by passively routing them past the fish screens into a downstream fish salvage facility. The collection piping or channels routed from the downstream limits of the fish screens are extended to holding tanks within a housing facility. The CHTR facilities would be further defined and developed based on lessons learned and owner preferences from the existing Skinner (DWR) and Tracy (Reclamation) Facilities.

The various components above are assembled into a working intake system. Examples of completed chevron screen intakes are provided in the photos below and on Figures 6-3 and 6-4.

Chevron Intake in the Dry

Chevron Intake in Service

The CHTR concept includes collection piping, return pumps, flow scalping screens within the collection channels, holding tanks, bridge crane, transport vehicles, and other ancillary provisions.

6.1.1.3 Optional San Joaquin River Tunnel Intake

According to preliminary hydraulic modeling results for the TDF Option, under future climate change conditions, salinity intrusion within Delta waters would increase with SLR. In order for the TDF Option to be a valid and reliable Delta conveyance solution, a physical bypass may be required where the TDF conveyance corridor and the San Joaquin River intersect. The proposed bypass is a tunnel with upstream and downstream control structures and an intake fish CHTR facility.

The facilities for the Victoria Canal Intake described in Section 6.1.1.2 would be the same for this optional San Joaquin River Tunnel Intake.

6.2 Construction Methodology

Construction of diversion facilities requires means, methods, and approaches unique to marine and heavy civil construction. Although a variety of intake configurations have been conceived for the project conveyance options, construction methodologies are relatively consistent regardless of intake type. This section describes the basic construction approaches and types of construction for intake facilities. A more detailed description of intake construction is provided in Appendix B.

6.2.1 General Constructability Considerations

In general, constructability considerations include, but are not limited to, mobilization and demobilization; contract administration; development of staging/storage areas and construction zones; earthwork, deep excavation, and shoring and bracing; levee construction, slurry cut-off walls, and deep soil mixing; cofferdamming, dewatering, and tremie slab construction; microtunneling, trenching, and pipeline installation; foundation preparation, stabilization, and foundation pile installation; conventional cast-in-place (CIP) concrete construction involving formwork, reinforcing, placement, and finishing; bridge construction (in-river intakes only);

metalwork fabrication, assembly, installation, and structural framing; and miscellaneous civil site and electrical work.

All intake options and sites would possess unique infrastructure complexities, foundation characteristics, and construction periods to complete. Significant temporary construction zones are required for staging and storage. Particular construction challenges include the following:

- Driving sheet and foundation piles to significant depths to achieve hydraulic cut-off. Subgrade of intake structures is in the order of 50 feet below sea level and conveyance piping is at even greater depths.
- Tunnel boring and conduit construction under levees and river channel.
- Underwater construction such as tremie slab placement and sheet pile trimming.
- Cofferdamming, shoring, and bracing.
- Site access and dewatering.

Particular construction elements common or unique to all intake facilities are listed below.

- Staging/storage area and construction zone prep (5 to 10 acres for each river intake/15 to 20 acres for centralized intakes, 20 to 40 acres for river intakes including sedimentation basins and pumping plants/40 to 60 acres for centralized facilities)
- Retrieval pit construction for microtunneling/pipe jacking operations
- Tunnel boring and pipe jacking under levees and river channel
- Sheet pile cofferdamming, shoring, bracing, and hydraulic cut-off
- General earthwork (e.g., excavation, spoil, backfill, levee construction)
- Dewatering wells, construction water treatment, return to watercourse
- Foundation preparation and mat slab construction inside sheet pile cell
- Vertical shaft construction and piping
- CIP reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing)
- Metalwork fabrication, machining, assembly, and installation (stainless steel fish screen panels or cylinders, embeds, flow control baffles, bulkheads, traveling brush screen cleaning system, gantry crane mechanical hoist system, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.)
- Erosion control (underwater placement of stone protection/geotextile)
- Prestressed I-girder bridge construction (in-river intake only)
- Miscellaneous civil site work (e.g. fencing, gates, access roadways and ramps, log booms/debris deflectors, hydroseeding, landscaping, etc.)
- Miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, traveling brush power transmission, flow/level/turbidity/limit/torque instrumentation, utility service)

6.2.2 Construction Variations for Centralized Intakes

Construction of the single-source intakes for the Victoria Canal/CCF and optional San Joaquin River Tunnel diversions is expected to be facilitated due to consolidation of single intake locations and because the construction sites are disconnected from the waterways. In addition, structure depth is not as substantial as the aforementioned technologies, so maintaining a dry working space is expected to be feasible with a conventional dewatering well field.

However, the footprint of the intake structures is considerable, roughly 600 feet in width by 1,000 feet in length. It is expected similar construction approaches used for the earlier intake types are needed including sheet pile cofferdam and support pile installation, mass excavations, dewatering, foundation stabilization, and concrete construction. Deep piping is one element of construction that is not required by centralized intakes. Once primary facilities are complete, the earth between intake and river channel could be removed in the wet. Topside facilities associated with the pumping equipment and fish salvage facilities would not be critical path and could be constructed as a contractor’s schedule allows.

It is estimated the construction of this intake type would require four years to complete, both sites being constructed concurrently by different contractors or construction crews. Because the sites are relatively remote and unencumbered, sufficient staging/storage area and public impacts are not foreseeable concerns. All weather access would be incorporated during the first season once channel realignment and bridge crossing work is completed. Installation of cellular cofferdamming, dewatering, mass excavation, and foundation preparation and pile installation are expected to be performed in the second season. Concrete work is expected for the third season, and mechanical, electrical, and topside facility construction is expected for the fourth season.

6.3 Maintenance Considerations

The proposed intake facilities would require routine or periodic adjustment and tuning to remain consistent with design intentions. Facility maintenance is part of long-term asset management and includes activities such as painting, cleaning, repairs, and other routine tasks to operate facilities in accordance with design standards after construction and commissioning.

Operation and maintenance (O&M) activities would involve performing routine, preventive, predictive, scheduled, and unscheduled maintenance aimed at preventing equipment/facility failure or deterioration. The goal of O&M is to increase efficiency, reliability, and safety. Like any operating facility, maintenance is an integral part of a functional and reliable project.

6.3.1 General Inspections

Routine visual inspection of the facilities would be important for monitoring and logging performance; recording the history of facility conditions and deterioration; identifying trends that occur with respect to river hydrology, climate conditions, and other factors; and preventing mechanical and structural failures of project elements. Continual inspections are important, not only while the facilities are in operation, but also during down times.

A deliberate monitoring program would increase awareness of conditions compromising operational performance and basic function. In terms of relative difficulty, inspections can vary from visual observations from the top surface of facilities to underwater examinations using specially trained underwater diving crews to dewatering for access and firsthand field verification. Video and photographic inspections, along with thorough record keeping of observations, would aid in understanding how well the facilities weather the elements and operate in the face of dynamic conditions. A proactive inspection program is consistent with the asset management of critical infrastructure and would extend the service life of the facilities.

6.3.2 Sedimentation Removal

Sediment deposition is a problem that commonly plagues manmade infrastructure in natural waterways. It can bury intakes in particular and either reduce their capability to divert or force shutdowns altogether until working conditions are restored. Attention to this issue during engineering and design can reduce or avert this problem. However, the dynamic riverine environment can be unpredictable and there is the chance that sedimentation can inhibit function and operations. Typical maintenance activities associated with river intakes can include the following:

- Suction dredging around intake structure using raft or barge mounted equipment and pumping sediment to a landside spoil area.
- Mechanical excavation around intake structures using track-mounted equipment and clamshell dragline from the top deck after installing a floating turbidity control curtain.
- Dewatering of intake/sedimentation basin/pumping plant bays to remove sediment buildup in conduits and channels using small front end loading equipment and manual labor.

The planned operation of proposed intakes would help mitigate sediment deposition within the intake bays and conveyance conduits when turbidity in the river exceeds a certain threshold. The sediment removal systems would be designed to keep sedimentation channels and wet well bays free of sediment buildup. It is expected only extreme conditions would give cause for the activities listed above.

6.3.3 Debris Removal

Should substantial debris become lodged at the leading edge or adjacent to the intake structure, removal of the material may require equipment and specialized labor. Although historically the in-river intake technology has not shown to be a debris trap, there may be incidents where large debris deposits in the vicinity of the structure compromises its function. In the wake of heavy to extreme hydrologic events, inspections should be conducted to visually confirm debris presence or the lack thereof. If large debris is found to have accumulated, removal would require underwater diving crews, boom trucks or rubber wheel cranes, and possibly a small barge and crew to rig the leads to the debris.

With respect to the centralized intake concept, it is expected debris loading in the form of vegetation would be a significant factor hampering facility performance. Accordingly, the proposed design of this intake type includes self-cleaning trash rack mechanisms at the upstream end of the intake facilities. The devices are fully automated and are used to continuously rake debris upward from the trash racks into a continuous debris conveyor at the top deck. The conveyor would transport the collected debris in a spoil area or roll-off containers for convenient off-hauling. The frequency of these efforts would depend on debris load conditions within the river.

As for screen operations, the continuous traveling brush mechanisms, or other screen cleaning technology applied, are expected to maintain a relatively clean screen face and adequate open area. The outbound current is relied upon with this type of cleaning system to transport raked debris past the screen banks once brushed from the surface. Cleaning frequency would need to be varied commensurate with debris load conditions in the river.

6.3.4 Biofouling

Accumulation of algae, freshwater sponge, mussels, and other biological organisms is a known ailment of structures in waterways. Over time, bio-fouling can occlude the screens and jeopardize function. The key design provision for intake facilities is that all mechanical elements can be removable from the top surface for convenience of inspection, cleaning, and repairs as needed. The intakes would feature top-side gantry crane systems for removal and insertion of screen panels, louver assemblies, and bulkheads.

It is expected that all panels would require annual removal (at a minimum) for pressure washing. Additionally, individual intake bays would require dewatering (one pair at a time) for inspection and assessment of bio-foul growth rates. Dewatering is accomplished by closing off portals with pre-fabricated bulkheads.

With the impending invasion of Quagga and Zebra mussels in inland waters of California, it may be simply a matter of time before these biologics would affect operations of intakes in the Central Valley. Frequency of bio-foul removal would intensify with the advancement of these organisms. Coatings and other deterrents would be more thoroughly investigated during preliminary and final design to protect against these invasive organisms.

6.3.5 Corrosion

Because a substantial amount of metalwork would be incorporated in the intakes, aerobic and galvanic corrosion would need to be monitored. Materials are expected to consist of plastics and austenitic steels (stainless), so generally speaking, corrosion is not expected to be detrimental to the life of the facilities. Passive cathodic protection systems may be employed to preserve the condition of submerged metals and thereby extend their service lives. Maintenance associated with these systems generally consists of replacing sacrificial (zinc) anodes at multi-year intervals. Removal of screen and baffle elements for cleaning would allow inspection of metalwork, thereby permitting the assessment of corrosion rates. Metal items receiving coatings would be more prone to localized corrosion attack and, therefore, would be more subject to a routine inspection process involving forensic material testing and metallurgical analyses, similar to that required by the American Water Works Association M42 and D100 standards for water storage reservoirs.

6.3.6 Impact Repairs

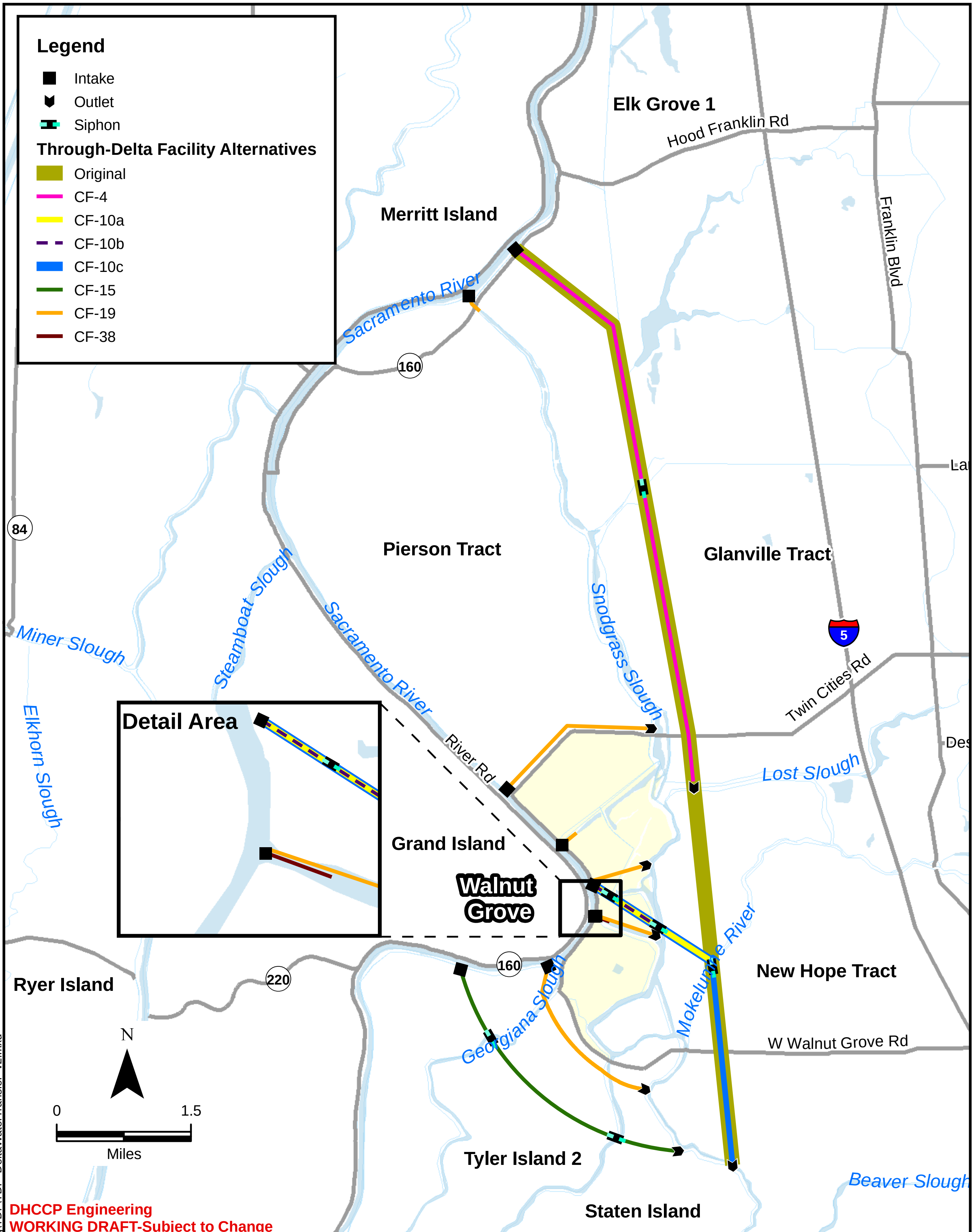
Impact damage incurred by the intake facilities can be the result of incidents such as boat collisions, debris impact, stone and sediment abrasion, etc. Impact damage is not as much a concern for centralized diversions as it is for river intakes. The centralized intakes would be shielded behind substantial upstream trash racks. Because the intakes would be aligned parallel with the predominant flow direction, the brunt of the impact would be borne by the substantial concrete pier nosing at the leading edge of the structure, serving to absorb and deflect downstream traveling debris. Impact damage is not a common problem for intake structures in the Sacramento River; nonetheless, should this occur, repairs would be required.

Elements considered to be most exposed to impact damage are the screen panels, baffle assemblies, and traveling brush mechanisms. The robust concrete structure that houses these elements is quite durable and not expected to suffer much damage. Should the less robust metalwork be damaged, maintenance would consist of removal and repair. With the aforementioned items being constructed of stainless steel alloys, spot repairs would require specialized skills most likely involving shielded gas, wire-feed welders and other common machining tools. Spares should generally be stored on site to minimize downtime should extensive repairs be needed.

With the majority of working components being submerged and adequate security provisions in place, vandalism associated damage should not be a significant problem.

6.3.7 Mechanical Equipment

The only systems associated with the intakes involving power-driven and routinely moving parts are the screen cleaning systems, gantry crane hoist systems, self-cleaning trash rake systems, and pneumatic spillway gates. Lubrication of bearings, continuity checks of limit/torque switches, and periodic inspections of equipment per manufacturer recommendations are the primary O&M tasks expected for these systems. Strip brushes for the screen cleaning systems would need replacement every several years. On-site vendor training and O&M manuals would equip staff with the knowledge of maintaining these systems for years of safe and reliable service.



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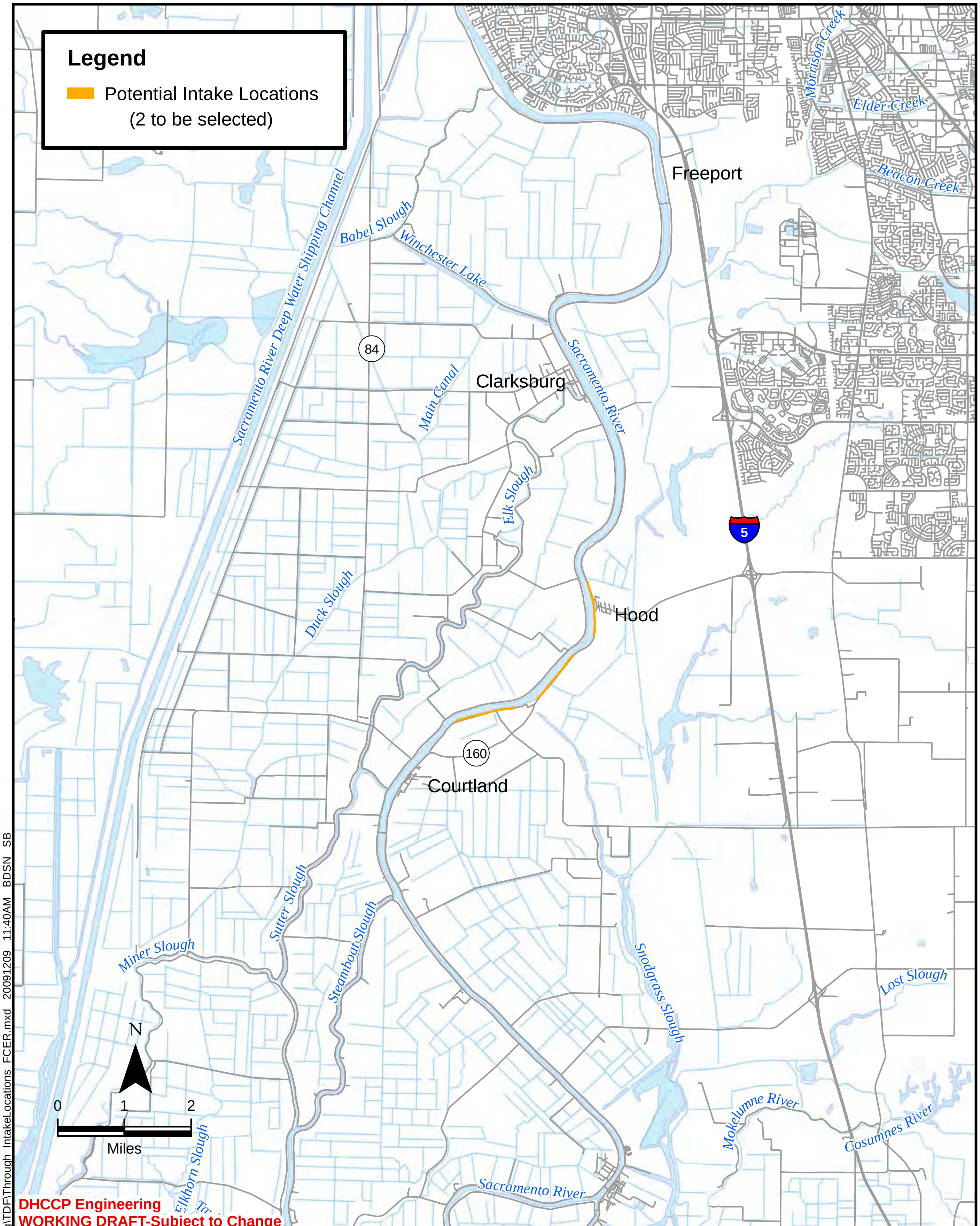
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Example Diversion Schemes for North Delta Water Transfer

FIGURE
6-1



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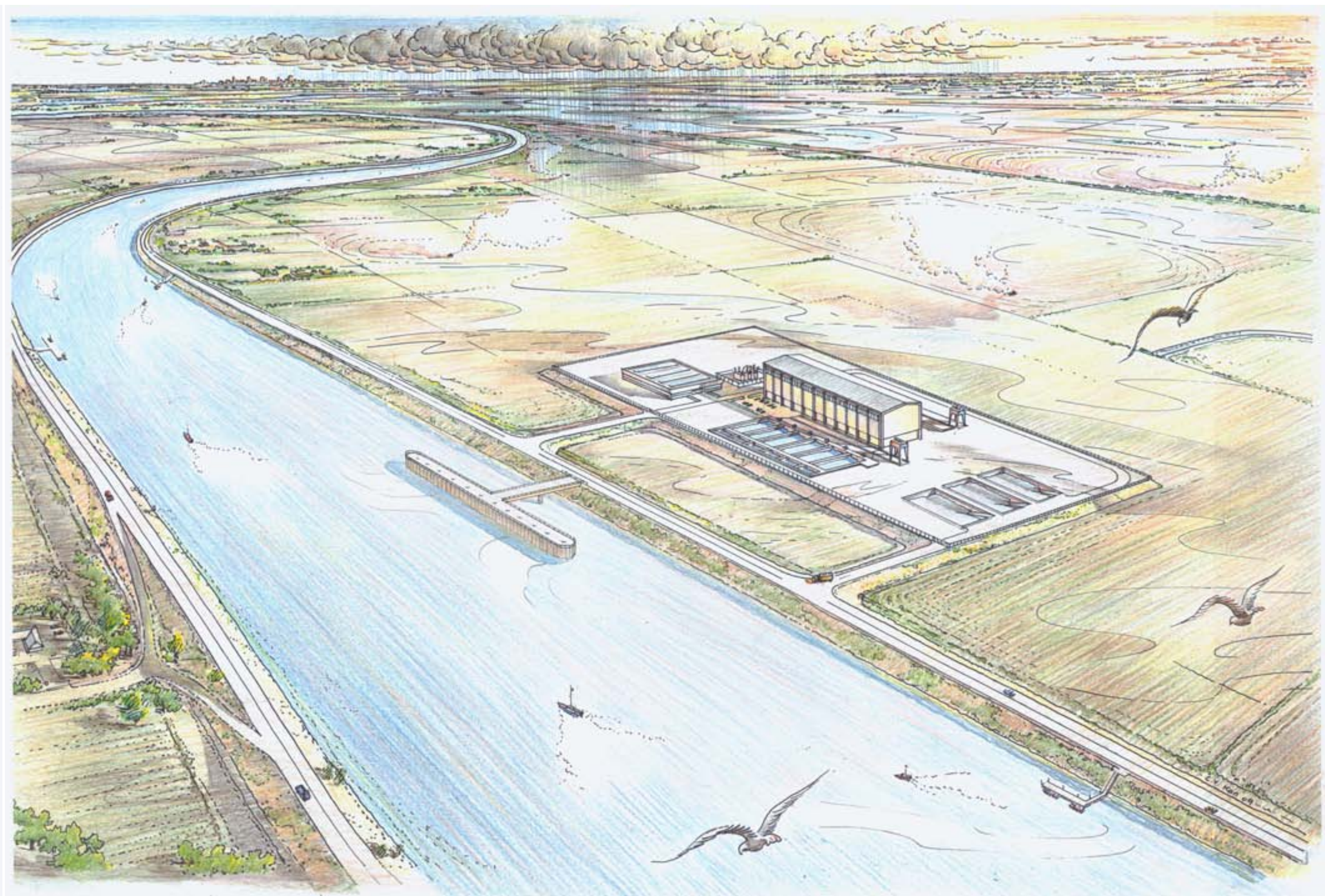
Potential Intake Locations

FIGURE
6-2



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In-River Intake

FIGURE
6-3

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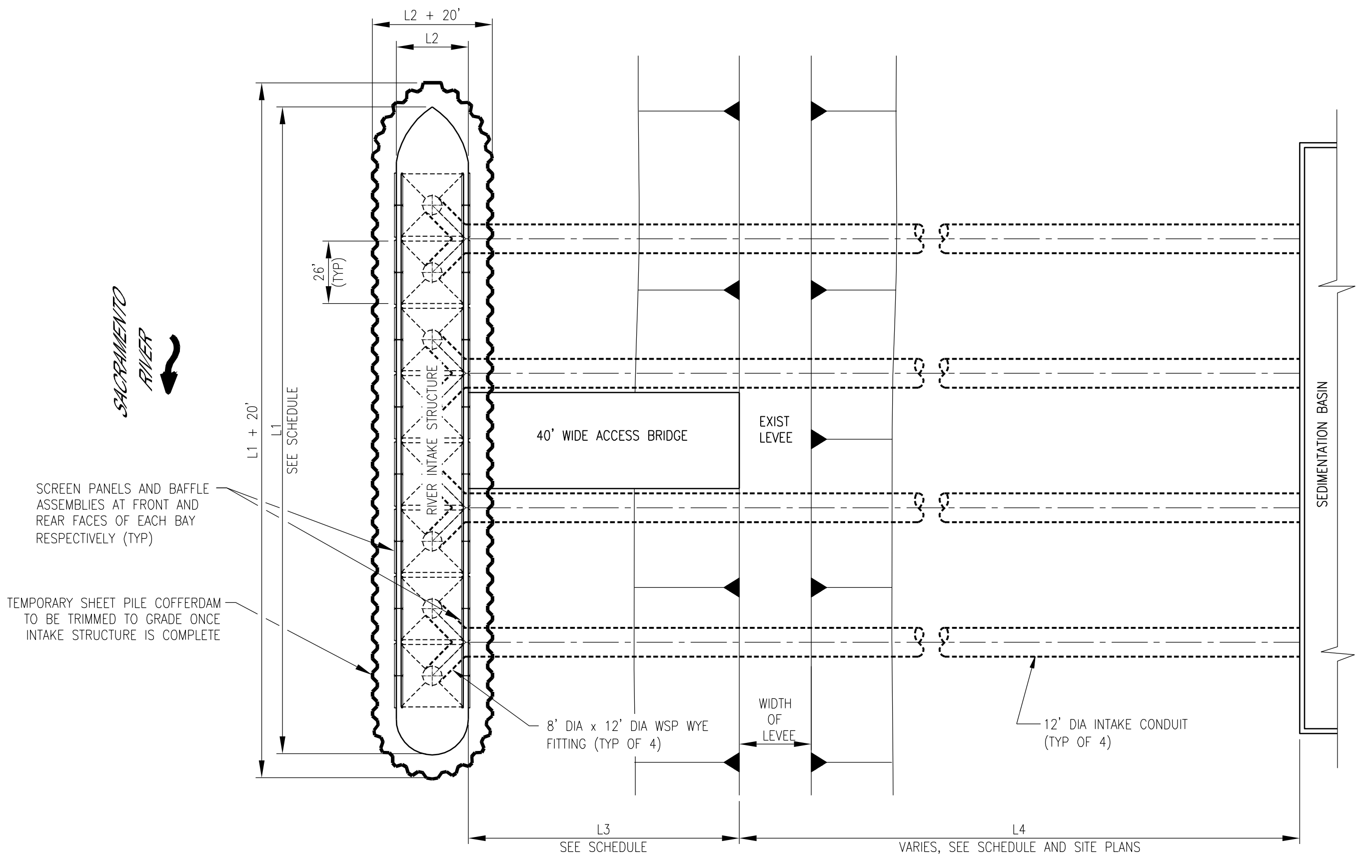
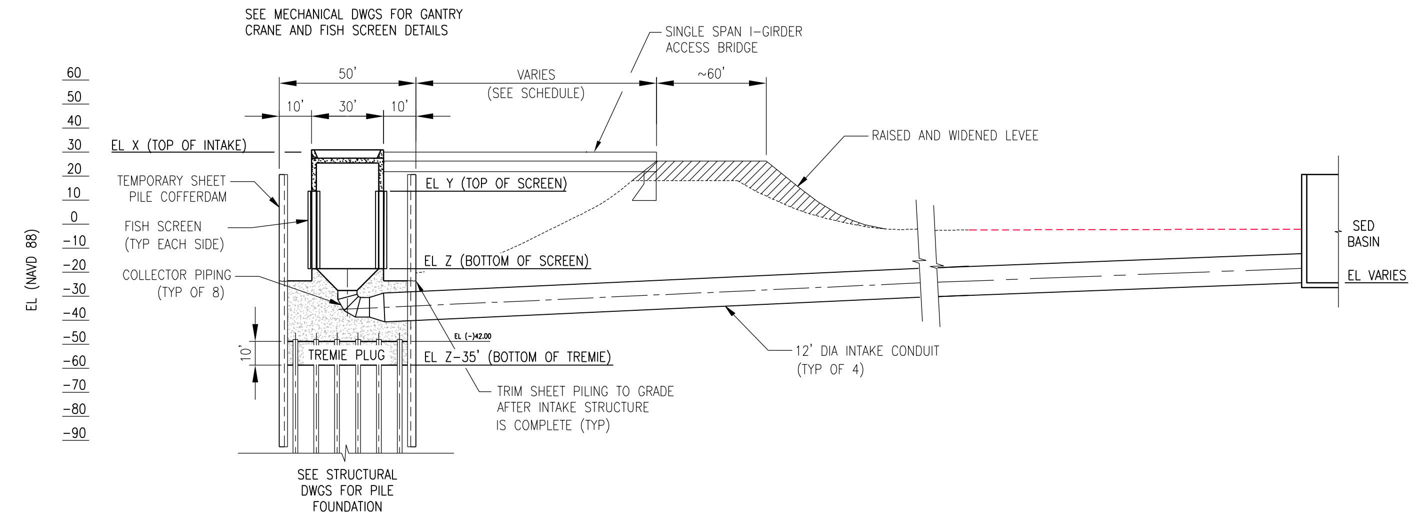


RIVER INTAKE - PLAN
SCALE: 1" = 40'



RIVER INTAKE - PROFILE
SCALE: 1" = 40'

DIMENSION SCHEDULE

INTAKE DESIGNATION	APPROX RIVER MILE	L1*	L2	L3	L4	EL X**	EL Y	EL Z
NORTH DELTA NO. 1	38.3	270'	30'	100'	365'	27.7'	10.0	-15.0
NORTH DELTA NO. 2	36.8	270'	30'	135'	380'	27.0'	9.2	-15.8

* BASED ON DELTA SMELT FISH SCREENING CRITERIA.
** BASED ON USACOE 2002 COMPREHENSIVE FLOOD STUDY FOR 200-YR WSEL PLUS SEA LEVEL RISE PLUS 3-FT OF FREEBOARD.

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THROUGH-DELTA FACILITY OPTION

In-River Intake Facility Plan and Profile

FIGURE
6-4

7.0 PUMPING PLANTS

The pumping plant facilities that would be needed to implement a TDF Option are:

- An intake pumping plant at two river intake structures drawing flow from the Sacramento River and lifting it from sedimentation basins into low-pressure pipelines and into the Supplemental Intake Canal (optional) for gravity flow southward.
- An intermediate pumping plant near the San Joaquin River (San Joaquin River Tunnel sub-option). The water from the natural channels would be lifted through a 15,000 cfs capacity pumping plant (optional SJRTPP) into tunnel entry shafts.
- An intermediate pumping plant at the Victoria Canal. A second, 15,000 cfs capacity pumping plant (VCPP) would lift water from the northern end of the alignment into a new forebay (Byron Tract Forebay) near the existing CCF.
- The channels and fish salvage facilities associated with the two intermediate pumping plants are discussed in detail in Sections 6.0 and 20.0.

Intake and intermediate pumping plant structures presented in this report are engineered to protect against a 200-year flood event, consistent with the DHCCP canal embankment design criteria.

7.1 Intake Pumping Plants

7.1.1 Description and Site Plan

The TDF Option intake pumping plants are summarized in Table 7-1 and depicted on Figures 7-1, 7-2, and 7-3.

Table 7-1: Intake Pumping Plant

Alignment	Total Pumping Capacity (cfs)	Number of Intake Pumping Plant Locations	Total Pumping Capacity at Each Pumping Plant (cfs)	Number of Pumps and Capacity per Pump (cfs)	Number of Sedimentation Basin Channels at Each Pumping Plant
TDF	4,000	2	2,000	4 at 500 Duty 1 at 500 Standby	4

cfs = cubic feet per second
TDF= Through-Delta Facility

Two in-river intakes, each of 2,000 cfs capacity, would be located along the Sacramento River. Flow would be diverted from the river via screened fish intakes. Each intake pumping plant would include a sedimentation basin subdivided into channels, a common transition channel and a pumping plant. Other pertinent facilities would include a solids pumping station, solids storage lagoons and a dewatering pumping station. A substation and transformer would be located on each site to supply power. The diversion channels and fish salvage facilities associated with the pumping plant are discussed in detail in Sections 6.0 and 20.0. This section of the report covers the Sacramento River intake pumping plant and associated equipment.

To protect the site and ancillary structures from flooding, the pumping plant, sedimentation basins and associated solids lagoons would be constructed on engineered fill, with a finished ground level between 27.0 and 27.7 feet (NAVD88) depending on the intake pumping plant location.

It is the hydraulic capacity of the individual diversion structure (intake) that sets the number of pumps and capacity at each intake pumping plant. Maximum flow would normally be passed by all of the pumps in a plant running in parallel, with a spare pump as standby. VFDs are required on all intake pumps to vary pumping rate and regulate velocity through the intake's fish screens.

The intake pumps would be orientated vertically and would operate in parallel. Each pump discharges into an individual, 96-inch (8 foot)-diameter pipe. This pipe size equates to a maximum flow of 500 cfs and a maximum velocity of 10 fps per pipe. Ultrasonic flow measurement on each discharge pipe is used to control VFDs. Flow would be discharged into a transition structure for transfer to the canal. The transition structure is designed to prevent discharge pipes from draining and to maintain a constant discharge head on the intake pumps.

The transition structure and the discharge arrangements at the canal are discussed further in Section 8.0. The details of overall system operation are discussed further in Section 4.0.

7.1.2 Pumping Plant General Arrangement

Pumping plants would be constructed of reinforced concrete and would have multiple floors to house mechanical and electrical equipment. The lower level would include the rectangular pump suction bays, dewatering sump, pump columns, and dewatering pipes. The intermediate level would include discharge piping and valves, access to the pump and motor shaft, hydraulic power units (HPUs), and other miscellaneous mechanical systems. The upper floor (operating level) would house mechanical and electrical equipment, including but not limited to pump motors, pump drives, electrical switchgear and motor control centers (MCCs), fire protection and heating, ventilation, and air conditioning (HVAC) equipment. The building access and upper floor would be located above the design flood level equal to a 200-year-flood EL +55 inches to account for long-term SLR and 3 feet of FB. An elevator and stairway would be provided for each of the pumping plants to allow easy access to the various floors. The roof would be at an elevation suitable to accommodate a bridge crane for hoisting and transporting the pump components and mechanical and electrical equipment.

Preliminary sizing of the pumping plants has been evaluated using vertical column pumps with a vertical motor arrangement. ***The Hydraulic Institute Standards for Pump Intake Design*** (Hydraulic Institute, 1998) identifies minimum pump bay widths and minimum submergence depths, which were considered in determining the pumping plant layout.

Preliminary geotechnical information indicates the presence of peat and softer alluvial material at the pumping plant foundation level. It is anticipated that the pumping plant structures, including the sedimentation basins, would require a piled foundation to transfer the loads to a stronger formation level.

The primary structural support systems used for the pumping plants would consist of reinforced concrete slabs and walls at and below grade, with steel framing and exterior metal wall and roof panels for the above-grade building. The upper floor level, located at grade level, would be reinforced concrete floor slab that would support the vertically mounted pump. It would be enclosed by a steel framed building that includes a traveling 75-ton bridge crane capable of lifting the pump motor or the pump shaft and impellor. The lower level would be a concrete mat slab wetwell that would include reinforced concrete partition walls located at each pump to separate and confine the water flow at each pump suction inlet. Deep foundation piles are anticipated to be necessary to support the heavy dead and operating loads of the building. Foundation piles and a common concrete mat would support the dead loads and operating loads of the pumps and equipment in the building.

The assumed live loads would include equipment loads such as the crane, pumps, and valves. The assumed values would be verified and revised as appropriate to match the most current information as the design is developed. When available, the geotechnical report would be used to confirm assumptions and refine the design with regards to site-specific soil issues.

7.1.2.1 Sedimentation and Solids Handling

Upstream of the intake pumping plant, flow would be diverted from the river via intakes protected with fish screens. The openings for the fish screens would be 1/16th inch in size and would filter out material greater than this size before it could enter the pumping plant. Removal of the larger solids in the river water prior to pumping would improve pump performance and longevity, minimize uncontrolled siltation in the transmission canal and forebay at Clifton Court, and increase the overall performance of the conveyance system. To remove the solids passing through the screen, a sedimentation basin would be constructed upstream of the pumps.

The sedimentation basin would require settled solids collection equipment in each channel, a solids pumping plant and drying/settlement lagoons.

Preliminary information on the nature of the silt that would be encountered in the Sacramento and San Joaquin Rivers is presented in Appendix D.

7.1.2.2 Connection Pipes

From each in-river intake facility, two 8-foot-diameter pipes would be required to convey water from the screened intake structure to the sedimentation basin. The pipes would be installed by micro-tunneling from the dry side of the land to the intake structure under the Sacramento River Levee. The length of pipe would vary with each individual site but in general it would be approximately 250 feet for each pipe.

7.1.2.3 Sedimentation Basins

At each intake pumping plant there would be a rectangular sedimentation basin downstream of the intake screens and upstream of the pump intakes, to allow a proportion of the sediment that has passed through the fish screens to be settled and collected before it reaches the pumps. The fish screens for the intake structures would have openings of 1/16th of an inch and would, therefore, act as an initial barrier to larger sediment. The use of a rectangular basin for subsequent settlement of the smaller particles would allow the water to flow horizontally through a long tank. This would suit the configuration of the intake pumping plant.

To provide uniform approach velocities through the fish screens, the sedimentation basin would be subdivided into narrower sedimentation channels, with each channel notionally serving one duty pump. Subdividing the sedimentation basin would allow each individual basin to be shut down for a prolonged period to remove mussel growth or to perform other scheduled maintenance. The channels would discharge into a common transition channel upstream of the intake pumps allowing any pump to be operated regardless of which channel may be isolated at any time.

The sedimentation basin and common transition channel would be a concrete structure that would accommodate the width of the pumping plant structure at the pump inlet. The basin would have interior concrete walls to create separate sedimentation channels, each sized to accommodate the width of one suction bay. The structural system for the basins consists of reinforced concrete walls and mat slab foundation supported on piles. The walls are to be designed to retain external soil loads and contain internal hydrostatic and dynamic loads. Due to the expected high groundwater level at each site, the basin design includes resistance to buoyancy uplift forces. The significant basin depth required (+/-48 feet) has resulted in exterior concrete walls becoming quite tall, potentially exceeding the economical limits of a conventional cantilever wall system. Given the unusual site conditions, it would seem reasonable that another alternative wall system may provide a more efficient solution. Some examples of alternative wall systems that could be considered are: counterfort/buttress walls; tie-back system; soldier pile system; sheet pile system. A geotechnical engineer should evaluate the compatibility of site-specific soil conditions with the proposed use of any alternative system.

Each sedimentation channel would contain a permanent, mechanical solids collection system to remove accumulated solids. The solids would be transferred using progressive cavity pumps to a solids lagoon for consolidation and dewatering prior to disposal off site. Clarified decant water would be returned to the common transition channel or to the head of the sedimentation basin.

As part of the preliminary design, the sedimentation basin depth was set based on river stage elevations and a minimum water depth in the river of 3.5 feet NAVD88. The flow-through velocity of each sedimentation channel was set as a function of basin depth and width at the given flow rate of 500 cfs per channel. Based on an analysis of the available sediment data and settling calculations for the anticipated particle sizes, an optimum bed length was selected for removing a ‘reasonable percentage’ of the “settleable” solids. The length selected was at least three times the channel width, to limit short circuiting of the flows. On this basis, each basin would be 120 feet long and 40 feet wide. Assuming an average water depth in the Sacramento River of 5 feet EL, and allowing for the design flood elevation, the basin would be approximately 48 feet deep overall. The bottom of the basin would be at an EL of -22.5 feet (NAVD88) and the top of the walls at the flood protection elevation. Based on the river flow exceedance charts, for 65% of the time, the basin would operate with water ELs of between +2 and +3.5 feet (NAVD88) after considering head losses through the inlet.

The typical velocity through each channel would be 0.48 fps. The water level in the basin would be dictated by the water level in the river. The flow of water through the basin would be dictated by the pumping rate of the intake pumps. The channel inlet and channel sedimentation zone would be separated by a fixed weir and baffle wall to distribute the flow uniformly into the inlet zone and reduce streaming at high river levels. Each channel would have two adjustable weirs (rising sluice gates) at the downstream end, discharging into the common outlet channel. The water level in this channel would be controlled by the minimum pump stop/start levels.

7.1.2.4 Pumps

The intake pumping plant concept includes five 500-cfs pumps (four duty and one standby). Flow would be directed to the pumps via a tapered, common transition channel downstream of the sedimentation channels. The transition channel would enable any of the pumps to be operated regardless of which sedimentation channel is offline for maintenance purposes. Pumps would be installed in parallel, and in the vertical position. All motors, VFDs, and electrical equipment would be located on or above the operating floor having an elevation above the flood protection elevation.

Each pump discharge pipe would include a hydraulically operated butterfly valve, a redundant motor operated butterfly valve, and an ultrasonic flow meter. The hydraulically operated butterfly valve would provide a check valve function; therefore, no separate check valves would be provided. All discharge piping would connect to a transition (weir) structure outside the pumping plant structure, then transition into two force mains which would connect to the proposed canal.

At this stage, vertical column axial/mixed flow and vertical volute mixed flow pumps have been identified as the most likely type of pumps to be used for the large flow and low head application.

Vertical column axial/mixed flow pumps are manufactured for wet well installation with the pump discharge elbow located either above or below the motor mounting floor. These pumps can also be connected to a formed suction inlet. The pump line-shaft is located inside the pump column and can be open type or enclosed type. Impeller submergence is required to prevent cavitation. Vertical axial/mixed flow pumps are typically started in the wet.

Vertical mixed flow volute pumps are manufactured with a vertical volute casing. They are typically installed in dry pits with the suction pipe connected to the wet well, or with a formed suction inlet. The pump impeller can be withdrawn from the pump casing without disconnecting the piping. The volute casing can also be embedded in the concrete structure to simplify pump support and reduce vibration and noise. Volute pumps can be started with the impeller either wet or dry. A compressed air system would be required to depress the water level below the impellers when starting dry. Starting the pump in the dry reduces the inrush current and hence reduces the instantaneous power load. When starting the pump wet, the pump casing would be filled with water resulting in a high inrush current and high instantaneous power requirement. One way to reduce the inrush current and starting power load for a wet start is to start the pump at a reduced speed or at a lower voltage.

Vertical mixed flow volute pumps have been installed at both Banks and Jones Pumping Plants and O&M staffs are familiar with the pump operation procedures and maintenance requirements.

The pumps are discussed in further detail in Appendix D.

All of the intake pumps would be located in separate pump bays with stop logs at the suction inlets, such that each individual pump can be isolated in its own bay for inspection and cleaning.

Flow-through/trash screens would also be provided to filter out any large debris that may have entered the common transition channel. Each pump bay would conform to Hydraulic Institute Standards, to provide adequate width for maintaining velocities less than 1.5 fps and sufficient submergence for acceptable pump performance.

The pump bays, sedimentation basins, valve vaults, and common transition channel would be dewatered using a dewatering pumping station. Preliminary estimates show that the dewatering sump would be approximately 20 feet long by 16 feet wide and deep enough to allow for gravity flow from the bottom of all basins. The pumping plant would be located at the lower level floor of the intake pumping plant. The dewatering pumps would be guide-rail mounted submersible pumps capable of passing solids, operating as one duty and one standby. The pump discharge would discharge into the upstream end of the sedimentation basin.

7.1.2.5 Solids Handling Facilities

The sedimentation basin located upstream of the intake pumps would remove the settleable solids that are smaller in size than the openings in the fish screens, but larger than the very fine silt that is predominantly present in the river water. The solids would mainly be comprised of medium-fine sand of between 0.25 to 1 millimeter in size. Solids collected by the mechanical sediment removal system in each sedimentation channel would be siphoned off to a solids pumping station at the downstream end of the basin. Solids would be pumped directly via positive displacement pumps/progressive cavity pumps to storage lagoons for further settling and disposal. The suction generated by the pumps would allow the dry well of the solids handling pumping station to be constructed at a shallower depth than the basins themselves.

Due to its depth, the dry well would require forced ventilation linked to a timer at the access. The main pump starters would be located on a mezzanine floor at the top of the well, with local control panels and emergency stops at the bottom. Access into the bottom of the well would be via a stairway with landings at every 12-foot change in elevation. A permanent sump pump would be installed for floor drainage.

Another arrangement would be to use self priming pumps, installed on the concrete deck (walkway) above the sedimentation basin, to collect and pump solids from the sedimentation basin to the solids lagoons. This arrangement would not require the construction of a deep, dry well sump. Due to the depth of the basin, the viability of this option would need to be evaluated as part of further design development.

7.1.2.6 Solids Lagoons

A mass balance approach was adopted for the calculation of the required lagoon size, using monthly suspended solids data. This allowed the lagoons to be sized based on measured data (i.e., the weight of suspended solids in the river water throughout the year). For the purposes of conceptual engineering, calculations were based on a worst case scenario by considering the throughput of the intakes for a maximum flow of 3,000 cfs. The assumed data, such as the throughput of the intake pumping plant and the volume of water transferred during solids pumping, would be refined as the engineering progresses to optimize the size and operation of the lagoons.

Assuming the intake is operating constantly at maximum throughput, the average daily mass of solids that would be expected to settle in the basin for subsequent transfer to the lagoons is approximately 137,000 pounds per day (lb/d). In the worst month (based on published Sacramento River studies), the daily mass of solids would be expected to peak at approximately 253,000 dry lb/d.

After a year, the accumulated volume of solids would notionally be 486,000 cubic feet on a dry solids basis. Using a bed load of 30 cubic yards (cy), this equates to 600 truckloads of dry solids for off-site disposal.

For this stage of engineering, it is proposed that three lagoons be constructed. The three lagoons would be used in a rotating cycle of lead, lag, and standby operation with one basin filling, one settling, and the third being emptied of settled and dewatered solids. The lagoons would be 10 feet deep and would have sloped sides with a top width of 86 feet and a top length of 165 feet. They would be concrete lined to prevent seepage to the groundwater or adjacent river bed. The lagoons would be installed on the raised area of on the elevated area of the site at an elevation that would allow them to function with other parts of the plant and during the design flood condition.

Clarified top water would be returned by gravity to the head of the sedimentation basin.

Once the preferred operational regime of the solids collection and removal system is developed, the solids transfer pumping from the sedimentation basin, the inlet and decant arrangements at the lagoons, and the number of lagoons would be refined to enhance solids transfer and subsequent disposal.

The nature of the solids anticipated to settle in the diversion channels at the fish salvage facilities would be different to that at the river intakes. However, a similar mass balance approach can be adopted to estimate the notional volume of solids that could settle within the diversion channels. Assuming that a diversion facility is operating constantly at a maximum throughput of 15,000 cfs, the average daily mass of solids that would be expected to settle in the basin for subsequent transfer to the lagoons is approximately 906,000 lb/d. In the worst month (based on published Sacramento River studies), the daily mass of solids would be expected to peak at approximately 1,269,000 lb/d.

After a year, the accumulated volume of solids would notionally be 3,308,000 cubic feet on a dry solids basis. Using a bed load of 30 cy, this equates to 4,100 truckloads of dry solids for off-site disposal.

7.1.2.7 Substation

One of the intake pumping plants would include a major substation while the second intake pumping plant would have a smaller substation. The location of the major substation would be determined during design development. The footprint for the major substation would be approximately 250 feet by 250 feet. The smaller substation would have a footprint of 150 feet by 150 feet. The substation would be located adjacent to the pumping plant and the elevation of the substation would be the same as the pumping plant operating floor.

7.1.2.8 New Levee and Levee Road

The intake pumping plants would be protected from a 200-year-flood event consistent with DHCCP embankment design criteria. The embankment would be constructed with a 3H:1V side slope, and a 200-foot distance would be maintained between the toes of embankment and the existing levee. See Figure 7-1 for general layout.

7.1.3 Pumping Plant Mechanical Systems

Mechanical building systems’ requirements would be similar for both the intake pumping plants and the intermediate pumping plants at the optional San Joaquin River Tunnel and the Victoria Canal. Mechanical building systems would conform to the requirements of the State of California Title 24, the California Mechanical Code and other applicable codes and would include HVAC, plumbing, and fire protection systems.

Various areas of the pumping plant would be provided with HVAC systems to maintain desired temperatures for equipment protection and human comfort, and to provide adequate ventilation (Table 7-2). Natural ventilation would be used for cooling where feasible; fans, louvers, and ductwork would provide additional capacity. Packaged or central type air cooled direct expansion refrigerant systems would be used for air conditioning where necessary; water cooled air conditioning equipment would be used where feasible. Air conditioning systems would include economizer components to utilize outdoor air for cooling. Heating would be provided by electric unit heaters and heat pumps.

Table 7-2: Pumping Plant HVAC System Overview

Room Type	Summer Design Temperature (°F)	Winter Design Temperature (°F)	Cooling System Type	Heating System Type
Mechanical Equipment Rooms, Storage Rooms	104	55	100% Outside Air	Electric Unit Heaters
Electrical Rooms	90	55	Air Conditioner	Electric Unit Heaters
Control Room, Restroom	78	72	Heat Pump	Heat Pump

°F = degrees Fahrenheit
% = percent

Plumbing systems would be provided for various areas of the pumping plant. Roof drains, where needed, would discharge to splashblocks at grade; floor drains and mechanical equipment drains would be provided with gravity drainage to a plant drainage sump equipped with duplex sump pumps. Restroom fixtures would be provided with sanitary gravity drainage to a wastewater holding tank. Oil rooms would be provided with separate sumps. A potable water supply system would be provided for the welfare facilities and any safety showers, if required. If available, potable water supply would be taken from the nearest clean water conveyance system. If not available, a self-contained water filtration and treatment system would be included. The availability of potable and non-potable water sources would be determined during detailed design. Raw water downstream of strainers would be evaluated for use in a non-potable water system serving hose faucets and water-cooled condensing units for electrical equipment and HVAC hydronic cooling systems.

Fire suppression systems would be provided as required by the authority having jurisdiction. The type of fire suppression systems would be evaluated for pump motors, electrical rooms, and oil storage rooms.

Stairs and an elevator would be provided for accessing different levels of the pumping plant structure. All pumping plants would include operational and storage areas, including those described below.

Restroom. Space would be allocated in the building for male and female restrooms. Each would be American Disabilities Act compliant and would include shower facilities and lockers.

Control Room. Space would be allocated in the building to provide for a control room. This room would include desk space, two operator work stations, a SCADA server, printers, programmable logic controller (PLC), and communication equipment.

Electrical Switchgear and MCC Room. The building would contain a room for housing the electrical switchgear equipment, MCC, lighting transformer, lighting panel, and other associated electrical equipment. The size of this room would vary depending on the quantity and size of the pumps and hence motors and pump drives.

Oil Storage Room. Each pumping plant would have a room for the hydraulic oil system for lubricating the pump bearings and the actuators for the discharge valves. This room would also provide space for oil drum storage, fire suppression system and other mechanical equipment that has yet to be identified.

Gate Storage. Each pumping plant would include space allocated for gate storage for the spare and temporary isolation gates for the suction inlets of the pumps. This room would also provide storage for the spare flow-through screens that would be located on the pump bay inlets. The isolation gates and/or flow-through screens would be hoisted and transported using a mobile, self-supporting gantry type crane. An outdoor space at each pumping plant would be provided for stop logs storage. The stop logs and flow-through screens would be hoisted and transported using a rail-mounted, mobile, self-supporting gantry type crane.

Pump and Motor Storage Room. From discussions with DWR staff, it is understood that the preference would be to service the pumps off site. Space would be provided for storing the pump motor or other mechanical equipment and to allow a hauling truck to access the equipment for transportation away from the site. A bridge crane would be included in the design for transporting mechanical equipment from the pump motor room to a laydown area.

Electrical Drive Room. Each pumping plant would have a room for housing VFDs for pump motors and other associated electrical equipment. The type of drives to be used for operating the pumps has not been finalized. Based on preliminary information, conservative values have been used to determine the size of the drives and thereby establish a footprint for the drive room. Electrical drives for pumps of this capacity can produce a great deal of heat. They would be fully enclosed in a separate room with its own HVAC system or, alternatively, the VFDs would be water-cooled using the motor cooling system.

Battery Room. Each pumping plant would have a room for housing 125-volt direct-current (DC) battery banks, two battery chargers, and DC switch board. The room would be continuously ventilated to exhaust hydrogen gas.

Pump Motor Room. The building footprints would vary the most depending on the number of pumps and hence the size of the pump motor room. This room would include pump mechanical equipment and motor, concrete vaults for accessing discharge piping and valves, HVAC system, fire suppression system, bridge crane, and other miscellaneous equipment.

Dewatering Sump. It is anticipated that each pump would be shut down periodically for routine maintenance. The pump would need to be dewatered on these occasions. The pumping plants would include space for a sump and duty/standby sump pumps for collecting and removing or recycling raw water that is retained in the suction and discharge lines during pump isolation and shutdown. The dewatering sump would also be utilized for dewatering the sedimentation basins and the common transition channel. An oil separator would be included on the discharge from the sump.

Flow Meter Vault. An ultrasonic flow meter would be installed in a flow meter vault on each pump discharge pipe. The flow meter would be the preferred method of controlling the speed of the pump VFDs to regulate velocity through the intake screens.

Cooling Equipment. Handling the heat rejection from the VFDs is anticipated to be difficult because they are large in size; close coordination with site and electrical design would be a priority during detailed design. With the mild climate and the heat rejection anticipated from the motors and electrical gears, there would not be much need for heating in the equipment and electrical rooms. Alternative energy sources such as solar hydronic were considered but were deemed to be cost prohibitive for the rare instances when heating would be needed.

Pump Discharge Valves and Actuators. At the discharge of each pump, two butterfly type isolation valves would be provided. The first isolation valve would be hydraulically actuated and the second one electrically actuated. No check valves are planned at the pump discharge.

The electrically actuated valve would normally remain open, and would be used only when the hydraulically actuated valve is not available. Power supply for the electrically operated valve would be 480 V/60 Hz/3 Phase. The electric actuator would have a manual override and hand wheel to facilitate operation of the valve even if there is a power failure. The valve would be designed for remote/local manual initiated operation.

The hydraulically operated valve would remain closed during pump start and stop. Either after a pre-set time after pump start or based on a pre-set pressure at the pump discharge line, the hydraulically actuated valve would start to open. This is done to create a backpressure on the pumps during start up and also to help in releasing air from the pump column through air release valves installed upstream of these valves. The size and pressure rating for the valves would be selected to match the pump discharge pipe diameter and pump shutoff head. Hydraulic actuators appropriate to the torque requirements would be provided. The operating time of the valves would be decided based on any specific requirements (like surge conditions) to be established during detailed design stage. Operating pressure for the hydraulic actuators would be 1,500 pounds per square inch (psi).

The pump discharge valves would be installed in vaults, along with accumulators for each valve and the valve actuators would be designed to open/close the valves against pump shut-off head. No bypass is planned for the valves. Sumps with pumps would be provided to collect any spillage in the valve vaults that drain to the pumping plant dewatering sump.

Hydraulic Power Unit for Valve Actuators. For actuating the hydraulically actuated valves a HPU would be provided at each pumping plant. The HPU would consist of an oil tank, two electrically operated pumps on a duty/standby configuration, a manually operated pump, oil filters, solenoid operated directional valves, isolation valves, relief valves, accumulators, piping, instrumentation, and controls. The hydraulic pumps and tanks would be sized to actuate one valve at a time. Preliminary estimates suggest a 400-gallon hydraulic oil tank. Adjacent to each of the hydraulically actuated butterfly valves, accumulators would be provided. The accumulators would have adequate capacity for two strokes of the hydraulic actuator without a recharge to achieve valve closure in the event of power failure. A common discharge header would feed each of the accumulators.

The accumulators would be bladder type pre-charged with nitrogen. No compressed air system would be provided. The oil tank, valves and hydraulic pipes would be made of stainless steel.

Motor Cooling Water System. Water-cooled motors would be provided for intake pumps. For jacket cooling of motors, some manufacturers recommend potable water and others prefer high quality water. In any case, the raw water being pumped by the intake pumps would not be suitable for direct cooling. Two possible options are either a single-pass potable water system or a closed-loop cooling water system

A closed loop cooling water system would consist of a tank, pump, a heat exchanger, pipe, and valves in the primary circuit. A secondary cooling water supply to the heat exchanger would be required. The secondary loop would include a pump to draw water from the wet well, pump it through the heat exchanger, and discharge it back to the wet well.

Each intake pump motor would be provided with a dedicated motor cooling water system linked to the pump start and stop. The cooling units would be interconnected to allow cooling of a unit when the cooling system of that unit is out of service. No standby cooling water pumps are planned, but spare pumps (for shelf storage) would be provided. The cooling unit may also cool the VFDs.

7.1.4 Pumping Plant Electrical Systems

The electrical systems would conform to the latest editions of the State of California Title 24, State of California Title 8, National Electrical Code National Electrical Safety Code, Life Safety Code, American National Standards Institute, Illuminating Engineers Society, Institute of Electrical and Electronics Engineers, Insulated Cable Engineers Association, and Underwriters Laboratories standards and codes.

7.1.4.1 Power Distribution Planning

Table 7-3 lists the distribution and equipment utilization voltages and ratings that generally would be used. Depending on the specific equipment requirements determined in design, there may be some exceptions:

Incoming power to the intake pumping plant would consist of two 69-kV feeders connected to two 69-kV/4,160-volts outdoor, oil-filled transformers. Two 4,160-volt feeders from the transformers would be connected to an indoor lineup of switchgear arranged in a main-tie-main configuration. The two incoming feeders would be installed in an underground ductbank system to supply approximately 8 megavolt amperes (MVA) power for each pumping plant. The switchgear breakers would feed the VFDs and station auxiliary transformers.

Two outdoor, oil-filled, station auxiliary transformers would step the voltage down to feed the 480-volt MCCs to distribute power and control to pump motors, process equipment, and auxiliary power loads such as the plumbing and HVAC systems, and power and lighting transformers and panels.

Table 7-3: Distribution and Equipment Utilization Voltages

Above 500 horsepower motors	4,000 volts or 6,600 volts, three-phase
Motors, 1/2 horsepower up to 500 horsepower	480 volts, three-phase
Motors, less than 1/2 horsepower	120 volts, single-phase
Motor Control Voltage	120 volts, single-phase
MVA Switchgear Control Voltage	125 volts direct current
Lighting	120 volts, single-phase
Convenience outlets	120 volts, single-phase

MVA = megavolt amperes

7.1.4.2 Lighting

Table 7-4 lists the following general types of light sources to be used.

Table 7-4: Pumping Plant Lighting

Area	Light Source
Office None above 8 feet; mount along walls for ease of bulb change	Fluorescent
Process, above mounting height of 14 feet	High-pressure sodium vapor
Storage, indoors	Fluorescent
Walkway, indoors	Fluorescent
Walkway, outdoor	High-pressure sodium vapor
General site	High-pressure sodium vapor

Where fluorescent lights are indicated, fixtures with energy-saver ballasts and lamps would be used. Outdoor lighting fixtures would be luminaries with individual photocells. Critical paths, entrances, and walkways would be illuminated. High bay lighting fixtures would be high-pressure sodium vapor instant-on lamps.

7.1.4.3 Control Modes and Control Basis

In general, all equipment at the pumping plant would be operated in local or remote control modes as described in Section 18.0.

In general, all equipment at the pumping plant would be operated in one or more of the following control modes listed in Tables 7-5, 7-6, and 7-7, and described in Section 18.0.

Table 7-5: Intake Pumping Plant

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel or VFD	Remote PLC/ SCADA Control
Pump P01 Typical	Pump (VFD driven)	Emergency stop Low suction level pump stop hardwired.	Start – stop, local-remote & speed control	Start – stop, & speed control based on operator entered Flow setpoint vs. total PP flow. Low pump suction level stop.
	Open, closed hydraulically actuated valves on each main pump P01 discharge typical	Local – Remote switch at valve, open, off, close switch at valve		Open-off-close, manual, auto in auto valve closes prior to pump start. Valve opens either preset time after pump start is initiated or at pre-set discharge pressure on pump discharge line.
	Open, closed motor actuated valves on each main pump P01 discharge typical. Valve is normally open and remains open. This valve is used to backup hydraulic valve.	Local – Remote switch at valve, open, off, close switch at valve		Open-off-close, manual, valve normally open. Manually initiated operation no auto mode
Dewatering Pump	Dewatering pump, submersible pump located in dewatering sump inside pumping plant		Local-Off-Remote at Local Control Panel	Manual operator-initiated start. Low-level transmitter pump stop.

PP = pumping plant
PLC = programmable logic control
SCADA = supervisory control and data acquisition
VFD = variable frequency drive

Table 7-6: Solids Pumping plant Area at Intake Pumping Plant

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel or VFD	Remote PLC/ SCADA Control
Solids Pump	Pump (constant speed)	Local – Off-Remote Local or remote start of solids vacuum collector initiates solids pump start		Start-stop, local-remote
	Open closed motor actuated valves on each solids pump suction typical	Local – Remote switch at valve, open, off, close switch at valve		Open-Off-Close manual, auto, auto operation TBD
	Open closed motor actuated valves on inlet to each sludge lagoon typical	Local – Remote switch at valve, open, off, close switch at valve		Open-off-close manual, auto, auto operation TBD

OOC = Open-Off-Close
PLC = programmable logic control
SCADA = supervisory control and data acquisition
TBD = to be determined
VFD = variable frequency drive

Table 7-7: Sedimentation Area at Intake Pumping Plant

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel or VFD	Remote PLC/ SCADA Control
Outlet Weir Gate	Modulating motor actuated rising weir gate at sedimentation basin outlet	Local – Remote switch at valve, Open-Stop-Close switch at gate		Operator selectable percent open manual, auto, in auto gate controlled to maintain sedimentation basin level setpoint.
Solids Vacuum Collector	Variable speed Solids Collector	Local-Off-Remote	Start – stop, Local – remote and speed control OIT at local control panel	Start – stop, speed control
	Open closed motor actuated valves on sedimentation basin discharge to solids pumping plant lagoon typical	Local – Remote switch at valve, open, off, close switch at valve		Open-off-close manual, auto, auto operation TBD

OOC = Open-Off-Close
PLC = programmable logic control
SCADA = supervisory control and data acquisition
TBD = to be determined
VFD = variable frequency drive

7.1.5 Construction Methodology

7.1.5.1 General Considerations

The pumping plants would be constructed in the following steps:

- Mobilization
- Clearing and grubbing
- Installing stormwater pollution prevention best management practice (BMP) devices.
- Rough grading
- Constructing access road and setback levee
- Installing dewatering system
- Constructing connection pipe between intake structure and sedimentation basins
- Installing sheeting and shoring system for cofferdam around excavation footprint
- Excavation
- Pile driving
- Constructing building and structures
- Installing substation
- Installing piping, conduits, and ductbank
- Installing mechanical equipment
- Remove sheeting and shoring system for cofferdam
- Commissioning and acceptance testing
- Final grading, landscaping, and paving
- Demobilization and cleanup

The construction site area is influenced by the depth of excavation and footprints of the pumping plant, sedimentation basins, solids handling facilities, and substation. The construction site area also includes space for staging, equipment, materials storage and laydown area, spoils storage, and parking. To meet the needs of construction, an area of approximately 30 to 35 acres would be required at each pumping plant site.

7.1.5.2 Site Layout

The size of equipment and infrastructure is based on hydraulic and project requirements, whereas the layout is based on ease of construction and operations. All pumping plants would include a CIP reinforced concrete structure and a superstructure, power substation, access road, flood protection levee, parking and security fencing. In addition, intake pumping plants would have concrete sedimentation basins and associated solids handling facilities.

Access to the construction site would be via existing levee roads and/or existing roadways. Additional temporary access roads and haul roads may need to be constructed between the existing roadways to the site.

Of the 30 to 35 acres required to construct each intake pumping plant, approximately 20 acres would be temporary use only for construction staging, equipment laydown, material storage, spoils storage, temporary office trailers, access road, stormwater detention pond, and parking area for contractors and construction equipment.

Table 7-8: Summary – Area Disturbed with Setback Levee at Intake Pumping Plant

	Total per Intake Pumping Plant	Total for Two Intake Pumping Plants
Permanent (acres)	16	32
Temporary – Construction (acres)	20	40
Total (acres)	36	72

Table 7-9: Summary – Area Disturbed without Setback Levee at Intake Pumping Plant

	Total per Intake Pumping Plant	Total for Two Intake Pumping Plants
Permanent (acres)	11	21
Temporary – Construction (acres)	20	39
Total (acres)	31	61

Table 7-10: Summary – Structure Footprint for Intake Pumping Plant

	Dimensions (feet x feet)	Area (ft^2)
Connection Pipes	170 x 250	42,500
Sedimentation Basin	170 x 120	20,400
Pumping Plant	258 x 108	26,580
Substation	150 x 150	22,500
Solids Wetwell and Pumps	35 x 35	1,225
Solids Lagoons (3 No.)	165 x 100	49,500

ft^2 = square feet

Heavy construction equipment, such as diesel powered dozers, excavators, rollers, dump trucks, fuel trucks, and water trucks, would be used during excavation, grading and construction of access/haul roads and the site. Dust or mud generated from these activities would be controlled using water trucks, street sweepers, and other stormwater BMPs as necessary.

Existing soils unsuitable for reuse as backfill would need to be removed off site. Suitable materials would be brought in to backfill around the structures, pipes, and electrical ductbanks.

The volume of material to be hauled is estimated in Table 7-11. The total number of truck trips during the course of earthwork is estimated to be 64,322 for the two 2,000-cfs intake pumping plants, assuming 10-cubic-yard dump trucks are used.

Table 7-11: Summary – Earthwork at Intake Pumping Plants

	Total per Intake Pumping Plant	Total for Two Intake Pumping Plants
Total Excavation (cubic yards)	230,000	460,000
Excess Soils to be Exported (cubic yards)	320,000	640,000
Imported Backfill Material (cubic yards)	16,800	33,600

7.1.5.3 Intake Pipes

Work breaching the existing levee would need to be done during the dry season. Installation of the 12-foot intake pipes would require berm or cofferdam construction. A berm could be constructed using alluvial mineral soil if excavation is done prior to intake pipe installation. If intake pipes are installed before site excavation, or if sufficient quality or quantity of alluvial mineral soil is not available on site, material would need to be imported on site to construct the berm. Depending on the soil conditions, piling may also be needed upstream and downstream of the intake pipes to protect the bermed material from erosion. If it is determined that a berm would not provide sufficient protection due to river conditions or extended duration of construction, cofferdam construction would be necessary. Material excavated from the existing levee would be stored on site to be used in reconstructing the Sacramento River levee after intake pipes installation is complete.

The intake pumping plants are physically connected to the intake facilities through pipelines; therefore, construction of a setback levee would also be needed to provide uninterrupted roadways for the public during construction. It is also anticipated that the tunneling method for construction of connecting piping between intake facilities and pumping plants would require building a temporary tunneling pit adjacent to the pumping plants.

7.1.5.4 Sedimentation Basins

The base slabs for sedimentation basins and pump buildings would be located below ground level, thus excavation to -35 feet would be necessary. Most of the soils from ground level to 30 feet below ground are expected to be unsuitable and would be removed.

7.1.5.5 Intake Pumping Plant Structures

Based on the available geotechnical information, the pumping plant structures would need to be supported on pilings. Piles would be driven into the soil until they reach refusal. The soil inside the steel pile would be removed by auger machine and replaced with reinforcing steel and concrete.

Interlocking steel sheeting and shoring systems are anticipated to hold back soils and groundwater and achieve required excavation depths. Vibratory hammers and/or drop hammers would be used to drive sheet piles and pilings.

Because of relatively high groundwater levels in all potential sites, an electric-powered dewatering system is anticipated to lower the groundwater during construction. Groundwater would be removed, treated as necessary, and disposed under a National Pollutant Discharge Elimination System (NPDES) permit. Dewatering would be a continuous operation until tremie concrete develops adequate strength and weight to resist buoyancy force. It has been assumed that up to 8 feet of concrete would be needed to overcome uplift forces from the groundwater. Diesel-powered standby power generator(s) would be used to power the dewatering pumps during power outages.

It is anticipated that temporary power would be brought on site for use during construction. Diesel powered trucks and generators also are likely to be used during construction.

During construction, concrete would be transported to the job site from a concrete batch plant. The number of deliveries is estimated to be approximately 3,900 for each intake pumping plants based on 9-cubic-yard concrete trucks. A temporary batching plant at the job sites to reduce the truck traffic on public roadways may be used.

The construction period for intake pumping plants is expected to be approximately three years.

7.1.5.6 Solids Drywell and Pumps

Solids pumps would be located at the end of the sedimentation basins. The structure would have a footprint of approximately 40 feet by 80 feet.

7.1.5.7 Solids Lagoons

Solids lagoons would be 10 to 15 feet below the flood protection level and would be formed when earthwork is undertaken for the engineered embankment for the intake pumping plant.

7.1.5.8 Substation

The substation would be located above ground at the flood protection level, thus excavation is not anticipated.

7.1.5.9 New Levee and Road

A new levee with a road would be constructed around each intake pumping plant. The levee would be constructed prior to excavation for the pumping plant and sedimentation basins. The road would be 24-feet-wide to allow access of large equipment. Excavated alluvial mineral soils may be used, though additional material may have to be imported on site. Paving or other surfacing may be required. A summary of the two possible levees is provided in Tables 7-12 and 7-13.

Table 7-12: Intake Pumping Plant Levee Construction – Setback Levee

	Total per Intake Pumping Plant	Total for Two Intake Pumping Plants
Length (feet)	2,380	
Width (feet)	40	
Height (feet)	21	
Volume of Material Needed (cubic yards)[1]	74,000	148,000
Material to be Imported (cubic yards)[1]	16,800	33,600

[1] Compacted in place

Table 7-13: Intake Pumping Plant Levee Construction – Without Setback Levee

	Total per Intake Pumping Plant	Total for Two Intake Pumping Plants
Length (feet)	1,980	
Width (feet)	40	
Height (feet)	21	
Volume of Material Needed (cubic yards)[1]	61,600	123,200
Material to be Imported (cubic yards)[1]	25,300	50,600

[1] Compacted in place

7.1.6 Maintenance Considerations

Maintenance would be required for the intake pumping plants, sedimentation basins, and solids lagoons. The anticipated maintenance activities would include scheduled routine maintenance and emergency maintenance. It is anticipated that one of the intake or intermediate pumping plants would be selected to provide the site with centralized maintenance service.

7.1.6.1 Sedimentation Basins and Solids Lagoons

All equipment in the sedimentation basins and solids lagoons would be serviced based on a schedule recommended by the manufacturers. The equipment includes solids collection equipment, pumps, piping, valves, sluice gates and actuators. Periodic mussel cleaning from the sedimentation basins, and removing solids from solids lagoon for off-site disposal would also be required.

7.1.6.2 Intake Pumping Plant

All equipment in the intake pumping plant would be serviced based on a schedule recommended by the manufacturers or developed by the Operator. The equipment would include:

- Mechanical process equipment: Intake pumps, dewatering pumps, motors, valves, hydraulic system, cooling water system, bridge crane, gantry crane, and elevators.

- Building mechanical equipment: HVAC system.
- Electrical equipment: Uninterruptible power supply, VFDs, switchgears, MCCs, transformers, and substations.
- Control equipment: PLC, SCADA, flow meters, and switches.
- Communication systems: Microwave system and fiber optics.

Typical maintenance would include checking the oil level or replacing oil, greasing bearings, checking for abnormal level of noise and vibration or over temperature, replacing wear rings or impellers, replacing motors, replacing light bulbs, etc. Other non-equipment related items would include painting, housekeeping, and cleaning.

7.1.6.3 Site Maintenance

Routine site maintenance would include landscape maintenance, trash collection, and outdoor lighting repair or replacement.

7.2 Intermediate Pumping Plants

7.2.1 Description and Site Plan

Conceptual engineering indicates that two intermediate pumping plants could be required as part of the TDF Option: one within Reach T1 for the San Joaquin Tunnel Option and one within Reach R20. The northern pumping plant would divert water into a tunnel under the San Joaquin River. The other pumping plant at the end of Victoria Canal would provide the hydraulic head for the siphons under Old River and West Canal into CCF. The intermediate pumping plants overcome the head loss (energy loss) due to friction of flow restrictions from siphons and tunnel.

The intermediate pumping plants would be located near the San Joaquin River (optional SJRTPP) and existing CCF (VCPP). A summary of the intermediate plants is provided in Table 7-14 and are shown on Figures 7-4, 7-5, 7-6, and 7-7.

Table 7-14: Intermediate Pumping Plants

Alignment	Intermediate Pumping Plants	Total Pumping Capacity (cfs)	Number of Pumping Plants	Location	Number of Pumps and Capacity for each Pump (cfs)
TDF	Optional San Joaquin River Tunnel and Fish Salvage Facility	15,000	1	San Joaquin River	15 Duty at 1,000 cfs 1 Spare at 1,000 cfs
	Victoria Canal Fish Salvage Facility and Siphon	15,000	1	Near Clifton Court	15 Duty at 1,000 cfs 1 Spare at 1,000 cfs

cfs = cubic feet per second
TDF = Through-Delta Facility

Upstream of each intermediate pumping plant, the diversion facilities at the San Joaquin River and VCPP would include an intake channel, self-cleaning trash rack structure, a series of operable (isolation) gates, a sedimentation basin with solids collection and handling facilities, five parallel fish screen structures and fish salvage facilities. The fish salvage facilities, diversion structures and associated solids handling equipment upstream of the pumping plant are discussed in Section 20.0. This section of the report covers the pumping plant and associated equipment.

Maximum flow at each pumping plant would normally be passed by 15 of the pumps in a plant running in parallel, with an additional 16th pump installed for standby. All of the pumps would be fitted with VFD. The optional San Joaquin River Tunnel and VCPP intermediate pumping plants have the same footprint and number of pumps, differing only in their discharge methodology.

The pumps at the San Joaquin River Facility would discharge into deep tunnels, as discussed in Section 11.0. At the VCPP, flow would be discharged into a siphon. Refer to Section 10.0 for details of the siphon and siphon control structure.

A substation and transformer would be located on the site to supply power.

In order to protect the site and ancillary structures from flooding, the pumping plant and associated equipment would be situated on an engineered embankment, with the top of the structures above the flood protection elevation. The general ground level at the intermediate pumping plant would be raised to an EL of 25.5 feet NAVD88 for the San Joaquin River Tunnel and Victoria Canal Pumping Plants.

7.2.2 Pumping Plant General Arrangement

The preliminary concept for the intermediate pumping plant arrangement is similar to the concept presented for the intake pumping plants. The pumping plant structure would be constructed of reinforced concrete and would have multiple floors to house mechanical and electrical equipment. The lower level would include the rectangular pump suction bays, dewatering sump, pump columns, and dewatering pipes. The intermediate level would include discharge piping and valves, access to the pump and motor shaft, HPUs, and other miscellaneous mechanical systems. The upper floor (operating level) would house mechanical and electrical equipment, including but not limited to pump motors, pump drives, electrical switchgear and MCCs, fire protection, and HVAC equipment. The upper floor would be located above the flood protection elevation. An elevator and stairway would be provided for each of the pumping plants to allow easy access to the various floors. The roof would be at an elevation suitable to accommodate a bridge crane for hoisting and transporting the pump components and mechanical and electrical equipment.

Preliminary sizing of the pumping plants has been determined using vertical column pumps with a vertical motor arrangement. The Hydraulic Institute Standards for pump intake design (Hydraulic Institute, 1998) identifies minimum pump bay widths and minimum submergence depth, which were considered in the pumping plant layout.

The general subsurface conditions for the areas planned for the intermediate pumping plant may be characterized by a soil profile comprised primarily of, in the descending order, a thin layer of loose sand or silty clay of the alluvial flood plain deposits, organic clayey silt and silty clay of the brackish marshes, followed by fibrous peat formed in the freshwater tidal marshes overlying the stiff silty clay and dense to very dense silty sand of the upper Pleistocene age. The organic clayey silt/silty clay and peat soil were generally encountered had a thickness of few feet to approximately 10 to 12 feet (DWR, 1965).

The primary structural support systems used for the pumping plants would consist of reinforced concrete slabs and walls at and below grade, with steel framing and exterior metal wall and roof panels for the above-grade building. The upper floor level, located at grade level, would be reinforced concrete floor slab that would support the vertically mounted pumps and motors. It would be enclosed by a steel framed building that includes a traveling 75-ton bridge crane. The lower level would be a concrete mat slab wet well that would include reinforced concrete partition walls located at each pump to separate and confine the water flow at each pump suction inlet. Deep foundation piles are anticipated to be necessary to support the heavy dead and operating loads of the building. Based on a preliminary pile foundation evaluation, using a 24-inch concrete-filled pipe pile, an estimated pile length in the order of 60 to 65 feet below the founding level of the intermediate pumping plant would be required.

The assumed live loads would include equipment loads such as the crane, pumps and valves. The assumed values would be verified and revised as appropriate to match the most current information as the engineering is developed. When available, the geotechnical report would be used to confirm assumptions and refine the engineering with regards to specific site soil issues.

The San Joaquin River Tunnel Fish Facility and Victoria Canal Fish Salvage Facility would also have sedimentation channels, as part of the diversion structure arrangements. The maximum throughput at these locations would be 15,000 cfs each, and the channels would be longer than the sedimentation channels at the river intake pumping plants. In addition, the channels would only be protected by coarse screens, rather than the fish exclusion screens that would be present at the river intakes. The nature and volume of the solids anticipated to settle in the diversion channels of the fish salvage facilities would, therefore, be different to those at the river intake pumping plants. This would require different equipment for collecting and pumping the solids.

7.2.2.1 Influent Channel and Discharge Piping

The fish salvage facilities and diversion structures upstream of the pumping plant are discussed in Section 20.0.

Flow into the pumping plants from the upstream intake diversion structure would be directed to each pump intake through pump bay openings with isolation gates. These would allow the pump wells to be dewatered for maintenance. Trash racks would be used upstream of the pumps for pump protection.

Downstream of the optional San Joaquin River Tunnel, flow would be discharged into two 33-foot-diameter tunnels. Flow would be discharged directly into a manifold attached to each one of the three tunnels. Each manifold may be connected to a surge tower. A maximum positive pressure head is required to maintain the flow through the tunnel. By connecting through a closed manifold, this pressure head can be maintained. It is proposed that five to six pump discharge pipes would be combined on each manifold. During periods of high pumping, all pumps would be in operation and pump discharge would be directed into all three tunnels. The maximum velocity through the main discharge pipes and tunnels would be approximately 8.7 fps. During low pumping periods (i.e., when Banks Pumping Plant is not in operation), three intermediate pumps would be operated to meet the needs of Jones Pumping Plant. Under this condition, flow would be directed into one or two discharge pipes and tunnels. The minimum velocity through two pipes and tunnels would be approximately 2.6 fps. For tunnel details, refer to Section 11.0.

7.2.2.2 Intermediate Pumping Plants

The intermediate pumping plants would be operated to sustain water levels required for optimal pump operations at both Banks and Jones Pumping Plants. Each pumping plant would have 15 1,000-cfs duty pumps and one 1,000-cfs spare pumps, all with VFD.

Pumps would be installed in parallel and in vertical positions. Pump discharge piping would include a hydraulically operated butterfly valve, a redundant motor operated butterfly valve and a flow meter. The hydraulically operated butterfly valve would provide a check function; therefore, no separate check valves would be provided.

Similar to the intake pumping plants, vertical column mixed flow and vertical volute mixed flow pumps have been identified as the most likely type of pumps to be used for the large flow and medium head application required at the intermediate pumping plants.

All of the pumps would be located in separate pump bays with stop logs at the suction inlets, such that each individual pump can be isolated in its own bay for inspection and cleaning. Flow-through screens would also be provided at each pump bay entrance to screen any large debris.

The pump bays, valve vaults and channels would be dewatered using a dewatering pumping station. The dewatering pumps would discharge back into the canal upstream of the isolation gates. The pumping plant would be oriented perpendicular to the influent canal and downstream canal. Additional information regarding pump selection is in Appendix D.

7.2.2.3 Substation

A substation would be included at the intermediate pumping plant site at the San Joaquin River Tunnel; and two new substation facilities at Clifton Court would be utilized to provide power to the Victoria Canal and Byron Tract plants at the flood protection elevation.

7.2.3 Intermediate Pumping Plant Mechanical Systems

Operational and storage room requirements would be similar to the intake pumping plants. Mechanical building systems' requirements would also be similar for both the intake and intermediate pumping plant and would consist of HVAC, plumbing, and fire protection. Pump discharge valves and actuators, HPUs and motor cooling water systems would also be similar to those for the intake pumping plants. Refer to Section 7.1.3 for details.

7.2.4 Pumping Plant Electrical Systems

Incoming power to the optional SJRTPP would consist of four, 69-kV feeders connected to four 69-kV/4,160-volt outdoor, oil-filled transformers. Four 4,160-volt feeders would be connected to two indoor lineups of switchgears arranged in a main-tie-main configuration. The four incoming feeders would be installed in an underground ductbank system to supply approximately 40-MVA power for the pumping plant. The switchgear breakers would feed the electrical drives and the station auxiliary transformers.

Incoming power to the VCPP pumping would consist of two 69-kV feeders connected to two 69-kV/4,160-volt outdoor, oil-filled transformers. Two 4,160-volt feeders would be connected to an indoor lineup of switchgear arranged in a main-tie-main configuration. The two incoming feeders would be installed in an underground ductbank system to supply approximately 22-MVA power for the pumping plant. The switchgear breakers would feed the VFDs and the station auxiliary transformers.

Two outdoor, oil-filled, station auxiliary transformers at each plant would step the voltage down to the 480-volt level to serve motors and other miscellaneous equipment. For additional information on power distribution planning and lighting, see Sections 7.1.4.1 and 7.1.4.2, respectively.

7.2.4.1 Control Modes and Control Basis

In general, all equipment at the pumping plant would be operated in one or more of the control modes described in Section 18.0.

Table 7-15: Optional San Joaquin River Tunnel and Victoria Canal Fish Salvage Facility Pumping Plants

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel MCC, or Switchgear	Remote PLC/SCADA Control
Pump P01 (typical)	Pump (constant speed)	Emergency Stop	Start – stop, local-remote	Start – stop, local-remote
.	Open closed hydraulic actuated valves on each main pump P01 discharge typical	Local – Remote switch at valve, open, off, close switch at valve	.	Open-Off-Close, Manual, Auto In Auto valve closes prior to pump start. Valve opens either preset time after pump start is initiated or at pre-set discharge pressure on pump discharge line.

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel MCC, or Switchgear	Remote PLC/SCADA Control
.	Open closed motor actuated valves on each main pump P01 discharge typical. Valve is normally open and remains open. This valve is used to backup hydraulic valve	Local – Remote switch at valve, open, off, close switch at valve	.	Open Stop Closed Manual, Valve normally open Manually initiated operation no Auto mode

MCC = motor control center
PLC = programmable logic control
SCADA = supervisory control and data acquisition

7.2.5 Construction Methodology

Intermediate pumping plants would be located at the San Joaquin River (optional SJRTPP) and at Clifton Court (VCPP), respectively.

Heavy construction equipment such as diesel powered dozers, excavators, rollers, dump trucks, fuel trucks and water trucks would be used during excavation, grading construction of access/haul roads and levee. It is anticipated that during earthmoving activities, dust or mud would be generated. Water trucks, street sweepers and other BMPs may be used to minimize generation of dust and mud.

The optional pumping plant at the San Joaquin River discharge piping would connect to a 33-foot-diameter common manifold (field assembly required), which would include three 18-foot-diameter pipes connecting to the tunnels. The VCPP discharge piping would discharge into a siphon and radial gate control structure. The area that would be disturbed is summarized in Table 7-16.

Table 7-16: Area Disturbed Intermediate Pumping Plants*

Area Criterion	San Joaquin Location	Victoria Canal Location
Permanent (acres)	21	24
Temporary – Construction (acres)	22	15
Total (acres)	42	39

* Does not include access roads

7.2.5.1 General Considerations

Figures 7-4 and 7-5 illustrate the limits of construction sites for the 15,000-cfs intermediate pumping plants. The construction site area is influenced by the depth of excavation and footprints of the pumping plant and substation. The construction site area also includes space for staging, equipment, materials storage and laydown area, spoils storage, and parking. To meet the needs of construction, an approximate area of 30 to 35 acres would be required for the pumping plant site. Other considerations are the same as for the intake pumping plants (see Section 7.1.5.1).

7.2.5.2 Intermediate Pumping Plant Structures

Based on available geotechnical information, the intermediate pumping plant structures may need to be supported on piles. Either continuous flight auger or precast concrete driven piles would be driven into soils until they reach refusal. Diesel hammers would be used if driven piles are selected. It is anticipated that the bottom of the pumping plant would need to be used if driven piles are selected. It is anticipated that the bottom of the pumping plant would need to be over-excavated and filled with tremie concrete.

An interlocking steel sheeting and shoring system would be installed due to unfavorable soil conditions and the depth of excavation. Vibratory hammers and diesel hammers would be used to drive sheet piles and concrete piles.

Because of a high groundwater table, a well system would be required to lower the groundwater table during construction. It is anticipated that a large amount of groundwater would need to be removed and either treated or disposed of off site. Dewatering would be a continuous operation until tremie concrete developed adequate strength and weight to resist buoyancy force. If necessary, groundwater treatment approaches would be determined at a later engineering phase. Diesel powered standby power generator(s) would be used to power the dewatering pumps during power outages.

It is anticipated that contractors would supply temporary power on site for use during construction for both the intake and intermediate pumping plants. Diesel-powered trucks and generators are likely to be used during construction.

During construction, concrete would be trucked to the job site from a concrete batch plant. The number of deliveries is estimated to be approximately 6,900 for each of the intermediate pumping plants based on 9-cubic-yard concrete trucks. Due to the large amount of concrete required for construction, the concrete supplier may set up a temporary batching plant at the job site to reduce truck traffic on public roadways.

It is also anticipated that existing near-surface soils are unsuitable for reuse and would need to be disposed of off site. Suitable materials would be brought in to backfill around the structures, pipes, and electrical ductbank. The volume of unsuitable soils that would need to be removed and disposed off site along with imported backfill material has been estimated and included in Table 7-17. The total number of truck trips during the course of earthwork is anticipated to be substantial. The total number of truck trips for construction of the intermediate pumping plants would be approximately 170,231.

Table 7-17: Summary – Earthwork for Intermediate Pumping Plants

Area Criterion	San Joaquin Location	Victoria Canal Location
Total excavation (cubic yards)	480,000	730,000
Excess soils to be exported (cubic yards)	680,000	1,020,000
Imported backfill material	—	—

The construction schedule is heavily influenced by the delivery schedule of major pumping and electrical equipment. The lead time of the first pump is approximately two years from shop drawing approval date, and the delivery of the rest of pumps would be one pump per month thereafter. Additional time should be included for factory testing, delivery, installation, wiring, startup and field testing. The overall completion schedule would exceed four years.

7.2.5.3 Forebay and Discharge Pipe

The intermediate pumping plants would receive water from the upstream fish intake structure and sedimentation basins, and common transition channel. These areas are discussed in Section 6.0.

The optional SJRTPP would discharge water into three tunnel shafts.

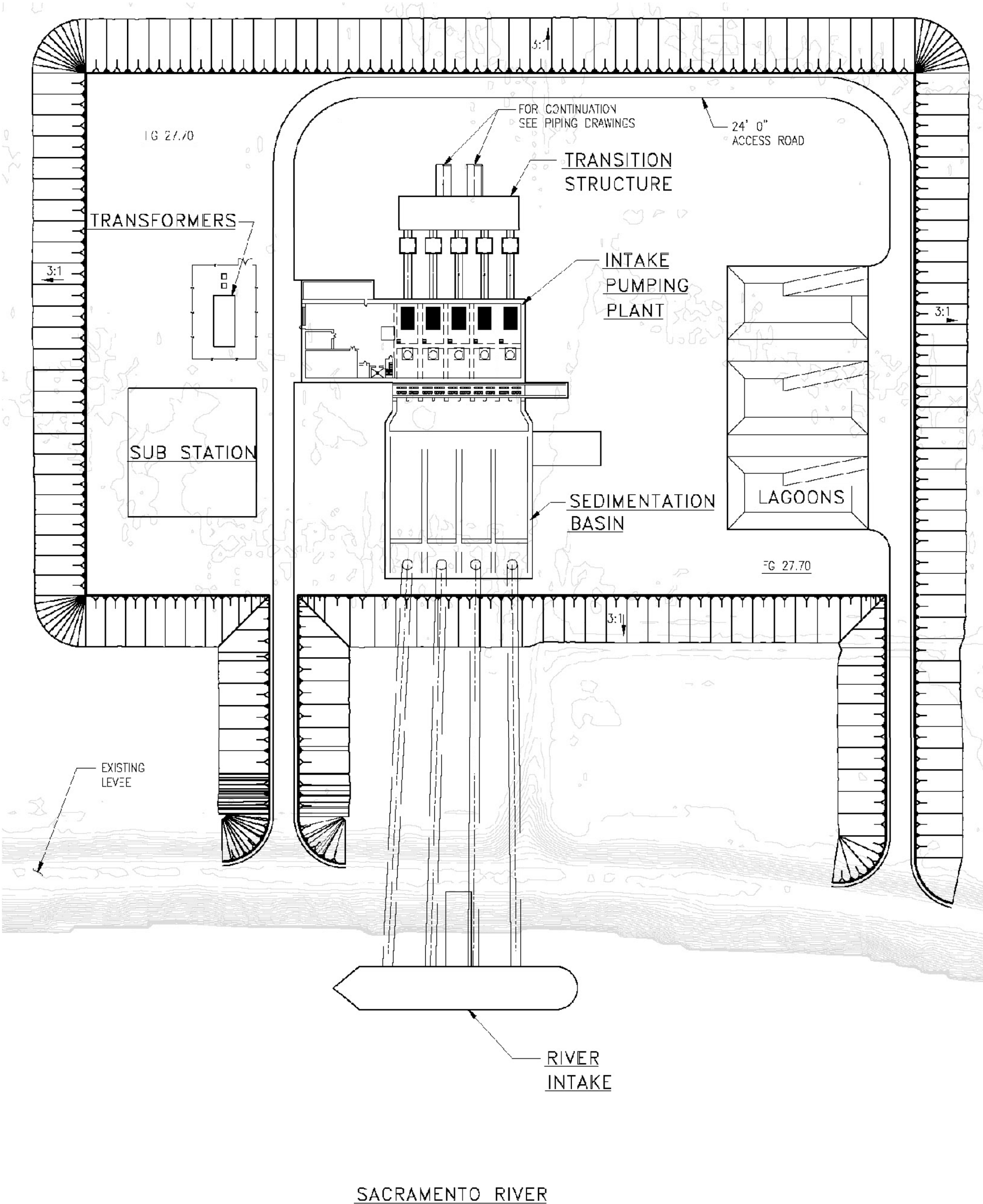
The VCPP would discharge water into a siphon and radial gate control structure.

7.2.5.4 Substation and Grounding Construction

As with the substation at the intake pumping plants, the substation at the intermediate pumping plant would be at the flood protection elevation. Significant earthwork would not be necessary to make grade because the terrain is fairly flat. Unlike the intake pumping plant substation, the intermediate pumping plant substation would be fairly isolated from other on-site work. Substation construction can be performed in parallel with pumping plant construction.

7.2.6 Maintenance Considerations

Maintenance considerations would be similar to those for the intake pumping plants. However, because the intermediate pumping plants have no sedimentation basin, no maintenance associated with the solids handling equipment and disposal would be needed. Refer to Section 7.1.6.



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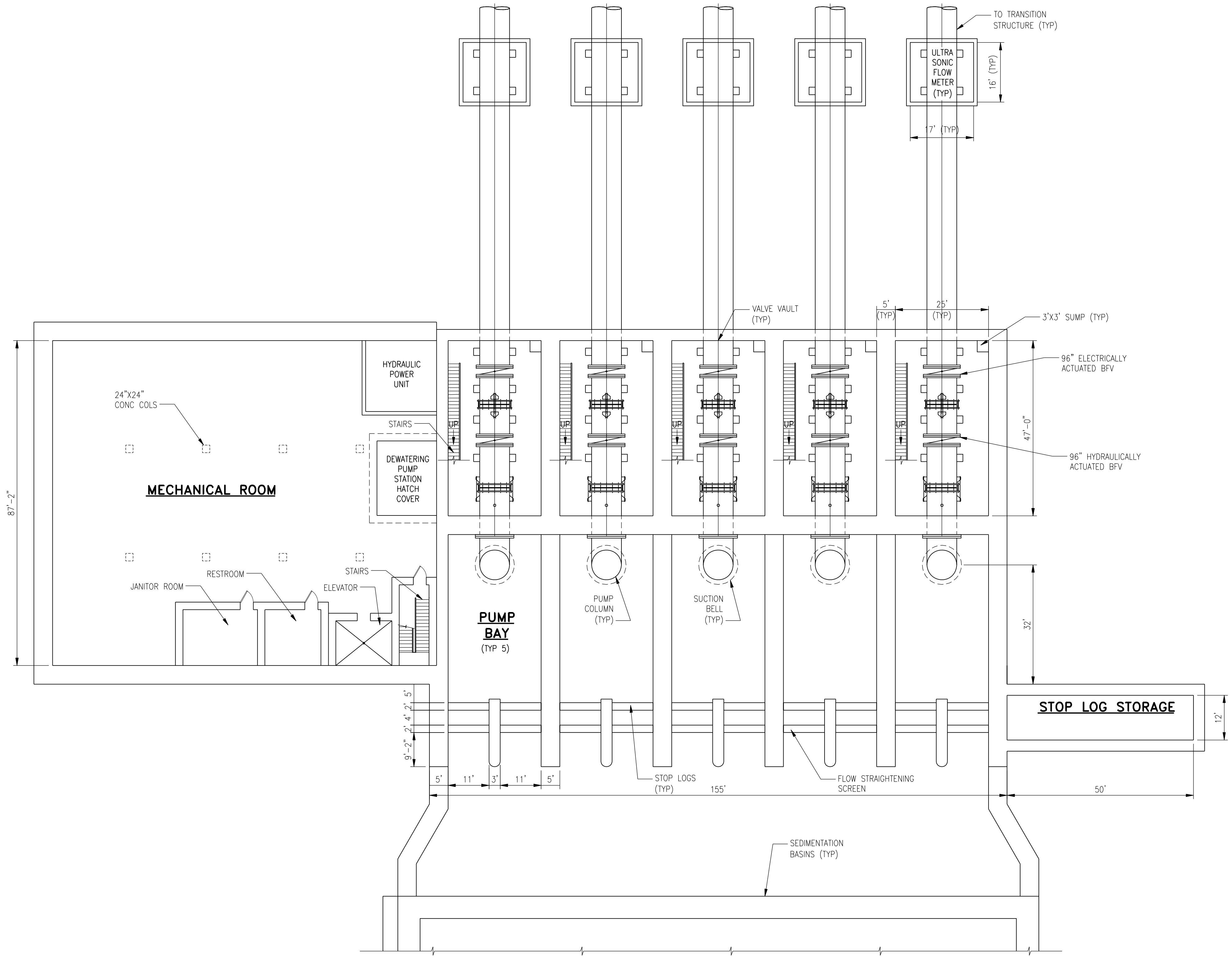
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Intake Pumping Plant Site Plan

FIGURE
7-1



INTERMEDIATE LEVEL PLAN

1/16" = 1'-0"

0 16'-0" 32'-0"

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Pumping Plant
Intake Pumping Plant - Intermediate Level Plan

FIGURE **7-2**

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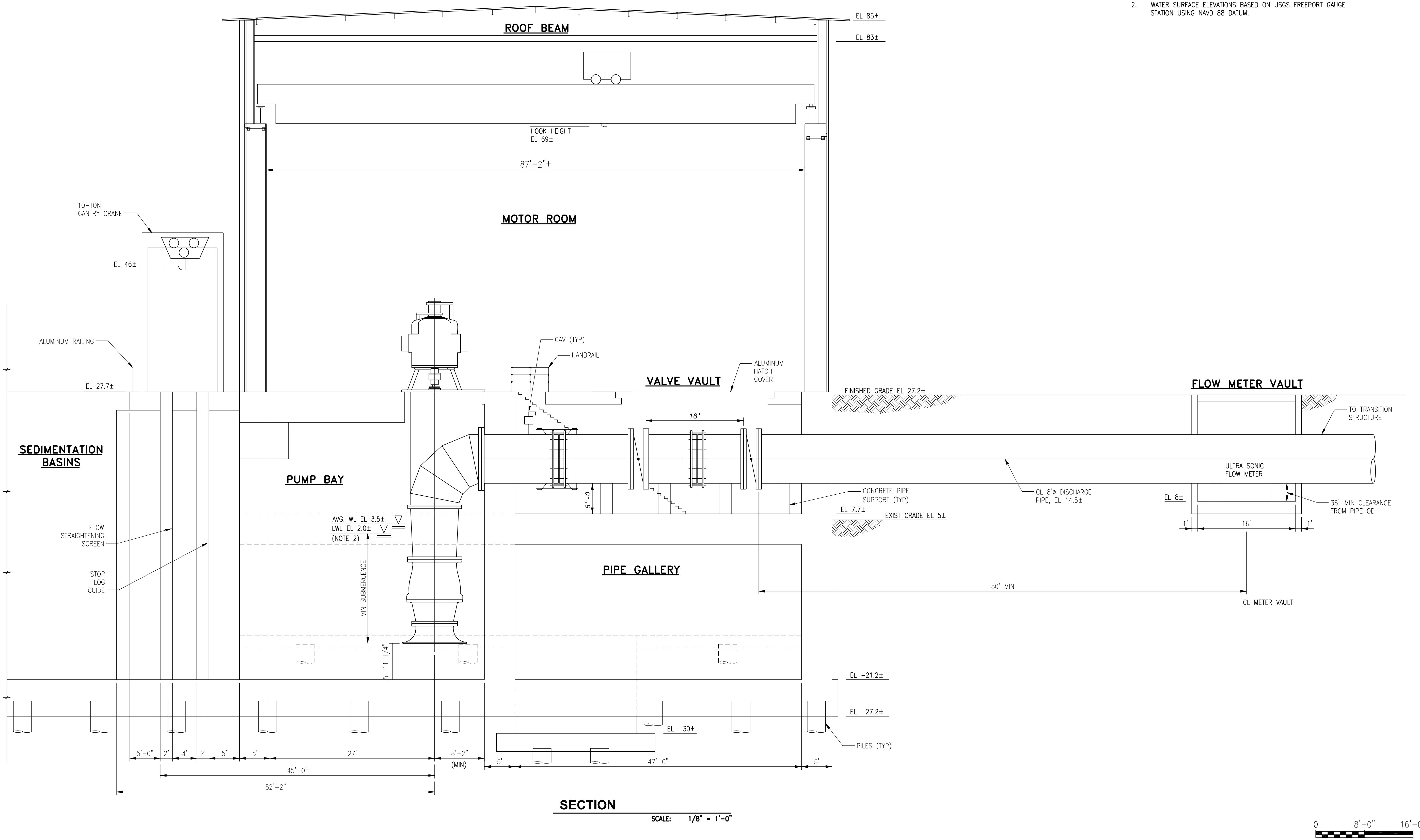
NOTES:

1. ELEVATIONS SHOWN ARE FOR INTAKE PUMPING PLANT NO. 1.
2. WATER SURFACE ELEVATIONS BASED ON USGS FREEPORT GAUGE STATION USING NAVD 88 DATUM.



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Pumping Plant
Intake Pumping Plant - Transverse Section

FIGURE
7-3

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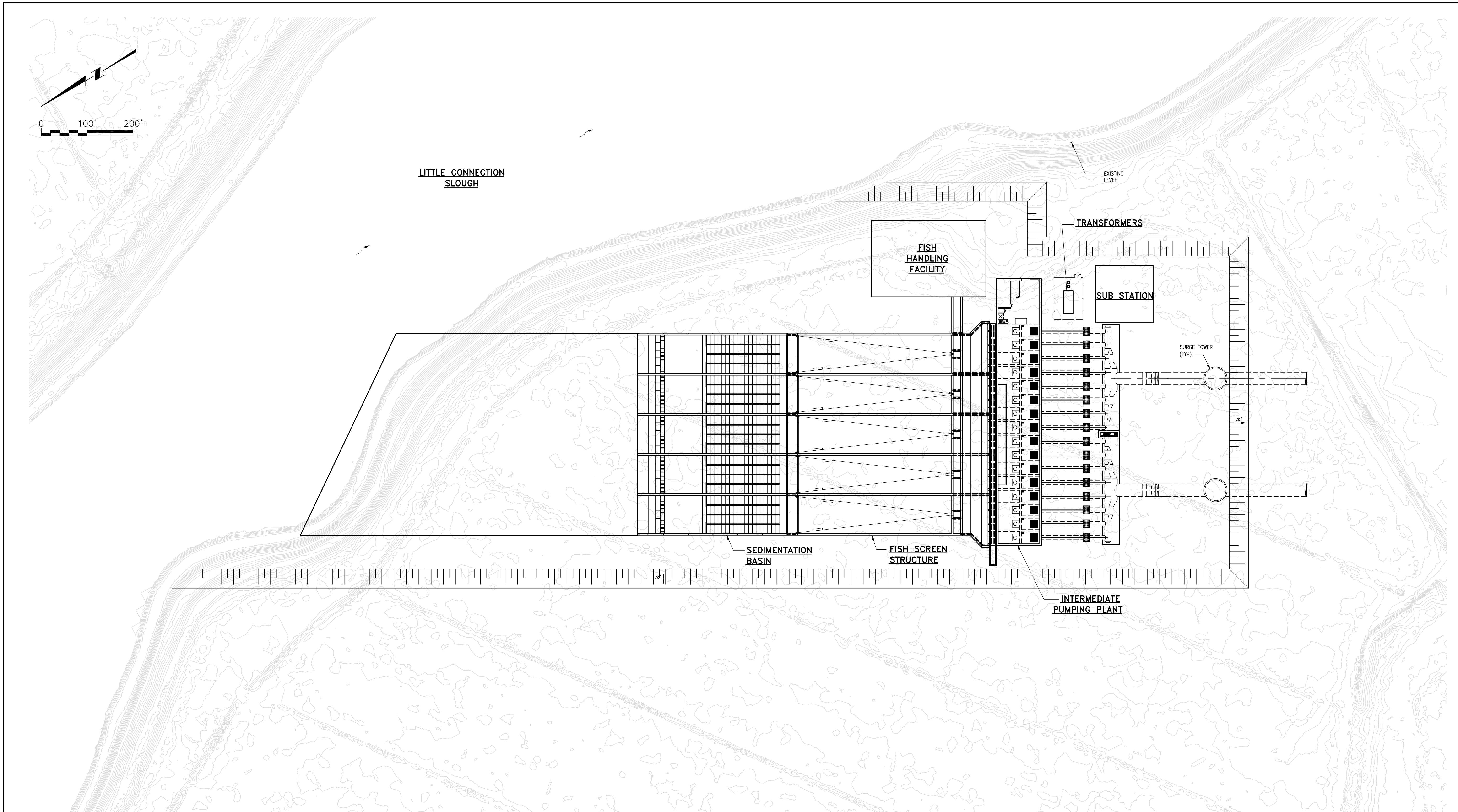

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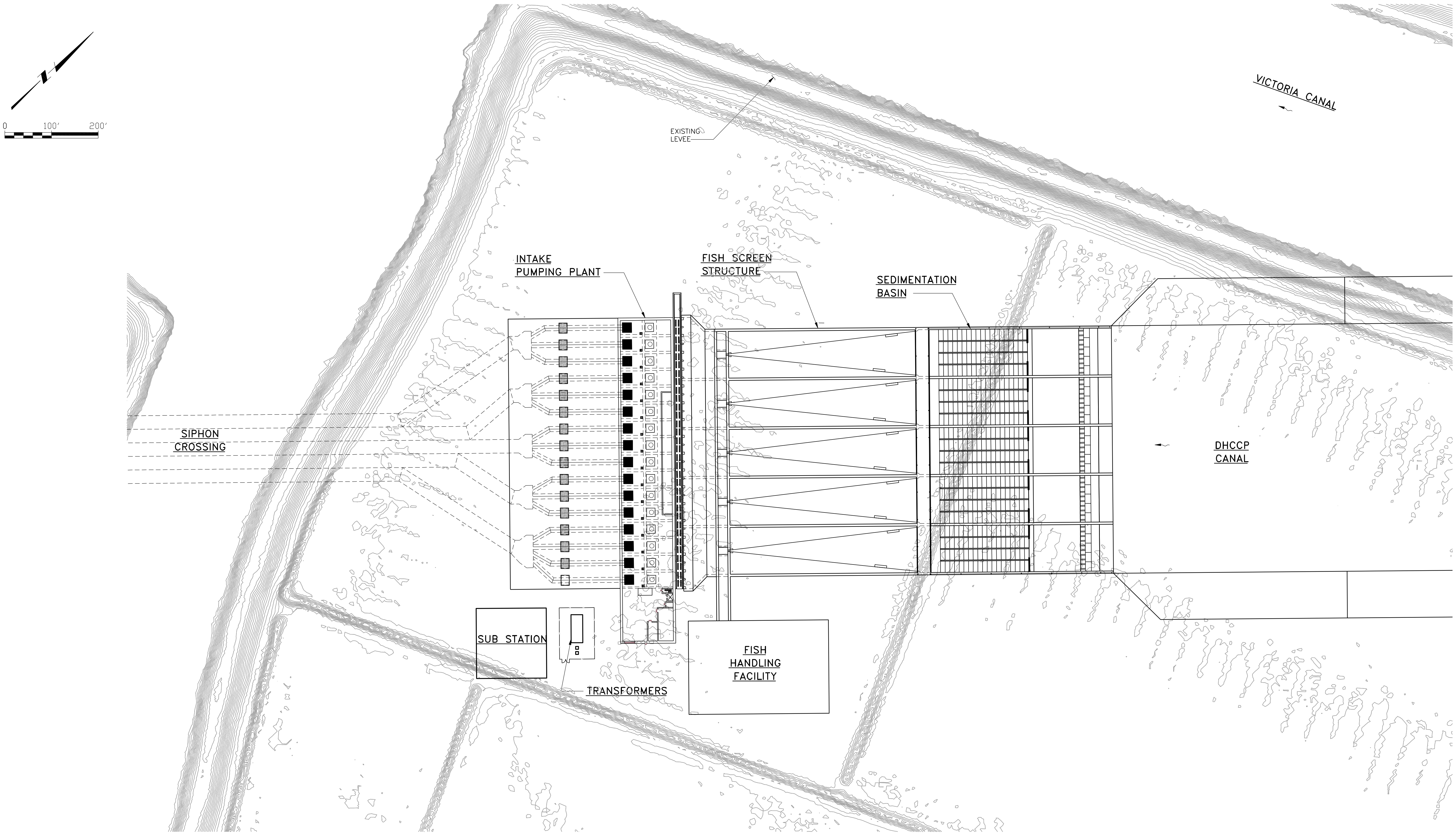
THROUGH-DELTA FACILITY OPTION

Pumping Plant Optional San Joaquin River Tunnel Pumping Plant - Site Plan

FIGURE
7-4



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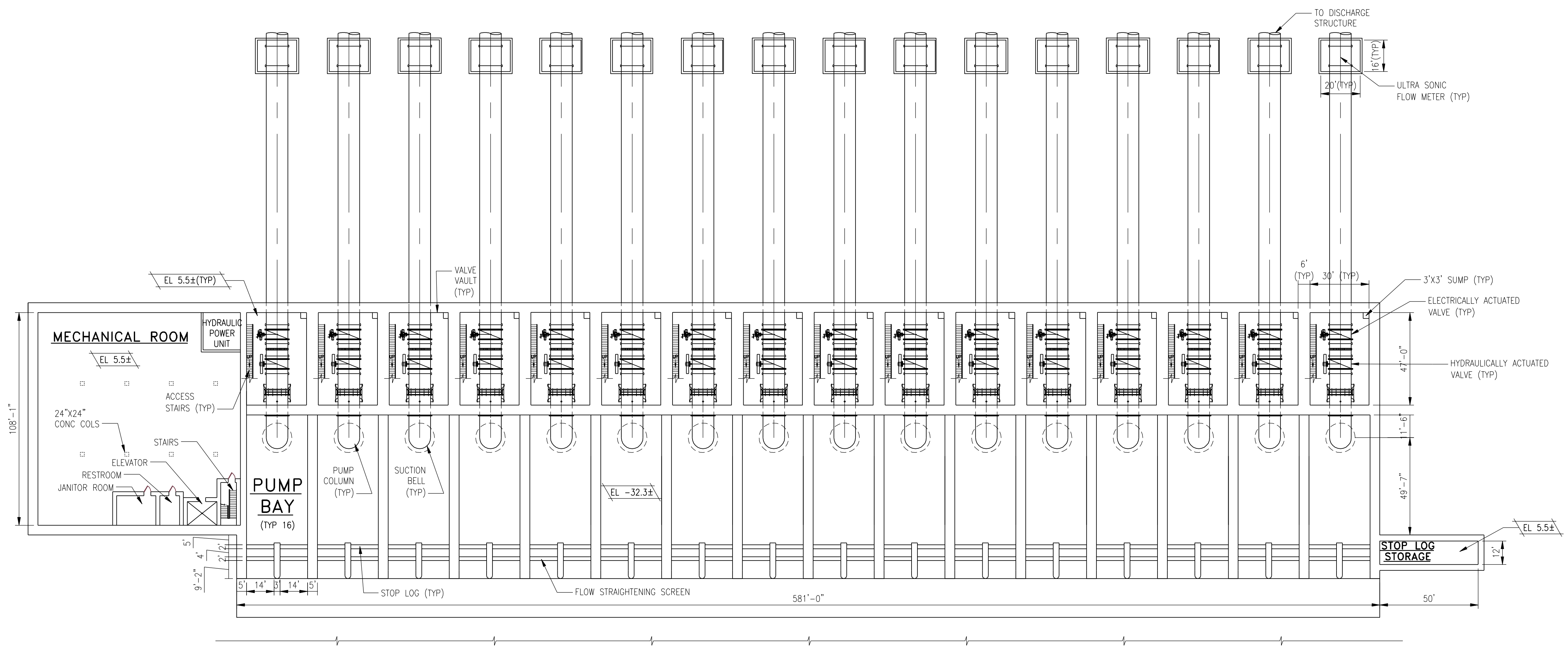
Pumping Plant Victoria Canal Fish Salvage Facility Pumping Plant - Site Plan

FIGURE
7-5



INTERMEDIATE LEVEL PLAN

1/32" = 1'-0"

0 16'-0" 32'-0"

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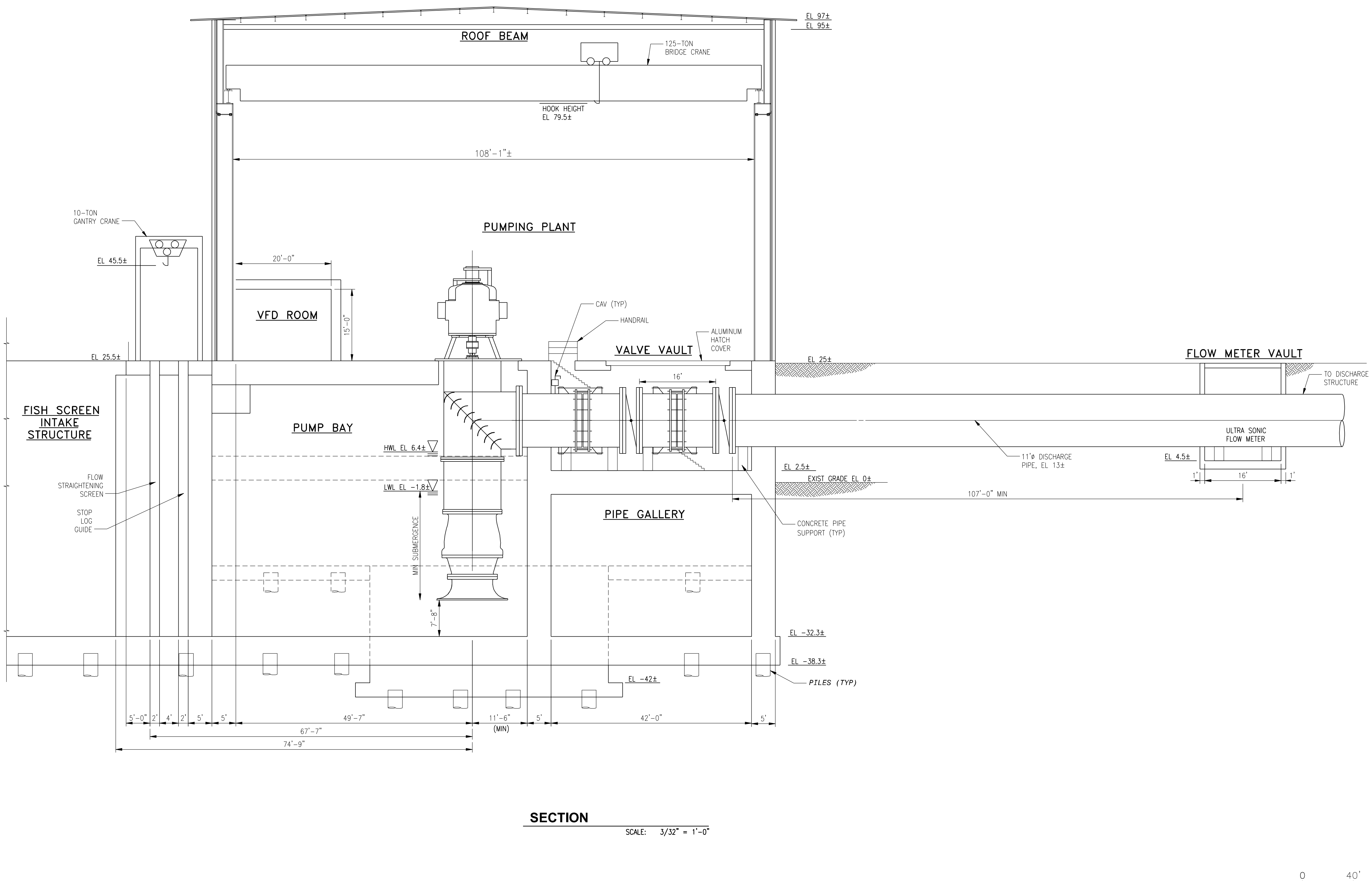
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Typical Intermediate Pumping Plant
Intermediate Level Plan

FIGURE
7-6



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THROUGH-DELTA FACILITY OPTION

Typical Intermediate Pumping Plant Transverse Section

FIGURE
7-7

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8.0 PIPELINE CONVEYANCE SYSTEM

The preliminary concept for the TDF conveyance option relies predominantly on a canal to convey water southward from the Sacramento River to the SWP and CVP export pumping facilities; however, pipelines would be used to convey water from the intake pumping plants to the canal.

8.1 Description of Facilities

The TDF Option begins with two intake facilities on the Sacramento River, each with a capacity of 2,000 cfs. Sacramento River water is drawn into the intake facility through fish screens. Each intake facility has two 8-foot-diameter pipelines that feed water to four sedimentation basins where solids are removed. The settled effluent from the sedimentation basins discharges to a common channel that serves as the intake for the pumping plant. Five pumping units (four duty plus one stand-by), each with a capacity of 500 cfs, deliver settled flow through a dedicated 96-inch pipeline to the pumping plant transition structure.

The pipeline conveyance system begins at the pumping plant transition structure and delivers water to the canal transition structure and into the canal system. The pipeline conveyance system must have the same hydraulic capacity as the intake facilities.

8.1.1 Intake Locations and Pipeline Alignments

Figure 6-2 shows potential river intake locations, as well as conceptual alignments for the conveyance pipelines. The intake, sedimentation, and pumping plant facilities are approximately 850 feet in plan length. Final plan length would vary at each intake location based on site-specific conditions. The lengths for the conveyance pipelines, beginning from the northernmost intake through the southernmost intake, are as shown in Table 8-1. All lengths are determined from the pipeline inlet at the pumping plant transition structure to the discharge at the primary canal transition structure. A typical intake facility pipeline profile is shown on Figure 8-1.

Table 8-1: Length of Pipeline Conveyance System

Intake Number	Intake Location	Approximate Conduit Distance (Miles)
1	Highway 160, 0.1 miles from Junction with Hood Franklin Road	0.4
2	At Highway 160, 0.8 miles northeast of Lambert Road	1.0

8.1.2 Pipe Material Type Alternatives

Pipeline or conduit alternatives have been investigated for application to the conveyance system for the TDF Option as follows:

- Circular CIP concrete pipe

- Reinforced concrete cylinder pressure pipe (AWWA C300) or reinforced concrete pipe (AWWA C302)
- Steel pipe (AWWA C200)
- Rectangular CIP concrete box
- Arch CIP concrete conduit

Appendix E, Pipeline Materials, summarizes the construction techniques, delivery and installation requirements, and other characteristics associated with each construction material and configuration.

For this report, design parameters and quantities for circular concrete CIP conduit are presented. Evaluation of other pipeline material alternatives is continuing.

8.1.3 Pipe Number and Size Selection

Alternative configurations of number of conduits and conduit size were evaluated. ***The Technical Memorandum – Pipe Size Optimization and Sensitivity Analysis***, attached in Appendix E, summarizes the evaluation and recommended pipe size for one type of pipeline conveyance system (Conduit Type I) based on hydraulic design capacity of 2,000 cfs.

The evaluation identified the optimum number of conduits, and the optimum pipe diameter to be constructed for each conduit type. The analysis included evaluation of hydraulic, right-of-way (ROW), energy consumption, construction impacts, and economic criteria. A sensitivity analysis was performed to evaluate to what extent variations in the selected design criteria would impact conduit size selection.

Table 8-2: Recommended Pipe Size (Circular CIP Concrete Alternative)

Type	Total Flow (cfs)	Number of Conduits	Flow per Conduit (cfs)	Recommended Conduit Diameter (ft)	Velocity (fps)
I	2,000	2	1,000	14	6.5

cfs = cubic feet per second
CIP = cast-in-place
fps = feet per second
ft = feet

The optimum pipe diameter for conduit Type I is 14 feet for the TDF concept.

Trench section dimensions and installation details for all of the pipe alternatives are shown on Figure 8-2.

8.1.4 Pipe Hydraulics and Pressure Criteria

The profile of each pipeline with the corresponding HGL associated with the operation of the pipeline at the design capacity of the intake pumping plant has been established. Table 8-3 summarizes the resulting internal design pressures for each pipeline.

Table 8-3: Pipe Internal Design Pressures

Intake Pipeline	Minimum Design Pressure		Maximum Design Pressure	
	ft	psi	ft	psi
1	38	16	41	18
2	41	18	44	19

ft = feet
psi = pounds per square inch

8.1.5 Pumping Plant Transition Structure

The pumping plant transition structure receives flow from the pumping plant and is the beginning of the pipeline conveyance system as shown on Figure 8-1. The pumping plant transition structure is configured to provide a convenient and hydraulically efficient means for transitioning from the seven discharge pipelines from the pumping plant into the two, larger conveyance pipelines. The structure is configured to facilitate isolation of the pipelines for inspection and maintenance, and to assure that the pumping plant discharge pipelines and flow meters always operate under a submerged condition.

Provisions for conduit isolation are provided with manual stop logs. A gantry crane would be permanently installed at the top of the structure to install and remove the stop logs, as needed. Access hatches are provided at each of the structures for inspection and maintenance. Venting is also provided to protect the pipelines during dewatering.

8.1.6 Canal Transition Structure

The pipeline conveyance from each intake would discharge to the canal through a canal transition structure as shown on Figure 8-3. The canal transition structure is configured to provide an efficient hydraulic transition (with minimal head loss) from the pipeline system to the canal. A dividing wall would be provided between each conduit and would extend to the discharge to the canal. Isolation gates would be installed to facilitate conduit isolation. A pier at the end of each structure is provided to hold the gates. Manual stop log facilities are provided downstream of the gates to facilitate inspection and maintenance of the gates.

A portion of the canal section upstream and downstream of the transition structure would be armored to protect the earthen embankment from erosion due to higher flow velocities.

8.1.7 Pipe Cover Depth and Floatation

Pipe cover depth could take into consideration several factors including (1) farming and agriculture needs, (2) high groundwater and floatation, and (3) location of existing utilities.

Investigations into farming practices indicated a soil disturbance of up to 6 feet under some conditions. Cover depth is also a critical consideration to address pipe floatation. Preliminary data indicates that groundwater is within 1 to 2 feet of existing ground elevation. Appendix F presents the pipeline floatation analysis.

The recommended pipeline depth of cover is 10 feet. Meeting this design criterion may raise flotation issues for several of the conduit types. The following floatation prevention alternatives would be further investigated to identify a preferred configuration:

- Increase conduit thickness
- Provide a concrete slab in between parallel conduits and anchor conduits to the slab
- Increase footing width
- Cap conduits with cement slurry
- Negotiate easements to prohibit disturbances, such as by farming
- Provide concrete collars
- Anchor conduit to piles

The conveyance profiles developed for this CER show that actual cover depth would be greater in many locations due to the minimum design slope. Other factors may also impact pipeline cover including conduit material type (lighter materials have a greater minimum cover requirement), clearance requirements beneath existing utilities, structures, river crossings and other obstructions, future construction developments, or wind erosion. These aspects would be investigated further.

8.1.8 Other Construction Components

Materials of Construction. Alternative conduit materials and configuration of construction are evaluated in Appendix E. Designing for a range of materials and configuration would maximize bidding opportunities and result in best cost per unit length of conduit. The following conduit options are considered feasible:

- In situ cement-mortar lined, coal-tar epoxy coated steel pipe protected with impressed current cathodic protection.
- Field-fabricated reinforced concrete cylinder pressure pipe (RCCP) (AWWA C300) or reinforced concrete pipe (RCP) (AWWA C302).
- CIP concrete options including circular CIP, rectangular box conduit, and arch-shaped conduit.

Pipe Embedment. Pipe embedment requirements depend on the pipeline material configuration and geotechnical conditions. Additional geotechnical explorations would supplement previous studies by DWR to better evaluate likely soil conditions and the depth of groundwater. It is anticipated that native materials are generally of good quality in the area of pipeline construction and excavated material from the pipeline trench would be used as embedment and backfill for the conduits and exported for use as fill elsewhere on the project. Pipeline embedment would be imported where suitable materials are not available. All embedment would be placed and compacted around the pipeline as required for pipeline support and to minimize surface settlement.

Table 8-4 summarizes the volumes of excavation, backfill, pipe bedding, and spoil per mile of conduit length for each type of conduit. These quantities assume that the native materials may be used for trench backfill.

Table 8-4: Summary of Trench Quantities Per Mile

Quantity	Conduit Type I
Diameter, feet	14
Number of parallel conduits	2
Excavation volume, cubic yards	688,800
Backfill volume, cubic yards	592,800
Pipe bedding volume, cubic yards[1]	14,300
Spoil volume, cubic yards[1]	826,600

[1] There are additional embedment and spoil volume requirements for steel pipe

Pipeline Floatation. Preliminary geographical information system (GIS) data show that the groundwater table in the proximity of the conduit alignments is shallow, approximately 1.5 feet below existing grade. The Pipeline Floatation Analysis (Appendix F), concluded that there would be potential for floatation with circular CIP concrete or RCCP at typical design thickness when cover depth was less than four feet. This potential floatation may be true either during maintenance or in the case of an emergency when pipe exposure is required. Protection against pipeline floating would be required and may include permanent dewatering facilities, concrete ballasts and/or integrated with tension piles, or other deep foundation systems.

Roadway Crossing. Roadway crossings would be constructed by open-cut trench construction methods. Local access would be maintained by detours, or temporary pavement. Compliance with local governing agency requirements would be performed for each roadway crossing.

Drainage Crossings. Where installed in or across existing substantial drainage courses, the pipeline would be protected by additional cover where necessary, concrete encasement, or rip-rap at open-cut installations. A scour analysis would be required to determine the limits and depth of each crossing to prevent exposure of the pipelines in the future as a result of channel erosion.

Air Vents. At high points in the pipeline, particularly at pronounced changes in grade, vent shafts would be required. Pipeline grades would be adjusted to minimize the number of high points. Air vents would also be provided at the pumping plant transition structure.

The vent shafts would prevent accumulation of air at high points within the pipeline by exhausting large volumes of air as the pipeline is filled and by releasing pockets of accumulated air while the pipeline is operational and under pressure. Vent shafts would provide vacuum relief by allowing air to enter the pipeline when it is drained.

Pipeline Dewatering Facilities. Pipeline dewatering facilities would be installed as part of construction (1) to provide a dry, stable excavation bottom for placement of bedding, pipe material, and backfill; (2) to dewater the lenses of silts and sands encountered during excavation; and (3) to dewater highly permeable prolific sand layers below the excavation. In addition, due to the high level of the groundwater table, dewatering facilities may also be considered post-construction for inspection, maintenance, or in the case of emergency. Two dewatering schemes are being considered: well point method and the deep well pump method.

Table 8-5 summarizes the characteristics of dewatering being considered at this time. Figure 8-2 shows 10-foot-wide dewatering benches for well point facilities at a depth of 15 feet from the bottom of the trench. The ability of the receiving water bodies to accommodate anticipated discharge rates, volumes and qualities, as well as the impact of dewatering activities on pipeline system down times, would be evaluated. Geotechnical and groundwater quality investigations would be required to assess whether treatment of groundwater prior to discharge is required.

Table 8-5: Summary of Dewatering Alternatives

	Dewatering Alternatives	
Description	**Well Point**	**Deep Well**
Application methodology	Dewater silts and sand above the bottom of planned excavation.	Depressurization of sand underneath the bottom of planned excavation.
Well screen diameter	2 to 4 inches	6 to 8 inches
Well depth	20 feet	75 to 300 feet
Well spacing	3 to 6 feet at top of trench 3 to 6 feet at intermediate bench	50 to 75 feet
Well yield	1 to 10 gpm	30 to 100 gpm

gpm = gallon per minute

Access Openings. Access openings would be provided at primary transition structures including the pumping plant, Sacramento River crossing, and the canal structures. Access openings would be configured to facilitate internal inspections and maintenance of the conveyance system.

<u>Corrosion Protection.</u> Corrosion control would be evaluated, based on the following:

- Pipeline materials of construction
- Design life
- Corrosivity of the environment (i.e., soil resistivity, pH, redox potential, sulfates, sulfides, chlorides, wetting and drying cycles, backfill, soil contamination, possible alternating current induction, bimetallic connections, direct current interference sources, and long-line corrosion cells)
- Consequence of a corrosion related leak or rupture
- Cost of providing the corrosion control method versus the actual benefit derived
- Owner preferences

Corrosion protection measures to be considered include protective linings and coatings, dielectric isolation of dissimilar materials, and cathodic protection systems consisting of either galvanic anodes or impressed current system. This aspect would be further assessed during subsequent engineering analysis.

8.2 Construction Methodology

Construction of the pipeline conveyance system is proposed to be open trench excavations for the majority of the alignment.

8.2.1 Trench Width

Trench widths would vary depending on the depth of cover, and geologic and hydrologic conditions. Preliminary geotechnical and hydrologic conditions have been investigated which indicate that groundwater may be close as 1.5 feet below the existing ground surface. Clear spacing of 18 feet between CIP conduits and 10 feet from conduit to the toe of the trench has been provided to allow for formwork and bracing needed for CIP concrete construction. Other conduit material alternatives would have smaller spacing requirements. While dewatering equipment is expected to be required, quantities have been shown with and without dewatering facilities. Details of the circular CIP trench sections are shown on Figure 8-2. Quantities for other methods of construction are included in Appendix E, Pipeline Materials. Additional ground surface would be required by the construction equipment, materials laydown areas, access, and dewatering equipment.

8.2.2 Construction Easement

Table 8-6 presents the maximum expected dimensions of the construction (temporary) and permanent ROW calculated for each conduit type. The temporary construction easement includes the top trench width with dewatering facilities, spoil area required for excavated soils assuming a 20% bulking factor, and access required for construction equipment. The permanent ROW includes the top trench width with dewatering facilities plus 35 feet of access either side for vehicles and equipment. Trench sections and construction easement requirements for conduit Type I are shown on Figure 8-2.

Table 8-6: Construction Easement Widths for Conduit Type I

Conduit Type	Nominal Size (feet)	Number of Pipelines	Permanent Easement (feet)	Dewatered Installation &Temporary Construction Easement Width (square feet)
I	14	2	280	620

(1) Based on 2 horizontal to 1 vertical ratio (2H:1V) maximum allowable trench side slope and a 4H:1V maximum spoil area side slope
(2) Temporary Construction Easement width is representative of the total construction width required for trench and spoil area.

Where high groundwater is encountered along portions of the alignment, a groundwater collection and disposal system would be installed and operated continuously during the construction period while the trench is open. Groundwater disposal may involve installation of a temporary above-grade pipeline for discharge into an adjacent waterway, irrigation ditch or into the surrounding fields. Treatment of water removed as part of dewatering activities may be required consistent with discharge permit conditions.

8.2.3 Description of Construction Methods and Procedures

Excavation. Except where crossing under a major waterway, intake conveyance pipelines would be installed via open cut. Excavation would include clearing, grubbing, excavation, disposal of excess spoil material and dewatering. In addition:

- All existing vegetation and trees would be cleared and grubbed along the pipeline easement and disposed of off site.
- Temporary construction access roads and haul roads would be constructed.

Open trench areas would have temporary fencing and barricades to prevent entry and entrapment of wildlife and livestock in trench excavations.

For sections of the alignment where the groundwater table is above the trench formation level, a conventional dewatering system would be installed and operated continuously while the trench is open, period to achieve a dry trench. Groundwater disposal may involve temporary above-grade pipelines for discharge into a waterway or onto the ground. If discharge into a waterway is selected, rip-rap erosion protection may be required. All discharges would meet the requirements of the NPDES permit.

Diesel-powered equipment are proposed for excavation. Excavated material initially would be sidecast and stockpiled along the pipeline alignment within the construction easement.

Surplus excavated material not used for backfill of the conduit trench would be hauled away for off-site use or disposal. The pipe bedding material and trench backfill may be controlled low strength slurry mixture, imported granular material, native material or some combination thereof. Topsoil would be set aside during excavation and saved for reapplication when construction is complete. A portion of the spoil area would be set aside as a separate topsoil storage area.

Dust control measures during construction would conform to all federal, state, and local requirements.

Erosion control measures such as silt fencing, straw mats and straw wattles would be placed to capture sediment and reduce erosion.

Dewatering the pipeline trench during construction would be accomplished in accordance with the NPDES permit and the Regional Water Quality Control Board (RWQCB) requirements.

After construction is complete, the alignment would be recontoured as required and seeding of all disturbed areas would occur.

Paved areas disturbed by construction would be repaved.

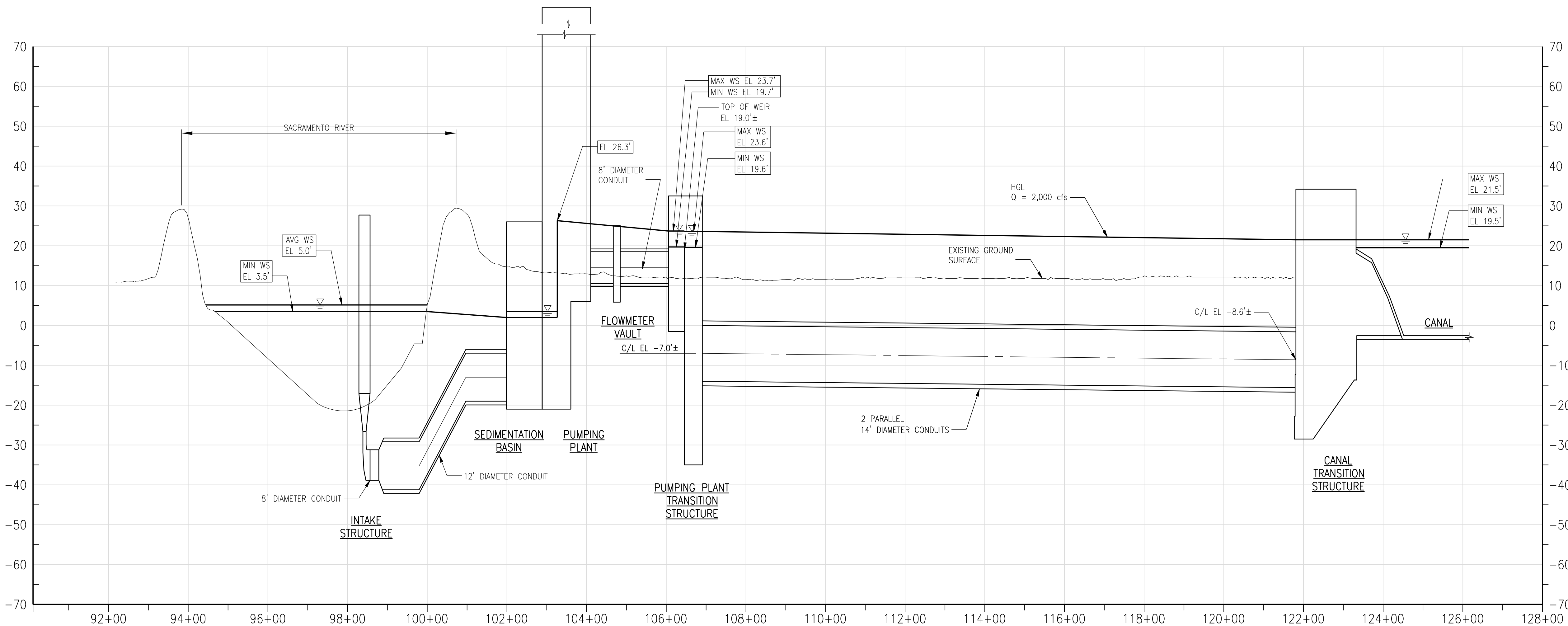
8.3 Maintenance Considerations

Maintenance of the conveyance pipelines is dependent on the materials of construction as summarized in Table 8-7.

Table 8-7: Summary of Maintenance Considerations

Material and Conduit Configuration	Maintenance Considerations
Steel Pipe	Maintenance and operation of an impressed current cathodic protection system. Periodic internal inspections and repair of cement mortar lining.
RCCP or RCP	Periodic internal inspections and repair of cement mortar lining at the joints. Periodic inspections of internal concrete. Repairs to concrete as needed including sealing cracks and repairing spalling to prevent exposure of steel.
CIP	Periodic inspections of internal concrete and joints. Repairs to concrete as needed including sealing cracks and repairing spalling to prevent exposure of steel.
All	Regular periodic operation of radial gates. Repairs as needed. Vent inspection and repairs. Regular inspections along the line for signs of leakage or erosion of soil cover.

CIP = cast-in-place
RCCP = reinforced concrete cylinder pressure pipe
RCP = reinforced concrete pipe



INTAKE NO. 1 PIPELINE

VERT SCALE: 1" = 15'-0"
HORZ SCLAE: 1" = 150'-0"

LEGEND:

AVG	AVERAGE
C/L	CENTERLINE
EL	ELEVATION
MAX	MAXIMUM
MIN	MINIMUM
WS	WATER SURFACE

NOTES:

1. CENTERLINE OF CONDUIT BASED ON A MINIMUM OF 10 FEET OF COVER AT ALL LOCATIONS AND A MINIMUM SLOPE OF 0.001.
2. FOR WATER SURFACE (WS) ELEVATIONS DOWNSTREAM OF WEIR IN TRANSITION STRUCTURE:
   MAX – 2,000 cfs 2 CONDUITS MAX CANAL ELEVATION
   MIN – 500 cfs 2 CONDUITS MIN CANAL ELEVATION
3. VERTICAL AND HORIZONTAL SCALE SHOWN AT 34"x22" SHEET.
4. FOR STRUCTURE ELEVATIONS, REFER TO SECTION DRAWINGS OF STRUCTURES.

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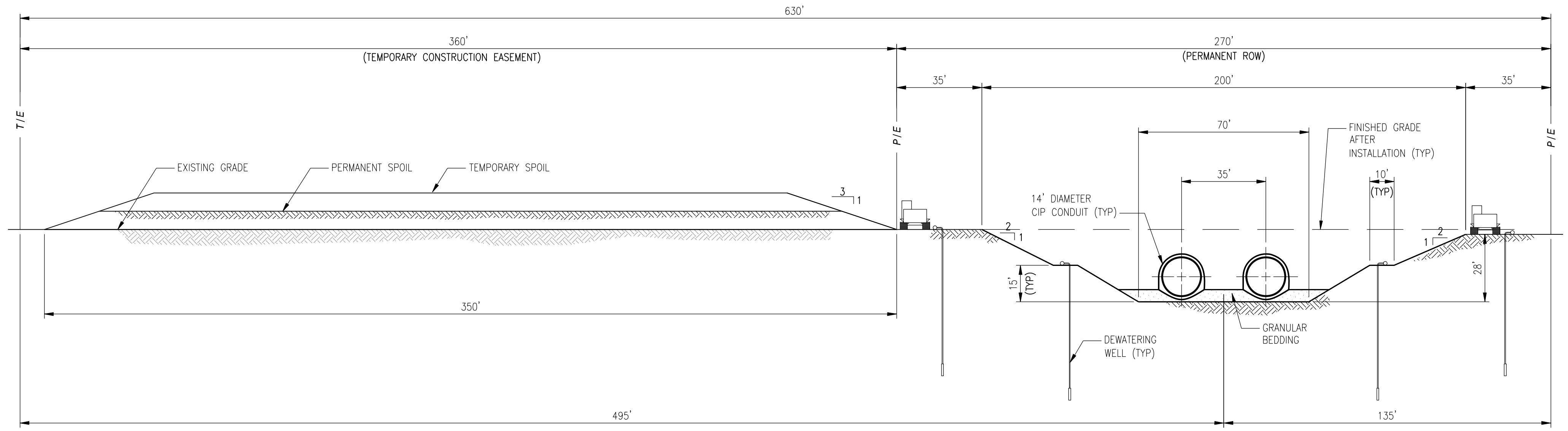
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

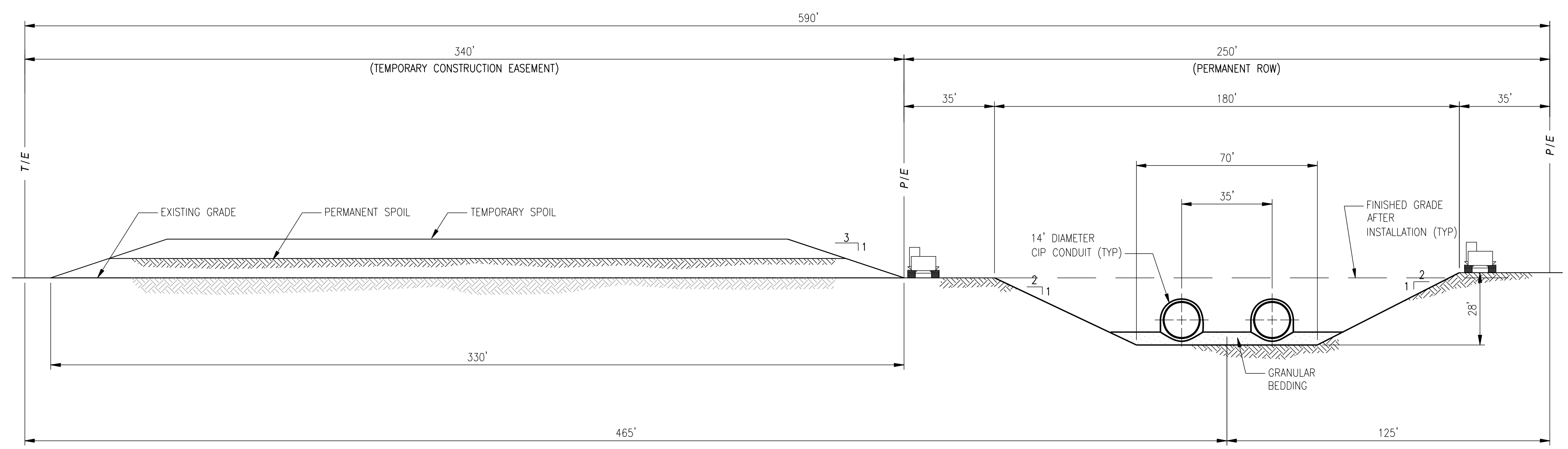
Pipeline
Intake No. 1 - Hydraulic Profile

FIGURE
8-1



CONVEYANCE PIPELINE TRENCH SECTION WITH DEWATERING

1" = 30'-0"



CONVEYANCE PIPELINE TRENCH SECTION W/O DEWATERING

1" = 30'-0"

NOTES:

1. EXCESS SPOIL TO BE UTILIZED IN OTHER AREAS OF THE PROJECT.
2. MINIMUM OF 10 FEET OF COVER FROM CROWN OF PIPE.

0 30' 60'

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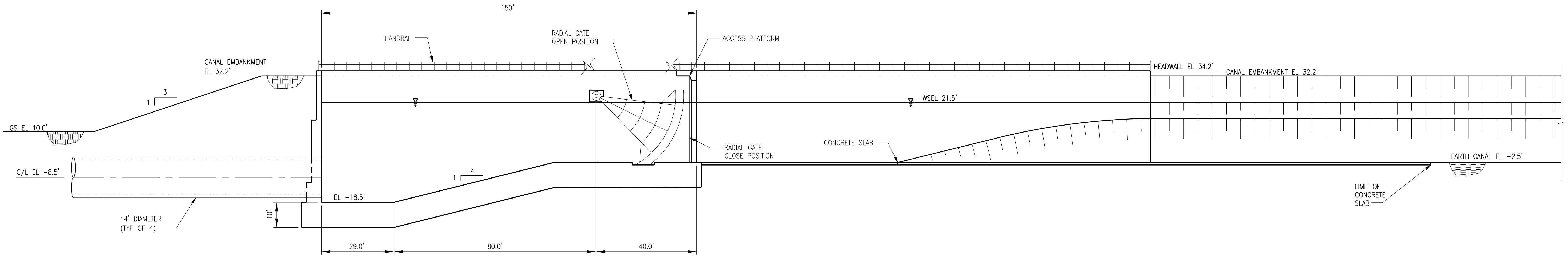
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Pipeline Conduit Type I (2,000 cfs)
Circular CIP Sections

FIGURE

8-2



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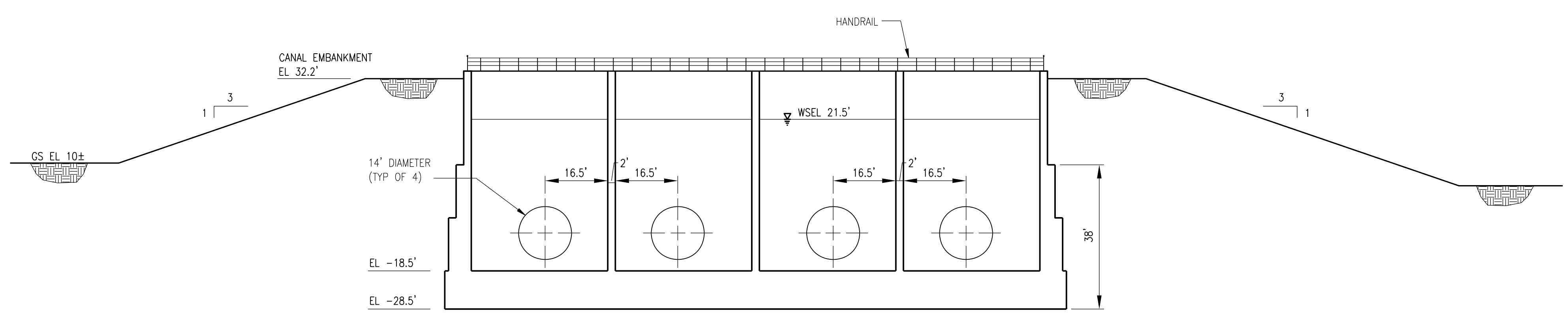

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CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Pipeline Intake No. 1 and 2
Canal Transition Structure - Sections

FIGURE
8-3

9.0 CANALS

The TDF Option includes canals at the northern and southern ends of the conveyance option. The optional Supplemental Intake Canal in the north (approximately 12 miles) would convey water from intakes near Hood to New Hope Tract where it connects with the Mokelumne River.

The southern reach runs along the Victoria Canal from Middle River to Old River near Clifton Court (approximately 4.9 miles). The Victoria Canal would be modified to convey the entire 15,000 cfs. Future hydraulic studies would optimize the size of the Victoria Canal. Additional details on Victoria Canal are provided in Sections 16.0 and 20.0.

9.1 Canal Geometry and Footprint

This section presents the preliminary concepts for the canal portion of the conveyance. Canal geometry, design and footprint would vary along the canal alignment. Figure 9-1 presents a typical cross section. The canal would be a trapezoidal open channel. Where the channel WSE is generally above existing ground, the canal would be formed by earth embankments constructed of compacted engineered fill. A conceptual site rendering is provided as Figure 9-2.

Where the canal is formed entirely by excavation below existing ground surface, compacted berms a minimum of 4 feet high would be constructed at the edge of the excavation for the purpose of diverting surface drainage away from the canal.

Some factors that would affect local canal geometry and footprint include:

- Embankment and channel slopes;
- Embankment top width;
- Existing ground;
- WSE;
- Channel depth;
- Freeboard requirements; and
- Embankment crest elevation.

9.1.1 Embankment and Channel Slopes

The maximum slope for the embankments and permanent excavation was assumed to be 3H:1V for the purposes of this report. Final slopes would be determined during later engineering phases and would consider seismic loading and other factors.

An 8H:1V slope is shown on the conveyance side of the embankment from the WSE down to approximately six feet below the WSE. The 8H:1V slope provides erosion protection by mimicking the natural slope of a shore, dampening the wave height and energy along the FB and operational zones. The 8H:1V slope also reduces seepage by widening the embankment section. Alternatively, the 8H:1V slope may be eliminated and replaced with erosion protection.

This would reduce the embankment width but may increase seepage and decrease slope stability.

9.1.2 Embankment Top Width

The width of the top of the embankments is based on the requirements for canal facility maintenance access roads that would be located on the crest of the embankments. The tops of the embankments could be wide enough to allow for two maintenance vehicles traveling in opposite directions to pass each other.

A primary access road with a 16-foot-wide paved section with 4-foot-wide shoulders is proposed on the east embankment. The proposed design pavement section would be 3 inches of asphaltic concrete over 6 inches of California Department of Transportation (Caltrans) Class 2 aggregate base is based on recommendations by DWR Field Division maintenance staff for the East Branch of the California Aqueduct.

A secondary access road with a 12-foot-wide gravel section with 4-foot-wide shoulders is proposed on the west embankment. The design section for the secondary access road would be 8 inches of Class 2 aggregate base.

Access roads may diverge from the embankment crest to accommodate bridges, overchutes, and other structures. Where soft soils are encountered, additional measures may be needed to strengthen the base and subgrade.

9.1.3 Existing Ground Elevation

The existing ground is relatively flat. Most of the existing ground along the alignment lies below sea level and is protected from flooding by levees at the sloughs and rivers forming what are essentially sunken islands. The ground surface is crossed by drainage and irrigation ditches and raised berms for local roads.

9.1.4 Water Surface Elevation

The WSE for both the Supplemental Intake and modified Victoria Canals would be controlled by the operation of the intake and VCPPs and the operation of downstream control gate structures. The WSE is further discussed in Section 5.0.

9.1.5 Channel Depth

The canal channel is currently designed for a water depth of 23.5 feet. The channel depth is further discussed in Section 5.0.

9.1.6 Freeboard

Freeboard is the vertical distance from the high water level during operations or from potential floods or tides to the embankment crest elevation. Freeboard prevents waves from wind, maintenance boats, and canal operations from overtopping and eroding the embankment. For this report, maintenance standard FB is 3 feet.

9.1.6.1 Landside Freeboard

Freeboard is required on the landside of the canal embankments to prevent wind waves generated during flooding from eroding, overtopping, and breaching the embankment. Earlier reports on the feasibility of the peripheral canal indicate a minimum FB requirement of 3 feet (DWR, 1973). Recent calculations for FB in the San Francisco Bay at Oakland Airport indicate 3 feet is a reasonable FB height (Oakland Airport, 2007). Freeboard calculations specific to the canal embankment would be performed during later engineering phases.

9.1.6.2 Conveyance Side Freeboard

Freeboard on the conveyance side includes considerations for the development of wind waves on the canal and transient waves generated by operational events. Closing the control gates in the canal may produce transient waves when the momentum of water in the pool is reflected upstream as the control gates close. Transient waves can be minimized by controlling the rate of closure of the gates. Large vertical differences in the hydraulic grade at pool boundaries would result in larger transient waves. DWR FB requirements for improvements to the East Branch of the California Aqueduct call for 2 feet of concrete-lined FB plus 2 feet of unlined FB, for a total of 4 feet of FB (DWR, 2008d). For the purposes of this report, four feet of conveyance side FB is assumed.

9.1.7 Embankment Crest Elevation

Embankment crest elevations for the proposed TDF Option have been set to the highest of the following three criteria:

- Flood Event Levee Failure: WSE including SLR with an additional FB allowance that is the higher of the 3-foot standard FB or the computed wave run-up. (SLR is based on a 100-year SLR of 4.55 feet at the Golden Gate Bridge).
- Seismic Event Levee Failure: MHHW including SLR with an additional FB allowance that is the higher of the 3-foot standard FB or the computer wave run-up.
- Operational Event: Conveyance WSE plus 4-foot FB.

The embankment crest elevations may be refined in later engineering phases.

9.1.7.1 Flood Event

Overtopping of the canal embankments during a flooding event could result in a breach failure and take weeks or months to repair, interrupting service from the conveyance system. Such a break in service would prevent the water diverted by the intakes from reaching the export pumps in the southern end of the Delta.

Protection against overtopping is therefore critical to the continued operation of the isolated conveyance facility and would be achieved by setting the crest elevation of the embankments for the 200-year-flood event as described in Section 3.5. This flood event controls the top of the embankment elevation along the alignment. The elevation of the 200-year-flood event along the alignment would be refined during future engineering phases.

Future engineering phases would also consider methods of preventing inundation of adjacent islands by floodwaters flowing into the canal through a potential breach of an embankment within an island.

9.1.7.2 Seismic Event

Island levees may fail after a major seismic event (DWR, 2008a). Multiple island levee failures could result in inundation of Delta islands with salt water from the San Francisco Bay and potentially overtop the canal embankments. Estimates indicate it would take many months to repair the levees and remove salt water trapped inside the levees. The duration for repairs would depend on the number of affected islands.

Protection against overtopping of the canal embankments resulting from seismically-induced flooding of the islands is provided by setting the embankment crest elevation for the highest reasonable water level that might occur after the design seismic event. The MHHW level was used in this report for protection after the design seismic event, which has a 200-year return period.

The top of embankment elevation for the seismic event was calculated and found to be less than for the flooding event, thus the flood event governs in this proposed concept.

9.1.7.3 Operational Event

In some cases, particularly near pumping facilities where the hydraulic grade is highest, the canal WSE and its FB may control top of embankment elevation.

9.1.8 Footprint

As depicted in Figure 9-1, the footprint for the canal includes the following elements:

- Conveyance channel
- Embankments with maintenance roads
- Spoils areas
- Drainage and irrigation ditches
- Toe road

In addition to the canal footprint, the limits of work for construction include borrow areas, a larger haul road at the toe of the embankments, grading for drainage, and drainage pumping stations. Excavation spoils may be placed in the adjacent islands in lieu of along side the embankments.

9.1.9 Right-of-Way

At a minimum, permanent ROW would be required from approximately 10 to 20 feet west of the landside toe of the west embankment to approximately 10 to 20 feet east of the landside toe of the east embankment. Additional ROW would be needed where drainage ditches, spoil areas and toe roads are to be maintained by DWR. Alternatively, some drainage and access features required for canal maintenance may become part of a permanent utility easement. Total ROW for the canal may vary between 1,000 and 2,000 feet in width.

9.1.10 Erosion Control

Erosion from wind and wave on the conveyance side of the embankments during normal operations and on the land side of the embankments during potential flooding of the islands would be addressed during future engineering. Erosion on the conveyance side can be controlled with lined or mild slopes. Some conveyance side erosion control alternatives currently under review include:

- Unlined with 8H:1V slope
- Concrete lined
- Riprap lined
- Articulated concrete mat-lined

Lined slopes could also be provided to protect only critical erosion control areas such as side slopes or the operating zone rather than lining the full section of the canal.

An unlined canal is assumed for this report. Lining alternatives would be evaluated in a subsequent engineering study.

9.2 Geotechnical Considerations

The geologic and subsurface conditions for the general area of the canal have previously been investigated by DWR in the late 1960s and early 1970s. The available general subsurface information is briefly summarized in Section 3.3. Soils underlying the canal alignment include clayey and silty soils, alluvial flood plain deposits, organic clayey silt and silty clays, and fibrous peats. The results of a subsurface investigation program to provide specific subsoil information along the alignment would be considered in a subsequent engineering phase.

9.2.1 Organic and Peat Soils

The top layer of soil along some portions of the canal segments consist of up to 25 feet of organic and peat soils deemed unsuitable for support of the canal embankments. Therefore, in these areas removal and disposal or treatment of the peat and organic soils would be necessary. The removal of the full depth of the peat and organic soil may be limited to the area of the embankment foundations.

Treatment alternatives for the organic and peat soils could be evaluated in subsequent engineering studies. A cost comparison among alternatives would determine the most cost-effective approach to organic and peat soils. The cost of each approach to strengthen and reduce compressibility of organic and peat soils would be evaluated.

9.2.2 Liquefiable Sands

Potentially liquefiable sands (typically loose, saturated sands that undergo a significant loss of strength during an earthquake) may be present below the organic soils. It would be necessary to remove or stabilize these soils as part of the excavation for the canal embankments.

9.2.3 Slope Stability

The proposed embankment slopes are based on engineering evaluations of stability. Preliminary assessments were performed to evaluate a range of conditions. The assessments accounted for varying WSEs, and the effects of phreatic surface conditions and related pore pressures.

The embankment engineering presented in Figure 9-1 and described in this report meets the DHCCP design criteria for stability and related geotechnical concerns.

9.2.4 Seepage

Seepage from the canal could occur where the normal water level in the canal is higher than the groundwater levels of the adjacent areas. Seepage could potentially raise the water table on the landside of the embankments and result in losses of water from the conveyance that may not be recoverable. Canal waters could seep through the generally homogenous embankment fill and the foundation soils. Higher seepage flows may occur through more permeable lenses of sand and/or gravel in the foundation. Control of seepage could include the following:

- Installation of a slurry cutoff wall through the canal embankments and foundation. A cutoff wall would be most effective in areas where the canal cuts through layers of permeable sands and gravels.
- Use of a drainage ditch parallel to the canal to control seepage and groundwater levels. Water in the drainage ditch would then be pumped into the sloughs or back into the canal.
- Installation of pressure relief wells along the drainage ditch to collect subsurface water and direct it into the parallel drainage ditch.

These, and other seepage control alternatives, may be considered in later engineering phases. Additional soil borings drilled during later engineering phases could assist in identifying reaches of the alignment underlain by permeable sand and gravel zones or other geologic features that could require seepage control measures.

9.2.5 Groundwater Table

This report assumes the groundwater table would be within a couple feet of the existing ground surface. Additional investigations to determine the elevation of the groundwater table along the alignment are planned during future engineering phases. This information could be used in designing dewatering systems for construction, as well as establishing parameters for embankment slope stability, seepage, and drainage requirements.

Water quality evaluations would be conducted to determine if water collected in drainage ditches or wells requires treatment prior to being returned to the canal or natural waterways.

9.3 Structures

9.3.1 Culverts

Culverts are used to pass drainage water underneath the canal. The figure on the following page shows a profile for a typical culvert on the California Aqueduct. Due to the flat terrain and depending on the invert of the canal, culverts on the Delta canal may be built as inverted siphons. Grading would be required upstream of culvert inlets to properly drain adjoining areas. Similarly, an outfall with energy dissipation and associated grading would be needed at the downstream end of the culvert.

9.3.2 Irrigation Ditches

Construction of irrigation ditches to supply water for agricultural use may be required in areas where irrigation water supply ditches are separate from drainage ditches. The irrigation ditches would likely need to be elevated above the existing ground to allow for gravity flow. New pumps or siphons may be required to supply the irrigation ditches.

Irrigation ditches are further discussed in Section 13.0.

9.3.3 Drainage Ditches

Construction of the canal would disrupt both natural and agricultural drainage patterns along the alignment. Currently, excess irrigation, seepage and stormwater runoff are drained to low spots in the islands and removed by pumping.

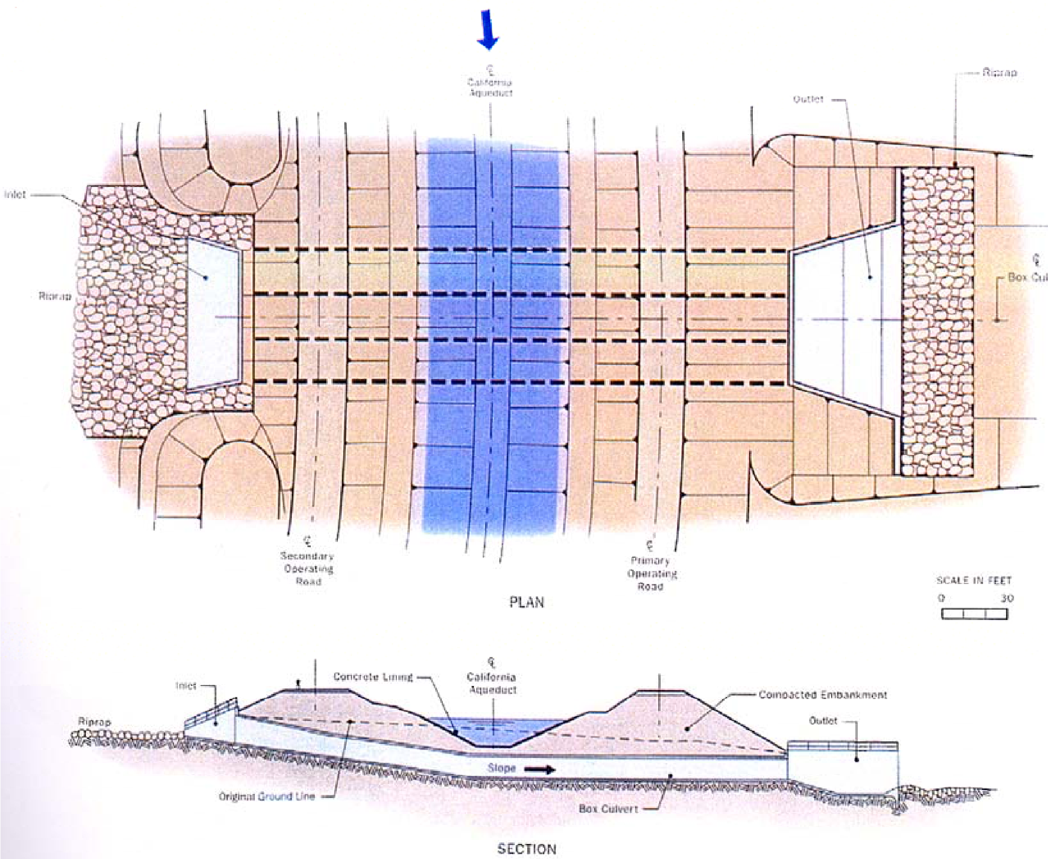
Generally, the existing ground slopes toward the canal on one side and away from the canal on the other side. If the existing ground slopes toward the canal on both sides, a drainage ditch would be constructed along both sides of the canal to collect water and direct it to collection points for removal by pumping. It is anticipated that these ditches would be approximately 5 feet deep and would connect to the existing drainage system.

In areas where the ground slopes away from the canal on both sides, or if surface runoff would be intercepted and conveyed around the canal by an existing drainage feature, no drainage areas would be constructed.

The risk to the canal from flooding in the adjacent islands may be reduced by providing a means for drainage water to pass from one side of the canal to the other. The water can be routed:

- Under the canal with a culvert to existing drainage systems,
- Over the canal with an overchute to existing drainage systems,
- Around the canal and through a gap between the existing levee and the ends of the canal embankments, or
- To new storm drain pumps that would pump the water to sloughs or the canal.

Drainage ditches are further discussed in Section 13.2.5.



Plan and Section Views – Culvert Crossing Under the California Aqueduct

9.3.4 Toe Roads

A toe road on each side of the embankment would be needed to provide maintenance to the drainage and irrigation ditches as well as providing access to areas otherwise cut off by the canal. The toe road would be paved where existing paved roads have been disrupted by the canal. In other areas where existing roads are gravel or not surfaced, the toe road is assumed to be gravel. The toe road would connect to the embankment maintenance road at locations where the embankment maintenance road is interrupted, at the ends of the embankments, and at bridges. The toe roads would tie into existing public roads and may or may not be publicly accessible.

9.4 Construction Methodology

Generally, construction of the canal channel and embankments would proceed in three main phases:

- Embankment Foundation and Channel Excavation (approximately 8.5 million cy)
- Embankment Construction (approximately 21 million cy)
- Spoils Placement

Excavation would proceed first with the excavated materials initially being hauled to off-site disposal areas or stockpiled nearby. Once a sufficient area has been excavated, the foundation for the embankments could be prepared and the embankments constructed. After a section of the embankments has been constructed, unusable materials excavated from the next section of the canal could be placed on the landside in the spoil areas shown on Figure 9-1.

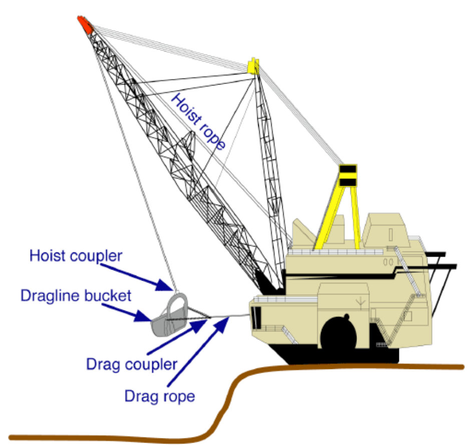
The canal and embankments could be constructed in independent segments possibly under separate contracts defined by island or designated reach. Determinations of the length for each construction contract reach should include consideration of the use of common borrow and spoil areas.

9.4.1 Excavation and Dewatering

Excavation of unsaturated soils could be performed using scrapers or excavators loading into large dump trucks. Excavated materials that are suitable for embankment fill could be hauled and placed directly into areas ready for embankment construction or stockpiled for future use; unusable material would be hauled to spoil disposal areas, as discussed in Section 22.0.

Excavating below the groundwater table using the same types of equipment would require extensive dewatering. Alternatively, excavation from approximately 2 feet above the groundwater table down to the canal invert could be formed in the wet using equipment like the walking dragline excavator shown below, thereby reducing the need for dewatering. The Bucyrus 680W is an example of such a dragline excavator. The Bucyrus 680W has a 225-foot boom and 30-cubic-yard bucket and would be supported by three large dozers. Together, this equipment is estimated to be capable of removing 1,260 cy per hour. The dragline excavator is electric-powered, thereby reducing the air quality impact compared with diesel-powered equipment.

An average depth of 8 feet of material would require excavation or treatment within the embankment foundation where the canal alignment passes through areas of organic materials. At this stage of engineering, it is assumed that the organic materials would be removed. Where removed, the organic materials would be replaced with compacted engineered fill. Placement and compaction of the engineered fill would require that the excavations be maintained in a dewatered condition. Dewatering of the excavations might require a combination of systems of wells with moveable collection and pumping systems and drainage ditches. Cutoff walls along the edge of the excavation may be required to control seepage into the excavation where permeable sand and gravel layers are exposed in the cut slopes or canal invert.



Dragline Excavator

Dewatering activities would comply with the NPDES permit and RWQCB requirements.

9.4.2 Embankments

It is unlikely that excavation of the canal would yield sufficient quantities of suitable material to build the embankments. Therefore, additional embankment material from off-site borrow locations would be needed. As previously indicated, embankment materials would be placed and compacted on the dewatered foundation. Moisture conditioning of the embankment materials would generally be performed in the borrow areas prior to hauling and placement in the embankments.

It may be necessary to include overbuild or camber in the embankments to accommodate some anticipated settlement in the embankments. Estimates of required camber, if necessary, would be determined during future design phases based on the foundation geology, embankment height and embankment materials.

9.5 Operations and Maintenance Considerations

9.5.1 Operations

The flow rate and water level within the canal would be controlled by control structures at each siphon and tunnel structure. Control structures might be radial gates similar to those shown below on the California Aqueduct. The control structures divide the canal into pools. Drawdown rates of water within the pools would need to be determined based on the stability of the conveyance side embankment slopes.

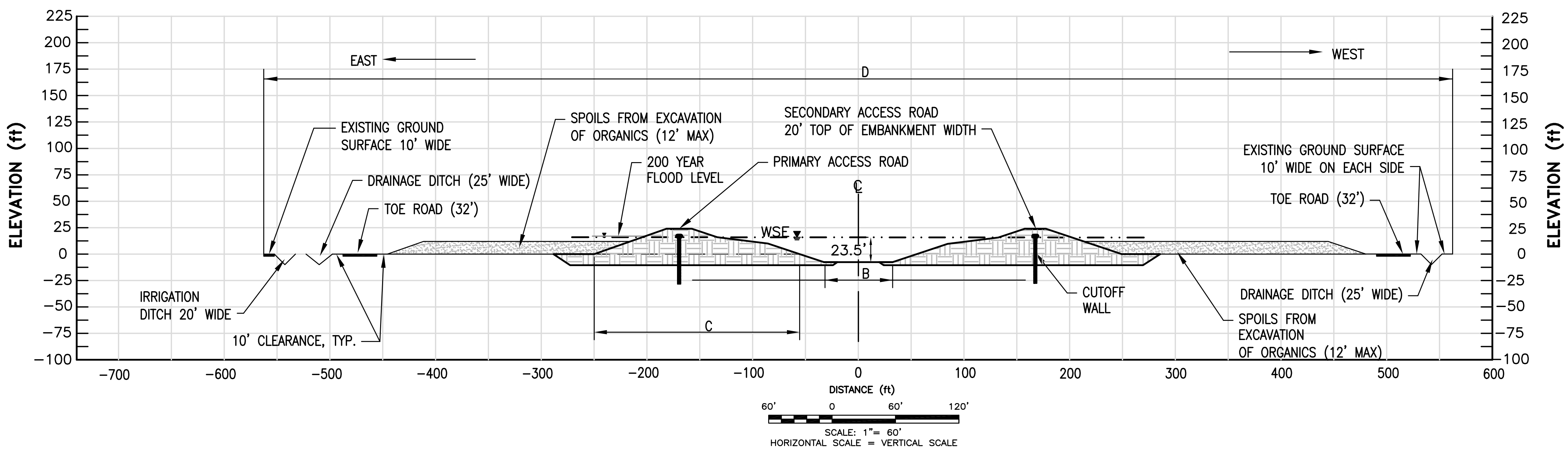
9.5.2 Maintenance

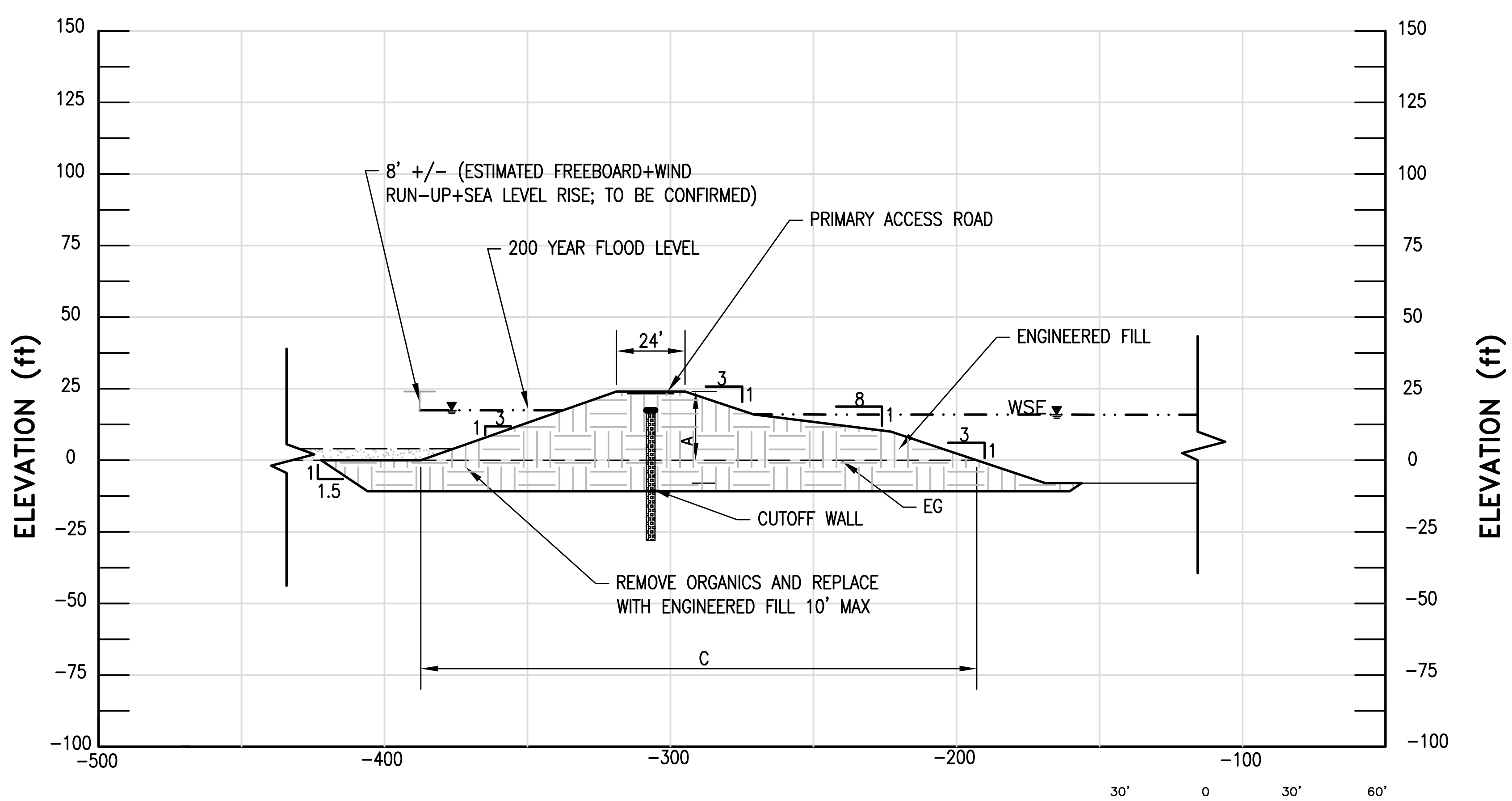
Maintenance requirements for an unlined canal would include control of vegetation and rodents, embankment repairs in the event of island flooding and wind wave action, and monitoring of seepage flows. Plant and animal life are expected on both sides of the canal embankments. Rodents, such as muskrat and beaver, have been known to undermine similarly constructed levee embankments causing embankment failure. Lining the canal with concrete, rip-rap or an articulated mat would reduce the maintenance effort required on the conveyance side of the embankments.

Radial Control Gates on the California Aqueduct

9.5.3 Sediment

Sediment would be expected to build up on the bottom of the canal. A measure that could be included in later engineering phases for reducing measure for reducing the sediment that would collect in the siphons and tunnels would be to construct sediment traps. The sediment traps could be formed by over-excavating portions of the channel upstream of the structures where the flow rate would be reduced to allow suspended sediment to settle at a controlled location. The sediment traps would be periodically dredged to remove the trapped sediment.





EMBANKMENT DETAIL SECTION

Flow (cfs)	Embankment Height (ft)	Canal Bottom Width (Unlined) (ft)	Approximate Embankment Footprint (ft)	Approximate Footprint (ft)
	A	B	C	D
4000	25	65	204	1450
4000	30	65	234	1500
4000	35	65	264	1575

LEGEND

- ENGINEERED FILL
- SPOILS
- WATER SURFACE

ABBREVIATIONS

WSE	WATER SURFACE ELEVATION
ELEV.	ELEVATION

NOTES:

1. ALL SLOPES ARE 3H:1V EXCEPT AS STATED.
2. IRRIGATION AND DRAINAGE DITCHES MAY NOT BE REQUIRED AT ALL LOCATIONS.
3. SPOILS AREAS, ROADS AND DITCHES ARE INCLUDED IN THE CANAL FOOTPRINT.
4. LOCATION AND DEPTH OF CUT-OFF WALLS TO BE DETERMINED.
5. FOOTPRINT MAY EXTEND TO 2000 FEET WHERE ADDITIONAL SPACE FOR SPOILS IS NEEDED.

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CANALS - TYPICAL CROSS SECTION

FIGURE
9-1

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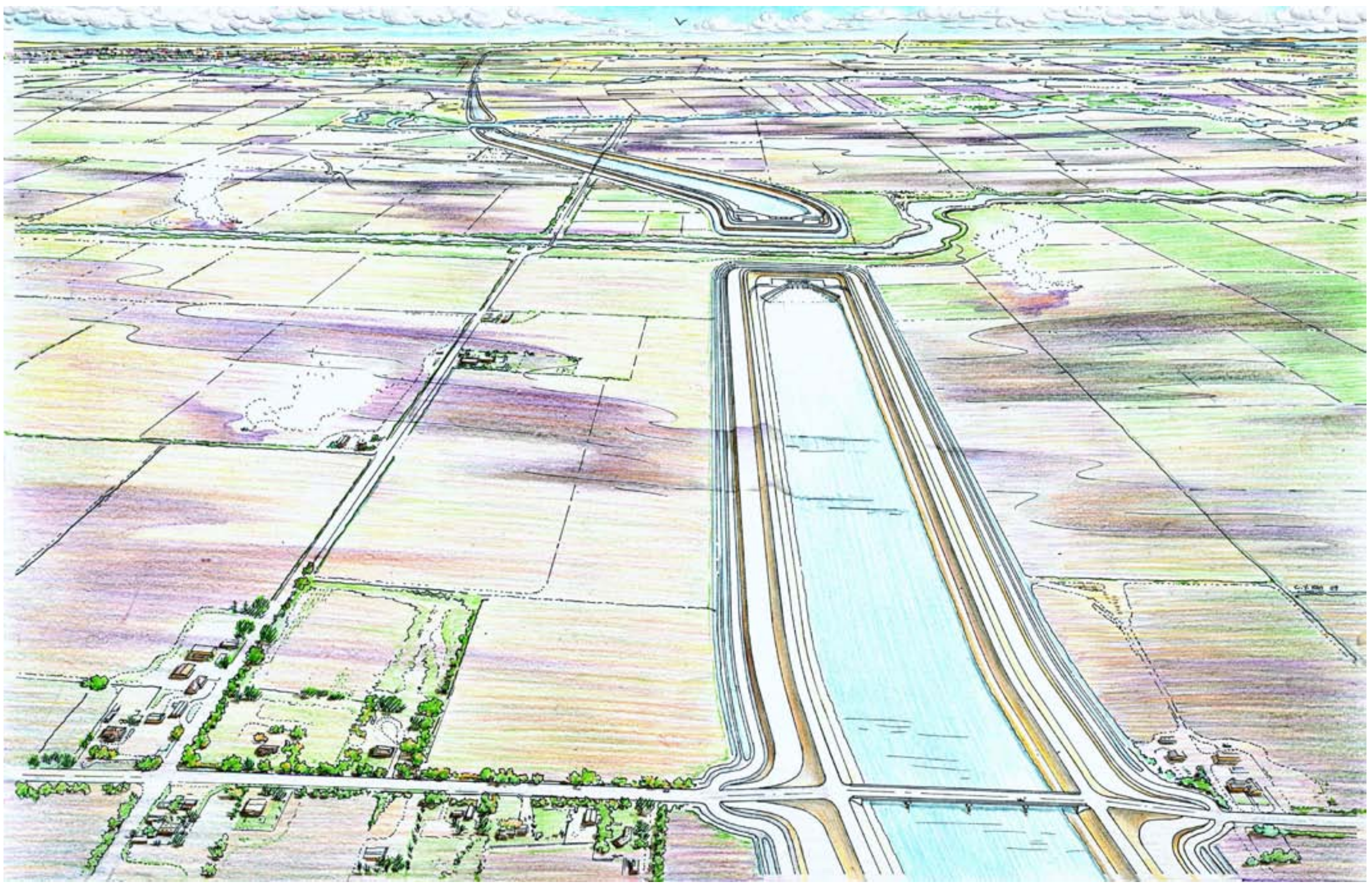

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FIGURE

9-2

Canal Rendering

10.0 CULVERT SIPHONS – SHALLOW CROSSINGS

Where the alignment crosses a major waterway, the preliminary engineering concept for the TDF Option uses inverted hydraulic siphons to convey canal water under the obstacle. When the waterway is relatively shallow, these siphons would comprise multiple cell box culverts. For simplicity, these structures are called culvert siphons in this report. Deeper waterways would be crossed using tunnels as described in Section 11.0.

10.1 Description, Locations and Site Plan

The TDF Option includes culvert siphons across the following three watercourses:

- Stone Lake Drain
- Old River
- West Canal

10.1.1 Proposed Culvert Size and Shape

The sediment characteristics for this project are not known at this time, but the water velocity in the canal approaching the culvert siphon is less than 2 fps; it is expected that most sediment would settle before reaching the siphons. The culvert size and shape was selected to optimize velocity and head loss. It is considered appropriate to size the siphon culverts to give a water velocity of between 5 and 6 fps.

Historically, record drawings show that culvert siphons on other systems have been built with either circular or rectangular sections. Because the culvert siphons required for this project are very large and are expected to be constructed in reinforced concrete within cofferdams, it is considered appropriate to use rectangular cells.

A typical culvert siphon is shown on Figures 10-1, 10-2, and 10-3 at the end of this section.

10.1.2 Dimensions and Levels

Using GIS data, including ground levels, bathymetry and peat depths where available, the main dimensions and levels for each culvert siphon have been assessed as shown in Table 10-1. The upstream and downstream canal invert levels have been taken from an assumed HGL with a canal depth of 23.5 feet but these levels would be adjusted to suit the final HGL. Invert levels at the low point of the siphon are controlled by the bathymetry, ground conditions, the need to establish full flow conditions, and depth of cover required to prevent scour or other damage to the structural integrity of the siphon.

The roof of the culvert has been set approximately 15 feet below the lowest point in the slough to establish full flow conditions. This assumption should be verified with additional hydraulic calculations. Regarding potential for uplift, preliminary calculations indicate that with one culvert dewatered approximately 7 feet of cover is required to prevent uplift with a factor of safety (FS) of 1.25 not including the additional resistance provided by the piles.

Many of the existing sloughs have flood control levees on one or both banks and in most locations, the new canal embankments would be higher than both the existing ground level and the existing slough levees. To provide for overland flood flows, a 100-foot buffer would be maintained between the toe of the slough levees and the toe of the canal embankment.

The culvert siphon inlet and outlet structures would be located within the footprint of the canal embankment. These structures would have flared wing walls to form a smooth transition from the much wider canal into the relatively narrow culvert siphon.

10.1.3 Foundations

The geologic and geotechnical setting is described in Section 3.3. The native subsoils may be characterized by the clayey and silty soils underlain by interlayers of silty and clayey sand and clay and silt sediments of the alluvial floodplain deposits. In general, the organic soils and fibrous peat are absent from the proposed locations for the culvert siphons (DWR, 1965).

In most cases, the geometry of the proposed culvert siphons would be such that peat would be excavated and the bottom of the structure would be founded on alluvial soils. Peat soils encountered below the inlet and outlet structures should be excavated and replaced with compacted imported material to the desired lines and grades.

At this stage of engineering and without detailed boring data at each siphon crossing, it is appropriate to assume that all siphon structures (inlet, barrel and outlet) would be supported on a pile foundation.

Table 10-1: Summary of Proposed Culvert Siphons & In-line Control Structures – TDF Option

Structure	ID	Location	Flow (cfs)	Number of Box Culverts	Box Culvert Width (ft)	Box Culvert Height (ft)	No. of Radial Gates	Estimated Siphon Length (ft)[1]
Siphon	1	Stone Lake Drain	4,000	2	19	19	2	1,736
Siphon	2	Old River	15,000	5	23	23	5	1,557
Siphon	3	West Canal	15,000	4	26	26	4	1,255
In-line Control Structure	1	End of Supplemental Intake Canal	4,000	NA[2]	NA[2]	NA[2]	2	255

1 Siphon Length = Length of Transition Structures + Length of Control Structures + Length of Culvert Box.
2 There are no box culverts associated with the in-line control facilities.

cfs = cubic feet per second
ft = feet
ID = identification

10.1.4 Mechanical and Electrical Systems

Control structures would be provided at the inlet to a culvert siphon to allow for regulation of upstream WSE. Control structures would also be provided at intermittent locations along the canal alignment to provide for improved control of the WSE where siphons are not required.

For this report, it is assumed that radial gates would be utilized to provide for control of the WSE within the canal. Radial gates provide for an efficient transfer of hydrostatic loads through the trunnion and also allow for a lower hoist capacity requirement as compared to other gates.

When all gates are closed, upstream water levels in the canal can be regulated, and the downstream siphon and canal can be isolated. When an individual gate is closed, the water velocity in the remaining cells can be increased to aid with sediment transportation as mentioned above.

It is assumed that each gate would be actuated using electric motors. A typical radial gate within the project area on the DCC (between the towns of Locke and Walnut Grove) is shown in the following photograph.

10.1.4.1 Mechanical – Gates, Hoists, and Other

Each gate would be provided with an independent electric hoist capable of remote operation. Provisions for gate operation during a power outage have not been provided. Operation during a power outage (if needed) could be accomplished by transporting portable generators to the site. It is assumed that permanent generators for the purposes of operating the gates during a power outage are not necessary.

Typical Radial Gate

The gates would include bottom and side seals to control leakage while in the closed position. Gate slots are not required which should reduce potential problems due to cavitation and debris collection.

10.1.4.2 Control Modes and Control Basis

In general, all equipment at each control structure would be operated either locally or remotely. Table 10-2 describes the controls equipment for the siphon control structure.

Table 10-2: Equipment Controls

Equipment Name/Tag	Description	Control		
		Local at Equipment in Field	Local at Control Panel or VFD	Remote PLC/SCADA Control
Control Structure (typical)	Modulating electric-operated radial gate.	Local-Off-Remote (LOR) Switch. OSC switch.	Calculate flow based on upstream level, downstream level, and gate position.	OSC and percent open control based on operator entered percent open set-point.

LOR = Local-Off-Remote
OSC = Open-Stop-Close
PLC = programmable logic controller
SCADA = supervisory control and data acquisition
VFD = variable frequency drive

10.1.4.3 Electrical – Cables, Ducts, and Conduits

Duct or conduits can be cast into the walls of the siphon culverts so that power and/or control cables can be carried across the waterways.

10.1.5 Site-Specific Culvert Siphon Details

The TDF consists of three proposed siphon locations. The names of these siphons from north to south are: Stone Lake Drain, Old River, and West Canal.

Further description of each siphon location is provided in the following sections.

10.1.5.1 Stone Lake Drain Siphon

The proposed Stone Lake Drain siphon is located in Sacramento County at the northern end of the TDF Option and is part of the Supplemental Intake Canal. This siphon lies within the Pierson District approximately 2 miles west of I-5, and less than 3 miles southeast of where SR 150 crosses Snodgrass Slough. According to a survey conducted in 1986 and 1987 (DWR Web site, 2009), there are no irrigation diversions, but at least one agricultural drain returns to the Stone Lake Drainage. There are no recreational areas in the vicinity of Stone Lake.

There are no major utilities within the general vicinity of the proposed Stone Lake Drain Siphon. No major floods have occurred within the Stone Lake Drain area between 1930 and 1992 based upon information obtained from the DWR Web site.

10.1.5.2 Old River

The proposed Old River siphon is part of the Victoria Canal to CCF improvements and is located in San Joaquin and Contra Costa Counties. This siphon lies between Union Island and Coney Island approximately 2 miles south of SR 4 and 3.5 miles northeast of County Route J4. There are two major constructed waterways extending east from the vicinity of Coney Island, Victoria/North Canal and Grant Line/Fabian & Bell Canal. Both Canal systems connect the Old and Middle Rivers. According to a survey conducted in 1986 and 1987 (DWR Web site, 2009), there are at least 15 irrigation diversions from and one agricultural drain returns to the small waterway that comprises part of the Old River east of Coney Island. There are at least 15 irrigation diversions from and three agricultural drain returns to Victory/North Canal and at least 32 irrigation diversions from and 14 agricultural drain returns to the Grant Line/Fabian & Bell Canal. There are no recreational areas in the vicinity of the Old River siphon.

Power transmission lines of up to 500 kV under the jurisdiction of the WAPA run in the northeast to southwest direction, approximately two miles to the southeast of the proposed siphon area. No major historical floods have occurred within the Old River siphon area between 1930 and 1992.

10.1.5.3 West Canal

The proposed West Canal siphon is also part of the Victoria Canal to CCF improvements and is located in Contra Costa County. This siphon lies on the east side of CCF approximately 1 mile southwest of the Old River siphon location. The CCF provides water to the Tracy Pumping Plant and Banks Delta Pumping Plant for supplying water to the DMC, California Aqueduct, and South Bay Aqueduct. Two constructed waterways extend to these pumping plants. According to a survey conducted in 1986 and 1987 (DWR Web site, 2009), there are at least no irrigation diversions from and at least one agricultural drain return to CCF. There are at least nine irrigation diversions from and two agricultural drain returns to the Tracy Pumping Plant waterway and no irrigation diversions or agricultural drain returns to the Banks Delta Pumping waterway. There is one recreational area located on the west side of CCF, which is approximately 2 miles west of the proposed siphon.

Similar to the Old River siphon location, there are power transmission lines of up to 500 kV under the jurisdiction of the WAPA run in the northeast to southwest direction, approximately 2 miles to the southeast of the proposed siphon area. No major historical floods have occurred within the West Canal siphon area between 1930 and 1992.

10.2 Construction Methodology

The culvert siphons identified in this section of the report would be constructed as large multiple-box culvert structures using cofferdams and open cut-and-cover construction methods with conventional CIP concrete structures.

There is a four-month window in the low water season (August 1 to November 30) for driving steel sheeting to construct a cofferdam, or performing any work activities in the water (e.g., excavation using a dragline). Once the cofferdam is completed and the enclosure is in place, work can continue (e.g., dewatering, excavation, pile driving, concreting) inside the cofferdam for the remainder of the year. That is, the work performed inside the cofferdam is not considered to be work in water. This means of working was previously envisaged for the South Delta Improvement Program. It is also assumed that no work can be done on or near the levees during the high-water seasons.

10.2.1 Duration

Depending on the width of the slough and the expected flow conditions, in most cases it should be possible to construct a bypass channel and redirect the slough away from the work area. In these cases, construction can be continuous for each slough.

For larger sloughs or where other restrictions exist, it is envisioned that the culvert siphons would have to be constructed in two phases, each phase lasting one year unless flood control or fisheries issues force a shortened work window. In the first phase, a temporary cofferdam would be installed approximately halfway across the slough, the encircled area would be pumped dry, and the exposed soil would then be excavated down to the desired lines and grade. Half of the total length of the culvert would then be constructed inside the cofferdam, temporarily plugged, and backfilled to the desired waterway bottom configuration.

During the second phase, the cofferdam would be re-installed across the other half of the channel, and the remainder of the structure would be constructed and backfilled. In this way, use of the waterway for recreational navigation would be allowed to continue during construction, albeit with appropriate temporary construction zone restrictions in place for marine safety.

For longer culvert siphons, it may be necessary to construct the culverts in three phases over three years.

10.2.2 Construction Footprint

In addition to the physical footprint created by the proposed works, the following footprint would be required during construction for the general and subcontractors:

- Construction area and laydown for each siphon inlet – 15 acres.
- Culvert siphon – 250 to 500 feet along the length of the slough, plus additional footprint area for backup levees and/or bypass channels to temporarily contain the slough.
- Construction area and laydown for each siphon outlet – 15 acres

10.3 Operations and Maintenance Considerations

10.3.1 Hydraulic Capacity and Sediment

Flow velocity through the siphon culverts and the resultant hydraulic head loss across the siphons are the two principal criteria used to determine the size. Because all siphon culverts have a low point below the waterway or obstruction they are crossing, there is potential for sediment deposition. Higher water velocity would help flush sediments through the siphon but at the same time it increases hydraulic head loss which would increase pumping costs. Furthermore, larger culvert siphons would be more expensive to build.

The potential for sediment deposition is a function of the sediment characteristics including grain size and density, sediment concentration and water velocity. Sediment may be brought into the new canal system from the upstream waterway, it may be picked up from the canal floor or sides, and it may be windblown material. The California SWP Vol. II Conveyance Facilities

Bulletin No. 200, November 1974, page 13, states: “Sediments can build up to sizable amounts in canal inverts and, where feasible, were excluded from the canal. Wind-blown soils in desert and agricultural areas, particularly during cropland preparations, can produce a considerable amount of sediment. Sediment traps were incorporated in the invert of the canal sections in certain reaches. These traps are rectangular hopper type structures…” (DWR, 1974a).

The water flow velocity used for previous work has been reviewed for this report. The Draft August 1974 Peripheral Canal EIR (DWR, 1974b) described the siphons envisaged at that time. By back calculation, the water velocity in the siphons would have been approximately 8.8 fps, which is assumed to be sufficient to keep sediments from depositing within the low point of the siphon. However, dimensions given in the Conceptual Analysis of Incised Canal Configuration by CALFED, Water Management Planning Branch, in September 1999 (CALFED, 1999) would give a significantly lower velocity of just below 3 fps for 15,000 cfs.

A design water velocity of between 5 and 6 fps is a compromise between potential sedimentation and head loss and supports the proposed culvert sizing of four cells of 26 feet by 26 feet per cell (barrel).

10.3.2 Ability to Close Individual Culvert Cells

If sedimentation proves a problem, it would be possible to close one of the control gates in the control structure (see below) at the siphon inlet, thereby temporarily increasing the water velocity and sediment transport capacity in the remaining open culvert cells. In this way, it should be possible to flush out some sediment deposits.

10.3.3 Stop Logs

Stop logs and stop log channels are provided at the inlet and outlet structures to allow individual cells to be isolated, dewatered, de-silted, and maintained. The stop logs would be located upstream of the inlet control structures, as well as downstream of the outlet structures, to allow a siphon and gate to be isolated for maintenance purposes.

10.3.4 Floating Barriers and Safety Chains

Floating debris have the potential to travel downstream towards the culvert siphon intake. As an aid to mitigate the impacts of floating debris and to enhance public safety, floating barriers with hanging safety chains would be provided across the canal upstream of the siphons. Additionally, notice boards are proposed to warn the public about the danger presented by the siphons.

10.4 Control Structures

Control structures are proposed at the discharge end of the optional Supplemental Intake Canal, at the CCF intertie to the CVP approach canal, and in the CVP approach canal. These structures are located where additional level control is needed, but a siphon crossing (with accompanying gate structure) is not required. These structures are in addition to the control structures at each siphon and provide water surface level control in the canal, forebay, and approach canals to the export pumping plants. The control structures would essentially consist of an inlet transition, gate structure, and outlet transition and allow for isolation of sections of the canal for maintenance, if necessary.

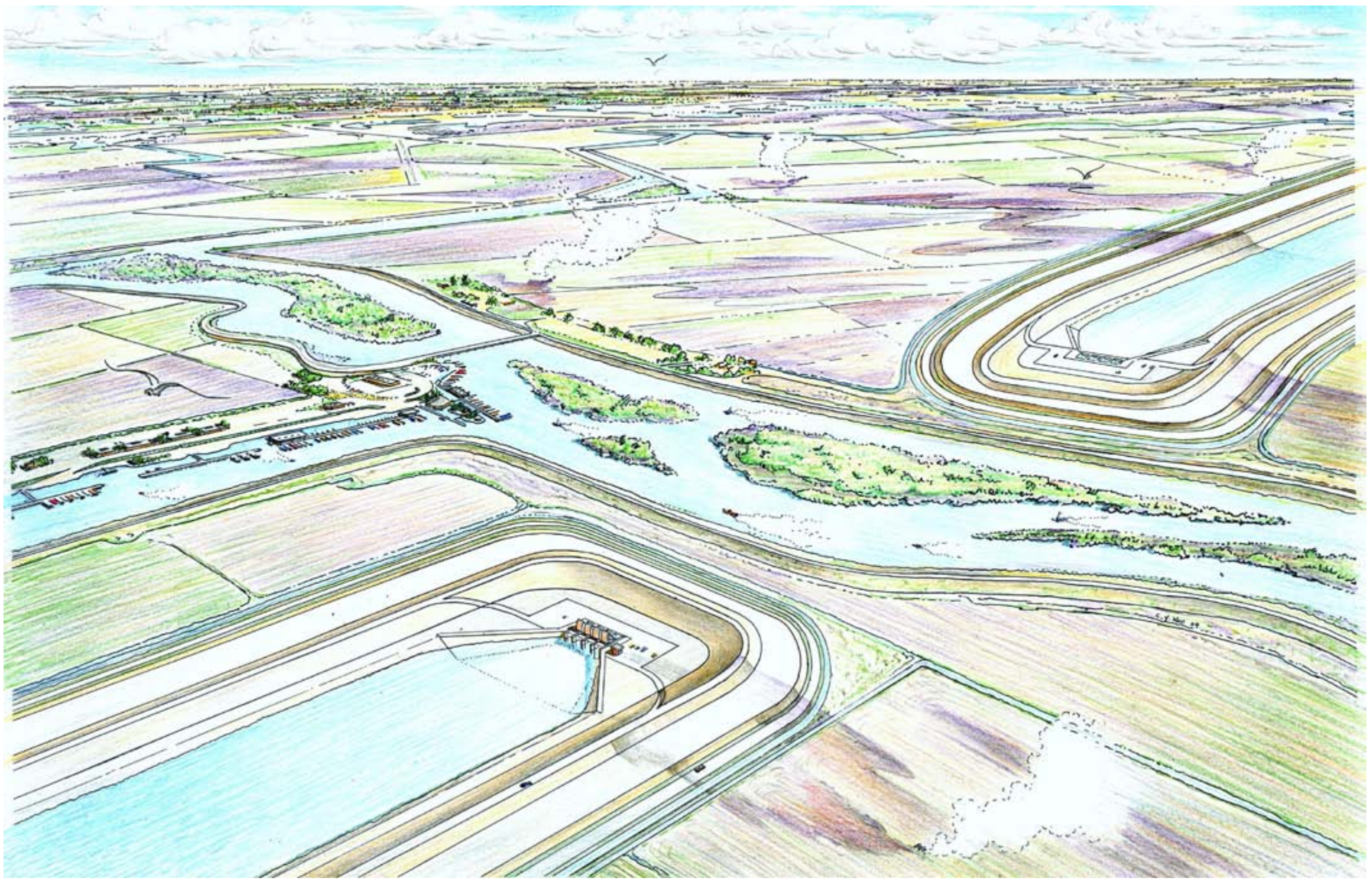

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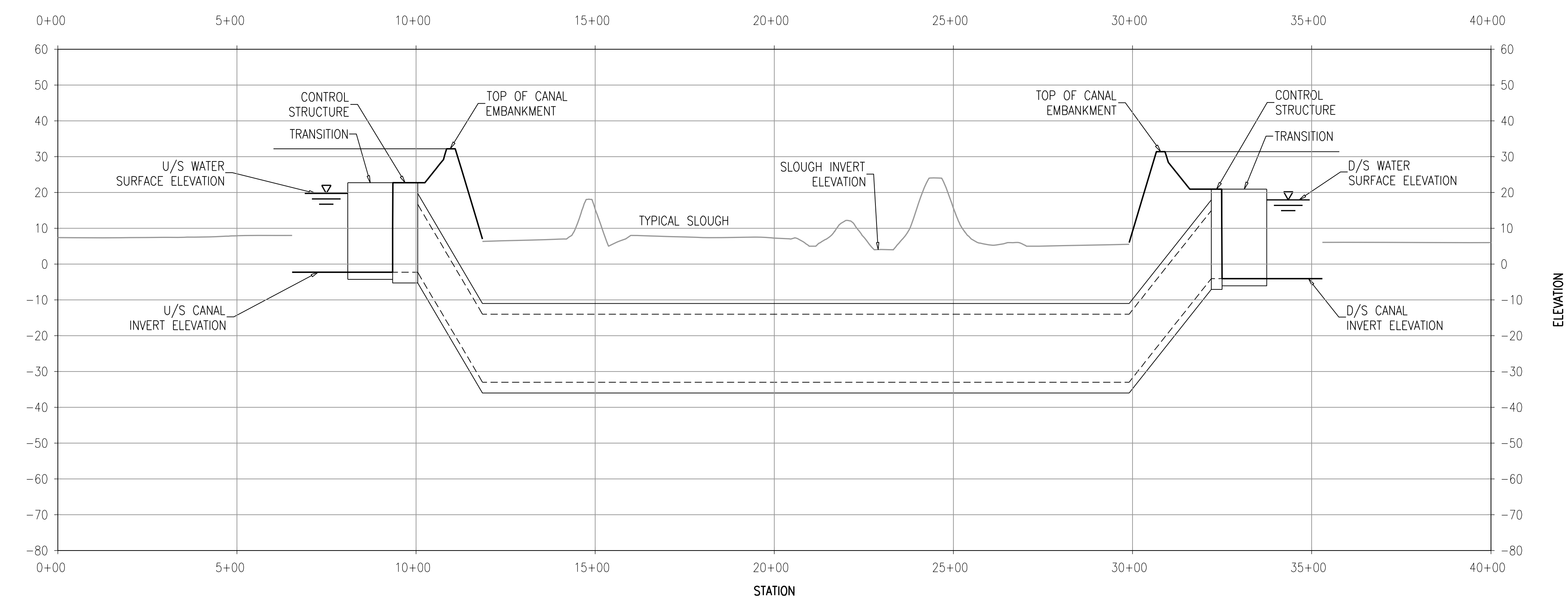
FIGURE

10-1

Culvert Siphon



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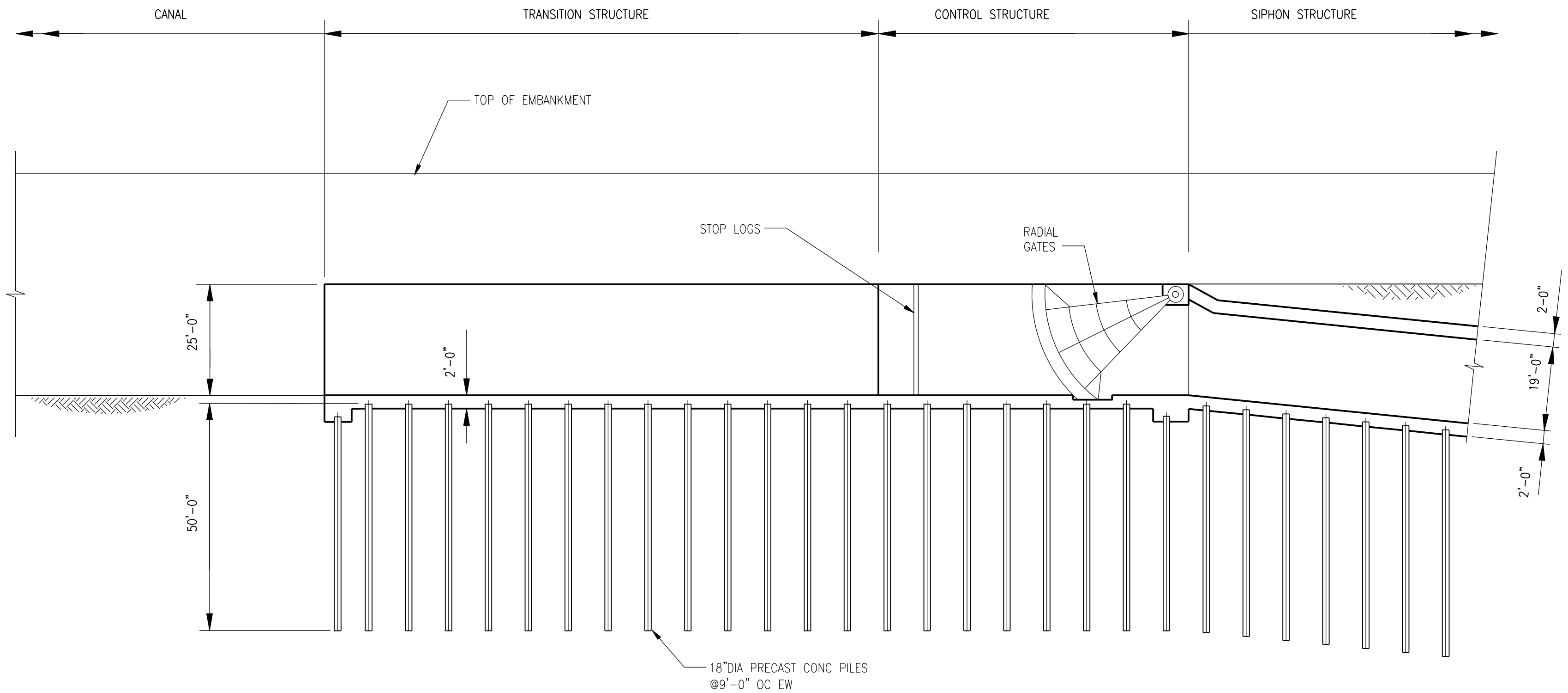
Culvert Siphon
Typical Section

FIGURE
10-2



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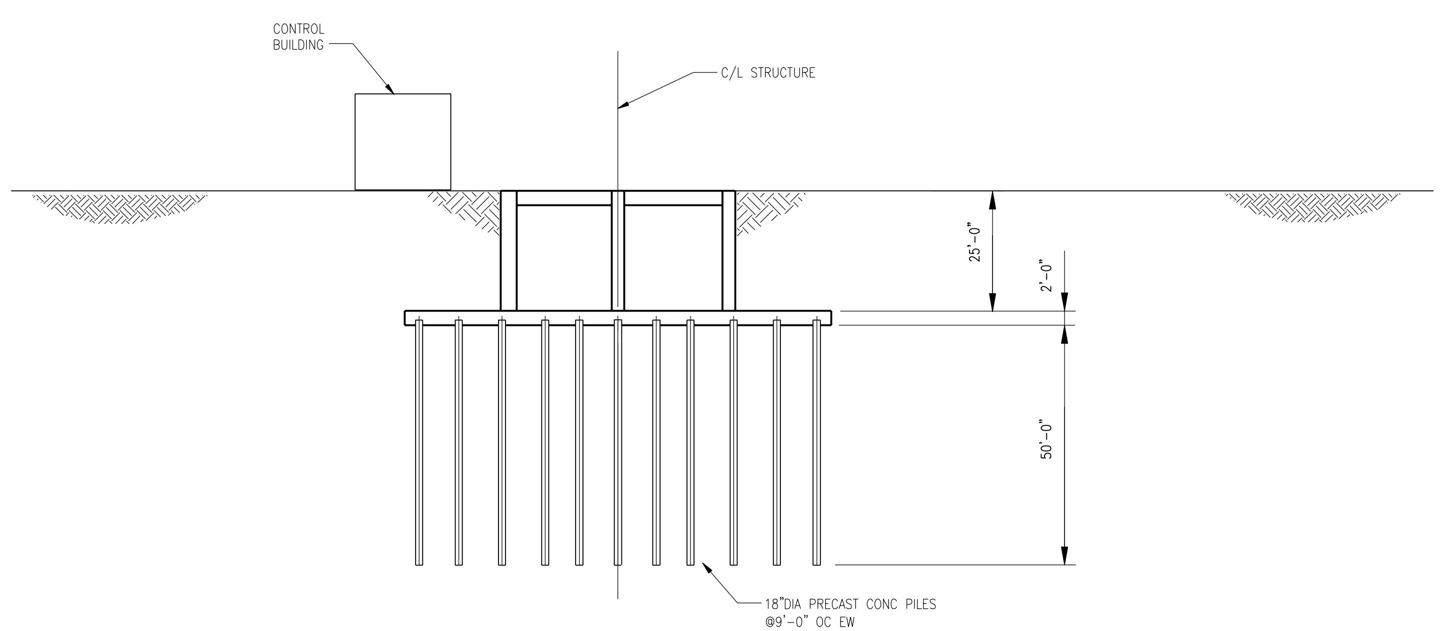

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Culvert Siphon
Inlet Sections

FIGURE
10-3

11.0 TUNNELS – DEEP CROSSINGS

The TDF Option includes two optional deep crossings necessary for environmental reasons and to sustain the long-term operation of the facility in the event of catastrophic levee failure. These deep crossings are:

- Lost Slough-Mokelumne River (Supplemental Intake Canal Tunnel)
- San Joaquin River

11.1 Description and Site Plan

The TDF Option proposed deep crossings are shown in Table 11-1.

Table 11-1: Tunnel Crossings

Crossing	Number of Tunnels	Inside Finished Diameter (feet)	Length (feet)
Mokelumne River (Supplemental Intake Canal Tunnel)	1	27	9,350
San Joaquin River	2	33	18,340

The configuration of each tunnel crossing is dictated by the required hydraulic capacity, the need to minimize the hydraulic head losses, and the required flow velocity to maintain any sediment in suspension and reduce its settling in the deep section of the tunnel. Assuming 15,000 cfs total capacity for conveyance, two bores—each 33 foot inside finished diameter—are proposed at each deep tunnel crossing.

The tunnels are anticipated to be approximately 150 feet below mean sea level (msl), primarily to avoid peat deposits and provide enough ground cover under deep-water courses, and spaced a minimum of 66 feet (two outside tunnel diameters) apart. The main construction shaft, or launching shaft, for each tunnel would be approximately 60 feet in diameter to accommodate construction and construction support operations, such as ventilation. The tunnel-boring machine (TBM) retrieval shaft would be approximately 45 feet in diameter. A typical tunnel profile is shown on Figure 11-1.

The tunnels would be designed as concrete lined with pre-cast bolted-and-gasketed segments as shown on Figure 11-2. The tunnel concrete liner serves as permanent ground support and would be installed immediately behind the TBM, essentially forming a continuous watertight vessel. The depth of the tunnels would be based on future geotechnical investigations and would need to balance avoiding extremely weak surface soils, such as peat and organic soils, with the additional costs associated with a deeper tunnel in more competent soil material. Until site-specific geotechnical data is obtained, the invert elevation of the tunnels is assumed to be at 150 feet below msl.

Support of construction operations would require approximately 15 acres per tunnel to allow location of offices, parking, shops, segment storage, fan line storage, daily spoil pile, power supply, water treatment, and other requirements. Depending on the methodology selected to construct the walls for the shafts, the staging areas would initially also include the slurry ponds used for slurry wall construction. Work area requirements of approximately 30 acres total for each end of each tunnel bore should be reserved at all of the potential starting locations, exclusive of muck handling and disposal.

11.1.1 Floating Barriers and Safety Chains

Public access to the canal in the vicinity of the tunnel inlet should be limited and recreational boating, fishing, and swimming should not be allowed within an unsafe distance upstream or downstream of the structures. Notice boards would be used to warn the public about the danger presented by the siphons.

11.1.2 Intake and Outlet Structures

It is assumed that each tunnel bore would need to be isolated and dewatered individually to allow for inspection and maintenance. For this purpose, each tunnel would start and end in concrete control structures. These structures would serve as transition with the canal both upstream and downstream, or the pumping facilities upstream. Sets of primary and guard large wheel gates would be used to isolate each tunnel bore and allow its dewatering for inspection. The gates would have removable emergency bulkheads for their maintenance. The gates and bulkheads would be operated using permanent frame gantries. At this time, no permanent pumping capabilities have been included, so the tunnel dewatering would be performed using mobile pumps. Backfilling the tunnels would be done through a piping system that draws water from the canal until the pressure on both sides of the gate is equalized and their operation is possible. For the San Joaquin River crossing, the upstream control would be with valves instead of gates.

11.1.3 Individual Tunneled Crossings

11.1.3.1 Lost Slough-Mokelumne River (Supplemental Intake Canal Tunnel) Crossing

An environmental habitat area controls this crossing, requiring transferring the water conveyed by the canal, through a set of intake structures, into a 27-foot-diameter, 9,350-foot-long tunnel, approximately 160 feet deep, and through outlet structures discharging into the canal.

11.1.3.2 San Joaquin River Crossing

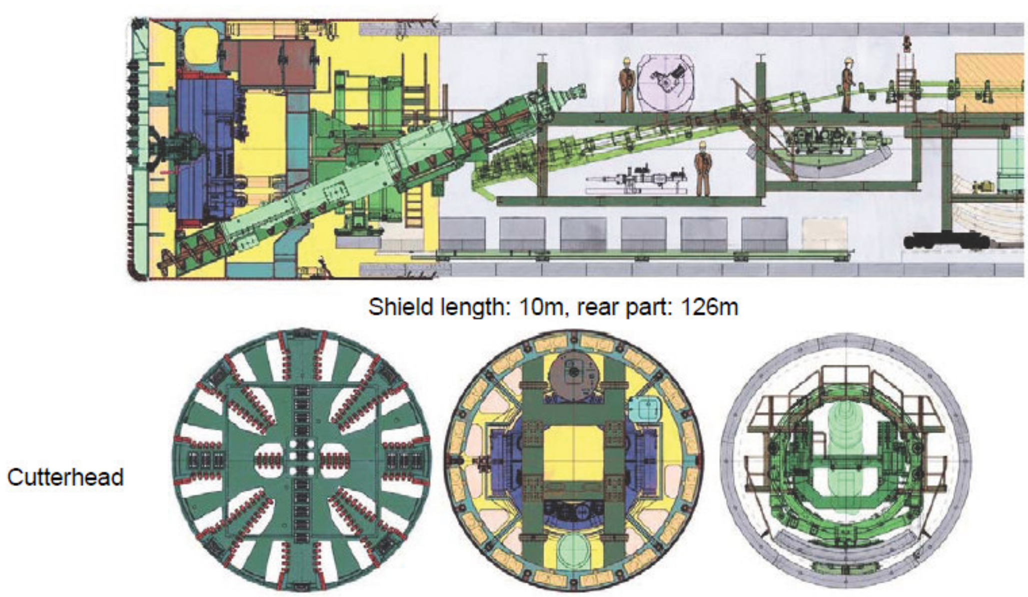
The San Joaquin River crossing is a navigable deep-water waterway that cannot be interrupted by construction. The canal flow would be transferred through a pumping station and into two 33-foot-diameter, 18,340-foot-long tunnels, approximately 160 feet deep, and through outlet structures discharging into the canal.

11.2 Construction Methodology

This section summarizes alternative tunnel construction methods. The compatibility of the tunneling excavation method with anticipated ground conditions is the most important consideration with respect to tunnel excavation and support. At this point, there are no borings that extend to the depth of the tunnel alignment. Once the geotechnical investigations have been performed, preliminary design evaluations would be required to recommend tunnel excavation and support methods compatible with anticipated geotechnical conditions.

11.2.1 Tunnel Excavation Methods

The proposed tunnels are anticipated to be constructed in alluvial soils (soft ground) at depths greater than 100 feet with high groundwater pressures. Because of this, the tunnels would be constructed using mechanized soft ground tunneling machines. Closed-face pressurized machines would be needed because of high groundwater and earth pressures. Pressurized face machines can be utilized in a wide range of conditions and are specifically designed to counterbalance high earth pressures at the face, and resist high groundwater pressures. Pressurized face mechanized tunneling machines include earth pressure balance (EPB) machines and slurry tunneling machines. EPB machines hold the excavated tunnel spoils in a pressurized chamber behind the cutter head. This is used to counterbalance earth pressures. Pressure is held at the tunnel face by carefully controlling the rate of spoils withdrawal from the chamber using a screw auger while the machine is pushed forward. EPB machines are suitable for tunneling particularly in silts and clays. They are also applicable to granular materials when foam conditioners are added to plasticize the excavated material in the pressurized chamber. Slurry machines utilize a highly viscous fluid pressurized in the chamber in front of the cutter head to counter balance earth pressures. The fluid pressure is developed and maintained by pumping the fluid into the chamber and mixing it with excavated material to form slurry. The pressure in the chamber is regulated through a plenum and the slurry is pumped out of the chamber at a specified rate to a slurry separation plant. The distance that the slurry can be effectively pumped is one of the constraints of a slurry machine.



Cross-Section EPB TBM

The tunneling methodology to be used for the TDF Option tunnels is most likely to be a closed-face TBM, specifically an EPB TBM. The use of an EPB TBM technique enables the construction of tunnels in soft ground conditions and a high water table. The TBM shield supports the walls and roof of the excavation until the precast segmental liner is erected at the end of the shield. The pressure at the face is maintained by the controlled release of excavated material via a screw conveyor. Muck is discharged into muck cars or onto conveyors to be removed off site. Proper use of the EPB technique allows only the removal of the theoretically correct amount of material, thus greatly reducing the potential of surface settlement. The proposed diameters of the tunnels are close to the largest ever used for an EPB tunnel in the United States, but are within the precedents utilizing the technology elsewhere.

Each tunnel would require appropriately sized shafts to accommodate TBM equipment. Each tunnel would also require a hand-mined starter tunnel for the TBM and a hand-mined tail tunnel for tracking and mucking equipment. The starter tunnels would be constructed in areas previously treated by grouting, jet-grouting, or ground freezing.

EPB TBM operations require the use of additives in order to control the behavior of excavated material. The additives include water, surfactant foam, polymers, bentonite, or any combination thereof, although modern practice uses foams and polymers that are more environmentally friendly than bentonite. Foam is essentially a mixture of air and diluted foaming agent in water. Foam and/or polymer enhance the EPB machine’s ability to control face pressure. They are also used to reduce the level of torque required to cut the ground, which, in turn, reduces the required power input to the motors. Foam makes the cuttings more plastic and less permeable. Polymers are used to condition the soil, either by absorbing water or by affecting the deformation and flow characteristics of the soil. The main purpose of polymers is also to help support the face and encourage loose, coarse-grained soils to move smoothly through the excavation chamber. Polymers can also be used to reduce the tendency of soils with large amounts of highly plastic clay to stick to the cutterhead. The modern additives are not toxic and are biodegradable.

The tunnel muck-handling system would likely consist of either muck cars or continuous conveyors. Each system has its own unique features, but they have good records in practice. Either method would lead to stockpiling of muck at the ground surface and subsequent transfer to disposal areas, which might be handled by conveyor, wheeled haul equipment, or barges, at the contractor’s selection.

Considering the required construction schedule for the balance of the TDF conveyance, a minimum of three TBMs would be required, one for each tunnel crossing. One of the machines could be refurbished and reused in the shorter crossing.

11.2.2 Tunnel Support

It is anticipated that bolted and gasketed precast segments would be required for tunnel support. The segmental lining would need to be designed to support external earth pressures and groundwater pressures as well as construction loading due to handling, erection, and thrusting of the TBM. Bolted and gasketed precast concrete segments consist of rings, typically of six pieces. These pieces are bolted together at the circumferential and longitudinal joints. The finished ring formed by the segments is smaller than the excavated cylinder, and the annular space between the ring formed by the segments and the ground needs to be grouted to positively support the ground, and to have successful performance of this type of tunnel support system.

Instrumentation and monitoring would be a requirement for tunnel construction. The three bores would not advance at the same rate. In actuality, they would be scheduled to advance “in echelon” or staggered to avoid adversely loading one tunnel during construction of a parallel tunnel. Instrumentation and monitoring would be designed to take advantage of this staggering and provide data to allow evaluation of construction safety.

11.2.3 Precast Segment Plant and Yard

A precast segment plant would be required to produce tunnel segments. The plant size would be dependent on the total number of segments required and the schedule for production but it is likely that a plant would require approximately 10 acres for offices, materials storage, concrete batch plant, materials storage, and casting facilities. The amount of segment storage space would need to be added on to the plant space requirements and could be several times the space required for the plant. It is likely that the contractors would want to locate the segment plant near the main construction portal to minimize hauling distance for the segments. The segments can be transported by barge, rail, or truck where these modes of transport are available; however, it is most likely that trucking of segments would be required.

Manufacturing of the tunnel segments would not necessarily be done at the same precast segment yard for each tunnel contract. Each contractor would probably want to set up his/her own plant and this would typically be part of the contractor means and methods determined during bidding. It is possible that one precast segment plant could serve all of the tunnel contracts as a subcontractor; however, the tunnel contractors would be free to make the decision whether to set up their own plant as part of the competitive bidding process and it is likely that with four contracts, more than one precast segment plant would eventually be set up.

Typical Casting Yard

11.2.4 Tunnel Gas Classification

All tunnels constructed in California are required by law to obtain a tunnel gas classification from the State of California Division of Mines and Tunnels. Typically, at the 60% level of engineering or after the geotechnical investigation is completed and the tunnel plan and profile have been established, the project documents are submitted to the California Department of Industrial Relations, Division of Industrial Safety and Health (Cal/OSHA) and a Tunnel Gas Classification is requested. The classification is then typically included in the project specifications. It would contain special requirements that the contractor is required to follow during construction related to tunnel construction safety, gas monitoring, fire prevention and safety, and explosion prevention.

There are active natural gas fields beneath the anticipated vertical alignment for the tunnels. During the geotechnical investigations, and in design, it would be necessary to evaluate how these gas fields affect the constructability of the tunnels. In any event, because these gas fields are present near the tunnel alignment, it is anticipated that Cal/OSHA would classify the tunnels as gassy or extra hazardous. This classification is likely to carry with it some very high levels of precautions related to tunnel construction safety. The tunneling machines would be required to be equipped with gas monitoring equipment that automatically shuts down the TBM if gas is detected. It is also likely that special ventilation requirements, as well as special access and egress requirements, would be imposed by Cal/OSHA (at a minimum).

11.2.5 Other Tunneling Issues

Construction of the tunnel crossings may have the following impacts.

- **Traffic.** An estimated 50 construction workers would travel to and from the work site (launching shaft area) every 8 hours for 5 or 6 days per week for the multi-year construction.
- **Noise and Vibration.** Tunneling equipment is electric-powered and the expected soft ground conditions would limit vibration.
- **Water Treatment and Disposal.** Water seeping into the shaft during construction would be collected, treated (if necessary) and disposed of in accordance with the NPDES permit and RWQCB requirements.

11.2.6 Ground Improvement

Ground improvement would be required to support the tunnel and to control groundwater at the locations of the shaft. Site-specific geotechnical investigations are needed to determine the extent and type of ground improvement that may be required. The types of ground improvement that would likely be considered include grouting (such as jet-grouting or compaction grouting) and ground freezing. The choice usually depends on the ground conditions and the methods preferred by the contractor.

11.2.7 Shaft Construction

Temporary construction shafts and permanent shafts would be required along the tunnel alignment. There would be a shaft to lower the TBM to its initial working position, and to support its operation, and there would be a shaft at the end of that machine drive to retrieve it.

Soil and groundwater conditions and the depth of the shafts would require specialized shaft construction methods. The launching and retrieval shafts would require sturdy ground support and bottom plugs to provide stable excavations, avoid ground loss and settlement of the ground around the shafts, and prevent invert blowouts. Potential construction methods include overlapping concrete caisson walls, panel walls, jet-grout column walls, secant piles walls, slurry walls, precast sunken caissons, and potentially other technologies. Shaft bottoms would need to be stabilized to resist uplift associated with external hydrostatic pressures, both during excavation and operation. Considering the ground conditions, it may be necessary to treat the shaft area continuously from the surface to the bottom of the shaft to control blowouts. Concrete working slabs capable of withstanding uplift would be required at all shaft locations to provide a stable bottom and a suitable working environment.

The launching and retrieving shafts would need ground stabilization outside of the shafts to reduce groundwater inflows and ground loss during machine launching and retrieval. Jet-grouting, deep soil mixing, and permeation grouting should all be considered during preliminary design.

TBM Assembly in the Launching Shaft

Shaft Casting and Backfilling

After construction of the tunnels, the launching and retrieval shafts would be backfilled around steel pipes, formed concrete pipes, or cast against reusable forms of the required diameter.

Due to the unusually deep nature of the shafts to be constructed for the project, structural stability of the shafts would be critical and Cal/OSHA may require design submittal and approval of this feature.

11.2.8 Tunnel Muck Disposal

The estimated volume to be excavated from the tunnels and shafts is approximately 1.5 million cy. Treatment and re-use options are being investigated. The muck would be plasticized with EPB foam or soil conditioner during excavation and would be saturated when removed from the tunnels. The muck would not be acceptable for use as engineered fill, but it could potentially be dried out and used as agricultural backfill.

11.3 Maintenance Considerations

Maintenance requirements for the tunnels have not been finalized. Some of the critical considerations in terms of maintenance would include determining if the tunnels need to be taken out of service for inspection, and, if so, how frequently this would be required. In addition, the equipment that the facility owner needs to put into the tunnel for maintenance needs to be determined so that the size of the tunnel access structures can be set. Equipment such as trolleys, boats, harnesses, camera equipment, or communication equipment would need to be described prior to finalizing shaft design.

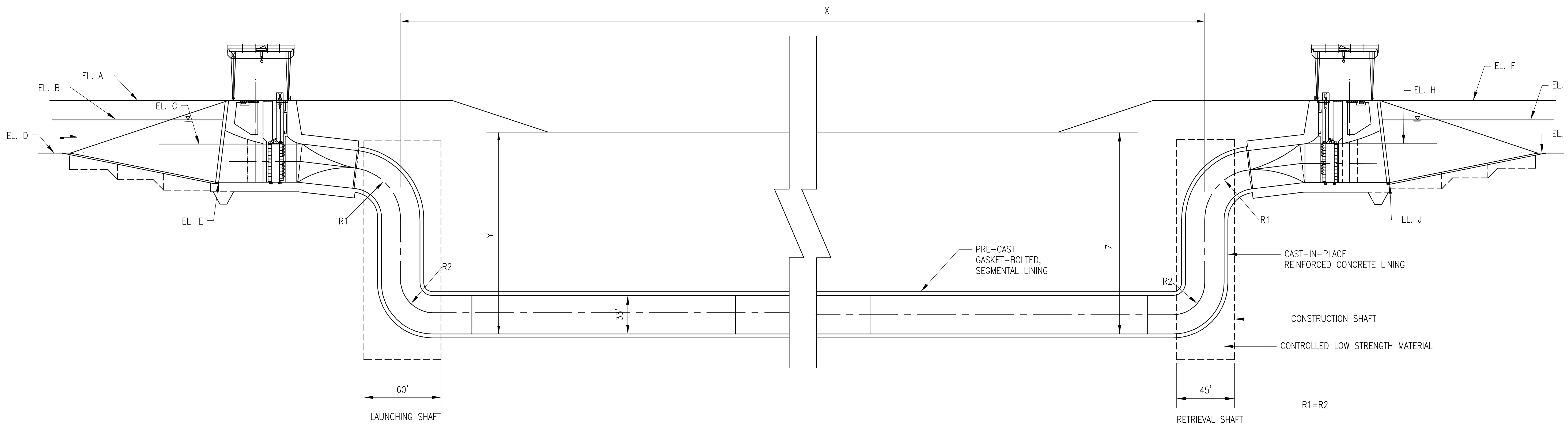
11.4 Design Criteria

11.5 Engineering analysis

During preliminary design, engineering analyses would be required to confirm the feasibility of the selected vertical and horizontal alignments and shaft locations, and the construction methodology selected for the tunnel and shafts.

Recommended engineering analyses include (but are not limited to):

- Anticipated geotechnical conditions
- Anticipated ground behavior
- Earth and groundwater loads on tunnel support
- Lateral earth pressures for shaft design
- Shaft bottom stability
- Settlement calculations
- Evaluation of potential impacts to adjacent structures and levees
- Groundwater treatment and disposal



LOST SLOUGH/MOKELUMNE RIVER		
X	ft.	5,022
Y	ft.	160
Z	ft.	160
A	ft NAVD 88	31.4
B	ft NAVD 88	13.0
C	ft NAVD 88	−3.88
D	ft NAVD 88	−10.48
E	ft NAVD 88	−33.88
F	ft NAVD 88	26
G	ft NAVD 88	7.8
H	ft NAVD 88	−9.06
I	ft NAVD 88	−15.66
J	ft NAVD 88	−39.06

SAN JOAQUIN RIVER		
X	ft.	18,340
Y	ft.	160
Z	ft.	160
A	ft NAVD 88	N/A *
B	ft NAVD 88	N/A *
C	ft NAVD 88	−18.6
D	ft NAVD 88	N/A *
E	ft NAVD 88	N/A *
F	ft NAVD 88	N/A *
G	ft NAVD 88	N/A *
H	ft NAVD 88	−22.52
I	ft NAVD 88	N/A *
J	ft NAVD 88	N/A *

* SHAFT CONNECT TO PIPELINE U/S

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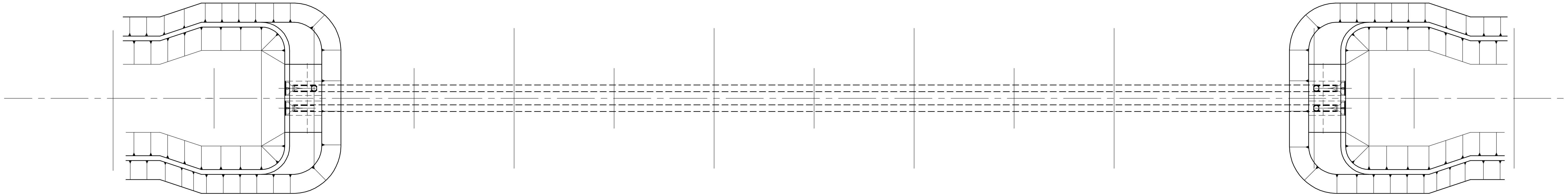
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH DELTA FACILITY OPTION

Tunnel Profile

FIGURE
11-1



PLAN (TYP)
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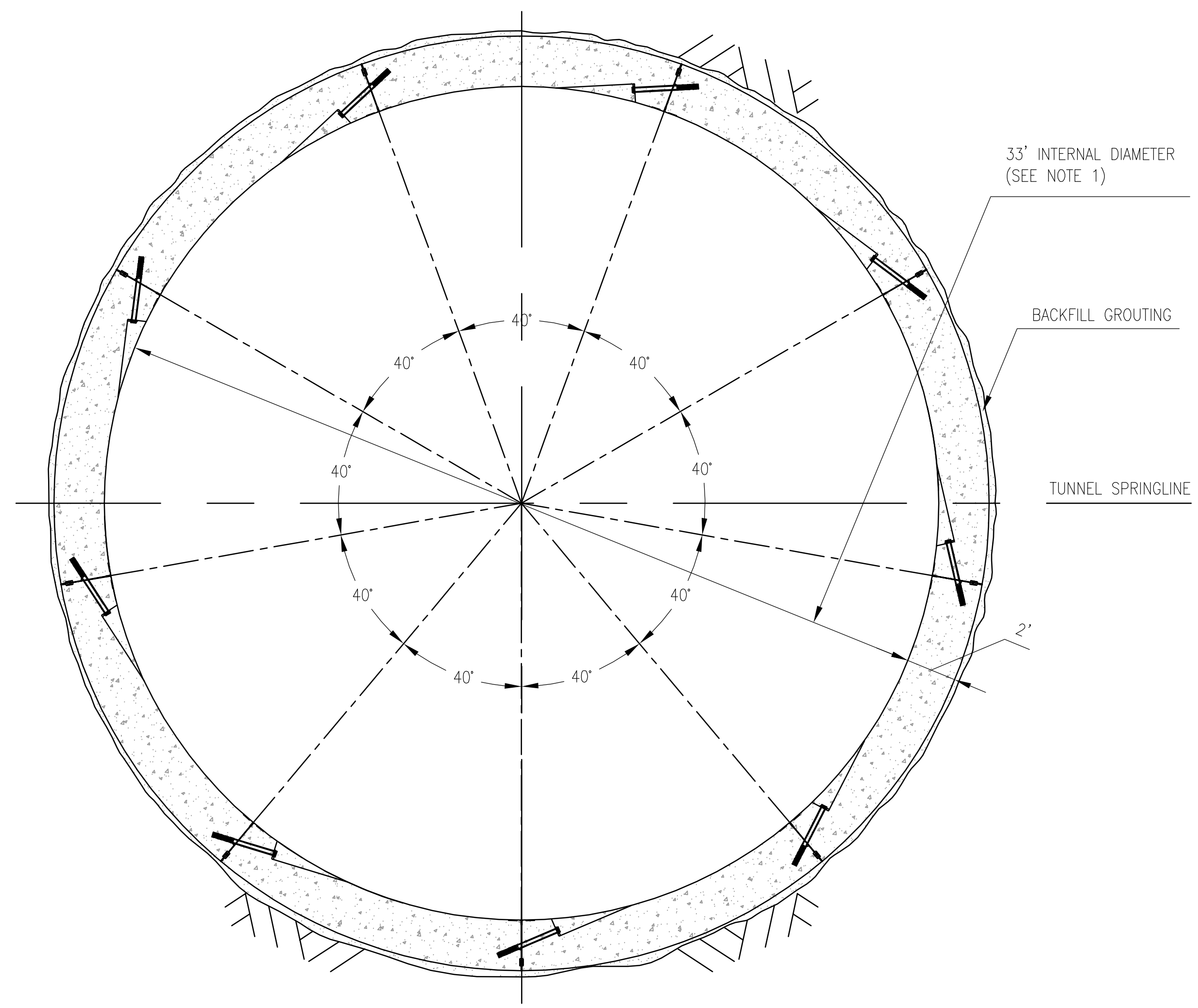

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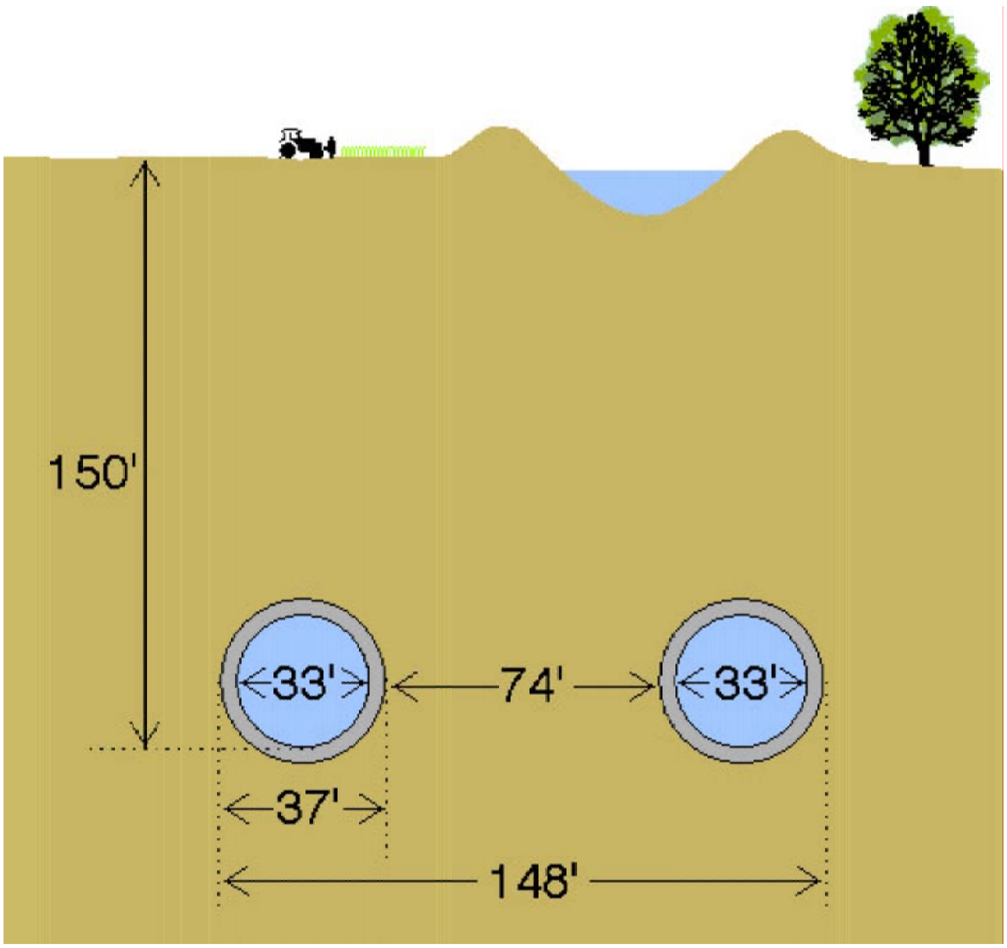
California Department of Water Resources **Advancing the Bay Delta Conservation Plan** **Delta Habitat Conservation & Conveyance Program** DHCCP Team	CONCEPTUAL ENGINEERING REPORT THROUGH DELTA FACILITY OPTION **Tunnel Plan and Section**	FIGURE **11-2**



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California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

FIGURE
11-3

Tunnel Configuration

12.0 BRIDGES – ROAD AND RAILROAD

The TDF Option would intersect several public roadways and one railroad, requiring bridges to be introduced at most of these locations to maintain connectivity across the canal. Bridges over existing channels would require modification where channels and levees are modified.

12.1 Description and Site Plan

Intersections or conflicts with public roadways would require bridges at multiple locations to maintain roadway connectivity. New bridges would be required where local roads are intersected by the proposed canal, and modification or replacement of several existing bridges would be required where they cross existing waterways where levees would be modified. An assessment of the cumulative impacts to private roads, driveways, and farm access roads should be conducted as part of this project.

The alignment crosses a railroad corridor at the active BNSF railroad in Contra Costa County where it crosses Middle River east of the City of Brentwood. A new bridge would be required across Middle River for the BNSF tracks to accommodate the new setback levees at this location.

12.1.1 Canal Crossings

Table 12-1 lists the public roads and railway corridors intersected by the proposed canal, the approximate roadway design speed in miles per hour (mph), average daily traffic (ADT), and a commentary regarding the proposed disposition at each location. Considerable coordination with affected local agencies remains to be done to further evaluate current and future traffic patterns and land use in support of decisions regarding bridge locations and roadway connectivity.

Table 12-1: Bridge Crossings Associated with the Supplemental Intake Canal

Facility Intersected	Design Speed (mph)	ADT	Comments
Lambert Road	45		Bridge required.
Dierssen Road	35		To be determined. Affects only three properties by increasing their westbound trip distance by 3 miles.
Twin Cities Road	55	4,900	Bridge required.
West Lauffer Road	35		To be determined. Very limited traffic as road serves few properties. Road may be abandoned or realigned and new crossing combined with North Vail Road or W. Barber Road.
West Walnut Grove Road	45	3,200	Bridge required.

ADT = average daily traffic
mph = miles per hour
N/A = not applicable
UPRR = Union Pacific Railroad

The construction of intakes and the associated pumping plants would impact levee roads along the Sacramento River at the two intake sites south of the town of Hood. Traffic on the levee road (SR 160) would be detoured around this construction area as shown in Section 7.0. Once construction is finished, the levee road would be restored back to its original locations.

12.1.2 Through-Delta Channel Crossings

The improvements in the through-Delta corridor may include levee armoring or retrofitting, channel dredging, and installation of setback levees. These measures would impact the existing bridges crossing these waterways. The bridges subject to impact are included in Table 12-2 which lists the fate of each bridge as determined thus far. Considerable coordination with owner agencies and regulatory agencies remains to be done to better define and evaluate the needs and alternatives available at each crossing.

In addition to the bridges affected that cross the TDF Option, there are several levee roads that would be affected when setback levees are installed. Most would simply be relocated onto the new levee and interruption to traffic can be minimized. Most do not see much public traffic.

12.1.3 Roadway Crossings

The typical bridge and roadway plan shown on Figure 12-1 depicts typical bridge and approach roadway profiles for all roadway bridges across the canal. Bridge type is assumed to be CIP or precast concrete superstructures supported on concrete pier walls and abutments, all founded on pile foundations. Preliminary span lengths are based on a maximum 145-foot length corresponding

Table 12-2: Bridge Crossings Associated with Through-Delta Corridor

Facility	Disposition/Comments
West Walnut Grove Road bridge over South Fork of the Mokelumne River (45 mph)	Bridge replacement is required. New bridge would need to be longer and higher to span to new western setback levee. New Hope Resort access roads may need modification to meet new grade of road.
SR 12 bridge over Little Potato Slough	Bridge over Little Potato Slough is assumed to be salvaged but would require modifications and/or replacement of up to three west side columns to accommodate setback levee and channel widening. Access to bridge control house would also need to be modified. Projected ADT on SR 12 for year 2015 is 22,900 vehicles.
Bacon Island Road bridge over Middle River	This existing bridge is proposed to remain in place and operable. Access roads along existing levees (South and West Bacon Island Roads) would be relocated onto the new setback levees. Existing levees are to remain. New access roads down to the bridge would be necessary from the setback levees. Current ADT estimated at 900 vehicles.
BNSF bridge over Middle River	A new railroad bridge is proposed to be constructed on a new alignment to the north. This new segment of bridge would be approximately 1.8 miles long and would tie back in to the existing alignment east of the existing bascule bridge over Old River. A new movable span similar to the existing, and aligned with the existing navigable waterway is proposed to maintain navigable access. Train traffic would be maintained on the existing structure during construction.

Facility	Disposition/Comments
SR 4 bridge over Middle River	Existing bridge is to be replaced with a new bridge to the north of existing alignment that improves the horizontal alignment and provides a vertical profile to improve navigational clearance under the bridge to approximately 40 feet clear. Projected ADT on SR 4 for year 2015 is 8,300 vehicles. Design speed would be 65 mph.

mph = miles per hour
ADT = average daily traffic
BNSF = Burlington Northern and Santa Fe (Railroad)
SR = State Route

to a practical limit for transportation of precast girders. A three-span configuration would suffice for all the new canal bridge crossings. Span lengths would vary depending on the angle at which the canal crosses the road. Preliminary hydraulic analysis has revealed that hydraulic head loss is relatively minor at bridges. For this reason, attempts to eliminate a pier in the water through the design of longer and deeper spans does not appear warranted.

The bridge supports are anticipated to be slender, 30-inch-thick concrete pier walls. The low chord of the bridge would be set to provide the prescribed FB above maximum WSE in the canal. The elevation of the top of the canal protective berm, however, may be governed by a different FB criterion that also includes flood protection and SLR. The top of bridge deck would always be at least as high as the top of the levee so that no breach is created.

Bridge widths and approach roadway design would meet the design criteria as outlined in the ***American Association of State Highway Transportation Officials (AASHTO) Geometric Design of Highways and Streets*** (2004). State bridges would also meet requirements in the Caltrans Highway Design Manual. Bridge widths would in general match approach roadway widths including shoulders. If the roadway owner agency identifies a plan to widen the roadway, at a minimum, the foundations and substructure elements of the bridge should be built to accommodate this eventuality to prevent intrusive construction in the canal in the future.

The length and overall footprint of the approach roadway would vary at each bridge location, dictated primarily by the height of the canal levee relative to the existing roadway. Another factor influencing the approach roadway footprint is the need to maintain continuity for levee maintenance roads which have to loop around the ends of bridges and bridge approach guard railings creating a bulge in the canal protective berm.

The bridge crossings required over the through-Delta waterways require a unique solution to accommodate the proposed waterway improvements while maintaining existing traffic**.** Efforts would continue to be made to modify conveyance designs in order to minimize impacts to existing bridges. Several factors would need to be taken into consideration in arriving at final solutions for these locations including the following:

- Navigational clearance requirements that would dictate span lengths, fender protection, bridge profiles, and/or the need for movable spans.
- Marine considerations including scour depths, salinity, water surface fluctuations, tidal influence, etc.

- Traffic considerations and detour/staging options.
- Regulatory agency permit requirements and restrictions.

The existing movable roadway bridges that would be affected include the SR 12 bridge, Bacon Island Road bridge and the SR 4 bridge. All are swing span structures although the SR 4 bridge has been inoperable for some time. Any action that would affect the status of the navigable waterway at these locations is subject to the approval of 11th Coast Guard District who has jurisdiction over these waterways. Although it is possible that a new Coast Guard permit could be obtained for any of these bridges that eliminates the need for the bridge to be movable, it is considered unlikely that commercial and recreational boating interests would allow this reduction in access. In addition, maintenance of the new levees requires barge and tug boat access in the waterways which require certain vertical clearances.

The available clearance currently under the SR 4 bridge is limited and the bridge can no longer open. For this reason a higher fixed bridge with a profile that provides approximately 40 feet of navigational clearance is expected to be acceptable at this location.

The current conceptual design allows the Bacon Island Road bridge to remain in place and operable.

The conceptual design of the SR 12 bridge includes only modifications to the existing bridge that do not affect the swing span.

12.1.4 Railroad Crossings

Criteria for bridge design could conform to American Railway Engineers and Maintenance of Way Association criteria. Railroads typically desire to have structures built to accommodate a future second track and room for maintenance vehicles to cross, which results in a bridge width of 75 feet. Railroad bridges are typically constructed of simple spans that can be removed and replaced quickly to minimize disruption to service. Spans are generally shorter for the same reason. As with roadway bridges, the foundations and substructure elements for this future bridge widening should be incorporated, at a minimum, in the original construction to avoid intrusive construction in the waterway in the future.

The existing BNSF crossing over Middle River includes a 110-foot long, single-leaf bascule lift span over the navigable waterway portion of Middle River, with 15 approach spans to the east and a series of precast concrete trestle spans stretching west to Old River where the same type bascule bridge is installed. BNSF has reported that traffic for this particular stretch of track varies but current estimated volumes include 62 trains per day, which includes 50 freight trains and 12 passenger trains. The freight trains are expected to reach speeds of 70 mph and the passenger trains would reach 80 mph. Service could be maintained on this track.

The elevation of the proposed setback levees is approximately 7 feet higher than the existing tracks. For this reason it is assumed that the bascule bridge span and the approach spans would require replacement on a new alignment adjacent to, and north of the existing alignment, in order to match the higher elevation of the proposed eastern setback levee and accommodate the proposed operable barriers across Railroad Cut. Train traffic would be maintained on the existing bridge during construction until the tie-ins for the new alignment are made at either end. A double track bridge is assumed to be necessary at a minimum for this new portion of bridge including the lift span.

12.2 Construction Methodology

Construction methodology for bridges over new canals would involve typical materials and bridge/roadway construction techniques. The construction of the bridge structures and the disturbance caused by it including excavation, pile driving for foundations, and stockpiling of materials, would all probably occur within the overall footprint of the proposed canal construction. activities unique to bridge construction that should be noted, including:

- **Deep Foundation Construction.** The bridge piers and abutments are anticipated to be founded on driven pile foundations typically installed with diesel hammer pile-driving rigs. The pile caps (footings) are to be constructed below the final canal grade with abutments founded in the levee embankments. Because scour depths in the canal are minimal, footings can be placed relatively shallow.
- **Superstructure Type.** It is anticipated that the bridge superstructures—or main load carrying members—would be comprised either of CIP concrete, precast concrete girders or steel girders. The ability to prefabricate these members ahead of time would expedite construction and allow more flexibility in sequencing.
- **Placement of Concrete.** While bridge superstructure material may vary, it all substructure elements would be comprised of CIP concrete. Because groundwater levels along the alignment are relatively shallow, dewatering may be required to place concrete for pier pile caps (footings). Depending on the depth below groundwater, this can be accomplished through the use of wells or sealed cofferdams. Water removed from the ground or from inside cofferdams is considered to be processed water and would need to be dealt with according to current NPDES and RWQCB regulations prior to discharge.

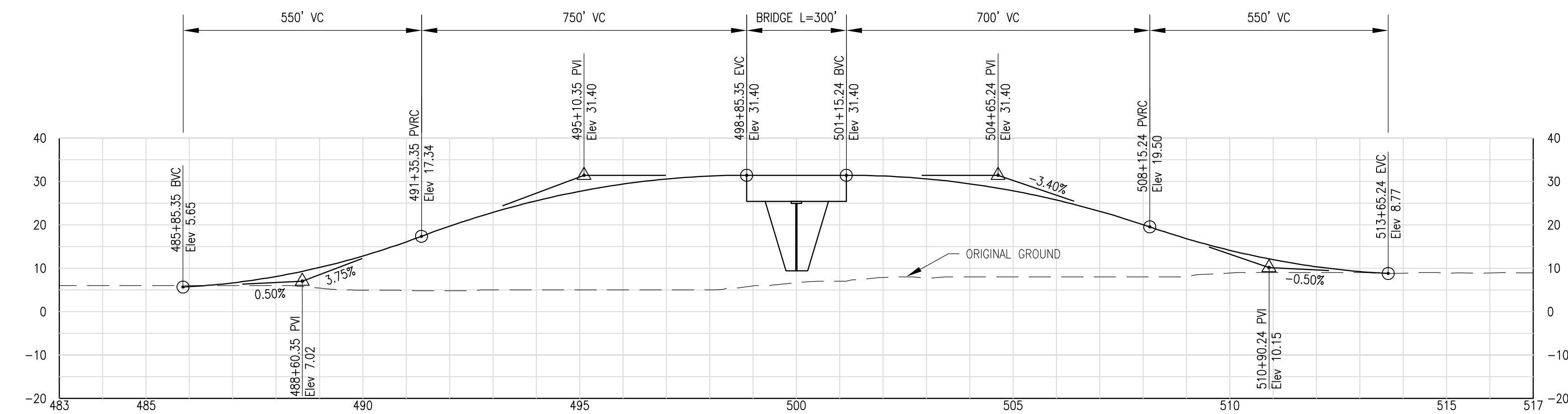
  Equipment to be used includes cranes, pile driving hammers, concrete trucks, and concrete pumps. Existing roadways would be used for delivering materials, which would be stockpiled within the canal footprint.

  Unlike new bridges over the new canal, the replacement structures over the existing Delta waterways each would be a unique bridge designed to meet unique site conditions and constraints. Construction of new bridges across the waterways would follow typical bridge construction techniques and methodologies mentioned above for canal bridges applied in a marine environment. This work in and above waterways would be subject to permit restrictions and mitigation measures imposed by the regulatory agencies issuing permits to construct. Foundations and columns placed within the waterway would be built within either cofferdams or large diameter casings anchored below river bottom and extending above water level essentially sealing off and isolating the work from the waterway. Temporary obstruction or restriction to navigation and recreational boating in the waterway may be unavoidable as a result of working platforms (trestles) extending into the waterway and/or overhead falsework

- **Staging and Traffic Management.** For new bridges across canals, the general sequence of construction can follow one of two basic schemes: either the bridge would be built first followed by canal construction, or the canal would be constructed first and the bridge built across it. It is easier to build a bridge over a finished canal, because most of the pier foundation excavation is complete, as well as the abutment embankment (levee). This sequence is possible where roadway(s) can be closed to traffic entirely, allowing canal construction to proceed uninterrupted. Where traffic needs to be maintained on a given roadway, canal and bridge construction would need to be coordinated and staged to allow traffic to be maintained on the existing or temporarily realigned roadway until the new bridge is ready for traffic. A traffic management plan would need to be developed for each crossing, or series of crossings, to arrive at a staging and detour scheme that minimizes traffic disruption. Detour routes would undoubtedly need to be planned and built to serve traffic for a considerable amount of time given the scale of construction and the time required for settlement periods for roadway approach embankments and canal embankments.

For the new bridges across canals, temporary alignment of the roadway in the vicinity of construction has been assumed with the new bridge built on existing alignment. Once the new bridge is finished, traffic is shifted onto it, and the temporary roadway would be removed as part of canal excavation.

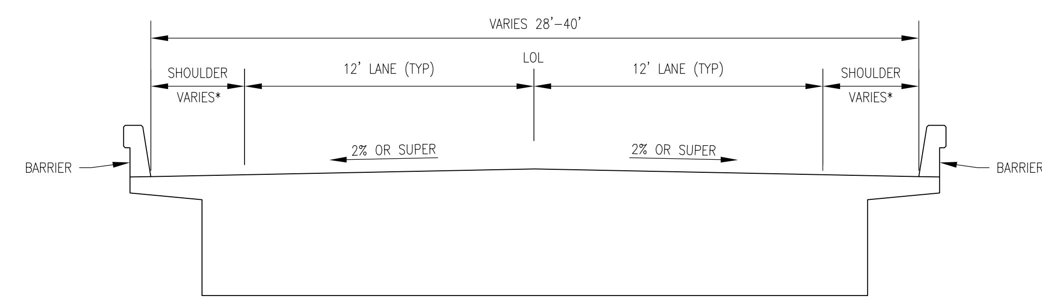
Maintaining traffic on existing routes during construction of the proposed new bridges over existing waterways at West Walnut Grove Road bridge, the BNSF bridge at Middle River and the SR 4 bridge would be accomplished by realigning the new structure so that traffic can be maintained on the existing one. At the Potato Slough bridge on SR 12 at Terminous, the proposed new setback levee to the east would require some temporary traffic realignment off the east end of bridge to allow for adjustment of the profile of Highway 12 to ramp up and over the new eastern setback levee.



PROFILE

SCALE: 1"=200'H, 1"=20'V

FACILITY INTERSECTED	DESIGN SPEED (MPH)	ADT
LAMBERT ROAD	45	3100
DIERSSEN ROAD	35	TBD
TWIN CITIES ROAD	55	4900
WEST LAUFFER ROAD	35	TBD
WEST WALNUT GROVE ROAD	45	3200



BRIDGE SECTION

NO SCALE

*STATE ROUTE SHOULDER = 8'
*LOCAL ROAD SHOULDER = 4'
*MINOR LOCAL ROAD SHOULDER = 2'

California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Bridge Profile and Section (Typical)

FIGURE **12-1**

13.0 UTILITY AND INFRASTRUCTURE CROSSINGS

The preliminary alignment of the TDF Option crosses various utilities. These include power transmission lines, communication transmission lines, buried gas transmission lines, agricultural irrigation water deliveries and drainage ditches, and potable water delivery systems. This section identifies the major utility crossings, for example East Bay Municipal Utility District's (EBMUD) Mokelumne Aqueducts and regional gas and electric transmission lines.

Smaller electrical power distribution lines and communication lines that require field verification and coordination with utility providers would be identified in later engineering phases.

Because several natural gas fields are present along the alignment, natural gas wells are included in the utility and infrastructure inventory. Building infrastructure within the proposed ROW is also addressed in this section. However, road and rail infrastructure crossings are discussed in Section 12.0.

This report provides a preliminary identification of utility crossings. Additional water supply impacts to existing and proposed future facilities of water users are anticipated. The extent of these impacts would be evaluated through the environmental program.

13.1 Utility and Other Infrastructure Crossings

Existing utilities and other infrastructure are located within the preliminary alignment of the TDF Option. These can be classified as linear features or distinct point features. Linear utility features include the following: power/electric transmission lines natural gas transmission lines, water transmission lines, sewer transmission lines, and communication transmission lines. Distinct point utility features include oil wells and natural gas wells. Other infrastructure features can include buildings and agricultural delivery canals and drainage ditches. This section identifies the major utility crossings that can be identified from existing aerial photography, USGS maps, and other sources. Field checking would be required in the future to verify the information provided herein.

In addition to the major utility crossings, there may also be small or minor utility crossings, such as low-voltage power/electric distribution lines and communication lines, which would require field verification and coordination with the utility providers. These would be identified in later design phases.

13.2 Description of Utilities and Other Infrastructures

13.2.1 Inventory

A preliminary identification of utility crossings, oil/natural gas wells, and structures was made by overlaying DWR, DRMS, USGS, and Reclamation geographical layers on 2006 high-resolution aerial photos.

The preliminary TDF Option alignment would cross or interfere with:

- Two overhead power/electric lines;
- Fifteen natural gas pipeline crossings;
- EBMUD Mokelumne Aqueduct;
- Thirty-one natural gas wells (5 active and 26 inactive);
- Fifty-three areas with structures, and
- Three hundred and seventy agricultural delivery canals and drainage ditches

Local electrical distribution lines and telephone/communication lines are present along the preliminary alignment but are not included in the above list.

Each feature or utility would require mitigation attention. Avoiding electrical transmission lines during design, crossing over or under existing pipelines (revising elevations where necessary), relocating/re-routing existing utilities, capping abandoned wells and revising drainage routing would be considered. Final measures would be determined on a case-by-case basis and would consider applicable state, county, city, and local agencies requirements. Additional discussion follows.

13.2.2 Utilities

13.2.2.1 Power/Electric Lines

The preliminary alignment of the TDF Option crosses two existing power/electric lines: a 115-kV transmission line and a 500-kV transmission line. In addition, numerous electrical distribution lines along most roads would require relocation.

13.2.2.2 Natural Gas Pipelines

Fifteen natural gas transmission lines were identified to be within the TDF Option alignment. Two of the natural gas pipelines are located within the Supplemental Intake Canal portion of the TDF Option and 13 are located where setback levees are proposed. Various purveyors operate and maintain these pipelines whose input is necessary to determine appropriate mitigation measures. The pipeline locations would need to be field verified.

13.2.2.3 Water

Three pipelines (Mokelumne Aqueduct Nos. 1, 2 and 3), owned by East Bay Municipal Utilities District, were constructed in 1929, 1949, and 1963, respectively. The pipelines cross the Delta predominantly through Jones Tract – Upper and Lower, Woodward Island, Orwood Tract, and Bixler Tract. The pipelines are located aboveground on top of saddle pipe supports on both sides of Middle Slough and below ground across Middle Slough.

13.2.2.4 Sewer

Sewer pipelines were not identified within the TDF Option alignment.

13.2.3 Oil and Natural Gas Wells

No oil wells were identified within the TDF Option alignment.

Thirty-one natural gas wells were identified within the ROW of the conveyance. Six of these wells are located in the Supplemental Intake Canal portion of the TDF Option, and 25 are located where setback levees are proposed. The six wells located within the Supplemental Intake Canal are identified as plugged and abandoned. The remaining 25 natural gas wells are located within the setback levee reaches of the conveyance facilities and all but 5 of these wells have been plugged and abandoned.

13.2.4 Structures

Fifty-three areas with structures were identified to be within the TDF Option alignment. These areas contain houses, out buildings, pump houses, and/or silos. Many locations contain multiple structures.

13.2.5 Agricultural Delivery Canals and Drainage Ditches

Three hundred and seventy agricultural delivery canals and drainage ditches were identified to be within the TDF Option alignment. Although the agricultural delivery canals and drainage ditches were identified using high definition aerials and USGS quadrangle maps, at times the resolution of the aerials or maps were not clear, and access to the property not available so the classification of the crossings as either an agricultural delivery canal or drainage ditch and their direction of flow would need to be determined and verified.

A field investigation of Twitchell and Sherman Islands revealed that irrigation and drainage systems are not always operated in the same manner; some manage the groundwater and others use surface water for irrigation. As a result, the systems may include siphons, pumping stations, delivery and drainage canals, and subsurface tile lines. Mitigating these crossings would require a variety of solutions.

13.3 Construction Methodology

13.3.1 General

Utility crossings would be designed and constructed in accordance with the applicable state, county, local and district standards, and the requirements of the utility owner (Pacific Gas and Electric [PG&E] for example). Existing buried pressure pipelines would be relocated either over or under the conveyance channel section as necessary and consistent with the needs of the local agency and/or landowner. Existing gravity pipelines, siphons, and pumping plants would be modified or replaced to maintain existing flows.

13.3.2 Utilities

13.3.2.1 Power/Electric Lines

The proposed alignment options of the conveyance have been altered to minimize conflicts with power/electric transmission lines. For aboveground power/electric distribution lines, the power poles would be relocated. Every effort would be made to adjust the alignments of the conveyance facilities to avoid impacts but relocation of power poles and towers may be required. Determination of whether poles or towers need to be relocated would require a field survey. Where the alignment crosses under 500-kV transmission lines, it is likely that use of the electrically powered draglines for canal excavation would be discontinued and conventional diesel-powered excavating equipment used instead.

13.3.2.2 Natural Gas Pipelines

Natural gas pipelines would be relocated as necessary and coordinated with the utility company. The facilities would be constructed either over or under these existing pipelines. The choice would depend on the depth and size of the existing pipelines. If the existing pipelines are large it might be more economical to place the conveyance facilities below the existing pipelines. The decision would be made as more site specific information becomes available.

13.3.2.3 Water

Recent DWR discussions with EBMUD revealed they do not want their existing pipelines going under the conveyance facilities. Therefore, the Mokelumne Aqueducts would be bridged over the conveyance canal. The bridge would be constructed south of the existing aqueduct alignment and tied-back into the existing pipelines on both east and west sides of the canal ROW. For construction footprint purposes, a 300-foot-wide corridor south of the existing pipeline alignment is assumed.

13.3.3 Oil and Natural Gas Wells

Management of the abandoned natural gas wells would depend on a number of factors which include whether the well is located within the permanent or temporary ROW of the TDF Option. For abandoned natural gas wells located within the permanent ROW, the top of the well would be capped off 10 feet below the bottom of the canal. For abandoned natural gas wells located within the temporary ROW, including haul roads and staging areas, the top of the wells would be excavated and capped off at a depth that would be appropriate to the site conditions and the well location.

Before any work, abandoned wells would need to be tested to confirm that it was properly abandoned. Wells not properly abandon may need to be completely re-abandoned. All abandonments would be in accordance with State of California, Division of Oil and Gas requirements.

13.3.4 Agricultural Delivery Canals and Drainage Ditches

Agricultural water delivery canals and drainage ditches that are interrupted by the conveyance facilities would be modified, or new facilities would be constructed to preserve the intended use of such facility. Such facilities would be designed to satisfy the O&M needs of the farming industry. To determine what is needed would require a better understanding of these agricultural systems. This knowledge would be obtained from local farmers, landowners, governing agencies, and surveys of the delivery and drainage systems. Potential planned measures may include, but are not limited to:

- New or modified irrigation pumping plants
- Extended delivery pipes
- New or modified drainage ditches
- New or modified drainage pumping plants

In some cases, the existing agricultural facilities can be abandoned, such as fields where the irrigation delivery canal and/or drainage ditch ends within the field itself. Additionally, it is possible that several existing water delivery or drainage systems could be combined. The actual mitigation would be determined on a site-specific basis and would be consistent with good practice, owner needs, and local agency requirements, as appropriate.

14.0 FOREBAYS

No new forebays are proposed for the TDF Option.

15.0 LEVEES

Delta levees are the most extensive constituent feature of the current system for conveying water through the Delta to the diversion pumping facilities at CCF. The levees confine water flows to the sloughs, rivers, and channel cuts that comprise the Delta, preventing the flows from spilling out over the adjacent island tracts, which are typically situated below sea level. The existing levees are vulnerable to failure and many Delta islands have flooded repeatedly.

The proposed TDF Option necessarily relies upon the Delta slough and river network as the primary mode of conveyance. As such, levees remain the primary constituent of the proposed conveyance system. Therefore, strengthening of existing levees and construction of new, more reliable levees is the most extensive aspect of construction of the TDF.

Whether to strengthen existing levees or to construct new levees depends on multiple considerations including levee reliability, design flood level elevations, environmental impacts and opportunities, construction feasibility, and other concerns. For most areas of the TDF Option, it is preferable to construct a new levee, set back behind the existing levee, instead of strengthening the existing levee. Such setback levees constitute the majority of levees that are proposed for the TDF Option. Several configurations of setback levees are proposed, as well as several approaches to modifications of existing levees.

15.1 Description and Site Plan

Most of the alignment length of the TDF Option is along existing Delta waterways. The TDF Option includes new levees or strengthened existing levees along the entire length of this part of the alignment. Generally, new levees or strengthened existing levees are situated on both sides of the waterways that comprise the alignment.

The primary purpose of these levees is to safeguard the conveyance of water through the Delta waterways, consistent with the goal of ensuring water supply reliability. The levees safeguard the conveyance in two ways. First, the levees contain conveyance water within the facility. While existing levees can overtop or breach and allow conveyance water to spill out of the facility, the new levees and strengthened existing levees would be designed to prevent overtopping or breaching. Second, the levees prevent salt water-contaminated water from intruding into the existing conveyance. While salt water that may flood Delta island tracts may also enter the existing conveyance waterways through breached existing levees, the new levees and strengthened existing levees would keep out salt water from inundated island tracts.

As can be seen in Figures 15-1 and 15-2, the levees form a nearly contiguous boundary that contains the water within the conveyance and also isolates the conveyance water from salt water intrusion. The only gaps in this levee boundary are located at river forks or confluences, where a side waterway connects to the conveyance waterway. At these locations, an operable barrier spans across the side waterway (refer to Section 17.0 for further description of operable barriers). The operable barrier connects to the levee on either side of the waterway, making the levee-barrier-levee boundary completely contiguous.

During normal operations, operable barriers would be open, allowing water to pass into and out of the side waterways. For these conditions, water moves through the Delta much as it does now

– conveyance water is not confined to only the TDF alignment waterways (refer to Sections 4.0 and 5.0 for further description of current conveyance conditions). During these conditions, the new levees and strengthened existing levees improve water supply reliability by reducing the likelihood of water losses due to levee failures along those parts of the conveyance.

During emergency operating conditions, such as after a significant earthquake, the operable barriers are closed. During these conditions, water does not pass into and out of the side waterways. Rather, the conveyance water is isolated within the TDF conveyance alignment, and the operable barriers and levees form a contiguous boundary between conveyance water and potentially intruding salt water.

The ability to create a contiguous boundary along the length of the alignment is key to ensuring water supply reliability. During emergency operating conditions such as after a significant earthquake, substantial salt water intrusion into the Delta is expected to occur. Many island tracts, including those adjacent to the TDF conveyance waterways, would be flooded with salt water. This condition is expected to occur throughout the southern and central areas of the Delta, and potentially into the northern part of the Delta. For reliable water supply, the TDF conveyance could be kept separated from the intruding salt water. Therefore, a contiguous boundary would be put in place along the entire length of alignment subject to adjacent flooding by salt water. This alignment area includes the entire southern and central areas of the Delta, extending as far north as Staten Island.

Additionally, for reliable water supply during such conditions, conveyance water must be prevented from leaking out of the TDF conveyance alignment. Current conveyance routes mostly depart from the TDF conveyance alignment in the northern part of the Delta and travel along routes more westerly within the Delta. During conditions where the southern and central areas of the Delta are intruded by salt water, conveyance water following current conveyance routes would pass through these salt water-intruded areas. For reliable water supply, conveyance water must be kept within the TDF conveyance. Therefore, a contiguous boundary would be put in place along the entire length of alignment subject to water leaking out of the conveyance. This includes the entire northern and central areas of the Delta, extending as far south as Mandeville Island.

For these two reasons – keeping conveyance water within the conveyance and keeping salt water out during emergency events – a contiguous boundary of levees and operable barriers is necessary along the entire length of the alignment. The resulting levee layouts are shown in Figure 15-1. Table 15-1 summarizes the lengths of levees by island tract along the alignment.

Table 15-1: New and Strengthened Levee Locations and Lengths

Location	Length (miles)	Location	Length (miles)
Bacon Island	5.2	Middle Roberts Island	0.3
Bouldin Island	3.9	New Hope	3.4
Brack Tract	2.0	Staten Island	11.4
Canal Ranch	2.4	Terminous Tract 2	6.4
Dead Horse Island	0.5	Union Island	3.1
Empire Tract	5.0	Venice Island	4.0

Location	Length (miles)	Location	Length (miles)
Jones Tract	8.0	Victoria Canal	6.2
Libby McNeil Tract 1	0.7	Walnut Grove	1.4
Mandeville Island	2.1	Woodward Island	2.6
McDonald Tract	6.3		
		TOTAL	**78.0**

As stated above, the primary purpose of these proposed levee improvements would be to safeguard the conveyance of water through the Delta waterways. The proposed levee improvements would keep fresh water within the conveyance and keep saline floodwaters out. The levees would also be designed to withstand the 200-year flood-level condition (as described in Section 3.5) in order to protect themselves (i.e., the conveyance facilities) from significant damage or destruction during such an event, not to provide flood protection to adjacent lands.

Where feasible, the levees would be designed to improve the Delta ecosystem by restoring a variety of riparian and aquatic habitats. Certain types of levees to address this goal (in addition to conveyance reliability) are situated at various locations along the TDF conveyance alignment. These types of levees are described in the following sections.

15.1.1 Types of Levees

Levees for the TDF are categorized into the following types:

- Adjacent Setback Levee
- On-Channel Levee at Widened Existing Channel
- On-Channel Levee at New Channel
- Offset Setback Levee with Notching of Existing Levee
- Offset Setback Levee with No Alteration of Existing Levee
- Strengthened Existing Levee
- Facilities Protection Levee

The following paragraphs describe the levee types referred to in this report are not intended to describe levee types or other projects or other applications.

Adjacent Setback Levee

An Adjacent Setback Levee is a new levee constructed adjacent to, along, and generally parallel to the landward side (i.e., landside) of an existing levee, with only 50 feet between the two levees. The existing levee is not removed, nor is modification performed on the side that is situated toward the waterway (i.e., waterside). As used herein, the term “waterside” refers to the side that is closest to the waterway, even though this side may not be (and in the case of an Adjacent Setback Levee is not) directly in contact with or adjacent to the waterway itself. A typical section of an Adjacent Setback Levee is shown on Figure 15-3.

On-Channel Levee at Widened Existing Channel

For an On-Channel Levee at Widened Existing Channel, a new setback levee is built and the existing levee is completely removed. The removed levee no longer serves as a lateral constraint to river flows, as water is allowed to pass over the former levee location and extend the river channel laterally out to the new setback levee. The river channel is thus widened by allowing flows to pass over the area between the new setback levee and the removed levee. The new setback levee forms the new channel boundary.

New On-Channel Levee at New Channel

A New On-Channel Levee at New Channel is a new levee constructed as part of development of a new river channel. A new river channel, as opposed to a widened existing channel, is created by constructing a pair of new levees on opposing sides of the new channel alignment. Such levees are not located specifically in relation to the alignments of existing levees (as setback levees are), but are located to create the lateral bounds of the new channel along its planned alignment and at the planned channel width. The area between the pair of new levees becomes the channel, and so is permanently inundated.

Offset Setback Levee with Notching of Existing Levee

An Offset Setback Levee is a new levee constructed at some distance back from the landside of an existing levee, instead of adjacent to the existing levee. An Offset Setback Levee may or may not be aligned parallel to the existing levee, but the setback levee defines an enclosed area between it and the existing levee (typically by each end of the setback levee being connected to the existing levee or to an Adjacent Setback Levee).

For an Offset Setback Levee with Notching of Existing Levee, the existing levee is modified by cutting a notch down through the levee at one or more locations. Each notch is cut deep enough to allow water to pass from the waterway through the notch and into the intra-levee area between the notched existing levee and the new setback levee. Water may pass in and out of the intra-levee area through the notches, filling or partially filling the area. The intra-levee area is not subject to the same river-way flow conditions (e.g., flow rates and related erosive forces) as the waterside of the existing levee. The intra-levee area may or may not be partially filled with soil. For configurations where the intra-levee area is not filled with soil, the intra-levee area is like a pond or flooded island feature when it is inundated. For configurations where the intra-levee area is partially filled with soil, the intra-levee area is like a wetland, overbank-flooding zone, or intertidal zone. A typical section of an Adjacent Setback Levee with Notching of Existing Levee is shown on Figure 15-3.

Offset Setback Levee with No Alteration of Existing Levee

For an Offset Setback Levee with No Alteration of Existing Levee, the existing levee is left intact (i.e., it is not removed, lowered, or substantially modified in configuration). For this setback levee configuration, the area between the new setback levee and the existing levee is also left intact and not modified. With respect to inundation, this intra-levee area remains intact as long as the existing levee does not fail or overtop.

Strengthened Existing Levee

A Strengthened Existing Levee is an existing levee modified substantially to increase its resistance to failure and overtopping. Strengthening of existing levees typically includes modifying the levee configuration by adding materials (e.g., rip-rap) to make it more resistant to erosion, seepage, and slope instability and raising the crest elevation. To do so, typically the landside slope of the existing levee is buried by new soil to expand the levee base and height (as opposed to expanding the levee on the waterside into the river).

Facilities Protection Levee

A Facilities Protection Levee is a new levee constructed around new conveyance facilities for their protection and has no particular spatial relationship to existing levees.

15.1.2 Levee Geometry and Composition

Each type of new levee has the same general levee geometry and composition. Differences in type are due mainly to the new levee's spatial relation to the existing levee and to the disposition of the area between the new and existing levees.

A typical new levee would have a broad-based, generally asymmetrical triangular cross-section. The waterside slope would be inclined at an angle of 3H:1V (approximately 18 degrees). The levee would have a roughly horizontal crest, approximately 20 feet wide. The landside slope would be a compound slope, with the lower portion being flatter than the upper portion. The upper part of the landside slope would be 3H:1V and the lower part 5H:1V (approximately 11 degrees).

Slope angles for the levee configurations are controlled mostly by levee slope stability characteristics. Steeper slopes are more susceptible to stability problems, which can result in levee failure. Safe stability conditions could be achieved, and there is generally not much flexibility in steepening levee slope angles. Flattening of levee slope angles is permissible, but such flattening widens the levee footprint and generally increases impacts of construction.

Some adjustments to slope angle may be possible for the lower part of the levee landside slope. Addition of an engineered drainage layer (i.e., a blanket drain) within the levee tends to decrease instability and allow for an increase in slope angle. This alternative configuration includes a blanket drain and a steeper landside slope is. For this alternative, the landside slope may be steepened to 3H:1V, typically reducing the levee footprint width by approximately 40 to 50 feet.

The levee height, as measured from the adjacent ground surface on the landside and vertically up to the elevation of the levee crest, would range from approximately 20 to 45 feet high, and would commonly be between approximately 30 to 40 feet. Correspondingly, the base width (the footprint) of the levee would range from approximately 180 to 360 feet, and commonly be approximately 250 to 320 feet wide.

Required levee height is controlled predominantly by the design WSEs and required FB, discussed in Section 3.5 of this report. On the waterside of the levee, the levee crest elevation could exceed the WSE associated with the design flood stage within the waterway. Note that for future conditions that account for SLR, settlement of the existing levee, the design-level flooding conditions, and other factors, the existing levee is expected to overtop at most locations, and waterway flood flows could be contained by the new levee. On the landside of the setback levee, the levee crest elevation could exceed the WSE associated with the design inundation level that would result from overtopping or a levee failure elsewhere along the perimeter of the island tract (i.e., a levee located along a different waterway).

The main body of the levee would be composed of compacted lean clayey and/or silty soils. Such soils include mixtures of clay, silt, sand, and/or gravel wherein the predominant material type is lean clay or silt (i.e., the soil mixture has a Unified Soil Classification System [USCS] symbol of CL or ML) or has a substantial fraction of lean clay or silt (i.e., USCS symbol of SC or SM with greater than or equal to 30% clay or silt, respectively). The soils would be graded, blended, prepared, placed, and compacted to engineering standards and densities. It is anticipated that a significant amount of material would need to be imported.

Beneath the levee, a zone of native soils would typically be removed and replaced. The depth of replacement is estimated to range from approximately 5 to 15 feet, but is expected to be 5 feet typically. The width of replacement would be slightly greater than the width of the base of the levee. This zone would be replaced with compacted clayey or silty soils as described above.

The typical configuration would include some type of in situ foundation improvement, to strengthen and stiffen the relatively weak and compressible soils present underneath most of the levee alignments. A zone of improved foundation materials would extend from the waterside levee toe to the landside toe. The zone of improved foundation materials would extend down to depths ranging from approximately 20 to 60 feet. The zone of improved foundation materials would typically be composed of a combination of existing in situ materials and added materials, mixed together. The type and quantity of added materials would depend on the type of foundation improvement methodology used. Additive materials may included other soil materials (typically sand and gravel), cement, cementitious or chemical grout, or a combination of these materials. The soils in the improved foundation zone would be significantly densified, or moderately to significantly bonded (e.g., cemented), or some combination of these effects. Levee configurations would include a zone of armoring material on the waterside slope of the levee. Armoring material would be rip-rap, which generally is composed of small to large angular boulders.

Different levee types have differing causes for slope armoring on the waterside. On-Channel Levees would be subject to waterway flows, and so could be armored for the full slope length on the waterside and have protection at the waterside toe. An Offset Setback Levee with No Alteration to Existing Levee is not planned to be subject to waterway flows, but is expected to be subject to inundation due to failure or overtopping of the existing levee. Therefore, the waterside slope of these Offset Setback Levees could be armored for the length of the slope. A similar requirement exists for the other new levee types, because the existing levee is vulnerable to overtopping or failure at most locations. In all of these cases of waterside slope armoring, details of the armoring design may be subject to modification, but the function of protecting the levee from erosion over the slope length could not be compromised.

Generally, an access road would be maintained either along the landside toe of the levee, along the levee crest, or along a combination of these locations.

A dedicated ROW would extend along the landside levee toe, principally to preclude encroachment of channels, ditches, trenches, or pits near the levee. New agricultural channels would need to be constructed at many locations due to disruption (and filling) of existing channels where new levees cross the existing channels. New agricultural channels would be positioned at planned locations near the outside margin of the ROW. The variability of existing conditions, both geotechnical and aboveground, are likely to cause variation from the typical designs for all types of levees. It should be noted that variations in the geotechnical conditions would not be fully understood until much later in the design process.

15.1.3 Design Considerations for New Levees

Some elements of the configurations are constrained by design criteria and safe engineering practice. Other elements, however, may be modified as needed to avoid site-specific features, accommodate special local concerns, and reduce overall impact.

15.1.3.1 Levee Type Selection

Each of the types of levees is situated along the alignment at certain characteristic or typical locations, each for a particular purpose or combination of reasons. At many locations, though, the type of levee that is currently planned may reasonably be replaced with a different type of levee. At some locations, it is not yet (at this stage of the design) confirmed what type of levee is needed or preferable. The following paragraphs summarize the typical rationale for selecting which type of levee to use at each location.

Adjacent Setback Levee

Adjacent Setback Levees are the most commonly sited type of levee improvement along the TDF conveyance alignment. Adjacent Setback Levees provide the reliability that is required with relatively little area of impact and taking of land, while creating some opportunity for habitat restoration in the area between the setback levee and the existing levee. Adjacent Setback Levees preserve the general alignment and flow of existing waterways. Adjacent Setback Levees preserve the general shape of existing properties and island tracts, and minimize the amount (for a setback levee) by which the limits of properties and island tracts are encroached upon. (Strengthened Existing Levees generally encroach less onto adjacent lands, as described in a subsequent section of this report, than do Adjacent Setback Levees, but have much greater impacts within existing waterways and on waterside slopes of existing levees.) Consequently, Adjacent Setback Levees are the default type of setback levee selected for most locations along the TDF conveyance alignment.

Generally, though, at most locations where an Adjacent Setback Levee is sited, a different type of setback levee could be specified if needed instead of an Adjacent Setback Levee. This type of change may be considered, for example, at an area where the adjacent land use is of particularly low value and having an Offset Setback Levee with Notching of Existing Levee to provide an intertidal marshland would provide especially valuable habitat restoration.

On-Channel Levee at Widened Existing Channel

An On-Channel Levee at Widened Existing Channel is sited most commonly along the northern parts of the conveyance alignment, with some additional locations along the southern part. This type of levee is located only in those areas where the existing river or slough channel does not have sufficient capacity to carry the design flow. At such locations, the capacity of the channel could be increased. To do so requires either widening the channel, deepening the channel, or a combination of the two approaches. Section 16.0 discusses channel enlargement and the selection of method of enlarging channels.

For locations where it is concluded that the channel could be widened, an On-Channel Levee at Widened Existing Channel can not be replaced with a different type of setback levee. Of the levee types, only the On-Channel Levee at Widened Existing Channel configuration increases existing channel capacity. The Offset Setback Levee with Notching of Existing Levee configuration allows for water to flow into the intra-levee area and can create significant on-stream storage volumes during flooding events, but this configuration does not increase the actual channel capacity or increase the volume that can be conveyed through the channel.

On-Channel Levee at New Channel

As an alternative to widening an existing channel to develop greater conveyance capacity, an entirely new channel may be constructed. On-Channel Levees at New Channel locations are sited along the northern part of the TDF alignment, in order to avoid removing habitat (which would occur if the existing channel was widened).

Offset Setback Levee with Notching of Existing Levee

An Offset Setback Levee with Notching of Existing Levee is most commonly sited where there is a tight bend or curve in the waterway and existing levee alignment. At such locations, an Offset Setback Levee typically runs a much straighter and significantly shorter course than would an Adjacent Setback Levee. The significantly shorter alignment substantially reduces the materials and resources consumed to construct the levee. The Offset Setback Levee forms an inland boundary of an intra-levee area, and where the existing levee is notched, the intra-levee area provides benefits as a habitat restoration area.

The intra-levee area serves a second, related purpose. Significant quantities of native soils would be excavated and removed from beneath the footprint of the levees during construction. A portion of this material may be disposed of on the levee slopes, but the remainder could be disposed elsewhere. Instead of hauling significant quantities of material off site to a distant disposal site, the native soils can be disposed of locally on site within the intra-levee area. In part for this purpose, an Offset Setback Levee with Notching of Existing Levee is situated periodically along the conveyance alignment. Specific intra-levee areas are not explicitly derived from anticipated disposal volumes, though, and the area (as well as planned fill heights) can be adjusted if needed. Consequently, the precise location of an Offset Setback Levee, of even the selection of an Offset Setback Levee instead of an Adjacent Setback Levee, is subject to change if needed. Generally, at most locations where an Offset Setback Levee with Notching of Existing Levee is sited, a different type of setback levee could be specified if needed instead of an Offset Setback Levee with Notching of Existing Levee.

Offset Setback Levee with No Alteration to Existing Levee

An Offset Setback Levee with No Alteration to Existing Levee is much less commonly sited than Adjacent Setback Levees along the TDF conveyance alignment. Offset Setback Levees cut further into island tracts and properties. Such encroachment is selected where it provides significant benefit (as described further in following paragraphs). Generally, an Offset Setback Levee with No Alteration to Existing Levee does not provide such significant benefits. At some locations, though, an Offset Setback Levee with No Alteration to Existing Levee has been deemed warranted to avoid impact to particular areas along the landside of existing levees. That is, instead of crossing such an area with an Adjacent Setback Levee, an Offset Setback Levee with No Alteration to Existing Levee is used to bypass the habitat area. It is important to note, however, that such avoidance is a relatively temporary benefit, in that most intra-levee areas are expected to be inundated during the life of the project due to failure or overtopping of the existing levees.

Generally, at most locations where an Offset Setback Levee with No Alteration to Existing Levee is sited, a different type of setback levee could be specified if needed instead of an Offset Setback Levee with No Alteration to Existing Levee. This type of change may be considered, for example, where an Offset Setback Levee with No Alteration to Existing Levee has been used to bypass a habitat area, but it is concluded that the impact of an Adjacent Setback Levee to the habitat area is actually less detrimental than the impact of the much longer alignment route (and footprint area) of the Offset Setback Levee.

Strengthened Existing Levee

Whether to strengthen an existing levee or to construct a new levee at a particular location depends on multiple considerations, including levee reliability factors, design flood level elevations relative to existing levee heights, environmental impacts and opportunities, construction feasibility, and other concerns. For most areas of the TDF alignment, it is preferable to construct a new levee, set back behind the existing levee, instead of strengthening the existing levee. For certain locations, strengthening of an existing levee instead of constructing a new setback levee is preferable. Typically, though, Strengthened Existing Levees are planned only where construction of a new setback levee is considered infeasible or would unavoidably cause significant impacts to existing features (and a Strengthened Existing Levee would not).

Facilities Protection Levee

Facilities Protection Levees are sited at limited locations, only for the purpose of providing flood protection to new TDF conveyance facilities.

Based on the selection considerations described in the preceding paragraphs, 78.0 miles of new and strengthened existing levees have been proposed for the TDF alignment. Table 15-2 identifies the total length of each type of levee improvement proposed.

Table 15-2: New and Strengthened Levee Lengths by Levee Type

Location	Length (miles)
Adjacent Setback Levee	38.6
On-Channel Levee at Widened Existing Channel	13.8
On-Channel Levee at New Channel	2.7
Offset Setback Levee with Notching of Existing Levee	10.5
Offset Setback Levee with No Alteration of Existing Levee	6.7
Strengthened Existing Levee	4.5
Facilities Protection Levee	1.2
TOTAL	**78.0**

15.1.3.2 Levee Alignment Location Selection

In addition to the factors discussed above, many other factors were considered in selecting alignment locations for the levees, including:

- Existing environmental features, habitats, uses, and protections;
- Existing land use, property, and agricultural boundaries, and impacts to land operations;
- Existing infrastructure, population centers, and transportation patterns;
- Existing irrigation, drainage, and flood control systems;
- Likeliness of presence of cultural resources;
- Geotechnical conditions;
- Current and future hydrodynamic, hydrologic, and hydraulic conditions;
- Other project levees and features, and connectivity to such;
- Curvature of alignments and waterways;
- Materials, equipment, resources, effort, and time required for construction;
- Character of island tracts and likely perceptions regarding alignment layouts;
- Likely value of habitat restoration opportunities;
- Likely future conditions of environmental features, habitats, uses, and protections;
- Likely future land use; and
- Reliability and sustainability of design.

Generally, for most of the levee alignments presented in this report, the location of the levee may be adjusted without compromising the design principles or conveyance reliability. For example, if an Offset Setback Levee alignment is considered to be too close to a habitat area containing protected species that are particularly sensitive to construction noise, it is likely that the levee alignment can be re-aligned at a distance further away from the habitat area.

Some types of levee realignments would not be possible or feasible, depending on the type of levee. An Adjacent Setback Levee generally cannot be realigned to be further toward the existing levee along which it is situated for stability reasons. Also, an On-Channel Levee at Widened Existing Channel generally cannot be realigned to be further toward the existing river channel along which it is situated for the purpose of achieving a specified channel capacity increase. At most locations where this type of levee is planned, however, it is not yet known (at this stage of the design) exactly how much additional capacity is needed, and it is likely that the eventual levee alignment locations would be situated further toward the existing river channel at many locations where this type of levee is shown.

15.2 Construction Methodology

The following subsections provide a description of each type of construction feature or process, including staging areas, as typically laid out or planned across the project. The descriptions include characteristic descriptive data or data ranges of key elements of typical layouts and configurations. Also discussed are key construction plan considerations that result in the proposed typical layouts and methods and the degree of flexibility in specifying or adjusting the layouts and methods.

15.2.1 Improving Foundations for New and Existing Levees

Throughout much of the alignment locations of new and existing levees, the foundation soil underlying the levee footprint area is composed of or includes organic peat or other weak soil materials. The presence of such soils would result in significant settlement of overlying levees and levee instability problems for levees constructed on such materials. Construction would include some form of foundation improvement, to improve the condition of the soils that underlie the levees.

15.2.1.1 Typical Proposed Construction Layouts and Methods

Foundation improvement for new levees would typically involve removal and replacement of a zone of native soils beneath the levee footprint and also in situ ground improvement of the soils beneath the removal and replacement zone.

Excavation to remove soils would typically be performed by common earthmoving equipment used for grading operations. Such equipment includes scrapers, bulldozers, excavators, front-end loaders, and related materials-handling machines, as well as materials-transport vehicles such as end-dumps, transfers, off-road mine haul trucks, etc. Excavated materials would be hauled along the levee alignment footprint area to material placement locations, generally located adjacent to the levee alignment footprint area. Some areas for staging of equipment and materials would be needed at intermittent locations along the alignment. Such areas would typically be rectangular in plan and range from approximately 200 to 1,000 feet on a side.

At most locations along the levee alignment, it would be infeasible to perform foundation improvement using excavation methods to depths exceeding approximately 5 feet. In such areas, improvement of the foundation soils would be performed either using pre-consolidation methods or by using in situ ground improvement methods.

Pre-consolidation involves preloading and consolidating the peaty soils using perforated vertical drains to facilitate drainage of the consolidating soils. Vertical drains are typically narrow (several inches) strips of geotextile-wrapped synthetic drainage panels, which are pushed down vertically into the ground by a telescoping shaft operated from a large track-mounted rig. After installing the wick drains, load would be added at the ground surface by placing soil fill, applied using a staged construction of the new levee, and likely would be completed over a two- to three-year period. As the foundation soils consolidate, water is slowly dispelled via the wick drains and routed from underneath the levee to adjacent areas or other designated water disposal areas. Time constraints for construction may limit the application of this approach.

Alternatively, in situ ground improvement methods are expected to be used to improve the foundation soils. In situ ground improvement methods involve adding materials to the existing in situ materials and mixing them together to form a stronger combined material. The type and quantity of added materials would depend on the type of ground improvement methodology used. In situ ground improvement methods include jet-grouting, compaction grouting, chemical grouting, shallow soil mixing, deep soil mixing, vibro-compaction, vibro-replacement, and other methods that impregnate the soil with the additive material and blend it into the soil mass. Additive materials may include other soil materials; cement, cementitious grout, or chemical-compound grout (for grouting or mixing methods); or a combination of these materials.

15.2.1.2 Construction Methods Considerations

The soft foundation soils vary in thickness and depth from the northern end of the alignment to the southern end. The methods for improving foundation soils would largely depend on the thickness in each reach. Additionally, thin layers of loose sand that may be present in the foundation soils are typically susceptible to liquefaction and would be the source of slope deformations during a seismic event; foundation improvement methods would typically target these soils as well.

Foundation improvements would depend not only on the type of material that requires increased stiffness or strength but also on the type of levee that is being constructed. In areas where existing levees would be strengthened, the levee may require more indirect access to perform the improvement. For example, excavation of undesirable material from underneath the levee would not be feasible, so foundation improvement would necessitate in situ methods that can be performed from the ground surface. Due to the existing levee configuration, as well as existing public and private uses adjacent to the levee, levee foundation improvements would need to be performed relatively quickly and involve indirect access methods such as jet-grouting or compaction grouting, which can be accomplished either vertically or at angle.

Generally, the most economical and feasible method of improvement is to either remove the soft soils and loose sand or construct the levee in prolonged stages to allow for consolidation of underlying materials. For this second approach, loose sand layers (where present) would still need to be improved through jet-grouting or another means of densification and strengthening. If pre-consolidation is used for foundation improvement, the consolidation of soft soils would also result in some settlement of the ground immediately adjacent to the levee. The amount of this settlement may range from less than a foot to several feet, depending on the peat thickness.

The method to be used in each area would also be dependent on the depth of the water table. Where the water table is shallow, it may be unsafe or otherwise infeasible to perform excavation and soil placement/compaction. In such areas, other methods of foundation improvement would be needed. Where groundwater is relatively deep, excavation methods would likely be preferable. At some locations, the width of the excavation area may expand in order to create safer side-slopes for the excavation (weaker soil materials need to be laid-back at flatter slopes to remain stable). Additionally, there would be limits on how much excavation can be performed in close proximity to existing levees. Activities would likely be limited to April through October for work on the waterside of the levees.

15.2.2 Constructing New Levee Embankments

Regardless of levee type, new levee embankments would be constructed generally using similar methods, and would vary slightly in methodology due to their physical spatial relation to existing levees and how the existing levees would be altered.

15.2.2.1 Typical Proposed Construction Layouts and Methods

Construction of the new levees would require large amounts of engineered fill to be hauled in from off site to staging areas along the levee alignment. Material would be moved along the levee alignment to locations where fill is to be placed for levee construction. Placement and compaction operations would be performed by common earthmoving equipment used for grading operations. Such equipment includes scrapers, bulldozers, excavators, front-end loaders, compactors, water trucks, and related materials-handling machines, as well as materials-transport vehicles such as end-dumps, transfers, off-road mine haul trucks, etc. Mostly, earthwork operations for levee construction would be confined to the limits of the levee footprint area. Additionally, some areas for staging of equipment and materials would be needed at intermittent locations along the alignment. Such areas would typically be rectangular in plan and range from approximately 200 to 1,000 feet on a side.

15.2.2.2 Construction Methods Considerations

Earthmoving activities performed by heavy equipment are generally relatively loud operations. The proximity of construction areas to habitat areas or to populated areas would need to be considered in selecting construction methodologies and equipment, as well as work times and possible environment constraints (e.g., due to wind direction and speed).

Placement and compaction of engineered fill would require water for moisture-conditioning of fill soils and for site dust control. Water would either need to be hauled to the site or, more likely, local sources of water would be developed along the levee alignment. Various methods and types of equipment are typically used during construction for pumping water from open water sources; methods selected would need to consider the water supply type and potential environmental impacts, and where water may be taken from would need to be carefully considered and controlled.

Fill placement and compaction operations can be severely hampered by rain. Seasonal impacts to construction schedules and methods would need to be considered in selection and review of proposed methodologies.

Construction methods of the various levee types would need to be flexible and contingency plans developed due to the wide variety of conditions within the Delta. The levees that would be constructed or improved are in areas that range from limited access with very little existing infrastructure to crossing highways and urban centers. Site access may require the development of new infrastructure, redesigning existing infrastructure, and negotiating with local interests to minimize construction impacts. These construction considerations are discussed further elsewhere in this report.

15.2.3 Strengthening Existing Levees

Strengthened Existing Levees are constructed by expanding and building onto existing levees, and the existing levee would undergo significant modification.

15.2.3.1 Typical Proposed Construction Layouts and Methods

Typical construction layouts and methods for strengthening of existing levees generally would be similar to those described in preceding sections for foundation improvements and construction of new levees. Additionally, however, most of the landside half of the levee would be effectively reconstructed by means of in situ ground improvement performed through this half of the levee.

Also, some in-water work would need to be performed to improve armoring along the waterside slope of the existing levee. Armoring typically would be placed using track-mounted excavators from on the existing levee crest, or from a crane mounted on a barge in the river.

15.2.3.2 Construction Methods Considerations

For each area where a Strengthened Existing Levee has been sited, the determination to improve the existing levee was because of the presence of unique features. Many areas are confined by existing infrastructure, protected habitat, or urban areas. This requires flexibility in not disturbing the local feature that requires less impact than if a standard setback levee had been designed. Construction schedules, staging areas, construction equipment, and ground improvement methods may all need to be altered to specifically address these areas in unique ways.

15.2.4 Notching or Removing Existing Levees

As described in previous sections, a notched levee would be created where an Offset Setback Levee with Notching of Existing Levee is sited. Existing levees would be removed at locations where an Offset Setback Levee with Widened Existing Channel is sited.

15.2.4.1 Typical Proposed Construction Layouts and Methods

Notching of an existing levee is performed by cutting a notch down through the existing levee along the waterway at one or more locations. Removal of an existing levee involves removal of the levee material all the way down to the underlying native soils, below typical low-flow river water elevations.

Notching or removal of existing levees may be performed by one or more of several methods. Notching or removal of the levee material may be performed using relatively agile but powerful excavation equipment, such as track-mounted excavators. Excavators typically would access the area to be excavated via the rest of the existing levee. Excavated material would be placed in materials-transport vehicles such as end-dumps, transfers, etc. Material would be hauled along the existing levee crest until reaching an access road that connects to the new setback levee work area.

Alternatively, notching or removal of levee material may be performed using dredging equipment. Dredging is described in subsections of Section 16.0 of this report.

Another alternative for notching or removing existing levee material is to isolate the work area with a cofferdam and to use earthmoving equipment within the isolated area to excavate the material. A cofferdam would be a temporary structure, most likely consisting of sheet piles, installed in the waterway a short distance from the levee. The cofferdam would be aligned parallel to the levee work area and would tie in to the levee beyond the limits of excavation work. The area between the cofferdam and the levee would be de-watered. Earthmoving equipment would then be used to remove levee material. Upon completion of earthwork, the cofferdam would be removed, allowing the river water to move back into and over the work area.

Armoring would be placed on the slopes and bottom of levee notches, and armoring may be placed along the ground surface (what would become the channel bottom) at some locations where existing levees are removed. Armoring may be placed using a track-mounted excavator from on the existing levee crest or from a crane mounted on a barge in the river.

15.2.4.2 Construction Methods Considerations

Earthmoving activities performed by heavy equipment are generally relatively loud operations, and in-water work disturbs sediments and water quality. The proximity of construction areas to habitat areas or species would need to be considered in selecting construction methodologies and equipment, as well as work times and possible environment constraints (e.g., due to river flows).

15.3 Maintenance Considerations

Levees would require maintenance to continue to perform reliably over the project life.

Permanent access roads along levees are needed for levee inspection, maintenance, and repair. Maintenance and repair of levees typically require the use of various earthen and armoring materials, as well as various types of equipment and supplies. All of these materials may be stockpiled and stored at various maintenance facilities situated intermittently along the levee alignment.

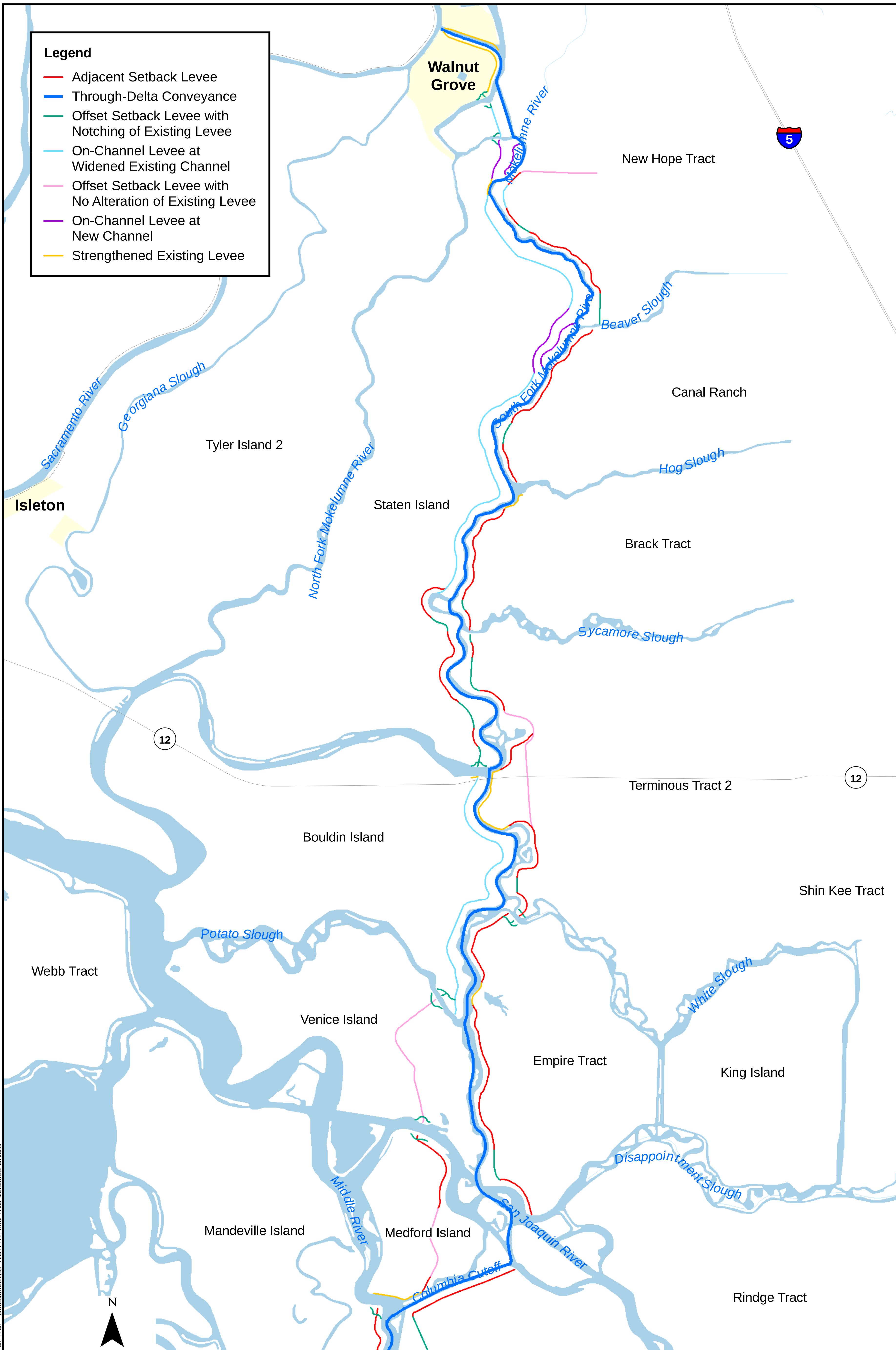
Generally, levee access roads would be unpaved but surfaced with aggregate baserock. Regular re-application of baserock and re-grading would be necessary to maintain these access roads. At some locations, where regular non-maintenance vehicular traffic could be conveyed along the crest or toe of the levee, roadways would be paved with asphalt-concrete pavement overlying aggregate base material.

Levee maintenance facilities would typically be composed of material stockpile areas, sized to accommodate materials, equipment, and sufficient area for staging and loading of materials. Such areas would typically be rectangular in plan and range from approximately 50 to 500 feet on a side, depending on the length of levee serviced by the maintenance facility.

Access roads would be used regularly for inspection of the levees. Inspection would be performed for both the waterside and landside slopes and features. Slopes could not be too heavily wooded or vegetated to inhibit observation and maintenance of the slope, and vegetation control measures would be performed as part of levee maintenance.

Maintenance activities include periodic addition of waterside armoring material, which may necessitate access and work either from the levee crest (e.g., using an excavator to place rip-rap) or from the water (e.g., using a barge and crane to place rip-rap).

Levee maintenance may also include operations designed to prevent and repair damage from animal burrowing within the levee.



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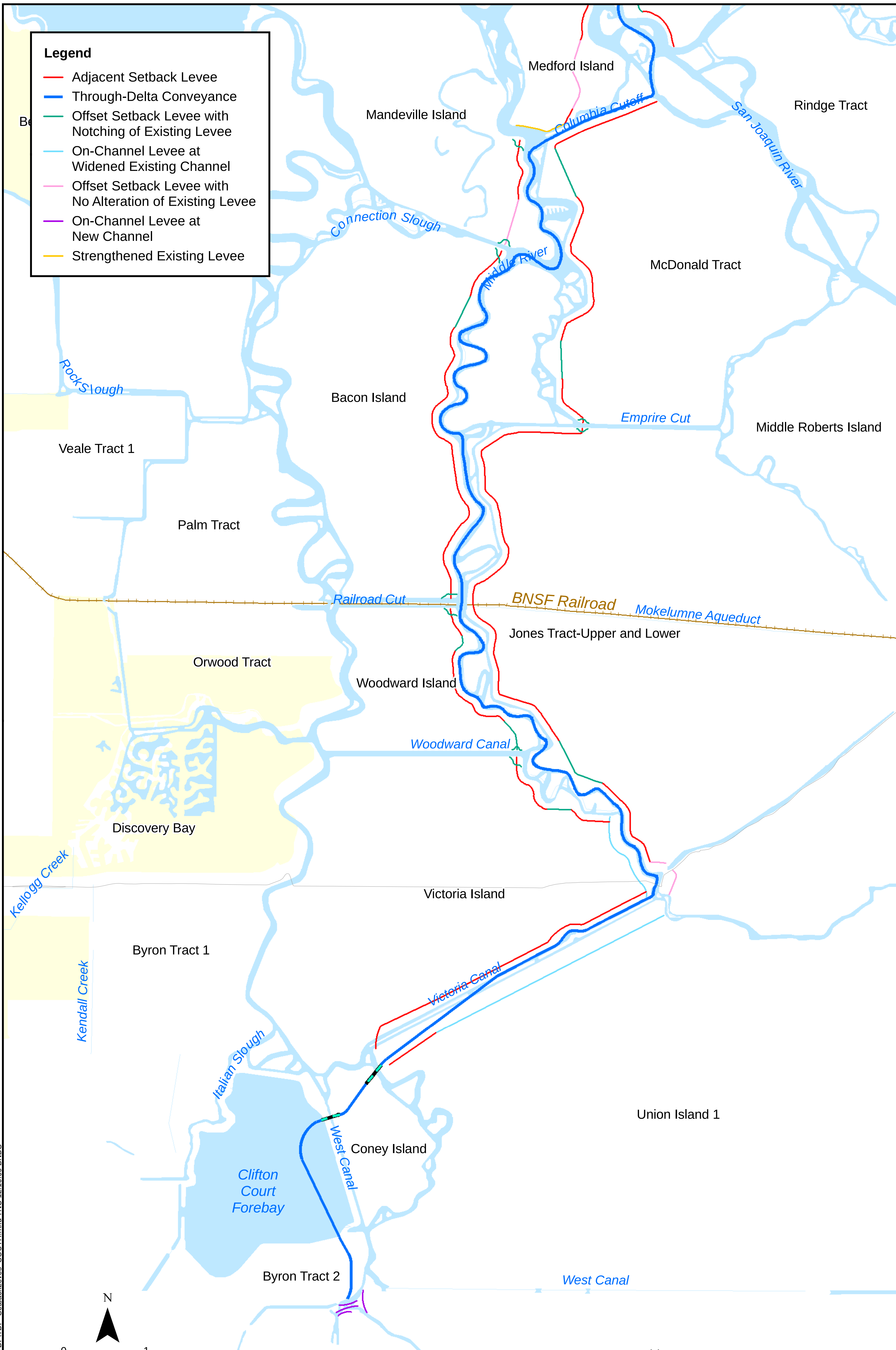
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Through-Delta Facility
North Levee Locations

FIGURE
15-1



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CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Through-Delta Facility
South Levee Locations

FIGURE
15-2

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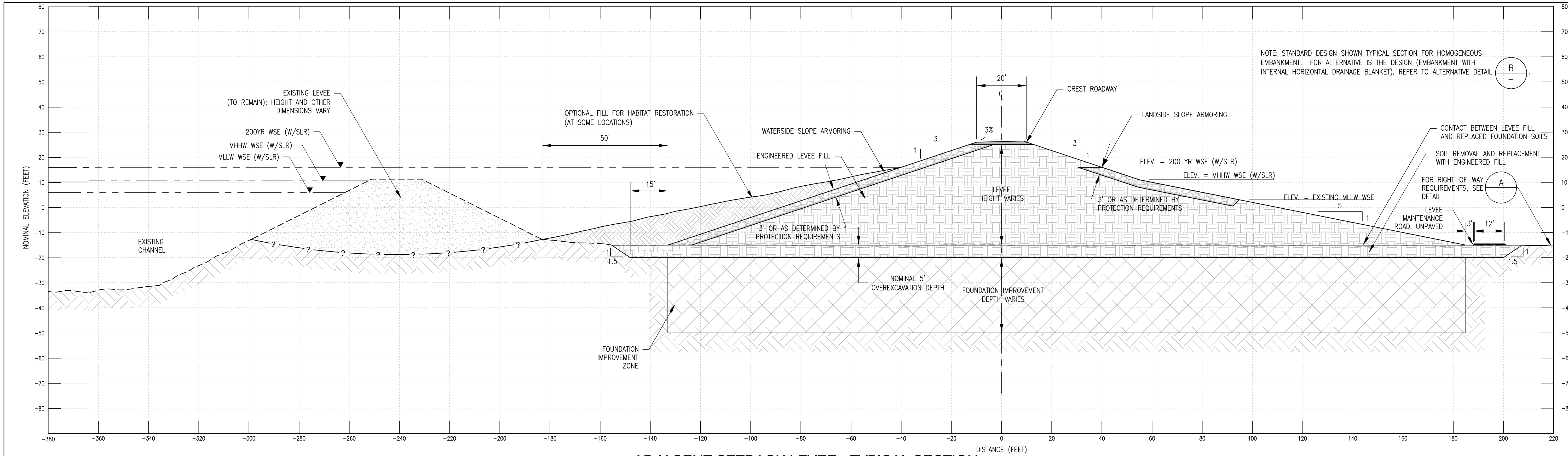
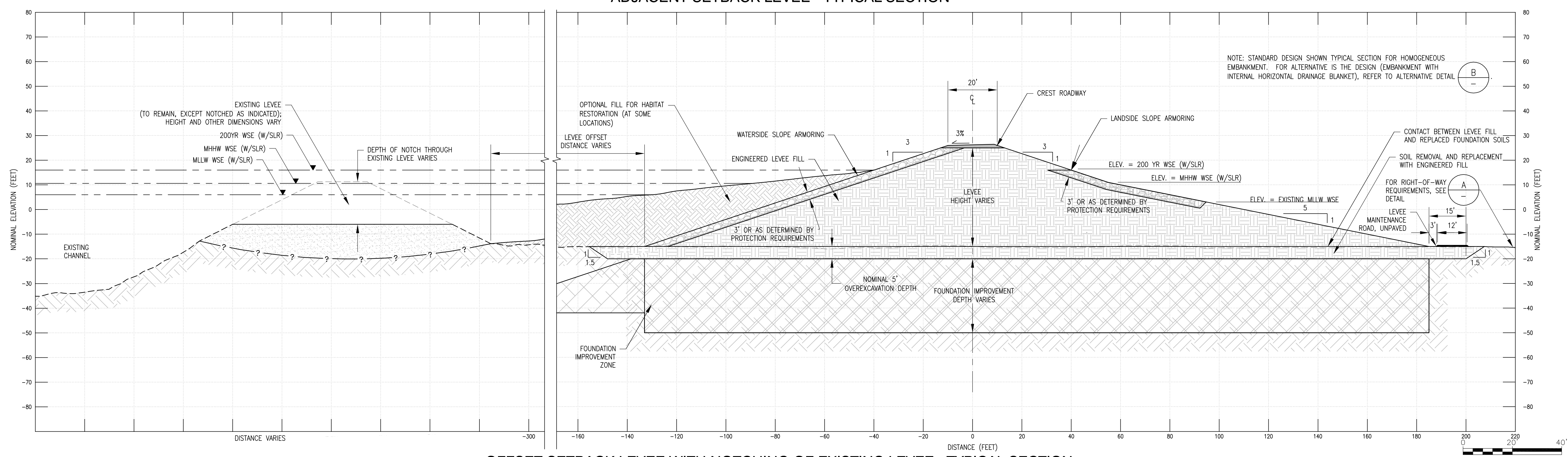


ADJACENT SETBACK LEVEE - TYPICAL SECTION



OFFSET SETBACK LEVEE WITH NOTCHING OF EXISTING LEVEE - TYPICAL SECTION

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California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT THROUGH-DELTA FACILITY OPTION	FIGURE
	Levee Sections	15-3

16.0 CHANNEL ENLARGEMENT MEASURES

Channel enlargement would be accomplished through dredging to deepen channels and the use of offset setback levees to widen the existing channel. Hydraulics for the improved through-Delta corridor are discussed in Section 5.0.

16.1 Description and Site Plan

Prior to human manipulation, the Delta drained the Central Valley in an essentially east to west manner, concentrating into larger and larger channels toward the heart of the Delta. Under the current configuration, much of the Delta's natural flow pattern has been altered by a series of diversions and navigable canals. The Through-Delta alignment follows a generally north to south pathway across the Delta, thus utilizing a number of canals and what would once have been tributary channels. This means that the size of channels on the alignment varies widely, depending on their original function. For the TDF Option, all channels south of the San Joaquin River are designed to carry 15,000 cfs. Conceptual engineering indicates that some channel enlargement would be required to reliably convey the requisite flows and to reduce the potential for scour.

16.1.1 Existing Channels

The Through-Delta alignment from the DCC to CCF consists of existing channels. These pre-existing waterways would be modified to meet the goals of the project, but retain essentially the same alignments and functions that they have historically held.

16.1.2 Hydraulic Considerations

To assess the capacity of the alignment, the cross sectional areas of each channel were needed. These areas were derived from application of the Delta Simulation Model II (DSM2) hydraulic model and comparison of model cross sections with updated bathymetry. The model yields cross sections with WSEs that can be used to compute area, but also provides feedback on the flow that results through each hydraulic reach. The model, as applied, does not analyze operations, but was used to contribute feedback to design, based on the hydraulic impact of potential modifications to the alignment. Once constricted areas were identified, the cross-sectional areas obtained from the modeling were compared to recently obtained Delta bathymetry to ensure that the additional area needed would be reflected in the design of channel enlargements. Further analysis is needed to optimize the total additional area needed on each cross section before accurate dredge quantities and setback levee locations can be developed.

16.1.2.1 Hydraulic Analyses

Because the response of the Delta system is highly dependent on the conditions being analyzed, it was important to develop a set of initial conditions for the DSM2 model that would provide WSEs relevant to project design. A selection of Delta inflows (and corresponding tidal, operations, and consumptive use) through the water years 1996 to 1998 was made. Within this time period, both high and low total Delta inflow conditions existed that would allow for a range of analyses. While a longer historical record of conditions exists, this period was deemed adequate to provide design feedback, while offering relatively rapid model run-times.

During the period from October 1996 to October 1998, conservative, high tide-high flow WSEs were sought. A high flow event occurred from February 5, 1998, to February 16, 1998. During this period the peak WSE at each alignment cross section and corresponding WSE were extracted from the model and plotted.

Reaches deemed to have an anomalously low conveyance area (shown in Table 16-1) were recommended for channel modification, either by widening or dredging, to achieve a target area. Target areas were established by interpolating areas between two sections bounding the constricted reach. The target areas were not based on any design criteria. Rather, they were established as a starting point for comparison between existing and proposed conditions, which would require further evaluation and optimization in future studies. Where additional flow through the alignment is deemed sufficient based on the results of applying the current target areas, no further modifications would be made. Where the increase is deemed insufficient, additional channel modifications may be considered.

Table 16-1: Cross-Sectional Area Modification Recommendations for Constricted Reaches of the TDF Alignment

Alignment Reach	Channel Name	Existing Area (feet^2)	Recommended Minimum Area (feet^2)	Area Increase (feet^2)
R4	South Fork of the Mokelumne River	10,662	13,839	3,177
R5	South Fork of the Mokelumne River	10,073	14,525	4,451
R6	South Fork of the Mokelumne River	9,004	11,657	2,653
R7	South Fork of the Mokelumne River	12,891	13,236	345
R9	Little Potato Slough	39,327	50,438	11,110
R10	Little Potato Slough	7,510	10,048	2,537
R17	Middle River	51,211	85,674	34,464
R18	Middle River	59,355	89,039	29,684
R19	Victoria Canal	37,620	81,689	44,069

Once target conveyance areas were established, channel modifications were made within the model to determine the revised flow through the alignment. A comparison in flow conveyance was made for 25-hour tidal cycles on a low system inflow on April 13, 1997, and a high system inflow on February 6, 1998. Such tidal averaging attempts to demonstrate the net flow direction and magnitude through the alignment.

16.1.2.2 Enlargement Design

Once satisfactory improvements were made in the modeled net flow through the alignment, the required additional area of each cross section was made available to the engineering design team. This team compared the model cross sections to the current cross-section data and enlarged sections, as needed, to match the target areas from the modeling. The design of channel enlargements was based on a range of factors, including slope stability, scour, and environmental concerns.

16.1.2.3 Results

Channel constrictions were identified in portions of reaches 4, 5, 6, 7, 9, 10, 17, 18, and 19 of the TDF alignment. These reaches comprise three work areas where either levee setbacks, dredging, or both, were considered to increase conveyance and to reduce scour. Reaches 17, 18, and 19 experience scour at high flows, necessitating modifications to convey 15,000 cfs through the Delta during normal operation. During a seismic event, when alignment barriers are closed, the remaining channel modifications are necessary to maintain the required flows.

Hydraulic modeling suggests that flow to the Central Delta during times of gate operation could be increased by setting back levees on both work areas in the northern end of the alignment. This would allow for increased conveyance area and hydraulic radius on Deadhorse cut, the South Fork of the Mokelumne, and Little Potato Slough. In addition to the operation of project barriers, the amount of additional flow through the alignment is impacted by the operation of the DCC gates and the amount of flow delivered by the Supplemental Intake Canal. The amount of additional flow during gate operation, with 4,000 cfs from the Supplemental Intake Canal, is indicated in Table 16-2, as an example of the effectiveness of increasing capacity. In future studies, an optimization of increased area should be completed to meet project goals for flow.

Table 16-2: Comparison of Flows Through the Alignment With and Without Channel Enlargement

	Low Flow		High Flow	
Channel	**Flow Without Channel Enlargement**	**Flow With Channel Enlargement**	**Flow Without Channel Enlargement**	**Flow With Channel Enlargement**
Mokelumne	3,015	3,306	14,682	21,393
Little Potato Slough	2,896	3,182	14,995	21,696
Middle River	8,126	8,188	2,957*	3,014*
Victoria Canal	8,062	8,151	4,332*	4,384*

* Majority of flows during high flow originates from the San Joaquin River and arrives at CCF via Grant Line Canal.

All units are cubic feet per second.

In the southern Delta, with barriers closed, the hydraulic constriction of reaches 17, 18, and 19 was significant enough that the DSM2 model would not function without channel modification at 15,000 cfs export pumping. These channels were re-simulated with larger cross sections, which also allowed channel velocity to be decreased to minimize scour. In this region of the Delta, some dredging was indicated by the propensity of these channels to “dry up” in the model, as well as a sudden drop in WSE just downstream of the work area. In future studies, an optimization of increased area should be completed to meet project goals for flow and velocity.

16.1.3 Setback Levees

This section describes two approaches of modifying levee alignments to enlarge channel capacity. Modifying levee alignments generally alters channel width (as opposed to channel depth) to achieve channel enlargement.

Section 15.0 describes the various types of levees to be constructed for the TDF; those which pertain to channel enlargement are further discussed in this section.

Figure 15-1 shows the locations and alignments of the levees. Please refer to the Figure 15-1 for depiction of alignments and configurations described in the following sections.

The first approach of modifying levees for channel enlargement is to directly widen the channel by (1) removing existing levees along one side, and (2) constructing new levees set back further away from the existing waterway centerline. The removed levee no longer serves as a lateral constraint to river flows, as water is allowed to pass over the former levee location and extend the river channel laterally out to the new setback levee. The river channel is thus widened by allowing flows to pass over the area between the new setback levee and the location of the formerly existing levee. The new setback levee forms the new channel boundary. This type of new levee is referred to in this report as an On-Channel Levee at Widened Existing Channel.

The second approach is to supplement the existing channel width by (1) constructing a new channel between a pair of new levees that is aligned next to the existing channel, (2) at each end of the new channel, curving and connecting the new levees to the existing channel levees, and (3) removing the existing channel levees between the connections to new levees at each end of the new channel. The resulting configuration is a waterway that splits into two adjacent channels, which rejoin shortly downstream. Another way of looking at this configuration is as a widened channel with an island in the middle, formed between the existing channel and the new channel. This approach to channel enlargement preserves the existing levee for the length of the island. The new levee that bounds the other side of the island, and the new levee on the opposite side of the new channel, are each referred to in this report as a New On-Channel Levee at New Channel.

These descriptions are intended to characterize each levee type as defined by and referred to in this report; the concepts and descriptions are project-specific to the TDF and do not apply to other projects or other applications.

16.1.3.1 Levee Geometry and Composition

Each type of new levee has the same general levee geometry and composition for the new levee itself (differences in type are due mainly to the new levee’s spatial relation to the existing levee and to the disposition of the area between the new and existing levees). Levee geometry and composition are described in Section 15.0.

Channel enlargement may be achieved by:

- Widening, by constructing new levees situated according to the relative locations and configurations described above;

- Deepening, by dredging along the bottom of the existing channel (dredging is discussed further in Section 16.1.4); or
- A combination of widening and dredging.

Selection of a preferred approach depends on several factors. In some areas, the amount of needed channel enlargement can not be achieved by dredging alone, and widening would be used either instead of or in conjunction with dredging. In other areas, where sufficient enlargement can be achieved by dredging alone, widening may still be preferable, depending on site-specific factors.

At some locations, dredging may potentially destabilize waterside slopes of existing levees. Dredging the channel bottom adjacent to levees would be expected to somewhat reduce the stability of levees. In some areas, the levees may have sufficient margins of safety to accommodate a slight reduction in stability. In many areas of the Delta, however, existing levees are only marginally stable. If dredging would be expected to further destabilize such levees, channel enlargement by widening would be preferable at these locations.

Another consideration for selecting widening or deepening relates to fluvial geomorphology and the dynamic nature of riverine environments. Deepening of channels along the conveyance alignment would result in a channel-bottom gradient change where tributary channels enter the conveyance channel. At these locations, the tributary channel bottom would step down to the deepened conveyance channel bottom. In riverine environments, such steps tend to result in down-cutting of the channel bottom upstream of the step. This down-cutting tends to migrate upstream until the river has achieved a new equilibrium between channel flows and channel bottom gradient. Down-cutting may be resisted by the channel bottom, depending on the materials that compose the channel bottom and the velocities of flows within the channel. If down-cutting were to occur, migrating into the tributaries of the conveyance channel, the down-cutting might destabilize waterside slopes of existing levees along the tributaries. At locations where this concern is considered a possibility, channel enlargement by widening may be preferable.

Environmental concerns may distinguish whether widening or deepening is preferable. Widening channels involves significant earthwork-related construction activities, disposal of excavation spoils, impacts to land where the levees and new channel areas are situated, and removal of existing levee environments. Deepening involves disturbance to waters during dredging, disposal of dredge spoils, removal of in-channel environments, and future occurrences of water disturbance, spoils disposal, and channel environment removal as initial dredging efforts would likely need to be re-performed on a regular and on-going basis to maintain channel capacity (refer to Section 16.3.2 for discussion of dredging maintenance). Site-specific environmental concerns may dictate which method of channel enlargement is preferable.

For channel widening, the alignment positioning, overall width, and footprint area of the combined new levee and widened channel, or set of new on-channel levees and new channel, depends on the amount of channel enlargement required. Required enlargement depends on existing channel capacity, WSE characteristics, and other factors. Additional design considerations for channel enlargement by widening include those discussed in Section 15.0 regarding levee geometry, composition, and other factors.

Typically, the levee offset and channel enlargement would be constructed on the west bank of the conveyance route. The offset distance was calculated based on the required additional cross-sectional area for the channel, and using existing ground surface elevations in comparison to the WSE for estimation of depth of the widened portion of the channel. The calculated offset distance was measured from the existing landside levee toe.

Additional cross-sectional channel area of 7,500 square feet would also be needed in Reaches R4 and R5, extending along the South Fork of the Mokelumne River from its confluence with Dead Horse Island Cut to its confluence with Beaver Slough. To achieve this enlargement, the new levee would be placed an offset distance of 585 feet from the existing levee on Staten Island. An exception is at the upstream end of Reach R4, near Supplemental Intake Landing. At this location, instead of widening the existing channel, a new channel would be constructed parallel to and west of the existing channel. This approach preserves habitat along the west bank of the existing channel while also providing improved channel hydraulics at the confluence of Dead Horse Island Cut with the Mokelumne River where it splits into the North and South Forks. To achieve the needed channel enlargement, the new channel would be 585 feet wide.

Reach R6, extending along the South Fork of the Mokelumne River from Beaver Slough to Hog Slough, would require an additional 4,000 square feet of cross-sectional channel area. For most of Reach R6, this enlargement would be achieved by widening the existing channel, setting back a new levee on Staten Island an offset distance of about 400 feet. An exception is at the upstream end of Reach R6, near Beaver Slough. At this location, instead of widening the existing channel, a new channel would be constructed parallel to the existing channel. This approach preserves habitat along the west bank of the existing channel. To achieve the needed channel enlargement, the new channel would be about 350 feet wide.

Additional cross-sectional channel area of 2,000 square feet would be needed along the northern part of Reach R7, from Hog Slough to approximately 2,000 feet north of Sycamore Slough. Channel enlargement would be accomplished by setting back the new levee along Staten Island an offset distance of approximately 250 feet.

Channel enlargement along Reaches R9 and R10 would be needed to provide and additional 2,000 square feet of cross-sectional channel area. From the confluence of the South Fork of the Mokelumne River and Little Potato Slough to the confluence of Potato Slough and Little Potato Slough, a levee would be set back a distance between 220 and 320 feet west of the existing levee.

Channel enlargement would also need to be effected near the southern end of the TDF alignment. Along Reach R18, it is intended to dredge the channel to increase channel capacity (refer to the next section of this report). To allow for flexibility in dredging layouts, the channel would also be widened by constructing an offset setback levee. The new levee would be set back an offset distance of 300 to 350 feet from the existing levee, which would allow for an additional cross-sectional channel area of up to 4,000 square feet from widening.

Reach 19, which is the existing Victoria Canal, would also require channel enlargement. The channel would be widened substantially, to accommodate the full 15,000 cfs flow without causing scour of the channel bottom. The needed additional cross-sectional area is about 12,000 square feet. To achieve this, a new levee would be constructed along the southeast side of Victoria Canal, set back an offset distance of 930 feet from the existing Victoria Canal levee. This offset distance would also allow for deepening of the widened part of the channel, such that the entire channel bottom may be cut down to the same elevation as the current channel bottom of Victoria Canal. This widening and deepening would begin where the conveyance enters into Victoria Canal and extend the length of Victoria Canal to the point where the channel could narrow into the Intake Facilities at the southwest end of Victoria Canal.

16.1.4 Dredging

Dredging was evaluated as a potential method of enlarging the cross sectional area of hydraulically under capacity reaches identified in the TDF alignment. In order for dredging to be considered in a reach, a selection of cross sections within each reach were plotted and compared to a “safe” dredging envelope section. The “safe” dredging envelope section was developed to identify the limits of the maximum dredging cut to prevent potential structural damage to the foundations of the existing levees. If the required amount of additional cross section area could be achieved by removing material without exceeding the limits of the “safe” dredging envelope section, dredging was considered as a potential channel enlargement method. Table 16-3 presents the potential in situ dredge volumes for nine reaches in the southern end of the alignment near the Victoria Slough. These volumes are preliminary and represent the “best guess” estimate based on the limited amount of survey data available. Prior to selecting dredging in these reaches, additional survey and bathymetry data, and detailed hydraulic analysis and modeling are required to determine actual conditions and anticipated increased hydraulic capacity.

Table 16-3: Potential Dredge Volumes Along Southern Reaches of the Through-Delta Alignment

DSM2 Reach Number	Reach Length (Approximate) (LF)	Approximate Dredge Volume (in situ) (cy)	Possible Methodology/Notes
135	3,000	300,000	Hydraulic or Mechanical Dredging
136	3,500		Insufficient data to determine if dredging is required
137	3,500	500,000	Hydraulic or Mechanical Dredging
138	4,000	400,000	Hydraulic or Mechanical Dredging
139	3,500	200,000	Hydraulic or Mechanical Dredging
141	6,500		Insufficient data to determine if dredging is required
142	3,500		Insufficient data to determine if dredging is required
143	5,500	200,000	Hydraulic or Mechanical Dredging
144	3,500		No dredging required – sufficient hydraulic capacity exists
Total	**36,500 LF (7 miles)**	**1,600,000 CY**	

cy = cubic yards
LF = linear feet

16.2 Construction Methodology

16.2.1 Setback Levees

Regardless of levee type, new levee embankments would be constructed generally using similar methods, and would vary slightly in methodology due to their physical spatial relation to existing levees and how the existing levees would be altered. Levee construction methodology and considerations are described in Section 15.0.

16.2.2 Dredging

The results of a preliminary evaluation determined that, in general, the potentially feasible channel enlargement alternatives for this alignment are offset levees in the middle and northern reaches of the alignment and dredging in approximately 7 miles of the southern reaches near the Victoria Canal. Figure 16-1 illustrates the areas where dredging may be feasible. This figure identifies potential dredging areas, potential dredging areas also requiring excavation of island areas, and areas requiring additional analysis to determine the appropriate enlargement method.

Dredging would only be used where necessary for channel enlargement. Dredging methods can generally be classified in two categories: hydraulic dredging, and mechanical dredging. Hydraulic dredging utilizes barge-mounted water power and inductor type pumps and hydraulic cutter jets to mobilize sediments. The water and dredge spoil volumes, referred to as slurry, are then pumped to settling ponds for gravity settling and dewatering of the dredge spoil sediments. This type of dredging requires large bermed areas to facilitate gravity settling. The size of the dewatering areas is dependent on slurry flow rate, amount of total dredge spoils, and settling rate of the material. This type of dredging results in the lowest developed sediment plumes in waterways, however, it requires the management of large volumes of water. The range of solids concentration in the slurry can be 2 to 15% by volume. Mechanical dredging utilizes barge-mounted clamshell type buckets or land-based drag line buckets to excavate the dredge spoils. Typically, the spoils are placed in holding areas on the barge for dewatering and eventual disposal at designated underwater placement areas for ocean applications, or land-based transfer and disposal. This dredging methodology results in higher underwater sediment plume development due to the aggressive disturbance of the sediment by the mechanical buckets. However, the mechanical systems do not require management of large volumes of water.

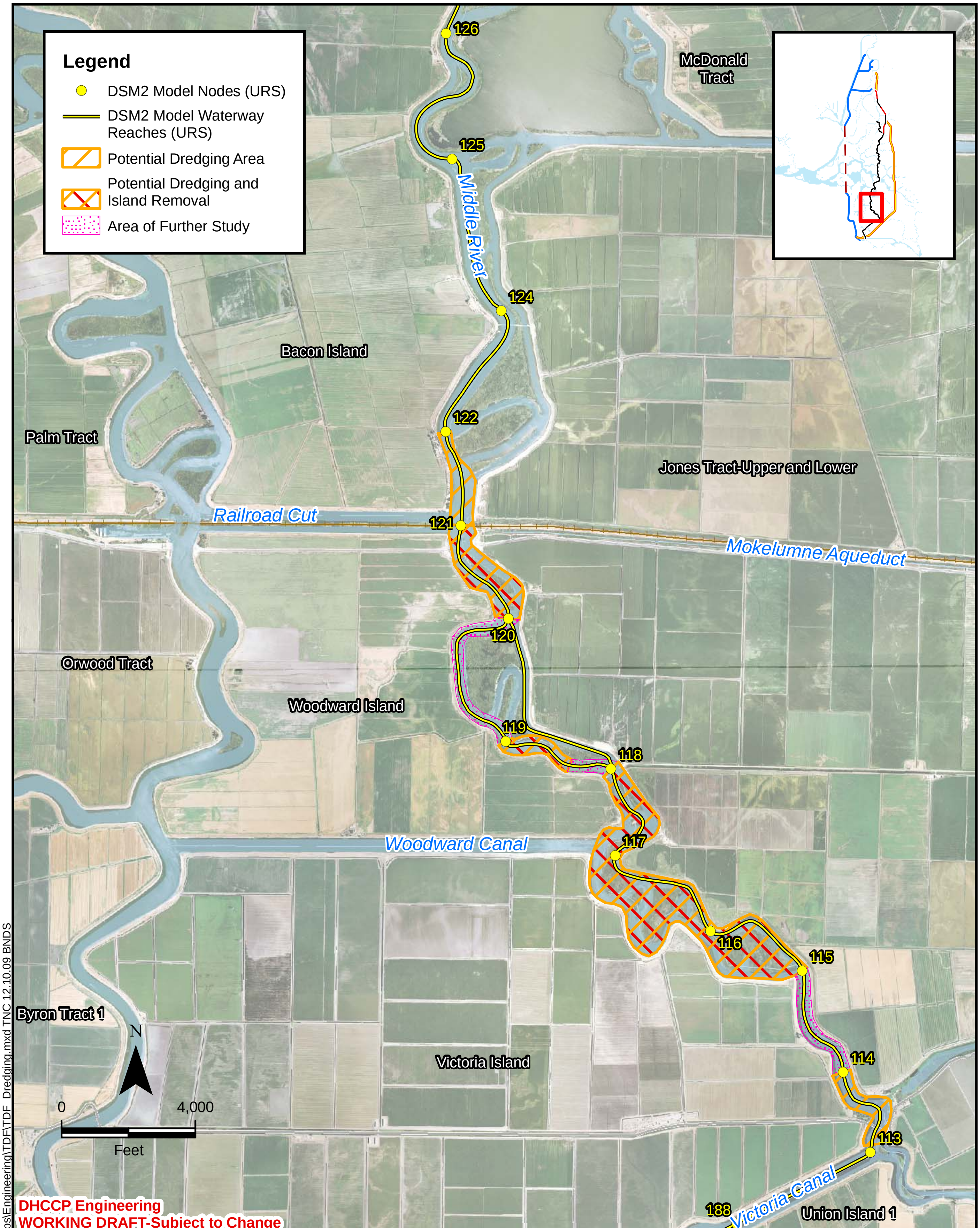
16.3 Maintenance Considerations

16.3.1 Setback Levees

Levee maintenance considerations are described in Section 15.0.

16.3.2 Dredging

All dredged channels and waterways require periodic maintenance dredging to maintain the desired channel cross sections. The frequency of the maintenance dredging is dependent on the sediment load carried by the waterway and the hydraulic scouring conditions in the waterway.



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California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
THROUGH- DELTA FACILITY OPTION

Channel Enlargement Measures
Potential Dredging Areas

FIGURE
16-1

17.0 OPERABLE BARRIERS

17.1 Overview

The proposed channel alignment for the TDF Option crosses rivers, sloughs, and manmade canals that hydraulically connect the project to the Delta. To protect water quality within the TDF conveyance system, operable barriers would be provided at each of these crossings. The barriers would be closed to prevent salt water intrusion into the conveyance system during an emergency situation. SLR is not an upset condition; it is an assumed future (normal) condition. In addition to preventing salt water intrusion, four barriers located north of the San Joaquin River (Snodgrass Slough, North Fork of the Mokelumne River, South Fork of the Mokelumne River, and Potato Slough) would periodically be closed to maintain the required HGL (water level) within the project channel and prevent unnecessary loss of water down the river or slough. These barriers are referred to as dual barriers.

The TDF conveyance incorporates a short section of the San Joaquin River, which also forms a part of the Port of Stockton Deep Water Ship Channel. An operable barrier would be required on the river across the ship channel to protect the TDF from salt water intrusion. As an alternative to using the San Joaquin River for part of the conveyance, the TDF alignment includes a tunnel option that would convey project water under the river and eliminate the need for the barrier on the river. The tunnel option would require a different arrangement of operable barriers in the area to function properly and prevent salt water intrusion. This section also describes the operable barriers that would be required north and south of the San Joaquin River for the tunnel option.

17.2 Description and Site Plan

17.2.1 Operable Barrier Locations – TDF

Nine operable barriers would be required on the west side of the TDF to isolate project water from the Delta, if necessary. Barriers would be provided at the locations shown in Table 17-1 where the TDF conveyance crosses rivers or sloughs. The barrier locations are also shown on Figure 17-1. Note that each barrier is securely tied into new project levees on both banks of the river or slough to fully protect the project.

Table 17-1: Operable Barriers, TDF

Barrier	Dual Barrier
Snodgrass Slough	Yes
North Fork of the Mokelumne River	Yes
South Fork of the Mokelumne River	Yes
Potato Slough	Yes
San Joaquin River	No
Middle River	No
Connection Slough	No
Railroad Cut	No
Woodward Canal	No

17.2.2 Operable Barrier Locations – TDF With Tunnel Option

As indicated previously, the TDF includes a tunnel option that would eliminate the need for an operable barrier on the San Joaquin River. For the TDF with the tunnel, a total of 11 operable barriers would be required. Barriers would be provided at the locations shown in Table 17-2 where the TDF conveyance crosses rivers or sloughs. The barrier locations are shown on Figure 17-2. Note that each barrier is securely tied into new project levees on both banks of the river or slough to fully protect the project.

Table 17-2: Operable Barriers, TDF with Tunnel Option

Barrier	Dual Barrier
Snodgrass Slough	Yes
North Fork Mokelumne River	Yes
South Fork of the Mokelumne River	Yes
White Slough – Tunnel Option	No
Potato Slough – Tunnel Option	Yes
Little Connection Slough – Tunnel Option	Yes
Middle River – Tunnel Option	No
Connection Slough	No
Empire Cut – Tunnel Option	No
Railroad Cut	No
Woodward Canal	No

The additional barrier required on Little Connection Slough for the tunnel option would be a dual barrier based upon available hydraulic model information. Under certain water level and flow conditions the barrier could be closed to maintain the upstream water level in Little Connection Slough high enough to insure proper flow into the tunnel. Likewise, the barrier would be closed to prevent salt water backflow into the project from the San Joaquin River, which no longer has a barrier for the tunnel option.

The additional barriers located on the east side of the TDF on White Slough and Empire Cut would also be required to isolate the project from salt water intrusion if there is no barrier on the San Joaquin River. Both channels indirectly connect the San Joaquin River to the project and available hydraulic model study results indicate that circulation of water from the San Joaquin River back into the project via these two channels is possible under certain hydraulic conditions.

17.2.3 Typical Operable Barrier Arrangements

A typical operable barrier is shown on Figure 17-3.

17.2.3.1 Selection of Barrier Type for Conceptual Design

There are a variety of gates that could be used to form operable barriers. However, the selection of gate style is severely limited by the depth of water to be handled under emergency or future SLR conditions. Typical water depths in the rivers and sloughs under design emergency high water conditions can range from 30 feet up to 50 feet based upon available bathymetric data, and differential heads across a barrier could be up to 20 feet. Minimizing obstructions in the waterway (piers, walls, etc.) would also be a concern. Considering only gates of proven technology for the water depths and heads applicable to the project, miter gates have been selected for the barriers at all locations for conceptual engineering. Such gates are used for ship locks on canals and rivers in deep water applications. In addition, USACE has developed several engineering manuals covering the design, construction, and maintenance of miter gates based upon a long history of use.

Like other gate styles (radial, Obermeyer, etc.), miter gates are effective in resisting differential water pressure in only one direction. Because some of the barriers could be dual barriers, resisting salt water intrusion in one direction at times or maintaining water level inside the project conveyance in the opposite direction at times, dual barriers include two sets of gates in each bay oriented in opposite direction, similar to a standard boat lock. The upstream or downstream gates would be closed depending upon the direction of water level control needed.

17.2.3.2 Miter Gate Description

A typical miter gate consists of two fabricated steel panels that form a three-hinged arch to resist water pressure when the panels are in the closed position. Each gate panel is composed of horizontal steel girders, vertical intercostals, vertical end diaphragms, a skin steel plate, and adjustable diagonal tension rods. To minimize leakage, seals are provided along the side and bottom of each panel and where the panels butt together when the gate is closed. Each gate panel is supported by fabricated top and bottom hinges that are firmly anchored to the concrete wall and floor of the supporting structure.

Gate panels can be opened and closed using electrical motors, reducing gears, and mechanical linkages, or using hydraulic cylinders that are supported on top of the concrete bay walls and linked directly to the top of each gate leaf. For conceptual design, it is assumed that hydraulic cylinders would be used for operation. This selection was made to avoid having higher maintenance electrical and mechanical equipment located on top of high pier walls spaced across wide rivers and sloughs. To minimize concern for environmental impact from hydraulic oil spills, food-grade hydraulic oils are available to operate the hydraulic cylinders.

17.2.3.3 Gate Structure Arrangements

The barrier structure would be constructed of reinforced concrete walls, piers, and foundation mats. The concrete thickness would depend upon the height and width of each structure bay. Because of the limited information available at the various barrier sites, it is assumed that all structures would be supported on foundation piles driven to a required bearing capacity of at least 60 tons (120 Kips). Lateral resistance capacity for piles would be determined assuming that the upper 10 feet of in situ soil below the foundation mat would contribute little resistance to lateral movement. Foundation piles would be standard, rolled, steel H-piles that can be field spliced (welded) to provide the required driven lengths. Driven lengths are anticipated to be between 60 and 80 feet below foundation level. Piles that could be subjected to uplift forces would be securely anchored into the concrete foundation.

Each barrier structure ties into new project levees on both sides of the waterway. To afford a uniform level of protection across a barrier installation, the top elevation of the structure walls was set to the same elevation as the proposed levee crest adjacent to the structure. This arrangement also facilitates access to the sides of the structure from the levees for O&M.

The number of gate bays required at any barrier depends upon the width and bottom profile of the existing channel. The height of each gate bay was selected to match the bottom channel profile as closely as possible. The width of each bay was then selected so that the height to width ratio of the individual gate panel would be approximately 1.0. An effort was made to standardize bay widths as much as possible to facilitate fabrication.

A critical aspect of barrier design and construction is the interface between the side walls and the new levee fill placed against them. The structure height at each end was selected such that the level of the foundation mat was at or below the anticipated excavation depth under the new levees. In this way, fill for the new levee would be placed fully against the sides of a new structure on the prepared foundation. In addition, the outside wall face of the structure would be battered to compress the soil if it settles along the wall interface. A number of battered buttresses would also be provided along the side walls for structural purposes to minimize deflection that could affect gate operation, and to minimize the possibility of developing a seepage path along the wall-soil interface.

Another critical aspect of barrier design is the possibility of differential settlement occurring between the barrier structure supported on piles and the adjacent levee fill over time. To minimize this concern, consideration has been given to providing additional ground stabilization under the levee fill for some distance before the structure.

A 20-foot-wide by 40-foot-long O&M building would be provided at each barrier location. The building would be a reinforced concrete masonry unit building on a concrete slab with standing-seam metal roof. The building would be insulated and ventilated to maintain preset temperatures for summer and winter. The building would include the hydraulic operating equipment for the gates, electrical service panels, instrument and control panels to facilitate local and remote monitoring and operation, and an area for maintenance equipment and maintenance activities. The hydraulic operating equipment would include hydraulic oil pumps, oil reservoirs, flow control valves, and high-pressure accumulator bottles that would permit a certain number of gate openings and closings in the event of a power failure. A bank of batteries would operate the control system, also in the event of a power failure. Hydraulic lines would be hard-piped from the control building to each hydraulic cylinder and would run within concrete walls and mats.

Power to each site would be provided from a 12-kV service line that would follow the new levee alignment. A ground-mounted transformer would be provided at each site with an adjacent pole to receive the overhead service connection. Electrical lines from the transformer to the building service panels would be in conduit underground.

For conceptual design, it is assumed that corrosion protection for steel gate panels and other steel members would be a coating system suitable for brackish water conditions with an impressed current cathodic protection system.

Some excavation (dredging) would be required for several hundred feet upstream and downstream of the gate structures to transition the sides of the natural channel to the required depth and width of the gate structure. It is anticipated that the conformed cut bottom upstream and downstream would be protected by rip-rap to control erosion.

17.2.3.4 Tethered Barge Gate

For the San Joaquin River, because of the presence of the ship channel, an alternative to the miter gate structure could be a tethered barge gate. In general, the tethered barge gate is a less proven design that could require further field testing and refinement. However, the advantages of a tethered gate may deserve further consideration in the future considering reduced site disturbance and no blockage of the river channel,

This concept involves a tethered barge structure that floats along the river bank when not deployed, and is pivoted across the full channel width when activated. The barge structure is built from modular pontoon barges (10 feet long by 30 feet wide by 7 feet deep) that are common in the marine construction industry. These relatively small modular barge units are interconnected to create a large floating structure capable of spanning the channel. The barge structure rotates about a large pivot pile located on one side of the channel. The structure is fitted with a hinged steel gate that hangs below the floating barge, extending to the bottom of the channel upon deployment.

17.2.4 Site-Specific Barrier Details

The configuration of the new levees would vary from site to site but the interface details with the gate structure would remain the same.

17.2.4.1 Barrier on Deadhorse Slough at Deadhorse Cut

This two-way barrier serves primarily to retain fresh water from the Sacramento River within the alignment, should a significant seismic event occur in the eastern Delta. However, it is possible that depending on the tide, the water level could be higher on either the east or west side of the barrier. Without this barrier, during such an event, water brought into the alignment through the Supplemental Intake Canal or DCC would tend to follow the natural hydraulic gradient westward down Deadhorse Slough to the North Fork Mokelumne, rather than following the TDF alignment. This could result in longer system downtimes, due to poor water quality. It is intended to be operated during a seismic event, in concert with barriers on North Fork Mokelumne River at South Fork of the Mokelumne River, South Fork of the Mokelumne River, and Potato Slough at Little Potato Slough. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.2 Barrier on North Fork Mokelumne River at South Fork of the Mokelumne River

This two-way barrier serves primarily to retain fresh water from the Sacramento River within the alignment, should a significant seismic event occur in the eastern Delta. However, it is possible that depending on the tide, the water level could be higher on either the east or west side of the barrier. Without this barrier, during such an event, water brought into the alignment through the Supplemental Intake Canal or DCC would tend to follow the natural hydraulic gradient westward down the North Fork Mokelumne, rather than following the TDF alignment. This could result in longer system downtimes, due to poor water quality. It is intended to be operated during a seismic event, in concert with barriers on Deadhorse Slough at Mokelumne River, South Fork of the Mokelumne River, and Potato Slough at Little Potato Slough. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.3 Barrier on South Fork of the Mokelumne River at Little Potato Slough

This two-way barrier serves primarily to retain fresh water from the Sacramento River within the alignment, should a significant seismic event occur in the eastern Delta. However, it is possible that depending on the tide, the water level could be higher on either the east or west side of the barrier. Without this barrier, during such an event, water brought into the alignment through the Supplemental Intake Canal or DCC would tend to follow the natural hydraulic gradient westward down the South Fork Mokelumne, rather than following the TDF alignment. This could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Deadhorse Slough at Mokelumne River, North Fork Mokelumne River at South Fork Mokelumne, and Potato Slough at Little Potato Slough. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.4 Barrier on White Slough at Little Potato Slough

This optional, one-way barrier serves primarily to prevent salt water from the San Joaquin River from entering the alignment, should a significant seismic event occur in the eastern Delta, increasing regional salinity. This barrier would only be included should the final alignment cross the San Joaquin River through a tunnel. If a barrier on the San Joaquin River were utilized, it is not anticipated that water entering or escaping from the alignment at White Slough would be a problem, as the alignment would be isolated elsewhere. And, if the final alignment included no hydraulic constraint on the San Joaquin River, this barrier would have little impact. However, in the event of tunnel implementation, salinity moving up the San Joaquin River could still enter the alignment without a barrier at this location and its impact could be significant. Without this barrier to support a tunnel crossing of the San Joaquin, longer system downtimes, due to poor water quality could result. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.5 Barrier on Potato Slough at Little Potato Slough

This two-way barrier serves primarily to retain fresh water from the Sacramento River within the alignment, should a significant seismic event occur in the eastern Delta. However, it is possible that depending on the tide, the water level could be higher on either the east or west side of the barrier. Without this barrier, during such an event, water brought into the alignment through the Supplemental Intake Canal or DCC would tend to follow the natural hydraulic gradient westward down Potato Slough, rather than following the TDF alignment. This could result in longer system downtimes, due to poor water quality. It is intended to be operated during a seismic event, in concert with barriers on Deadhorse Slough at Mokelumne River, North Fork Mokelumne River at South Fork Mokelumne, and South Fork Mokelumne at Little Potato Slough. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.6 Barrier on San Joaquin River between Middle River and Little Potato Slough

This barrier would be an alternative to the construction of a tunnel across the San Joaquin River or the option to leave the San Joaquin River hydraulically connected to the TDF alignment. This barrier would have distinct advantages and disadvantages to these options. In terms of water quality, it would have a similar effect to that of the tunnel across the San Joaquin, but would not require the supporting barriers on White Slough and Empire Cut that the tunnel would. However, it may be less feasible to construct. The construction of this barrier would allow the TDF alignment to be hydraulically isolated from the tidal influence of the San Joaquin River, which, during critical times, could bring a very high volume of salt water into the alignment. This consequence could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Deadhorse Slough, North Fork and South Fork Mokelumne, Potato Slough, Middle River at Columbia Cut, Connection Slough, Railroad Cut, and Woodward Canal at Middle River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.7 Barrier on Middle River at Columbia Cut

This barrier would not be required if the 2-Gates project is constructed. This one-way barrier serves primarily to exclude salt water from the alignment that could otherwise be drawn in, should a significant seismic event occur in the vicinity. This barrier potentially could be utilized during the low flow season to control salinity at the pumps on an as-needed basis, as well. Without this barrier, during such an event, the large volume of salt water coming up the San Joaquin River could fill the alignment creating a rapid increase in salinity at the pumps. This consequence could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Connection Slough, Railroad Cut, and Woodward Canal at Middle River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.8 Barrier on Connection Slough at Middle River

This one-way barrier serves primarily to exclude salt water from the alignment that could otherwise be drawn in, should a significant seismic event occur in the vicinity. This barrier potentially could be utilized during the low flow season to control salinity at the pumps on an as-needed basis, as well. Without this barrier, during such an event, the large volume of salt water coming up the San Joaquin River could fill the alignment creating a rapid increase in salinity at the pumps. This consequence could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Middle River at Columbia Cut, and the barriers on Railroad Cut, and Woodward Canal at Middle River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.9 Barrier on Empire Cut at Middle River

This optional, one-way barrier serves primarily to prevent salt water from the San Joaquin River from entering the alignment, should a significant seismic event occur in the eastern Delta, increasing regional salinity. This barrier would be included only if the final alignment crosses the

San Joaquin River through a tunnel. If a barrier on the San Joaquin River were utilized, it is not anticipated that water entering or escaping from the alignment at Empire Cut would be a problem, as the alignment would be isolated elsewhere. And, if the final alignment included no hydraulic constraint on the San Joaquin River, this barrier would have little impact. However, in the event of tunnel implementation, salinity moving up the San Joaquin River could still enter the system without a barrier at this location and its impact could be significant. Without this barrier to support a tunnel crossing of the San Joaquin, longer system downtimes, due to poor water quality could result. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.10 Barrier on Railroad Cut at Middle River

This one-way barrier serves primarily to exclude salt water from the alignment that otherwise could be drawn in, should a significant seismic event occur in the vicinity. It potentially could be utilized during the low flow season to control salinity at the pumps on an as-needed basis, as well. Without this barrier, during such an event, the large volume of salt water coming up the San Joaquin River could fill the alignment creating a rapid increase in salinity at the pumps. This consequence could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Middle River at Columbia Cut, and the barriers on Connection Slough and Woodward Canal at Middle River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.11 Barrier on Woodward Canal at Middle River

This one-way barrier serves primarily to exclude salt water from the alignment that could otherwise be drawn in, should a significant seismic event occur in the vicinity. It potentially could be utilized during the low flow season to control salinity at the pumps on an as-needed basis, as well. Without this barrier, during such an event, salt water coming up Old River could fill the alignment creating a rapid increase in salinity at the pumps. This consequence could result in longer system downtimes, due to poor water quality. This barrier is intended to be operated during a seismic event, in concert with barriers on Middle River at Columbia Cut, and the barriers on Connection Slough and Railroad Cut at Middle River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.2.4.12 Barrier on Old River at Italian Slough

This one-way barrier serves primarily to exclude salt water from the alignment that could otherwise be drawn in, should a significant seismic event occur in the vicinity. It potentially could be utilized during the low flow season to control salinity at the pumps on an as-needed basis, as well. This barrier would be included in the final alignment only if the proposed siphon crossing on Old River were not implemented. If a siphon on Old River were utilized, the alignment would be isolated from salinity intrusion from Old River and the barrier would not be needed. Without this barrier, or a siphon, salt water coming up Old River could fill the alignment during a seismic event, creating a rapid increase in salinity at the pumps. This consequence could result in longer system downtimes, due to poor water quality. The barrier is intended to be operated during a seismic event, in concert with barriers on Middle River at Columbia Cut, and the barriers on Connection Slough, Railroad Cut, and Woodward Canal at Middle River, but only if no siphon is built across Old River. Individual water quality sensitivity analyses have not been performed on this barrier to quantify its impact in isolation.

17.3 Construction Methodology

The barriers would be constructed in multiple stages during summer low flow periods. A closed steel sheet pile cofferdam would be constructed across part of the channel, leaving the remainder of the channel open to pass natural flows. The structure would be constructed within the cofferdam after dewatering. The configuration of the cofferdam would include the upstream and downstream retaining walls adjacent to the main structure. When part of the structure is competed, the cofferdam would be removed and a new cofferdam installed for the next adjacent section to be constructed. Water flowing in the channel would pass through completed structure bays and through any open natural channel that is not blocked by cofferdam. It is possible that some of the longer structures could require multiple construction seasons to complete. However, construction through the winter high flow periods is not anticipated. Additional temporary cofferdams may also be required upstream and downstream of the deeper gate bays after the entire structure is completed to facilitate dewatering for the installation of the gate panels in each bay.

A work area of up to 15 acres could be required in the vicinity of each barrier structure. This area would be needed for the temporary storage of materials (sheet piling, foundation piling, etc.), concrete form fabrication, possible field fabrication of miter gate panels, stockpiles, office trailers, shops, construction equipment maintenance, and the like.

17.4 Maintenance Considerations

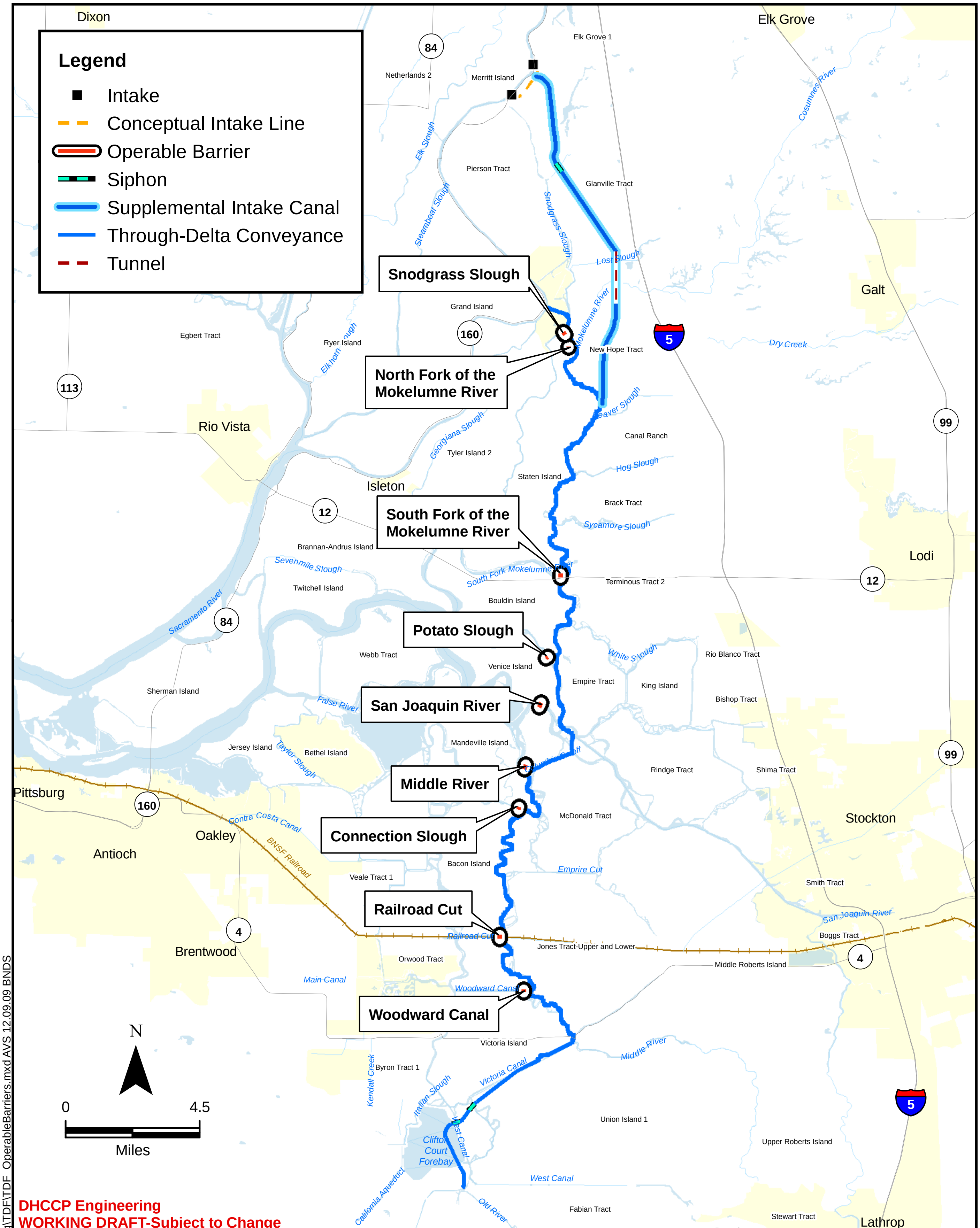
The miter gates would require routine annual inspection of coating systems, cathodic protection systems, gate seals, panel alignment, hydraulic cylinders, and other associated equipment. An important aspect of the inspection would be to locate and repair any visible fatigue cracking in structural members or welds and repair any coating failures.

Some gates may not be required to operate for long periods of time. For this reason, all such gates should be fully exercised at least two times per year, including one test before the onset of the wet season each year.

Each gate bay should be inspected annually at the end of the wet season for the accumulation of sediment on the bottom that could impede gate operation. Sediment should be removed during the summer.

To facilitate maintenance during the summer, each miter gate bay is equipped with stop log guides in the walls and pockets in the floor slab for steel stop log posts. At low water periods for the smaller gate installations, these could be used to dewater individual bays for inspection and maintenance. Major maintenance on deep gate bays could require a temporary cofferdam upstream and downstream of the bay to facilitate dewatering.

Routine maintenance would also include periodic inspection of oil reservoirs and the condition of the accumulator bottles/tanks that would operate the system in the event of a power failure.



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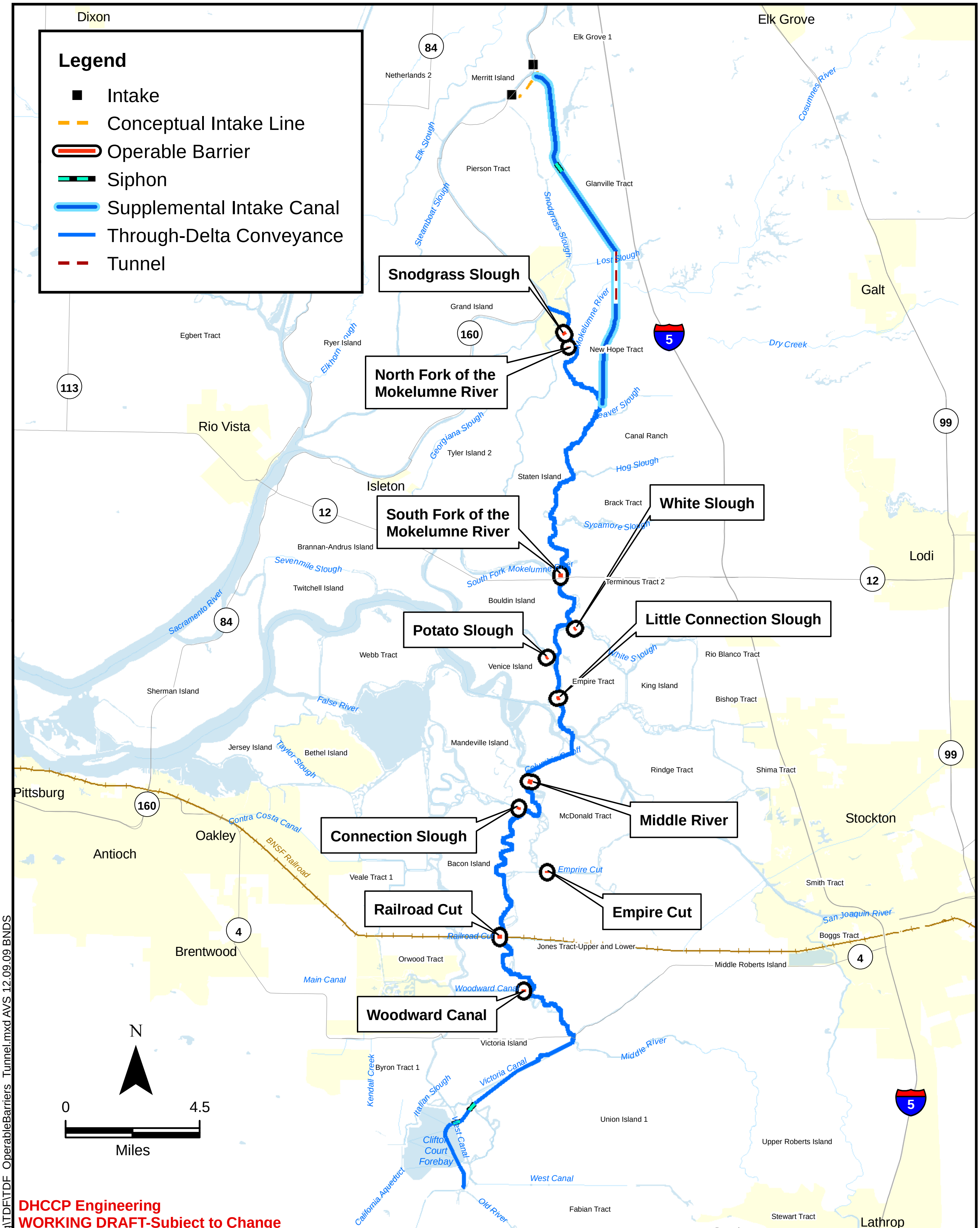
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Operable Barriers Location Plan

FIGURE
17-1



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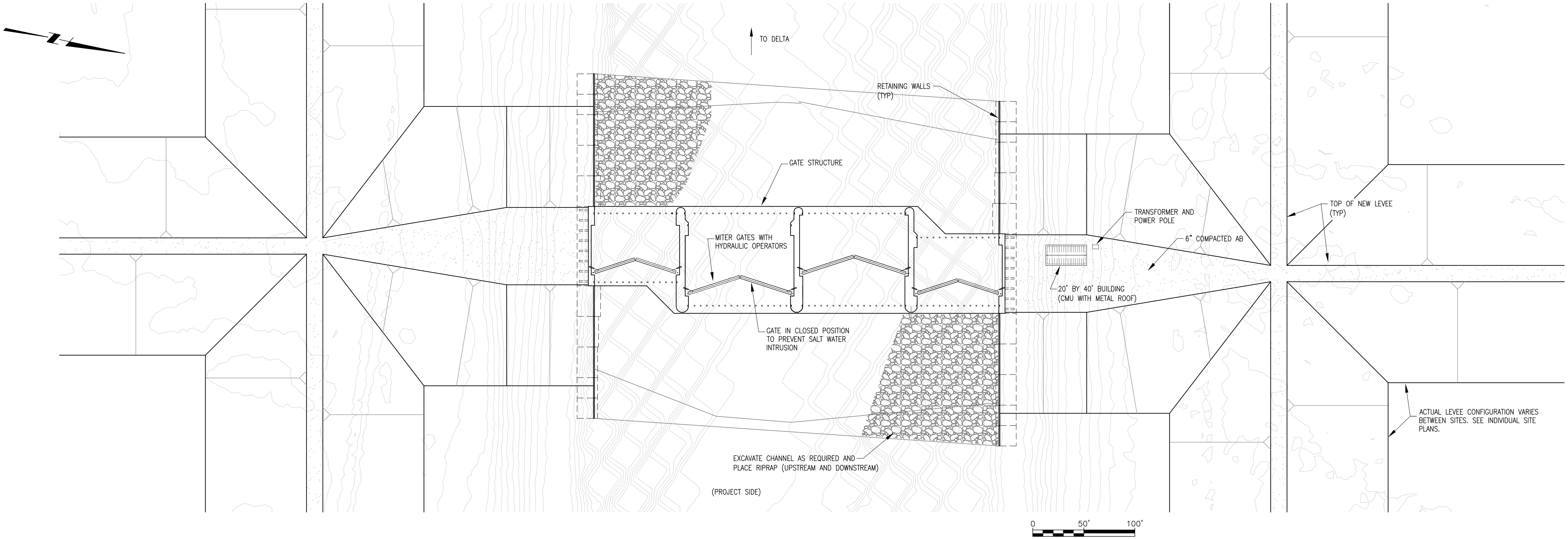
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

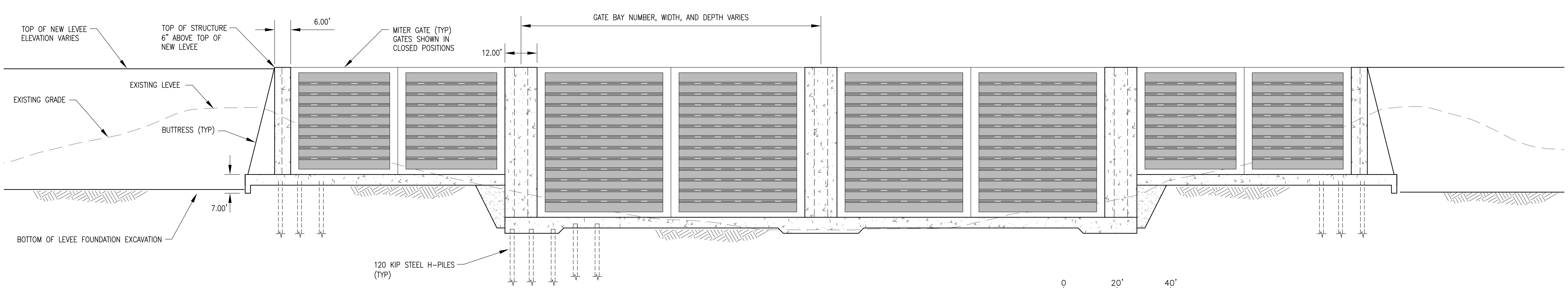
Operable Barriers with Tunnel Option Location Plan

FIGURE **17-2**



PLAN – SINGLE BARRIER GATES

SCALE: 1" = 50'-0"



PROFILE – SINGLE BARRIER GATES

SCALE: 1" = 20'-0"

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THROUGH-DELTA FACILITY OPTION

Single Barrier - Typical Plan & Profile

FIGURE
17-3

18.0 CONTROLS AND COMMUNICATIONS

The facility control system can provide local and remote automatic and manual control and monitoring of the facilities.

18.1 Description and Site Plan

Sites along the TDF Option alignment that would communicate with the SCADA system include:

- Two optimal supplemental intake pumping plant structures
- Optional SJRTPP
- VCPP
- Operable barriers
- Each siphon and tunnel structure (each of which would include radial gate control structures, gate position monitoring and control, and monitoring of both upstream and downstream water surface level)

Each of these sites includes control and monitoring equipment for the site and can communicate with the Joint Operations Center (JOC) located in Sacramento, as well as the Area Control Center (location to be determined).

The communications system is planned to be implemented using a mix of fiber optic cable in conduit, and potentially microwave radio or leased Telco lines. The communications would connect to the Delta Field Division O&M Center on the south side of the alignment, and DWR Headquarters at 9th and O Streets on the north side of the alignment. The existing communications system provides project communications to the JOC and the Area Control Center (location to be determined). The communications system forms a ring by using the existing communications system, providing redundant paths for reliability.

Buried fiber optic cable installed in conduit would be utilized along all canals, levees, and tunnels being constructed for this alignment.

Communications would be provided between the intake pumping plant and the DWR Headquarters at 9th and O Streets in Sacramento. The method of communications would be fiber optic cable buried in conduit, microwave radio, or via leased lines from a telecommunications provider.

If fiber optic cable in conduit is utilized for this segment, the conduit route is to be determined, but would run adjacent to roads, highways, railroads, utilities, or other easements.

If microwave radio is utilized for this segment, parabolic antennas would be located on the roof of the 9th and O Street building, and at the intake pumping plant. The antenna at the 9th and O Street building would be mounted directly on the building roof adjacent to existing antennas. The antenna at the intake facility would be installed on a new antenna mounting pole located on the levee. The height of this pole would be determined via a radio propagation study. Preliminary investigation indicates the pole would be approximately 50 feet high. The feasibility of a microwave radio link would need to be determined following verification of the availability of frequencies to be licensed by the Federal Communications Commission, performing a path propagation study, and investigation of any potential new building construction planned in the intended path.

Buried fiber optic conduit is planned to run from the south end of the new canal to the Delta Field Division O&M Center. The conduit would run adjacent to CCF, along the inlet canal to Banks Pumping Plant, then to the Administration Building at the Delta Field Division O&M Center.

A global positioning satellite (GPS)-based time clock located at each pumping plant would support the control system. This equipment requires a small dish antenna mounted on the roof of the pumping plant, and provides time-reference data for data-collection purposes.

In addition, two satellite-based clocks, with similar small dish antennas support the communications system. The two communications clocks would be located at the VCPP, and the northernmost intake.

18.2 Control Modes and Control Basis

In general, all equipment would operate in local and/or remote control modes.

- **Local.** In the local mode, the equipment is manually controlled locally or from a nearby MCC, Switchgear, VFD, local control panel, valve actuator or hand station.

  For operable barrier sites and siphon and tunnel crossing structure sites, the local-remote switch enables local control from an operator interface terminal located on the PLC panel.

  When equipment is in local mode, all remote-mode control of the equipment is disabled.

- **Remote.** In the remote mode, the equipment is controlled through the PLC based on automatic control strategies, commands issued from any plant control system (PCS) workstation located at the facility, or commands issued from the AREVA workstations located at the Area Control Center or Project Operations Center (located within the JOC). A software-based jurisdiction scheme is used to coordinate controls from PCS workstations located at the facility and commands issued from AREVA workstations located at the Area Control Center or Project Operations Center.

The control mode would be selectable, where applicable, based on local/remote switches located at the field equipment, or local control panels. Selector switch position feedback would be wired to the PLC, allowing an operator using the operator workstation to know whether control from the operator workstation is active.

Some non-process equipment (e.g., sump pumps and HVAC equipment) may be provided with local manual controls only. This includes equipment such as sump pumps and HVAC equipment.

18.3 Construction Methodology

Control and communications equipment is installed inside buildings or in outside electrical panels, where required. Panels are installed adjacent to facilities and structures. Equipment security and protection from vandalism would be considered during the design phase.

Specific considerations are as follows:

- **Intake Facilities.** Conduit with fiber cable are run in the same trench as the intake pipelines to the north end of the new canal. Conduit with fiber cable runs on both sides of the pipeline. Pull boxes along pipelines are buried after fiber installation.
- **Canals.** A fiber optic conduit system with fiber cable is installed on the top of the canal, adjacent to the access road on both sides of the canal.
- **Culvert Siphons – Shallow Crossings.** The fiber optic conduit system is buried in the trench with the system structures. Conduit with fiber cable is be placed on both sides of the structure.
- **Tunnels – Deep Crossings.** A conduit with fiber cable is be placed inside each parallel tunnel bore after the concrete lining is installed. The conduits are attached to the side walls of the lining.
- **Levees.** The fiber optic conduit system is planned to be installed along both sides of the conveyance during levee construction, near the top of the setback levees. For the eastern setback levees, the conduit would cross sloughs through horizontal directional drilling. For the western setback levees, the conduit would cross sloughs as described for the operable barriers. Pullboxes would be buried after fiber installation.
- **Operable Barriers.** Each operable barrier would be connected to the SCADA system via the fiber optic cable system. The fiber optic conduit system would be installed adjacent to the operable barriers.

18.4 Maintenance Considerations

The existing SWP is operated by common controls and communications systems. To maintain a common operational platform, the SCADA and communications systems for this new conveyance facility concept is an extension of the existing system maintained in the same manner as the existing systems.

19.0 POWER SUPPLY AND GRID CONNECTIONS

Electric power is required for the optimal supplemental intake pumping, optional San Joaquin River Tunnel pumping, Victoria Canal Fish Salvage Facility pumping, and gate controls at the entrances of each of the control structures and inverted siphons along the alignment.

19.1 Description and Site Plan

For this alignment, the total intake pumping electrical demand is estimated at approximately 7 megawatts (MWs). Additionally, the optional SJRTPP electrical demand is estimated at 33 MWs, and the VCPP electrical demand is estimated at 33 MWs. Including non-pumping loads (e.g. security lighting, communications, controls, and movable barriers) the total conveyance option electrical demand is estimated at 77 MWs.

Electrical power for the entire project would be delivered through a single 230-kV transmission line, owned by either the utility or the project, which would interconnect with a local utility at a new utility substation assumed to be constructed within or adjacent to the utility's existing transmission ROW. Some utility grid reinforcement and upgrade is likely to be needed in order to accommodate this large new pumping load. The transmission line would terminate at the conveyance option's main 230-kV substation, which would be located adjacent to the optional SJRTPP.

At the conveyance option's main 230-kV substation, electric power would be transformed from 230 kV to 69 kV and delivered to the adjacent main 69-kV substation. From there, power would be distributed north to the two intake substations near Hood, and southwest to the VCPP Substation over 69-kV subtransmission lines constructed along the TDF Option on wood poles. At the various substations, electric power would be transformed from 69 kV to the voltage needed for the pumps and auxiliary equipment at the various pump plants.

To supply power for communications, monitoring, and control of the movable barriers in the channels and the gates at the tunnel and siphon entrances along the canal, 12-kV distribution lines would be extended north and south from the main 69-kV substation, and north from the VCPP substation. Wherever possible, this 12-kV line would be constructed on the same poles as the 69-kV subtransmission line.

The electrical power grid connection plan is illustrated on Figure 19-1, and shown schematically on Figure 19-2.

For this alignment, there are three utility grids which could supply power to the project: PG&E (under the control of CAISO), Sacramento Municipal Utility District (SMUD), and WAPA. Physical interconnections to the electrical grid are being analyzed and the results would be included in a future document. The electrical power needed for the conveyance facilities would be procured in time to support construction and operation of the facilities. As the operator of the SWP, DWR is an active participant in the activities of the California electric grid, from long-term planning to day-to-day operation. The DWR Planning and Risk Office leads the process of identifying, evaluating, and establishing the electrical interconnection for DWR projects to the California electric grid.

19.2 Construction Methodology

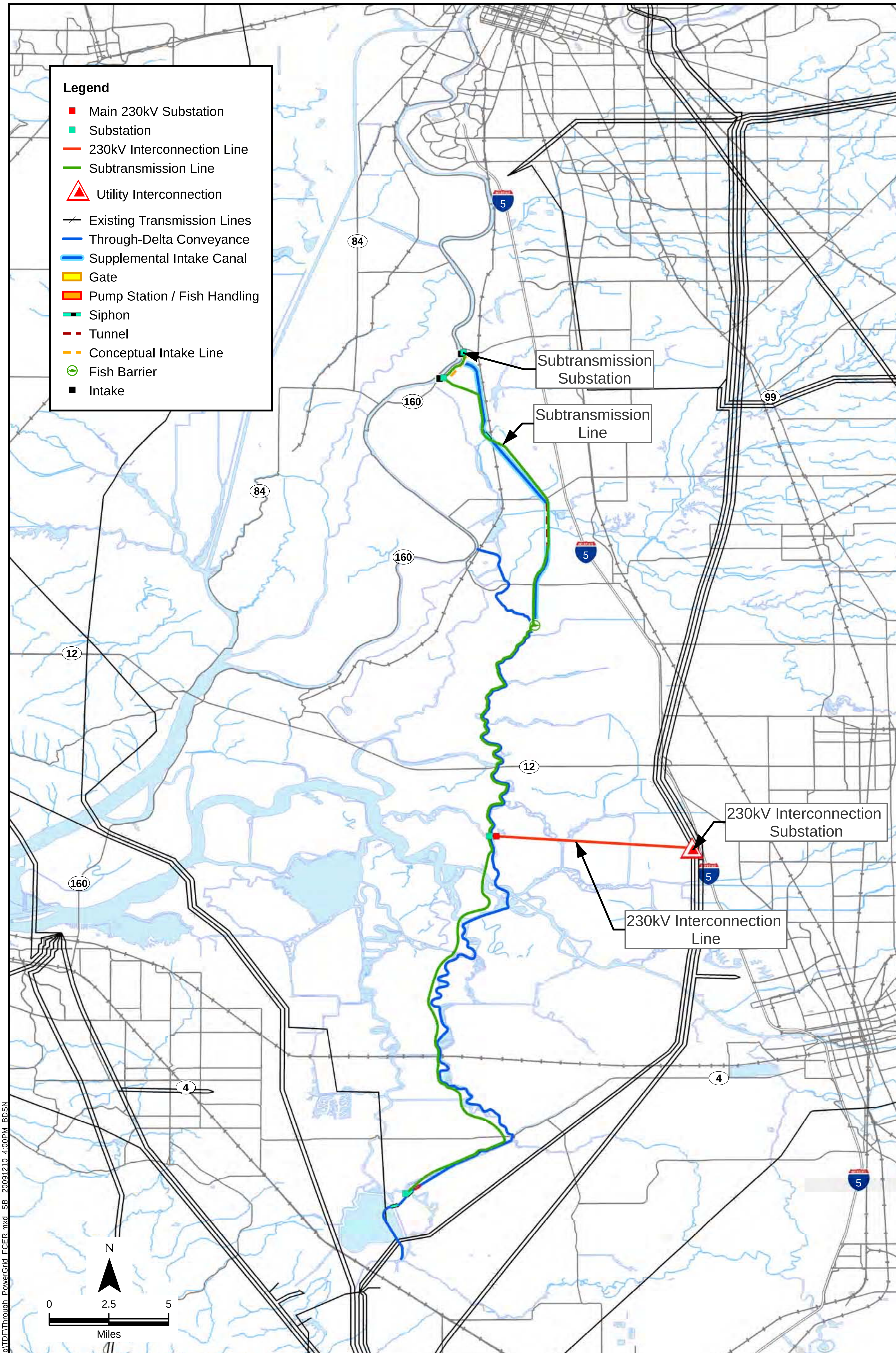
In order to supply power during construction of the intake and pumping plant structures, and power for the tunneling and canal dredging machines, it is desirable to construct the project main 230-kV substation, the main 69-kV substation, the 230-kV transmission interconnection line, the intake substations, and temporary 69-kV lines between the main 69-kV substation and the intake substations to the north, and the VCPP Substation to the south, early in the construction schedule. Additionally, temporary power lines would be needed to provide power for levee and canal construction (draglines) and tunnel boring equipment. Once the canal construction is completed, the temporary power lines would be relocated to their final routes alongside the waterways and within the canal and alignment ROW.

19.3 Grid Interconnection Reliability Discussion

The plan for electric power supply to this project would utilize a single 230-kV transmission interconnection to a suitable local utility. This places the entire project at risk of a power supply outage if the 230-kV transmission line fails or is interrupted, or the local utility has an outage affecting the interconnecting substation. However, the cost of a second, independent interconnection to improve delivery reliability could be weighed against both the likelihood of such an interruption, and the consequences of an interruption. For 230-kV transmission lines, the typical outage frequency (North American Electric Reliability Corporation [NERC] Transmission Availability Data System) is 0.1527 outages per circuit per year, which is equivalent to one outage in 6.55 years. For such outages, the mean duration is 30.32 hours. Together, these two statistics suggest that by relying on a single transmission interconnection to the utility grid, the project could expect to lose its power supply roughly once every 7 years for 30 hours. While the project is not expected to operate at its full capacity of 15,000 cfs at all times, for the purposes of a reliability assessment, it would be prudent to assume that the transmission interconnecting line failed while the project was attempting to deliver at full capacity.

Based on an assessment of the Sacramento River flows, it is estimated that full project flows could only be attempted approximately 22% of the time, or roughly 80 days per year. Thus, a transmission line failure lasting 30 hours would eliminate 30 hours of full-flow diversion out of 80 days. Assuming that there would be no opportunity to make up the lost full-flow diversions, the project’s annual water delivery capability would be reduced by approximately 1.6 percent. However, during an outage of this project, some pumping capability from CCF can still be maintained at the Delta pumping facility. Furthermore, while there would only be a single 230-kV transmission line interconnecting the project with the utility substation, the utility substation would likely be connected to the grid by way of two or more transmission lines, increasing the likelihood that the utility substation would be operational. Finally, unless the interconnecting transmission line is physically damaged, such as with multiple towers collapsed, repairs to transmission lines can typically be completed in a very short time.

To reduce the likelihood of loss of electrical power to the project, a second interconnecting transmission line would be needed, extending to a second utility substation along a different route than the first transmission line (to eliminate the chances of a single disaster damaging both lines). The cost to construct a second, independent interconnection would more than double the cost of the transmission line. While the reliability of water delivery to the SWP is important, this project is not critical to that water delivery on an hour-to-hour basis, but rather to the overall long-term quality of that water delivery and to the protection of species and habitat in the Delta. A sustained outage of the project would reduce the benefits that the project would provide, but it would not interrupt water deliveries through the SWP. Thus, a single line interconnection should provide sufficient reliability to meet project needs.



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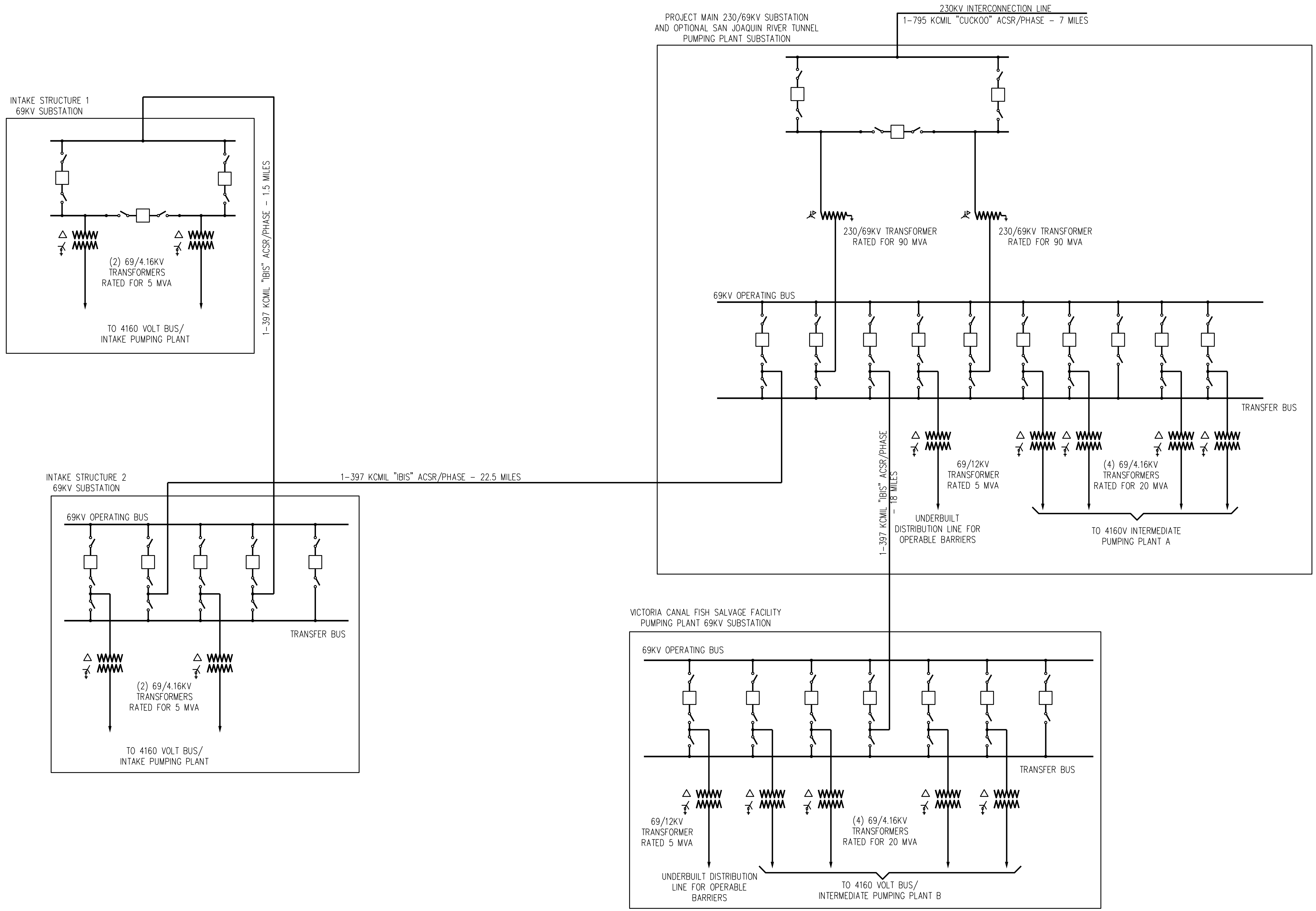
CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY CONVEYANCE OPTION

Electric Power Grid Connection Plan (Optional)

FIGURE
19-1



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CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Electric Power Grid
One Line Connection Diagram

FIGURE
19-2

20.0 CCF AND VICTORIA CANAL MODIFICATIONS

A new fish salvage facility is proposed to improve the protection of endangered fish species. Anecdotal reports indicated that greater than 80% of juvenile salmonids and juvenile/adult smelt entering CCF do not survive. Fish mortality is said to be primarily attributed to predation occurring in the forebay between the entrance to the gates and the Skinner Facility. A smaller component of fish loss is caused by the export pumping and salvage operations required to relocate the screened fish to other points in the Delta. The approach to improving fish survival focuses on preventing their entry into the forebay using more modern fish screen and salvage facilities. Redirecting the Victoria Canal directly into the forebay, isolating Victoria Canal from Old River, constructing a new fish screening facility on the realigned section of Victoria Canal, and closing the existing inlet gate structure to CCF at the southeast corner would prevent fish from the forebay entering. The existing SWP (Skinner) and CVP fish salvage facilities would continued in operation in the TDF Option to screen any fish remaining in the CCF.

20.1 Description and Site Plan

20.1.1 Site Layout

The proposed plan for preventing fish entry into the forebay includes a new fish screening facility constructed on a realigned and isolated (from Old River) Victoria Slough with a new spillway directly into the forebay (see Section 6.0 for details on the fish screening facility). The approach canal to the Jones Pumping Plant would be modified to include a new canal to provide access to forebay water. Figure 20-1 illustrates the proposed concept for CCF modifications. The new Victoria Canal CHTR facility would be located on the northeastern side of CCF on Union Island. Figure 20-1 illustrates the features required to develop a valid CHTR facility.

The proposed CCF modifications are described below.

20.1.1.1 Realign Victoria Canal into CCF

Victoria Canal would be realigned starting at approximately 4,000 feet before the confluence with the Old River and redirected approximately 10 to 15 degrees to the southeast to accommodate an inverted siphon crossing under Old River at a narrow point. The siphon would discharge to an approximately 3,200-foot-long earthen canal which would cross Coney Island to a second inverted siphon passing under West Canal and into a new outlet structure constructed on the bank of CCF. This canal would become the only inlet into CCF and would be sized to accommodate the total combined design flow rate for the SWP Banks Pumping Plant (10,300 cfs) and the CVP Tracy Pumping Plant (4,600 cfs). The 4,000 feet segment of the Victoria Canal that connected to the former Old River would be filled in with earth fill to eliminate the dead end section of Victoria Canal resulting from the realignment geometry.

20.1.1.2 Construct a New CHTR Facility on the Realigned Victoria Canal

The CHTR facility (described in Section 6.0) would be a single facility located on the realigned segment of Victoria Canal to capture all fish that follow the flow direction into CCF. Earthen embankment construction and a new access road would be required to the facility. The alignment of the access road has not been determined.

20.1.1.3 Construct a New CCF Inlet Pumping Plant on Victoria Slough

A new pumping plant would be required to convey water from Victoria Canal to CCF. The pumping plant would be located just downstream of the new fish screens. The pumping plant would be similar in design to the canal pumping plant located at the north end of this conveyance option, other than it would be twice as large from a facility footprint viewpoint.

20.1.1.4 Close the Existing Inlet Structure Located at the Southeast Corner of CCF

The existing inlet structure would be closed to allow a single point inlet through the realigned Victoria Canal. (There should not be any new fish entering CCF after these improvements.)

20.1.1.5 Reconfigure the Jones Pumping Plant Intake Canal

Intake control would be accomplished by constructing an approximately 6,000-foot-long cross connection canal between the Jones Pumping Plant intake canal (Delta Mendota Canal) and CCF to provide access to the water in the forebay. A new gate and control structure would be constructed on the cross connection canal to control the source of water to the Jones Pumping Plant.

20.1.1.6 Maintain Operation of the Existing CHTR Salvage Facility (Skinner Facility)

Operation of the existing CHTR salvage facility could be maintained under this proposed configuration.

20.2 Construction Methodology

CCF Fish Facilities project includes several separate construction elements The primary construction methodologies required for each element are presented Table 20-1.

Table 20-1: Construction Elements for CCF Upgrades

Construction Element	Construction Elements
Realign Victoria Canal Into CCF	Install cofferdam or sheet pile wall at junction of Old River and Victoria Canal during construction to prevent flooding. Earthwork required to construct new canal. Two new inverted siphons: one under Old River, and one under West Canal Inlet structure at CCF (construct concrete or earth embankment). Dredging in CCF near the inlet to support flow capacity of the canal.

New CHTR Facility	Installation of inlet screens across 500-foot canal width. Six chevrons installed in stream across 500-foot canal width. Fish handling facility adjacent to canal. Two-lane access road from facility to adjacent roadway (location to be determined)
New CCF Inlet Pumping Plant (Victoria Slough)	See other sections for construction elements.
Close the existing inlet structure located at the southeast corner of CCF	This should be completed by raising the existing gate, and would require no new construction.
Reconfigure the CVP Tracy Pumping Plant Intake Canal	Install new 6,000-foot-long canal between the Jones and Banks Pumping Plants intake canals. Earthwork to construct cross connection canal. New gate control structure in cross connection canal.
Maintain Operation of the existing CHTR Salvage Facility	Operation and maintenance of facility (in-stream and on land facilities).

CCF = Clifton Court Forebay
CHTR = collection, handling, transport, and release
CVP = Central Valley Project
Skinner Facility = Skinner Delta Fish Protection Facility

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CONCEPTUAL ENGINEERING REPORT
THROUGH-DELTA FACILITY OPTION

Overall Forebay Plan

FIGURE
20-1

21.0 BORROW SITES

Borrow materials are required for canal embankment construction, setback levee materials, in-river rock slope protection (RSP), agricultural soil for vegetation restoration, and haul roads. The primary borrow material need is soil suitable for use as engineered embankment fill, but rock, gravel, and sand would also be required. The purpose of this section is to identify general areas that could prove suitable as sources of borrow material in the general vicinity of the proposed construction. Potential sources of borrow material were screened on the basis of suitable geotechnical properties and physical settings that may be practically mined and the material readily transported to construction sites.

21.1 Description and Site Plan

21.1.1 Suitable Sources of Borrow Material

Identifying sources of suitable borrow material is an iterative process that continues until initiation of construction activities. The initial search for borrow sites was guided by the following assumed criteria:

- Borrow material can have between 20 and 80% fines (i.e., material passing a #200 sieve).
- Borrow material should have a liquid limit less than 50 (ASTM D4318).
- Borrow material should not require post-excavation processing.
- Borrow material should be exposed at surface and require no, or very limited, overburden removal.
- Borrow source areas should be as close as possible to the construction site.
- Borrow areas should be of sufficient size to accommodate large excavation and material handling equipment.
- Borrow areas not immediately adjacent to construction areas should be in close proximity to transportation facilities capable of handling the anticipated quantity of borrow produced.
- Total amount of borrow material for engineered fill is approximately 200 million cy (bank yards).
- The multiplier to convert “bank yards” to “truck yards” is 1.3.
- The multiplier to convert “bank yards” to “yards compacted in place” is 0.75.
- Borrow areas should not require dewatering prior to or during excavation.
- Borrow areas should be selected to minimize the impact or encroachment on existing surface and subsurface development.

Based on these criteria, the project area and the surrounding area were screened to identify potential sites for borrow material. Borrow sites within the project area were identified based on geologic data presented through the DRMS study. Borrow site locations identified outside the project area were based on reviews of published geologic maps. The regional geologic map series published by the California Division of Mines and Geology was used (Map No. 1A Sacramento Quadrangle and Map No. 5A San Francisco – San Jose Quadrangle).

21.1.2 Potential Borrow Sources

The soils in the area surrounding the proposed alignment are generally characterized by floodplain deposits consisting of clayey soils with various amounts of sand, silt, and peat. Starting with soils maps of the area developed for DRMS (DWR, 2007e), potential target areas were identified for borrow material for engineered fill, based on soil properties and practicality of mining and transport. Areas dominated by highly plastic or organic soils (e.g., those soils within the USCS groups CH, MH, OH, PT, and OL) were eliminated from consideration as a source of engineered fill (Office of Surface Mining, 1998). For the purpose of this study, a highly plastic soil is defined as having a liquid limit of greater than 50 (ASTM D4318). Soil types not excluded based on the above criteria are deemed potentially suitable for use as engineered fill. Further refinement of the target soil types would be conducted as more information becomes available, including geotechnical data.

The following must be avoided when mining and transporting target soils from new local or existing commercial sites:

- Extensive dewatering operations.
- Landfarming to reduce the moisture content of the existing soils.
- Significantly impacting existing drainage patterns, including engineered drainage.
- Significantly impacting existing development or infrastructure.
- Significantly impacting cultural and environmental resources.

Based on the impracticality of mining and transport, potential borrow source areas have been excluded from further consideration as potential engineered fill borrow areas. These areas include locations:

- More than 10 miles from the alignment being considered.
- Within 100 feet of existing residential or commercial development.
- Within 100 feet of a military installation.
- Within 100 feet of existing roads, rail lines, levees, waterways, and utilities that can be identified on current aerial photography of the project area.
- With an estimated size less than 100,000 square feet.

These criteria would be refined as more information becomes available. Due to the presence of a shallow groundwater table throughout the project area, individual potential borrow areas in the immediate vicinity of the facility tend to have limited quantities of usable material. Therefore, an alternative method for sourcing borrow material is proposed to take advantage of the economics of scale. Three potential borrow source areas for engineered fill have been identified outside the project area (Bald Hills, Bear Creek, and Montezuma Hills), and one potential source of clay borrow has been identified within the project area (Clarksburg). Each of these areas encompasses a contiguous area of a few square miles and potentially has sufficient reserves to provide borrow material for construction of the entire project. Final selection of a source for borrow material is dependant upon field investigations to evaluate the geotechnical quality of the source material and other factors, such as environmental considerations and excavation, handling, and transportation methodology.

These areas would be evaluated as potential borrow areas once quantities are defined (including DWR's Port of the Stockton facility). The two greatest unknown variables at this stage of the investigation are the groundwater level at potential borrow sites and the volume of borrow material required at various construction locations. Both of these factors need to be resolved before specific borrow locations and excavation configurations can be evaluated. In addition, the presence of hazardous materials or environmental working conditions in or adjacent to potential borrow material sites was not evaluated.

It is anticipated that imported durable rock would be needed for in-river repairs. Both RSP and bedding materials would be needed. The maximum size of RSP is estimated to be 400 pounds (California DOT, 2000), and the maximum diameter of bedding rock is 4 inches. Crushed rock would also be needed for all-weather haul roads that may be required and for work pads at construction sites, such as tunnel portals and siphons to enable all-weather construction activity. The most cost-effective source of crushed rock and aggregate likely would be existing commercial operations with the capability to transport the material as close as possible to the location where it is needed.

Borrow material for construction of setback levees in the vicinity of Walnut Grove would come from areas of New Hope Tract and areas immediately east of Georgiana Slough. Borrow material for setback levees between Beaver Slough and White Slough would be obtained from areas along the eastern alignment and the Interstate 5 corridor. Material may also be barged from selected areas of the Delta where suitable soils can be excavated or from the Montezuma Hills or Clarksburg contiguous borrow areas, or trucked from the Bear Creek contiguous borrow area.

Borrow material for setback levees south of the San Joaquin tunnel crossing to the Victoria Canal would be obtained from locations near the levees or from one of the three borrow source areas outside the project area (Montezuma Hills, Bear Creek, or Bald Hills).

Table 21-1 characterizes the geologic units that outcropped in the general vicinity of the project area that are considered most likely to provide suitable source material for engineered fill, based on the criteria listed above.

Table 21-1: Summary of Potential Borrow Source Characteristics

Unit Name	Symbol	Age	Description	General Location	Potential Borrow Material	Suitability for Borrow	Rippability	Construction Considerations
Yuba River Gold Fields	YGF	Modern	Well-graded gravel.	East of Yuba City.	Gravel	High	High	Unit consists of washed river rock, which may not be suitable for many applications.
Floodplain Basin Deposits	Qb	Holocene	Fine-grained silt and clay derived from the same sources as modern alluvium. Distal facies of unit Qa. Thickness varies from 1 or 2 meters to 60 meters.	Found throughout the Sacramento and San Joaquin Valleys; prevalent in the Delta. A number of different quaternary deposits have been grouped with this single unit based upon similar geotechnical characteristics as potential borrow material.	Silt and Clay	Variable	High	Most areas underlain by Quaternary basin deposits have extensive surface development, either agricultural or urban. Localized units may have highly variable grain-size distribution. Although satisfactory borrow sites may exist throughout this formation, generally in pre-historic fluvial channels, the reserves at a specific location are typically limited. Depth to groundwater is highly variable. The highly variable nature of this unit over short distances indicates this unit would not be a suitable source for large quantities of borrow material.

Unit Name	Symbol	Age	Description	General Location	Potential Borrow Material	Suitability for Borrow	Rippability	Construction Considerations
Modesto Formation (alluvium)	Qm	Late Pleistocene	Gravely sand, silt, and clay.	Alluvial deposits in the center of the Sacramento and San Joaquin Valleys.	Sand, Silt, Gravel, and Clay	Medium	High	Shallow groundwater is also associated with this unit in some areas. Dewatering of even small borrow areas would likely be required and there is a potential for cross-contamination of near-surface aquifers.
Montezuma Formation (poorly consolidated, clayey sand)	Qmz	Early Pleistocene	Poorly stratified clayey sand and pebbly sand.	Montezuma Hills, southwest of Rio Vista.	Clay and Sand	High	High	The Montezuma Hills property is currently owned by an environmental land trust. Purchase of alternative property would probably be required. A 500-kV line transects the property, which also overlies the producing Rio Vista gas field. There are numerous producing gas wells and collection piping that would need to be addressed.
Turlock Lake Alluvium	Qtl	Early Pleistocene	Sandstone, siltstone, and conglomerate derived mainly from Sierran granitic and metamorphic rocks; non-marine. Also includes Corcoran Clay.	Eastern edge of the Sacramento and San Joaquin Valleys.	Sand, Silt, Gravel, and Clay	Medium	Medium	Would require a large surface area be excavated. The unit is thin and located in areas with little relief. However, the available property is generally not developed, and existing railroad lines border the northern portion of the property.

Unit Name	Symbol	Age	Description	General Location	Potential Borrow Material	Suitability for Borrow	Rippability	Construction Considerations
San Pablo Group (marine sediments)	Msp	Late Miocene	Sandstone, mudstone, siltstone, and shale with minor quantities of tuff.	Southwestern border of Sacramento and San Joaquin Delta area.	Sand and Silt	Low	Low	A substantial amount of processing may be required to achieve the desired grain size distribution.
Upper Cretaceous Marine Sedimentary Rocks	Ku	Late Cretaceous	Sandstone and shale.	West of Clifton Court Forebay.	Sand	Low	Low	A substantial amount of processing may be required to achieve the desired grain-size distribution.
Panoche Formation	Kp	Late Cretaceous	Sandstone, shale, siltstone, conglomerate lenses; marine.	West and Southwest of Clifton Court Forebay.	Sand, Silt, and Gravel	Low	Low	Property underlain by this unit is currently developed into several large-scale wind power farms. A substantial amount of processing may be required to achieve the desired grain-size distribution.
Franciscan Complex (melange)	Kjf	Late Cretaceous to Jurassic	Melange, greenstone, sandstone, shale, conglomerate, metagraywacke, limestone, chert, serpentinized ultramafic rock.	Coastal Ranges west of Interstate 5 and south of Interstate 580.	Sand and Gravel	Low	Low	Mineral composition, degree of lithification, and grain-size distribution make this unit unsatisfactory for use as engineered fill, RSP, or crushed rock.

21.2 Construction Methodology

Conventional earthmoving equipment, such as bulldozers, loaders, and scrapers, would be used to excavate the borrow material above the groundwater table. In the event that sufficient quantities of borrow material above the groundwater table cannot be obtained, temporary dewatering operations in selected areas may be considered where this would be more economical than hauling borrow material distances of more than 10 miles from the construction site. Timing of dewatering and excavation of dewatered borrow pits could be coordinated with the placement of spoil in the borrow excavation to prevent the creation of new wetlands. If one of the large contiguous borrow source areas is employed, then it is unlikely that dewatering operations would be required and large scale mining and excavating equipment could be employed to speed construction, lower excavation costs, and reduce the footprint of the borrow areas for the project.

Spoil generated during construction would be placed in spent borrow areas as part of the restoration process. Spoil, with the exception of tunnel muck and dredge tailings would consist primarily of peat, organic silts, organic clays, and high plasticity clays. Once excavation is underway, it would be possible to coordinate delivery of borrow material with removal of spoil and placement of that spoil within excavated borrow areas. The spoil would not be compacted, but would be graded and seeded. Tunnel muck would require a period of time for additives used in the excavation process to naturally break down before the spoil site can be graded and other restoration activities pursued. The specific restoration process for each individual borrow area would depend upon existing site conditions, such as surface and groundwater elevations, slope angle and direction, local fauna, and anticipated post-construction use.

22.0 SPOILS SITES

Construction activity would generate spoils or materials deemed unsuitable for use as engineered fill may be temporarily or permanently placed for disposal. Significant thicknesses of non-supportive or organic soils would be removed in the course of canal and embankment, siphon, and pipeline construction . In addition, tunnel cuttings consisting of saturated soils mixed with bio-degradable polymers would be generated by tunneling operations.

These materials must be characterized and disposed of appropriately.

22.1 Description and Site Plan

Much of the area surrounding the proposed alignment consists of low-lying floodplain developed as agricultural land. Depending on the properties of the materials to be spoiled, some predominantly organic spoils may be deposited on portions of this land without adversely affecting its agricultural use. Depending on the timing of construction, it may be possible to treat the tunnel materials and reuse them as engineered fill. However, this would require a temporary treatment stockpile. It may be possible to dispose of much of the spoil material by placing it near the landside toes of the canal embankments and as the safety berm along the roads constructed parallel to the landside toes of the canal embankments.

Refinement of potential stockpile areas was based on the potential disruption to existing development, infrastructure, drainage patterns, and cultural and environmental resources. To provide a preliminary elimination of sites based on the impracticality of transport, the following criteria were applied:

- Spoil would be placed in borrow areas created for this project whenever possible
- Spoil areas should be located within 10 miles of the construction feature
- Spoil areas should not be located within 100 feet of existing residential or commercial development
- Spoil areas would not be located within 100 feet of a military facility
- Spoil areas would not be located within 100 feet of existing roads, rail lines, or infrastructure

These distances may be further refined and specific areas identified as more information about the nature and volumes of soil generated becomes available.

Spoils generated from construction of the setback levees would be disposed of in the area between the setback levee and the river, and also along the landside toe of the levee. Spoils generated from tunnel construction underneath the San Joaquin River would be disposed of in areas between the setback levees and Little Potato Slough north of the San Joaquin River and between the setback levees and Middle Slough north and west of Mildred Island, as well as areas adjacent to the landside portion of levees. If borrow material is sourced from one of the large contiguous borrow areas outside the project area all spoil material may be disposed of in the off-site borrow area.

22.2 Construction Methodology

Conventional earthmoving equipment, such as bulldozers and graders, may be used to place the spoil. Spoil, with the exception of tunnel muck and dredge tailings, may be placed on the landside toes of canal embankments and/or setback levees. This may require temporary placement of the soil in borrow pits or temporary spoil laydown areas pending completion of embankment or levee construction. Borrow pits created for this project would be the preferred spoil location. In the event that limited dewatering is required to excavate a borrow pit, construction would be timed to allow placement of spoil in the borrow excavation to prevent the creation of new wetlands, if appropriate.

Spoil placed in borrow pits or other laydown areas would be placed in 12-inch lifts with nominal compaction. Tunnel muck and dredge tailings would not be compacted. The maximum height for placement of spoil is expected to be 12 feet above pre-construction grade and have side slopes of 5H:1V or less. After final grading of spoil is complete, the area would be restored based on site-specific conditions following project restoration guidelines.

23.0 STOCKPILES, HAUL ROUTES, AND OTHER CONSTRUCTION-RELATED ELEMENTS

This section describes a variety of temporary facilities associated with construction activities.

23.1 Stockpiles

Materials to be stockpiled may include:

- Strippings from canal embankment footprints, for possible reuse in landscaping
- Tunnel cuttings that are slated for reuse after treatment for embankment or fill construction;
- Peat spoils for possible use on agricultural land, or as safety berms on the landside of haul roads, or as toe berms on the landside of embankments (cannot be part of the structural section)
- Borrow materials for canal embankments, setback levees, and in-river RSP
- Other materials being stockpiled on a temporary basis prior to hauling to permanent stockpile areas

Such materials can be stockpiled in the construction areas of the proposed canals, or on the landside of canal embankments and setback levees for later use. As noted above, some stockpiles may be used for material conditioning and potential reuse of the material. Temporary stockpile areas may also allow for the staging of deliveries (offloading), for equipment/materials storage, and for temporary field offices for construction.

Site clearing and grubbing, work area limits, and site access to stockpile locations would be developed. Silt fencing and straw bale dikes would be installed, as needed, to address drainage issues. Dust abatement and other environmental concerns relating to stockpiles would need to be addressed. Stockpile areas may require security fences, gates, and/or cameras.

23.2 Haul Routes

Haul routes and access roads consist of three types: wet weather roads, dry weather roads, and existing public and/or private roads. Dust abatement would be addressed in construction areas.

Wet weather roads are required for year-around construction at concrete and steel structures including tunnel portals, tunnel shafts, pumping plants, intakes, siphons and bridges, and for access to delivery areas and permanent spoil piles.

Dry weather roads are required for construction activities restricted to the dry season (i.e., weather permitting). This construction includes:

- Earthwork structures built without wet weather roads
- Roads from borrow areas to embankment and/or setback levee construction areas

- Other earthwork-related construction, such as material imports, permanent spoil piles, and temporary stockpiles

Typically, the surface for a dry weather road consists of a minimum of 12 inches of gravel. This thickness may be increased over peat or in other wet areas.

Existing public and/or private roads would be utilized as needed for year-round access to all of the construction areas.

23.3 Laydown Areas

Laydown areas would be needed for all elements of proposed construction. For embankment or setback levee construction, the laydown areas would be located along the embankment alignments or in the canal areas themselves, and would not require all-weather access. Laydown areas for tunnel construction would need to be accessible in all weather conditions for as long as such construction is ongoing. Laydown areas for other facility structure construction are expected to require all-weather access depending on the construction schedule for each structure. Laydown areas may require security fences, gates, and/or cameras.

24.0 CONSTRUCTION AND CONSTRUCTABILITY CONSIDERATIONS

24.1 Overview

Anticipated construction methodologies have been provided in previous sections for each project facility or feature. This section presents an overview of the preliminary tasks to facilitate construction, discusses factors affecting the constructability and selection of the various construction methodologies, and identifies other aspects affecting construction.

24.2 Preliminary Construction Tasks

After contracts are awarded and prior to any major construction activity taking place, each successful bidder or contractor has to (1) acquire the necessary permits and prepare plans (e.g., stormwater pollution prevention plan [SWPPP], safety plan, work plan, site layout); (2) mobilize; and (3) conduct site work.

24.2.1 Permitting and Plan/s Preparation

The contractor must have all permitting requirements fulfilled prior to any construction activity taking place. One specific state requirement is the SWPPP, which must be approved and implemented before construction starts. Another major permitting effort is acquiring USACE approval for in-river construction. ROWs and easements must also be in place before the contractor can begin.

Because of the impacts, the contractor would be required to develop site utilization plans, traffic control plans, and work plans. In these plans, the contractor would indicate his/her anticipated activities that require use of the site as described in the utilization plan. Depending on the extent of the available site area as well as anticipated demands, a contractor would set up his/her own crushing and concrete batching plants. Considering the magnitude of concrete needed for the TDF option and the schedule demands of the project, the contractor would be required to provide his/her own batch plant. After plans have been developed and approved, the contractor would be allowed to begin work.

24.2.2 Mobilization

During mobilization, the contractor moves manpower, materials, and equipment to the site. During this time, the contractor sets up his/her working area to best expedite the construction activities, locates offices, warehouses, staging, or laydown areas, and considers the best configuration to move labor, materials, and equipment in and out of the site.

24.2.3 Site Work

Site work pertains to clearing and grubbing activities as well as defining and constructing access roads for temporary construction purposes. The contractor could implement erosion and sediment control measures in accordance with an approved SWPPP before conducting site work. Although contractors are likely to make the most out of the existing levee roads, bridges, and highways during construction, some roads and bridges may need to be constructed to expedite construction activities and minimize impact to existing commuters and the environment. Maintaining access roads and environmental controls would require enforcement of BMPs.

After mobilization and site work, the contractor continues activities for the major project features in the TDF. While the methodologies have been discussed in the preceding sections, the following discussion would focus on factors which govern the constructability of the structure.

24.3 Constructability

24.3.1 Definition

Constructability is defined as:

- The extent to which the design of the work facilitates ease of construction, subject to the overall requirements for the completed project. (Construction Industry Research and Information Association definition)
- A system for achieving optimum integration of construction knowledge and experience in planning, engineering, procurement and field operations in the building process and balancing the various project and environmental constraints to achieve overall objectives. (Construction Industry Institute definition)
- A system for achieving optimum integration of construction knowledge in the building process and balancing the various project and environmental constraints to achieve maximization of project goals and building performance. (Construction Industry Institute, Australia definition)

These definitions point out the fact that for a structure or project to be “constructible,” it has to consider the effects of construction equally across the existing spatial, social, and environmental conditions. The next section addresses the conditions to be considered when selecting a preferred construction methodology.

24.3.2 Factors Affecting Constructability

The TDF alignment would pass through areas of residence and agriculture in the Delta region. The construction would pass in the vicinity of railroads and aqueducts, cultural resource lands, and navigable and recreational waterways. Accordingly, selecting construction methods would consider:

- Impacts to river hydrology
- Fish and wildlife protection
- Land use
- Presence of weak soils (i.e., low-bearing capacity)
- Site accessibility
- Availability of staging or laydown area
- Availability of new technology

Although factors can be generalized across the project, each feature is assessed according to governing conditions or constraints local to the area and specific to the type of structure. In the following sections, the discussion would focus on the constructability of each conveyance project feature as it is pertains to the TDF alignment.

24.3.3 TDF Facility Constructability

This section follows the order of appearance of the project facilities from the previous sections in this report.

24.3.3.1 Intake Facilities

Construction Methodology:

- The approach would be in-river construction with a cofferdam around the work area.
- Foundation piles would be driven to required depths where a structure would be constructed.
- The cut-and-cover approach for the pipe trenches would connect with the manifold of the on-shore pumping plant. Microtunneling would be used if the soil and work space is not available for cut-and-cover operations.

Constructability Concerns or Issues:

- For the intake structure to connect to the pumping plant, the pipes have to penetrate beneath the existing levee; therefore, the stability of the levee is a concern.
- Because the construction is conceived to be in-river, measures have to be in place to provide adequate fish protection during the construction phase.
- In-river construction would depend on the water level in the river. Construction would only occur during the low level season, thus construction “windows” would need to be established.
- The river needs to remain navigable during construction.
- Construction below the water table necessitates dewatering and water treatment before disposal.
- A dry work area needs to be maintained during construction.
- In-river construction necessitates approval and permitting from USACE.

24.3.3.2 Pumping Plants

Construction Methodology:

- The approach would be “open hole” construction with either vertical cut walls with soil retaining structures such as post and lagging or benched excavation.

Constructability Concerns or Issues:

- The soil or geologic condition in the plant location would dictate the depth of the excavation as well as the type of foundation to use.

- Construction below the water table necessitates dewatering and water treatment before disposal.
- The pipes leading to the manifold of the pump would penetrate beneath the levee; therefore, bank stability is a concern.

24.3.3.3 Pipelines

Construction Methodology:

- The general approach for pipe construction would be cut-and-cover excavation in areas where land disturbance is possible.
- When land disturbance should be kept at a minimum, trenchless construction or tunneling would be considered.

Constructability Concerns or Issues:

- The presence of a high groundwater level limits the depth of excavation in the dry.
- Construction below the water table necessitates dewatering and water treatment before disposal.
- Trench sizes would be kept at a minimum, thereby limiting equipment access.

24.3.3.4 Canals

Construction Methodology:

- Excavation is done in the dry, using diesel excavators, scrapers, trucks, and conveyors, and in the wet, using excavator on barges or draglines.

Constructability Concerns or Issues:

- The presence of existing utilities can conflict with the size and reach of excavation equipment.
- Site access could be a limitation to the size and number of equipment that should be on site.
- Construction below the water table necessitates dewatering and water treatment before disposal.

24.3.3.5 Culvert Siphons

Construction Methodology:

- To work across the full width of the river, the approach could be to construct a bypass system as a river diversion.
- Another approach could be cofferdamming half the river width and then closing the other half during the next construction season.

- An alternate construction method would be to microtunnel or jack culverts but this would only apply to limited culvert sizes.

- Precast concrete siphon segments could be placed in the wet.

Constructability Concerns or Issues:

- Because the construction is conceived to be in-river, measures are required to maintain adequate fish protection during the construction phase, including turbidity controls.
- In-river construction depends on the water level in the river. Construction would only occur during the low level season, thus construction windows must be established.
- The river needs to remain navigable during construction.
- In-river construction necessitates approval and permitting from USACE.
- A dry work area needs to be maintained during construction.

24.3.3.6 Tunnels

Construction Methodology:

- The approach would be tunneling using EPBTBM at least 100 feet below surface.
- Shafts for tunnel access would be open-hole construction with a soil retaining structure such as post and lagging, or precast panel wall construction.

Constructability Concerns or Issues:

- High groundwater levels would necessitate extensive dewatering and groundwater control in the tunneling operation.
- The level of earth pressure would determine the type of TBM to be used.
- A gassy tunnel classification would require intensive safety measures during construction.
- Soil or geologic conditions would affect the type of TBM and depth of tunnel and shafts.
- Removal and disposal of tunnel muck would affect the cycle and configuration of tunnel operations.
- The limits of ground support for shafts would limit the height of shaft walls.

24.3.3.7 Bridges

Construction Methodology:

- Bridge foundations would require pile driving.
- Large girders requiring multiple-crane lifts would be launched to span across the canal with or without piers.

Constructability Concerns or Issues:

- Large equipment and a large quantity of material would require reliable access to the site.
- Site constraints on laydown and staging areas must be addressed.
- Foundation work would extend below the water table, which necessitates dewatering and water treatment before disposal.
- The soil or geologic condition and bearing capacity would dictate the depth of driven piles.
- Conflicts with construction of other features (i.e., canal construction) affects the timing of construction.
- Temporary road or detour construction must be completed to avoid impact to traffic.

24.3.3.8 Utilities and Infrastructure Crossings

Construction Methodology:

- The various construction approaches would include trenching, pipe rerouting, pole relocation, and others.

Constructability Concerns or Issues:

- High groundwater levels necessitate dewatering and water treatment before disposal..
- Water, sewer, power or other service interruptions during construction.
- Resolve conflict with:
  - Power transmission or distribution lines
  - High-pressure gas lines and wells
  - Railroads and aqueducts
  - Existing water and sewer lines
  - Agriculture drainage and water supplies
  - Oil product lines

24.3.3.9 Forebay

Construction Methodology:

- Large open-hole excavation using excavators and haul trucks.
- Embankments would be constructed in lifts (e.g., 12-inch lifts) using bulldozers and graders and compacted to approved density.

Constructability Concerns or Issues:

- Availability of engineered fill needed for embankments.
- Tying in to existing system with minimum interruption.

- Construction below the water table would necessitate dewatering and water treatment before disposal.
- Construction would occur only during the low water level season, thus construction windows could be established.

24.3.3.10 Levees

Construction Methodology:

- The approach would be to build embankment in lifts (e.g., 12-inch lifts) using bulldozers and graders and compact embankments to approved density.

Constructability Concerns or Issues:

- Availability of engineered fill would be needed for embankment (requires identification of appropriate borrow sites and haul routes).
- Construction below the water table would necessitate dewatering and water treatment before disposal.

24.3.3.11 Operable Barriers / Gates

Construction Methodology:

- The approach would be in-river construction with a cofferdam around the work area.
- An alternative approach would be to construct a bypass system as a river diversion to work across the full width of the river.
- Another option would be to barge precast or procured materials, such as gates, into place and set them in the wet.

Constructability Concerns or Issues:

- Because the construction would be in-river, measures must be in place to ensure that there is adequate fish protection during the construction phase.
- In-river construction would depend on river water level. Construction would occur only during the low water level season, thus construction windows would be established.
- The river would need to remain navigable during construction.
- A dry work area would need to be maintained during construction.
- In-river construction would necessitate approval and permitting from USACE.
- Available skilled manpower would be needed specifically for specialized in water construction.

24.3.3.12 Controls and Communications

Construction Methodology:

- The general approach would be to run conduits parallel to the facility.
- Conduits also could be buried in pipe trenches during construction.
- Duct banks also could be constructed to run parallel to canals and other aboveground facilities.

Constructability Concerns or Issues:

- Installed materials would need to be protected during construction.
- Integrating the system would depend on finalizing components at the appropriate time.
- Presence of high groundwater levels limits trench depth.

24.3.3.13 Power Supply and Grid Connections

Construction Methodology:

- The general approach would be to install high-voltage lines using wooden or steel poles embedded into the ground.
- Generators and substations would be placed on concrete pads with sufficient ground preparation.

Constructability Concerns or Issues:

- Construction below the water table would necessitate dewatering and water treatment before disposal.
- New power poles would need to be located without conflict to existing utilities.
- Power lines would need to be constructed to provide both temporary and permanent power supply.
- A grounding grid would need to be constructed for the substation.

24.3.3.14 Borrow Sites

Construction Methodology:

- The approach would be to create shallow excavations using conventional earthmoving equipment.
- Restoration efforts would include grading, erosion control, and revegetation.

Constructability Concerns or Issues:

- High groundwater levels would limit depth of excavation.

- Stockpiled materials may require moisture conditioning before being hauled to work areas.
- The distance from the work site would need to be minimized.

24.3.3.15 Spoils Disposal Sites

Construction Methodology:

- The approach would be conventional earthmoving using bulldozers and graders. Spoils would be handled in 12-inch lifts with nominal compaction. Dewatering spoils may be required.

Constructability Concerns or Issues:

- The distance from spoils site to work site would need to be minimized to maintain short haul distances.
- Dust mitigation would be implemented along haul routes.
- Dewatering spoils would require a significant effort.

24.4 Other Aspects Related to Construction

Early identification of constructability concerns and issues allows these issues to be addressed in the design phase. Schedule and cost also play a major role in identification of construction methods. The project schedule may require that some operations finish sooner or later to meet overall project goals. In the same way, the cost-effectiveness of one construction method over another could be the determining factor. Construction and constructability go hand in hand with schedule and cost estimates to determine how project features should be built.

Addressing constructability issues and proposing construction methods could also allow for contractor flexibility and ingenuity. Efforts to address construction matters are intended to be inclusive and are offered for the furtherance of the environmental permitting process.

As design progresses, construction methods and constructability issues must be further developed.

25.0 DUAL CONVEYANCE FACILITY CONSIDERATIONS

Portions of the TDF Option described in this CER are a part of the Dual Conveyance Facility configurations. See Section 25.0 of the CER for the Dual Conveyance Facility Option for additional information

26.0 PERMITS NEEDED

The proposed implementation of any conveyance option (project) would have to meet the requirements of various federal state and local regulations, laws, policies and acts. A forthcoming Permitting Handbook (Draft Version #2, August 2009, and its subsequent updates) prepared for the BDCP provides information on the major requirements for permitting, and environmental review and consultation for implementation of the TDF Option. Certain federal, state and local regulations require issuance of permits prior to project implementation; other regulations require agency consultation but may not require issuance of any permits prior to project implementation. The Permitting Handbook identifies the major permits or actions, the agency in charge, agency authority, permit implementing entity, permitting process, and other relevant information. It does not include all of the permits or actions that may be needed for the project.

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APPENDIX A

Geology and Seismicity

1.0 GEOLOGY

1.1 Regional Geology

The project area is located within the northwestern section of the Central Valley geomorphic province of California, also known as the Great Valley province (Figure A-1). The Central Valley province is characterized by a large northwest trending asymmetrical synclinal trough filled with a prism of upper Mesozoic-age (Bartow, 1991) through recent sediments up to 30,000 feet thick (Figure A-2). Most of these sediments consist of upper Mesozoic-age marine sandstone, shale, and conglomerate, known as the Great Valley Sequence, which accumulated in a forearc ocean basin that lay to the west of the Mesozoic North American margin (Harden, 2004). The Great Valley Sequence is overlain by a range of Tertiary-age marine, terrestrial, and volcanoclastic sedimentary rocks. These rocks are in turn overlain by a thick accumulation of alluvial, eolian and deltaic deposits associated with late Quaternary glacial cycles.

The Central Valley sedimentary basin is divided into the Sacramento Valley in the north and the larger San Joaquin Valley in the south, separated by the buried, transverse Stockton arch and Bakersfield arch. The Stockton arch, which is a broad structure bounded on the north by the Stockton fault, separates the San Joaquin and Sacramento sedimentary basins.

Under the central and western parts of the valley, the sediments rest on mafic and ultramafic rocks of a presumed Jurassic-age ophiolite. Along the western side of the valley, the Great Valley Sequence is juxtaposed with the Franciscan Complex of the Coast Ranges province along a boundary fault termed the Coast Range thrust. Under the eastern part of the valley, the sediments rest on a westward-tilted block of crystalline basement composed of Sierra Nevada plutonic and metamorphic rocks.

1.2 Delta Geologic History

The Delta has a complex geologic history. During the Cretaceous and Tertiary periods (Figure A-2), the future location of the Delta received thick accumulations of sediments from the Sierra Nevada and the Coast Ranges. Approximately 620,000 years ago (early Quaternary), a lake that had formed in the Central Valley spilled over a low spot in the Coast Ranges and began flowing through the San Francisco Bay Area via the Carquinez Straits. This drainage outlet provided the framework for the evolution of the Delta as known today.

Fluctuations in global climate and sea level since late Quaternary time have produced several cycles of deposition, non-deposition, and erosion. The cycles resulted in the accumulation of thick, poorly-consolidated to unconsolidated sediments overlying the Cretaceous and Tertiary formations. The present geomorphology and surficial geology of the Delta have been shaped by the landward spread of tidal environments resulting from sea level rise after the last glacial period, approximately 15,000 years ago.

In the Delta, relative sea level rise is the sum of eustatic (global) sea-level rise, tectonic land movements, and local subsidence (typically soil decomposition and consolidation). During the last glacial period, around 15,000 years ago, the Pacific coast was at least 6 miles west of its present position, and the relative sea level was approximately 300 feet lower than today. During this time, the area of the present day Delta, at the confluence of the Sacramento and San Joaquin rivers, formed part of the arid alluvial floodplain of the Central Valley. As a consequence, alluvial sand deposits together with eolian sand deposits underlie most of the late Holocene Delta soils (Figure A-3).

Between 10,000 and 5,000 years ago, relative sea-level rise was rapid, out-stripping the rate of deposition of flood-borne sediments supplied by the river systems (Atwater and Belknapp, 1980, and URS/JBA, 2007c). This resulted in the landward transgression of the ocean through the Carquinez Strait and into the Central Valley, forming the Suisun Bay and the Delta. This period of time saw the widespread deposition of organic silt and clay across the alluvial floodplain surface.

About 5,000 years ago, relative sea-level rise slowed, halting landward transgression of the tidal wetlands. At this time, the deltaic environment remained in approximately its present position, with slow relative sea-level rise balanced by vertical marsh growth through biomass accumulation and sediment deposition (Atwater et al., 1979). A transition, from deposition of organic silt-clay to peat formation in the Delta, largely reflects the decline in inundation frequency and the maturation of the marsh plain towards mean higher high water (MHHW) elevations.

The historical Delta east of Browns Island evolved laterally as two overlapping geomorphic units. The Sacramento Delta to the north comprised about 30 percent of the total area and extends as far as Sherman Island to the west. Its morphology was created by the interaction of rising sea level, alluvial river-flood deposition, and tidal marsh peat formation. This created an inland “bird’s foot delta” of distributary channels, bordered by higher supratidal natural levees, and surrounded by marsh plains (Atwater and Belknap, 1980).

In contrast, the larger south-centrally-located San Joaquin Delta (about 70 percent of the total area), with its relatively small flood flows and low sediment supply, formed as an extensive uniform freshwater tule (assemblage of bulrush, cattails, and common reed) tidal marsh dominated by tidal flows and organic (peat) accretion (Atwater and Belknap, 1980). Here, the channel system was determined almost entirely by tidal flows that created an extensive sinuous dendritic channel network. Because of the differential amounts of inorganic sediment supply, the peat of the south-central Delta (San Joaquin River system) grades northwards into peaty mud and mud toward the natural levees and flood basins of the Sacramento River system (Atwater and Belknap, 1980). This is reflected in the thickness of peat across the Delta, which can be up to 30 feet thick in the central Delta, and thinning towards the north and south (URS/JBA, 2007c).

At the margins of the Delta, the freshwater tidal marshes merged with flood basin marshes at slightly higher elevations. Although the wetland species were the same, the underlying soils were different because the flood basins dried out every summer, preventing peat accumulation.

Over the last 150 years, the natural landscape elements of the Delta have been transformed by human activities. The large freshwater tidal marsh of the Delta has been converted by levee building into a highly dissected region of channels and levee-encircled islands used for agriculture (Simenstad et al., 2000). Today, the Delta contains over 55 “dry” islands or tracts that are protected from flooding by more than 1,100 miles of levees. Islands that were originally near sea level are now well below sea level and large areas of many islands are now more than 15 feet below sea level.

1.3 Regional Subsidence

For the last 5,000 years up to the 1850s, relative sea-level rise in the Delta was balanced by vertical marsh growth through biomass accumulation and sediment deposition (Atwater et al. 1979), resulting in the accumulation of great thicknesses of organic rich soils within the delta. Starting in the mid-1800s, many hundreds of miles of levees were constructed, allowing the isolation and draining of vast areas of the delta for agricultural use. The construction of these levees and drainage systems was largely completed by 1930 and the Delta had taken on its current appearance, with most of its 1,100-square-mile area reclaimed for agricultural use (Thompson, 1957). The original levees were usually less than 5 feet high, but had to be raised to keep up with the settlement of levees and the subsidence of the interior island soils. Prior to the agricultural development of the Delta, island surface elevations were at or near sea level. As the Delta islands have subsided, levee heights have become progressively greater. Some levees are now up to 25 feet above the interior island surfaces.

The dominant cause of this land subsidence in the Delta is decomposition of organic carbon in the peat soils (Ingebritsen and Ikehara, 1999). Prior to agricultural development, the soil was waterlogged and anaerobic (devoid of oxygen), so organic carbon accumulated faster than it could decompose. Drainage for agriculture led to aerobic (oxygen-rich) conditions that favor rapid microbial oxidation of the carbon in the peat soil. In some areas, groundwater extraction and gas field pumping can also contribute to local and regional subsidence.

The principal control on the magnitude of subsidence is the composition of the marsh soils. At a landscape scale, the soils of the central Delta, which are generally more organic-rich, exhibited the highest average historical rates of subsidence, between 0.10 and 0.16 feet per year (ft/yr) (Mount and Twiss, 2005). The more inorganic soils of the northern Delta exhibited lower rates of subsidence. On a local scale, the surface profile of individual islands is generally “saucer-shaped,” due to oxidation of the exposed peats in the interiors of the islands. Inorganic soils may be more prevalent at the island perimeter because of depositional processes.

Rates of subsidence on the Delta islands have declined since the 1950s because of improved land-use practices (Deverel and Rojstaczer, 1996; Deverel et al., 1998). Further subsidence is also constrained by the thickness of organic-rich sediments deposited during the mid- to late-Holocene. In the south and east Delta, historical subsidence has reduced or eliminated the organic-rich soils, whereas the thicker organic soils of the central and west Delta continue to subside. Mount and Twiss (2005) found that post-1950 subsidence rates were 20 to 40 percent less than the average rate between 1925 and 1981 (URS/JBA, 2007b).

2.0 SEISMICITY

2.1 Seismotectonic Setting

Active faulting and earthquakes in central California result from transpressional deformation related to movement of the Pacific plate is to the northwest relative to the North American plate. Most of this movement is accommodated along the major strike-slip fault systems of the San Andreas and Hayward-Calaveras fault systems, which lie to the west of the Delta (Figure A-4). Other strike-slip faults nearer the Delta also accommodate the motion between the tectonic plates, and some plate motion is taken up on reverse and thrust faults like those in the Coast Ranges-Sierran Block boundary zone (CSRB). The Delta lies in the central western part of a broad asymmetric trough whose western limb dips more steeply than its eastern limb because the western limb is being deformed by tectonism at the eastern margin of the Diablo Range. The Diablo Range is cored by late Mesozoic Franciscan rocks that are overlain by Great Valley Sequence strata. These rocks have been pervasively folded, with fold axes generally subparallel to the San Andreas fault. This tectonic setting has been in place since about 5 million years ago when the San Andreas fault system became established at the latitude of central California and when uplift of the Diablo Range began about 3.5 million years ago (Wakabayashi and Smith, 1994).

Historical earthquakes, like the 1983 moment magnitude (**M)** 6.4 Coalinga earthquake and the Vacaville-Winters **M** > 6 earthquakes in 1892, shed light on the nature of late Cenozoic tectonics in Central California (Figure A-4). These earthquakes occurred at the western margin of the Central Valley and the eastern margin of the Diablo Range and are interpreted to have occurred on structures that formed in response to northeast-southwest compression resulting from divergence between the San Andreas fault and the orientation of the Pacific-North American plate motion (Wong et al., 1988, Wentworth and Zoback, 1990). Wakabayashi and Smith (1994) described a series of west-dipping faults that are responsible for these earthquakes and that separate the Coast Ranges from the Central Valley. More recent research (e.g., WGNCEP, 1996; O’Connell et al., 2001; and USBR, 2001) has been used to refine the Wakabayashi and Smith (1994) model and improve the characterization of these faults. The CSRB faults are associated with a buried fold and thrust belt and, in most cases, do not rupture the surface. Although the geometry and recurrence of these faults are still not as well understood as the major strike-slip faults to the west, they are included in compilations of active faults used to evaluate earthquake hazards (e.g., WGCEP, 2008).

2.2 Seismic Sources

A model of the active and potentially active seismogenic faults in the greater San Francisco Bay region was developed as part of the Delta Risk Management Strategy (DRMS) study (Figure A-4). Each seismic source was characterized using the latest geologic, seismological, and paleoseismic data and the currently accepted models of fault behavior. A major study by the Working Group on California Earthquake Probabilities (WGCEP, 2003) entitled “Earthquake Probabilities in the San Francisco Bay Region: 2002-2031” describes and summarizes the current understanding of the major faults in the San Francisco Bay area. The DRMS study adopted the WGCEP (2003) seismic source model for the San Andreas, Hayward/Rodgers Creek, Concord/Green Valley, San Gregorio, Greenville, and Mt. Diablo thrust faults. The characterization of the Calaveras was slightly modified by William Lettis and Associates (WLA) and URS for DRMS (URS/JBA, 2007a). We will not describe these fault sources in this report.

“Blind” faults beneath the Delta and the Western Tracy and Vernalis faults, part of the CRSB (Wong et al., 1988), are of particular significance to the assessment of seismic hazards in the Delta (Figure A-5). The Delta sources include the Northern Midland zone, the Southern Midland fault, the Thornton Arch zone, and the Montezuma Hills source zone (Figure A-5). As is the case for many “blind” faults, the characterization of the Delta seismic sources is highly uncertain because of the very limited amount of available data. What is known about these sources primarily has come from subsurface seismic data. Descriptions of the Delta faults (or fault zones) and four faults in the CSRB are provided in the following paragraphs. These descriptions are based on work conducted as part of the DRMS seismology study (URS/JBA, 2007a).

2.2.1 Delta Faults and Fault Zones

Midland Fault. The Midland fault is a roughly north-striking, west-dipping fault underlying the central Delta region that accommodated extension and subsidence in the early Tertiary Sacramento Valley forearc basin (Krug et al., 1992). As shown on the California State geologic map, the fault is at least 60 kilometers (km) long (Wagner et al., 1981). The Midland fault is not exposed at the surface and is known primarily from natural gas exploration in the greater Delta region. Proprietary seismic reflection profiles indicate that the dip of the fault is relatively steep at shallow depths and decreases with depth, suggesting a downward-flattening or listric geometry. Although the Midland fault is commonly shown as a single buried trace on state maps along its entire length (Wagner et al., 1981; Jennings, 1994), subsurface mapping by the California Division of Oil and Gas (1982) and Krug et al. (1992) indicate the fault breaks into a series of northwest-striking splays north of the town of Rio Vista exhibiting a right-stepping, ***en echelon*** pattern. The northwest-striking splays of the fault are associated with a series of active and abandoned gas fields in the Sacramento Valley between the towns of Rio Vista and Woodland (California Division of Oil and Gas, 1982).

Based on reverse offset of Quaternary strata inferred from interpretation of seismic reflection data, the Thrust Fault Subgroup (1999) adopted a range of weighted values for the long-term average reverse slip rate on the Midland fault that is centered on 0.15 millimeters per year (mm/yr). To develop estimates of maximum earthquake magnitude for the Midland fault, the Thrust Fault Subgroup (1999) considered several scenarios and adopted a weighted range of earthquake magnitudes centered on **M** 6.25.

DRMS study authors performed additional research, met with experts, and reviewed proprietary information to revise the Thrust Fault Group’s description of the Midland Fault (URS/JBA, 2007a). The DRMS analysis reveals systematic west-side-up anomalies in the contact at the base of peat across the Midland fault in Webb Tract, Franks Tract, and Holland Tract. If it is assumed that these anomalies are because of Holocene movement on the Midland fault, then the implied vertical separation rate is about 0.3 to 0.6 mm/yr, which is comparable to the reverse slip rate of 0.1 to 0.5 mm/yr for the Midland fault estimated by the Thrust Fault Subgroup (1999). Based on the change in character of the Midland fault at about the latitude of Rio Vista (Krug et al., 1992), the DRMS authors separated the fault into two distinct sources: the Southern Midland fault, which is characterized as a single, potentially seismogenic fault; and the Northern Midland zone, which is characterized as an areal source zone to encompass the numerous right-stepping, northwest-striking splays of the Midland fault.

DRMS authors interpreted that net slip on the Southern Midland fault is probably oblique, with components of dextral and reverse displacement and modified the weighted range of slip rates for the Midland fault developed by the Thrust Fault Subgroup (1999) to account for a component of right-lateral motion. Slip rates used in the DRMS model included 0.1 mm/yr (weighted 0.3), 0.5 mm/yr (0.4), and 1.0 mm/yr (0.3). DRMS adopted the same weighted slip rate values for the Northern Midland Zone as for the Southern Midland fault (URS/JBA, 2007a).

Two scenarios were considered in evaluating earthquake magnitude for the Southern Midland fault: (1) unsegmented rupture of the entire length of the fault (**M** 6.6); and (2) rupture of only part of the fault in a single event, with the same weighted range of floating earthquake magnitudes centered on **M** 6.25 as adopted by the Thrust Fault Subgroup (1999). The DRMS study placed higher weight on the floating earthquake model because geomorphic expression of activity is not uniform along the entire mapped length of the fault. For the Northern Midland Zone, DRMS considered a floating earthquake model only and adopted the same weighted range of magnitudes as for the floating earthquake on the Southern Midland fault (URS/JBA, 2007a).

<u>Thornton Arch Source Zone.</u> The Thornton Arch source zone encompasses the possibility that a buried structure in the vicinity of the Thornton and West-Thornton-Walnut Grove gas fields is an active fault (Figure A-5). The motivation for this is the observation that the Mokelumne River does not continue along a straight course across the Delta from the point where it exits the western Sierran foothills, but rather it appears to be deflected to the north in an anomalous loop north and west of the town of Thornton (URS/JBA, 2007a). The deflection of the Mokelumne River occurs around the “Thornton Arch”, an antiformal structure that comprises the Thornton and West-Thornton-Walnut Grove gas fields (California Division of Oil and Gas, 1982). Available data on the structure of the gas fields are limited to structure contour maps on Eocene stratigraphic markers and cross sections developed from borehole data (California Division of Oil and Gas, 1982). The “Thornton Arch” is a roughly east-west-trending antiformal closure in Eocene and older strata. The California Division of Oil and Gas (1982) has interpreted the presence of several north-northwest-striking faults in the gas fields from analysis of borehole data, but it is not clear how these structures are related to the development of the fold.

Based primarily on the possibility that the northward deflection of the Mokelumne River is because of localized Quaternary uplift of a blind structure, DRMS defined a source zone to encompass the Thornton Arch and associated faults as potential causative structures (URS/JBA, 2007a). DRMS assumed that the primary causative fault(s) for the deformation have an approximately east-west strike similar to the trend of the antiform and that earthquake magnitudes are limited by the relatively small dimensions of the Thornton Arch source zone and structures encompassed therein. DRMS adopted a range of maximum magnitudes with a weighted mean of **M** 6.25. Given the lack of geomorphic expression of surface deformation within the Thornton Arch source zone other than the possible deflection of the Mokelumne River, DRMS inferred that deformation rates must be very low, and adopted a weighted range of slip rates centered on a mean of 0.10 mm/yr (URS/JBA, 2007a).

<u>Montezuma Hills Source Zone.</u> DRMS defined the Montezuma Hills source zone to encompass the uncertainty about whether Quaternary uplift of the Montezuma Hills is due exclusively or

even primarily to west-side-up motion on the Midland fault (URS/JBA, 2007a). The motivation for this is a structural geologic interpretation by Dr. Janine Band of a grid of proprietary seismic reflection lines that cross the Montezuma Hills. Given Dr. Band's observations, DRMS defined a source zone to encompass possible undetected active structures that may be responsible for the uplift of the Montezuma Hills. DRMS extended the zone southward along the general trend of the Sherman Island fault system in the subsurface (Figure A-5). DRMS assumed that earthquake magnitudes will be limited by the northwestern/southeastern (NW-SE) dimensions of the zone, and thus adopted a range of maximum magnitudes with a weighted mean of **M** 6.25. The preferred range (0.05 to 0.5 mm/yr) and weighting of slip rates reflects the interpretation that tectonic activity in the Montezuma Hills, if independent of the Midland fault, may be related to transfer of slip from the Vernalis and West Tracy faults to the Pittsburg-Kirby Hills fault zone.

2.2.2 CRSB Boundary Zone Faults

The following paragraphs describe the elements of the CRSB that were characterized as part of the DRMS study (URS/JBA, 2007a): West Tracy, Vernalis, Black Butte, and Midway faults.

<u>West Tracy Fault.</u> The West Tracy fault strikes northwest-southeast and is mapped for a total distance of about 34 km along the eastern flank of the northern Diablo Range between Corral Hollow south of Tracy and the town of Byron (Figure A-5). The fault has no documented surface trace on small-scale geologic maps published by the State of California (Rogers, 1966; Wagner et al., 1991), and is known primarily from analysis of proprietary borehole data and seismic reflection data acquired for oil and gas exploration (Sterling, 1992). The West Tracy fault is well imaged as a moderately to steeply west-dipping fault on seismic reflection lines. The reflection data provide clear evidence for west-side-up reverse displacement on the fault, including offset of reflectors associated with Cretaceous marine strata at depth and monoclinal folding above the fault tip (Sterling, 1992). Geologic mapping at 1:250,000 scale by the State of California (Rogers, 1966) shows a contact between older and younger Quaternary deposits that follows the buried trace of the West Tracy fault. The older deposits are preferentially associated with the hanging wall of the fault, consistent with Quaternary uplift. DRMS interpreted these map relations as prima facie evidence for Quaternary uplift and fault-propagation folding above the West Tracy fault (URS/JBA, 2007a).

Very limited data are available to estimate the rate of slip and recent behavior of the West Tracy fault. DRMS assumed that the slip rate of the West Tracy fault is less than that of the Midway/Black Butte fault zone because it lies farther to the east, consistent with geodetic data that document eastward decreasing rates of dextral motion across the Pacific-Sierran plate boundary (Prescott et al., 2001; d'Alessio et al., 2005). A lower bound of 0.07 mm/yr on the slip rate is estimated based on total vertical separation of about 800 ft (244 m) of a basal Miocene unconformity across the fault as reported by Sterling (1992) and an assumed duration of deformation (active during the past ~3.5 Ma). A maximum slip rate on the West Tracy fault of 0.5 mm/yr was assumed; this value is 50 percent of the maximum slip rate on the Midway/Black Butte fault (URS/JBA, 2007a).

<u>Vernalis Fault.</u> The Vernalis fault is an approximately northwest-striking, moderately to steeply west-dipping fault in the subsurface of the western San Joaquin Valley, about 9 to 12 km east of the physiographic front of the Diablo Range (Figure A-5). The Vernalis fault extends for a minimum of 31 km between Tracy and the town of Patterson to the southeast (Sterling, 1992).

Exploration geologists who have examined proprietary subsurface data suggest that the fault may continue an unknown distance south of Patterson, so the full length of the fault is poorly known.

The Vernalis fault is known primarily from analysis of proprietary borehole data and seismic reflection data acquired for oil and gas exploration (Sterling, 1992). Sterling (1992) describes stratigraphic and structural relationships imaged by seismic reflection data indicating “movement as recently as late Pliocene.”

DRMS inferred Quaternary activity of the Vernalis fault based on the systematic occurrence of older Quaternary deposits on the upthrown hanging wall block (URS/JBA, 2007a). Geologic maps of the 2 degree San Jose (Rogers, 1966) and San Francisco-San Jose quadrangles (Wagner et al., 1991) published by the State of California show Pleistocene fluvial deposits on the upthrown western side of the fault, and generally younger basin deposits on the downthrown side. The contact between the older and younger deposits closely follows the buried fault trace in the subsurface. The Vernalis fault also may exert control on local stream and drainage patterns.

Given the possible link between the structures, DRMS assumed the slip rate on the Vernalis fault is comparable to the estimated rate for the West Tracy fault (0.07 to 0.5 mm/yr), and adopted a range of weighted magnitudes centered on a mean magnitude of **M** 6.5, encompassing the possibility of rupture of all or part of the fault in a single event (URS/JBA, 2007a).

<u>Black Butte and Midway Faults.</u> The Black Butte fault is a northwest-striking, moderately to steeply west-dipping Quaternary fault along the physiographic boundary between the northern Diablo Range and northwestern San Joaquin Valley, located approximately 10 km southeast of the city of Tracy (Figure A-5). Sowers et al. (1992) documented about 180 m of west-side-up displacement of an early to middle Quaternary pediment surface across the Black Butte fault in the vicinity of Corral Hollow. Although these geomorphic and structural relations provide evidence for Quaternary activity on the fault, there is significant uncertainty in the age of the deformed surface, as well as the correlation of the pediment across the fault.

The late Cenozoic Midway fault strikes northwest and is separated from the northwest end of the Black Butte fault by a left ***en echelon*** step across a small west-northwest-trending anticline that deforms Miocene-Pliocene strata (Crane, 1995; Midway 7.5 minute quadrangle). Geologic mapping by Crane (1995) documents about 800 m of apparent right-lateral offset of an unconformable contact between Cretaceous and Miocene strata across the Midway fault in the SW 1/4 of section 19, T.2S., R.4E. Paleoseismic trenching investigations of the Midway fault conducted in 2004 by Geocon, Inc. documented late Pleistocene surface rupture on the fault (David Bieber, Geocon, Inc., personal communication in URS/JBA, 2007a). Slickensides on the exposed fault plane indicate dominantly subhorizontal displacement (Bieber, personal communication, 2007).

Based on the above data and observations, DRMS concluded that the Midway fault is an active structure that primarily accommodates strike-slip displacement (URS/JBA, 2007a). Based on the preponderance of evidence, DRMS characterized the Black Butte and Midway faults as a single structure that accommodates dextral-reverse displacement. DRMS estimated a range in slip rate for the Black Butte fault from the inferred displacement of the pediment and middle to early Pleistocene age estimates (Sowers et al., 1992), and an inferred horizontal to vertical (H:V) ratio for the components of slip. If it is assumed that the offset pediment ranges in age from about 300 ka to 1 Ma, then the corresponding range in long-term average vertical separation rate is about 0.2 to 0.6 mm/yr. With an assumed ≤ 3:1 ratio of strike-slip to dip-slip displacement, the implied rate of net oblique slip is less than 0.6 to 1.8 mm/yr. For the Midway fault, DRMS estimated a long-term average rate of dextral offset of about 0.2 mm/yr based on 800 m of late Cenozoic right-separation and an assumed duration of deformation (active during the past ~3.5 Ma). For maximum magnitude, DRMS adopted a floating earthquake model with a weighted range of magnitudes that favors rupture of all or most of the combined length of the Black Butte and Midway faults (URS/JBA, 2007a).

2.3 Historical Seismicity

The Delta has exhibited a low level of historical seismicity, and the seismicity that has occurred is difficult to correlate to any of the Delta seismic sources, which is not an unusual observation for buried faults. No **M** ≥ 4.0 events have occurred in the past 40 years in the Delta and no **M** ≥ 5.0 earthquakes have occurred in the Delta in historical times (Figure A-4). The absence of significant seismicity in the Delta does not necessarily indicate the absence of seismogenic structures. The neighboring CRSB boundary zone (Figure A-5) has been, for the most part, not seismically active and yet the occurrence of large earthquakes (**M** > 6) such as the 1892 Vacaville-Winters and 1983 Coalinga earthquakes are testimony to the seismogenic potential of buried faults (Wong et al., 1988). There have been about 15 earthquakes of approximately moment magnitude (**M**) 6.0 or greater in the San Francisco Bay region in historical times (Figure A-4).

The most significant earthquakes to the Delta are discussed in more detail below.

<u>October 21, 1868.</u> This local Richter magnitude (M_L) 6.8 earthquake occurred on the southern Hayward fault. It was one of the most destructive in California history. Heavy damage was sustained in towns along the Hayward fault in the eastern San Francisco Bay area, as well as in San Francisco and San Jose. The fault is thought to have ruptured from its southern end, in the eastern Santa Clara Valley, to northern Oakland or southern Berkeley. There is little information about this earthquake's effects on the Delta.

<u>April 19 and 21, 1892.</u> In April 1892, a series of earthquakes struck the western Sacramento Valley (Figure A-4). The epicenters of the largest earthquakes were near Winters and Vacaville, both very small towns at the time. The first earthquake, felt most strongly in Vacaville, occurred on Tuesday, April 19, in the early morning. Damage was more apparent in brick buildings than wooden ones. The April 19 earthquake had an estimated magnitude of about **M** 6.5. This earthquake damaged the communities of Vacaville, Dixon, and Winters, and the surrounding rural areas in the western part of the lower Sacramento Valley (Bennett, 1987). The second earthquake struck Winters on Thursday, April 21, at 9:40 a.m. It was stronger than the April 19 earthquake, although only estimated to be an **M** 6.2, damaging all remaining brick and stone buildings in Winters. The April 21 earthquake also resulted in the death of man who was injured by falling brick. These earthquakes are thought to have occurred on the Trout Creek and Gordon Valley segments of the CRSB (O'Connell et al., 2001).

<u>March 31, 1898.</u> On this date, the San Francisco Bay region was shaken by an earthquake that appears to have been centered near Mare Island in San Pablo Bay (Figure A-4). The maximum

intensity was modified Mercalli (MM) VIII or greater and buildings were damaged in areas around the Bay Area. Toppozada et al**.** (1981) re-evaluated the magnitude of this event through comparisons with other historical earthquakes and assigned it a M_L 6.7.

<u>April 18, 1906.</u> The **M** 7.9 Great San Francisco earthquake of 1906 was the most destructive earthquake to have occurred in northern California in historical times. The earthquake was felt from southern Oregon to south of Los Angeles, and as far east as central Nevada. It ruptured the northernmost 430 km of the San Andreas fault, from San Juan Bautista to the Mendocino Triple Junction. Damage was widespread in northern California and injury and loss of life was particularly severe. Ground shaking and fire caused the deaths of more than 3,000 people and injured approximately 225,000. Damage from shaking was most severe in areas of saturated or loose, young soils.

<u>May 2, 1983.</u> The **M** 6.4 Coalinga earthquake caused about $10 million in property damage and injured 94 people. The most significant damage outside the Coalinga area occurred at Avenal, about 30 km southeast of the epicenter. This earthquake was accompanied by an 0.5-m uplift of Anticline Ridge northeast of Coalinga, but surface faulting was not observed. Ground and aerial searches immediately after the earthquake revealed ground cracks and fissures within about 10 km of the instrumental epicenter, none of which appeared to represent movement on deeply rooted fault structures. This earthquake was felt from Los Angeles to Susanville to western Nevada (http://earthquake.usgs.gov/regional/states/events/1983_05_02.php). This earthquake is thought to be an analog for earthquakes that might occur in the western Delta.

<u>April 24, 1984.</u> The **M** 6.2 Morgan Hill earthquake occurred on the Calaveras fault about 18 km east of San Jose and 22 km north of Morgan Hill. This earthquake had a focal depth of 8 km and ruptured about 30 km of the fault. It was felt in California and Nevada over an area of 120,000 square km and caused damage estimated at $7.5 million. In San Jose, cracks formed in some walls and plaster fell, many items were thrown from store shelves and some chimneys cracked. Very strong shaking (~1.3 g) was measured at Coyote Dam approximately 20 km south of the epicenter. This earthquake is thought to have been very similar to an earthquake that affected the area in 1911.

<u>October 17, 1989.</u> The **M** 6.9 Loma Prieta earthquake occurred on or adjacent to the Santa Cruz segment of the San Andreas fault. The cities of Los Gatos, Watsonville, and Santa Cruz were severely damaged; San Francisco and Oakland were also damaged. Shaking was felt throughout the San Francisco Bay area and as far away as San Diego and Nevada. While the Loma Prieta earthquake was one of the most expensive natural disasters in U.S. history, causing in excess of $6 billion damage, the loss of life was significantly less than in 1906. Sixty-two people died and about 3,500 were injured. About 12,000 people were displaced from their homes. As in the 1906 earthquake, the worst damage from shaking occurred on unconsolidated or saturated soils, or with unreinforced masonry or inadequately designed structures. No damage was caused in the Delta.

<u>October 30, 2007.</u> The **M** 5.6 Alum Rock earthquake occurred on the Calaveras fault southeast of Calaveras Reservoir and northeast of San Jose, at a depth of about 8 to 9 km (about 5 miles). The event caused strong shaking in the epicentral region and was felt from Santa Rosa in the north, to the Sierra in the east, and King City to the south. The earthquake ruptured an approximately 5-km-long patch at depth on the near-vertical Calaveras fault, which is based on

the distribution of aftershocks, focal mechanisms, and moment tensor solutions (U.S. Geological Survey web page http://earthquake.usgs.gov/eqcenter/eqinthenews/2007/nc40204628/nc40204628.php). Geologists did not find any surface rupture along the fault; surface rupture is unusual for an earthquake of this size and depth.

2.4 Background Seismicity

To account for the hazard from background (floating or random) earthquakes that are not associated with known or mapped faults, regional seismic source zones were used in the DRMS seismology study (URS/JBA, 2007a). In most of the western United States, the maximum magnitude of earthquakes not associated with known faults usually ranges from **M** 6 to 6½. Repeated events larger than these magnitudes generally produce recognizable fault-or-fold related features at the earth's surface (e.g., dePolo, 1994). An example of a background earthquake is the 1986 **M** 5.7 Mt. Lewis earthquake that occurred east of San Jose.

For a probabilistic seismic hazard analysis (PSHA), like that performed for the DRMS study, earthquake recurrence estimates of the background seismicity in each seismic source zone are required. The DRMS site region was divided into two regional seismic source zones: the Coast Ranges and Central Valley (URS/JBA, 2007a). The recurrence parameters for the Coast Ranges source zone were adopted from Youngs, et al. (1992). They calculated values for background earthquakes based on the historical seismicity record after removing earthquakes within 10-km-wide corridors along each of the major faults. The recurrence values for the Central Valley zone were estimated by URS as part of the DRMS study (URS/JBA, 2007a and Figures A-6 and A-7). The maximum earthquake for the source zones is **M** 6.5 ± 0.3.

2.5 Seismic Hazards

The DRMS study (URS/JBA, 2008) evaluated the vulnerability of levees to seismic hazards. Historically, there have been 166 Delta and Suisun Marsh flood-induced levee failures leading to island inundations since 1900. No reports have been found to indicate that seismic shaking has ever induced significant damage. However, the lack of historical damage is not a reliable indicator that Delta levees are not vulnerable to earthquake shaking. Furthermore, the present-day Delta levees, in their current configurations, have not been significantly tested by the moderate to high seismic shaking that can be expected. Unlike flood-induced failures, earthquake-induced levee failures tend to extend for thousands of meters if not kilometers.

The largest earthquakes experienced in recent history in the region include the 1906 Great San Francisco earthquake and the 1989 Loma Prieta earthquake. The 1906 earthquake occurred while the levees were in their early stages of construction, were much smaller than they are today, and were not representative of the current configuration. The epicenter of the 1989 Loma Prieta earthquake was too distant and registered levels of shaking in the Delta too small to cause perceptible damage to the levees. Nonetheless, the DRMS seismic analysis team performed a special simulation analysis of the 1906 Great San Francisco earthquake to evaluate the potential effects of this event on the current levees (URS/JBA, 2008).

In addition to the simulation of these largest regional earthquakes, the DRMS study also evaluated recent smaller and closer earthquakes (URS/JBA, 2008). The earthquakes, and their impacts, that were evaluated include the 1980 Livermore earthquake (**M** 5.8) and the 1984 Morgan Hill earthquake (**M** 6.2). Except for the 1906 earthquake, which would have caused deformations of some of the weakest levees, the other earthquakes were either too small or too distant to cause any significant damage to the Delta levees. These results are consistent with the seismic vulnerability prediction model developed for the DRMS study (URS/JBA, 2008).

For further information on the DRMS levee vulnerability study, which evaluated existing levees, please see the DRMS study (URS/JBA, 2008).

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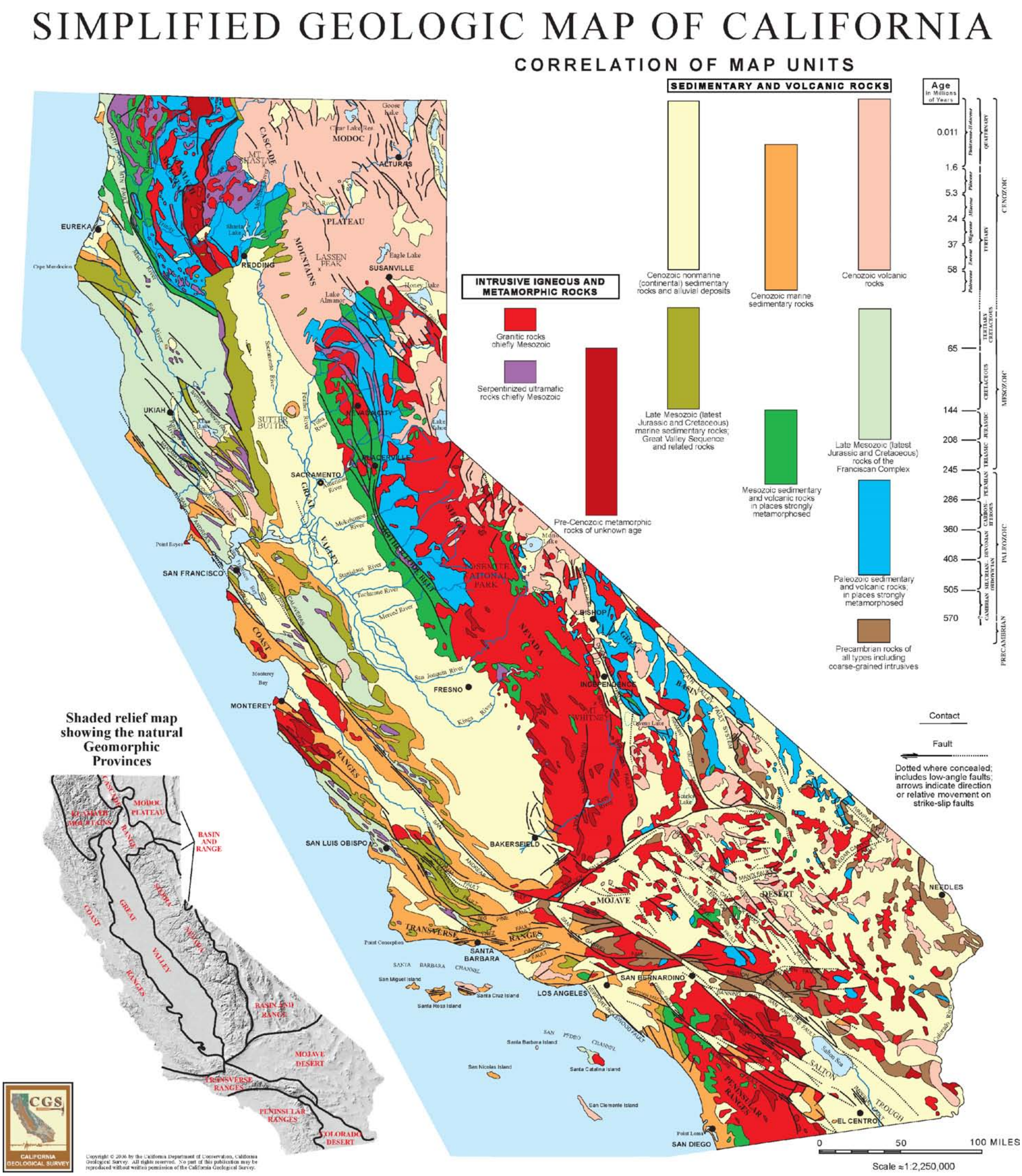

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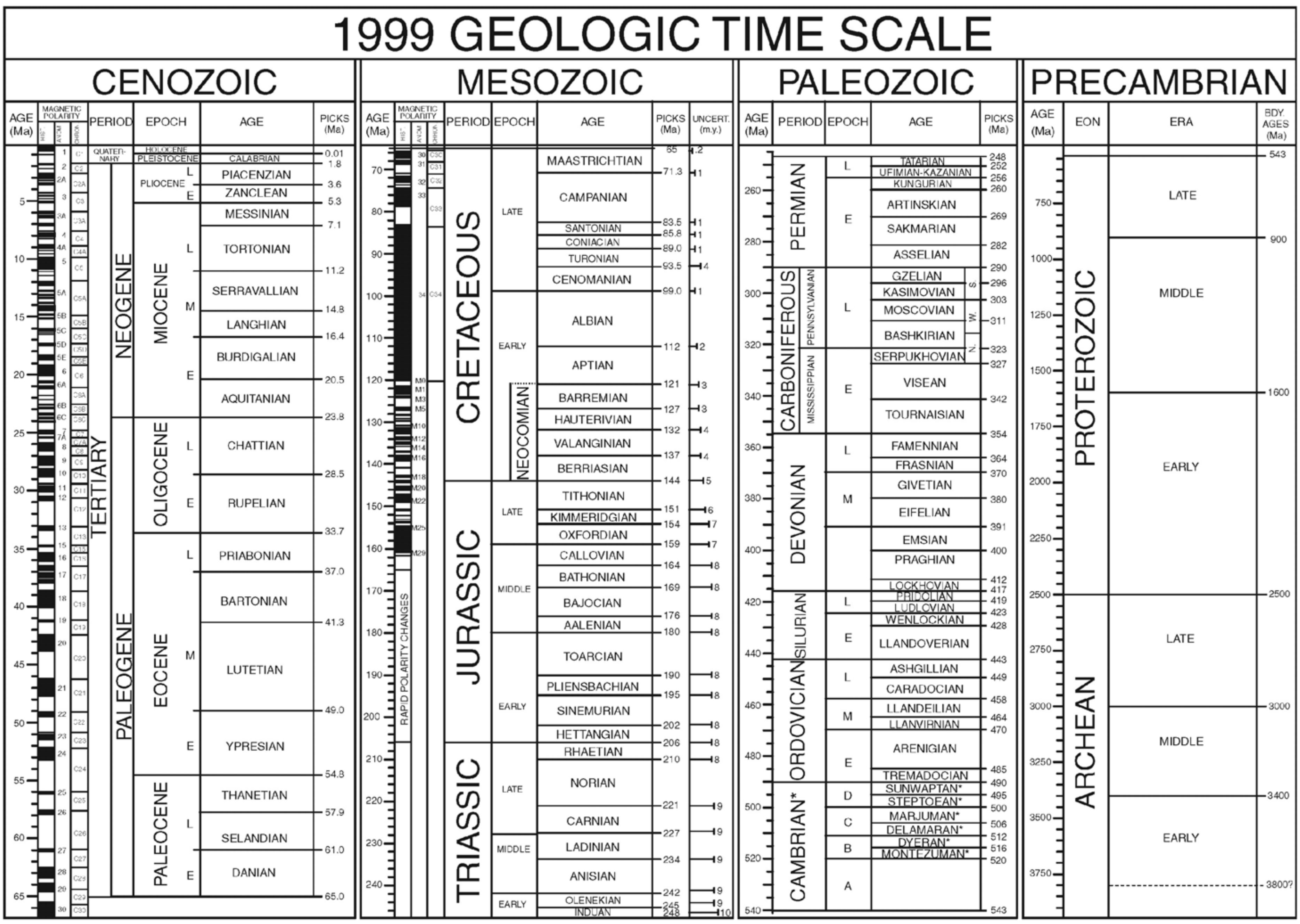
Source:
California Geological Survey, 2006, Simplified Geologic Map of California, Map Sheet 57, California Department of Conservation

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT ALL OPTIONS **Regional Geologic Setting**	FIGURE **A-1**



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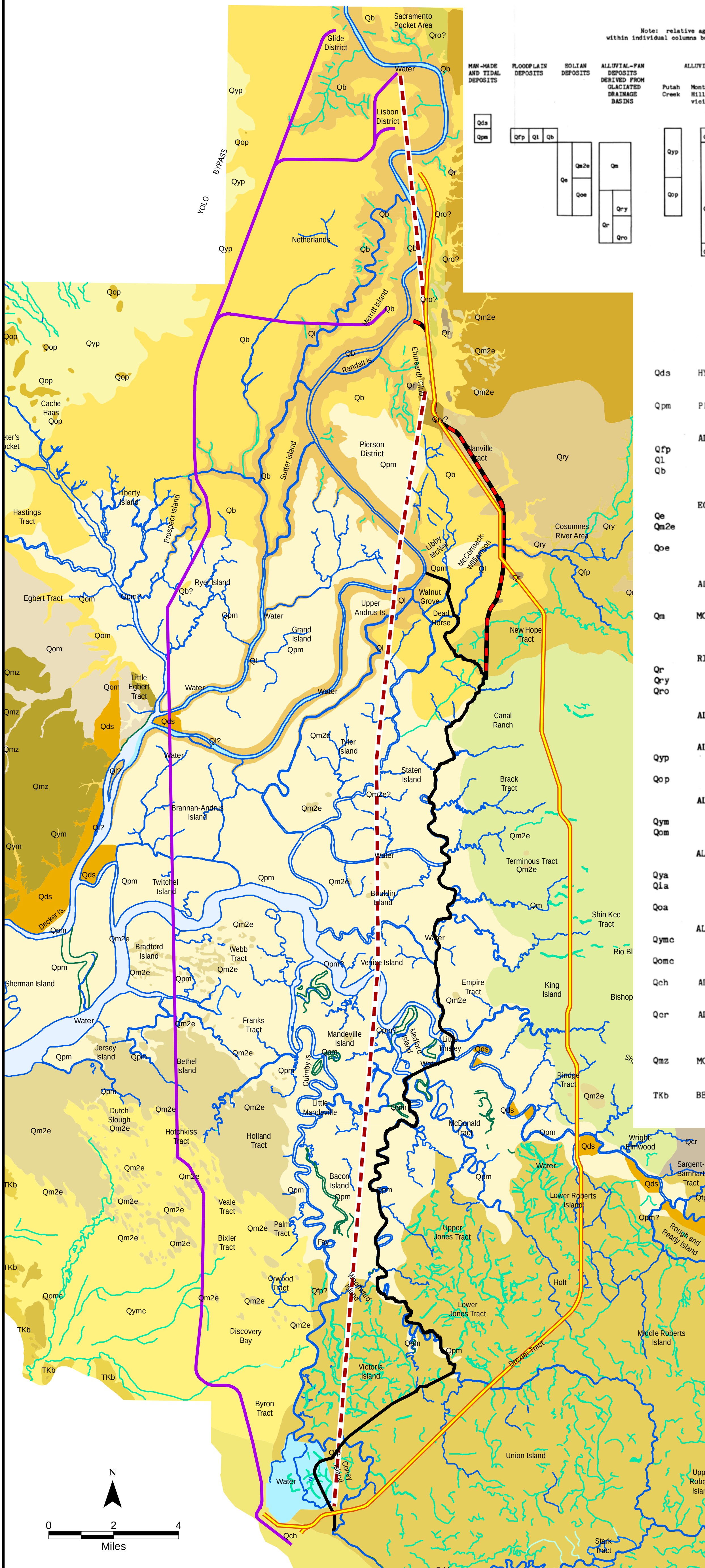
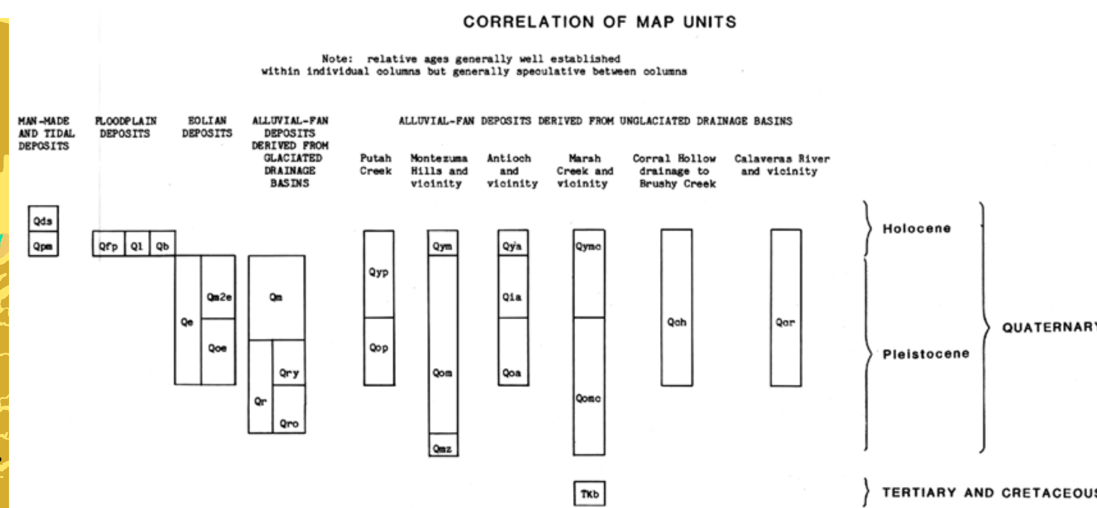
*International ages have not been established. These are regional (Laurentian) only. Boundary Picks were based on dating techniques and fossil records as of 1999. Paleomagnetic attributions have errors, Please ignore the paleomagnetic scale.

Sources for nomenclature and ages: Primarily from Gradstein, F., and Ogg, J., 1996, *Episodes*, v. 19, nos. 1 & 2; Gradstein, F., et al., 1995, SEPM Special Pub. 54, p. 95–128; Berggren, W. A., et al., 1995, SEPM Special Pub. 54, p. 129–212; Cambrian and basal Ordovician ages adapted from Landing, E., 1998, *Canadian Journal of Earth Sciences*, v. 35, p. 329–338; and Davidek, K., et al., 1998, *Geological Magazine*, v. 135, p. 305–309. Cambrian age names from Palmer, A. R., 1998, *Canadian Journal of Earth Sciences*, v. 35, p. 323–328.

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT ALL OPTIONS **Geologic Time Scale**	FIGURE **A-2**





DESCRIPTION OF MAP UNITS

Qds	HYDRAULIC-DREDGE SOILS (Holocene; post-1900)
Qpm	PEAT AND MUD OF TIDAL WETLANDS AND WATERWAYS (Holocene)
	ALLUVIUM OF SUPRATIDAL FLOOD PLAINS (Holocene) -- Consists of:
Qfp	Alluvial-floodplain deposits, undivided
Ql	Natural-levee deposits (Holocene)
Qb	Flood-basin deposits (Holocene)
	EOLIAN DEPOSITS (Pleistocene) -- Consist of:
Qe	Eolian deposits, undivided (upper Pleistocene)
Qm2e	Eolian deposits of upper member of the Modesto Formation (upper Pleistocene
Qoe	Older eolian deposits (upper Pleistocene)
	ALLUVIAL-FAN DEPOSITS DERIVED FROM GLACIATED DRAINAGE BASINS (Pleistocene) -- Consist of:
Qm	MODESTO FORMATION (Pleistocene) -- Dense stipple - loose sand, probably eolian; light stipple - loose sand and silt, chiefly fluvial; no stipple - compact silt and very fine sand
	RIVERBANK FORMATION (Pleistocene) -- As mapped, divided into:
Qr	Riverbank Formation, undivided (upper Pleistocene)
Qry	Younger unit of the Riverbank Formation (upper Pleistocene)
Qro	Older unit of the Riverbank Formation (upper Pleistocene)
	ALLUVIAL-FAN AND ALLUVIAL-TERRACE DEPOSITS DERIVED FROM UNGLACIATED DRAINAGE BASINS (Holocene and Pleistocene) -- Consist of:
	ALLUVIUM OF PUTAH CREEK (Holocene and Pleistocene) -- Divided into:
Qyp	Younger alluvium of Putah Creek (Holocene and(or) upper Pleistocene)
Qop	Older alluvium of Putah Creek (upper Pleistocene)
	ALLUVIUM OF MONTEZUMA HILLS AND VICINITY (Holocene and Pleistocene) -- Divided into:
Qym	Younger alluvium of Montezuma Hills and vicinity (Holocene)
Qom	Older alluvium of Montezuma Hills and vicinity (Pleistocene)
	ALLUVIUM OF ANTIOCH AND VICINITY (Holocene and Pleistocene) -- Divided into:
Qya	Youngest alluvium of Antioch and vicinity (Holocene)
Qia	Intermediate alluvium of Antioch and vicinity (upper Pleistocene)
Qoa	Oldest alluvium of Antioch and vicinity (upper Pleistocene)
	ALLUVIUM OF MARSH CREEK AND VICINITY (Holocene and Pleistocene)
Qymc	Younger alluvium of Marsh Creek and vicinity (Holocene and upper Pleistocene)
Qomc	Older alluvium of Marsh Creek and vicinity (Pleistocene)
Qch	ALLUVIUM OF CREEKS FROM THE CORRAL HOLLOW DRAINAGE TO BRUSHY CREEK (Holocene and(or) upper Pleistocene)
Qcr	ALLUVIUM OF CALAVERAS RIVER AND VICINITY (Holocene and(or) upper Pleistocene)
Qmz	MONTEZUMA FORMATION (Pleistocene)
TKb	BEDROCK (Tertiary and upper Cretaceous)

LEGEND

Holocene channels

- Channel subject to non tidal flow
- Channel subject to non tidal flow - queried
- Tidal channel
- Tidal channel - queried
- Tidal channel - probably abandoned before 1850

BDCP Initial Alignment Options

- East Alignment
- Through Delta Alignment, Existing Waterway
- Through Delta Alignment, Hood-New Hope Outlet
- West Alignment
- All Tunnel

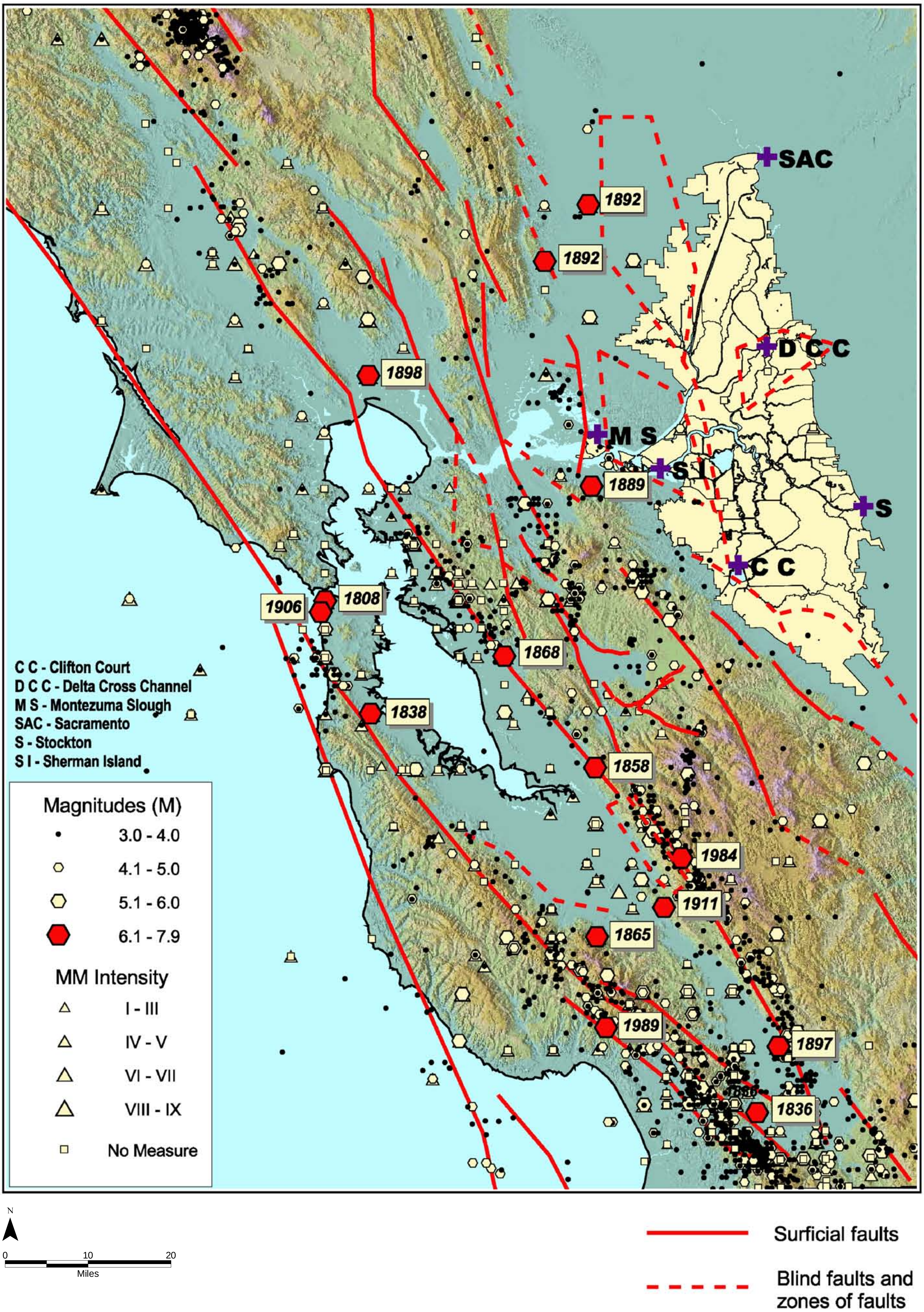
DATA SOURCE:
Atwater, Brian F., 1982. Geologic Maps of the Sacramento-San Joaquin Delta California. U.S. Geological Survey Miscellaneous Field Study MF-1401, Denver Colorado, 1982.

Atwater's report was converted to the digital geospatial format by David T. Hansen, G. James West, Patrick Welch, and Barbara Simpson, of the U.S. Bureau of Reclamation, Mid-Pacific Region, MPGIS Service, 2001.

This digital geologic data is based Atwater's hard-copy report. The digital data captures surficial geologic units, 1850 tidal wetland areas, late Holocene tidal and non tidal channels, and shorelines of perennial lakes, circa 1910. The field studies covered period of about 1978 to 1981. Data was compiled on 7.5-minute USGS topographic quadrangle maps for the area by Brian Atwater in 1982.

Geology of the Sacramento-San Joaquin Delta

FIGURE
A-3



Source: DWR, 2007a, DRMS Seismology TM

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT ALL OPTIONS	FIGURE A-4
	Active Faults and Historical Seismicity of the San Francisco Bay and Delta Region (M 3.0), 1800-2006	

Source: DWR, 2007a, DRMS Seismology TM

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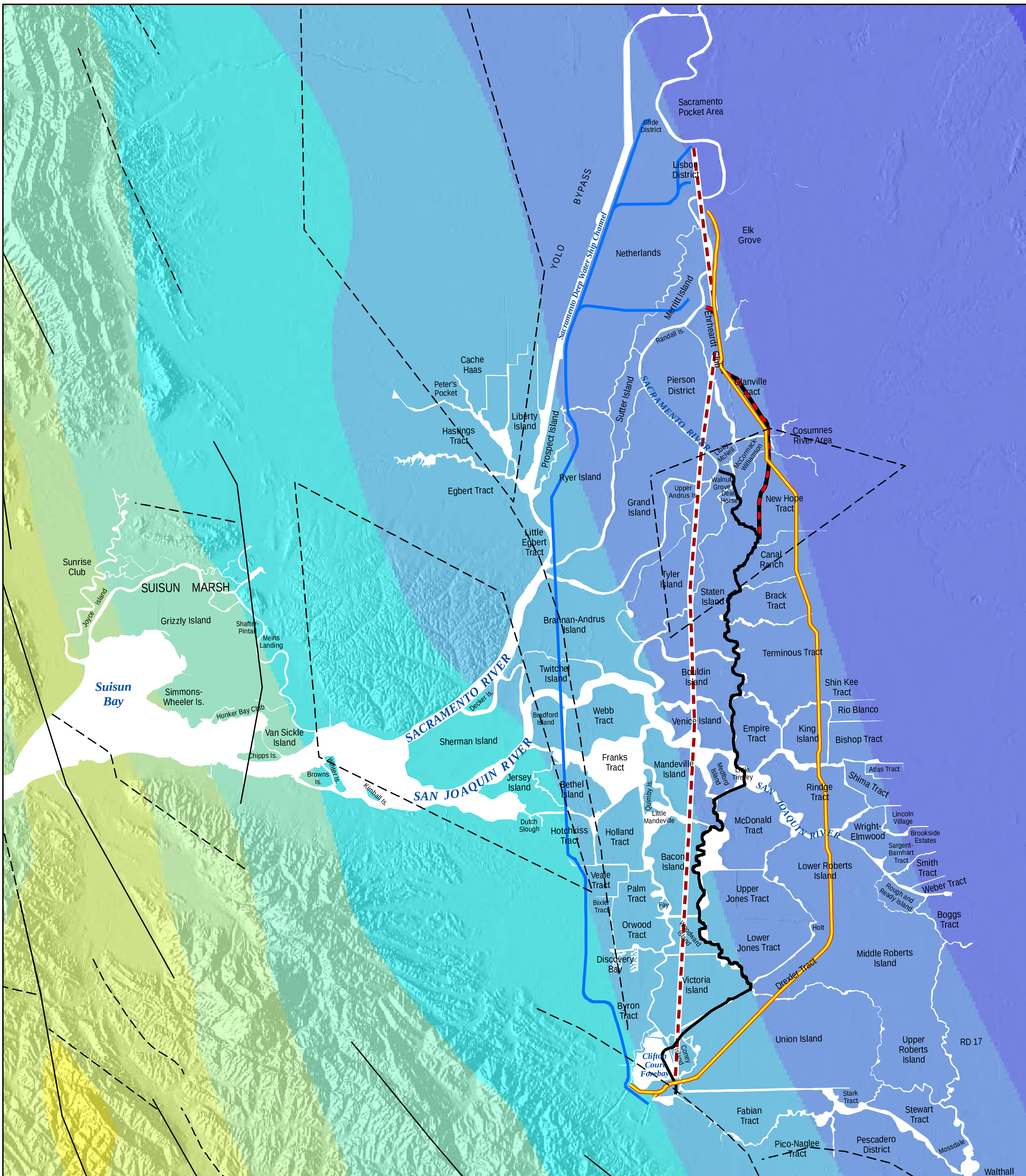
CONCEPTUAL ENGINEERING REPORT
ALL OPTIONS

Active Faults in the Delta Region

FIGURE
A-5



Faults

- Surficial fault used in the hazard analysis
- Blind fault used in the hazard analysis

BDCP Initial Alignment Options

- East Alignment
- Through Delta Alignment, Existing Waterway
- Through Delta Alignment, Hood-New Hope Outlet
- West Alignment
- All Tunnel

PGA (g), 200-year return period

- 0.00 - 0.10
- 0.11 - 0.15
- 0.16 - 0.20
- 0.21 - 0.25
- 0.26 - 0.30
- 0.31 - 0.35
- 0.36 - 0.40
- 0.41 - 0.45
- 0.46 - 0.50
- 0.51 - 0.55
- 0.56 - 0.60
- 0.61 - 0.65
- 0.66 - 0.70

Source: DWR, 2007a, DRMS Seismology TM

N

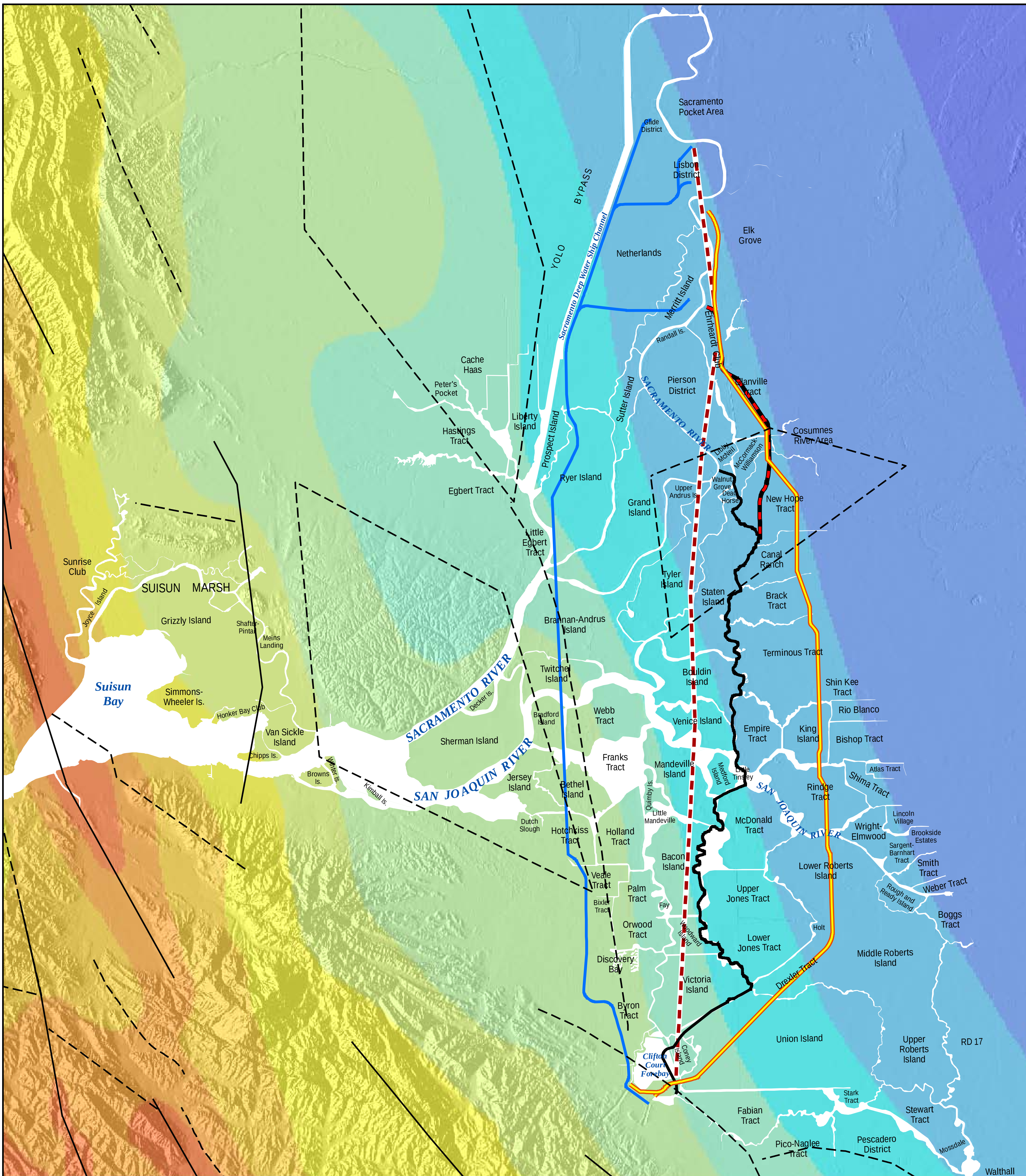
0 4 8
Miles

California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
ALL OPTIONS

PGA Hazard for a 200-year Return Period

FIGURE
A-6



Faults

- Surficial fault used in the hazard analysis
- Blind fault used in the hazard analysis

BDCP Initial Alignment Options

- East Alignment
- Through Delta Alignment, Existing Waterway
- Through Delta Alignment, Hood-New Hope Outlet
- West Alignment
- All Tunnel

PGA (g), 500-year return period

- 0.00 - 0.10
- 0.11 - 0.15
- 0.16 - 0.20
- 0.21 - 0.25
- 0.26 - 0.30
- 0.31 - 0.35
- 0.36 - 0.40
- 0.41 - 0.45
- 0.46 - 0.50
- 0.51 - 0.55
- 0.56 - 0.60
- 0.61 - 0.65
- 0.66 - 0.70

Source: DWR, 2007a, DRMS Seismology TM

N

0 4 8

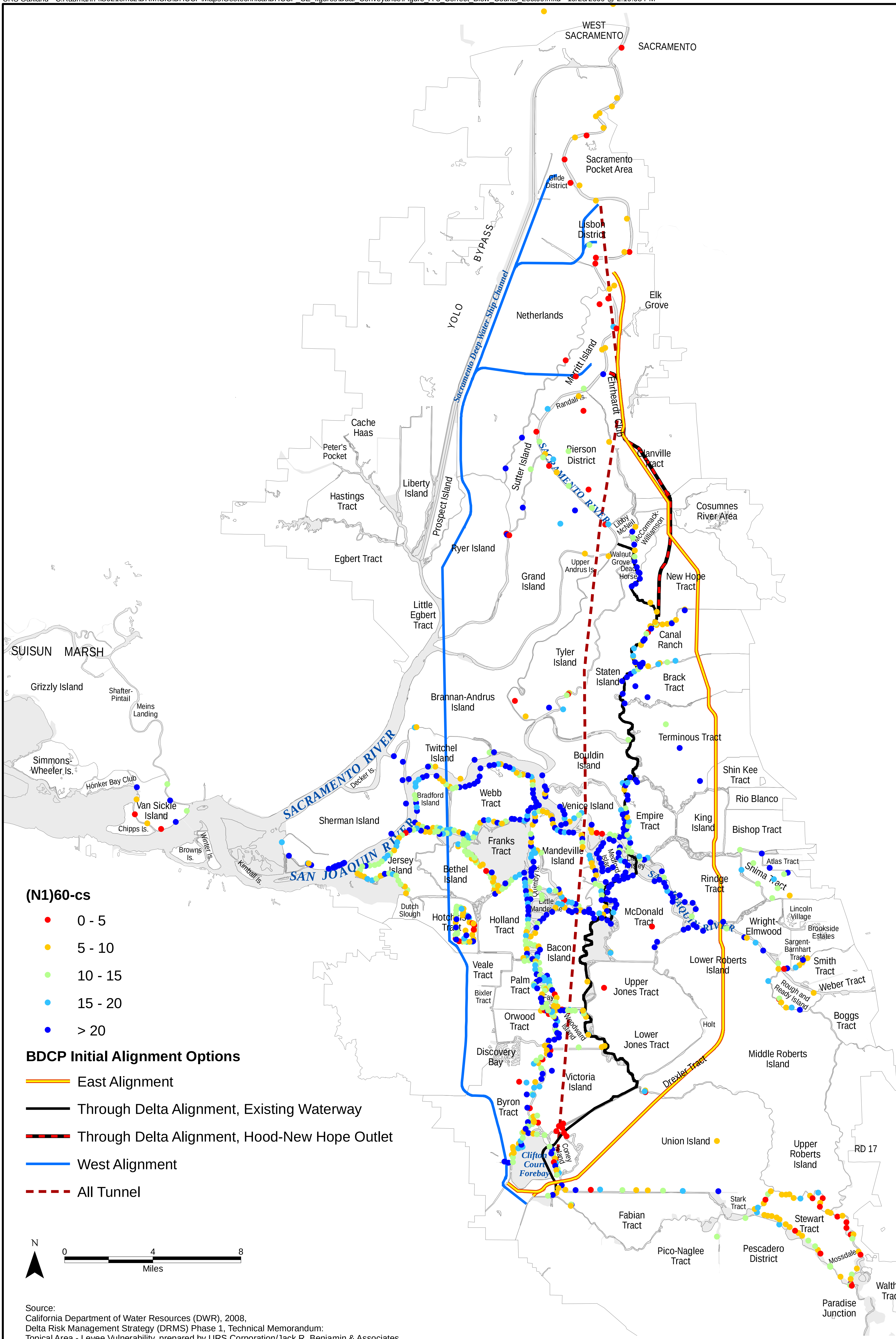
Miles

California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
ALL OPTIONS

PGA Hazard for a 500-year Return Period

FIGURE
A-7



Source:
California Department of Water Resources (DWR), 2008,
Delta Risk Management Strategy (DRMS) Phase 1, Technical Memorandum:
Topical Area - Levee Vulnerability, prepared by URS Corporation/Jack R. Benjamin & Associates

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	CONCEPTUAL ENGINEERING REPORT ALL OPTIONS **Corrected Blow Counts (N1)60-cs Distribution for Foundation Sand**	FIGURE **A-8**

APPENDIX B

TO THE

CONCEPTUAL ENGINEERING REPORTS

FOR CONVEYANCE OPTIONS IN THE DELTA

INTAKE FACILITY DEVELOPMENT AND SELECTION PROCESSES

DELTA HABITAT CONSERVATION AND CONVEYANCE PROGRAM

OCTOBER, 2009 VERSION

WORKING DOCUMENT

1.0 INTRODUCTION & BACKGROUND

Since the 1970s, several methods of conveying water through and/or around the Delta have been suggested and evaluated. Two general approaches (or a combination thereof) have been proposed to date for diverting Delta water while preventing the entrainment and impingement of juvenile to adult life stages of smelt, salmonids, and other fish species endemic to the region: a single consolidated intake at Clifton Court Forebay (CCF) or alternate points of diversion further to the north on the Sacramento River delivering water to CCF through an isolated conveyance facility.

Since 1991, actions have been proposed by the California Department of Water Resources (DWR) and the U.S. Bureau of Reclamation (Reclamation) to improve water supply for south Delta agriculture, improve fish protection, and improve the reliability of water supply for the State Water Project (SWP) and the Central Valley Project (CVP). In 2000, these proposed actions were incorporated into the state and federal multi-agency CALFED Bay-Delta Program (CALFED Program) which proposed a preferred program alternative that included both a single consolidated fish screened intake at CCF and new fish screened diversions on the Sacramento River near Hood. A programmatic EIR/EIS was prepared but later challenged in court.

In 2006, DWR and Reclamation prepared a joint environmental impact statement/environmental impact report (EIS/EIR) on implementation of the South Delta Improvements Program (SDIP). An alternative to install screened intake facilities at the north end of CCF was considered for that EIS/EIR (DWR and Reclamation, 2006).

In this CALFED alternative, CCF would become an isolated reservoir, receiving water from existing Delta water courses through screened intake facilities located at the north end of CCF. Both the Banks (SWP) and Jones (CVP) Pumping Plants would then draft off of the CCF. The proposed intake facilities, in conjunction with a new fish salvage facility, would prevent fish from entering the CCF and support continued collection and release of fish back to the Delta. This CALFED alternative was eliminated from further consideration in the SDIP EIS/EIR because of the overall uncertainty about the ability of a fish screen to operate sufficiently to protect the fish and about maintenance and operational constraints.

The Bay Delta Conservation Plan (BDCP) is now under preparation to develop a recovery plan for endangered and threatened species. This process will define the criteria that must be met by intakes for future diversions in the Delta. The Delta Vision Blue Ribbon Task Force has recognized BDCP as the primary mechanism for assessing and resolving impacts to endangered and threatened species.

Diverting water from the Sacramento River through screened intake facilities at the northern end of the Delta via an isolated conveyance facility has been proposed as a way of avoiding impacts to at-risk fish species and their habitats (the original peripheral canal). By relocating the point of diversion to the northern limits of the legal Delta, it is expected the threat to vulnerable species and life stages can be significantly decreased. For example, implementing new points of diversion on the Sacramento River will, for the most part, avoid the exposure of smelt species altogether.

To identify how best to protect at-risk fish species from entrainment and impingement at new intake facilities on the Sacramento River, a consortium of regulatory agency and industry experts, known as the Fish Facilities Technical Team (FFTT), was assembled in 2008 as part of the Bay Delta Conservation Plan (BDCP) process. Consistent with conventional fish protection technologies and practices, concepts proposed by the FFTT involved positive exclusion barriers, otherwise termed fish screens. Although alternative methods for fish exclusion such as physical guidance and behavioral devices were considered and have been studied since the 1960s (i.e., guidance louvers, lights, acoustics, temperature devices, chemicals, electric fields, bubble curtains), their reliability for diversion of the volume of water used by the SWP and CVP is unproven, and some studies have reported that these fish exclusion measures seldom exhibit efficiencies above 60% (Congress' Office of Technology Assessment, 1995). Modern fish screen designs offer much greater efficiencies, upwards of 90% (National Marine Fisheries Service [NMFS], 1997).

There is a broad array of options for fish screen technologies and intake arrangements/locations for diversion of water from the Sacramento River. Based on a maximum diversion of 15,000 cubic feet per second (cfs) of water, the FFTT developed its best assessment of potential fish-screened intake facilities, including design approach velocities; fish screen type; size and number (multiple versus a single intake); configuration and geometry; and location(s) to support both through Delta and isolated conveyance options. The intake options on the Sacramento River identified by the FFTT varied from three 5,000 cfs intakes each separated by approximately 2 miles, to as many as ten 1,500 cfs intakes spaced roughly 1 mile apart. Three different screen technologies were proposed for further consideration: in-river, on-bank, and cylindrical configurations.

The intake concepts identified by the FFTT can accommodate virtually any isolated conveyance option. These concepts were developed strictly looking at the requirements of diverting water from the river and not conveying that water beyond the limits of the levees bordering the river. Because conveyance methods and other factors outside the river limits play a role in the logistics and composition of feasible intake options, a "Value Planning" team (VPT) was assembled to further examine intake and conveyance compatibility.

The VPT used a value planning approach to formulate and analyze potential intake locations and sizes and fish screen technologies using an array of factors including fish protection, operational flexibility, maintainability, community impacts, conveyance requirements, and economics among others. An intake criteria and evaluation matrix was developed as a decision-support tool to compare the performance of a series of intake concepts using a list of factors. The purpose of this exercise was to identify features or trends among intake configurations that appeared to provide the greatest benefit.

The results of the value planning study conducted by the VPT, as well as the FFTT recommendations, were used to further evaluate the most promising intake options. Based on subsequent planning exercises, a preliminary arrangement of intakes was developed for the options to convey water from the Sacramento River to CCF. Intake facilities proposed for isolated conveyance options have the following attributes:

- In-river, vertical plate screen intakes
- Intake capacity of 3,000 cfs for a total of five intakes to convey a maximum of 15,000 cfs.

- Intakes located between the Pocket Area in Sacramento south to Courtland (Figure 1)

These parameters for intake facilities are used for all Isolated Conveyance options considered.

The Through Delta Facility (TDF) conveyance option would convey water to CCF through existing Delta water courses. With this option, a new fish screen intake and salvage facility would be located north of CCF to exclude fish from entering the reservoir. Also, in addition to the Delta Cross Channel, an arrangement consisting of two 2,000 cfs in-river intakes located between Hood and Courtland to supplement conveyance into the existing channels is proposed.

The Dual Conveyance Facility option combines conveyance through existing Delta water courses to CCF in combination with an isolated conveyance facility. The preliminary arrangement of intakes for the isolated conveyance is proposed for this option.

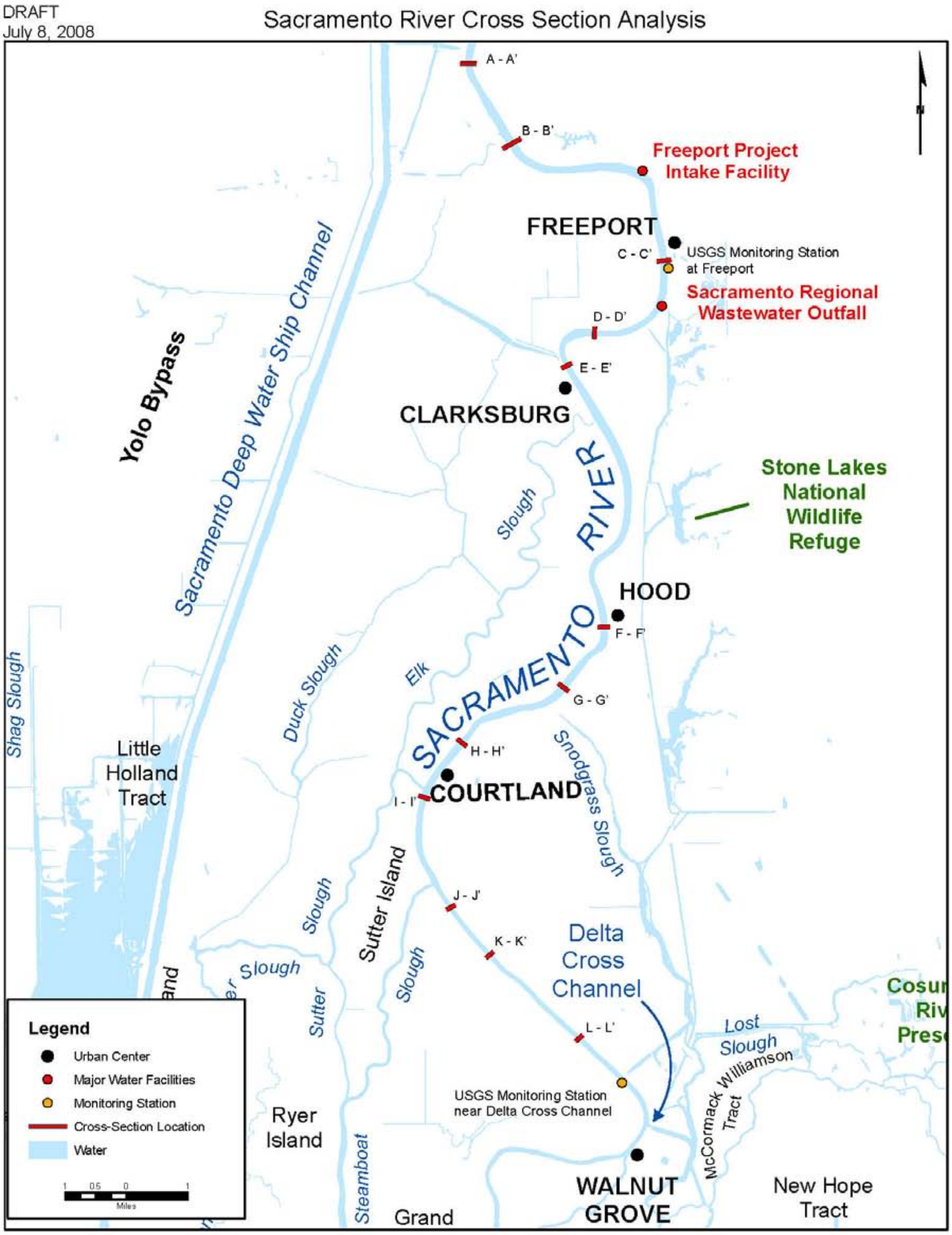


Figure 1. Overview of Potential Diversion Locations on the Sacramento River

2.0 DESCRIPTION OF AVAILABLE INTAKE TECHNOLOGIES

Two methods exist for preventing entrainment of fish in diverted water; either positive exclusion barriers (fish screens and infiltration galleries) or alternative behavioral devices. Fish screening physically prohibits fish from being diverted from the natural watercourse, whereas the latter relies on eliciting avoidance behavior of the target species. Although behavioral devices are under ongoing study, they have yet to demonstrate the consistency and effectiveness of physical barrier screens. Very few behavioral devices have been permanently installed, and most that have been installed have ultimately been replaced with positive barrier screens. Behavioral devices are considered experimental and are only accepted by regulatory agencies if evidence of effectiveness can be proven.

On the other hand, several fish screen types and configurations are available. Numerous configurations exist and have been employed with varying degrees of success. Commonly used screen configurations are as follows: vertical, horizontal, or inclined fixed plate screens; drum screens; cylindrical screens; conical screens; Eicher screens; modular inclined screens; and submersible traveling screens (see photos below). The screen technologies determined by the FFTT to be most appropriate for the BDCP are flat plate (vertical or inclined) and cylindrical.

Cylindrical Screens

Drum Screens

Chevron Screen (Dry)

Chevron Screen (In-Service)

Inclined Flat Plate Screen

Conical Screen

Traveling Screens

Horizontal Coanda Screen

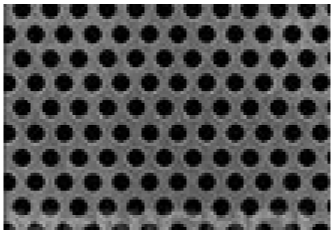
Vertical Cylindrical Screens

ISI Cylindrical Screen

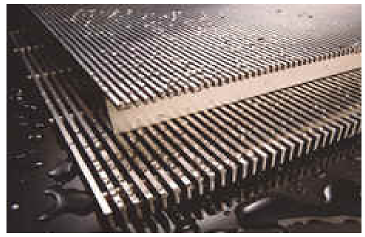
Inclined Flat Plate Screen

Screen with Air Burst Operation

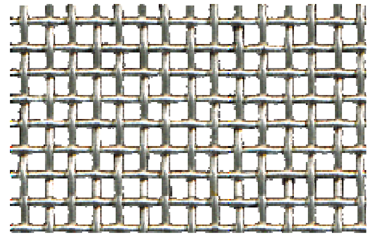
Screen materials typically used to exclude fish from raw water diversions are: slotted wedge wire; perforated plate; or woven wire fabric (see photos below).



Perforated Plate



Wedge Wire



Woven Wire

In consideration of the above variants, several screened intake concepts are proposed for the initial conveyance alignments. Fish screened intakes, in the context of this document, consist of hydraulic structures located at discrete points on the Sacramento River and at the north end of CCF. These facilities are arranged to divert raw water via gravity to a receiving pipeline, reservoir, or pumping plant. The intakes are designed to prevent entrainment and impingement of at-risk fish species, spanning from juvenile to adult life stages. River intakes keep fish in the main stem river channel, whereas the CCF intake diverts fish into collection facilities or a return channel.

The proposed intakes will be operated in response to river hydrology, tidal influence, time of use, water demand, and other biological, operational, and stressor constraints yet to be defined. The intakes may function independently or as a collective network to suit diversion needs and specific operating criteria. Intakes will function to screen fish from diverted water while offering the flexibility of optimizing export water quality and quantity.

In general, intakes include both static and dynamic elements tailored to suit site-specific conditions and design constraints. Generally speaking, intakes consist of a foundation element (e.g., cellular sheet pile perimeter/hydraulic cut-off, end-bearing or friction piles, and mat slab or subgrade reinforcement); reinforced concrete slab, walls, and diaphragms; fish screening and flow baffle elements; mechanical screen cleaning systems; electromechanical hoist systems (e.g., gantry or bridge cranes); diversion conduits; control elements (e.g., programmable logic controller [PLC], supervisory control and data acquisition [SCADA], instrumentation and control [I&C], power distribution, lighting); and miscellaneous civil site modifications.

The river intake options proposed by the FFTT consist of in-river, on-bank, and cylindrical intakes sized to accommodate their respective design flows while meeting fish protection criteria mandated by National Oceanic and Atmospheric Administration (NOAA) fisheries and California Department of Fish and Game (CDFG) for anadromous salmonid species (0.33 feet per second [fps] approach velocity). The river intakes would also be expected to provide reasonable protection for the weaker swimming Delta fish species (Delta smelt, ***Hypomesus transpacificus***) given the limited temporal distribution patterns and the ability to adjust diversions during the times of year when these species occupy the river near the proposed intake(s). The response would be to curtail diversions when smelt are present in the proximity of the intakes. According to United States Fish and Wildlife Service (USFWS) trapping data, this could be required between the months of February and June.

For the CCF intake proposed for the Through Delta and Dual Conveyance options, the intake would be a single-source intake made up of several chevron screen intake modules coupled with either a common collection, handling, transport, and release (CHTR) facility similar to the existing Skinner Fish Facility or a common return channel conveying screened fish back to Delta waters. This intake option is sized and configured to suit a more conservative fish protection criteria applicable to Delta fisheries (0.20 fps approach velocity) since the distribution of Delta smelt in the vicinity of the intake is greater than in the Sacramento River. This approach velocity is not a published criterion but corresponds to previous biological opinions and guidelines produced by the USFWS and NOAA Fisheries.

Descriptions and preliminary illustrations of potential intake configurations under consideration are provided below. The types of intake technologies and configurations considered are as follows:

1. In-river intakes
2. On-bank intakes (single or dual)
3. Cylindrical intakes
4. Centralized (chevron) intake

2.1 Intake Types

All four types of screened intakes listed above constitute biologically protective options and will contribute to the conservation and restoration of listed fisheries within the Delta by minimizing entrainment and reducing the potential for predation. A variety of river intake technologies have been evaluated for diverting water from the Sacramento River including single in-river intakes, dual or single on-bank intakes, and a bank of cylindrical screens at each diversion location. Centralized intakes have been considered for terminus diversion locations in the south Delta for the Through Delta and Dual Conveyance options.

Each river diversion facility consists of an intake structure, a series of conveyance conduits, a partitioned sedimentation basin, solids dewatering lagoons, a low-head pumping plant, and conveyance piping to a point of discharge into the proposed isolated conveyance canal. The aforementioned intake facilities were considered in detail and are further described below. Based on an intake technology selection workshop conducted in January of 2009, the preliminary intake technology used for conceptual design for isolated conveyance options is the in-river type. An example of this intake type is provided on Figure 2.

2.1.1 In-River Intakes

In-river intakes are a type of screened diversion constructed at an intermediate point in the river cross-section, detached from the bankline, with an aerial bridge span to provide access from a bordering levee. This configuration consists of a long, narrow, streamlined structure in the river housing dual, vertical screen banks at opposing faces. The economy of this concept capitalizes on greater river depths, due to constructing the facility mid-channel, and dual screen faces which reduces the structure’s overall length by more than half that of a traditional single screen bank structure. The benefits of in-river intakes are that they can be constructed with greater screen height and provide more than twice the screen surface area of an intake with a single screen bank. This means an in-river intake can divert more than twice the flow rate of a single-sided intake.

Relatively speaking, the footprint of an in-river intake is roughly one-third that of an intake with a single screen bank. This theoretically reduces the exposure time of juvenile fish and the threat of impingement by a similar margin, assuming all other hydraulic factors were to remain constant. Additionally, an in-river intake can divert more than twice the flow rate of a single-sided intake constructed on the bankline due to greater screen height from locating it in the deeper section of the river and because of dual screen banks. An in-river intake, considering a potential 25-foot screen height, requires a structure roughly 270 feet long when designed to state (California Department of Fish and Game [CDFG]) and federal (National Oceanic and Atmospheric Administration [NOAA]-Fisheries) salmonid criteria requiring 3 square feet (ft^2) of screen area for each cubic foot per second of water diverted. When designed to smelt criteria (United States Fish and Wildlife Service [USFWS]) guidelines requiring 5 ft^2 per cfs, a structure approximately 380 feet long is needed.

This intake type consists of a reinforced concrete structure subdivided into individual bays that can be isolated and individually managed. Each bay is fitted at opposing faces with vertical flat plate screen panels, flow control baffles, and bulkhead provisions for isolation. Water is routed from each bay through respective diversion conduits under the adjacent levee to a receiving sedimentation basin(s) equipped with sediment removal mechanism(s). The sedimentation basin structure is directly coupled to a low-head pumping plant.

Examples of in-river intakes are the City of Sacramento's raw water diversions on the Sacramento and American Rivers providing raw water from the rivers to the City of Sacramento's surface water treatment plants (see photos below). These facilities feature design capacities between 250 to 300 cfs and include pumping equipment that is integral to the structures including pumps and motors, electrical switchgear, and motor control centers.

City of Sacramento's In-River Intake

The example depicted in Figure 2 is an in-river intake with a 25-foot screen height. This intake requires a structure roughly 270 feet long to meet salmonid criteria versus a structure 380 feet long designed for smelt criteria. The intakes discussed in this document are sized to provide sufficient screen area in accordance with federal and state standards for entrainment and impingement prevention of smelt and salmonids. The intakes would include ample screen area to accommodate a 0.2 fps average approach velocity at their design diversion rate and a maximum screen opening size of 1.75 mm (0.0689 inch).

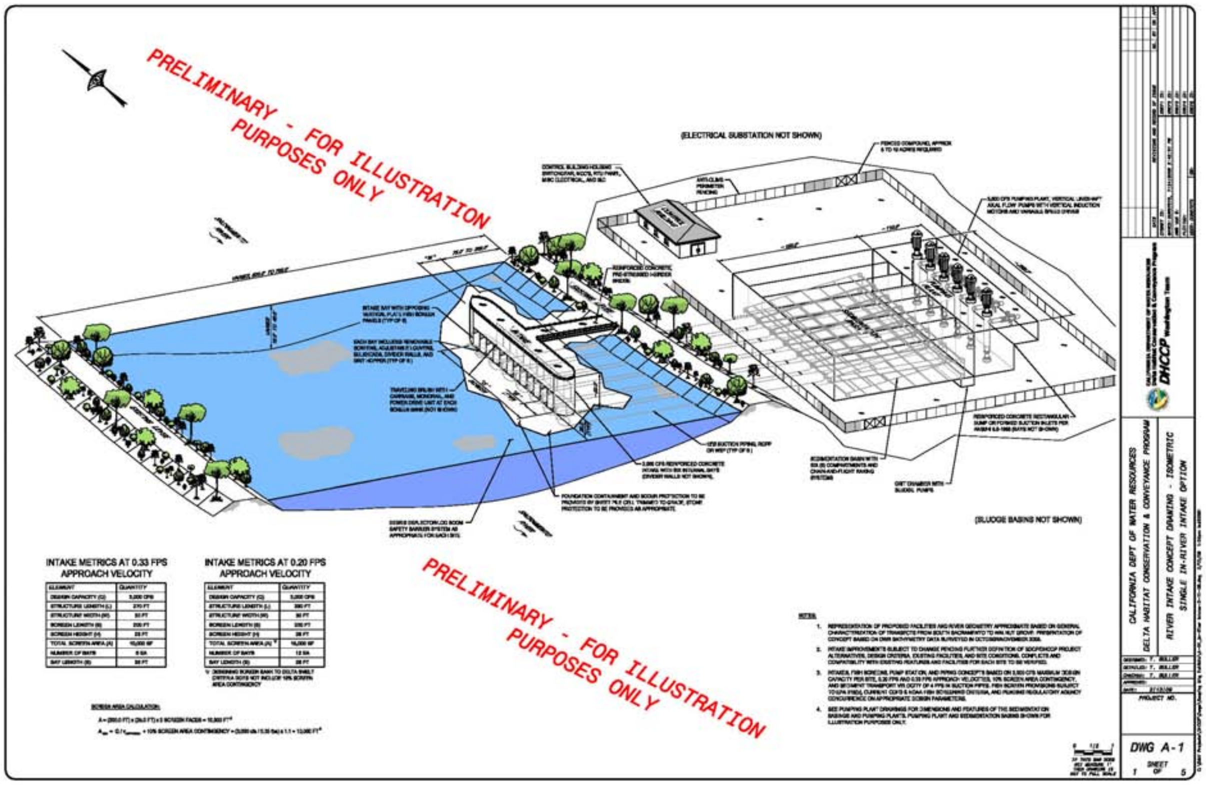


Figure 2. In-River Intake Concept

2.1.1.1 Intake Facilities

In-river intakes require physical activity within the river channel primarily involving heavy civil and marine construction and reinforced concrete work. In general, in-river intakes require a cellular cofferdam or sheet pile curtain for producing a dry work space during construction. The sheeting also ultimately serves as an integral foundation containment system interlocking subgrade and foundation elements into a cohesive unit. Either a dewatering field is constructed within the cell or an underwater cap or tremie slab is placed to provide a lower sealing membrane. The sheet pile cell is trimmed underwater after interior work is complete, leaving a permanent hydraulic cut-off about the perimeter of the finished structure.

Relatively soft soils are expected to be present in the Sacramento River; therefore, foundation support elements are likely required to underpin intake structures. Over-excavation and import of engineered fill is expected to be cost prohibitive and impractical given the depth of material removal needed to expose suitable foundation material. Interior to the sheet pile cell, a matrix of piles is a common means of supporting the foundation slab-on-grade of intake structures. This approach provides structural stability and prevents differential settlement of the intake by transferring loads to competent, underlying soil strata. This is a critical component of providing a sound platform for vertical intake construction.

An in-river intake consists of a reinforced concrete structure segmented by divider walls into discrete sub-compartments or bays. The structure is subdivided to provide manageable, stand-alone partitions that can be individually isolated. Each bay contains its own screen and baffle elements that can be readily removed at the top side for inspection, repairs, and maintenance for reasons such as bio-fouling, mussel attachment, and repairs. The bays would be aligned to lend symmetry in velocity fields and screen hydraulics at each bay and to maintain interchangeability of components. With a compartmentalized approach to the intakes, structural efficiency is improved, operational flexibility and reliability are enhanced, and sediment management features can be reasonably incorporated into the structure to minimize the impacts to operations.

The invert of the screen portals through the intake structure would be offset roughly 3 to 5 feet above the river floor to act as a sill for preventing excessive sediment load entry into the structure. Since duning within the river is a known phenomenon according to USGS studies, this design feature will help to reduce the impacts of sediment buildup which commonly plagues river intake performance. In addition, to mute the effects of sedimentation interior to the intake structure, the floors of the individual bays are contoured and diversion conduits are sized to maintain sediment movement through the intake.

The principal screen elements of the fish screening component of the intake consist of:

- A bank of vertical, structurally reinforced wedge-wire screen panels. Flat plate screen panels are self-contained and include stainless steel wire fabric, backing rods, and structural support and framing members. The function of the screen panels is to prevent fish entrainment into the intake structure. The screen fabric features an open area and wire spacing to prevent impingement and entrainment of fry-sized salmonids and juvenile smelt.
- A series of self-contained flow control baffle assemblies (e.g., ganged louvers or porosity control panels). Adjustable baffles are placed behind the screens to uniformly distribute approach velocities at the screen face.
- Structural bulkheads or blanks will be used for isolation of the individual bays.

The exterior faces of the in-river intake feature slot embeds or guide rails for insertion and removal of the above components. A rolling gantry crane with an electromechanical hoist system is proposed at the top side of the structure to facilitate panel removal and installation from either side. As an alternate, and possibly an owner preference, a boom truck or mobile crane could be used in lieu of a permanent on-site crane for the requisite activities. This removal/insertion concept is critical to the O&M of the intakes. Not only does it enable the individual diversion circuits to be isolated and dewatered, but it allows screens and baffles to be readily examined, cleaned, repaired, and restored to working condition. Construction of intakes on the Sacramento River in recent years with fixed features reveals the importance of isolatable and removable elements.

Another basic element for keeping the intake in operable condition is the incorporation of a screen cleaning system. A variety of technologies are used in the industry such as mechanical brushing, compressed air scouring, and submersible water jets. Based on performance history of all types, a traveling brush cleaning system is proposed to maintain unimpeded screen faces. This cleaning system consists of a monorail beam, brush carriage, brush arm, strip brushes, drive leads, limit switches, and a stationary power transmission unit. The brush system is designed to make a complete travel cycle at intervals as required to maintain a clean screen face. A dedicated system would be provided for each screen bank. As an alternate to the fixed screen panel and brushing system described, a traveling screen system with a polymer screen belt and stationary brush/water jet system may be a viable option for consideration.

With the exception of screen cleaning systems, the intakes function statically with no moving parts to exclude fish from diverted water. The top of the screens, or portal soffit through the intake wall, is set at an elevation corresponding to river stage under a given hydrologic/ diversion condition. It is important that the submerged screen surface area be consistent with the diversion rate allowed at a given river flow to avoid exceeding the design approach velocity. By designing the fish screen panels to capitalize on river depth, the intake structure footprint can be reduced. The required screen surface area and height of the fish screen panels dictate the longitudinal dimension of the intake structure. Hydraulics and stability drive the width dimension. The ultimate dimensions of the intakes will need to correspond to final operating rules being developed by BDCP.

In the vertical dimension, the intake structure is elevated to match the top of the bordering levee for flood protection and access reasons. The intake features a top deck that can be readily accessed by O&M equipment. An aerial bridge span is needed to link the intake structure with the adjacent embankment and roadway. It is proposed that a pre-stressed I-girder bridge be used, resting atop an abutment at the levee end and atop the intake structure at the opposite end. This bridge type is commonly used for utility and roadway bridges involving spans consistent with those under consideration and require little to no maintenance. More importantly, it is expected the crossing can be made in a single span, therefore requiring no piers in the river to support it.

Ancillary features of the intake structure include hatches and ladders/landings for man-way access. The structure perimeter is bounded by fencing and/or guardrail. The structure includes shaped pier nosings at the upstream and downstream limits to deflect debris load, minimize hydraulic drag, and avoid producing adverse hydraulics.

2.1.1.2 Conveyance Facilities

In-river intakes can include an integral pump gallery located on top of the intake or buried conduits underneath the adjacent levee to connect the intake with a landside pumping plant. As the concept is currently envisioned for this project, a series of conduits is preferred because constructing a pumping plant and sediment management facilities within the river section requires considerably more structural area in the river channel. Additionally, the electromechanical equipment of a landside pumping plant is less susceptible to flood damage than if this equipment were located on the river side of the levee. Finally, a landside pumping plant would cause less construction impacts to the bordering levees and roadways.

Multiple parallel conduits are needed for several reasons. The sheer size of the intakes exceeds the capacity of single, commercially available conduits. Also, the ability to isolate individual conduits for inspection, maintenance, and repair is important. Connecting independent piping allows individual intake/sedimentation trains to be taken off-line, while leaving the remaining others in service.

Each conduit is connected to its respective intake bay via a downward converging hopper bottom. A special pre-fabricated fitting will connect the hopper to the conveyance conduit, and also combine flow from pairs of adjacent intake bays. The fittings and conduits interior to the cofferdam shell will be encased in concrete and supported by the foundation pile matrix to form a cohesive system that is not susceptible to differential settlement. With a relatively deep intake structure and an upward, vertical conduit connection to the bays/hoppers as proposed, piping profiles are roughly 30 feet below the toe of the adjacent levee. Conduits will be installed using single-pass microtunneling and pipe jacking methods. Welded steel or reinforced concrete pipe with bell-and-spigot joints are likely candidates for conveyance piping. Cathodic protection and anti-fouling coatings will need to be further evaluated in final design.

2.1.1.3 Sediment Management Provisions

River diversions throughout the Central Valley are highly susceptible to high bed loads and sedimentation requiring labor intensive removal to keep the intakes functioning as designed. Several measures are proposed to offer a practical sediment management operation minimizing the impacts of sedimentation. The first is to offset the floor of the intake several feet above the river bottom to provide an intake sill. Large concentrations of bed load are transported through the Sacramento River corresponding to hydrology, most of which is transported along the channel bottom. Raising the invert of the intake helps to reduce sediment entry into the structure.

By incorporating aggressively sloped hoppers or recesses into the floor of the intake structure, buildup in the intake bays cannot occur as long as each diversion train is frequently used. This design will allow accumulated material to slough into the connected conduit which is used as a transport mechanism for moving sediment from the intake to the landside of the bordering levee. Sizing the conduits to produce incipient velocity for the largest grain size capable of passing through the fish screen material (1.75 mm or 0.0689 inch) can minimize sedimentation within the conduits. However, periods of inactivity in piping circuits can lead to sediment deposition. Therefore, the intakes include mechanisms for installing bulkheads at each bay so each diversion train can be isolated and dewatered for physical access and material removal if needed.

The in-river intakes draw water from the river, convey it beneath the adjacent levee, and discharge it into an in-line sedimentation basin system upstream of a landside pumping plant. The object of a sedimentation basin is to settle out some portion of the sediment grain size distribution before the water reaches the pumping equipment. This will reduce the effects of abrasives on pump impellers and wear rings and also lessen sediment loading in the downstream conveyance infrastructure.

The sedimentation basin is channelized such that one trough in the basin can be dedicated to a pair of intake bays. The sedimentation basin can include either individual chain-and-flight raking systems to drag deposited sediment from their respective channels to solids pumps or a hydraulic removal/vacuum system. In either case, the removal systems pump the concentrated sediments to solids lagoons which decant the water and return the supernatant to the pump wet well. Either system can be automated to alter cycle frequency in response to sediment loading conditions.

2.1.1.4 Conceptual Pumping Facilities

In general, pumping plants consist of rectangular sumps housing a series of vertical column, enclosed lineshaft axial flow pumps. The pumps are suspended within individual bays of the wet well and paired with corresponding intake bays and sedimentation basin channels. Pumping equipment is coupled with medium-voltage, vertical lineshaft motors. The pumping plants lift water diverted from the river and discharge it into conveyance pipelines which route it to points of discharge within the isolated conveyance canal via energy dissipaters or afterbays. Design of pumping equipment requires duty capabilities corresponding to variable river stage and canal tailwater elevations. The ability to adjust output is important to avoid exceeding approach velocity limits at the screen banks. This output control will be accomplished using variable speed (variable frequency) drives.

2.1.2 On-Bank Intakes (Single or Dual)

On-bank intakes consist of a single bank of flat plate screen panels, either aligned vertically or with an angle of repose. This type of intake configuration is located near and parallel to the bankline, is accessible from the adjacent embankment, and produces little relative projection or silhouette within the water column. This screened intake configuration is used in river systems for facilities of comparable diversion capacities.

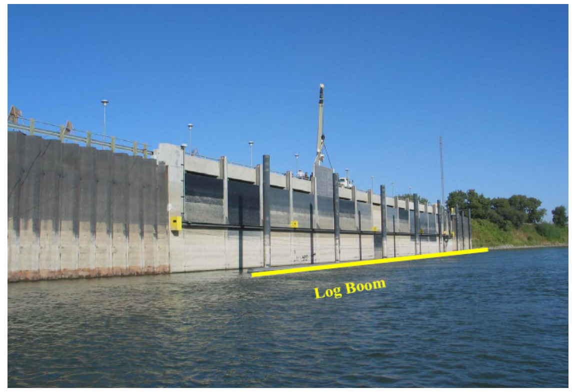
Similar to in-river intakes, this type of intake structure consists of a reinforced concrete shell with an integral steel sheet pile curtain. The primary differences are:

- On-bank intakes involve greater in-river disturbance because construction footprints are more than twice the area of an in-river intake for a given rate of flow.
- On bank intakes span a much greater distance of river bankline than in-river intakes.
- On-bank intakes involve shallower screen heights due to lesser depth at the bankline as opposed to mid-river.
- On-bank intakes are embedded within the adjacent levee and therefore do not require an access bridge.
- On-bank intakes involve construction of a set-back levee to maintain vehicle passage and to provide ample staging area during construction.

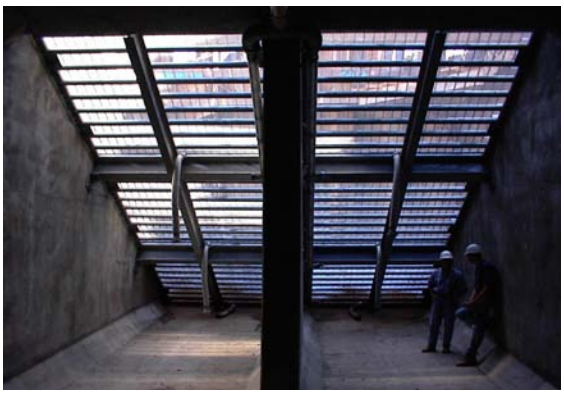
Otherwise, the components of an on-bank intake are essentially the same as for in-river intakes as described in Section 2.1.1.

Similar installations include the Tehama Colusa Canal’s point of diversion at Red Bluff (3,000 cfs), and other Sacramento River diversions for the Glenn-Colusa Irrigation District (3,000 cfs), Sutter Mutual Water Company (960 cfs), Freeport Regional Water Authority/East Bay Municipal Utility District (286 cfs), Princeton-Codura-Glenn/Provident Irrigation Districts (600 cfs), and Reclamation District 108 (Wilkins Slough and Pondstone Pumping Plants [300 cfs]). The proposed concepts require the use of one or more on-bank screens at each location. For reference, examples and renderings of on-bank intakes are provided on Figures 3 and 4 and in the photographs below.

Sutter Mutual On-Bank Intake



RD 108 On-Bank Intake



PCGID/PID On-Bank Intake

PCGID/PID On-Bank Intake

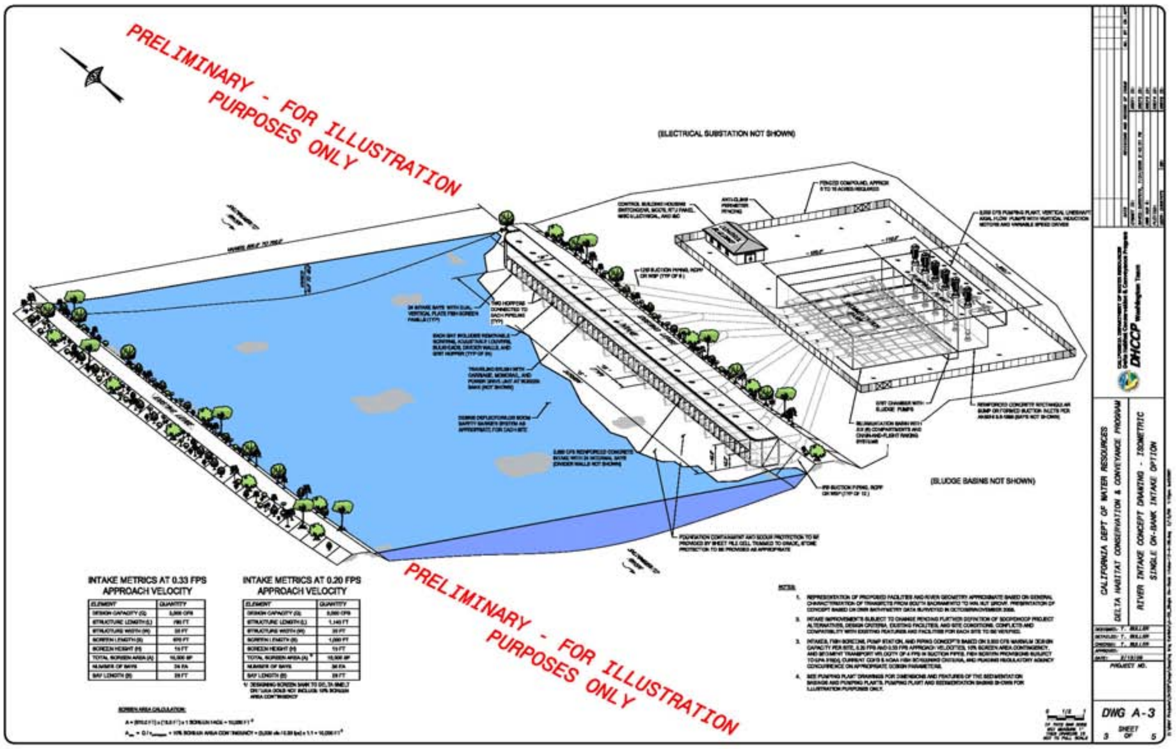


Figure 3. Single On-Bank Intake Concept

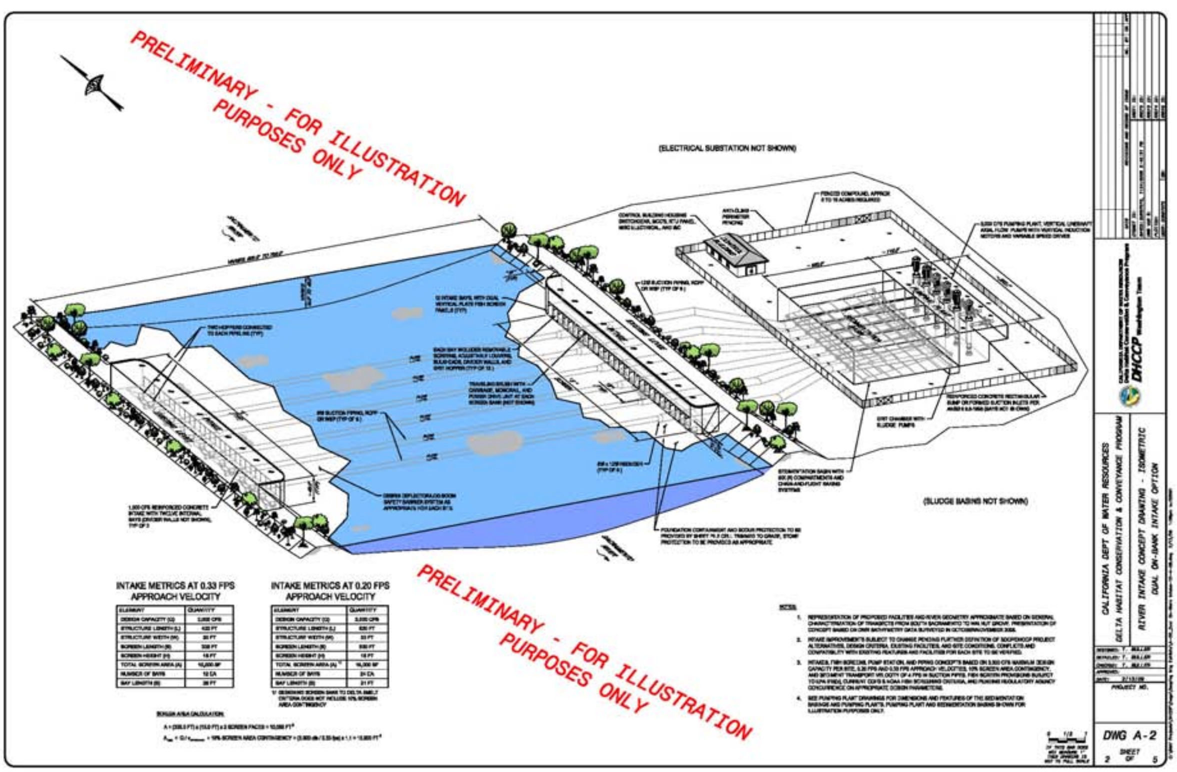


Figure 4. Dual On-Bank Intake Concept

2.1.3 Cylindrical Intakes

Cylindrical screens are a significantly different style of fish screen compared to those described above. This type of screen is more aptly suited to smaller diversions (100 cfs and less) due to its relatively small screen area. This screen configuration is generally used to retrofit existing unscreened diversions but can also be used for new intake facilities. A bank of multiple units can be combined to provide greater capacities.

Cylindrical screens are pre-manufactured modules assembled in a T-type configuration with screen elements at each side of a manifold connection. The screens are self-cleaning and retrievable. Each module requires above grade guide rails or tracks for guiding the screen into the water body over the conduit inlet or docking station. An alternate arrangement involves superstructure in the channel to support an intake conduit/pump casing and the guide rail system.

The advantages of this screen configuration are its relatively low cost, its shallow profile, its retrievable and interchangeable design, and its ability to be constructed in the "wet" without cofferdams and dewatering systems. Drawbacks to this screen configuration include the number of moving parts and hydraulic components, exposure to impact damage from debris/bed load, single-source manufacturing, and potential for producing structure in the watercourse which supports predation. Application of this screened intake concept requires further evaluation but has proven to be viable for other small diversion installations on the Sacramento River.

Examples of this screen type on the Sacramento River are a variety of installations for Reclamation District 999 near Clarksburg, Alameda County Water District (ACWD), and Maxwell Irrigation District (MID). The proposed concept requires a series of cylindrical screens to provide the desired capacity at each location, as well as in combination with other screen-type facilities. Photographs of cylindrical screened intakes are provided below.

RD 999 Cylindrical Screen

RD 999 Cylindrical Screen

ACWD Cylindrical Screens

ACWD Cylindrical Screens

Cylindrical Screen on Ramp

MID Cylindrical Screens

This intake concept consists of T-type screens lowered into and raised from individual docking stations using electromechanical winching and guiderails. The cylinders screen water prior to entering suction pipe headers which feed individual or multiple pumps. Traditionally, this screen type is used at a single suction pipe on the waterside of a river levee where it is lowered into place on an inclined pump. The pumps and motors are typically positioned on the landside of the river to pump over the top of the levee.

For this project, however, a series of dual cylindrical screens would be used to supply water to a common pumping plant on the landside of the levee to meet the high capacity requirements. A localized bank of screens would be arranged at select locations, as opposed to individual screens scattered abroad. Each pair of screens would share a common suction pipe, docking station, and retrieval guiderail system. The individual suction pipes from each pair of screens would be manifolded as necessary to supply water by gravity to a landside pumping plant. The conduits would be constructed in a similar fashion to that for the in-river and on-bank intake concepts, aligned beneath the river levee and coupled on the landside. The pumping plant and sediment management facilities would be identical to those planned for the other intake types. An illustration of the cylindrical intake concept proposed for this project is provided on Figure 5.

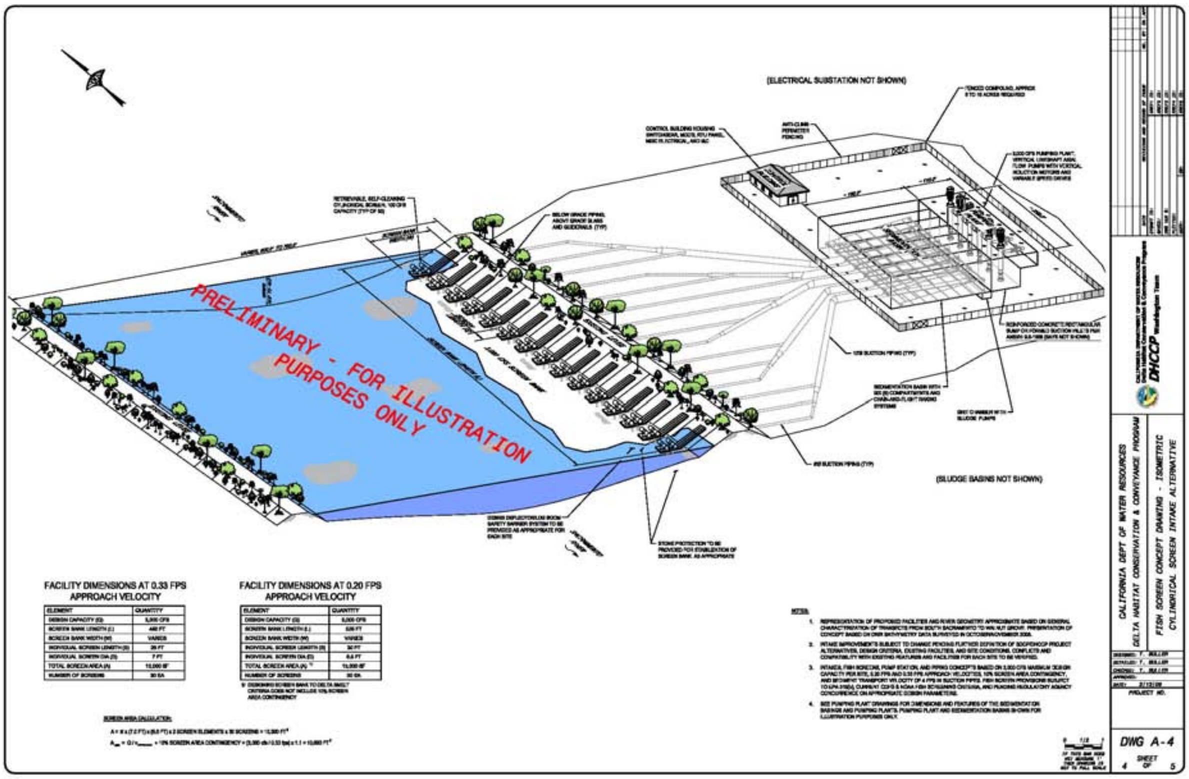


Figure 5. Cylindrical Intake Concept

2.1.4 Chevron Intake

The CCF diversion improvements mentioned above involve an altogether different screened intake concept given the location in the watershed. Since the point of diversion for the SWP and CVP export facilities is at a terminus in the Delta network, sufficient carriage flows are not available to sweep fish past a screened intake. This “dead end” condition requires an intake configuration with a means of escape from excessive exposure to the screen bank and subsequent impingement. The transport or sweeping velocity component is a fundamental requirement of an off-channel fish screen design. Accordingly, a chevron screen configuration is appropriate for the application, as was proposed in the original SDIP of 2001.

A chevron intake involves opposing screen banks within a common diversion structure. Like the in-river intake concept, the chevron screen configuration involves dual screen banks within a single intake thereby reducing structure magnitude and footprint for a given flow rate. The opposing screen banks converge in the downstream direction toward a central bypass channel or pipe inlet. As flow is “peeled off” of the diverted water column through the screens, the transport velocity in between the screen banks is maintained and gradually reduced as the bypass inlet is approached. This provides a constant downstream sweeping velocity within the structure to carry fish into a return channel or fish collection piping.

To suit a maximum diversion rate of 15,000 cfs, multiple chevron intake modules are required. The facilities are aligned in parallel and butted up to one another to provide separate diversion trains creating manageable compartments that can be individually isolated and accessed. An example rendering of the facility is provided on Figure 6 for reference.

A series of five side-by-side chevron intakes are proposed with an individual capacity of 3,000 cfs. The chevron intakes involve similar construction approaches and elements as the river intakes described above. In general, the chevron intake concept consists of the following:

- Foundation support system comprised of a cofferdam cell, pile support matrix, mat slab, and reinforced subgrade, as needed
- Reinforced concrete flume with access decking and structural steel support framing
- Converging banks of vertical plate, wedge-wire fish screen panels
- Adjustable baffle assemblies behind the screen panels
- Traveling brush cleaning mechanisms at each screen bank
- Bridge or gantry crane systems for screen and baffle removal/insertion
- Intermediate bypasses to prevent excessive fish exposure
- Collection or return piping/channel at downstream limit of fish screen banks
- Fish salvage facility (CHTR)

The various components above are assembled into a working intake system. Several illustrations are provided on Figure 7 to define a chevron screen, as well as the complete diversion system.

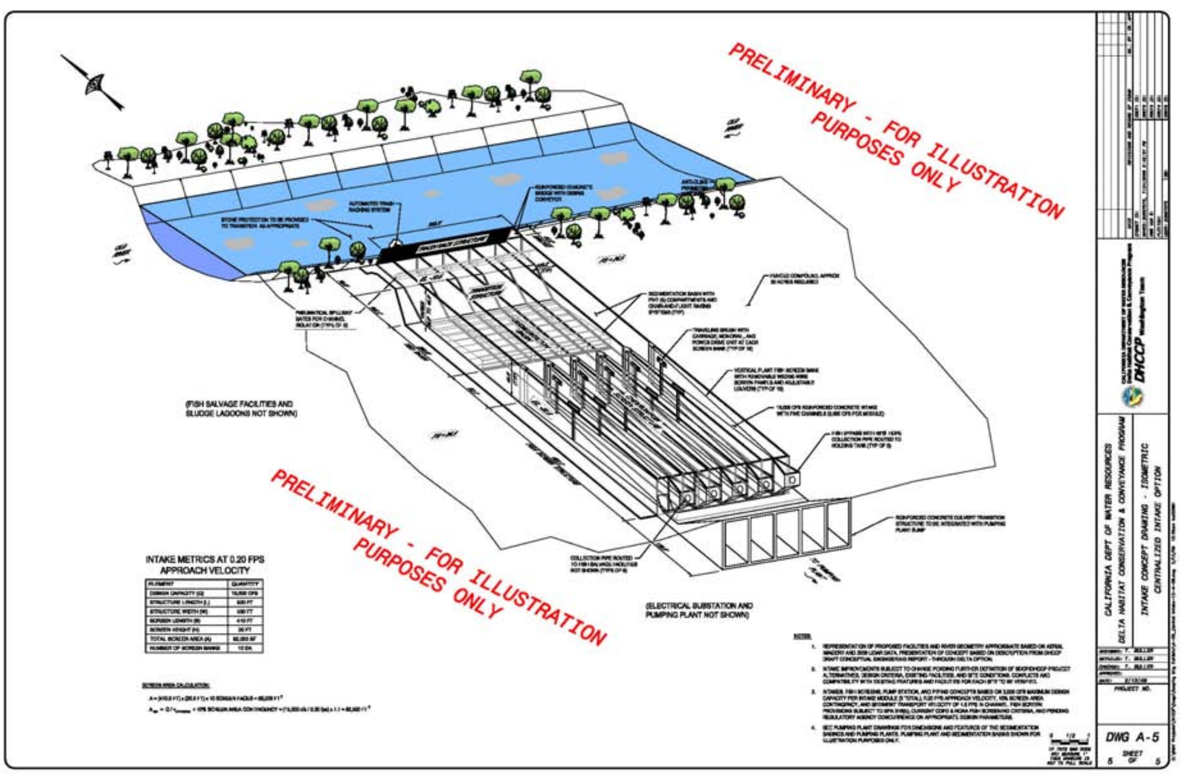


Figure 6. Example Chevron Screened Intake

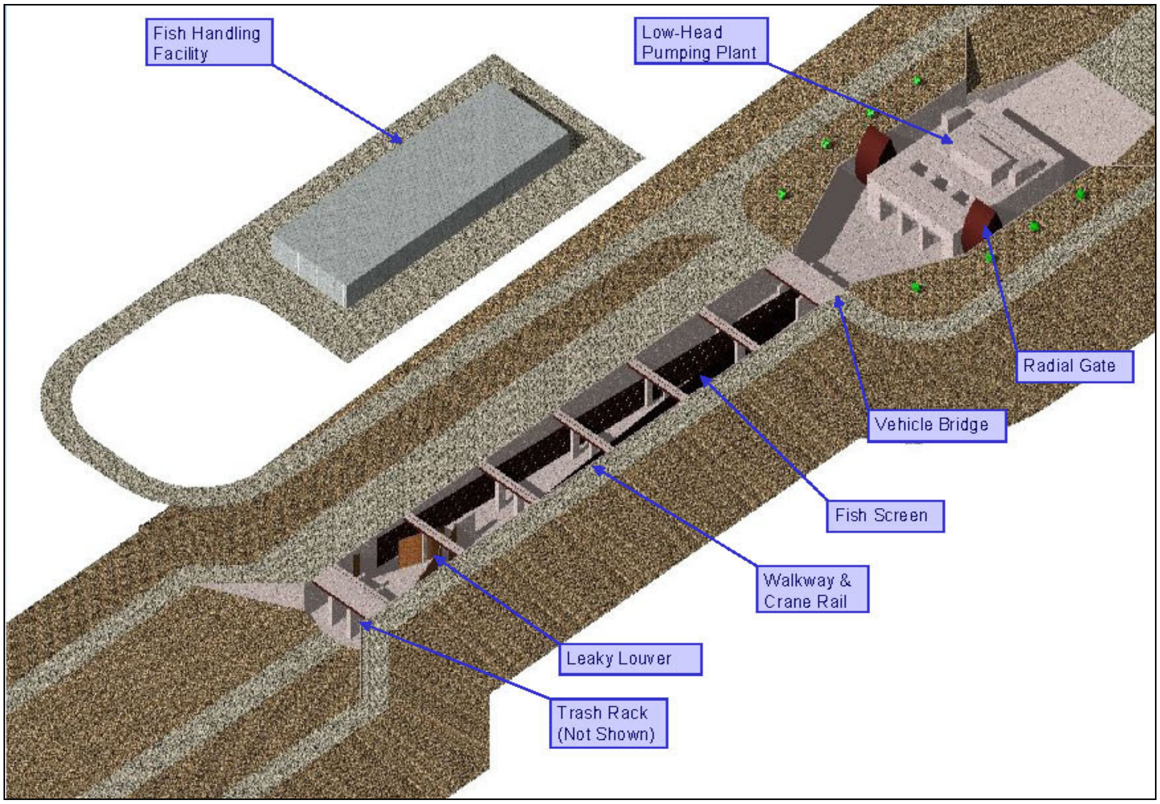


Figure 7. Example Chevron Screened Intake – Plan, Profile, and Section

Chevron Intake in the Dry

Chevron Intake in Service

2.1.5 General River Intake Technology Comparison

A general comparison has been made to contrast differences and illustrate points of similarity between the river intake technologies being considered for the project. Additionally, two example projects recently constructed or currently under construction on the Sacramento River have been included in the comparison for frame of reference. These two projects are the City of Sacramento's Sacramento River Intake (In-River Intake) near Richards Boulevard and East Bay

Municipal Utility District's Freeport Diversion (On-Bank Intake) near Freeport. Table 2 presents the various specifications and performance characteristics associated with the river intake technologies discussed above.

Table 2. Comparison of Intake Technologies

Criteria/Attribute	In-River Intake	On-Bank Intake	Cylindrical Intake	Sacramento Intake	Freeport Intake
Geometry (Five Intakes at 3,000 cfs Each)	270-ft Long x 30-ft Wide (salmonid) 380-ft Long x 30-ft Wide (smelt) Effective Screen Height: 25-ft	790-ft Long x 30-ft Wide (salmonid) 1,140-ft Long x 30-ft Wide (smelt) Effective Screen Height: 15-ft	450-ft Long x 30-Screens (salmonid) 525-ft Long x 30-Screens (smelt) Screen Dimensions: 8.5-ft Φ x 30-ft	210-ft Long x 34-ft Wide (salmonid and smelt) 250 cfs Capacity (In-River Intake Type) Effective Screen Height: 8-ft	610-ft Long x 80-ft Wide (salmonid and smelt) 290 cfs Capacity (On-Bank Intake Type) Effective Screen Height: 11.5-ft
Fish Exposure Time	3.3 minutes for Salmonid Screen 5 minutes for Smelt Screen (Assumes a Sweeping Velocity of 1.0 fps)	11.2 minutes for Salmonid Screen 16.7 minutes for Smelt Screen (Assumes a Sweeping Velocity of 1.0 fps)	6.5 minutes for Salmonid Screen 7.5 minutes for Smelt Screen (Assumes a Sweeping Velocity of 1.0 fps)	~2 minutes (Assumes a Sweeping Velocity of 1.0 fps)	~3 minutes (Assumes a Sweeping Velocity of 1.0 fps)
Impacts to Lands and Terrestrial Species (Acreage)	40 acres	40 acres	40 acres	8 acres	12 acres
Construction Schedule (Assume Same Resources)	2 Months – Sheet Pile Cofferdam 2 Months – Foundation Piles 2 Weeks – Excavation 6 Months – Tunneling/ Pipe Jacking (2 Wks setup/1 Wk Teardown per run) 8 Months – Concrete Structure 3-Months – I-Girder Bridge	5 Months – Sheet Pile Cofferdam 6 Months – Foundation Piles 6 Weeks – Excavation 15 Months – Tunneling/Pipe Jacking (2 Wks Setup/1 Wk Teardown per Run) 10 Months – Concrete Structure 0-Months – I-Girder Bridge	3 Months – Sheet Pile Cofferdam 3 Months – Foundation Piles 4 Weeks – Excavation 12 Months – Tunneling/Pipe Jacking (2 Wks Setup/1 Wk Teardown per Run) 16 Months – Concrete Structure 0-Months – I-Girder Bridge		
	Total Construction Period: 2½ Yrs	Total Construction Period: 3½ Yrs	Total Construction Period: 3 Yrs	Total Construction Period: 2½ Yrs	Total Construction Period: 3 Yrs

Table 2. Comparison of Intake Technologies

Criteria/Attribute	In-River Intake	On-Bank Intake	Cylindrical Intake	Sacramento Intake	Freeport Intake
In-Channel Permits	Rec Board/USACE/ CDFG 1600 (City of Sacramento Intake)	Rec Board/USACE/ CDFG 1600 (Freeport Diversion)	Rec Board/USACE/ CDFG 1600	Rec Board/USACE/ CDFG 1600	Rec Board/USACE/ CDFG 1600
Navigation	50-ft Wide Temporary Obstruction 30-ft Wide Permanent Obstruction	50-ft Temp Projection from Bankline 40-ft Permanent Projection from Bankline	50-ft Temp Projection from Bankline 20-ft Permanent Projection	35-ft Wide Permanent Obstruction	80-ft Permanent Projection from Bankline
Hydraulic/Flood Impacts (200-Yr Event) (Per CCHE2D Modeling with 50-ft Wide Cofferdam)	Not Modeled (TBD)	Not Modeled (TBD)	Not Modeled (TBD)	Unknown	Unknown
Approximate Distance from Bankline/Levee Crown	75-ft to 150-ft	30-ft to 50-ft (Assumes modification of channel bottom)	75-ft to 100-ft	300-ft	150-ft
Riverbank Impacts	~100-ft Disturbed for Salmonid and Smelt Screens due to Bridge Abutment	~900-ft for Salmonid Screen ~1,300 for Smelt Screen	~900-ft for Salmonid Screen ~1,300 for Smelt Screen	Unknown	Unknown
Set-Back Levee Required	Yes (~400-ft)	Yes (~1,200-ft)	Yes (~800-ft)	No	Yes
Reliability	Good	Good	Subject to Debris Impact and Substantial Damage	Good	Good
Maintenance	Good	Good	Most Maintenance Expected of all Technologies Compared	Sediment Problems	Good
Aesthetics and Visibility	380-ft x 30-ft Intake 50% Exposed Pump Station Bldg Exposed 30,000 sf x 75-ft high	1,140-ft x 30-ft Intake 50% Exposed Pump Station Bldg Exposed 30,000 sf x 75-ft high	No Intake Exposure Pump Station Bldg Exposed 30,000 sf x 75-ft high	210-ft x 50-ft Intake 50% Exposed Pump Station Bldg Exposed 10,000 sf x 50-ft high	610-ft x 80-ft Intake 50% Exposed Pump Station Bldg Exposed 20,000 sf x 50-ft high

3.0 CONCEPT DEVELOPMENT

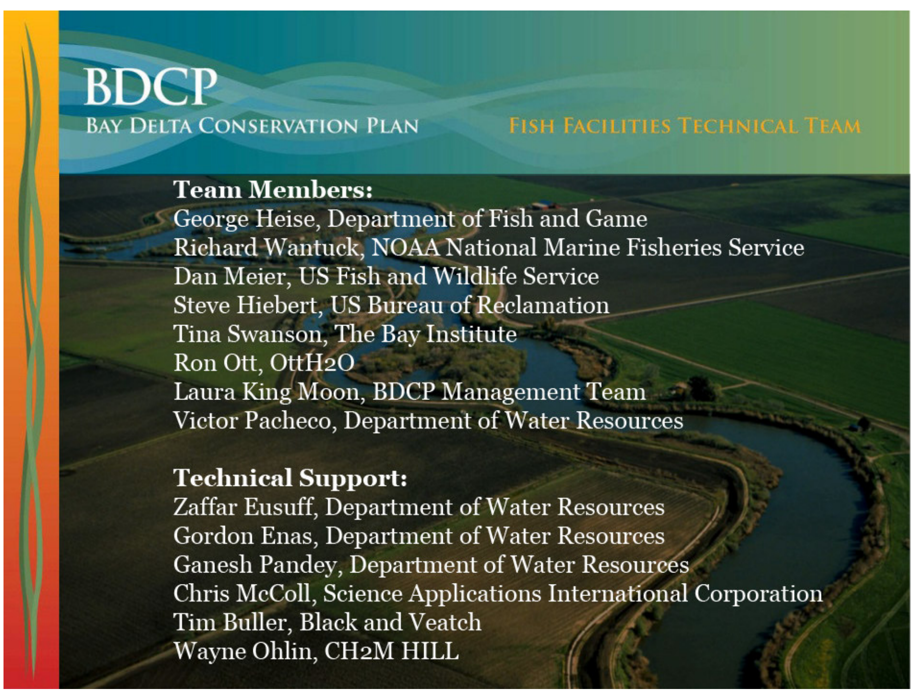
Three planning steps were involved in the shaping of screened intake concepts on the Sacramento River. These steps were:

1. FFTT conceptual intake development
2. Value Planning Study (VPS) for screened intakes on the Sacramento River
3. Integration of data and selection tools provided by the first two planning steps

Key considerations and outcomes associated with this planning process are explained below. Planning approaches relied on the best judgment of participants having expertise in the fields of fisheries and civil engineering, water resource planning, and conservation biology. As there are an infinite number of intake options and combinations with no absolutes, the processes and decisions explained below represent a subjective approach to determining reasonable solutions based on best available information and technical resources. The object of concept development, as it relates to intake selection, is to apply a series of step-logic procedures to converge on sound intake concepts which best meet program objectives.

3.1 BDCP FFTT Contributions

The FFTT was assembled under the BDCP to identify and explore fish-screened diversion concepts on the Sacramento River best suited to protect listed species from entrainment and impingement. Key participants making up the FFTT are identified below.



Fish Facility Technical Team Agency (FFTT) Participants

3.1.1 FFTT Intake Development Constraints

The focus of the FFTT has been to provide the BDCP Conveyance Workgroup with initial direction and recommendations regarding location, composition, and arrangement of fish protective diversion facilities. The planning activities conducted by the FFTT were subjected to the following key considerations and constraints.

- Water supply is provided for a maximum cumulative diversion rate of 15,000 cfs from the Sacramento River between the cities of Sacramento and Walnut Grove. Locating intakes further upstream is prohibitive due to substantial improvements and land use impacts, while further downstream is subject to greater tidal influence and smelt habitat encroachment.
- Intake concepts must prevent fish entrainment using proven best industry practices.
- The key project objective of conceptual intake development is to provide biologically protective and reliable technologies.
- Intakes must be sited relative to river hydraulics and fish distribution to minimize impacts to at-risk fisheries.
- A focus was placed on selecting and siting intakes without necessarily considering conveyance linkages and other factors outside the limits of the river section.
- Intakes must not produce adverse flood control impacts that cannot be remedied by common design features.

Development of screened intake concepts are subject to fish screen criteria prescribed by NOAA Fisheries and CDFG, as well as USFWS guidelines. These criteria are primarily based on fish swimming ability and behavior to prevent entrainment and impingement of selective fish species. Adopted fish screen design criteria are tailored primarily to protect salmonids. The FFTT intake options are designed to be protective of various State and federally listed fish species

Listed Fish Species of Consideration

A variety of considerations were taken into account by the FFTT to properly select and configure fish screen and intake technologies. Fish screening criteria and design considerations applied to the planning of diversions consist of the following key elements:

- River bathymetry and hydrology
- Intake placement within the river
- Temporal and spatial distribution of salmonid and smelt species
- Screen technologies, capacities, and locations
- Approach velocity and uniform distribution
- Sweeping velocity and exposure time
- Screen materials and cleaning mechanisms
- Opportunities to minimize predation
- Sediment management and O&M issues
- Isolatable and removable elements
- Flood control and navigation impacts

3.1.2 FFTT Proposed Intake Recommendations

According to the above factors, the FFTT identified three river intake technologies showing promise and the capability of meeting current screen criteria and best industry practices. These technologies are presented in Section 2.1. The array of transect locations selected by the FFTT for consideration of intake siting can be seen on Figure 1. Based on this technology and location information, the FFTT proposed four diversion schemes or concepts as presented in Table 3.

Table 3. FFTT Proposed Diversion Concepts

Diversion Concept	Facility Type/Location	Number and Capacity
A	Combined In-River (Dual) and On-Bank Intakes at Cross-Section Locations C (Freeport), F (Hood), and H (Courtland)	Three sites at 5,000 cfs each
B	Series of Cylindrical Screens at Locations from A (Sacramento) to L (Walnut Grove)	Ten sites with fifteen screens per site for a maximum of 1,500 cfs per site
C	Combined In-River (Dual) and On-Bank Intakes at Cross-Section Locations from A (Sacramento) to L (Walnut Grove)	Ten sites at 1,500 cfs each
D	Combined In-River (Dual) and Cylindrical Screens at Cross-Section Locations from A (Sacramento) to L (Walnut Grove)	Ten sites at 1,500 cfs each

Several key conclusions or recommendations were provided by the FFTT upon completing its evaluations. These items are as follows:

- A single intake facility is not considered reliable or protective of fish. Multiple intakes are preferable from both standpoints.

- Off-channel intakes are not advocated because their concentration of fish and single points of discharge increase predation opportunities.
- Intakes should be located as far north as possible to minimize encroachment on Delta smelt habitat. This also improves sweeping velocities at intakes as a result of muted tidal backwater effects.
- Intakes should be located within straight reaches of the river to avoid complex flow patterns, scour, and sediment issues associated with river bends.
- Positive exclusion barriers or fish screens should be provided as opposed to using experimental technologies, behavioral devices, and infiltration galleries for safely preventing entrainment.
- The best available fish screening technology that has been field proven on the Sacramento River should be used.
- Intakes should be designed with a maximum screen approach velocity of 0.33 fps (salmonids) as opposed to 0.2 fps (smelt). Designing screens to smelt guidelines requires a 60% increase in screen area contributing to greater fish exposure, cost, and river impacts. Given that smelt distribution and occupancy is expected to be short-lived in the reach of the Sacramento River being targeted, the application of a lower approach velocity was not necessarily considered appropriate by the FFTT for the project. The FFTT expressed a preference for operational adjustments rather than larger structures.
- The maximum diversion capacity should be limited to 5,000 cfs per diversion location. Multiple intakes with modules capable of diverting from 500 to 1,500 cfs per screen face should be used.
- The length of screens should be minimized to reduce the duration of fish exposure to the screen surface.
- All baffles, bulkheads, and screen panels should be removable from the top surface of the intake structure. Intake compartments should be capable of being isolated for dewatering and desiccation of mussels in the future.
- Sediment management provisions should be provided on the landside of the levee to minimize in-river infrastructure.
- Existing riparian habitat should be avoided.

The FFTT study provided general direction as to biologically protective and reliable diversion facilities. The FFTT also identified additional information needs necessary to properly locate points of diversion and further develop the team's recommendations. Additional fish seining/trapping studies, execution of multi-beam hydrographic surveys, modeling of river hydraulics using finite element computer techniques, identification of operating protocols, and scaled physical model studies were identified by the FFTT as essential for further refinement of river diversion concepts.

3.2 Value Planning Study Contributions

Subsequent to the FFTT study, DWR conducted a value planning study (VPS) to further evaluate potential intake schemes considering factors beyond the limits of the river boundaries. The value planning team was made up of independent participants spanning a broad cross-section of disciplines. However, three members of the FFTT were included to maintain continuity and to assist with appropriate knowledge transfer. The list of team participants is provided below.

Staff	Expertise	Provided by
Management Team		
Mark Benson	Deputy Program Manager	Washington Division
Jobaid Kabir	Task Manager	URS
Instructors		
John Robinson	Value Engineering	Strategic Value Solutions
Dawn-Marie Bennett	Value Engineering	Strategic Value Solutions
Value Engineering Team		
Gordon Enas	Engineering	DWR
Mike Gillespie	Constructability	Washington Division
Steve Hiebert	Environmental	Reclamation
Edward E. Donahue	Screens & Passage	HDR
Mike Melanson	Environmental	MWD
Bob Zylman	Estimation	Washington Division
Courtney Lyerly	Estimation	URS
Rahim Nasserziayee	Intake Hydraulics	Washington Division
Ron Ott	Fish Screens	HDR
Wilbur Huang	Hydraulics	URS
Ben Tiedt	Safety and Security	DWR
Al Davis	Right of Way	DWR
John Balance	Power	B&V (sub consultant)
John Chueh	Pumps	B&V
Tracey Hinajosa	Operations and Maintenance	DWR
Suzenne Tang	Observer	Washington Division
Marianne Kirkland	Engineering	DES Fish Facility

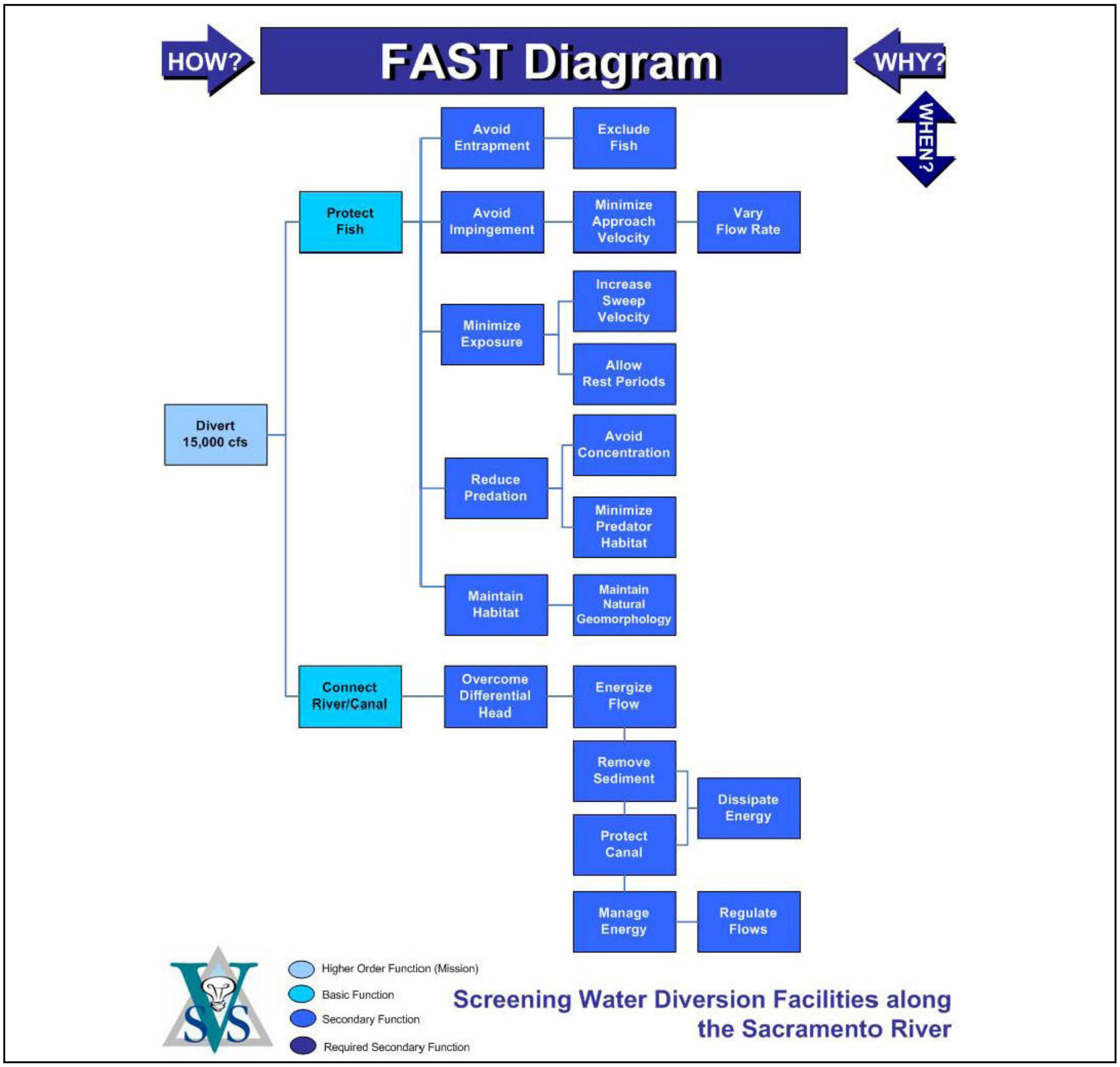
Value Planning Study Participants

The purpose of the VPS was to apply FFTT recommendations toward the development and comparison of a broad range of potential river intake concepts, as well as conveyance arrangements. The study objectives were as follows:

1. To formulate and analyze reasonable intake arrangements considering conveyance options and other constraints beyond the limits of the river cross-section.
2. To solicit independent, qualitative input from industry experts on appropriate intake arrangements for isolated conveyance options.
3. To develop a planning based methodology with decision-support tools for identifying reasonable intake solutions on the Sacramento River that are capable of protecting at-risk fish species while at the same time reliably diverting up to 15,000 cfs from the river.
4. To recommend key intake and conveyance features from various perspectives and disciplines for integration into each conveyance scheme.

The VPS consisted of a one-week workshop in October 2008 in which a panel of consultants and DWR staff convened to perform a cursory evaluation of viable diversion schemes for diverting water from the Sacramento River and delivering it to a common conveyance canal. A sequence of discrete tasks was undertaken in the study process. Prior to meeting, the team conducted a review of the FFTT study, conveyance option materials, and other associated design-related documents to gain an understanding of the program and background.

At the onset of the workshop, the team was presented with general background information, a detailed presentation of the FFTT study, and the goals of the VPS. A half-day tour of the Sacramento River was conducted thereafter to provide the team with perspective on existing conditions and constraints for intake siting. With a general understanding of program requirements under its belt, the VPT developed a function analysis to define key functions and objectives for the subsequent tasks of the VPS. The diagram below illustrates the function analysis tool developed by the team.



Function Analysis Diagram

Based on the above, it was possible for the team to formulate diversion schemes considering FFTT recommendations, key performance objectives, and supplemental conveyance considerations. A list of roughly 40 intake concepts was developed for the east and west conveyance routes, with varying capacities, locations, and technologies. Ultimately, 23 options were advanced for comparison reciprocal to both east and west conveyance alignments along with an additional 8 options specific to the western conveyance alignment only.

To provide a uniform basis by which to compare diversion concepts, the team identified a series of performance factors. In summary, eight performance factors were applied. These factors are condensed as follows:

1. Operational flexibility
2. Maintainability
3. Constructability/construction ease
4. Fish protection/fish benefits
5. Landowner and community impacts
6. River impacts
7. Safety
8. Security

With the options and factors defined, a weighted decision matrix was prepared to compare options. An evaluation matrix was used as a practical decision-support tool that applies a quantitative approach to ranking options based on performance factors. The process was used to compare options on a holistic level by evaluating, rating, and combining cumulative values. There are four basic steps to the matrix as it applies to this study:

1. Performance factors are defined and weighted based on their relative importance on a scale from 1 to 10. Note that weighting of factors was not performed by the VPT, but was deferred to DWR management and stakeholders.
2. Each option is evaluated by how well it addresses each performance factor on a scale from 1 (worst) to 10 (best).
3. Each option is scored for each performance factor, and the weighted score is the product of the weighting and scoring.
4. The weighted scores are summed for each alternative, and the best score is normalized to 100 with all others prorated accordingly for convenient ranking.

This method of comparison provides an opportunity to objectively assess how well each alternative performs and which features are best aligned with project objectives. A select subgroup was assembled from the team comprised of Agency staff from the Division of Environmental Services, Division of Engineering, and the O&M Division to apply values within the matrix. Assignment of relative values was made “blind,” meaning the subgroup assigned an initial value to each performance factor for each option based on its assessment of performance and best judgment. The subgroup, however, did not assign weighting to the performance factors. Rather this step was deferred to Lead Agencies and stakeholders to obtain a better collective assessment of the importance of individual factors.

To supplement the matrix, quantity take-offs and order of magnitude cost estimates were prepared. Project cost is a basic element of option comparison, and the value of cost versus benefit is subject to the opinions of those responsible for operating, maintaining, and financing the project. Therefore, cost was not scored as a factor within the evaluation matrix; rather, values were provided for informational purposes. Estimates were based on team members' prior involvement developing opinions of cost for similar conveyance evaluations.

The VPS did not identify the best performing option(s), but it did provide a tool by which to compare and rank the options against one another. Although identification of a preferred solution was not the object of the study, key findings included:

1. Time-of-use vs. continuous pumping provides no cost benefit
2. On-river pumping plants provide the greatest operational flexibility
3. On-river pumping plants reduce conveyance requirements
4. Forebays are not required with on-river pumping
5. Sediment management provisions at the intake are preferable
6. A single intake per site is more constructible and poses less builder's risk
7. Operational relationship of diversion flow to river flow is 35% (assumed)
8. Diversions are best limited to 3,000 cfs due to proven track record

The focus of the VPS was to formulate and analyze potential solutions according to factors not necessarily considered by the FFTT such as operational flexibility, maintainability, community impacts, conveyance requirements, economics, etc.

3.3 Integration Meeting Contributions

The goal of planning efforts up to this point had been to establish a rationale for identifying reasonable diversion solutions for each of the conveyance alignment options. The information produced by the FFTT and VPT sets the stage for developing and selecting reasonable and defensible intake solutions consistent with conveyance options. Although subsequent hydraulic and physical modeling and other studies are required to further refine intake concepts, the evolution in the decision making process serves as the basis for defining intake facilities for the EIR/EIS.

A meeting was conducted on December 3, 2008, to present the above planning activities and to provide for interactive use of the evaluation matrix. Additionally, a pros versus cons comparison of FFTT recommended intake technologies was conducted by meeting participants. A representation of this comparison is provided in Table 4.

Table 4: Comparison of Intake Technologies

Intake Technology	Benefits	Liabilities
Cylindrical Screens	Low impacts to river hydraulics/fish Screens are retrievable Greatest redundancy of screen types Less prone to sedimentation	Sole-source manufacturer Low capacity/many screens required Screen banks involve large footprint High perceived maintenance/lots of moving parts High exposure to impact damage Less reliability No history of similar applications Produces structure in river for predator holding
In-River Intakes	Smallest structure footprint Limited fish exposure Standard flat plate screen technology Retrievable/isolatable elements Commonly available screen elements/ many manufacturers	Greatest river encroachment Greatest navigation impacts Greatest potential for hydraulic impacts Visual impacts
On-Bank Intakes	Less exposure to impact Least river encroachment Standard flat plate screen technology Retrievable/isolatable elements Commonly available screen elements/ many manufacturers Only technology with field history up to 3,000 cfs Low hydraulic/navigation impacts	Large structure footprint/long structure Requires set-back levees for construction Greatest fish exposure

An interactive gaming session was subsequently undertaken using the evaluation matrix produced by the VPS. An iterative process was conducted in which the least scoring options for primary performance factors were dismissed. An example of the evaluation matrix at the beginning of the process is presented on Figure 8. Table 5 presents the sequence of trials conducted and resultant actions taken.

SCREENED INTAKE DIVERSIONS - VALUE PLANNING STUDY
EAST SIDE CONVEYANCE EVALUATION MATRIX

PERFORMANCE FACTOR		Operational Flexibility	Maintain-ability	Constructability/ Construction Ease	Fish Protection/ Fish Benefits	Impact on Landowners & Communities	River Impacts	Safety	Security	Normalized Weighted Scores	Rank	Capital Cost [2/]
	Value/Weight	10	10	3	10	7	8	1	1			
Alt ID	ALTERNATIVE DESCRIPTION[1/]											
DE-01	Five 3,000 cfs Single On-Bank Intakes (C, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	5	5	6	5	5	9	9	7	90.1	6	"Base"
DE-01A	Five 3,000 cfs Combined On-Bank and In-River Intakes (C, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	6	2	5	8	5	4	5	3	77.4	17	95%
DE-02	Six 2,500 cfs Single On-Bank Intakes (C, D, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	6	5	6	6	5	9	9	6	96.0	3	100%
DE-02A	Six 2,500 cfs Combined On-Bank/Cylindrical Intakes (C, D, E, F, G, H) with combined discharge to Canal at Clarksburg and Hood	7	6	8	5	5	8	5	2	96.0	4	94%
DE-02B	Six 2,500 cfs Combined On-Bank/In-River/Cylindrical Intakes (C, D, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	8	4	8	7	5	5	4	2	91.3	5	93%
DE-03	Three 5,000 cfs Combined In-River/On-Bank Intakes (F, G, H) with Combined Discharge to Canal at Hood	4	5	4	5	7	2	7	9	72.1	22	79%

Figure 8. Intake Option Evaluation Matrix

PERFORMANCE FACTOR		Operational Flexibility	Maintain-ability	Constructability/ Construction Ease	Fish Protection/ Fish Benefits	Impact on Landowners & Communities	River Impacts	Safety	Security	Normalized Weighted Scores	Rank	Capital Cost [2/]
Value/Weight		10	10	3	10	7	8	1	1			
Alt ID	ALTERNATIVE DESCRIPTION[1/]											
DE-04	Five Intakes from 2,000 cfs each to 5,000 cfs each (F, G, H, I, J) with a Combination of all Three Screen Types and Combined Discharge to Canal East of Clarksburg	7	5	5	7	5	4	4	2	86.1	9	106%
DE-05	Five 3,000 cfs Combined On-Bank, In-River, and Cylindrical Intakes (A, E, F, G, H) with Pipeline Under River from A and Discharge to Canal at Clarksburg and Hood, Canal Parallel to Sac River	9	4	5	6	4	5	4	2	86.4	8	110%
DE-06	Five 3,000 cfs Combined On-Bank/In-River Intakes (D, E, E1, F, G) with Discharge to Canal at Clarksburg and Hood, Canal Parallel to Sac River	5	4	4	7	6	4	7	7	80.5	14	81%
DE-07	Ten 1,500 cfs On-Bank/Cylindrical Intakes (E, E1, E2, E3, F, G, H, I, J, K) with Pipeline Corridor Parallel to Sac River	9	5	8	6	4	8	5	1	99.7	2	99%
DE-07A	Ten 1,500 cfs On-Bank/Cylindrical Intakes (C, D, E, E1, E2, E3, F, G, G1, H) with Pipeline Corridor Parallel to Sac River	9	5	8	6	4	8	5	2	100.0	1	102%
DE-08	Three 5,000 cfs Combined In-River and On-Bank Intakes (C, E, F) with Pipeline Corridor Parallel to Sac River	5	5	4	6	6	4	7	9	81.1	12	89%
DE-09	Five 3,000 cfs In-River and Combined On-Bank/Cylindrical Intakes (F, G, H, I, J) and Combined Discharge East of Clarksburg	6	4	5	6	4	5	4	3	77.4	17	104%

Figure 8 (continued)

PERFORMANCE FACTOR		Operational Flexibility	Maintain-ability	Constructability/ Construction Ease	Fish Protection/ Fish Benefits	Impact on Landowners & Communities	River Impacts	Safety	Security	Normalized Weighted Scores	Rank	Capital Cost [2/]
Value/Weight		10	10	3	10	7	8	1	1			
Alt ID	ALTERNATIVE DESCRIPTION[1/]											
DE-09A	Five 3,000 cfs In-River and Combined On-Bank/Cylindrical Intakes (F, G, H, I, J) and Combined Discharge East of Clarksburg	6	5	5	5	4	6	4	3	79.9	15	104%
DE-10	Four 3,750 cfs Combined In-River/On-Bank Intakes (E, E1, E2, F) with Pipeline Corridor Parallel to Sac River	5	6	4	6	6	4	7	8	83.9	11	74%
DE-11	Three 5,000 cfs Combined In-River/Cylindrical Intakes (C, E, F) with Combined Discharge at Hood	5	7	5	4	7	4	3	2	80.8	13	73%
DE-13	Three 5,000 cfs Combined In-River/On-Bank/Cylindrical Intakes (I, J, K) with Combined Discharge at Clarksburg	3	5	5	4	7	3	4	3	66.6	23	81%
DE-14	Four 3,750 cfs Combined In-River/On-Bank Intakes (D, E, F, G) with Pipeline Corridor Parallel to Sac River	5	6	4	7	6	5	6	7	88.9	7	79%
DE-17	Three 5,000 cfs Combined In-River/On-Bank/Cylindrical Intakes (F, G, H) with Combined Discharge at Clarksburg	4	6	5	4	7	4	4	3	75.2	20	77%
DE-21	Six 2,500 cfs In-River Intakes (D, E, E1, E2, F, G) with Pipeline Corridor Parallel to Sac River	5	2	3	9	6	3	6	5	76.2	19	72%

Figure 8 (continued)

PERFORMANCE FACTOR		Operational Flexibility	Maintain-ability	Constructability/ Construction Ease	Fish Protection/ Fish Benefits	Impact on Landowners & Communities	River Impacts	Safety	Security	Normalized Weighted Scores	Rank	Capital Cost [2/]
Value/Weight		10	10	3	10	7	8	1	1			
Alt ID	ALTERNATIVE DESCRIPTION[1/]											
DE-22	Ten 1,500 cfs On-Bank and Cylindrical Intakes (A thru J) with Pipeline Parallel to Sac River	10	2	5	5	1	5	5	2	74.0	21	134%
DE-23	Three 5,000 cfs Dual In-River and In-River/On-Bank Intakes (C, F, H) with Pipeline Parallel to Sac River (A and B with Pipelines on West Side with Undercrossing)	7	4	4	7	6	3	7	9	84.8	10	87%
DE-24	Three 5,000 cfs On-Bank Intakes with Oxbow Cut-Off Channels and Pipeline Parallel to Sac River	3	8	7	5		7	9	7	78.3	16	90%

1/ Based on description of options from Value Planning Study dated December 2008.

2/ Costs are provided at a relative scale referencing the base cost for the first option shown.

Figure 8 (continued)

Table 5. Evaluation Matrix Gaming Session – Sequence of Events

Iteration	Description of Actions	Resulting Outcomes
1	Fish protection factor assigned a weighting of 10 and all other factors given 0 weighting	Options DE-11, DE-13, and DE-17 dismissed
2	Scoring for options involving three 5,000 cfs intakes reduced to two for operational flexibility factor and weighting of 10 assigned with all other factors given 0 weighting	Options DE-3, DE-8, DE-23, and DE-24 dismissed
3	Scoring for options involving ten 1,500 cfs intakes and cylindrical and combined intake technologies reduced to two for maintainability factor and weighting of 10 assigned with all other factors given 0 weighting	Options DE-1A, DE-2B, DE-4, DE-7, DE-7A, DE-21, and DE-22 dismissed
4	Landowner/community impacts factor assigned a weighting of 10 and all other factors given 0 weighting	Options DE-5, DE-9, and DE-9A dismissed
5	River impacts factor assigned a weighting of 10 and all other factors given 0 weighting	Option DE-23 dismissed
6	Final weighting of factors on a cumulative 100 scale as follows: Operational flexibility – 25 Maintainability – 25 Constructability – 10 Fish protection – 25 Landowner/community impacts – 10 River impacts – 10 Safety – 0 Security – 0	Remaining options ranked as follows: DE-2A DE-2 DE-14 DE-1 DE-10 DE-6

A similar process was undertaken for the eight options developed specific to the west conveyance alignment with two options surviving the process, DW-1 and DW-2. In summary, the remaining options consist of four to six intakes involving either on-bank or in-river intake technologies. The east conveyance alignment favors intakes from Freeport to Courtland whereas the west conveyance alignment favors intakes from Sacramento to Hood.

A summary matrix and comparison chart are provided on Figures 9 and 10 to illustrate how the screened options compare against one another per the above process.

SCREENED INTAKE DIVERSIONS - VALUE PLANNING STUDY
EAST AND WEST ISOLATED CONVEYANCE FACILITIES EVALUATION MATRIX

	PERFORMANCE FACTOR	Operational Flexibility	Maintainability	Constructability & Construction Ease	Fish Protection/ Fish Benefits	Impact on Landowners & Communities	Impacts on Species and Habitat	River Impacts	Acceptability/ Permitability	Safety	Security	Normalized Weighted Scores	Rank	Capital Cost [2/]
	Value/Weight	25	25	5	25	10	10	10	10	0	0			
Alt ID	OPTION DESCRIPTION[1/]													
DW-1	Five 3,000 cfs On-Bank Intakes (A, B, C, D, E) and gravity flow to canal	6	5	6	7	6	0	9	0	9	8	100.0	1	"Base"
DE-02A	Six 2,500 cfs Combined On-Bank/Cylindrical Intakes (C, D, E, F, G, H) with combined discharge to Canal at Clarksburg and Hood	7	6	8	5	5	0	8	0	5	2	98.4	2	101%
DW-2	Four 3,750 cfs On-Bank Intakes (A, B, D, E) and gravity flow to canal	5	7	6	6	6	0	8	0	9	9	98.4	2	101%
DE-02	Six 2,500 cfs Single On-Bank Intakes (C, D, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	6	5	6	6	5	0	9	0	9	6	94.4	4	107%
DE-14	Four 3,750 cfs Combined In-River/On-Bank Intakes (D, E, F, G) with Pipeline Corridor Parallel to Sac River	5	6	4	7	6	0	5	0	6	7	92.1	5	84%
DE-01	Five 3,000 cfs Single On-Bank Intakes (C, E, F, G, H) with Combined Discharge to Canal at Clarksburg and Hood	5	5	6	5	5	0	9	0	9	7	86.5	6	107%
DE-10	Four 3,750 cfs Combined In-River/On-Bank Intakes (E, E1, E2, F) with Pipeline Corridor Parallel to Sac River	5	6	4	6	6	0	4	0	7	8	86.5	6	79%
DE-06	Five 3,000 cfs Combined On-Bank/In-River Intakes (D, E, E1, F, G) with Discharge to Canal at Clarksburg and Hood, Canal Parallel to Sac River	5	4	4	7	6	0	4	0	7	7	82.5	8	87%

1/ Based on description of options from Value Planning Study dated December 2008.

2/ Costs are provided at a relative scale referencing the base cost for the first option shown.

Figure 9. Summary Evaluation Matrix

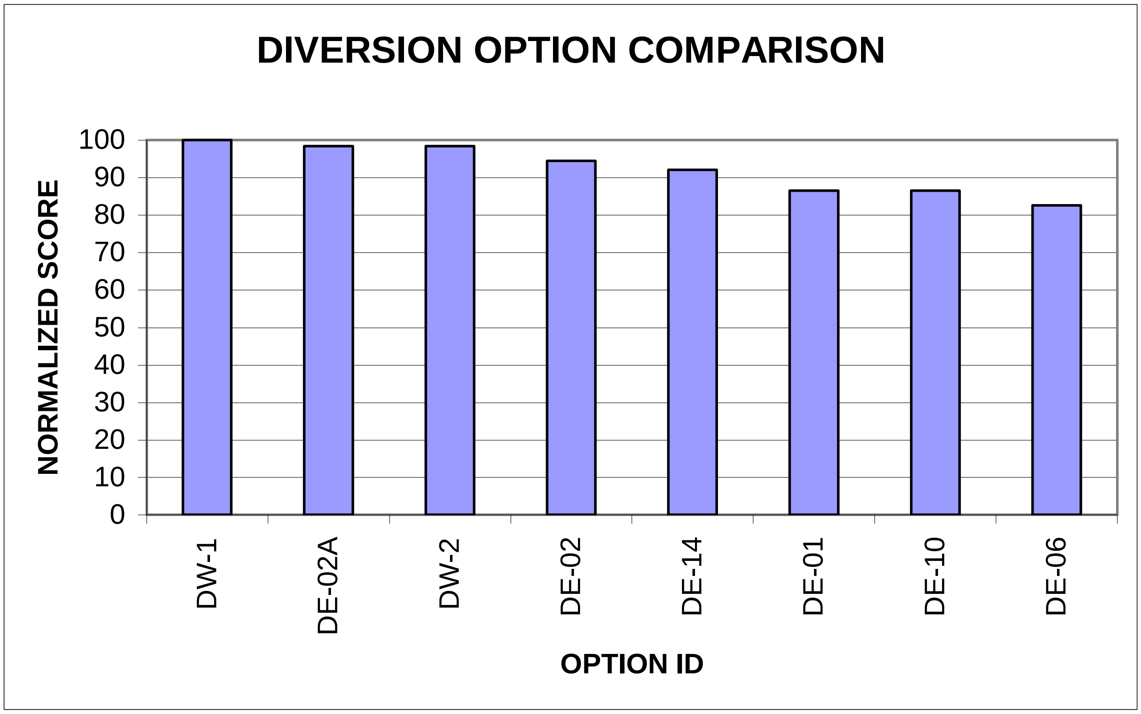


Figure 10. Option Comparison Graphic

In addition to the gaming session described above, a subsequent meeting was conducted on January 21, 2009, in which available geographical information system (GIS) datasets were utilized to refine locations of intake sites according to various environmental and land impact factors. A collaborative process was administered to adjust intake sites in an attempt to minimize impacts. A list of GIS datasets used in the selection process are as follows:

1. Property boundaries/parcel lines
2. Rare species habitat zones
3. Points of diversion on the Sacramento River
4. Land use
5. Wetland delineation
6. River cross-sections
7. USFWS fish trapping data
8. Ground level surveillance

The process resulted in adjusting physical locations of intake sites between Sacramento and Walnut Grove from that identified in the FFTT study, including the elimination of one particular site due to prohibitive existing features and conditions. Figure 11 shows the preliminary intake locations resulting from this process.

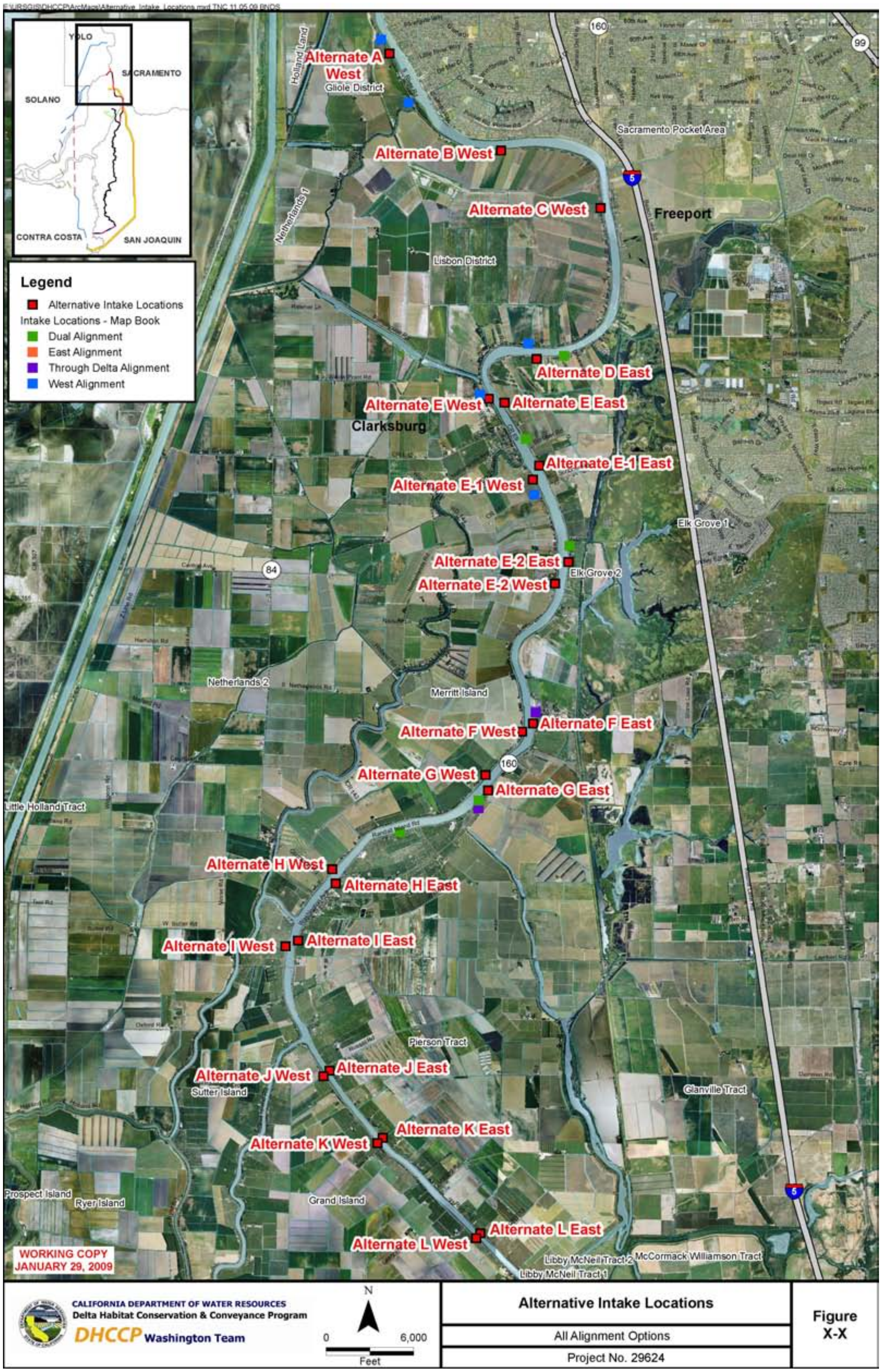


Figure 11. GIS-Based Site Refinement

3.4 Further Evaluation – Intake Siting and Sizing

To facilitate selection of diversion schemes for isolated conveyance facilities, intake technology was extracted from the equation and evaluated independently. The method used to identify the best intake technology to use for conceptual engineering described in Section 3.5. To determine the appropriate size (capacity) and location of intakes, an approach was used in the manner explained below.

With a total of 31 intake options (23 common to the east and west conveyance options and 8 exclusive to the west) formulated by the VPT for comparison, a more rigorous evaluation was needed. In the VPS, intake options were assigned values for a range of performance factors based primarily on professional judgment. The study did not weight the relative importance of individual performance factors. Therefore, ranking of options was not provided in the VPS evaluation matrix (Figure 8).

However, the evaluation matrix was used in an interactive gaming session to develop a better understanding of option performance. Nonetheless, this analysis (provided in Table 5 and on Figures 9 and 10) did not include sufficient consideration of factors influencing the practicality and environmental effects nor did it identify a reasonable set of options that could be carried forward in preliminary engineering design and the EIR/EIS analysis. The gaming session did result in the identification of design and environmental factors that could be used to logically screen options. Those factors are listed in **bold type** below.

Diversions should be limited to 3,000 cfs. Based on the information developed by the FFTT and VPS, it was decided that the maximum intake size should be limited to 3,000 cfs. No other intakes larger than this have been constructed in California, and there are few intakes larger than 3,000 cfs in the western United States. An intake of 3,000 cfs is the largest diversion known to exist on the Sacramento River meeting Pacific Northwest screening criteria. Larger intakes could result in undesirable performance, substantial local impacts on river hydrology, and greater construction impacts. Intakes less than 3,000 cfs would result in greater cumulative land use impacts, and it is uncertain whether the smaller intakes would improve fish protection.

Each 3,000 cfs intake, including landside pumping facilities, would disturb roughly 40 acres inland from the river during construction. Onshore facilities at each site, such as the pumping plant, sediment basin, and electrical substation, would result in a permanent facility footprint of roughly 20 acres. Smaller intakes on the order of 1,500 cfs would require a smaller area for construction and operation, but this reduction in land use would not be linear. A 1,500 cfs intake would result in roughly 30% less temporary and permanent land disturbance than a 3,000 cfs intake. However, to meet the project water delivery goal of 15,000 cfs, twice as many 1,500 cfs intakes would be required, resulting in greater total land use/terrestrial species habitat impacts than 3,000 cfs intakes. 1,500 cfs intakes would also require more canals or pipelines to convey water from the intakes to the conveyance facility. This would again increase land use and terrestrial species habitat impacts.

The use of 1,500 cfs intakes instead of 3,000 cfs intakes would produce less fish exposure at a particular intake site. However, fish exposure would be the same regardless of intake size because the total screen surface area would be equivalent for either option. Therefore, there is no certainty that fish protection would be improved with intakes smaller that 3,000 cfs.

Omit options exclusively involving cylindrical screen technology. As discussed in Section 2.1.3, the individual capacity of cylindrical screens is 100 cfs. Assuming the screens are ganged in sets of two, a total of 75 coupled units would need to be deployed to deliver a maximum of 15,000 cfs. At 3,000 cfs per intake, 15 pairs of cylindrical screens are required. Each screen module would include abovegrade guide rails and a hydraulic system for screen cleaning and removing the screens for maintenance and repair. This is a substantial increase in moving parts relative to an in-river or on-bank intake, resulting in higher operating and maintenance costs.

Cylindrical screens are more vulnerable to debris damage than in-river or on-bank screens because of their exposure in the water column. They are typically used for agricultural diversions, i.e. deployed during the summer and removed during the winter when debris load is greatest. With the proposed project, the screens would be deployed throughout the winter and subject to significant damage by large debris such as trees traveling down the river. Debris barriers could be constructed upstream of the screens to reduce this problem, but deflectors present their own set of problems. Substantial debris typically accumulates at the barriers, is difficult if not impractical to remove, can cause navigation hazards, potentially increases the local flood hazard, and increases risk of damage to the levee.

Use a single screening technology. A single intake type instead of multiple technologies is preferable from an O&M standpoint.

Eliminate intake options at the southern end of the study reach. Options involving intake locations exclusively in the southern region of the river reach under consideration should be eliminated due to greater tidal influence, higher probability of Delta smelt abundance, and potential for producing reverse flows in Sutter and Steamboat Sloughs.

The above factors were applied to the VPS intake options in several iterations. The results of this analysis are provided in Table 6.

Table 6. Option Matrix – Dataset Reduction

Iteration	Description of Actions	Resulting Outcomes
1	Remove all options comprised of intakes greater than 3,000 cfs	Options DE-3, DE-8, DE-10, DE-11, DE-13, DE-14, DE-17, DE-23, DE-24, DW-2, DW-3, DW-4, DW-10, and DW-15 dismissed; DE-4 modified to consist of five 3,000 cfs intakes
2	Eliminate options only utilizing cylindrical screen technology	Option DW-9 dismissed
3	Modify options to eliminate reference to intake technology	All option descriptions modified
4	Remove options that become duplicative as a result of Step No. 3	Options DE-1A, DE-2A, DE-2B, and DE-9 dismissed
5	Remove options involving 10 intakes	DE-7, DE-7A, and DE-22 dismissed
6	Remove options involving 6 intakes	DE-2 and DE-21 dismissed
7	Remove options involving intakes exclusively in southern reach of river	DW-11 dismissed
8	Options remaining after conducting iterations 1 thru 7	DE-1, DE-4, DE-5, DE-6, and DW-1 remain

A summary matrix is provided on Figure 12 identifying the remaining options for further comparison.

Alt ID	PERFORMANCE FACTOR / OPTION DESCRIPTION [1/]	Fish Protection	Impact on Landowners & Communities	Terrestrial Species Impacts	Raw Sum of Scores	Normalized Scores	Rank	Capital Cost [2/]
DE-01	Five 3,000 cfs Intakes (C, E, F, G, H) with Discharge Pipelines to Canal Beginning East of Clarksburg and at Hood	1	1	1	3	100.0	1	"Base"
DE-04	Five 3,000 cfs Intakes (F, G, H, I, J) with Combined Pipeline Discharge to Canal East of Courtland	1	1	1	3	100.0	1	106%
DE-05	Five 3,000 cfs Intakes (A, E, F, G, H) with Pipeline Under River from A and Discharge to Canal Beginning East of Clarksburg and at Hood	1	1	1	3	100.0	1	110%
DE-06	Five 3,000 cfs Intakes (D, E, E1, F, G) with Discharge to Canal Beginning East of Clarksburg and at Hood	1	1	1	3	100.0	1	81%
DW-1	Five 3,000 cfs Intakes (A, B, C, D, E) with Common Pipeline Discharge to Canal Beginning West of Freeport	1	1	1	3	100.0	1	94%

1/ Based on description of options from Value Planning Study dated December 2008.
2/ Costs are provided at a relative scale referencing the base cost for the first option shown.

Figure 12. Summary Evaluation Matrix

The performance factors or criteria developed for the VPS were revisited to determine if they could be further clarified for use in evaluating the five remaining intake options. The criteria for safety and security developed for the VPS were eliminated from further consideration. Standard engineering practices require safe and secure designs regardless of the intake option being considered. Therefore, intake options could not be differentiated based on these performance criteria.

The performance factors or criteria developed for the VPS were supplemented and organized into a flowchart (Figure 13) that relates to this fundamental purpose and further breaks down the criteria into measures consistent with the following decision analysis guidelines:

- Evaluation criteria must represent measures of the project objectives. By explicitly connecting evaluation criteria to the underlying fundamental project objectives, the evaluation criteria are not simply the means to get to the project objectives, but truly reflect the achievement of fundamental objectives themselves.

- Ideally, a natural scale should be defined to measure the impact on each evaluation criterion.
- Evaluation criteria should not overlap; that is, there should not be double-counting of impacts among different evaluation criteria.
- Exclude any evaluation criteria that do not vary significantly among the options being considered, even if they reflect some important project objectives. Such criteria do not help in distinguishing among options.

The following flowchart was developed to reflect the project objectives.

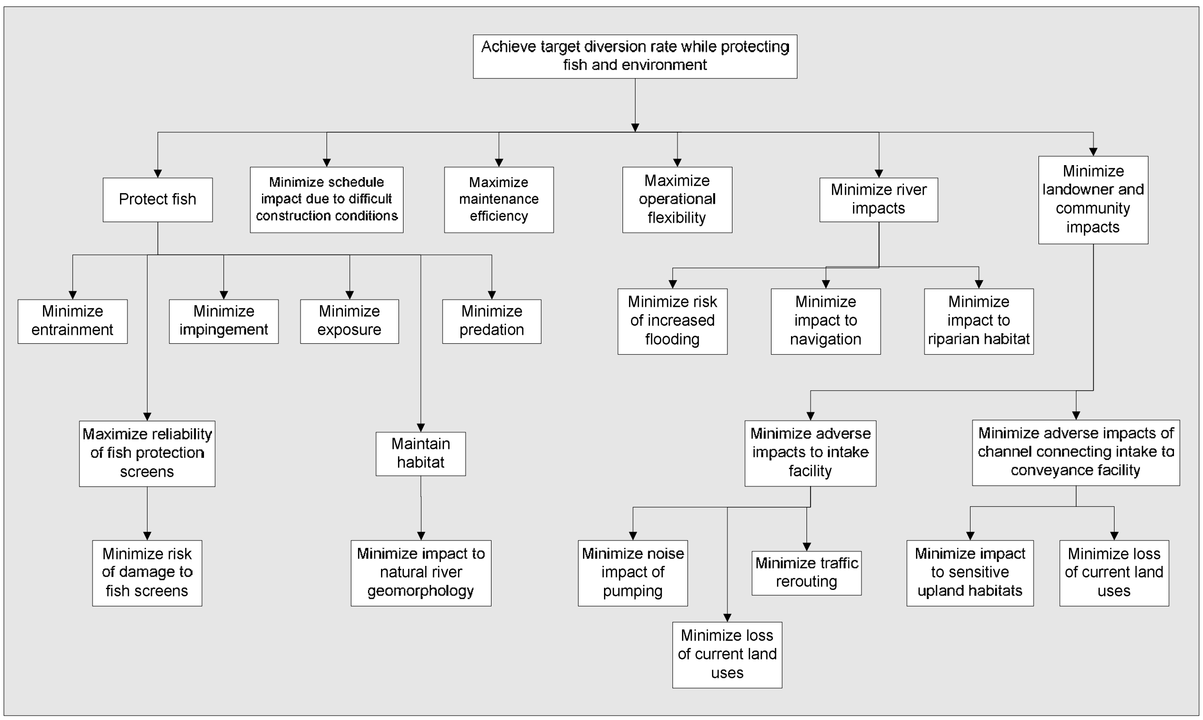


Figure 13. Project Objectives Flowchart

As shown on Figure 13, the fundamental purpose of the intake facilities is to achieve the target diversion rate from the Sacramento River (a maximum of 15,000 cfs) while protecting fish and the environment. The performance factors from the VPS are reiterated in **bold** below and augmented to reflect the key project objectives.

Operational Flexibility. Operational flexibility is largely a matter of being able to vary pumping rates over a range of flows and the ability to divert water irrespective of the distribution of special status fish species in the Delta system throughout the year. Because this criterion is primarily related to avoiding impacts to fish, it is shown under the objective “Fish Protection” on Figure 12. Operational flexibility can be accomplished primarily by having a number of relatively small intakes distributed over a large reach of the Sacramento River as opposed to a single large intake. As discussed above, it was decided to eliminate intake options that include a few or one large intake because intakes larger than 3,000 cfs have not been tried on California rivers, larger intakes could result in substantial local impacts on river hydrology, and large intakes could cause greater construction impacts. Therefore, all remaining options would provide roughly the same level of operational flexibility. For this reason, operational flexibility was eliminated as a criterion for evaluating intake options.

Maintainability. Potential for damage to fish screens was selected as the measure for the maintainability performance criterion. This criterion was dropped from further consideration because it was decided to eliminate cylindrical screens and use a single screen technology for the intakes, either an on-bank or in-river technology. This would make the screen technology essentially the same for each option. Therefore, the criterion would not provide a means of differentiating intake options.

Constructability/Construction Ease. All of the attributes identified for the VPS for the constructability/construction ease criterion influence the length of the construction schedule. Because the schedule can be measured on a natural scale that can be readily estimated for a given set of design and physical conditions, it was chosen as the measure for this performance criterion.

Length of the construction schedule and therefore the criterion of constructability/construction ease was eliminated from the evaluation of the remaining intake options. Regardless of the option selected for the project, construction would result in only short-term impacts to the environment. While these impacts may be substantial and constructability influences overall project cost, the long-term effects of intake O&M are of greater importance to the public and regulatory agencies. It was felt that if the constructability/construction ease criterion was carried through the evaluation of intake options, it could either inappropriately distort the analysis or have such a low weight relative to criteria that captured long-term effects of the intake that it would not provide a means of differentiating between intake options.

Fish Protection/Fish Benefits. Five potential measures were identified for the evaluation of intake options with regard to fish protection: minimizing entrainment, predation, impingement, exposure to the intake, and maximizing operational flexibility to avoid water withdrawals in areas of high concentrations of special status fish species. As discussed above, further evaluation following the VPS narrowed the range of intakes to include only those with relatively high operational flexibility. Resource agency criteria for screened intakes have established standards for minimizing entrainment and impingement which would be applied without differentiation to design of all intake options.

It is postulated that intake design could potentially influence predation by creating holding water favorable for predatory fish. This phenomenon has yet to be proven and no good scale was identified to measure predation potential among intake options. Entrainment and impingement of fish at an intake can be correlated to the length of time that various life stages of fish species are exposed to an intake screen. Exposure time can also be estimated for vulnerable life stages of fish species of concern. For these reasons, exposure time of fish to the intake screens was included as a criterion for evaluating the remaining intake options.

Landowner and Community Impacts. Four measures were developed for the performance criterion of minimizing landowner and community impacts: minimizing risk of collisions between vessels using the Sacramento River and the intake structure, loss of current land uses, impacts to other beneficial users, and construction traffic impacts. Risk of vessel collision was eliminated from consideration because of the difficulty of accurately estimating the risk level among intake options. Construction traffic impacts were eliminated from consideration because this represents a short-term impact that is less important than the long-term changes in communities that could result from an intake option. Loss of current land uses and impacts to other beneficial users were included as criteria for evaluating intake options. These criteria can be measured in terms of acreage of land where current uses would be changed.

River Impacts. The VPS identified flood, navigation, and hydrodynamic impacts as the attributes for the river impacts performance criterion. In developing the flowchart, navigation impacts were considered to fall under landowner and community impacts rather than river impacts; therefore, navigation was moved to that performance criterion. Engineering design would essentially eliminate the potential for hydrodynamic impacts regardless of the intake option selected for the project. Therefore, this attribute was eliminated for the river impacts performance criterion.

The measure selected for flood impacts was increased water surface elevation. Regardless of the intake option selected for the project, the design would take into account the potential for flooding and future sea level rise that could lead to flooding. Therefore, this measure would not be substantially different among intake options and was eliminated from further consideration.

Protect Terrestrial Species. A performance criterion that was not evaluated for the VPS was minimizing impacts to special status species other than fish. It will be necessary to construct pipelines or canals from the intakes to the isolated conveyance facility. The pipelines or canals will be constructed in riparian, upland, and possibly wetland habitats important to terrestrial special status species. To capture this aspect of the intake options, a criterion was added to evaluate the impact of intake options on special status terrestrial species.

Summary of Siting and Sizing Selection. With intake size or capacity established at 3,000 cfs per intake per the rationale described above, selecting intake locations is the final step in this process. To this end, the performance factors developed for the VPS were further evaluated to identify measures to be used to differentiate among the remaining intake options. It was determined that the following measures capture the important aspects of the intake options relative to their permitability and public acceptance and are likely to differentiate among option locations:

- Exposure of special status fish species to intake screens
- Acreage of special status terrestrial species impacted by project
- Acreage of land where existing uses would be changed by intake facilities

With the three basic comparison criteria established above, the ability to compare options is simplified. Rather than using a formal decision analyses process, a quantifiable comparison approach is employed. Decision analysis provides a formal method to analyze complex decision problems characterized by multiple and often competing objectives, many stakeholders, and uncertainties in assessing the impacts of alternatives. Because the performance factors developed for the VPS were reduced to three criteria that could be used to differentiate the remaining five intake options, an effort was made to evaluate the intake options with these criteria without applying a formal decision analysis approach.

3.4.1 ICF-West Preliminary Intake Site Selection

Example intake locations for an isolated conveyance facility (Figure 14) were used in conceptual engineering. The first preference in making this selection was to use intake locations as far north as possible. Tidal influence decreases the further north the intakes are located. With less tidal influence there is more constant sweeping velocities across the screens, thus reducing the exposure of fish to the screens. In addition, Delta smelt are assumed to be less abundant at the northern end of the Sacramento River reach being considered for the project.

Intake location C (refer back to Figure 1, page B 1-4, for letter designation locations) was not selected for the western conveyance facility because of its proximity to an existing intake at Freeport and its location about 1/2 mile north of the Sacramento Regional County Sanitation District (SRCSD) treatment plant outfall. Placing an additional intake just upstream of the SRCSD outfall could make it difficult for the SRCSD to meet its discharge (National Pollutant Discharge Elimination System [NPDES]) permit requirements. Intake locations E and E1 were eliminated from consideration for the west conveyance option because of their close proximity to Clarksburg. With one intake already selected in the vicinity of Clarksburg, adding another could substantially impact local recreational uses of the river important to this community. Intake locations F and G were the next most northern intake locations and they can be connected to the west conveyance option with a single pipeline corridor or canal, thus reducing land use impacts and impacts to terrestrial habitats.

3.4.2 ICF-East Preliminary Intake Site Selection

Intake locations B, D, E, F, and G on Figure 1 were selected for the eastern isolated conveyance facility (Figure 14). Intake location B is as far north as an intake can be for ICF-East without substantially impacting urban development in Sacramento. Intake location C was eliminated from consideration because construction of a pipeline or canal to transport the water from this location to an east conveyance facility would impact historic buildings in Freeport and a golf course. Additionally, the corridor for a conveyance route is highly constrained between the river levee and existing railroad.

Locations D and E were selected for conceptual engineering because they continue to be at the north end of the study reach, and water from these two intakes and an intake at location B can be transported to an east conveyance facility with a minimum of land use disturbance. Intake locations F and G were selected for conceptual engineer because they can also be joined to a single canal to move the water from all five intakes to the east conveyance facility with a minimum of land use disturbance and impacts to terrestrial habitats.

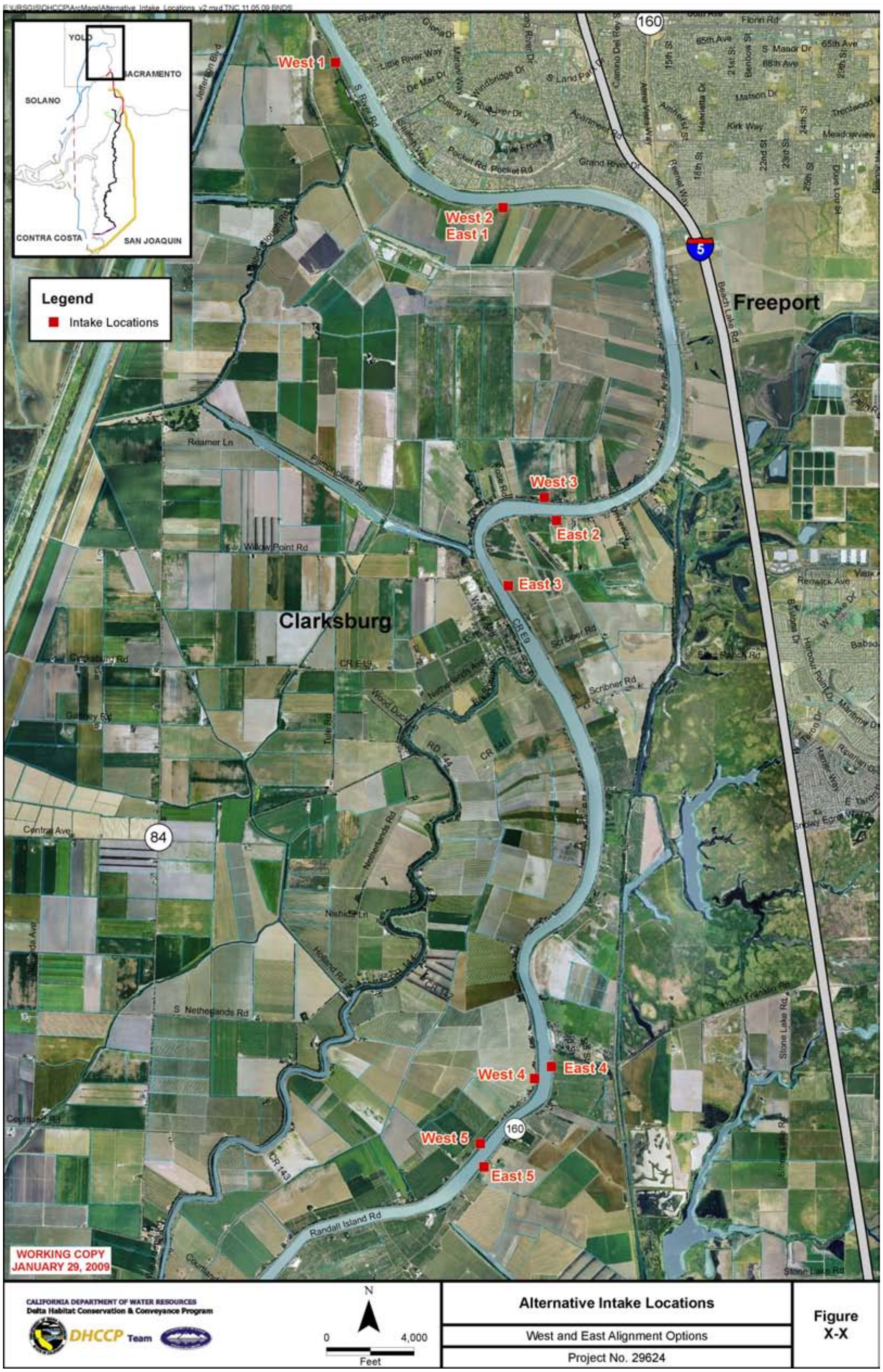


Figure 14. Example Intake Locations for Isolated Conveyance Facilities

3.5 Intake Technology Selection Workshop

A two-day intake technology selection workshop was conducted in January of 2009 to understand and compare the attributes of the four river intake types applicable to the proposed project. These four intake types consisted of the following:

- Single In-River (Figure 2)
- Single On-Bank (Figure 3)
- Dual On-Bank (Figure 4)
- Cylindrical Intake (Figure 5)

In order to evaluate the best technology for the proposed project, it is important to understand the basic geometry, construction approaches, and performance characteristics of each intake type being considered. The purpose of the workshop was to provide a basis for selecting a preferred intake technology and configuration for diverting water from the Sacramento River. To accomplish this objective a rational process of comparison, similar to that provided for intake siting and sizing, was applied.

Although the technologies identified above all are expected to produce similar levels of fish protection and riverbank impacts within some degree, a more focused analysis was required to discriminate between the technologies. The object of the workshop was to establish a basis for selecting an intake type based on various factors such as fish protection, riverbank impacts, constructability, risk of construction uncertainties, and cost of construction.

This activity required a broad cross-section of expertise canvassing a wide variety of topics and disciplines to obtain an objective consensus. A comprehensive list of participants was developed to ensure that the appropriate expertise was enlisted for the process. The expertise spanned a variety of subject areas including environmental, land use, fish biology, fish passage engineering, hydraulics and civil engineering, O&M, construction, and cost estimating. Additionally, several resource agency representatives from the FFTT participated in the workshop. (See the Attendance Record which follows.)

Upon providing an introductory synopsis of the FFTT study and VPS, a site tour was conducted to give participants a sense of the physical setting and existing site conditions. This was an instrumental part of providing the work group with firsthand perspective of the site conditions that exist for all of the intake locations and technologies considered.

Attendance Record for
Intake Technology Selection Workshop
Diversion Facilities on the Sacramento River

Monday January 26, 2009 @ 8:00 A.M.

Name	Signature
George Heise (CDFG)	George Heise
Dan Meier (USFWS)	Dan Meier
Brent Mefford (USBR)	[illegible]
Ron Ott (Ott H2O)	[illegible]
Gordon Enas (DWR-DOE)	[illegible]
John Moe (DWR-O&M)	[illegible]
Soheil Loghmanpour (DWR)	[illegible]
Zaffar Eusuff (DWR-Delta Branch)	
Teresa Engstrom (DWR-DOE)	
Andrea Turk (DWR-DES)	Andrea Turk
Marianne Kirkland (DWR-DES)	
Anthony Carpenter (DWR)	
Roger Padilla (DWR)	Roger Padilla
Shawn Mayr (DWR)	[illegible]
Tajeran Reza (DWR)	[illegible]
Sukhbir Singh (DWR)	
Steve Summy (WGI)	Steve Summy
Mike Cherry (WGI)	[illegible]
Patrick Liu (WGI)	[illegible]
Chris Mueller (B&V)	Chris Mueller

Name	Signature
Nhicolas Ly (B&V)	[illegible]
Victor Coquilla (B&V)	[illegible]
Randy Fiorucci (B&V)	[illegible]
Gerry Berg (B&V)	[illegible]
Paul Knietz (B&V)	
Jim Watson (B&V)	
Tim Buller (B&V)	
Carolyn Dabney	C Dabney
Bill Sutliffe	Wm Sutliffe
[illegible]	[illegible]
Ryan Irlmeier	[illegible]
Allan Davis	[illegible]
Ganesh Pandey	Ganesh

January 2009 Workshop Attendance Record

Upon return, a discussion was undertaken to understand the sensitivity of approach velocity criteria with respect to required screen surface area and overall structure footprint. This particular subject is critical to the geometry of the river intake facilities, since salmonid criteria requires 3 square feet of screen area per cubic foot per second of water diverted whereas smelt criteria requires 5 square feet of screen area. According to the FFTT study, it was suggested that salmonid criteria be used to reasonably reduce the amount of screen area fish would be exposed to and the magnitude of structures that would have to be constructed within the river. However, the rate of diversion would have to be reduced during periods when Delta smelt were occupy the river in the vicinity of the intakes. Other contingencies discussed by the group were deferral of additional intakes in the future, screen area provisions between 3 and 5 square feet per cubic foot per second of water, and bio-monitoring. The outcome of the discussion was that unless a variance was issued from the resource agencies, the only safe approach would be to design the intakes consistent with smelt approach velocity guidelines. Intakes designed to smelt criteria would avoid regulatory uncertainties, provide the largest structure footprint for EIR/EIS impact assessment, and avoid having to go through a subsequent permitting process should performance be found to be unfavorable to at-risk fish species.

For the purpose of technology comparison during the workshop, the initial assumption was that all intake technologies would be configured to divert 3,000 cfs per structure with adequate screen area to comply with salmonid fish screening criteria (0.33 fps). Exhibits were displayed using this assumption to present the scale of the four intake types, understanding that dimensional information was linearly proportional to whatever screen criteria was applied.

With a fundamental understanding of intake types and site conditions, an evaluation matrix was prepared to compare the technologies. The following comparison criteria were developed:

1. Maintenance demands
2. Navigation impacts
3. Fish exposure time
4. Construction impacts to landowners and communities
5. Riverbank impacts
6. Reliability
7. Aesthetics

Predation opportunity and flood impacts were additional criteria considered but deemed unsubstantiated by adequate information or analyses, and therefore were not included.

Using the above criteria, a comparison matrix was assembled and relative values were applied to each technology with respect to each criteria. A collective discussion was engaged to allow for collaborative input. Once values were assigned to how well each intake type met each criteria, the relative importance of each criteria was collectively assigned. As a result, a cumulative scoring comparison was produced and two separate trials run to understand the relative performance of technologies (Figures 15 and 16).

INTAKE TECHNOLOGY SELECTION
EVALUATION MATRIX

PERFORMANCE FACTOR		Maintenance	Navigation	Fish Exposure Time	Construction Impact to Landowners & Communities	Riverbank Impacts	Reliability	Aesthetics	Normalized Scores	Rank	Capital Cost 2/
Value/Weight		20	5	25	20	5	20	5			
Alt ID	OPTION DESCRIPTION 1/										
1	Single In-River	8	8	10	5	9	9	8	100.0	1	"Base"
2	Single On-Bank	9	10	1	5	2	9	5	69.9	3	156%
3	Dual On-Bank	7	10	6	1	1	9	5	69.9	3	n/a
4	Cylindrical Intakes	6	10	8	5	5	6	10	81.6	2	132%

1/ Based on description of options from Intake Technology Selection Workshop Jan 26-27, 2009.
2/ Costs are provided at a relative scale referencing the base cost for the first option shown.

Figure 15. Technology Comparison Matrix – Trial 1

INTAKE TECHNOLOGY SELECTION
EVALUATION MATRIX

PERFORMANCE FACTOR		Maintenance	Navigation	Fish Protection	Construction Impact to Landowners & Communities	Riverbank Impacts	Reliability	Aesthetics	Normalized Scores	Rank	Capital Cost 2/
Value/Weight		25	5	25	10	5	20	10			
Alt ID	OPTION DESCRIPTION 1/										
1	Single In-River	8	8	10	5	9	9	8	100.0	1	"Base"
2	Single On-Bank	9	10	1	5	2	9	5	69.8	4	156%
3	Dual On-Bank	7	10	6	1	1	9	5	73.4	3	n/a
4	Cylindrical Intakes	6	10	8	5	5	6	10	82.2	2	132%

1/ Based on description of options from Intake Technology Selection Workshop Jan 26-27, 2009.
2/ Costs are provided at a relative scale referencing the base cost for the first option shown.

Figure 16. Technology Comparison Matrix – Trial 2

Although further evaluation and refinement will be needed, the process provided a concept for the conceptual engineering of the isolated conveyance facilities. The second day of the workshop was spent discussing construction means, methods, and approaches for constructing each of the intake technologies. This was necessary in order to produce an assessment of cost.

During the discussion of likely construction elements and procedures, the following sequence of events was developed and deemed similar and appropriate for all intake technologies.

1. Mobilization/clearing/grubbing/site preparation
2. Establish traffic detour around site/widen existing levee
3. Excavate sedimentation basin/retrieval pit and provide shoring and bracing
4. Drive sheet piling cofferdam around intake footprint (barge mount)
5. Construct temporary bridging and access to cofferdam

6. Over-excavate interior of cofferdam cell and drive foundation piling
7. Place mat slab/tremie plug and dewater cofferdam cell
8. Install temporary walers, cross bracing, and pipe seals inside cofferdam
9. Perform single pass microtunneling and Jack 8'ϕ/12'ϕ WSP (number per intake type)
10. Install pipe elbows/manifolds, encase in concrete, and mortar/epoxy line piping
11. Construct intake slab-on-grade or cylindrical screen docking stations
12. Construct cast-in-place concrete walls and top deck
13. Construct pre-stressed i-girder bridge (in-river only) and site access provisions
14. Improve levee and install stone protection on embankment
15. Construct upstream deflector (cylindrical only)
16. Install cylindrical fish screens, baffles, screen cleaning systems, and appurtenances
17. Trim sheet piles to grade/remove bracing and walers
18. Place rip rap around cofferdam cell remnant perimeter (underwater)
19. Construct screen retrieval tracks (cylindrical screens only)
20. Construct top-side docking stations (cylindrical screens)/guardrail/fencing/lighting
21. Install hydraulic/electrical (cleaning) and winch (removal) systems
22. Install pipe manifolds and fittings on landside of levee
23. Demob/restore and re-establish former traffic route(s)

In discussing the construction approach for the dual in-river technology, it became apparent that not only did this intake type perform least favorably (Figures 15 and 16), but also was by in large much more complicated and costly to construct. Therefore, it was deemed appropriate to dismiss this intake technology from further consideration. Accordingly, costs were not developed for this technology. For the remaining three options, the construction costs were prepared based on the elements and approaches described above, including industry standard percentages for engineering, construction management, and construction cost contingency.

According to these preliminary opinions of cost, the in-river intake technology provided the lowest cost, followed by the cylindrical intake concept at 32% greater, and the single on-bank concept at 56% greater.

3.6 Summary of Diversion Scheme Selection

To restate the key outcomes of the concept planning and development described above, the following preliminary arrangements for river intake concepts are proposed for the ICF-East, ICF-West, ICF-All Tunnel, and TDF conveyance options:

- Five 3,000 cfs intakes;
- In-river intake technology appears to best meet project objectives;

- For ICF-East, intakes located at B (south boundary of the Pocket Area), D (southern east-west leg of the Freeport Bend), E (due east of Clarksburg), F (just downstream of Hood), and G (between Hood and Courtland);
- For ICF-West, intakes located at A (west of the Pocket Area, B (south boundary of the Pocket Area), D (southern east-west leg of the Freeport Bend), F (just downstream of Hood), and G (between Hood and Courtland);
- For TDF, two 2,000 cfs in-river intakes will be located at F (just downstream of Hood), and G (between Hood and Courtland).

Based on the planning process and step-logic procedures performed, this arrangement of intake facilities forms the basis by which conveyance facilities would be supplied raw water from north Delta diversions on the Sacramento River. Conceptual engineering has proceeded based on these decisions.

4.0 TOPICS FOR FURTHER CONSIDERATION

There are a number of issues that surround the optimization and configuration of diversion schemes for conveyance options, all of which are intended to be addressed in subsequent phases of planning and design. The preliminary intake concepts are arranged according to best available information and industry expertise. However, there are subject areas requiring further investigation that could influence the ultimate outcome of a final diversion scheme for the program. Particular topics that require further analysis and definition to best configure intake arrangements are as follows:

- BDCP Proposed Operations
- Geotechnical Conditions
- Real Property / Land Issues
- Water Quality Conditions and Discharge Sources
- Finite Element Hydraulic Modeling
- Flood Impacts and Mitigation
- Levee Integrity Considerations
- Navigation Impacts
- Sediment Transport Modeling
- Scaled Physical Modeling
- Environmental Studies
- Architectural Treatments
- Invasive Species (Quagga Mussel) Provisions

5.0 CONSTRUCTION APPROACHES, MEANS, AND METHODS

Construction of diversion facilities requires means, methods, and approaches unique to marine and heavy civil construction. Although a variety of intake configurations have been conceived for the project conveyance options, construction methodologies are relatively consistent regardless of intake type. Primary construction constraints associated with each intake include weak foundations, deep excavations, pile support, river hydrology, builder’s risk of constructing in natural waterways, cofferdamming and dewatering, producing and maintaining a dry working environment, multi-year construction windows at each site, potential conflicts with work performed by others, site access challenges, and staging area availability to name a few. This section describes the basic construction approaches and types of construction for intake facilities.

5.1 General Constructability Considerations

In general, constructability considerations include, but are not limited to, mobilization and demobilization; contract administration; development of staging/storage areas and construction zones; earthwork, deep excavation, and shoring and bracing; levee construction, slurry cut-off walls, and deep soil mixing; cofferdamming, dewatering, and tremie slab construction; microtunneling, trenching, and pipeline installation; foundation preparation, stabilization, and foundation pile installation; conventional cast-in-place concrete construction involving formwork, reinforcing, placement, and finishing; bridge construction (in-river intakes only); metalwork fabrication, assembly, installation, and structural framing; and miscellaneous civil site and electrical work.

All intake options and sites will possess unique infrastructure complexities, foundation characteristics, and construction periods to complete. Significant temporary construction zones are required for staging and storage. Particular construction challenges include the following:

- Driving sheet and foundation piles to significant depths to achieve hydraulic cut-off. Considering subgrade of intake structures is in the order of 50 feet below sea level and conveyance piping is at even greater depths, pile installation will be challenging.
- Tunnel boring and conduit construction under levees (and river channel).
- Underwater construction such as tremie slab placement and sheet pile trimming.
- Cofferdamming, shoring, and bracing.
- Site access and dewatering.

Particular construction elements common or unique to all intake facilities are listed below.

1. Staging/storage area and construction zone preparation (5 to 10 acres per each intake structure, 20 to 40 acres including sedimentation basins and pumping plants)
2. Retrieval pit construction for microtunneling/pipe jacking operations
3. Tunnel boring and pipe jacking under levees and river channel

4. Sheet pile cofferdamming, shoring, bracing, and hydraulic cut-off
5. General earthwork (e.g., excavation, spoil, backfill, levee construction)
6. Dewatering wells, construction water treatment, return to watercourse
7. Foundation preparation and mat slab construction inside sheet pile cell
8. Vertical shaft construction and piping
9. Cast-in-place reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing)
10. Metalwork fabrication, machining, assembly, and installation (stainless steel fish screen panels or cylinders, embeds, flow control baffles, bulkheads, traveling brush screen cleaning system, gantry crane mechanical hoist system, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.)
11. Erosion control (underwater placement of stone protection/geotextile)
12. Pre-stressed I-girder bridge construction (in-river intake only)
13. Miscellaneous civil sitework (e.g., fencing, gates, access roadways and ramps, log booms/debris deflectors, hydroseeding, landscaping, etc.)
14. Miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, traveling brush power transmission, flow/level/turbidity/limit/torque instrumentation, utility service)

Individual construction approaches unique to each intake type are described in Sections 5.2 through 5.5.

5.2 Single In-River Intakes

Because the in-river intake technology is used as the preliminary intake concept for the CER, a greater emphasis and level of detail is provided herein to describe the construction of this style of intake. Although there are unique permutations to constructing each type of intake, for the most part, construction approaches are quite similar for all intake technologies. For the purpose of describing intake construction, it should be noted that intakes are coupled with onshore pumping plants and sediment management facilities. Although those elements are not described in detail here, some aspects of construction, such as detour construction and staging, apply to the intake and pumping facilities as a whole.

A single in-river intake structure is approximately 380 feet in length (designed per smelt criteria) and 30 feet in width located approximately 100 feet to 200 feet from the adjacent levee crown in the deeper part of the river cross-section. Although construction access is more challenging than an intake facility constructed flush with the river levee, an in-river intake involves less levee disturbance and may offer the benefit of not having to construct a set-back levee in advance (unless needed for flood impact migitation).

Prior to beginning construction, it will likely be necessary to establish a roadway detour around the approximate 40-acre construction zone for each intake to provide site safety and security.

Since major roadways are situated on top of the river levees, significant detour roadways are likely to be needed for traffic circulation around the work areas. It is expected that earthen ramps will be required to realign the roadways from levee crown to landside ground elevation. It is expected gradual vertical and horizontal curves, grade transitions, signage, and barricades will be needed to conform to County and State Department of Transportation (DOT) requirements. Surfacing and section requirements will also need to comply.

While detour routes are under construction, preparation of the contractor’s staging area can occur simultaneously. Construction of the subject facilities will require ample space (roughly 40 acres maximum) for equipment, material storage and laydown, temporary facilities and job trailers, working space, and spoil/stockpiles areas. At some sites, selective demolition of existing improvements, realignment of existing utilities, and clearing of significant vegetation, such as vineyards and orchards, may be required.

Once the contractor’s staging area is developed and roadway detours are established, the contractor may proceed with widening and fortifying the existing flood control levees in accordance with United States Army Corps of Engineers (USACE) permitting requirements. Levee widening is necessary to provide for permanent owner ingress/egress to the facility. Slurry walls or deep soil mixing may be a required counter-measure for preventing seepage due to conduit construction underneath the levees. It is expected the existing levee crown needs to be widened roughly 30 feet several hundred feet upstream and downstream of the facility centerline for ingress/egress. This will also aid access provisions during construction.

A significant constraint affecting the construction of in-river intakes is the ability to conduct in-channel work. CDFG streambed alteration agreements (1600 permits) typically limit construction within natural waterways between July and October . The most important aspect of constructing an intake in the river is implementing a reliable and stable cofferdam system. Because of the significant sheet piling needed to accomplish this task, it is likely a barge-mounted crane is needed to drive the sheets. Since pick lengths are limited to less than 100 feet, a barge-mounted unit would be the most practical approach to picking and driving the sheets.

Prior to commencing sheet pile installation, the contractor will install temporary piles in which to attach his template for driving sheets. This provides the contractor with the means of guiding and controlling sheet placement. Because of the tip elevations needed to ensure hydraulic cutoff and structural stability relative to the subgrade depth needed for a tremie plug (~El. -50 NAVD), it is expected heavy sheets in the order of 100 feet in length will be needed to drive effectively. This is expected to leave pile butts at elevations comparable to adjacent levee crowns. Depending on load conditions and soil structure interaction, it may be necessary to alternate king piles into the cofferdam shell.

Upon establishing the shell, over-excavation of the cell interior is needed to reach subgrade either using a barge-mounted clamshell dragline or suction dredging equipment. Excavation can either be performed in the wet or in the dry. If attempted in the dry, a substantial dewatering well field would be required to pull down the water table. With this approach, conventional excavation in the cell could be conducted and only a nominal tremie plug or mat slab would be needed. If attempted in the wet, a dragline or suction dredge unit would be needed to remove material underwater. A floating suction dredge could be inserted inside the cofferdam, whereas a dragline would likely need to operate from a barge or via decking installed over the top of the cofferdam. The excavated material would need to be off-hauled or pumped to a spoil area on shore.

In either case, a temporary bridge deck would be constructed to provide equipment access for performing work inside the cofferdam, similar to that which was done for construction of the City of Sacramento's in-river intake facility. Temporary piles would be driven to support welded steel girders and likely timber decking. In addition, temporary decking at the top side of the cofferdam would be needed to provide access for a lattice boom crane to support construction inside the cofferdam.

With access established and over-excavation complete, a substantial matrix of foundation piles (end bearing or friction) would be needed to underpin the intake structure. It is initially expected a closed end pipe pile would be used and concrete-filled when depth is achieved. A combination of vibratory and diesel-powered drop hammers would likely be needed to achieve design tip elevations for piling construction.

Given that the intake structure and piping elements are significantly below typical water surface elevations in the river, producing a dry working space will be challenging. It is expected a contractor would need to underwater place a substantial tremie slab to provide a seal at subgrade for installing conduits under the intake. Sheet pile interlocks would likely also need to be sealed to limit infiltration. Following the installation of sheet piles, excavation of the cell, and installation of foundation piles, the substantial tremie slab would be placed underwater bonding to the sheets and the foundation piles. To resist the substantial hydrostatic pressure, as the interior water surface is pumped down, a series of walers and struts or cross-braces would be welded to the interior of the cofferdam. This would make the cofferdam self-supporting and allow for nominal dewatering provisions to maintain a dry working space.

With the cofferdam and temporary bridging established, work inside the cofferdam could be carried out year round. Installation of the conduits connecting the intake to the landside sedimentation basin would be accomplished by single-path microtunneling using an earth pressure balance or slurry tunnel boring machine (TBM). For each conduit, a pipe seal would be welded to the inside face of the cofferdam and an opening would be cut into the sheet piling to allow for pass-through. Exterior stabilization may be needed around the location where the conduits penetrate the sheet piling. This ground improvement and stabilization could be accomplished using jet-grouting techniques, if required.

Once the tunneling operation is initiated, the TBM would be advanced and pipe segments would be jacked into the bore, chasing or following the TBM. A jacking frame would be constructed inside the cofferdam for each conduit run. Pipe with flush bell-and-spigot or butt welded joints would be used. The open excavation for the sedimentation basin on the landside would serve as the retrieval pit for the TBM. A sheet pile head wall would be constructed to maintain a rigid face for the TBM to break through. Once each conduit run is completed, the TBM would be broken down, returned to the cofferdam, and the process repeated. As conduits are completed, pre-fabricated special Y fittings would be installed, mated up with the respective conduit, and encased in structural concrete.

With the conduits in place, the next step would be to carry out conventional concrete construction within the cofferdam. The conical hoppers and slab-on-grade would be cast, followed by the exterior and divider walls, and concluded with placement of the structural top deck. Embeds would be integrated into the formwork to serve as guide channels for the screen panels, baffle assemblies, and bulkheads. As the concrete placement is carried up vertically, temporary struts would be removed and the walers would be braced off of the structure. Upon curing, stripping, patching, and sacking of the structure, various metalwork and mechanical items would be installed.

At the same time, construction of the permanent bridge could be undertaken. It is expected that the bridge can be constructed in a single span using pre-stressed concrete I-girders. The landside and intake abutments would be constructed to receive the pre-cast girders, likely to be placed using tandem cranes, one on the levee and the other on a barge. Once the bearing pads are placed the girders would be set and the decking cast. The temporary bridge piling and decking would be removed prior if in conflict with the permanent bridge.

With the major infrastructure elements complete, ancillary installation of the metalwork and mechanical items could be completed. Screen cleaning systems, gantry cranes, and miscellaneous instruments and electrical features would be installed. Screens and baffles would be inserted and checked for fit-up. On the exterior sides of the cofferdam, stone protection would be placed underwater to transition and tie the structure into the river bottom. Upon completion of this work, the cofferdam could be flooded and underwater divers and torches/plasma cutters could be used to trim sheet piling to the finished grade of the structural slab.

After completing the intake, site restoration would ensue, the permanent roadway would be constructed, and demobilization would be carried out. The various civil site improvements and miscellaneous electrical work would be accomplished to complete the intake. It is estimated that construction of this intake type will require approximately two and a half to three years to complete.

5.3 Single On-Bank Intakes

The length of a single on-bank intake is nearly three times that of an in-river intake in order to provide the same effective screen surface as an in-river intake due to a shallower net depth and single screen face. A single on-bank intake structure is on the order of 1,140 feet in length (designed to smelt criteria). Although not as challenging to construct as an in-river intake, this option will require substantially greater river disturbance and much greater concrete and cofferdam construction at each location as compared to the in-river type.

Since the components of an on-bank intake are essentially the same as for an in-river intake, construction methodologies will be similar short of access bridge construction. The same sequence is expected with a three- to four-year construction schedule required to complete.

5.4 Dual On-Bank Intakes

To provide comparable screen bank length as an in-river intake, dual on-bank intakes could be constructed at a single site. A dual on-bank intake scenario would involve two intakes at opposing sides of the river in the order of 620 feet long. This will involve significantly greater construction footprints, impacts, and challenges than the two former intake types described. Although individual intake construction of a dual on-bank intake is identical to that of a single on-bank intake, connection piping between the two requires significant tunneling and shafting activities. Launch and retrieval portals are required and commercially available TBMs would be needed to plumb the two intakes together. The work and schedule is roughly double for the dual intake option in contrast to a single intake option, even though the net structure footprint is only increased by roughly 25 percent.

In addition, sheet pile installation and providing a hydraulic cutoff at the outside perimeter of the cofferdam cell will be complicated due to pile tip conflict with tunneled piping installed in advance of driving piles. In all, set-back levees, construction areas, and scope of work will be twice that of a single on-bank intake. Unless multiple crews are used, the construction of a dual on-bank intake is expected to require four years to complete.

5.5 Centralized Intakes

Construction of the single-source intakes for the CCF and San Joaquin River Bypass diversions is expected to be facilitated due to consolidation of single intake locations and because the construction sites are disconnected from the waterways. In addition, structure depth is not as substantial as the aforementioned technologies, so maintaining a dry working space is expected to be feasible with a conventional dewatering well field.

However, the footprint of the intake structures is considerable, roughly 500 feet in width by 1,000 feet in length. It is expected that similar construction approaches used for the earlier intake types are needed including sheet pile cofferdam and support pile installation, mass excavations, dewatering, foundation stabilization, and concrete construction. Deep piping is one element of construction that is not required by centralized intakes. Once primary facilities are complete, the earth between intake and river channel could be removed in the wet. Topside facilities associated with the pumping equipment and fish salvage facilities would not be critical path and could be constructed as a contractor’s schedule allows.

It is estimated that construction of this intake type will require four years to complete, both sites being constructed concurrently by different contractors or construction crews. Since the sites are relatively remote and unencumbered, sufficient staging/storage area and public impacts are not foreseeable concerns. All-weather access would be incorporated during the first season once channel realignment and bridge crossing work is completed. Installation of cellular cofferdamming, dewatering, mass excavation, and foundation preparation and pile installation are expected to be performed in the second season. Concrete work is expected for the third season, and mechanical, electrical, and topside facility construction is expected for the fourth season.

6.0 MAINTENANCE CONSIDERATIONS

The proposed intake facilities will require routine or periodic adjustment and tuning to ensure operations are managed consistent with design intentions. Facility maintenance is part of long-term asset management and includes activities such as painting, cleaning, repairs, and other routine tasks that ensure the facilities are operated in accordance with design standards after construction and commissioning.

O&M activities will involve performing routine, preventive, predictive, scheduled, and unscheduled maintenance aimed at preventing equipment/facility failure or deterioration. The goal of O&M is to increase efficiency, reliability, and safety. To ensure project objectives are met, certain O&M exercises must be carried out. Like any operating facility, maintenance is an integral part of a functional and reliable project.

6.1 General Inspections

Routine visual inspections of the facilities will be important for monitoring and logging performance; recording the history of facility conditions and deterioration; identifying trends that occur with respect to river hydrology, climate conditions, and other factors; and preventing mechanical and structural failures of project elements. Continual inspections are important, not only while the facilities are in operation, but also during down times.

A deliberate monitoring program will increase awareness of conditions compromising operational performance and basic function. In terms of relative difficulty, inspections can vary from visual observations from the top surface of facilities to underwater examinations using specially trained underwater diving crews to dewatering for access and firsthand field verification. Video and photographic inspections, along with thorough record keeping of observations, will aid in understanding how well the facilities weather the elements and operate in the face of dynamic conditions. A proactive inspection program is consistent with the asset management of critical infrastructure and will extend the service life of the facilities.

6.2 Sedimentation Removal

Sediment deposition is a problem that commonly plagues man-made infrastructure in natural waterways. It can bury intakes in particular and either reduce their capability to divert or force shutdowns altogether until working conditions are restored. Attention to this issue during engineering and design can reduce or avert this problem. However, the dynamic riverine environment can be unpredictable and there is the chance that sedimentation can inhibit function and operations. Typical maintenance activities associated with river intakes can include the following:

- Suction dredging around intake structure using raft- or barge-mounted equipment and pumping sediment to a landside spoil area.

- Mechanical excavation using track-mounted equipment and clamshell dragline from the top deck after installing a floating turbidity control curtain.
- Dewatering of intake/sed basin/pumping plant bays to remove sediment buildup in conduits and channels using small front end loading equipment and manual labor.

Planned operation of proposed intakes will prevent sediment deposition within the intake bays and conveyance conduits when turbidity in the river exceeds a certain threshold. The sediment removal systems will be effective in keeping the sedimentation channels and wet well bays free of sediment buildup. It is expected only extreme conditions would give cause for the activities listed above.

6.3 Debris Removal

Should substantial debris become lodged at the leading edge or adjacent to the intake structure, removal of the material may require equipment and specialized labor. Although historically the in-river intake technology has not shown to be a debris trap, there may be incidents where large debris deposits in the vicinity of the structures compromise its function. In the wake of heavy to extreme hydrologic events, inspections should be conducted to visually confirm debris presence or the lack thereof. If large debris is found to have accumulated, it is expected removal would require underwater diving crews, boom trucks or rubber wheel cranes, and possibly a small barge and crew to rig the leads to the debris.

With respect to the centralized intake concept, it is expected that debris loading in the form of vegetation will be a significant factor hampering facility performance. Accordingly, the proposed design of this intake type includes self-cleaning trash rack mechanisms at the upstream end of the intake facilities. The devices are fully automated and are used to continuously rake debris upward from the trash racks into a continuous debris conveyor at the top deck. The conveyor will transport the collected debris in a spoil area or roll-off containers for convenient off-hauling. The frequency of these efforts will depend on debris load conditions within the river.

As for screen operations, the continuous traveling brush mechanisms, or other screen cleaning technology applied, are expected to maintain a relatively clean screen face and adequate open area. The outbound current is relied upon with this type of cleaning system to transport raked debris past the screen banks once brushed from the surface. Cleaning frequency will need to be varied commensurate with debris load conditions in the river.

6.4 Biofouling

Accumulation of algae, freshwater sponge, mussels, and other biological organisms is a known ailment of structures in waterways. Over time, bio-fouling can occlude the screens and jeopardize function. The key design provision for intake facilities is that all mechanical elements be removable from the top surface for convenience of inspection, cleaning, and repairs as needed. The intakes will feature topside gantry crane systems for removal and insertion of screen panels, louver assemblies, and bulkheads.

It is expected that all panels will require annual removal (at a minimum) for pressure washing. Additionally, individual intake bays will require dewatering (one pair at a time) for inspection and assessment of bio-foul growth rates.

With the impending invasion of Quagga and Zebra mussels in inland waters of California, it may be simply a matter of time before these organisms will affect operation of intakes in the Central Valley. Frequency of bio-foul removal will intensify with the advancement of these organisms. Coatings and other deterrents will be more thoroughly investigated during preliminary and final design to protect against these invasive organisms.

6.5 Corrosion

Since a substantial amount of metalwork will be incorporated in the intakes, aerobic and galvanic corrosion will need to be monitored. Materials are expected to consist of plastics and austenitic steels (stainless), so generally speaking, corrosion is not expected to be detrimental to the life of the facilities. Passive cathodic protection systems may be employed to preserve the condition of submerged metals and thereby extend their service lives. Maintenance associated with these systems generally consists of replacing sacrificial (zinc) anodes at multi-year intervals. Removal of screen and baffle elements for cleaning will allow the opportunity for inspection of metalwork, thereby permitting the assessment of corrosion rates. Metal items receiving coatings will be more prone to localized corrosion attack, and therefore more subject to a routine inspection process involving forensic material testing and metallurgic analyses, similar to that required by American Water Works Association (AWWA) M42 and D100 for water storage reservoirs.

6.6 Impact Repairs

Impact damage incurred by the intake facilities can be the result of incidents such as boat collisions, debris impact, stone and sediment abrasion, etc. Impact damage is not as much a concern for centralized diversions as it is for river intakes. The centralized intakes will be shielded behind substantial upstream trash racks. The river intakes will be aligned parallel with the predominant flow direction; therefore, the brunt of impact will be borne by the substantial concrete pier nosing at the leading edge of the structure, serving to absorb and deflect downstream traveling debris. Although impact damage is not a common problem for in-river intake structures in the Sacramento River, should it occur, repairs will be required.

Elements considered to be most exposed to impact damage are the screen panels, baffle assemblies, and traveling brush mechanisms. The robust concrete structure that houses these elements is quite durable and not expected to suffer much damage. Should the less robust metalwork be damaged, maintenance would consist of removal and repair. With the aforementioned items being constructed of stainless steel alloys, spot repairs would require specialized skills most likely involving shielded gas, wire-feed welders and other common machining tools. Spares should generally be stored on site to minimize down time should extensive repairs be needed.

With the majority of working components being submerged and adequate security provisions in place, vandalism is not expected to be a significant problem.

6.7 Mechanical Equipment

The systems associated with intakes involving power-driven and routinely moving parts are the screen cleaning systems, gantry crane hoist systems, self-cleaning trash rake systems, and pneumatic spillway gates. Lubrication of bearings, continuity checks of limit/torque switches, and periodic inspections of equipment per manufacturer recommendations are the primary O&M tasks expected for these systems. Strip brushes for the screen cleaning systems will need replacement every several years. On-site vendor training and O&M manuals will equip staff with the knowledge for maintaining unique and specialized components. Following these instructions will ensure years of safe and reliable service.

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APPENDIX C

Solids Sedimentation

1.0 CONVENTIONAL SOLIDS SEDIMENTATION

1.1 Overview

Conventional solids sedimentation is a process in which a basin of a suitable size and capacity is provided, such that flow velocity is reduced sufficiently to allow solid materials to fall out of suspension by gravity. Sedimentation basins can be circular or rectangular. Rectangular basins have a number of advantages over circular basins: predictability, cost-effectiveness, and low maintenance.

At the intake pumping plant, it is proposed to construct rectangular sedimentation basins downstream of the intake screens and upstream of the pump intakes, to allow a majority of the grit size particles that have passed through the fish screens to be settled and collected before they reach the pumps.

A conventional sedimentation process would be simple in terms of operation and maintenance (O&M), with a minimal amount of mechanical equipment and no chemical addition. The sedimentation basins would require settled solids collection equipment, a solids pumping station, and drying/settlement lagoons.

Removal of the grit size particles prior to pumping would improve pump longevity, minimize uncontrolled siltation in the transmission canal and forebay at Clifton Court and increase the overall efficiency of the conveyance system.

The fish screens for the intake structures would have openings of $1/16^{th}$ of an inch and would therefore act as an initial barrier to larger sediment. The use of a rectangular basin for subsequent settlement of the smaller particles would allow the water to flow horizontally through a long tank. This suits the configuration of the intake pumping plant.

The water level in the proposed basin would be dictated by the water level in the river. The flow of water through the basin would be controlled by starting and stopping the intake pumps. Upstream fixed weirs and baffles, and downstream adjustable weirs, would be used to reduce streaming at high river levels and potential short-circuiting of the settlement area of the individual channels.

The nature of the solids and the temperature of the water will influence the performance of the channels. Laboratory tests of the raw water should be carried out as part of the detailed design. As the configuration and purpose of the sedimentation basins does not conform to the conventional sedimentation tank model, computation fluid dynamics modeling of the tanks should be used to optimize the basin performance during the detailed design stage.

1.2 Suspended Solids and Bed Sediment Data

A preliminary desktop design was performed using sediment data available through the United States Geological Survey (USGS) sample station at Freeport on the Sacramento River to identify the particle size factors (USGS, 1997-2007) and calculate sedimentation basin size. The available suspended solids data for the last 11 years was reviewed.

Suspended sediment concentration (SSC) in river water is expressed as a mass of suspended sediment per unit volume (in milligrams per liter [mg/L]). USGS data for the Sacramento River indicates a range of suspended sediment from 1 mg/L to a maximum of 512 mg/L.

Suspended sediment in river water is predominantly fine sediment of less than 0.063 millimeter (mm) in diameter (Table 1). Based on available data from 1996 to 2007 (USGS, 1996-1999; 2005-2007), the average percent of the total entrained solids by weight of this fine suspended sediment is 75%; the median value is 84 percent.

Table 1: Suspended Sediment Percent < 0.063 mm

Record Year	1996	1997	1998	1999	2005	2006	2007	Summary
Median Suspended Sediment < 0.063 mm (%)	75	48	84	87	87	83	89	84
Average Suspended Sediment < 0.063 mm (%)	65.5	44.3	83.9	86.3	84.8	73.8	87.4	75

The percentage of suspended solids comprised of fines (i.e., <0.063 mm) varies with river stage and flow, with a minimum to maximum sample range of 26% to 99% of the total solids loading (USGS, 1996-1999 & 2005-2007).

In the absence of suspended solids data for grain sizes larger than 0.063 mm, it has been assumed that solids larger than the fines correspond to bed sediment makeup. This assumption is supported by further analysis of the solids loading and river flows, which shows a marked connection between flow volume and carried sediment, as shown in Table 2 and on Figure 1 below. It is apparent that the Sacramento River experiences suspended solid spikes during heavy rainfall/wind events and when the river stage rises. These events evidently result in an increase in bed sediment being stirred up and entrained in the flow due to the increase in river velocity.

Table 2: Suspended Sediment Concentration Summary

Suspended Sediment Concentration (mg/L)	
Average Monthly Minimum	8.9
Average Monthly Maximum	178.2
95th Percentile	196.25
Five Year Average	59.3
Dry Season Average (May - September)	39.3
Wet Season Average (October - April)	73.4

mg/L = milligrams per liter

Average monthly suspended solids peaks during high river discharge or wet weather conditions are up to 179 mg/L.

USGS data was analyzed to determine the percentage of potential particle sizes anticipated above 0.063 mm in size. Since flow would be routed to the sedimentation channels via fish screens that will screen out particle sizes greater than 1/16th of an inch (1.6 mm), the size analysis was further refined to consider only grit particles smaller than this size.

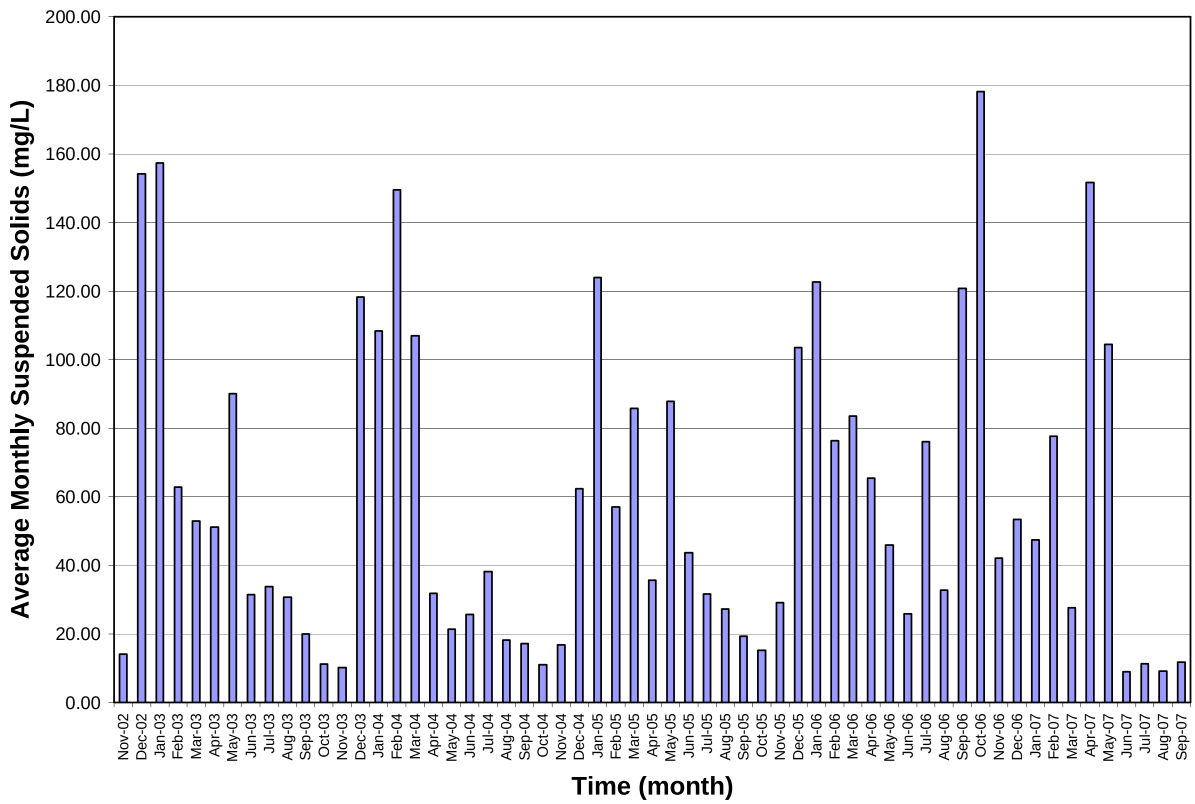


Figure 1: Average Monthly Suspended Sediment Concentration

A review of available bed sediment samples identifies the makeup of potential suspended solids passing into the sedimentation basin as shown in Table 3. Primary particles of concern are between 0.125 mm and 1 mm with nearly 100% of particles passing the 2 mm sieve in all cases (USGS, 1996-1999 & 2005-2007). The majority of the particles are anticipated to be between 0.25 mm and 0.5 mm in size.

In summary, based on USGS sample data, a median value of 84% of the total suspended solids in the river are fines (clays and silts) and cannot be settled out using plain sedimentation without chemical addition (Table 4). Of the remaining median 16% of solids, it is estimated that an average of 95% of the potential sediments larger than 0.063 mm in diameter and less than the fish screen size (1.6 mm) can be removed by plain sedimentation.

Table 3: Bed Sediment Percent Passing Sieve

Record Year	<0.063 mm	<0.125 mm	<0.25 mm	<0.5 mm	<1 mm	<2 mm	<4 mm	<8 mm
1997	6	11	26	62	93	98	100	100
1998	5	7	14	67	90	97	99	100
1999	1	4	17	71	94	98	99	100
2005	6	9	20	75	95	98	100	100
2006	8	11	26	77	97	99	100	–
2007	5	14	30	70	93	99	99	99
Average	**5**	**9**	**22**	**70**	**94**	**98**	**99**	**100**

Table 4: Sediment Composition (By Size)

<table>
<tr><th rowspan="2"></th><th>Settleable (16% of All Solids)</th><th>Non-Settleable (84% of All Solids)</th><th>Settleable and Non-Settleable (100% of All Solids)</th><th rowspan="2" colspan="2"></th></tr>
<tr><th>% Total</th><th>% Total</th><th>% Total</th></tr>
<tr><td>D_p >2 mm</td><td>2</td><td>0</td><td>0.32</td><td colspan="2">Blocked by Fish Screen</td></tr>
<tr><td>2 mm> D_p >1 mm</td><td>5</td><td>0</td><td>0.80</td><td rowspan="6">Can be Settled by Plain Sedimentation</td><td rowspan="6">Passes Through Fish Screen</td></tr>
<tr><td>1 mm> D_p >0.50 mm</td><td>23</td><td>0</td><td>3.68</td></tr>
<tr><td>0.50 mm> D_p >0.25 mm</td><td>48</td><td>0</td><td>7.68</td></tr>
<tr><td>0.25 mm> D_p >0.125 mm</td><td>13</td><td>0</td><td>2.08</td></tr>
<tr><td>0.125 mm> D_p >0.063 mm</td><td>4</td><td>0</td><td>0.64</td></tr>
<tr><td>0.063 mm> D_p</td><td>5</td><td>100</td><td>84.80</td></tr>
</table>

D_p = particle diameter
mm = millimeter

Notes:
Particles >2 mm in diameter will not flow through fish screens.
Particles approximately < 2mm and ≥0.063 mm in diameter are settleable by plain sedimentation.
Particles <0.063 mm in diameter are not settleable by plain sedimentation.

On average, it is estimated that 15% of total suspended solids in river water can be settled out using conventional sedimentation without chemical addition.

1.3 Sedimentation Basin Design

As part of the preliminary design, the flow-through velocity of each channel was set as a function of basin depth and width at a given flow rate. Basin depth was set based on river stage elevations and a minimum water depth in the river of 3.5 feet North American Vertical Datum of 1988 (NAVD88).

Industry standard-specific gravity and settling velocities for fine-to medium-grained particles were used to determine an optimum basin length (Lindeburg, 2008). For each discrete particle size, calculations were performed to determine the horizontal distance that the particle would travel before it reached the bottom of the sedimentation basin. This was then re-calculated for different basin depths to identify the optimum length and depth combination for the desired percentage removal. The product of these calculations is presented graphically on Figure 2.

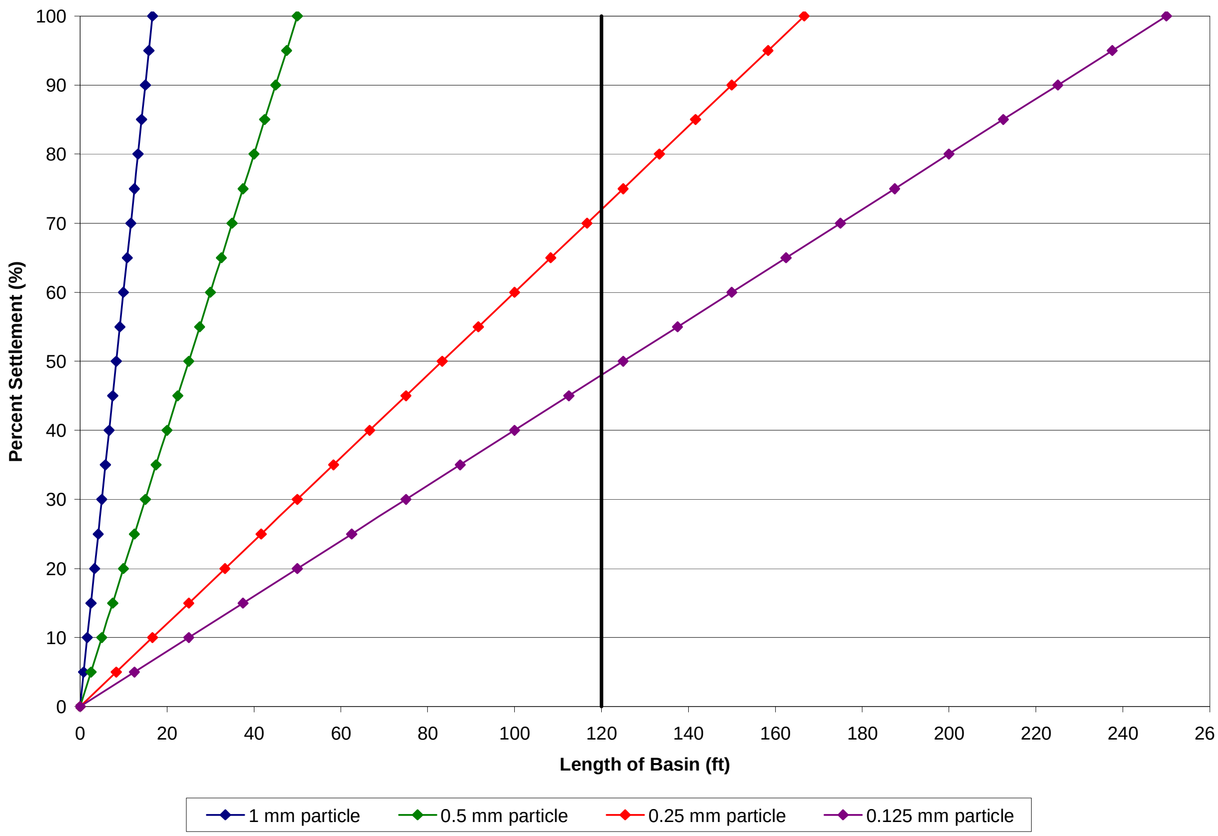


Figure 2: Basin Length vs. Percent Settlement by Particle Size

From the analysis shown on Figure 2, a basin length of 120 feet was determined to be the optimum length for removing a reasonable percentage of the settleable solids. The minimum depth of the basin was fixed to ensure the flow would be sufficiently low enough to facilitate particle settlement. The sedimentation basin would be a concrete structure that would run the width of the pumping plant suction, with interior concrete walls to create separate sedimentation channels. The channels would divide the flow, and each channel would be capable of being independently isolated for maintenance. A channel length that would allow for maximum effective settlement and was at least 3 times the basin width was chosen to limit short circuiting of the flows. Each basin would be 120 feet long, 40 feet wide and approximately 50 feet deep. The typical velocity through each channel would be 0.48 feet per second (fps). The final depth of the basin was determined by the requirement for the top wall level to be at the project flood protection level.

On average, it is estimated that approximately 14% of the total suspended solids entering the basin can be settled (Table 5).

As each intake pumping plant would include a standby pump, the channels would discharge into a common transition channel to ensure that any pump could operate, regardless of which sedimentation channel was isolated.

Table 5: Settleable Sediment (By Size)

	Settleable and Non-Settleable (100% of All Solids)	Settleable by Plain Sedimentation in 120-Foot-Long Basin
	% Total	% Total
2 mm> D_p >1 mm	0.80	0.80 (at 100%)
1 mm> D_p >0.50 mm	3.68	3.68 (at 100%)
0.50 mm> D_p >0.25 mm	7.68	7.68 (at 100%)
0.25 mm> D_p >0.125 mm	2.08	1.50 (at 72%)
0.125 mm> D_p >0.063 mm	0.64	0.31 (at 48%)
Total	14.88%	13.97%

D_p = particle diameter
mm = millimeter

The channel inlet and channel sedimentation zone would be separated by a fixed weir and baffle wall to distribute the flow uniformly into the inlet zone and reduce streaming at high river levels.

Each channel would have adjustable weirs at the downstream end discharging into the common outlet channel. Discharge level from the sedimentation basin would be controlled by the river water level to reduce streaming of flow at high river levels.

1.4 Solids Handling

The sedimentation basins located upstream of the intake pumps would remove the grit size particles that are smaller in size than the openings in the fish screens but larger than the very fine silt that is predominantly present in the river water. With the intake operating at maximum capacity, the estimated average daily mass of solids that would be expected to settle in the basin for subsequent transfer to the lagoons would be approximately 133,000 pounds per day (lb/d). The solids would mainly be comprised of medium-fine sand of between 0.25 to 1 mm in size. Each sedimentation channel within the basin would have a mechanical sediment removal system (Figure 3). The system would collect and siphon off settled solids into a solids pumping station at the downstream end of the basin. Solids would be pumped by positive displacement/progressive cavity pumps to storage lagoons for further settling and disposal. By maximizing the suction lift from the pumps, the dry well of the pumping station could be constructed at a shallower depth than the basins themselves.

The depth of the dry well would require forced ventilation linked to a timer at the access. The main pump starters would be located on a mezzanine floor at the top of the well, with local control panels and emergency stops at the bottom. Access into the bottom of the well would be via a stairway with landings at every 12-foot change in elevation. A permanently-plumbed sump pump would be installed for floor drainage.

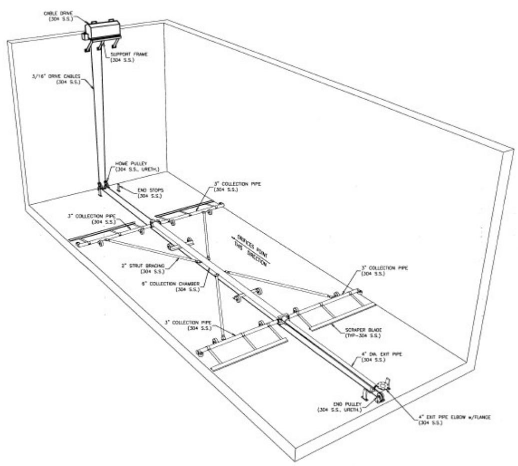


Figure 3: Mechanical Sediment Removal System

A mass balance approach was adopted for the calculation of the required lagoon size, using monthly suspended solids data (Figure 3). This allowed the lagoons to be sized based on measured data (i.e., the weight of suspended solids in the river water throughout the year). For the purposes of preliminary design, calculations were done on a worst case basis, by considering the throughput of the intakes for the Isolated Conveyance Facility (ICF) East Alignment: a maximum flow of 3,000 cubic feet per second (cfs). The assumed data, such as the throughput of the intake pumping plant and the volume of water transferred during solids pumping, will be refined as the design progresses to optimize the size and operation of the lagoons.

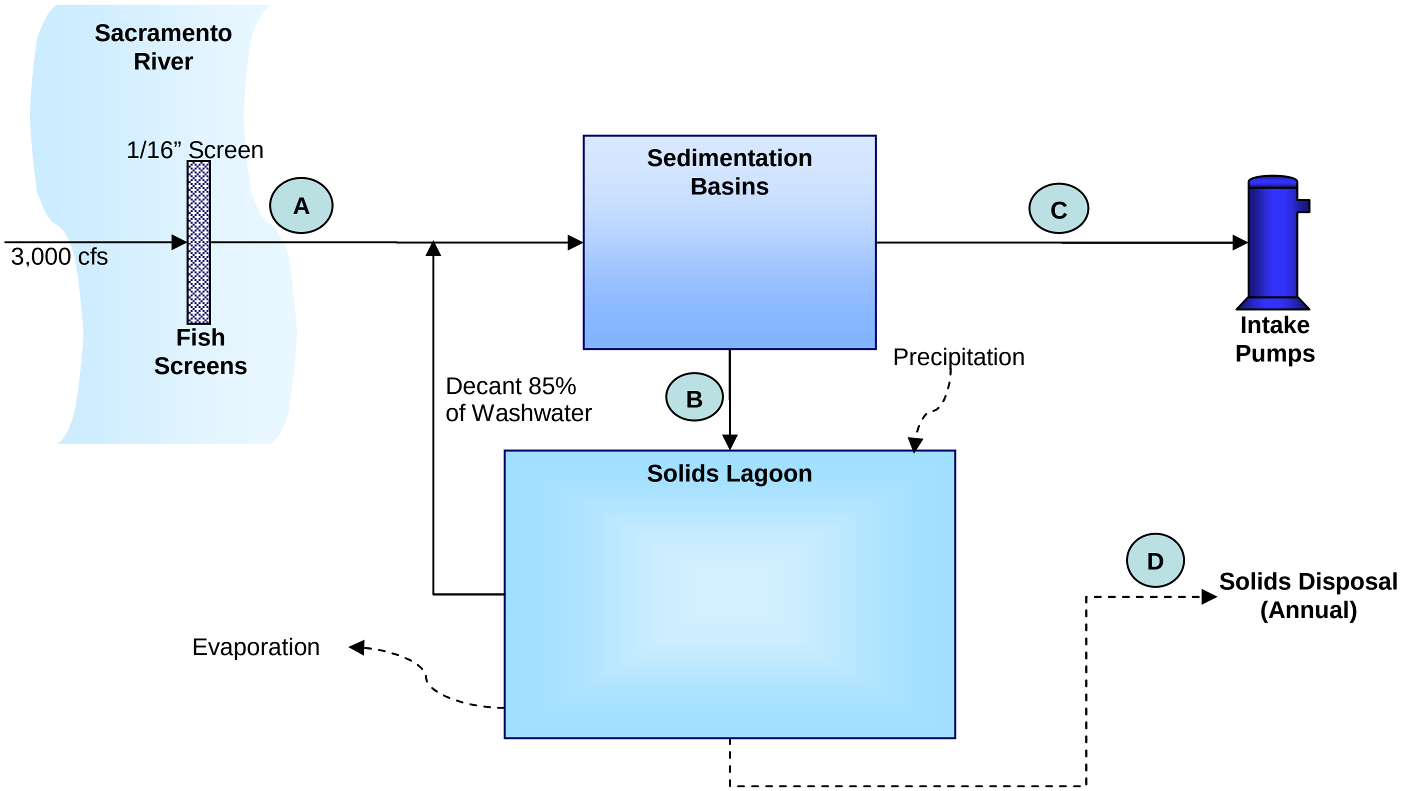
With the intake operating at maximum throughput, the average daily mass of solids expected to settle in the basin for subsequent transfer to the lagoons is approximately 133,000 lb/d. In the worst month, the daily mass of solids would be expected to peak at approximately 253,000 lb/d.

After a year, the accumulated volume of solids would notionally be 486,000 cubic feet (ft^3). Using a bed load of 30 cubic yards, this equates to 600 truckloads of solids for off-site disposal.

At this stage, it is proposed that 3 lagoons would be constructed. The 3 lagoons would be used in a rotating cycle of lead, lag, and standby operation with one basin filling, one settling, and the third being emptied of settled and dewatered solids. The lagoons would be 10 feet deep and would have sloped sides with a top width of 86 feet and a top length of 165 feet. They would be concrete lined to prevent seepage to the groundwater or adjacent river bed, and set at an elevation that would allow them to function during the design flood condition.

Clarified top water would be returned to the head of the sedimentation basin or to the common transition channel just upstream of the intake pumps.

Once the preferred operational regime of the solids collection and removal system is developed, the solids transfer pumping from the sedimentation basin, the inlet and decant arrangements at the lagoons, and the number of lagoons would be refined to enhance solids transfer and subsequent disposal.



	Flow	Grit ($D_p \geq 0.063$ mm)		
			Mass	
	cfs	%	Median	Max
			lb/d	lb/d
A	3,000.00	98	145,714	269,543
B	1.55	92	136,752	252,965
C	2,999.77	6	8,962	16,578
D	0.00	92	136,752	252,965

DHCCP Washington Team	Project No: 29624 ICF – EAST OPTION	SOLIDS MASS	Figure 4

Figure 4: Solid Mass

1.5 Fish Salvage Facilities

It should be noted that the proposed San Joaquin River Tunnel Fish Facility and Victoria Canal Fish Salvage Facility would also have sedimentation channels, as part of the diversion structure arrangements. The maximum throughput at these locations would be 15,000 cfs each, and the channels would be longer than the sedimentation channels at the river intake pumping plants. In addition, the channels would only be protected by coarse screens, rather than the fish exclusion screens that would be present at the river intakes. The nature and volume of the solids anticipated to settle in the diversion channels of the fish salvage facilities would therefore be different to those at the river intake pumping plants. This would require different equipment for collecting and pumping the solids. However, a similar solids analysis and mass balance approach can be adopted to estimate the notional volume of solids that could settle within the diversion channels.

Table 6: Suspended Sediment Percent < 0.063 mm

Record Year	1994	1995	1996	1997	1998	1999	Summary
Median Suspended Sediment < 0.063 mm (%)	85	62	81	86	84	80	82
Average Suspended Sediment < 0.063 mm (%)	85.1	69.3	79.4	85.3	81.5	78.5	79.1

mm = millimeter

Table 7: Suspended Sediment Concentration Summary

Suspended Sediment Concentration (mg/L)	
Average Monthly Minimum	17
Average Monthly Maximum	1,090
95th Percentile	163
Five Year Average	80.1
Dry Season Average (May - September)	89.04
Wet Season Average (October - April)	76.85

mg/L = milligrams per liter

Assuming that a diversion facility is operating constantly at a maximum throughput of 15,000 cfs, the median daily mass of solids that would be expected to settle in the basin for subsequent transfer to the lagoons is approximately 906,000 lb/d. In the worst month, the daily mass of solids would be expected to peak at approximately 1,269,000 lb/d.

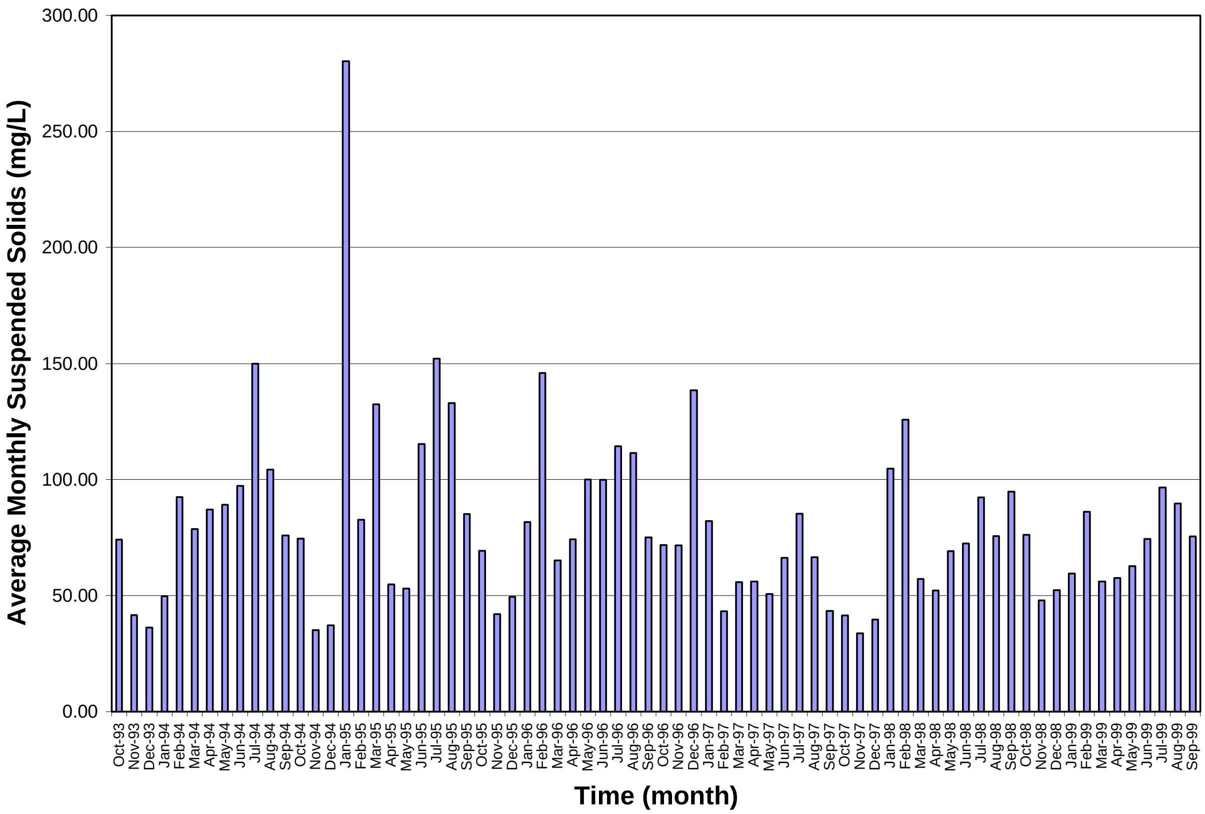


Figure 5: Average Monthly Suspended Sediment Concentration

Table 8: Bed Sediment Percent Passing Sieve

Record Year	<0.062 mm	<0.125 mm	<0.25 mm	<0.5 mm	<1 mm	<2 mm	<4 mm	<8 mm	< 16 mm
1994	--	2	8	57	95	99	100	--	--
1995	--	23	32	63	93	97	98	99	100
1996	--	1	8	58	94	99	100	100	--
1997	--	42	56	68	92	98	99	99	100
1998	--	11	29	66	93	98	98	100	--
1999	--	1	3	43	89	98	100	100	--
Average	**--**	**13**	**23**	**59**	**93**	**98**	**99**	**100**	**100**
% Total	***--***	***23***	***36***	***33***	***6***	***1***	***1***	***0***	***0***

mm = millimeter

Table 9: Sediment Composition (By Size)

	Settleable Solids (21% of All Solids)	Non- Settleable Solids (79% of All Solids)	Settleable and Non-Settleable (100% of All Solids)	Total % that can settle by Plain Sedimentation Basin
$D_p \geq 4$ mm	1	0	0.21	21
4 mm> $D_p \geq 2$ mm	1	0	0.21	
2 mm> $D_p \geq 1$ mm	6	0	1.26	
1 mm> $D_p \geq 0.50$ mm	33	0	6.93	
0.50 mm> $D_p \geq 0.25$ mm	36	0	7.56	
0.25 mm> $D_p \geq 0.125$ mm*	23	0	4.83	

D_p = particle diameter
mm = millimeter

Settleable Solids = Bed Sed Summary %Total
Settleable and Non Settleable = (Settleable Solids)*(21%)*(Percent Settlement (from Sed Basin Sizing GP)+Non-Settleable Solids

21% comes from the total number of solids that are not suspended and 79% is the average suspended solids amount, as estimated from the USGS Sample data (USGS, 1993-1999)

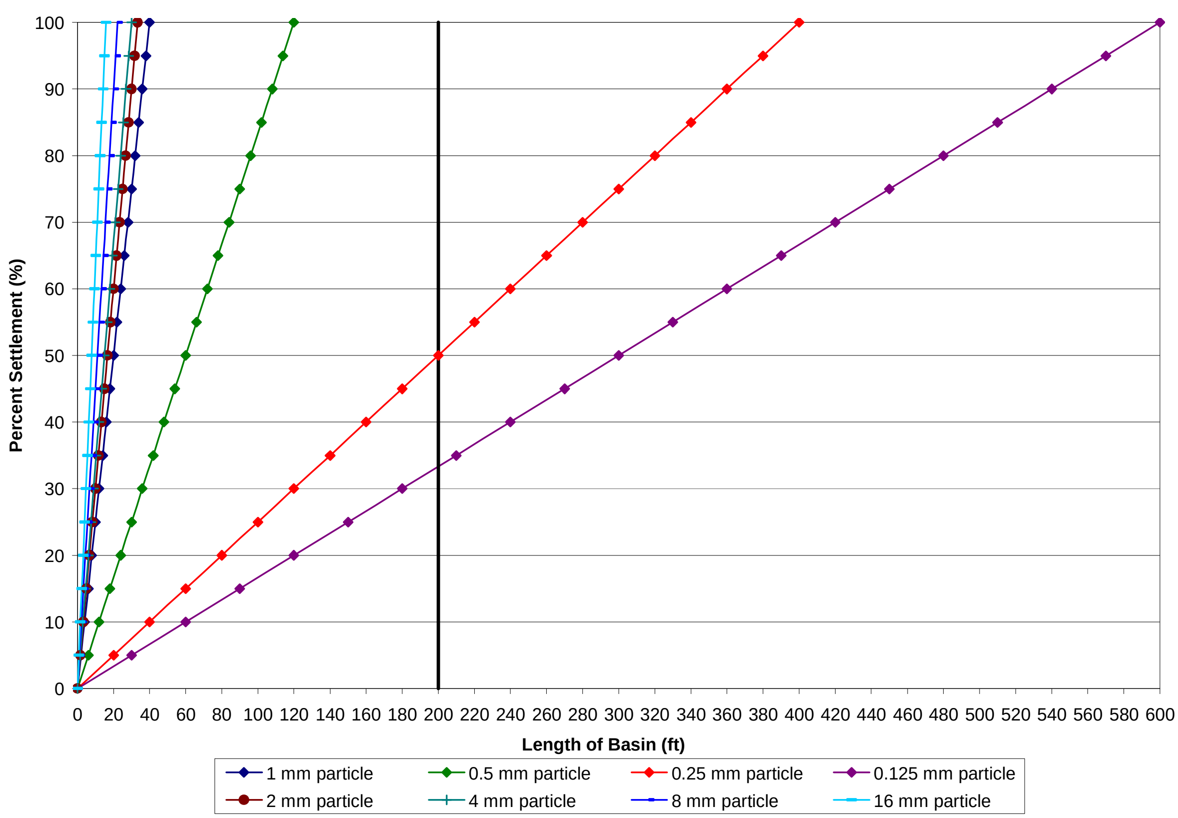


Figure 6: Basin Length vs. Percent Settlement by Particle Size

Table 10: Settleable Sediment (By Size)

	Settleable and Non-Settleable (100% of All Solids)	Settleable by Plain Sedimentation in 200-Foot-Long Basin
	% Total	% Total
D_p >4 mm	0.21	0.21 (at 100%)
4 mm> D_p >2 mm	0.21	0.21 (at 100%)
2 mm> D_p >1 mm	1.26	1.26 (at 100%)
1 mm> D_p >0.5 mm	6.93	6.93 (at 100%)
0.5> D_p >.25 mm	7.56	3.78 (at 50%)
0.25> D_p >.125 mm	4.83	1.59 (at 33%)
Total	21%	13.98%

D_p = particle diameter
mm = millimeter

References

Lindeburg, Michael, 2008. Civil Engineering Reference Manual: Eleventh Edition, p. 26-6, 35-1 – 35-8, A-13.

United States Geological Survey (USGS), 1997-2007. Daily Suspended Solids Concentration Data: 11447650 Sacramento River at Freeport.

USGS, 1996-1999 & 2005-2007. California Hydrologic Report: 11447650 Sacramento River at Freeport.

USGS, 1993-1999. California Hydrologic Report: 11303500 San Joaquin River at Vernalis, CA.

APPENDIX D

Pump Selection – Though-Delta Facility Option

1.0 PUMPING PLAN DESIGN

1.1 Pumping Plants Operation Criteria

Jones Pumping Plant is currently operated on an almost continuous basis mainly because it has no plant forebay. Banks Pumping Plant is operated on an on-peak/off-peak schedule. Banks Pumping Plant is operated to pump during the off-peak hours, and any additional pumping is to the on-peak hours in an attempt to minimize pumping during peak energy hours, typically between 5 P.M. to 10 P.M. The Department of Water Resources (DWR) will flatten out the pumping schedule to continuous pumping when real time conditions require a high export or weed loading.

The existing Banks and Jones Pumping Plants operate in a narrow band. It has been assumed that operation of new pumping plants would be to sustain pump operation in both Banks and Jones Pumping Plants; however, the operation of the intake pumps is influenced by the flows in the Sacramento River and tidal conditions. Therefore, during low-flow periods, water export would be curtailed. The amount and schedule of water export would be determined by the SWP/CVP Joint Operations Center. Local selection of pumps in operation and duration of pumping would be determined by the pumping plant operators based on direction by the SWP/CVP Joint Operations Center for the water export requirements.

To protect Delta smelt and salmon fries from being drawn into the pumps, pump operation must be controlled to limit the maximum velocity through the intakes to 0.2 foot per second (fps) to prevent impingement of Delta smelt in the screen openings.

Records show that the Sacramento River water surface elevation at the Freeport Gauging Station varies from elevation (EL) 27.3 to EL 0.0 (North American Vertical Datum of 1988 [NAVD88]). The proposed intake screens are located between EL -12 and EL 13. Figure 1 depicts the percentage of time that the river stage exceeds certain elevations. It shows that 10% of the time, the intake screens would be completely submerged. The pumping plants can operate at full capacity when the screens are completely submerged. For the remaining 90% of the time, the screens would be partially submerged and pumping plant operations should be adjusted to limit the velocity through intake screens to no greater than 0.2 fps.

1.2 Intake Pumping Plants Design Criteria

Intake pumping plants would be sized for a total firm pumping capacity of 4,000 cfs. Each intake pumping plant and its associated sedimentation basins would be sized for a firm capacity of 2,000 cfs. Pumping capacity could be varied by reducing the number of pumps on line and/or adjust pump operating speed. Variable frequency drives (VFDs) and flow meters would be required on all pumps to vary the pumping rate. The maximum turn down of pump speed is 50%, which corresponds to the 50% reduction in pumping rate and the 75% reduction in pressure.

Four intake pumps with 500 cfs capacity each were selected for each pumping plant to match the design capacity of the intake screens. One additional 500 cfs pump with a VFD would be installed to provide redundancy and improve reliability. Pumps would be installed in parallel and in vertical positions. All motors, VFDs, and electrical equipment would be installed above the 200-year flood protection level.

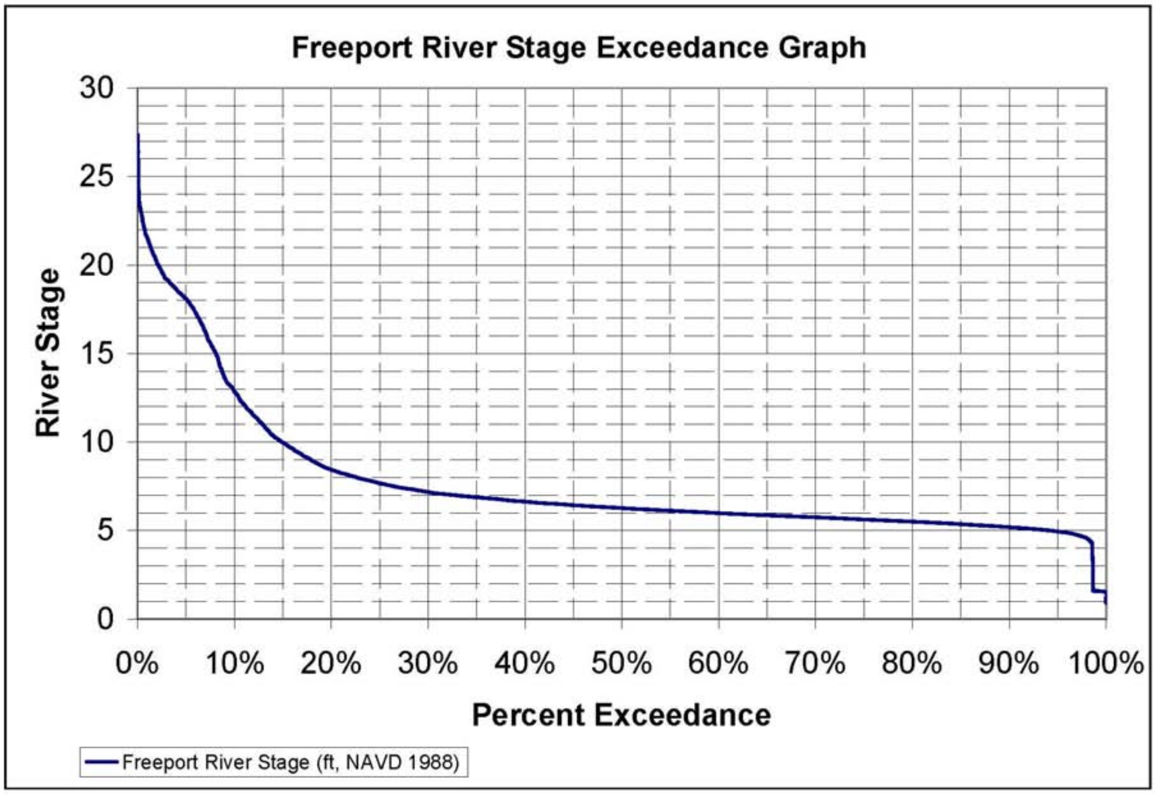


Figure 1: Sacramento River Freeport Stage Exceedance Graph

Discharge piping for each pump would include a hydraulically operated butterfly valve, a redundant motor operated butterfly valve, and a flow meter. Hydraulically operated butterfly valves would provide a check function; therefore, no separate check valves would be provided. All pump discharge piping would connect to a transition structure located outside the pumping plant then transition into two force mains which would connect to the new canal.

Table 1 summarizes the intake pumping plant capacities, number of pumps, pump design conditions, pump operating speed, and motor horsepower requirements. The total dynamic heads (TDHs) were calculated based on the assumptions that the design water surface elevation in the Sacramento River is at the top of screens, EL 13 (NAVD88), and the weir crest elevation in the transition structure is at EL 26 (NAVD88). Motor horsepower is significantly higher than the shaft power due to higher horsepower requirements at the shut-off conditions. It is anticipated that pumps would be started against closed valves which would put pumps starting against the shut-off head. The motor horsepower requirements listed are with reduced speed start and full speed start of intake pumps.

1.3 Intermediate Pumping Plant Design Criteria

The intermediate pumping plants for the San Joaquin River Option and for the Victoria Canal Fish Salvage Facility would be operated to overcome the head loss (energy loss) due to friction of the canal and flow restrictions from siphoning and tunnels. Intermediate pumping plants would include fifteen 1,000 cfs duty pumps and one 1,000 cfs spare pumps. This pump combination would meet the needs of the existing pumping plants and provide operational flexibility and reliability. VFDs would not be required on all pumps because the operations staff has the ability to adjust pump run time to meet the export requirement. However, as described in Article 3.2.2, VFD may be required in the event one pump operation is deemed necessary.

Pumps would be installed in parallel and in vertical positions. The pump discharge piping would include a hydraulically operated butterfly valve, a redundant motor operated butterfly valve, and a flow meter. The hydraulically operated butterfly valve would provide a check function, and therefore, no separate check valves would be provided. The pumps would be operating in parallel and pump discharge piping would be grouped into three groups and discharged into the tunnels. The details of connection at the tunnel entry have not been finalized at this point and pump TDH would need to be revisited during the final design stages.

Table 1 summarizes intermediate pumping plant capacities, number of pumps, pump design conditions, pump operating speed, and motor horsepower requirements. The total dynamic heads were calculated based on the design WATER SURFACE ELEVATION in the approaching canal of EL 15 (NAVD88), all pump discharge piping would be grouped into three groups which would transition into three tunnels. Motor horsepower is significantly higher than the shaft power due to higher horsepower requirements at the shut-off conditions. It is anticipated that pumps would be started against closed valves which would put pumps starting against the shut-off head. Table 1 shows the motor horsepower requirements with constant speed start and operation.

Table 1: TDF Option Pumping Plants Summary

	Pumping Plant Firm Capacity (cfs)	Number of Pumps	Duty Pump Capacity (cfs)	Spare Pump Capacity (cfs)	Pump TDH (feet)	Vertical Column Type Pumps: Pump Speed (rpm)	Vertical Column Type Pumps: Shaft Power at Design Point (bhp)	Vertical Column Type Pumps: Motor – HP (with Reduced Start/with Constant Speed Start)	Vertical Volute Type Pumps: Pump Speed (rpm)	Vertical Volute Type Pumps: Shaft Power at Design Point (bhp)	Vertical Volute Type Pumps: Motor – HP (with Constant Speed Drive)
Intake Pumping Plant No. 1	2,000	4 Duty + 1 Spare	500	500	21	208	1,350	2,000/3,500	160	1,370	2,000
Intake Pumping Plant No. 2	2,000	4 Duty + 1 Spare	500	500	21	208	1,350	2,000/3,500	160	1,370	2,000
Optional San Joaquin River Tunnel Intermediate Pumping Plant	15,000	15 Duty + 1 Spare	1,000	1,000	30	207	3,780	5,000/7,000	142	4,000	5,000
Victoria Canal Fish Salvage Facility Intermediate Pumping Plant	15,000	15 Duty + 1 Spare	1,000	1,000	30	207	3,780	5,000/7,000	142	4,000	5,000

bhp = brake horsepower
cfs = cubic feet per second
ft = feet
hp = horsepower
rpm = revolutions per minute
TDH = total dynamic head

2.0 PUMP SELECTION

2.1 Pump Type

Intake pumps exhibit high flows and low head characteristics and intermediate pumps exhibit high flows and medium head characteristics. Pumps that meet these requirements are generally in the category of axial flow pumps, also known as propeller pumps, and mixed flow pumps. Table 2 summarizes pumps recommended by various manufacturers. Vertical column mixed flow pumps and vertical mixed flow volute pumps could be used in either intake or intermediate pumping plants.

Table 2: Recommended Pump Types and Manufacturing Capabilities

Pump Capacity (cfs)	Location	TDH (ft)	ITT Flygt	Fairbanks Morse	Andritz	Patterson	Ebara
500	Intake Pumping Plant	31	VM, CV	VM	VM	VM	VM, CV
500	Intermediate Pumping Plant	65	VM, CV	NIP	VM	VM	VM, CV
1,000	Intermediate Pumping Plant	65	VM, CV	NIP	VM	VM	VM, CV

cfs = cubic feet per second
CV = Vertical Mixed Flow Volute Pumps
ft = feet
NIP = no information provide
TDH = total dynamic head
VM = Vertical Column Mixed Flow Pumps

2.1.1 Vertical Mixed Flow Volute Pumps

Vertical mixed flow volute pumps are manufactured with vertical volute casing and are typically installed in dry pits with a suction pipe connected to the wet well or with a formed suction inlet. Pump impellers can be withdrawn from the pump casing without disconnecting any piping. Volute casing can also be embedded in the concrete structure to simplify the pump support and reduce vibration and noise. Figure 2 shows a typical vertical mixed flow volute pump with casing exposed in the air. Vertical mixed flow volute pumps have been installed at both Banks and Jones Pumping Plants, and the operation and maintenance (O&M) staff are familiar with the pump operation procedures and maintenance requirements.

Volute pumps can be started with the impeller in either the dry or wet mode. A compressed air system would be required to depress the water to a level below the impellers when started in the dry mode. Starting the pump in the dry mode reduces the inrush current and the power load. When the pumps are started in the wet mode, the pump casing would be filled with water, resulting in a high inrush current and high power requirements. One way to reduce the inrush current and starting power load is to start the pump at a reduced speed or at a lower voltage.

Figure 2: Vertical Mixed Flow volute Pump

2.1.2 Vertical Column Mixed Flow Pumps

Vertical column mixed flow pumps are manufactured for wet well installation with the pump discharge elbow located either above or below the motor mounting floor. Pump line-shafts are located inside pump columns and could be either the open type or the enclosed type. Figure 3 depicts a typical vertical column mixed flow pump installation. Impeller submergence is required to prevent cavitation. Vertical column mixed flow pumps are typically started in the wet mode.

Figure 3: Vertical Column Mixed Flow Pumps

2.1.3 Vertical Column Axial Flow Pumps

Vertical column axial flow pumps are manufactured for wet well installation with the pump discharge elbow located either above or below the motor mounting floor. Pump line shafts are located inside pump columns and could be either the open type or the enclosed type. Figure 4 depicts a typical vertical axial flow pump. Propeller submergence is required to prevent cavitation. Vertical axial flow pumps are typically started in the wet mode.

Figure 4: Vertical Axial Flow Pump

2.1.4 Vertical Mixed Flow Pumps with Concrete Volute

A vertical mixed flow pump with concrete volute is a variation of volute pumps and has been used to convey large capacity flows for water intake applications. It is identical to a vertical mixed flow pump with the exception that cast-in-place concrete volute is used in lieu of metal volute. Figures 5 and 6 illustrate typical concrete volute construction.

Figure 5: Forming Volute

Figure 6: Forming Volute

Concrete volute pumps could reduce pump manufacturing time as the volute could be constructed on site while pumps are being fabricated in the shop. Similar to metal casing mixed flow pumps, pump impellers, shaft, bearings, wear rings, and shaft seals are all removable without disconnecting piping. However, because the concrete volute is constructed in the field, the pumps cannot be tested in the factory as one complete package.

2.2 Pump Selections

2.2.1 Intake Pumps

As shown in Table 2, pump manufacturers recommended vertical column mixed flow pumps and vertical mixed flow volute pumps for the intake pumping applications. Figure 7 represents a family system head curve and a pump curve of pump manufacturer ITT Flygt's vertical column mixed flow pump and Figure 8 represents a family system head curve and a pump curve of their vertical mixed flow volute pump. The system curve was developed based on current intake layouts, location, connection piping, and canal configuration and sizes and is subject to change.

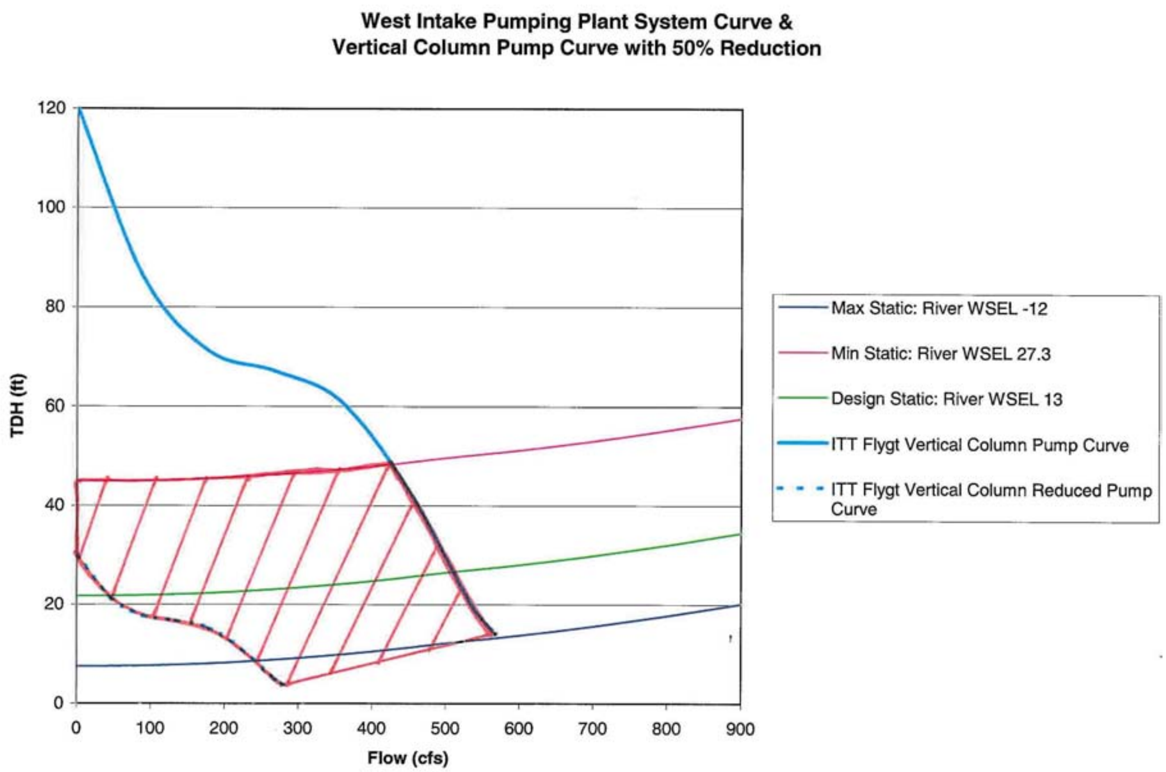


Figure 7: Intake Pumping Plant Vertical Column Mixed Flow Pump Operating Envelope

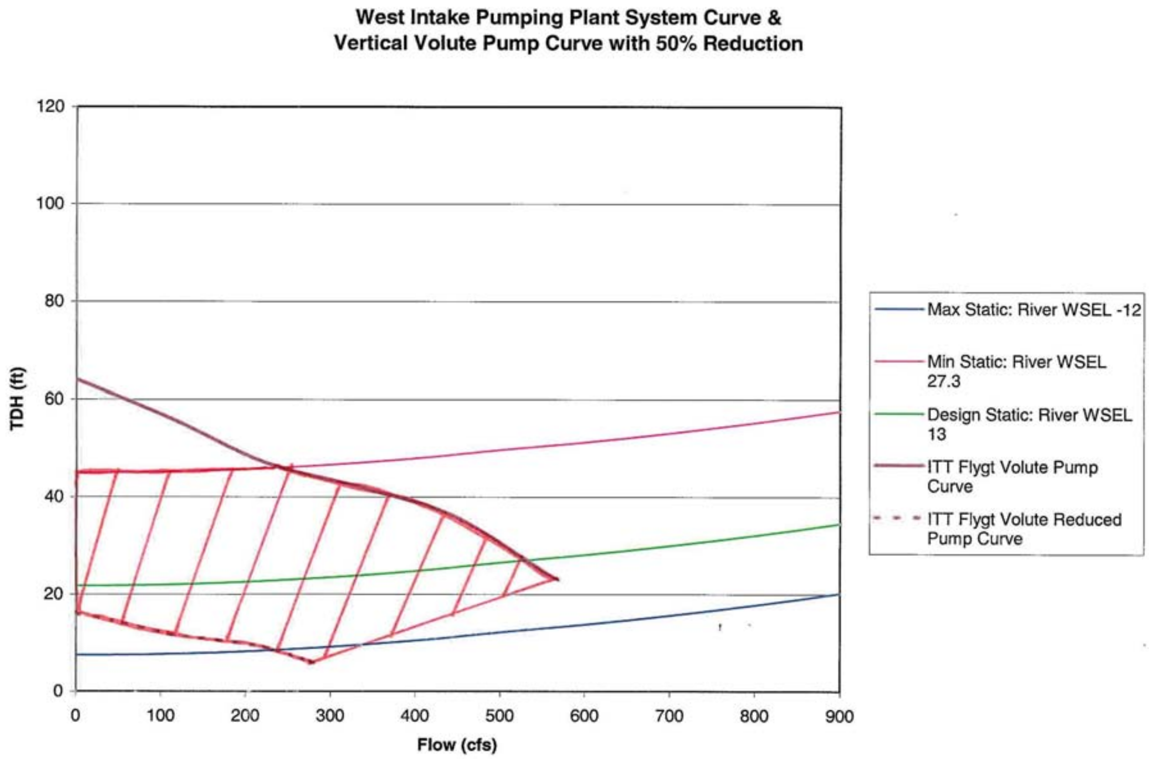


Figure 8: Intake Pumping Plant Vertical Mixed Flow Volute Pump Operating Envelope

Both types of pumps meet the design requirement of 500 cfs (224,400 gallons per minute [gpm]) at the design static head and with four pumps operating would meet the 2,000 cfs (897,700 gpm) design capacity. The design static head represents the condition when the river WATER SURFACE ELEVATION is at the top of the intake screens, EL 13 (NAVD88). When the water surface elevation is at the bottom of the screens, EL 12, represented by the maximum static head curve, the intake screens would be fully exposed, and water export from the Sacramento River would be stopped. The number of pumps in operation and their speeds would be adjusted to meet both the velocity and export constraints. During a high water level condition, represented by the minimum static head curve, the output from the vertical column mixed flow pumps would exceed the maximum pumping rate, which could result in a higher velocity through the intake screens than what is permitted. To avoid this problem, the number of pumps in operation and/or pump operating speed would be adjusted to meet design flows. The pump curve of the vertical mixed flow volute pump does not extend to the minimum static head curve, and pumps operating under this condition would have a cavitation problem.

One concern is that the net positive suction head requirement (NPSHR) for vertical column mixed flow pump rises sharply when flow rate exceeds the design flow, as shown on Figure 9. Figure 10 shows a milder rise in NPSHR on the vertical mixed flow volute pump. A higher NPSHR would require greater impeller submergence and could increase the depth of pump sump.

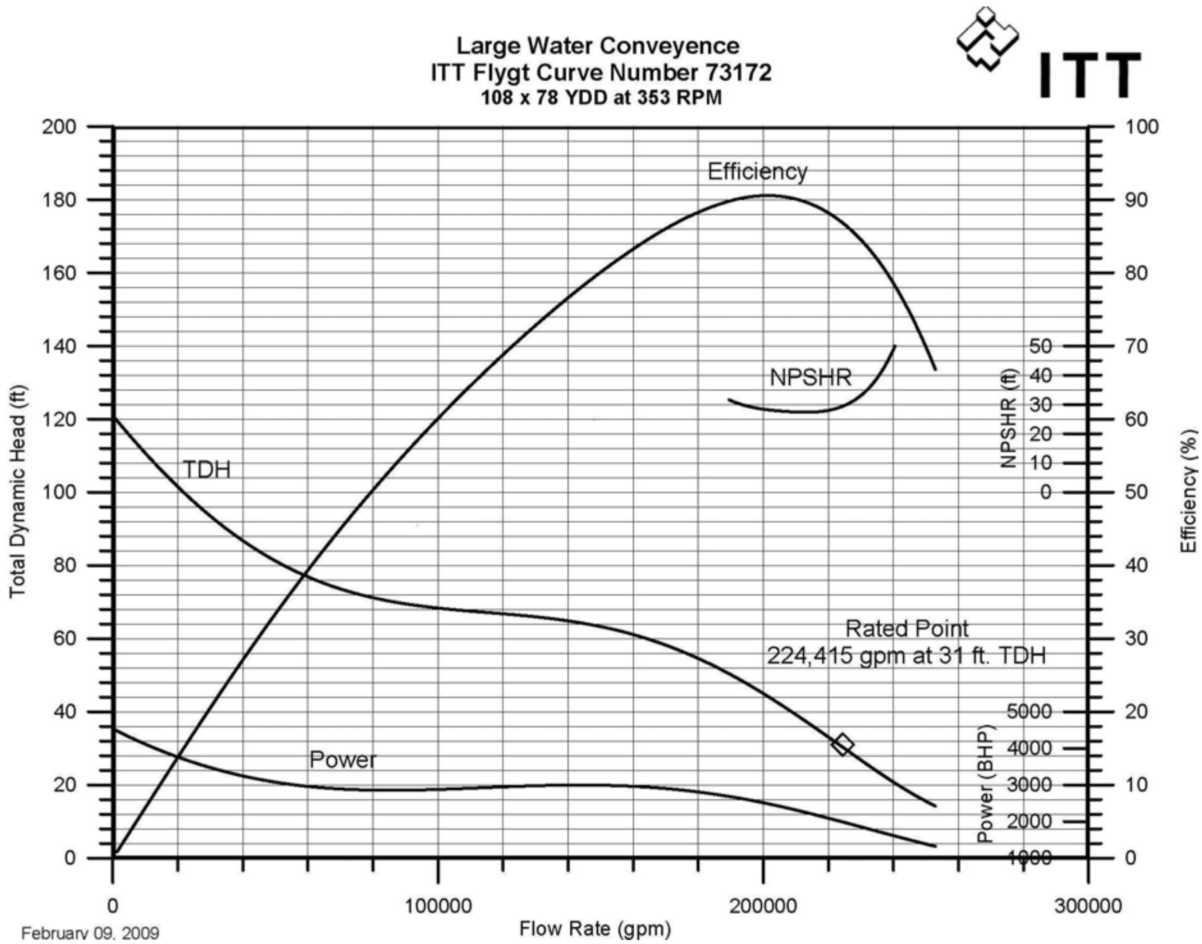


Figure 9: Vertical Column Mixed Flow Pump Curve

2.2.2 Intermediate Pumps

Figure 11 represents the system head curve and pump curves of five ITT Flygt vertical column mixed flow pumps in each group (three groups total) operating in parallel. Total pump output with all five pumps operating in each group would meet the design flow of 5,000 cfs. Additional column pump information is provided on Figure 12. Figure 13 represents the system head curve and pump curves of five ITT Flygt vertical mixed flow volute pumps in each group operating in parallel to meet the design flow. Additional volute pump information is provided on Figure 14. The system curve was developed based on the current pumping plant location, layout, discharge piping, and siphon structure configuration and is subject to change.

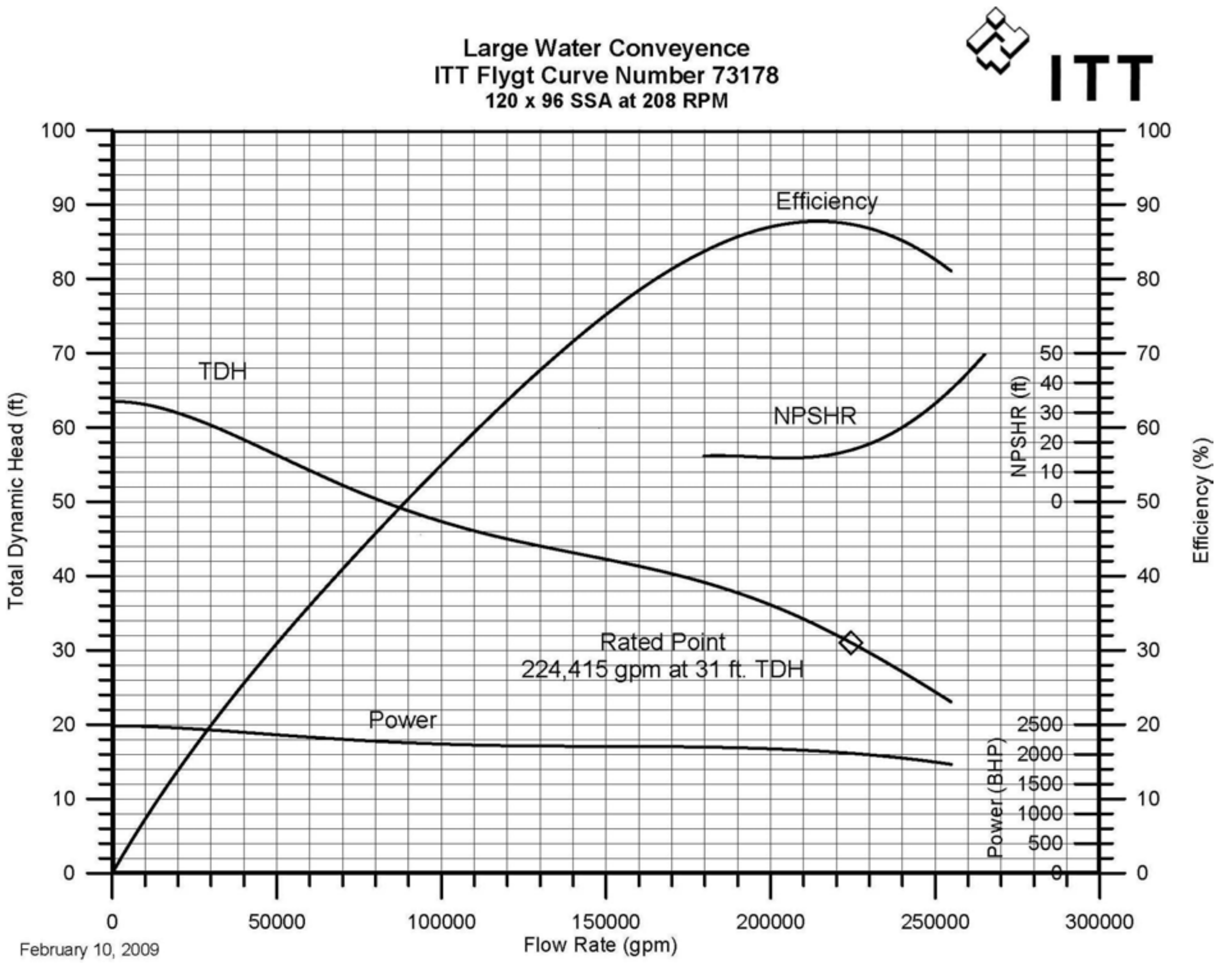


Figure 10 : Vertical Mixed Flow Volute Pump Curve

Figures 11 and 13 show that both types of pumps can meet the design requirement of 5,000 cfs (2,244,000 gpm) at 65 feet of head. The power requirements at the design point are about the same for both pumps except that vertical column mixed flow pumps have a higher motor horsepower requirement due to the higher shut-off head when compared to volute pumps. Higher shut off head would require larger motors and electrical systems for vertical column mixed flow pumps. Figures 11 and 13 also show that for the system to function properly, a minimum of two pumps must be in operation when pumps are called to operate. This generally is not a problem because the intermediate pumps are controlled to fill the Clifton Court Forebay (CCF) as fast as possible. However, if single pump operation is determined necessary, VFDs would be provided on pumps to adjust pump speed and flow rates.

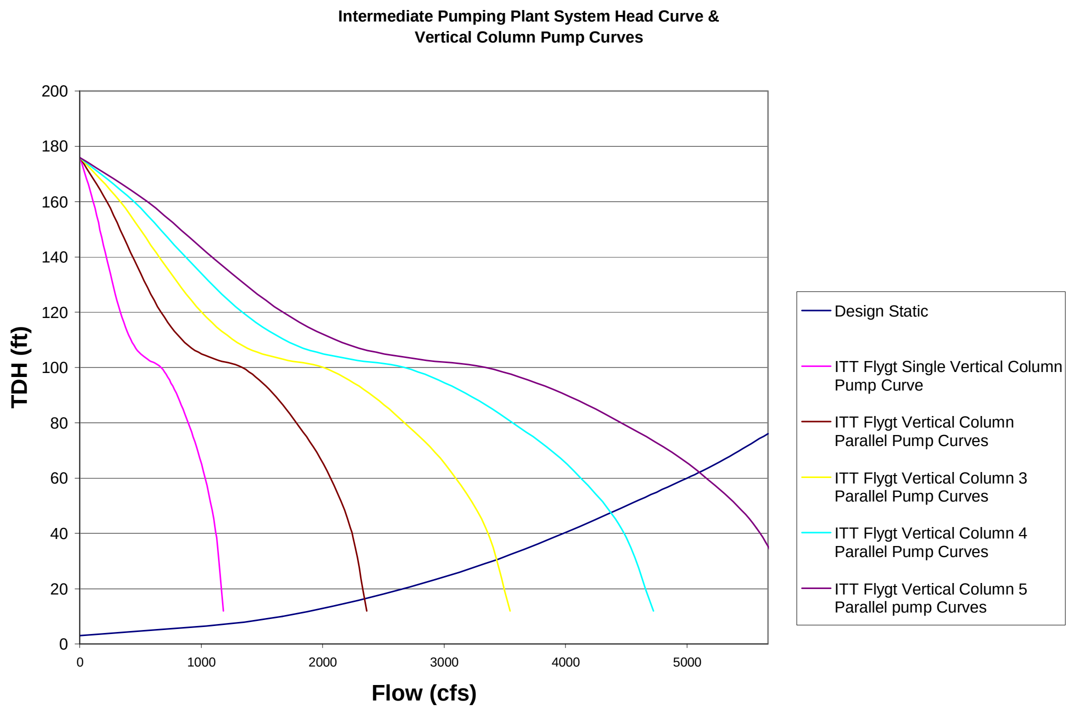


Figure 11: Intermediate Pumping Plant Vertical Column Mixed Flow Pump Curve and System Head Curve

2.3 Motors and VFDs

4,160-volt power is required for all intake pumps, and 6,900-volt power is required for all intermediate pumps. Both induction motors and synchronous motors are available in this voltage and the horsepower range; however, due to the low motor speed and high horsepower requirements, it is anticipated that the motors would be custom designed and manufactured. A motor of this size and voltage category is typically cooled using water. An auxiliary cooling water system would be provided at each pumping plant. Similar to the motors, the VFDs for 4,160-volt and 6,900-volt motors are also considered one-of-a-kind designs. The delivery time for the motors and VFDs will be investigated during future design.

Induction motors pull the voltage down when they are running, due to a high demand for current out of phase with the voltage -- so-called reactive power. To manage the voltage, it is necessary to add capacitors to the circuit to supply the out-of-phase component of the current. This would likely require several steps of capacitors and switches to energize and de-energize them, in addition to a control system.

Synchronous motors have unity power factor and generally have a higher efficiency than induction motors. Synchronous motors do not require nearly as much out-of-phase current to run, and by adjusting the voltage/current that is going into the rotor, the voltage on the motor and voltage on the power distribution system feeding the motor can be controlled. Synchronous motors behave better during electrical disturbances, which will inevitably happen.

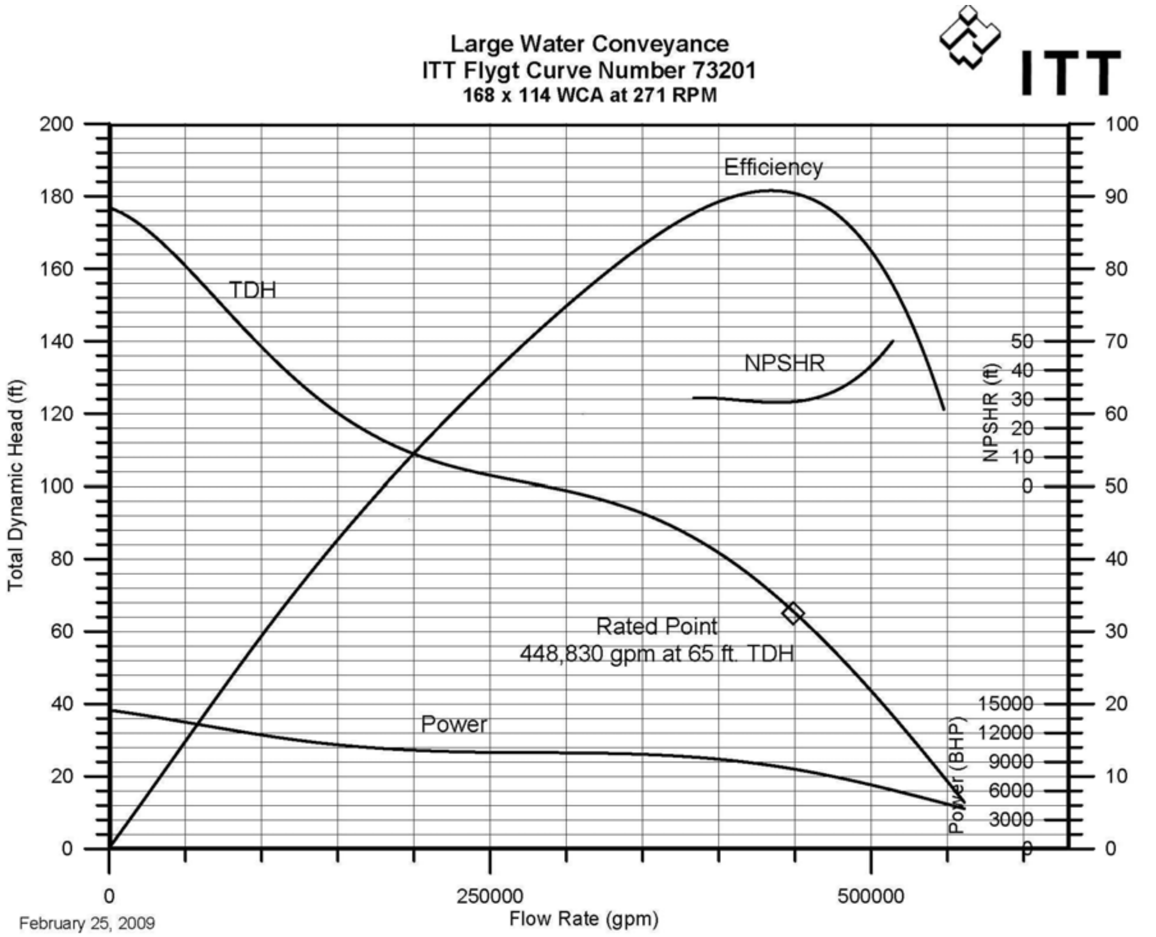


Figure 12: Intermediate Pumping Plant Vertical Column Mixed Flow Pump Curve

Both induction motors and synchronous motors could be used with or without VFDs. An induction motor with a VFD combination is preferred for the intake pumps since DWR has not utilized VFD controlled synchronous motors. The synchronous motor is selected for the intermediate pumps based on higher efficiency, unit power factor, and low inrush current. In addition, synchronous motors have been used at both Banks and Jones pumping plants, and the O&M staff are very familiar with motor O&M.

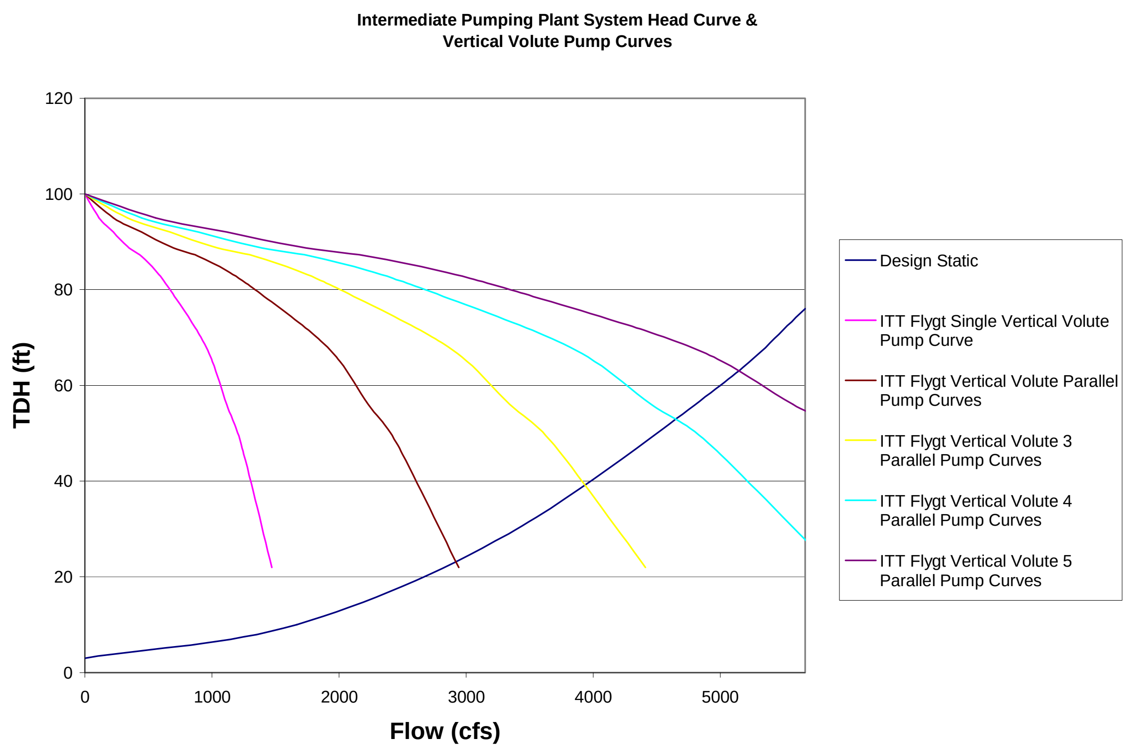


Figure 13: Intermediate Pumping Plant Vertical Mixed Flow Volute Pump Curve and System Head Curve

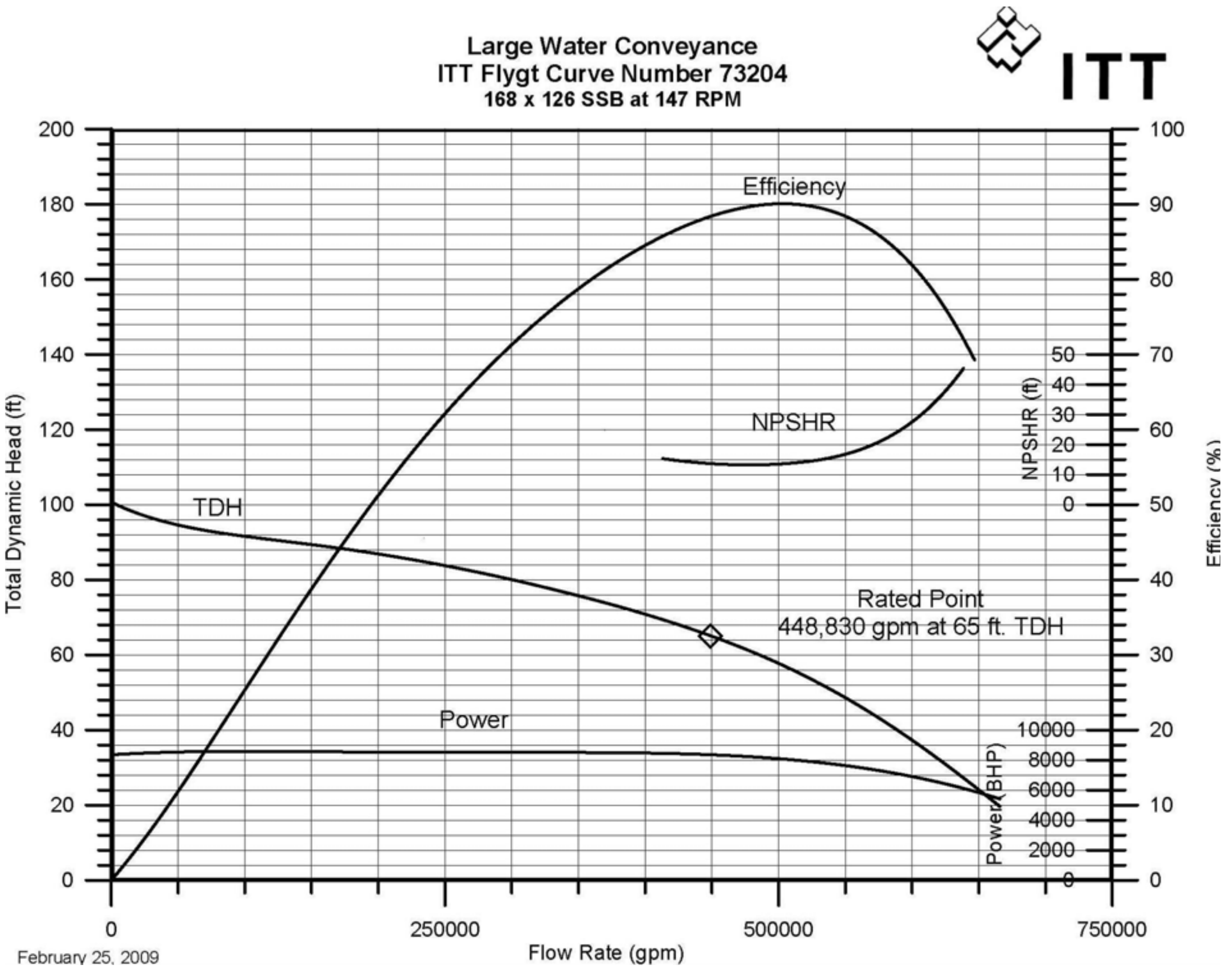


Figure 14: Intermediate Pumping Plant Vertical Mixed Flow Volute Pump Curve

3.0 CONCLUSION

The vertical mixed flow volute pump is eliminated from consideration at the intake pumping plants because the pump curve does not extend to the minimum static head curve, and a pump operating under this condition would have a cavitation problem. Vertical column mixed flow pumps could be used at the intake pumping plants.

Vertical column mixed flow pumps and vertical mixed flow volute pumps could be used at the intermediate pumping plant. Vertical column mixed flow pumps are generally slightly more efficient than the vertical mixed flow volute pumps. At the design point, the ITT Flygt vertical column mixed flow pump is 2% more efficient for the intermediate pumping applications. However, the vertical column mixed flow pump requires 30 feet of NPSHR at the design point for the intermediate pumping applications, compared to 15 feet for the volute pumps. The vertical column mixed flow pumps have higher shut off head than volute pumps, and this would require larger motors unless the pumps are starting at a reduced speed. Vertical mixed flow volute pumps are recommended for the intermediate pumping plant.

For vertical mixed flow volute pumps, concrete encasement of the pump volute is recommended in order to simplify the pump support and reduce vibration and noise. Concrete volute mixed flow pumps can shorten the manufacturing time, but they are not recommended since a completely assembled pumping system is required during factory testing.

A variable frequency drive (VFD) is recommended on all intake pumps for velocity control through the screens and for lowering the starting power requirement. Both induction motors and synchronous motors are compatible with VFD drives and may be used; however, induction motors are recommended for all intake pumps because O&M staff are more comfortable with using VFD controls on induction motors. Synchronous motors are recommended for intermediate pumps due to higher efficiency and O&M staff's familiarity with the equipment.

APPENDIX E

Pipe Materials

1.0 OVERVIEW

Materials and construction for the conveyance conduits between the Intake Pumping Plants and the new Canal are evaluated.

2.0 GENERAL DESCRIPTION OF CONVEYANCE SYSTEMS

Options to convey up to 15,000 cubic feet per second (cfs) of Sacramento River water from upstream of the Delta to the Banks and Jones Pumping Plants are under review. The Isolated Conveyance Facility (ICF)-West, ICF-East, and Dual Conveyance facilities each include five 3,000 cfs intakes and pumping plants on the Sacramento River that will pump through conveyance conduits to new canals for conveyance to the Banks and Jones Pumping Plants. The fourth option, Through Delta Facility (TDF), has two 2,000 cfs intake pumping plants that will pump through conveyance conduits to a new canal that feeds into the existing Delta canals.

3.0 PURPOSE AND SCOPE

To identify, evaluate, and recommend materials of construction for the conduits between the Intake Pumping Plants and the Canals.

A conduit size optimization evaluation indicated two conduits with a 200-square-foot open area per conduit (16 feet diameter if circular) is the optimum conduit size for 3,000 cfs conveyance capacity. For the 2,000 cfs two conduits with a 155-square-foot open area per conduit (14 feet diameter if circular) is the optimum conduit size.

4.0 LARGE CONVEYANCE INSTALLATION HISTORY

Since the optimum conveyance line size is very large compared with typical water transmission projects, a history of large diameter conveyance projects was first gathered to summarize industry experience in comparable sizes. Several large-scale projects have been implemented overseas, and the design and construction information may be applicable to this project. Pipe suppliers and manufacturers were also contacted to solicit their input with regard to large diameter conduit manufacturing and installation. Generally, a project of this scale will require special fabrication, transportation, and installation methods.

Table 1 summarizes the findings. The table shows that materials of construction successfully used for this size of pipe in the past includes CIP concrete, welded steel, and pre-stressed concrete cylinder pipe (AWWA C301).

Table 1: Large Conveyance Installation History

Pipe Type	Date	Size (ft)	Material Type	Length	Location
Steel Pipe					
Conveyance Pipe					
Central Arizona Project	1990s	21	Steel	1.7 miles	Arizona
Pacific Corp Swit 2 Project	--	11.5-16	Steel plates	--	--
Penstock					
Ghazi Barotha	--	5 @ 34.75	Steel-lined penstocks	100 meters	--
Gauley River Penstock	1990s	10-17	Steel	350 ft	West Virginia
Concrete Cylinder Pipe					
Pre-stressed concrete cylinder					
Central Arizona Project	1976	21	PCCP fabricated near site	4.5 miles	Arizona
San Onofre Project	1979	10-18	PCP non-cylinder	3.3 miles	California
MWD Castaic Project	1971	16.75	PCCP fabricated near site	5.9 miles	California
USBR Navajo Project	1975	15.75-17.5	PCCP fabricated near site	4.4 miles	New Mexico
Great man-made River	Phase 1 mid 1980s Phase 2 mid 1990s	2 @13	PCCP fabricated near site	490 & 510 miles	Libya
China South north	--	10	PCCP		China
Cast-in-Place Concrete Pipe					
Arch					
LACSD Joint Outfall	1973	12	Concrete	3.8 miles	California
Circular					
Tracy pump Plant d/c Lines	1947	15	Concrete	0.4 miles	California
Cast-in-Place Box Culvert					
Roundhill Reservoir	--	2 @ 19.6 x 19.6	Cast-in-place concrete	--	--
Batang Padang Hydro	--	40 ft x 40 ft	Cast-in-place concrete	200-300	--
KUMPP	Proposed	4 & 13' x 13'	Cast-in-place concrete	--	--
Other					
FRP					
Jubail Cooling water bypass	--	13.1	FRP	2 km	-
Tunnels					
Mangla	--	5 @ 26'	Steel grouted annular space	1800 ft each	--
Shanxi Wanjizsha Yellow River Diversion	--	18.3	--	--	China
Dokan dam	--	2 @ 36' and 39'	concrete lined tunnels 3'thck	--	--
Bombay outfalls under "sea"	--	11.4	--	7 km	India

FRP = fiberglass reinforced plastic
ft = feet
km = kilometer
KUMPP = Krishnapatnum Ultra Mega Power Project
LACSD = Los Angeles County Sanitary District
MWD = Metropolitan Water District of Southern California
PCCP = prestressed concrete cylinder pipe
Reclamation = United States Bureau of Reclamation

5.0 OPERATIONAL CONDITIONS AND REQUIREMENTS

For 3,000 cfs conveyance two 16-foot equivalent diameter lines are proposed. Similar material constraint conditions and economies would apply to the conveyance lines used for 2,000 cfs (two 14-foot equivalent diameter lines.)

5.1 Hydraulic Capacity and Pressure

5.1.1 Velocity

For each 3,000 cfs intake pumping plant the optimum conduit size provides a 200-square-foot open area per conduit and two conduits per intake pumping plant. With the planned pump capacity and transition structure configuration, the anticipated flow velocity ranges from 1.5 feet per second (fps) to 8.0 fps depending on the number of pumps in operation.

5.1.2 Pressure

The maximum design pressure between pumping plants and the canal is less than 30 pounds per square inch (psi). The water surface in the canal at the down stream end of the conveyance pipeline is above the conduit centerline at the canal and at the conveyance pipeline connection to the pumping plant transition structure. This will prevent the pipeline from draining during periods of no flow. The canal water surface elevation also sets the static pressure for the conveyance system. Static pressure is less than 15 psi.

5.2 Other Design Requirements

5.2.1 Depth of Cover

The minimum cover depth over the conduit will be 10 feet. This provides allowances for smaller size utility crossings, allows for some limited type of agricultural use over the top of the easement, and has allowances for erosion of the topsoil over time. The maximum cover depth will be at the pumping plant transitions structure, where berming around the pumping plant to raise it above flood level results in higher grade at the transition structure. In that area, the depth of cover may be up to 20 feet. Additional cover depths may be required to prevent conveyance line floatation. The conveyance line will be designed to withstand the earth load resulting from this depth of cover.

5.2.2 Depth-to-Groundwater

The depth-to-groundwater varies by intake and new canal location. Near the intake locations next to the Sacramento River the depth-to-groundwater fluctuates with river stage and local shallow aquifers.

Information available indicates the following general information:

Table 2: Depth-to-Groundwater

Option	General Depth-to-Groundwater
ICF-West	Varies from a few feet to 10 feet bgs
ICF-East and Dual Conveyance	Varies from a few feet to 6 to 8 feet bgs
Through Delta	Varies from a few feet to 6 to 8 feet bgs

bgs = below ground surface
ft = feet
ICF = Isolated Conveyance Facility

Because of the shallow groundwater elevations shown in Table 2 the pipeline material selected should be designed for continuous operation below the groundwater level. The presence of groundwater can impact pipeline corrosion rates and corrosion control design requirements depending on the conveyance material type. Also, with the conveyance lines installed below groundwater level, floatation must be prevented.

5.2.3 Length of Pipeline

Manufacturers that typically supply to water conveyance providers do not have fabrication facilities tooled to manufacture and handle pipeline with a diameter above 144 inches. Options to shop fabricated pipelines include field fabrication shops or fabricating in situ such as CIP circular or rectangular conduits. Field fabrication shops can require up to 50 acres for the fabrication facility but can be cost-effective if there is sufficient length of pipeline in a project.

The pipeline lengths between the intake pumping plants and the new canals for this project are relatively short and are estimated to total less than 11 miles per conveyance option.

5.2.4 Design Life

The pipeline conveyance facilities between the pumping plants and the new canals have a design life of 50 years. Material selection shall consider this design life requirement relative to corrosion control and maintenance.

5.2.5 Design Requirements Summary

The following summarizes conveyance conduit design requirements applicable to the materials and construction evaluation.

Table 3: Conveyance Conduit Materials Design Requirements

Criteria	Requirement
Flow Velocity Range, fps	1.5 to 8
Pressure, psi	< 30
Depth of Cover, ft soil	10 to 20
Groundwater	below groundwater
Design Life, yrs	50
Equivalent diameter, ft	14 to 18

fps = feet per second
ft = feet
psi = pounds per square inch
yrs = years

6.0 CONVEYANCE CONDUIT MATERIALS GENERAL

For pipelines 12 feet in diameter and smaller there are numerous pipeline material types commonly available. For line sizes above 12 feet in diameter, the range of material options is limited. Five conveyance conduit material/dimensional options were selected for evaluation based on the design criteria listed in Section 5.0, discussions with fabricators and industry experts, and the history of previous projects of this magnitude (discussed in Section 4). Conveyance conduit material option cross-sections are included on Figure 1. The cross-sections are conceptual and are suitable for general evaluations. Additional design refinements will be completed during the preliminary design phase of the project. The five material options selected for evaluation are:

- Steel pipe (AWWA C200 and plate welded)
- Concrete cylinder and concrete non-cylinder pipe (AWWA C300, C302, or C303)
- Circular CIP concrete pipe (ACI 301 and ACI 350-06)
- Rectangular CIP concrete box (ACI 301 and ACI 350-06)
- Arch CIP Concrete Conduit (ACI 301 and ACI 350-06)

Pre-stressed concrete pressure pipe with a steel cylinder (AWWA C301) has been installed in diameters up to 21 feet, but was not considered for this project due to a history of failure of the high tension wire from corrosion and the fact that pre-stressed concrete pressure pipe with steel cylinder (AWWA C301) is not currently accepted by the California Department of Water Resources (DWR).

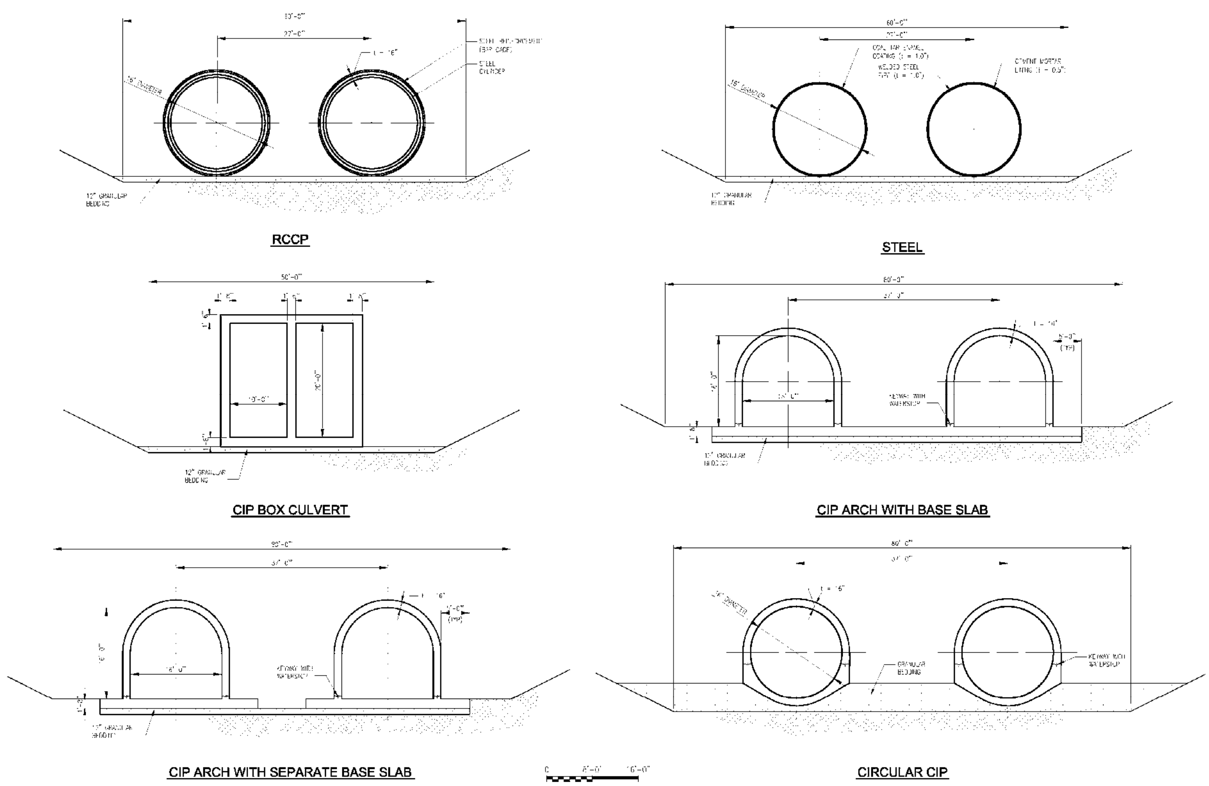


Figure 1 – Conduit Material Options

7.0 CONSTRUCTABILITY

As a result of the large diameter and pipeline weights, the steel pipe and concrete cylinder pipeline material options will require unique fabrication and installation techniques. Heavy lifting equipment for both factory and field fabricated options will be required, both for fabrication and installation. Manufacturers of steel pipe and concrete cylinder pipe indicate their factory fabrication tooling is not designed for pipe sized over 12 feet in diameter. New fabrication equipment may need to be developed and built specifically for this project. In addition there are challenges associated with transport of the pipe from the factory to the field.

Field fabricated pipe has constructability issues including costs for installation and operation of the temporary field fabrication equipment. Space requirements for the fabrication site must also be considered.

7.1 Transport Considerations

In the case of pipeline shop-fabricated off site then shipped to the site there are three main options for shipping:

- Boat or barge (along the Sacramento River)
- Railway
- Truck

The first two options will require transport by truck from the river or railway station. Therefore, roadway transport requirements become a critical consideration for moving fabricated pipe to the site. California Department of Transportation (Caltrans) regulations for dimensions and weight govern requirements for use of public roads to transport pipe (steel or concrete). Regarding established maximums, Caltrans states:

“In the instance where a load exceeds 14’0” in width, 135’0” in overall length, over permit weight ranges or requires multiple width hauling equipment, the transporter, owner or manufacturer may request a variance to allow movement of the vehicle and/or load which exceeds these limits.”

The key submissions required to obtain a permit for a variance are:

- Proof that no other mode of transportation is reasonably available. This includes letters from railroad and/or barge companies for verification. A letter from the railroad is required only when the load is less than 16 feet 0 inches wide. A letter from the barge company will not be required when ports are not reasonably available for both points of origin and destination within the state.
- Scale drawings and/or photographs to establish that the load will be transported in the smallest size possible.

- Certification by the manufacturer/designer that the critical nature or technical or structural requirements prohibit field fabrication of small component pieces. Economics or fabrication ease is insufficient justification without a bona fide economic comparison furnished by the manufacturer/designer indicating the range of total costs for the various options of fabrication and transportation.

The maximums for weight and height are dependent on the delivery route to be taken. Weight limits are dependent on the bridge loading capacities along the route, but are generally limited to a load weight of 40,000 pounds (lbs). Height limits are dependent on the vertical clearance of any overhead structure required for the vehicle and/or load. A 3-inch minimum clearance must be maintained at all times. Written route review may be required from the applicant for heights greater than 17 feet 0 inches.

For extreme weights and dimensions, a California Highway Patrol (CHP) escort may be required. Table 4 summarizes the applicable conditions whereby a CHP escort is required.

7.1.1 Transport Comparisons

To compare transport limitations and the number of truck trips required for each material option, some general pipeline assumptions were made. As discussed previously, the load limits (weight, height, width) are assumed based on general legal dimensions established by Caltrans. The limits may decrease or increase depending on the route taken and the permit approval process. The assumptions for each material option are summarized in Attachment A.

Table 4: CHP Escort Table

Route Classification	Width (ft)	Length (ft)	Height (ft)	Speed (mph)
Multi-lane freeway or express way with 12' lanes, 4' shoulders	17'+ if 3 or more lanes. 16'+ if 2 lanes	Unlimited	No maximum[1]	Below 30 mph below posted maximum speed limit
Substandard freeway or 2 lane road with 12' lanes and 0' to 4' shoulders	15'+	Unlimited on controlled access roadways. 135' for others	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. 11' lanes. 0 – any shoulder	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. 10' lanes. 0 – any shoulder	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. Less than 10' lanes	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit

The results of the pipeline transport comparison are summarized in Tables 4 and 5.

[1] Written route review may be required from the applicant for heights greater than 17 feet.

ft = feet
mph = miles per hour

Table 5: Shop Fabricated Steel and RCCP Transport

	Length of Section (ft)	Weight of Section (lbs)	Sections per Truck	Number of Truck Trips per Mile(1)
Steel (w/CML&C)	13	39,400	1	406
Steel (no CML&C)	20	41,500	1	264
RCCP	4	43,500	1	1,320

Note: (1) This is the number of trucks required to transport the pipe to the site. Additional truck trips may be required for other conveyance line requirements that are under investigation such as for cast-in-place concrete anchorage blocks required to prevent pipeline floatation.

CML&C = cement mortar-lined and coated
ft = feet
lbs = pounds
RCCP = reinforced concrete cylinder pipe
yd^3 = cubic yard

Table 6: CIP Circular Pipe and CIP Box Culvert Transport

	Total Volume of Concrete per Mile (ft^3)	Number of Trucks per Mile (10 yd^3 per truck max) (1)
2 CIP Circular Pipes	766,716	2,840
2 CIP Arches with Common Base Slab	1,571,011	5,819
2 CIP Arches with Separate Base Slab	1,119,096	4,145
CIP Box Culvert (2 w/common wall)	925,613	3,428

Note: (1) This is the number of trucks required to transport the concrete material for the conveyance line construction to the site. Additional truck trips may be required for other conveyance line requirements that are under investigation such as for CIP concrete anchorage blocks required to prevent pipeline floatation.

CIP = cast-in-place
ft^3 = cubic feet
yd^3 = cubic yard

Given a 14- to 16-foot-diameter pipeline, the limiting transport factor for shop-fabricated steel or concrete cylinder pipe is the weight. For the shop-fabricated pipeline options (Table 5) the number of individual sections and resulting number of joints impacts the number of welds required which in turn impacts cost and installation time. Shop-fabricated steel pipe can be shipped in longer lengths, thereby resulting in fewer truck trips and also less field joints. As shown in Tables 5 and 6, steel pipeline that is fabricated in the shop, and then assembled into pipe sections and lined and coated in the field, would require the least amount of transport trucks compared to other options.

8.0 MATERIALS

8.1 Steel

Spirally formed, fusion welded steel pipe has been manufactured and used in the United States since the late 1940s. Steel pipe has a demonstrated 50-years-plus service history. Steel pipe has many desirable qualities, which include durability, strength, economy, and reliability. Shop-fabricated steel pipe up to about 144 inches in diameter has been used extensively. There is limited experience for shop-fabricated steel pipe above 12 feet in diameter. However, there are a number of large (>14 feet) penstock applications where rolled steel plates have been shop manufactured and shipped to the construction site in sections and welded on site to form the pipe. In addition, steel tank erectors have confirmed that it is feasible to weld rolled steel plates (shipped or fabricated on site) vertically on site with a jig system and then to rotate the cylinder section for installation. Figure 2 shows a 21-foot-diameter spiral formed, fusion welded, steel pipe in shop fabrication conditions.

8.1.1 Availability

Due to the volume of steel that would be required for this project, advanced coordination and purchase agreements with fabricators should be considered. It may be necessary to assemble multiple bid packages should it be determined that there aren't any fabricators capable of constructing the entire conveyance system. Steel pipe would most likely be fabricated into pipe on or near the constructions site(s). Several steel pipe manufacturers have fabrication facilities that could be used for this project:

- Ameron (Tracy, CA)
- Northwest Pipe (Adelanto, CA and Portland, OR)
- Schuff Steel (Phoenix, AZ)
- Chicago Bridge and Crane (Claremont, CA)
- Advance Tank and Construction Company Inc. (Wellington, CO)

Figure 2: Fabrication of 21-Foot-Diameter Steel Pipe by Schuff Steel Company

Figure 3 shows field fabrication of a steel cylinder from bent plates.

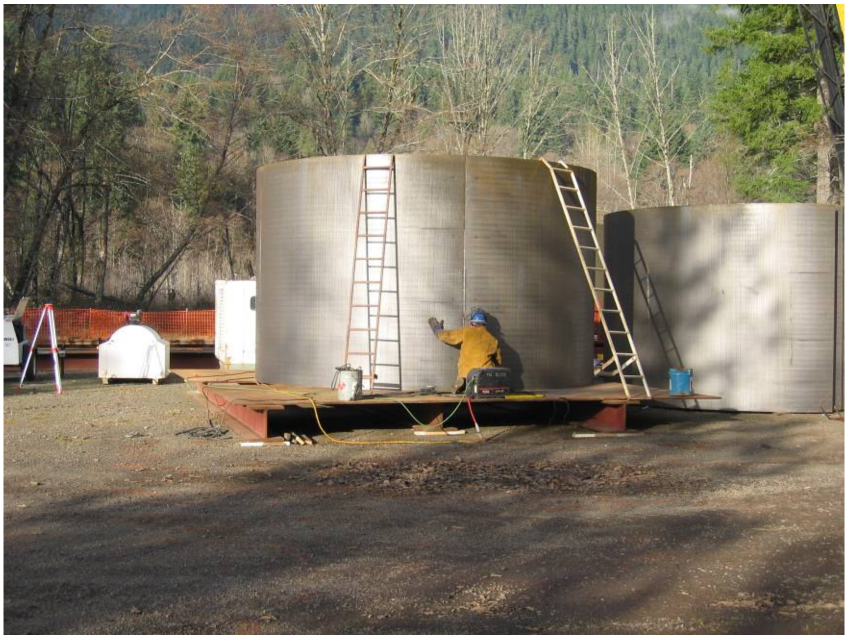


Figure 3: On-Site Fabrication of Bent Plate Steel Pipe

8.1.2 Transport/Installation

It is possible to ship steel pipe on trucks, but may not be economically feasible. The longest section of 16-foot-diameter bare steel pipe that can be shipped is 20 feet. The weight of the pipe and not the length is the controlling factor for shipment with truck transportation. Depending on the location of the fabrication shop, specialized pipe transport trucks may be used, such as the one shown on Figure 4.

Figure 4: Transportation of 21-Foot-Diameter Steel Pipe by Schuff Steel Company

Bare steel pipe can be supported with interior spider supports to prevent the pipeline from excessively deflecting during handling, shipment, and installation. If the steel pipe were cement mortar lined and coated in the shop, and then shipped, the increased weight would reduce the length of pipe per truck load from 20 feet to 13 feet. There would be concerns regarding pipeline deflection and potential damage to cement mortar lining and coating resulting from deflection during shipping. While shop-fabricated steel pipe may be a viable option, shipping it with cement mortar lining and coating does not appear to be practical.

The steel pipe transport equipment shown on Figure 4 was used both for transport from the fabrication shop as well as transporting the pipeline to the trench.

Rolled steel plates would be subject to the same trucking weight restrictions. For a 16-foot-diameter pipe, three sections (equivalent to a 20-foot-long pipe) can be trucked from the fabrication shop and assembled at the installation site. The rolled steel plates would be welded vertically on site with a jig system and then rotated for installation. A large crane could be used to move the steel cylinder into the trench.

Steel pipe is considered a flexible pipe system. To limit deflection of the pipeline it is important that the pipe embedment provide support around the pipe. Steel pipe will require more stringent and greater embedment requirements compared with the concrete and CIP conveyance options.

8.1.3 Maintenance

Much of the maintenance associated with steel pipe is related to preventing corrosion. Linings are used to protect the steel pipe from interior corrosion and erosion and coatings are used for corrosion protection on the exterior.

Linings are selected based on design flow velocities, handling and installation requirements and costs, service life requirements, and the physical and chemical characteristics of the water.

Cement mortar lining is relatively inexpensive, has been widely used, and has shown it can protect steel water pipelines under most operating conditions. Cement mortar lining can be used for continuous flow velocities up to about 15 to 20 fps. The expected velocities for the intake pumping plant conveyance are below this maximum allowable velocity. The water quality will need to be evaluated relative to sulfate and other constituents which could react with cement in the mortar. However, there are special cements and mortar mixes that can be used if needed.

Factory-installed cement mortar lining is a superior lining system; it will, however, add weight to the pipe and reduce the length of line that can be shipped per truck by about half of that for bare steel. It also reduces the allowable pipe deflection during handling, shipment and installation. If the steel pipeline is cement mortar lined prior to installation, the lining at the joints will have to be hand applied after the pipe is joined.

There are contractors who are able to apply a cement mortar lining to the pipe while it is in the trench. The field-applied cement mortar lining is more susceptible to variations in thickness and is not as dense as the lining applied centrifugally at the factory. A modified cement rich mortar lining mix should also be used for the field applied cement mortar lining. For a lining system applied in the field, a mobile applicator is constantly fed cement and centrifugally applies the cement mortar lining to the inside of the pipe. The in situ lining contractor estimated that with a 16-foot-diameter steel pipe and a 1/2-inch cement mortar lining, each machine could do 300 feet in a 10-hour day. The advantage of lining the pipe post shipping is that it significantly reduces the shipping weight of the pipe and reduces the likelihood for damage to the lining during shipping and installation. Once cement mortar lining is applied, the maximum pipeline deflection is 2 percent. This requires careful quality control of the pipe embedment compaction to prevent pipe deflection from soil loading. In situ cement mortar lining would be a viable option for the conveyance pipelines.

Cement mortar coating is usually applied in the shop by centrifugally spraying the pipe with cement mortar slurry to a thickness of at least 3/4 inch in conformance with American National Standards Institute (ANSI)/AWWA C205. If installed and handled correctly it has a superior service life history. Cement mortar coating will add weight to the pipe and reduce the length of line that can be shipped per truck by about half of that for bare steel. It also reduces the allowable pipe deflection during handling, shipment and installation. While it may be possible to cement mortar coat in the field by placing each bare steel section of pipe in the vertical position on a turn table and applying the mortar in a uniform fashion, handling and deflection considerations become a significant factor because of the low allowable deflection limits (less than 2%) with cement mortar coating.

Hot-applied coal tar enamel coating, ANSI/AWWA C203, has been used since the 1930s. To protect the coal tar enamel coating it is usually followed by a single layer of outerwrap consisting of glass-fiber felt, polyethylene-kraft paper, or polyethylene-elastomer laminate. Steel pipe with field-coated, hot-applied coal tar enamel coating (ANSI/AWWA C203) with the protective reinforced glass fiber inner and outer wrap would be a viable option for the proposed pipeline.

Regardless of the steel pipeline coating and lining, an impressed current cathodic protection system is recommended for longevity and reliability. With the high groundwater conditions, the cathodic protection system will require a higher current, for protection. In addition, it will require careful coordination with other utilities as stray currents can be an issue.

8.1.4 Joint Types

Steel pipe sections would be joined by welding. Several types of welded steel joints are available; however, lap welds are generally considered the most economical. At the expected operating pressures for this project, the lap welded joints can be fillet welded either internally or externally, or both. Butt welding of joints is more time-consuming because of the potential out-of-round pipe that makes welding tougher to control. Butt strap joints are more difficult to install, but are often used to install pipe assemblies or at changes of the pipe thickness. Butt strap joints should be considered for pipe closure assemblies.

If the steel pipeline is factory cement mortar lined, the lining at the joints will have to be hand applied after the pipe is joined.

8.1.5 Reliability

The advantage of steel pipe is that it is not typically prone to leakage. The concerns with steel pipe are exterior and interior corrosion, and flexure of the pipe. Correctly applying coating and lining, providing an impressed current cathodic protection system and providing steel pipe with adequate thickness would significantly reduce failures due to corrosion of the steel cylinder or joints.

8.1.6 Design Basis

Water industry standards for steel pipe are set forth in ANSI/AWWA C200. Design guidelines are set forth in AWWA M11. The assumptions in Table 8-1 were made for comparison purposes only.

Table 6: Steel Pipeline for Comparison Purposes

Inside Diameter after Lining (feet)	Wall Thickness (inches)	Lining Type	Coating type	Other
16	1.0	Field-installed cement mortar	Field-installed coal tar AWWA C203	Impressed current protection system

AWWA = American Water Works Association

8.1.7 Steel Pipe Summary

Hot-applied, coal tar, enamel-coated steel pipe that is cement mortar lined after installation is a viable option for the conveyance pipelines.

8.2 Concrete Cylinder and Concrete Pipe

The following three types of AWWA concrete pressure pipe are under consideration:

- Reinforced concrete cylinder pipe (AWWA C300)
- Reinforced concrete noncylinder pipe (AWWA C302)
- Bar-wrapped steel cylinder pipe (AWWA C303)

Design guidelines for these pipe types are included in AWWA Manual M9, Concrete Pressure Pipe.

Prior to acceptance of pretensioned concrete cylinder pipe (AWWA C303) in the 1960s, most of the concrete cylinder pipes used for pressure service above 55 psi was concrete cylinder pipe (AWWA C300). Both types have a thin steel cylinder embedded in concrete. The concrete cylinder pipe has mild steel reinforcing cages cast into the wall of the pipe and is suitable for pressures up to 250 psi. Concrete cylinder pipe is usually more expensive than bar-wrapped steel cylinder pipe.

Reinforced concrete noncylinder pipe (AWWA C302) has been used since the 1900s. It is used for pressure applications less than 55 psi and cover depths up to 20 feet are common. It is made with one or more reinforcing bar cages embedded in concrete. The concrete is placed by vertical or centrifugal casting method. Rubber gaskets joints have steel or concrete bell and spigot surfaces.

Bar-wrapped steel cylinder pipe (AWWA C303) has been manufactured and used extensively in the western United States since the 1960s. It consists of a concrete-lined and coated welded steel cylinder, helically wrapped with mild steel bar reinforcement under measured tension. It can be used working pressures up to 400 psi.

8.2.1 Availability

The manufacturers for concrete cylinder pipe (AWWA C300) and concrete noncylinder pipe (AWWA C302) indicate that their factory tooling is set up to fabricate 24- to 144-inch-diameter pipe. Larger diameter pipe would require special tooling and would result in more expensive pipe on a per unit basis. In addition, the weight of shop-fabricated concrete cylinder pipe would require manufacturing of pipe in shorter lengths to meet shipping weight limits. As a result, about three times as many truck trips would be required to transport concrete cylinder pipe to the project site as compared to bare steel pipe.

Manufacturers of AWWA C300 and C302 concrete cylinder pipe were asked about the feasibility of on-site fabrication of the pipe and indicated that on-site fabrication may be economically feasible depending on the pipeline lengths required for this project.

AWWA C303 is typically only provided in diameters 72 inches or less. Larger size bar-wrapped steel cylinder pipe is not considered a viable option by manufacturers who currently produce this material (Ameron or Northwest Pipe).

8.2.2 Transport/Installation

Figure 5 shows the on-site fabrication utilized by Ameron for their 21-foot-diameter AWWA C301 pipeline installed at the Central Arizona Project in the 1970s. Fabrication facilities required a 50-acre site. As discussed, AWWA C301 pre-stressed concrete cylinder pipe is not a recommended option for this project. However, similar area requirements would be needed for field fabricated C300 or C302 pipe.

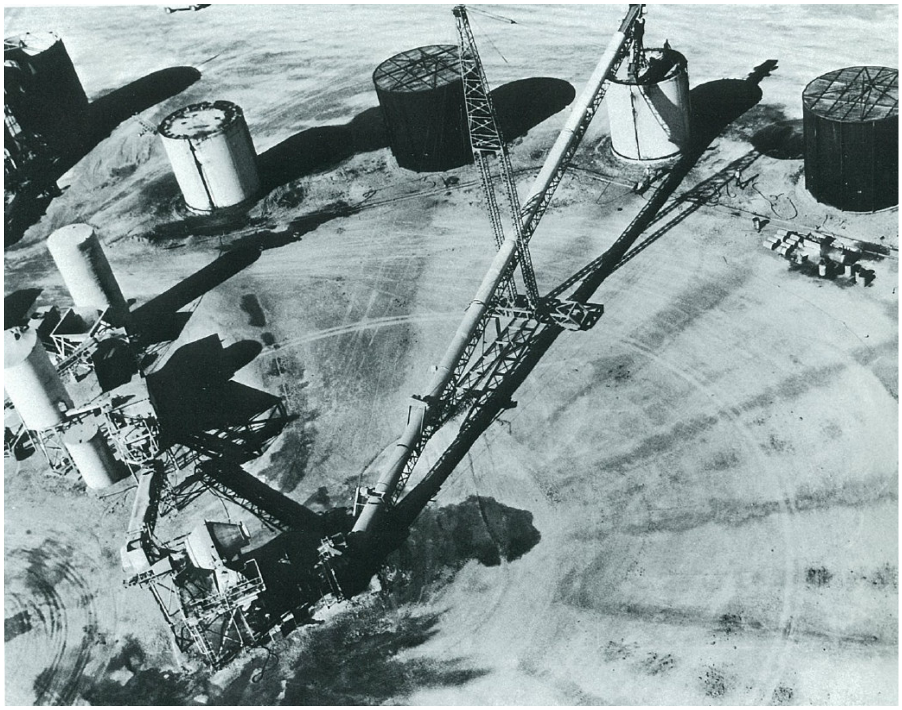


Figure 5: Ameron's On-Site Fabrication of Pre-Stressed Concrete Cylinder Pipe

Ameron has developed a process whereby they can pour, lift, and move sections of concrete cylinder pipe up to 21 feet in diameter. They use a radial stacker to pour concrete into several vertical pipe forms. The "Liftmobile" picks the pipe up and lays it horizontally on the ground. The "Pipemobile" then drives through the pipe section, hydraulically lifts it with the Pipemobile's mid-section, and drives into the trench and abuts to the previously laid pipe section (Figure 6).

In general, it is more economical to design rigid pipes (C300 and C302) to accommodate external loading with minimal bedding support than it is to require that the pipe be installed in highly compacted backfill. Bedding is required to avoid laying the pipe on hard, unyielding surfaces.

8.2.3 Maintenance

For most operating conditions, concrete pressure pipe is relatively maintenance free. The soil conditions and water quality will need to be evaluated relative to sulfate and other constituents which could react with the concrete. However, there are concrete mixes and cements that can be used if needed depending on any special water and soil chemistry conditions.

The field-applied cement mortar lining applied at the pipe joints after the pipe is joined is susceptible to thickness variations and can crack and spall, exposing the steel Carnegie type bell and spigot joints to the elements and increasing corrosion potential. The interior pipe joints should be periodically monitored for mortar cracking and damage and, if needed, should be repaired.

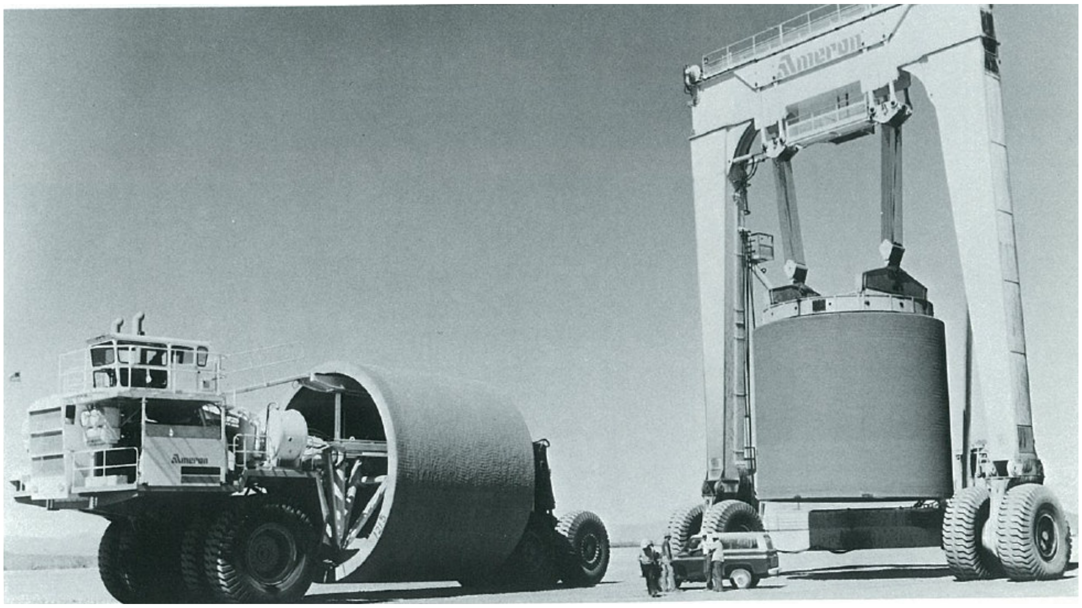


Figure 6: Ameron Used the "Pipemobile" and "Liftmobile" to Fabricate and Install 21-Foot-Diameter Pre-Stressed Concrete Cylinder Pipe On Site

8.2.4 Joint Types

Concrete cylinder pipe (AWWA C300), concrete noncylinder pipe (AWWA C302), and pre-stressed concrete cylinder pipe (AWWA C303) all can be provided with steel bell and spigot, Carnegie type joints. However, the joint connection for the concrete noncylinder pipe (AWWA C302) has less strength because it is welded to a steel collar piece and not welded to a steel cylinder.

Hydraulic Thrust can be resisted by the use of anchor blocks, by field welding adjacent pipe joints, or by a combination of both. Similar to steel pipe, field welding adjacent pipe joints instead of anchor blocks is a cost-effective option.

A mortar lining will have to be hand applied at the joints after the pipe is joined.

8.2.5 Reliability

Concrete pressure pipe is less susceptible to corrosion because the dense concrete layer at the interior and exterior of the line protects and passivates the steel reinforcing and cylinder. The hand applied cement mortar lining applied at the joints after the pipe is joined is more susceptible to thickness variations and can crack and spall exposing the steel Carnegie type bell and spigot joints to the elements and increasing corrosion potential.

8.2.6 Design Basis

Water industry standards for concrete pressure pipe are described in AWWA C300, AWWA C302 and AWWA C303. Design guidelines are set forth in AWWA M9. The assumptions in Table 8-3 were made for comparison purposes only.

Table 8-3: Concrete Pressure Pipeline for Comparison Purposes

ID After Lining (ft)	Pressure Limits AWWA C300 (psi)	Pressure Limits AWWA C302 (psi)	Pressure Limits AWWA C303[(1)] (psi)	C300 Maximum Earth Cover Depth (ft)	C302 Maximum Earth Cover Depth (ft)
16	250	55	400	>20	>20

Note: (1) Not available in diameters greater than 72 inches.

AWWA = American Water Works Association
ft = feet
ID = inside diameter
psi = pounds per square inch

8.2.7 Concrete Pressure Pipe Summary

As shown in Table 8-3, concrete cylinder pipe (AWWA C300) and reinforced concrete noncylinder pipe (AWWA C302) meet both the pressure and depth-of-cover requirements for this project. Pre-stressed concrete cylinder pipe (AWWA C303) cannot meet the optimum pipeline size requirements.

8.3 CIP Concrete Conduit

As a result of the potential fabrication and transportation costs and challenges with shop or field fabricated steel pipe and concrete pressure pipe, three configurations for CIP conduit are considered. They include:

- Circular CIP concrete pipe
- Rectangular CIP concrete box
- Arch CIP concrete conduit

There are two arch conduit configurations: arch conduits with a common base slab, and arch conduits without a common base slab. Arch conduits with a common base slab would require more concrete than the option of separate base slabs for each, but may provide constructability advantages with a common slab requiring less formwork and a larger surface for form placement and support.

8.3.1 Availability

The materials required for CIP concrete conveyance options are readily available and do not require specialized fabricators or equipment like other conduit options (steel pipe and concrete cylinder pipe).

8.3.2 Transport/Installation

For the CIP pipe options, the primary transportation concern is associated with the large number of concrete delivery trucks that may be required. Conceptual estimates for the number of 10-cubic-yard (yd^3) trucks required for a representative 1-mile length of conveyance facilities are included in Table 7-3. The volume of concrete utilized for CIP conveyance options will likely result in the contractor setting up a concrete batch plant at the project site rather than purchasing the concrete batches from outside suppliers. However, a suitable source for clean water will be required for the field batch plant.

Installation of CIP conveyance options would require significant formwork. In addition, the CIP options will require the trench to be open for about 2 months to provide time for formwork placement, concrete placement and curing, and stripping forms. This is two to three times more than what is required for the steel pipe or concrete pressure pipe options.

The CIP circular options would require specialized formwork. The rectangular shape culvert options would also require formwork, but it would be less complex due to common wall construction and flat shapes. The walls would be keyed into the base slab and a water stop will be provided at each joint.

For the arch options, the arches will be keyed into the base slab and a water stop would be provided at each joint. For the arch with separate base slab option, the base could be placed at the same time as the stem walls with water stops at the joints and the arch will be formed and placed after the walls are cured.

8.3.3 Maintenance

CIP concrete conveyance lines should be relatively maintenance free. The soil conditions and water quality will need to be evaluated relative to sulfate and other constituents which could react with the concrete. However, special concrete mixes and cements could be used, if needed, depending on any special water or soil chemistry conditions.

The interior pipe joints should be periodically monitored for cracking and damage and, if needed, should be repaired.

8.3.4 Joints

Wall to slab joints will be keyed where applicable and water stops will be provided at the joints to minimize leakage.

8.3.5 Reliability

Due to the number of construction joints and contraction/expansion joints, and the pressure requirements, CIP conveyance options may have a higher probability for leakage at the joints. A minimal amount of leakage may be acceptable due to the rural location of the project and the recharge potential to the surrounding groundwater. There will be less water loss through the CIP conveyance options than through the open and unlined canals.

8.3.6 Conceptual Design Basis

Table 8-5 has concrete wall thickness requirements for the CIP conveyance options based on a conceptual design analysis.

Table 8-5: Cast-In-Place Conveyance Options for Comparison Purposes

Option	Wall Thickness (inches)	Base thickness (inches)	Top thickness (inches)
Circular	16	--	--
Rectangular	Exterior Walls = 20 Interior Wall = 18	20	18
Arch	16	20	--

8.3.7 CIP Options Summary

CIP concrete is a viable option for the conveyance conduits between the pumping plants and the new canals and does not require construction of special potentially expensive on-site fabrication shops. As an added benefit, there are more companies who can construct these conveyance structures compared with the limited number of manufacturers who can fabricate large diameter pipelines.

9.0 SUMMARY

Based on the preliminary evaluations, the conveyance materials and configurations viable for the proposed conveyance pipelines between the pumping plants and new canals are summarized in Table 9-1.

Table 9-1: Conveyance Line Summaries

Material	Shape	Fabrication	Advantages	Disadvantages
Steel (AWWA C200) In situ cement mortar lined Coal tar enamel coated (Welded steel plates also included in this option evaluation)	Circular	Shop or Field Field	▪ Shop fabrication usually results in better weld quality. ▪ Lighter weight compared to other options ▪ Trench can be "closed up" in less time compared to cast-in-place options which has advantages relative to dewatering costs, dust control and safety. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Hauling pipe results in more opportunities for damage, such as excessive deflections. ▪ Flexible pipe requires more embedment compaction and greater quality control of the embedment. ▪ Will likely require cement mortar lining once installed and that lining is less dense and more susceptible to variations in thickness compared with factory-installed cement mortar lining. ▪ Will require impressed current cathodic protection.
Concrete Pressure Pipe (AWWA C300 & AWWA C302)	Circular	Shop or Field (field is more likely)	▪ Does not require impressed current cathodic protection. ▪ Requires less embedment compaction. ▪ Trench can be "closed up" in less time compared to cast-in-place options which has advantages relative to dewatering costs, dust control and safety. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Heavier than steel pipe ▪ Hand-applied cement mortar at joints can be susceptible to cracking. Requires periodic inspections and repairs. ▪ Few suppliers have indicated they can fabricate concrete pressure pipe greater than 12-foot-diameter. ▪ Will likely require field fabrication shop requiring up to 50 acres.
Cast-in-Place	Circular	In Trench	▪ Thinner wall thickness compared to other cast-in-place options. ▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Bedding preparation under pipe haunches will be more difficult to place compared with other options. ▪ Curved formwork and reinforcement requires more labor, higher installation costs. ▪ More difficult to support formwork bracing compared with other cast-in-place options that have a flat slab at the base. ▪ Requires more time before the trench can be "closed up" compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures.

Material	Shape	Fabrication	Advantages	Disadvantages
				▪ Concrete joints are more susceptible to leakage.
Cast-In-Place Concrete	Rectangular	In Trench	▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Formwork is less costly and labor intensive compared with cast-in-place circular conduits. ▪ Embedment materials and embedment compaction not required. ▪ Bottom slab can be placed and cured first providing surface for formwork building up. ▪ Utilizes common wall construction, making conveyance system more compact and the narrowest trench width.	▪ Requires more time before the trench can be “closed up” compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures. ▪ Concrete joints more susceptible to leakage. ▪ Not as favorable from a failure isolation standpoint compared to two separate circular conduits. ▪
Cast-in-Place Concrete	Arch	In Trench	▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Formwork is less costly and labor intensive compared with cast-in-place circular conduits. ▪ Embedment materials and embedment compaction not required. ▪ Bottom slab can be placed and cured first providing surface for formwork building up conveyance sections. ▪ Two separate arches are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Requires more time before the trench can be “closed up” compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures. ▪ Concrete joints more susceptible to leakage. ▪ Arch formwork more labor intensive compared to rectangular formwork.

ATTACHMENT A

Truck Transportation Assumptions

Truck Transportation Assumptions

General

- Diameter (or equivalent diameter) = 16 feet
- Number of pipes between intake and canal = 2
- Unit Weight of Mortar = 150 lb/ft^3
- Unit Weight of Steel = 490 lb/ft^3
- Maximum total length of conveyance for East and Dual Conveyance = 10 miles
- Max Transport Load Weight = 40,000 lbs
- Max Transport Dimensions: Height = 17 feet, Width = 15 feet
- Concrete Truck Capacity = 10 yd^3

Steel Pipe

Option 1: Fabricated and Cement-Mortar-Lined and Coated Off Site

- Steel Cylinder Thickness = 1 inch
- CML and CMC thickness = 1.5 inches

Option 2: Fabricated Offsite, Cement-Mortar-Lined and Coated On Site

- Steel Cylinder Thickness = 1 inch
- CML and CMC thickness = 0 inch

Reinforced Circular Concrete Pipe (Fabricated Off Site)

- Thickness = 16 inches

Cast-In-Place Circular Concrete Pipe

- Wall thickness = 16 inches

Cast-In-Place Arch with Common Base Slab

- Arch thickness = 16 inches
- Base Slab Height = 20 inches
- Base Slab Width = 66 feet

Cast-In-Place Arch with Common Base Slab

- Arch thickness = 16 inches
- Base Slab Height = 20 inches
- Base Slab Width = 58 feet

Cast-In-Place Box Culvert

- Exterior Wall thickness = 20 inches
- Interior Wall thickness = 18 inches
- Top slab = 18 inches
- Bottom slab = 20 inches

APPENDIX F

Pipeline Floatation Analysis

1.0 OVERVIEW

Conduit floatation is analyzed for various pipeline construction alternatives, including:

- Circular concrete pipe
- Steel pipe
- Concrete arches
- Concrete box conduits

1.1 Introduction

Conveyance alternatives to carry water from intake facilities to canals or tunnels at an assumed maximum flow of up to 15,000 cubic feet per second (cfs) are under review. Some conveyance alternatives include pressurized pipeline sections configured to deliver up to 3,000 cfs water from intakes on the Sacramento River to a new canal, forebay, or tunnel system connecting to the existing pumping plants in the south Delta.

Several types of conduit are being considered and conduit floatation is an important design consideration. Floatation is an issue in areas with a high groundwater table, such as the Delta project area. The groundwater table is, on average, 1.5 feet below ground surface, but is assumed to be at ground surface for the purposes of this conservative analysis. The future installed conduit would displace existing groundwater creating a buoyant force. If the buoyant force is larger than the weight of the conduit plus the cover on top of the conduit, floatation may occur.

This analysis for pipe floatation considers four types of conduits for conveyance pipelines: cast-in-place concrete, steel pipe, cast-in-place arches, and box conduits. Only permanent conditions of the conduit are considered, not include temporary conditions during construction.

1.2 Purpose and Scope

This purpose of this analysis is to identify the floatation potential of each conduit type and the sensitivity of floatation by modifying various pipeline design criteria.

This TM presents the assumptions and methodology for conduit floatation analysis and includes:

- Summary of general floatation design basis
- Floatation sensitivity analysis for circular conduit in terms of:
  - Depth of cover
  - Wall thickness
- Floatation sensitivity analysis for arches and box culverts in terms of depth of cover.
- Conclusions

1.3 General Design Criteria

The pipe floatation analysis was based on three types of conduits, including conveyance pipelines, both cast-in-place concrete and steel pipe, concrete arches, and concrete box conduits. The conduit sizing is based on a design flow rate of 3,000 cfs per intake since this is the conveyance capacity of the majority of the initial options.

The rationale for floatation consists of the following equation. The downward force caused by pipe weight and cover must be larger than the upward force of displaced water and buoyancy, or floatation would occur. A positive value indicates that the combined forces of weight are greater than that of buoyancy and the conduit would remain in place, whereas a negative value indicates the opposite and the conduit would float, damaging the structure. It should be noted that because the volume of water within the conduits may vary, the conduits are assumed to be empty and the weight of water within the conduits is not considered in the course of this investigation. This also allows for draining the pipeline for interior inspections and maintenance without installing groundwater dewatering facilities.

Floatation = Weight of Conduit + Weight of Soil – Buoyancy

The safety factor was also calculated to quantify the ratio of force of weight versus force of buoyancy. A ratio, or safety factor, equal to greater than 1 indicates that the force of weight is greater than that of the buoyancy force and the pipe would not float. A ratio of less than 1 signifies floatation would occur.

$$\textit{Safety Factor} = \frac{\textit{Weight of Conduit} + \textit{Weight of Soil}}{\textit{Buoyancy}}$$

Table 1 presents the assumptions used in the floatation calculations.

Table 1: Calculation Assumptions

Description	Units	Value
Unit weight concrete (W_{Conc})	lb/cf	150
Unit weight water (W_{H2O})	lb/cf	62.4
Unit weight non-saturated soil (W_{Soil})	lb/cf	110
Unit weight steel (W_{Steel})	lb/cf	490
Minimum Safety Factor	N/A	1.1

lb/cf = pound per cubic feet
N/A = not applicable

2.0 FLOATATION SENSITIVITY ANALYSIS

The following sections provide the results of the floatation analysis conducted for each conduit type. The effect of depth of cover and wall thickness are also analyzed for both pipe materials. However, because infinite thickness variations exist for concrete arches and boxes, individual investigations for these conveyance options will be conducted during subsequent design updates.

2.1 Cast-In-Place Concrete Pipe

Two floatation analyses were performed for cast-in-place concrete pipe: depth of cover, and wall thickness. The conveyance options for each 3,000 cfs intake include two 16-foot-diameter pipes. The pipes were evaluated independently, assuming lateral forces would be minimal and resulting in identical results for each pipe. Figure 1 provides an illustration of this installation and the forces considered for floatation.

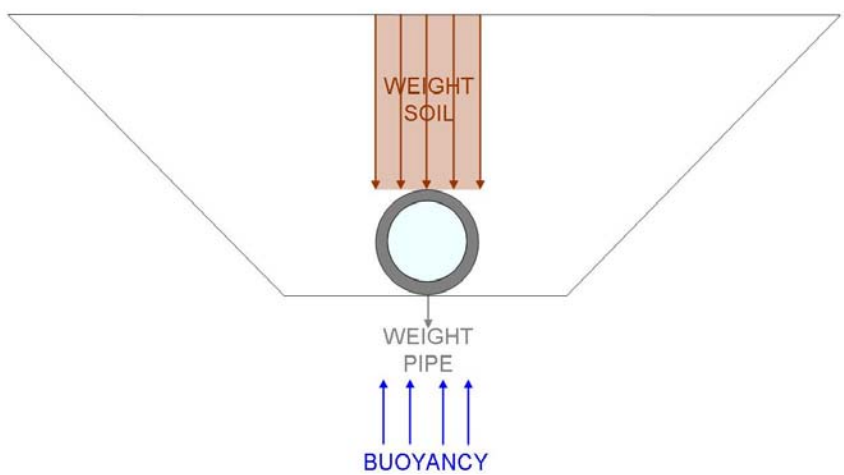


Figure 1: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the pipe, the width of the pipe outer diameter (OD), and height equal to the depth of cover (D=depth). The resulting area was then multiplied by the unit weight of non-saturated soil, equal to 110 pounds per cubic foot (lb/cf) specified in Table 1, resulting in pounds per foot (lb/ft) of soil.

Weight of Soil (lb/ft) = OD x D x W_{Soil}

The weight of pipe consisted of the weight of the cast-in-place concrete. This was determined by subtracting the area of the inner diameter (ID) of the pipeline from the area of the OD multiplied by the unit weight of concrete, assumed to be 150 lb/cf, resulting in lb/ft of concrete.

Weight of Pipe (lb/ft) = $(OD/2)^2\pi - (ID/2)^2\pi \times W_{Conc}$

Buoyancy is equal to the weight of water displaced by the pipe. This was calculated by determining the area occupied by the pipe multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the cover soil over the conduit must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

Buoyancy (lb/ft) = ((OD/2)$^2\pi$ + OD x Cover Depth) x W_{H20}

2.1.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. A pipeline diameter of 16 feet was assumed, as determined in the Pipeline Optimization TM (URS Group, Inc. [URS], 2009a, pending) with a wall thickness of 1 inch per foot diameter, 16 inches. Table 2 and Figure 2 provide the results of this investigation.

Table 2: Concrete Pipe - Depth of Cover (16-foot-diameter)

Depth of Cover (ft)	Pipe Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	10.9	0.0	17.1	0.6
2	10.9	4.1	19.4	0.8
4	10.9	8.2	21.7	0.9
6	10.9	12.3	24.1	1.0
8	10.9	16.4	26.4	1.0
10	10.9	20.5	28.7	1.1
12	10.9	24.6	31.1	1.1
14	10.9	28.7	33.4	1.2

ft = foot
lb/ft = pound per foot

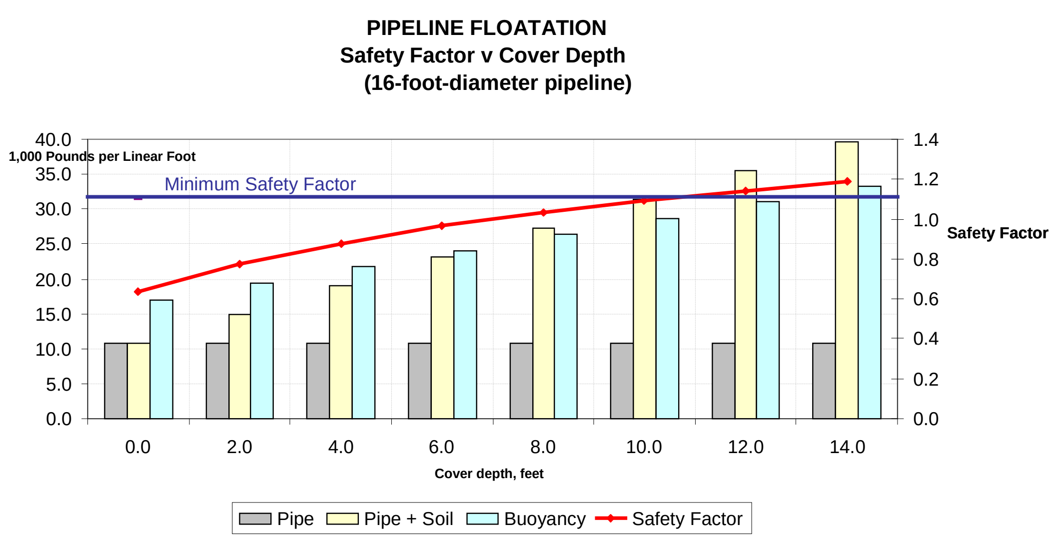


Figure 2: Concrete Pipe – Depth of Cover (16-foot-diameter)

As discussed in Section 1.3, a ratio of less than 1 indicates that the pipeline would float. Figure 2, shows that at a cover depth of less than 6 feet, the buoyant force would overcome the force of weight of the 16-foot-diameter pipe and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 10 feet is required.

Farming practices may cause disturbance of up to 6 feet of earth. An initial 16 feet of cover depth allows for 10 feet of pipe cover and 6 feet of soil for farming disturbance or erosion.

2.1.2 Wall Thickness

The second analysis provided for cast-in-place concrete pipes involved concrete thickness. This investigation assumed 16-foot-diameter pipes at various depths of cover, 0, 4, and 10 feet, and determined the floatation safety factor. While external loads and internal pressure design criteria would likely drive the design of concrete thickness, the results shown on Figure 3 present the potential effect on floatation.

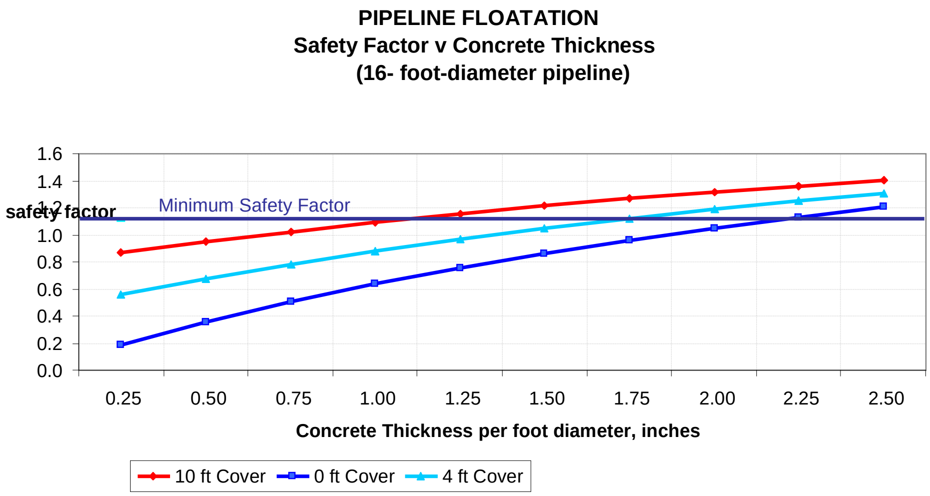


Figure 3: Concrete Pipe – Pipeline Wall Thickness

Figure 3 shows the sensitivity of pipe wall thickness to floatation prevention requirement that with 4 feet of cover, a thickness of 1.75 inch per foot diameter would prevent floatation, while a thickness of 2.25 inch per foot diameter would prevent conduit floatation without cover.

2.2 Steel Pipe

The same investigations conducted for cast-in-place concrete pipe were performed for steel pipe, using the same methodology as described in Section 2.1.

The weight of steel pipe consisted of the weight of steel, as well as the weight of the cement mortar lining. The lining thickness was assumed to be 0.5 inch, while steel thickness was assumed to be 1.0 inch thick. The pipe will likely be coated using coal tar epoxy, which is assumed to be negligible in this investigation. The approximate area per foot of each material was determined and multiplied by the unit weight, provided in Table 1. The cement mortar lining was assumed to have the same unit weight as concrete, 150 lb/cf.

$$\textit{Weight of Pipe (lb/ft)} = (OD_{Coating}/2)^2\pi - (OD_{Steel}/2)^2\pi \times W_{Conc} + (OD_{Steel}/2)^2\pi - (ID_{Steel}/2)^2\pi \times W_{Steel} + (ID_{Steel}/2)^2\pi - (ID_{Lining}/2)^2\pi \times W_{Conc}$$

2.2.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Assuming a pipeline diameter of 16 feet with a steel thickness of 1.0 inch steel and 0.5 inch thick lining, Table 3 and Figure 4 provide the results of this investigation.

Table 3: Steel Pipe - Depth of Cover (16-foot-diameter)

Depth of Cover (ft)	Pipe Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	2.4	0.0	12.8	0.2
2	2.4	3.6	14.8	0.4
4	2.4	7.1	16.8	0.6
6	2.4	10.7	18.9	0.7
8	2.4	14.2	20.9	0.8
10	2.4	17.8	22.9	0.9
12	2.4	21.3	24.9	1.0
14	2.4	24.9	26.9	1.0
16	2.4	28.5	28.9	1.1
18	2.4	32.0	31.0	1.1
20	2.4	35.6	33.0	1.2

ft = foot
lb/ft = pound per foot

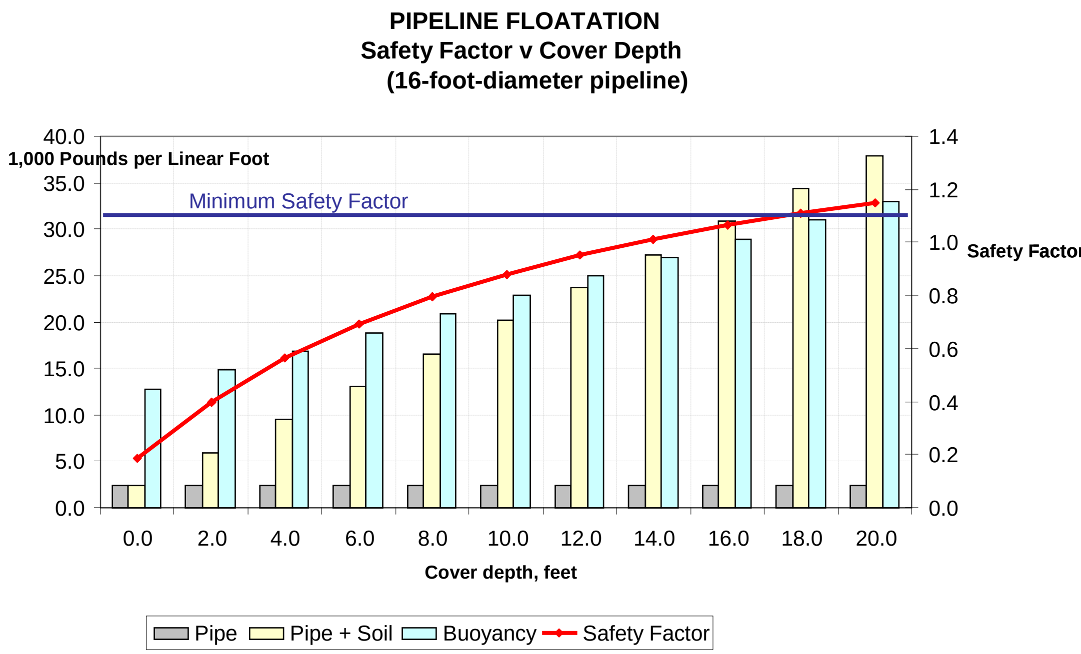


Figure 4: Steel Pipe – Depth of Cover

As discussed in Section 1.3, a ratio of less than 1 indicates that the pipeline would float. Figure 4, shows that at a cover depth of less than 12 feet the buoyant forces would overcome the force of weight of the 16-foot-diameter pipe and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 16 feet is required.

Farming practices may cause disturbance of up to 6 feet of earth. Accounting for 16 feet of pipe cover and 6 feet of soil for farming, the previous assumption of 10 feet of cover may not be sufficient to prevent floatation of steel pipe. A cover of approximately 22 feet would be necessary for steel pipe.

2.2.2 Wall Thickness

The second analysis provided for steel pipe involved steel thickness. This investigation assumed 16-foot-diameter pipes at various depths of cover, 0, 4, 7, and 10 feet, and determined the floatation safety factor at various diameter to thickness (D/t) ratios. While external load, deflection, and internal pressure design criteria would likely drive the steel thickness, the results provided on Figure 5 presents the potential effect on floatation. Table 4 provides the D/t ratios for the corresponding thicknesses.

Table 4: D/t Ratio to Steel Cylinder Thickness (16-foot-diameter)

Diameter to Thickness(D/t) ratio	Steel thickness, inch
160	1.200
200	0.960
240	0.800

The differences in thicknesses are minimal between D/t ratios, merely fractions of an inch, having a large impact on structural integrity and cost implications, although as shown on Figure 5 the effect on floatation is minimal. The figure shows that variance of the D/t ratio alone will not achieve the minimum safety factor of 1.1.

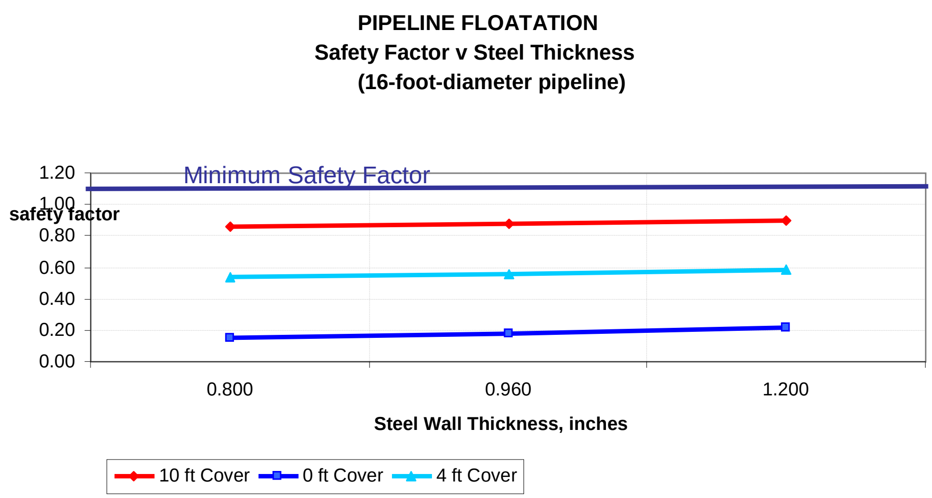


Figure 5: Steel Pipe – Pipeline Thickness

2.3 Concrete Arches

A sensitivity analyses was performed to observe the effect of various depths of cover for floatation with the concrete arch conduit option. Table 5 provides the assumed design criteria for the arches, while Figure 6 displays the layout of design.

Table 5: Arch Design Criteria

Design Criteria	Value
Number of arches	2
Arch height, feet	16
Arch width, feet	16
Arch thickness, inches	18
Distance between arches, feet	18
Distance from arch to edge of slab, feet	5
Slab width, feet	66
Slab thickness, inches	20

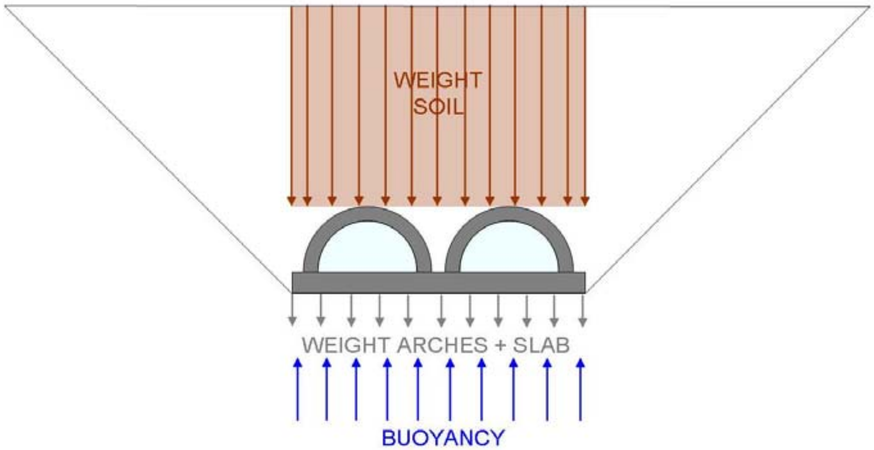


Figure 6: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the arches the width of the conduit slab (ws), and height equal to the depth of cover (D). The resulting area is then multiplied by the unit weight of soil, equal to 110 lb/cf specified in Table 1, resulting in lb/ft of soil.

Weight of Soil (lb/ft) = $w_S \times D \times W_{Soil}$

The weight of the conduit consisted of the weight of the cast-in-place concrete arches and slab. The weight of the arches was determined using the same method as described in Section 5.1 for circular pipes although divided by two and multiplied by the number of arches. The weight of slab was also determined by multiplying the width of slab (ws) by slab thickness (ts) and again by the unit weight of concrete.

Weight of Conduit (lb/ft) = $(2 \times ½ ((OD/2)2\pi - (ID/2)2\pi) + wS \times tS) \times WConc$

Buoyancy is equal to the weight of water displaced by the structure constructed, which equals the area occupied by the arches and slab multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the soil making up the conduit cover must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

Buoyancy (lb/ft) = $(2 \times ½ (OD/2)^2\pi + w_S \times t_S + \text{Cover Depth} \times OD) \times W_{H20}$

2.3.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Table 6 and Figure 7 provide the results of this investigation.

Table 6: Arches - Depth of Cover

Depth of Cover (ft)	Conduit Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	28.9	0.0	24.6	1.2
2	28.9	14.5	32.8	1.3
4	28.9	29.0	41.0	1.4
6	28.9	43.6	49.3	1.5
8	28.9	58.1	57.5	1.5
10	28.9	72.6	65.7	1.5
12	28.9	87.1	74.0	1.6
14	28.9	101.6	82.2	1.6

ft = foot
lb/ft = pound per foot

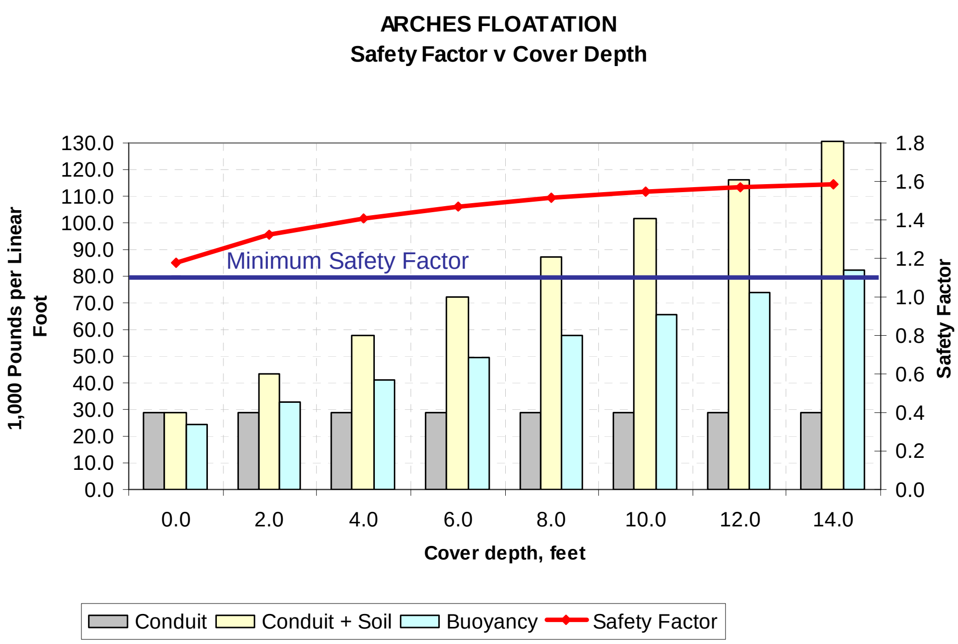


Figure 7: Arches – Depth of Cover

As discussed in Section 1.3, a weight to buoyancy ratio of less than 1 indicates that the pipeline would float. Figure 7 shows that without any cover the arches will not float, meeting the assumed safety factor.

Accounting for 6 feet of farming practices and no pipe cover, the total necessary depth of cover for the arches would be 6 feet.

2.4 Concrete Box Conduits

A floatation sensitivity analyses was also performed on the concrete box conduit alternative in terms of various cover depths. Table 7 provides the assumed design criteria for the boxes, while Figure 8 displays the layout of design.

Table 7: Box Conduit Design Criteria

Design Criteria	Value
Number of boxes	2
Box height, feet	20
Box width, feet	10
Wall thickness, inch	20
Top slab thickness, inch	18
Bottom slab thickness, inch	20
Total conduit width, feet	25
Total conduit height, feet	23

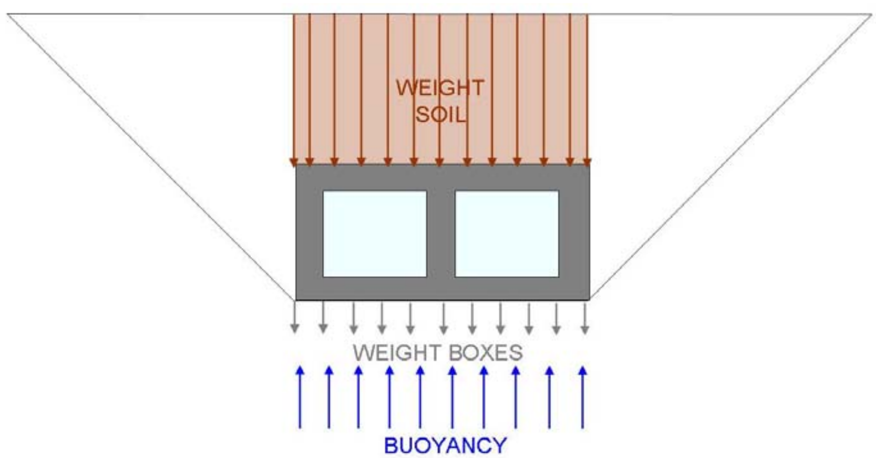


Figure 8: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the box conduit the width of the conduit (wc), and height equal to the depth of cover (D). The resulting area is then multiplied by the unit weight of soil, equal to 110 lb/cf specified in Table 1, resulting in lb/ft of soil.

Weight of Soil (lb/ft) = wc x D x WSoil

The weight of the conduit consisted of the weight of the cast-in-place concrete box subtracting the inner conduit boxes. The weight of conduit was determined by multiplying the wc by conduit height (hc), subtracting the area of the two boxes, box height (bh) multiplied by box width (bw), multiplying by the unit weight of concrete.

Weight of Conduit (lb/ft) =(wc x hc – 2 x (bh x bw)) x WConc

Buoyancy is equal to the weight of water displaced by the structure constructed, which equals the area occupied by the arches and slab multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the soil making up the conduit cover must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

Buoyancy (lb/ft) = (wc x hc + Cover Depth x wc) x WH20

2.4.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Table 8 and Figure 9 provide the results of this investigation.

Table 8: Box Conduit- Depth of Cover

Depth of Cover (ft)	Conduit Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	26.3	0.0	35.9	0.7
2	26.3	5.5	39.0	0.8
4	26.3	10.9	42.1	0.9
6	26.3	16.4	45.2	0.9
8	26.3	21.9	48.3	1.0
10	26.3	27.3	51.4	1.0
12	26.3	32.8	54.5	1.1
14	26.3	38.2	57.6	1.1
16	26.3	43.7	60.7	1.2
18	26.3	49.2	63.8	1.2

ft = foot
lb/ft = pound per foot

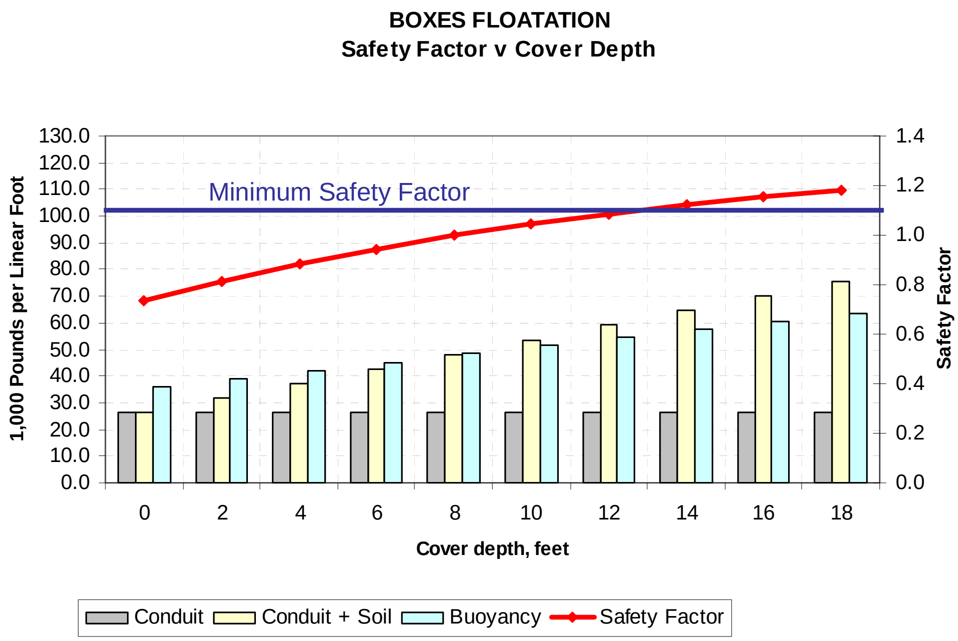


Figure 9: Concrete Box – Depth of Cover

A safety weight to buoyancy ratio of less than 1 indicates that the pipeline would float. As shown, a cover depth of less than 8 feet would allow the buoyant force to overcome the force of weight of the box conduits and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 12 feet is required, resulting in a necessary cover depth of 18 feet, including 6 feet for farming practices.

3.0 CONCLUSIONS

The floatation sensitivity analysis resulted in the following conclusions.

1. Concrete thickness variations affect floatation for cast-in-place circular pipes, whereas steel thickness has a minimal effect. Table 9 provides the thicknesses required to achieve the minimum factor of safety at various depths of cover for a 16-foot-diameter concrete pipe.
2. Each conduit type investigated would require varying cover depths to achieve the assumed safety factor. Table 10 provides these depths. It should also be noted that farming practices can cause disruption as deep as 6 feet. This is also accounted for in the following table.

Table 9: Required Thickness for Circular Concrete Pipe

Depth of Cover (feet)	Required Thickness to Prevent Floatation (inch)
0	32
4	28
10	16

Table 10: Required Depth of Cover

Conduit	Minimum Depth of Cover Required (feet)	Farming Depth (feet)	Total Depth of Cover (feet)
Concrete Pipe	10	6	16
Steel Pipe	16	6	22
Concrete Arches	0	6	6
Concrete Boxes	12	6	18

4.0 RECOMMENDATIONS

The preferred conduit minimum depth of cover is 10 feet. As shown in Table 10, meeting this design criterion may cause issues with floatation for several of the conduit types. The following floatation prevention alternatives will be further investigated and a preferred alternative will be identified:

- Increase conduit thickness
- Provide a concrete slab in between parallel conduits and anchor conduits to the slab
- Increase footing width
- Cap conduits with cement slurry
- Negotiate easements to prohibit disturbances, such as by farming practices, to conduit cover
- Provide concrete collars intermittently along conduit
- Anchor conduit to piles along alignment