Conceptual Engineering Report
California WaterFix
Byron Tract Forebay Option (WaterFix BTO)

July 2018

Prepared for:
DEPARTMENT OF WATER RESOURCES
Division of Engineering
901 P Street, Suite 313B
Sacramento, CA 95814

Prepared by:

California WaterFix: Program
Preliminary Not for Construction
Limitations and Acknowledgements

This report was prepared in accordance with the 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.

Many aspects of this report are adapted from the following sources:

- **Conceptual Engineering Report All Tunnel Option** dated March 10, 2010 (California Department of Water Resources [DWR], 2010a) and hereinafter referred to as the Draft ATO CER
- **Addendum to the Conceptual Engineering Report for the Isolated Conveyance Facility Pipeline/Tunnel Option (formerly All Tunnel Option)** dated October 22, 2010 (DWR, 2010b) and hereinafter referred to as the PTO CER
- **Conceptual Engineering Report for the Dual Conveyance Facility Modified Pipeline/Tunnel Option** dated October 1, 2013 and hereinafter referred to as the MPTO CER
- **Conceptual Engineering Report for the Dual Conveyance Facility Modified Pipeline/Tunnel Option Clifton Court Forebay Option** dated July 1, 2015 and hereinafter referred to as the MPTO/CCO CER

Information from the Draft ATO CER, PTO CER, MPTO CER and MPTO/CCO CER that was directly applicable to this conceptual engineering report was directly incorporated into this report. Information in the Draft ATO CER, the PTO CER, MPTO CER, MPTO/CCO CER that was similar to the characteristics of the concepts in this report was modified and adapted into this report. Additional information was provided as applicable for new or changed concepts. No specific reference to the Draft ATO CER, PTO CER, MPTO CER and MPTO/CCO CER beyond that described here, is provided for adapted or incorporated content.

The purpose of this report is to provide conceptual engineering of facilities required for the California WaterFix 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 as well as the 2018 Supplemental Environmental Impact Statement (S-EIS) and Supplemental Environmental Impact Report (S-EIR). It is also expected that the Lead Agencies, the S-EIR/EIS consulting team, and other stakeholders would recommend modifications to these facilities over the coming months, and such changes would be evaluated as needed to support the S-EIR/EIS.

This document is part of an iterative process of developing options that can be used as alternatives in the S-EIR/EIS. Therefore, all locations, dimensions, quantities, design concepts, construction techniques, and other information 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. All of the information presented in this report is considered conceptual or preliminary and will need to be verified as part of additional investigations and detailed design.
## Appendices

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Geology and Seismicity</td>
</tr>
<tr>
<td>B</td>
<td>Conceptual Level Construction Sequencing of DHCCP Intakes</td>
</tr>
<tr>
<td>C</td>
<td>MPTO/BTO Conceptual Construction Schedule</td>
</tr>
<tr>
<td>D</td>
<td>Surge Analysis Technical Memorandum</td>
</tr>
<tr>
<td>E</td>
<td>Pipe Materials</td>
</tr>
<tr>
<td>F</td>
<td>Pipe Floatation Analysis</td>
</tr>
<tr>
<td>G</td>
<td>Summary of Tunneling Contractor Workshops</td>
</tr>
<tr>
<td>H</td>
<td>Hydraulic Analysis Technical Memorandum</td>
</tr>
<tr>
<td>I</td>
<td>Basis of Conceptual Design for Tunnel Lining Systems</td>
</tr>
<tr>
<td>J</td>
<td>Alternative Designs for Tunnel Lining Systems</td>
</tr>
<tr>
<td>K</td>
<td>Potential Water Leakage Rates from Tunnels Technical Memorandum</td>
</tr>
<tr>
<td>L</td>
<td>DWR Seismic Design Criteria</td>
</tr>
<tr>
<td>M</td>
<td>Seismic Review of Tunnel Liner Performance</td>
</tr>
</tbody>
</table>
Volume 2: Concept Drawings (Bound Separately)
Volume 3: Map Book (Bound Separately)
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>AF</td>
<td>acre-feet</td>
</tr>
<tr>
<td>ANSI</td>
<td>American National Standards Institute</td>
</tr>
<tr>
<td>ASCE</td>
<td>American Society of Civil Engineers</td>
</tr>
<tr>
<td>ASTM</td>
<td>ASTM International (formerly American Society for Testing and Materials)</td>
</tr>
<tr>
<td>ATO</td>
<td>All Tunnel Option</td>
</tr>
<tr>
<td>ATS</td>
<td>Automatic Transfer Switch</td>
</tr>
<tr>
<td>AWWA</td>
<td>American Water Works Association</td>
</tr>
<tr>
<td>Banks PP</td>
<td>Harvey O. Banks Pumping Plant</td>
</tr>
<tr>
<td>BBID</td>
<td>Byron Bethany Irrigation District</td>
</tr>
<tr>
<td>BDCP</td>
<td>Bay Delta Conservation Plan</td>
</tr>
<tr>
<td>BEP</td>
<td>best efficiency flow capacity point</td>
</tr>
<tr>
<td>bgs</td>
<td>below ground surface</td>
</tr>
<tr>
<td>BMP</td>
<td>best management practice</td>
</tr>
<tr>
<td>BTF</td>
<td>Byron Tract Forebay</td>
</tr>
<tr>
<td>BTO</td>
<td>Byron Tract Forebay Option</td>
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<tr>
<td>Cal/OSHA</td>
<td>State of California Division of Occupational Safety and Health</td>
</tr>
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<td>CALFED</td>
<td>California Bay-Delta Authority</td>
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<tr>
<td>CCF</td>
<td>Clifton Court Forebay</td>
</tr>
<tr>
<td>CCO</td>
<td>Clifton Court Option</td>
</tr>
<tr>
<td>CER</td>
<td>conceptual engineering report</td>
</tr>
<tr>
<td>CFD</td>
<td>computational fluid dynamics</td>
</tr>
<tr>
<td>cfs</td>
<td>cubic feet per second</td>
</tr>
<tr>
<td>CIP</td>
<td>cast-in-place</td>
</tr>
<tr>
<td>CPT</td>
<td>cone penetrometer test</td>
</tr>
<tr>
<td>CVFPB</td>
<td>Central Valley Flood Protection Board</td>
</tr>
<tr>
<td>CVP</td>
<td>Central Valley Project</td>
</tr>
<tr>
<td>CWF</td>
<td>California WaterFix</td>
</tr>
<tr>
<td>cy</td>
<td>cubic yard</td>
</tr>
<tr>
<td>DCC</td>
<td>Delta Cross Channel</td>
</tr>
<tr>
<td>Delta</td>
<td>Sacramento River and San Joaquin River Delta</td>
</tr>
<tr>
<td>DHCCP</td>
<td>Delta Habitat Conservation and Conveyance Program</td>
</tr>
<tr>
<td>DMC</td>
<td>Delta-Mendota Canal</td>
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<tr>
<td>DSOD</td>
<td>Division of Safety of Dams</td>
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<tr>
<td>DRMS</td>
<td>Delta Risk Management Strategy</td>
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<td>DWR</td>
<td>California Department of Water Resources</td>
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<td>E&amp;W</td>
<td>East and West</td>
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<tr>
<td><strong>ACRONYMS AND ABBREVIATIONS</strong></td>
<td></td>
</tr>
<tr>
<td>--------------------------------</td>
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</tr>
<tr>
<td><strong>EBMUD</strong> East Bay Municipal Utility District</td>
<td></td>
</tr>
<tr>
<td><strong>EIR</strong> environmental impact report</td>
<td></td>
</tr>
<tr>
<td><strong>EIS</strong> environmental impact statement</td>
<td></td>
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<tr>
<td><strong>EPB</strong> earth pressure balance</td>
<td></td>
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<tr>
<td><strong>FB</strong> freeboard</td>
<td></td>
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<tr>
<td><strong>FFTT</strong> Fish Facilities Technical Team</td>
<td></td>
</tr>
<tr>
<td><strong>fpm</strong> feet per minute</td>
<td></td>
</tr>
<tr>
<td><strong>fps</strong> feet per second</td>
<td></td>
</tr>
<tr>
<td><strong>FRWA</strong> Freeport Regional Water Authority</td>
<td></td>
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<tr>
<td><strong>g</strong> measurement of peak ground acceleration</td>
<td></td>
</tr>
<tr>
<td><strong>gpd</strong> gallons per day</td>
<td></td>
</tr>
<tr>
<td><strong>ghg</strong> greenhouse gas</td>
<td></td>
</tr>
<tr>
<td><strong>GHGRP</strong> Greenhouse Gas Reduction Plan</td>
<td></td>
</tr>
<tr>
<td><strong>gpm</strong> gallons per minute</td>
<td></td>
</tr>
<tr>
<td><strong>GPS</strong> global positioning satellite</td>
<td></td>
</tr>
<tr>
<td><strong>HI</strong> Hydraulic Institute</td>
<td></td>
</tr>
<tr>
<td><strong>HGL</strong> hydraulic grade line</td>
<td></td>
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<tr>
<td><strong>H:V</strong> horizontal to vertical ratio</td>
<td></td>
</tr>
<tr>
<td><strong>HMA</strong> hot mix asphalt</td>
<td></td>
</tr>
<tr>
<td><strong>HMI</strong> Human Machine Interface</td>
<td></td>
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<tr>
<td><strong>hp</strong> horsepower</td>
<td></td>
</tr>
<tr>
<td><strong>HPU</strong> hydraulic power unit</td>
<td></td>
</tr>
<tr>
<td><strong>HVAC</strong> heating, ventilation, and air conditioning</td>
<td></td>
</tr>
<tr>
<td><strong>ID</strong> inside diameter</td>
<td></td>
</tr>
<tr>
<td><strong>ICF</strong> Isolated Conveyance Facility</td>
<td></td>
</tr>
<tr>
<td><strong>IPP</strong> Intermediate Pumping Plant</td>
<td></td>
</tr>
<tr>
<td><strong>IF</strong> Intermediate Forebay (Intermediate Pumping Plant Forebay)</td>
<td></td>
</tr>
<tr>
<td><strong>JOC</strong> Joint Operations Center</td>
<td></td>
</tr>
<tr>
<td><strong>Jones PP</strong> C.W. “Bill” Jones Pumping Plant</td>
<td></td>
</tr>
<tr>
<td><strong>km</strong> kilometer(s)</td>
<td></td>
</tr>
<tr>
<td><strong>kV</strong> kilovolt</td>
<td></td>
</tr>
<tr>
<td><strong>LCI</strong> Load commutated inverter</td>
<td></td>
</tr>
<tr>
<td><strong>MCC</strong> motor control center</td>
<td></td>
</tr>
<tr>
<td><strong>MCE</strong> Maximum Considered Earthquake</td>
<td></td>
</tr>
<tr>
<td><strong>mm</strong> millimeter(s)</td>
<td></td>
</tr>
</tbody>
</table>
mm/year  millimeter(s) per year
MPTO  Modified Pipeline/Tunnel Option
msl  mean sea level
MVA  megavolt-amperes
MW  megawatt

NAVD88  North American Vertical Datum of 1988 (all elevations in this report use NAVD88)
NCCF  North Clifton Court Forebay
NEMA  National Electrical Manufacturers Association
NGA  next generation attenuation
NGVD29  North Geodetic Vertical Datum of 1929
NPDES  National Pollutant Discharge Elimination System
NPSH  Net positive suction head
NPSHR  net positive suction head required
NTP  Notice To Proceed

O&M  operation and maintenance
PARO  Power and Risk Office
PGA  peak ground acceleration
PG&E  Pacific Gas and Electric Company
PLC  programmable logic controller
POR  preferred operating range
PP  Pumping plant
PP2  Intake No. 2 Pumping Plant
PP3  Intake No. 3 Pumping Plant
PP5  Intake No. 5 Pumping Plant
PTO  Pipeline/Tunnel Option
PWM  Pulse width modulated

Reclamation  United States Bureau of Reclamation
RCC  roller-compacted concrete
RCCP  Reinforced concrete cylinder pressure pipe
RCP  Reinforced concrete pressure pipe
ROW  right-of-way
rpm  revolutions per minute
RSP  rock slope protection
RTM  re-usable tunnel material

SCADA  supervisory control and data acquisition
SCCF  South Clifton Court Forebay
SHC  system head curve
Skinner Facility  Skinner Delta Fish Protection Facility
<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
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<tbody>
<tr>
<td>SLDMWA</td>
<td>San Luis and Delta-Mendota Water Authority</td>
</tr>
<tr>
<td>SLR</td>
<td>sea level rise</td>
</tr>
<tr>
<td>SPB</td>
<td>Slurry pressure balance</td>
</tr>
<tr>
<td>SMUD</td>
<td>Sacramento Municipal Utility District</td>
</tr>
<tr>
<td>SPRR</td>
<td>Southern Pacific Railroad</td>
</tr>
<tr>
<td>SWP</td>
<td>State Water Project</td>
</tr>
<tr>
<td>SWPPP</td>
<td>Stormwater Pollution Prevention Plan</td>
</tr>
<tr>
<td>TBD</td>
<td>to be determined</td>
</tr>
<tr>
<td>TBM</td>
<td>tunnel-boring machine</td>
</tr>
<tr>
<td>TDH</td>
<td>total dynamic head</td>
</tr>
<tr>
<td>TEWAC</td>
<td>totally enclosed water-to-air-cooled</td>
</tr>
<tr>
<td>UPRR</td>
<td>Union Pacific Railroad</td>
</tr>
<tr>
<td>URS</td>
<td>URS Corporation</td>
</tr>
<tr>
<td>USACE</td>
<td>United States Army Corps of Engineers</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
<tr>
<td>VFD</td>
<td>variable frequency drive</td>
</tr>
<tr>
<td>w day</td>
<td>working day</td>
</tr>
<tr>
<td>WAPA</td>
<td>Western Area Power Administration</td>
</tr>
<tr>
<td>WGCEP</td>
<td>Working Group on California Earthquake Probabilities</td>
</tr>
<tr>
<td>WSE</td>
<td>water surface elevation</td>
</tr>
<tr>
<td>%</td>
<td>percent(ile)</td>
</tr>
<tr>
<td>24/7</td>
<td>24 hours per day/7 days per week</td>
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</table>
Executive Summary

The California WaterFix (WaterFix) is an effort by federal and state agencies and other stakeholders that began in 2006 to stabilize water deliveries from the Delta while providing benefits to the Delta ecosystem as a whole. In April 2015, Governor Edmund G. Brown, Jr. and federal officials announced plans to accelerate restoration of the Delta’s ecosystem and fix the State’s aging water infrastructure. At that time, the WaterFix program was announced as the method that the State now supported to build a reliable delta conveyance system. In coincidence with the formation of the WaterFix, the State also announced the formation of the EcoRestore program to address the timely work on long-stalled Delta environmental projects. The combination of WaterFix and EcoRestore replaced the previous Bay Delta Conservation Plan (BDCP, which included Conservation Measure 1 (CM1) which focused on the construction and operation of the water conveyance infrastructure under the previous Habitat Conservation Plan.

The WaterFix conveyance facilities were previously documented in the Conceptual Engineering Report (CER) dated July 1, 2015 and included a 9,000 cfs conveyance system, three equally sized north Delta River intakes, a gravity-fed tunnel system, twin pumping facilities at Clifton Court Forebay, modifications to Clifton Court Forebay, connections to the Banks and Jones pumping plants, and the Head of Old River Gate facility at the divergence of the San Joaquin and Old Rivers. That version of the CER was comprised of three volumes; Volume 1 was the CER itself, Volume 2 contained the conceptual engineering drawings for the WaterFix, and Volume 3 contained the facility map book. The facilities described in the 2015 CER were the basis of the Alternative 4A which was the DWR recommended project in the 2016 Final EIR/EIS on the WaterFix initiative. This Final EIR/EIS eventually resulted in a Notice of Determination issued by DWR in July 2017.

The current CER represents a modified WaterFix configuration when compared to the July 1, 2015 version, and was developed to implement a series of system optimization measures, and to support the development of a Supplemental EIR/EIS that was produced by DWR in 2018. Some of the significant differences between this current alternative and the July 2015 WaterFix configuration include: establishing a new Byron Tract Forebay (BTF) at the south end of the Delta conveyance system, eliminating the need for any WaterFix related work inside Clifton Court Forebay, reconfiguring the Reusable Tunnel Material (RTM) storage locations so as to minimize wetland impacts, constructing new tunnels and connection channels south of BTF to convey water from BTF to the Banks and Jones Pumping Plants, and adjusting the North Tunnel alignment to avoid sensitive receptors.

The WaterFix Byron Tract Option (BTO) retains the capability for Isolated South Delta, Isolated North Delta, or dual mode operation that was described in the July 2015 CER. Under single mode operation, Jones and Banks Pumping Plants receive water from either North Delta diversions or South Delta diversions. Dual mode operations allow Jones and Banks to simultaneously receive water from North and South Delta diversions.

The current CER consists of three volumes:

- Volume 1 – Conceptual Engineering Report
- Volume 2 – Concept Drawings (Bound Separately)
- Volume 3 – Map Book (Bound Separately)

This CER is a conceptual engineering effort, representing a 5-10% design effort, depending on configuration of each specific facility. Facility locations, dimensions, and elevations are approximate and subject to change during the preliminary engineering phase, however maximum footprints and construction impacts have been estimated in the CER to facilitate related environmental planning efforts.

ES.1 Project Overview

- The WaterFix BTO as described in this CER is an isolated conveyance system that has the capability to divert flows from the North Delta portion of the Sacramento River and convey that water to the State Water Project and Central Valley Project export pumps located near Tracy, California. The facilities described in this CER have been examined from an environmental planning perspective through a
Supplemental EIR/EIS that was developed by DWR in 2018. The WaterFix BTO overall alignment and facility arrangement is shown in Figure ES-1.

The WaterFix BTO will include the following key facilities:

- Three Intake Facilities along the Sacramento River in the north Delta with fish-screened on-bank intake structures and conveyance tunnels (North Tunnels).
- An Intermediate Forebay (IF) to receive flow from each Intake Facility and to equally split flows to the main tunnels
- Dual main tunnel to provide gravity flow to the Byron Tract Forebay (BTF) Pumping Plant.
- BTF to provide operational storage. BTF will be capable of receiving flows by gravity directly from the main tunnels, or from the pump stations located at the terminus of the main tunnels, depending on hydraulic conditions in the Sacramento River and in the BTF.
- A BTF Pumping Plant located west of DWR’s existing Clifton Court Forebay (CCF).
- South Conveyance Facilities located at the outlet of BTF including: tunnels, outlet structure, and open channels
- Head of Old River Gate – Operable Barrier

**ES.1.1 WaterFix BTO Assumptions**

The facilities included in the WaterFix BTO assume the following:

- The WaterFix BTO will deliver up to 9,000 cubic feet per second (cfs) from the Sacramento River in the north Delta to the south Delta export pumping plants
- At the low water level of the Sacramento River, the WaterFix BTO has been configured to be able to deliver the design flow rates more than 99 percent of the time.
- The WaterFix BTO is configured to:
  - Transport water from North Delta diversions in conveyance facilities that are isolated from existing Delta Rivers and sloughs.
  - Provide for potential simultaneous utilization of both north Delta intakes and south Delta diversion facilities to meet the SWP and CVP demands
  - Protect WaterFix conveyance facilities against a 200-year flood event with the sea level rise (SLR) predicted from climate change.
  - Use gravity flow to convey water through the North and Main Tunnels to the Byron Tract pumping facilities
  - Use gravity flow to convey water from the river intakes to BTF under certain hydraulic conditions.
  - Provide an operable barrier with control gates at the head of Old River to reduce migration of San Joaquin River watershed salmonids into the South Delta through the Old River
- Key assumptions for the project implementation approach remain largely unchanged from the July 2015 CER.
**Legend**
- **Intake Site**
- **North Tunnels**
- **Main Tunnels (Twin-Bore)**
- **South Tunnels (Twin-Bore)**
- **Operable Barrier**
- **Channels**
- **Forebay**

**Waterfix Byron Tract Forebay Option**
- 3 intake facilities with fish screens along the Sacramento River
- Combined pumping plant @ BTF
- 60 miles of main tunnels
- 13.5 miles of north tunnels
- 3.2 miles of south tunnels
- 28-acre Intermediate Forebay at bottom el. -20 ft
- 820-acre BTF

**Figure ES-1:** Location of Facilities
ES.1.2 Implications of WaterFix BTO on Current California State Water Code

The WaterFix BTO facilities conveying water to the SWP and CVP export pumping plants comply with the following California State Water Code Sections:

- **Water § 259.** Law governing condemnation of railroad, public utility or state agency property: When the department condemns the property of any common carrier railroad, other public utility, or state agency, or the appurtenances thereof, it shall be governed by Article 3 (commencing with Section 11590) of Chapter 6 of Part 3 of Division 6.

- **Water § 11590.** Substitution of facilities; agreement: The department has no power to take or destroy the whole or any part of the line or plant of any common carrier railroad, other public utility, or state agency, or the appurtenances thereof, either in the construction of any dam, canal, or other works, or by including the same within the area of any reservoir, unless and until the department has provided and substituted for the facilities to be taken or destroyed new facilities of like character and at least equal in usefulness with suitable adjustment for any increase or decrease in the cost of operating and maintenance thereof, or unless and until the taking or destruction has been permitted by agreement executed between the department and the common carrier, public utility, or state agency.

- **Water § 11592.** Public Utilities Commission; submission of controversies: In the event the department and any common carrier railroad, other public utility, or state agency fail to agree as to the character or location of new facilities to be provided as required in this article, the character and location of the new facilities and any other controversy concerning requirements imposed by this chapter shall be submitted to and determined and decided by the Public Utilities Commission of the State.

ES.2 WaterFix BTO Component Descriptions

Table ES-1 summarizes the physical characteristics of the Intake Facilities, North Tunnels, Intermediate Forebay, Main Tunnels, Byron Tract Pumping Plant (BTPP), BTF, South Tunnels, and Connection Channels. Figure ES-2 illustrates a conveyance schematic of the WaterFix BTO, with references to sections within this CER where detailed facility information is presented.

**Table ES-1: Summary of WaterFix BTO Physical Characteristics**

<table>
<thead>
<tr>
<th>Feature Description</th>
<th>Approximate Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Overall Project</strong></td>
<td></td>
</tr>
<tr>
<td>Conveyance Capacity</td>
<td>9,000 cfs</td>
</tr>
<tr>
<td>Overall Length of All Tunnels</td>
<td>76.5 miles</td>
</tr>
<tr>
<td><strong>Intake Facilities</strong></td>
<td></td>
</tr>
<tr>
<td>Number of In-River-Screened Intakes</td>
<td>3</td>
</tr>
<tr>
<td>Flow Capacity at Each Intake</td>
<td>3,000 cfs</td>
</tr>
<tr>
<td><strong>Byron Tract Pumping Plant</strong></td>
<td></td>
</tr>
<tr>
<td>Total Number of Pumps (both Pumping Plants)</td>
<td>12</td>
</tr>
<tr>
<td>8 Large Pumps, Capacity per Pump (cfs)</td>
<td>1,125</td>
</tr>
<tr>
<td>4 Small Pumps, Capacity per Pump (cfs)</td>
<td>563</td>
</tr>
<tr>
<td>Total Dynamic Head</td>
<td>37 feet</td>
</tr>
<tr>
<td>Estimated Pump Load (MW, both Pumping Plants)</td>
<td>36</td>
</tr>
</tbody>
</table>
Table ES-1 (CONTINUED): Summary of WaterFix BTO Physical Characteristics

<table>
<thead>
<tr>
<th>Feature Description</th>
<th>Approximate Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>North Tunnels (From Intakes to IF)</strong></td>
<td></td>
</tr>
<tr>
<td>North Tunnel – Intake No. 2 (Connecting to a Junction Shaft near Intake No. 3)</td>
<td></td>
</tr>
<tr>
<td>Maximum Flow</td>
<td>3,000 cfs</td>
</tr>
<tr>
<td>Tunnel Length</td>
<td>1.99 miles</td>
</tr>
<tr>
<td>Number of Tunnel Bores; Number of Shafts (total)</td>
<td>1; 1</td>
</tr>
<tr>
<td>Tunnel Finished Inside Diameter</td>
<td>28 feet</td>
</tr>
<tr>
<td>North Tunnel – Intake No. 3 (from Junction Shaft to IF)</td>
<td></td>
</tr>
<tr>
<td>Maximum Flow Tunnel (Intake Flow)</td>
<td>6,000 cfs (3,000 cfs)</td>
</tr>
<tr>
<td>Tunnel Length</td>
<td>6.74 miles</td>
</tr>
<tr>
<td>Number of Tunnel Bores; Number of Shafts (total)</td>
<td>1; 3</td>
</tr>
<tr>
<td>Tunnel Finished Inside Diameter</td>
<td>40 feet</td>
</tr>
<tr>
<td>North Tunnel – Intake No. 5</td>
<td></td>
</tr>
<tr>
<td>Maximum Flow</td>
<td>3,000 cfs</td>
</tr>
<tr>
<td>Tunnel Length</td>
<td>4.77 miles</td>
</tr>
<tr>
<td>Number of Tunnel Bores; Number of Shafts (total)</td>
<td>1; 3</td>
</tr>
<tr>
<td>Tunnel Finished Inside Diameter</td>
<td>28 feet</td>
</tr>
<tr>
<td><strong>Intermediate Forebay</strong></td>
<td></td>
</tr>
<tr>
<td>Surface Area (at river elevation is 10 feet, IF WSE is at 0 ft)</td>
<td>37 acres</td>
</tr>
<tr>
<td><strong>Main Tunnels (From IF to BTF Pumping Plant)</strong></td>
<td></td>
</tr>
<tr>
<td>Number of Tunnels</td>
<td>2; parallel</td>
</tr>
<tr>
<td>Tunnel Length (each)</td>
<td>29.94 miles</td>
</tr>
<tr>
<td>Tunnel Finished Inside Diameter</td>
<td>40 feet</td>
</tr>
<tr>
<td><strong>Byron Tract Forebay</strong></td>
<td></td>
</tr>
<tr>
<td>Surface Area at Low Water Level (at water elevation 2.5 ft.)</td>
<td>810 acres</td>
</tr>
<tr>
<td><strong>South Tunnels (From BTF Outlet Structure to South Tunnel Outlet Structure at Connection Channel)</strong></td>
<td></td>
</tr>
<tr>
<td>Number of Tunnels</td>
<td>2; parallel</td>
</tr>
<tr>
<td>Tunnel Length (each)</td>
<td>1.59 miles</td>
</tr>
<tr>
<td>Tunnel Finished Inside Diameter</td>
<td>40 feet</td>
</tr>
<tr>
<td><strong>Connection Channels</strong></td>
<td></td>
</tr>
<tr>
<td>From South Tunnel Outlet Structure to Banks PP Intake Channel</td>
<td></td>
</tr>
<tr>
<td>Channel Length</td>
<td>2,800 feet</td>
</tr>
<tr>
<td>From South Tunnel Outlet Structure to Jones PP Intake Channel</td>
<td></td>
</tr>
<tr>
<td>Channel Length</td>
<td>4,800 feet</td>
</tr>
<tr>
<td><strong>Operable Barrier (Head of Old River)</strong></td>
<td></td>
</tr>
<tr>
<td>Approximate Dimensions</td>
<td>210 feet x 30 feet</td>
</tr>
<tr>
<td>Number of Bottom-Hinged Gates</td>
<td>5</td>
</tr>
</tbody>
</table>

Notes:  
AF = acre-feet  
cfs = cubic feet per second  
ft = feet  
MVA = Mega Volt Ampere
Figure ES-2: Conveyance Schematic
ES.2.1 River Intakes
The three River Intake Facilities (Intakes No. 2, 3, and 5) will each have a capacity of 3,000 cfs as proposed by DWR and the Fish Facilities Technical Team (FFTT), for a total capacity of 9,000 cfs. The Intake Facilities will be located along the Sacramento River, as shown in the CER Volume 2 (Concept Drawings) and Volume 3 (Map Book). The WaterFix BTO intake locations were selected in prior work activities by DWR in consultation with the FFTT, through various studies and previous CERs associated with the prior FEIR/EIS. Intake location and numbering are unchanged from the 2015 CER. Additional studies to finalize the configuration of the intakes will commence during preliminary design in consultation with the FFTT.

Each Intake Facility will consist of the following:

- A fish-screened intake structure that employs state-of-the-art on-bank fish screens, sized to provide an approach velocity of 0.2 feet/second under design diversion conditions
- Twelve large gravity collector box conduits that will include flow meters and control gates, and will convey flow from the screened intakes into a sedimentation system
- A sedimentation system consisting of gravity settling basin to capture sand-sized sediment and a drying lagoon for sediment drying and disposal
- Drop shaft structures to convey water from the sedimentation basins into the North Tunnel system
- Support equipment and features, including a relocated Highway 160, and such electrical equipment required to operate the intake equipment
- Potential future provisions, not described in this CER, to reintroduce dried sediment back into the Sacramento River

ES.2.2 Tunnels
The proposed conveyance tunnels include the North Tunnels, which consist of three separate tunnel reaches totaling approximately 13.5 miles that connect the three Intake Facilities to the Intermediate Forebay. Two 40-foot ID parallel Main Tunnels will convey water from the Intermediate Forebay to the BTF, each approximately 30 miles long. Each tunnel will convey up to 4,500 cfs, for a total of 9,000 cfs to the BTF. The South Tunnels will convey flow from the BTF Outlet Structure at the southern end of BTF to the South Tunnel Outlet Structure, south of Byron Highway, to the connection channels leading to the existing Banks PP Intake Channel and Jones PP Intake Channel. Each of these parallel 40-feet ID tunnels is approximately 1.6 miles in length.

The compatibility of the tunneling excavation method with anticipated ground conditions is a critical design/construction consideration. Currently, geotechnical information for the proposed tunnel alignment is limited. Once adequate geotechnical investigations have been performed, preliminary design evaluations will refine the recommendations of this CER for tunnel excavation and support methods. Based on available geotechnical data collected to date, it is anticipated that pressurized-face tunnel-boring machines will be utilized, either Earth Pressure Balance (EPB) or Slurry machines.

ES.2.3 Intermediate Forebay
The Intermediate Forebay is on the Glanville Tract, east of the Pearson District and west of Interstate 5. The Intermediate Forebay serves as an atmospheric break in the system from the inlet of the dual Main Tunnels. This break in the system allows the flows from each Intake to merge and distribute equally to each of the Main Tunnels, improving operational stability in the Byron Tract pumping plant, and allows independent operation of each of the North Tunnels and the Main Tunnels. The Intermediate Forebay also provides a location to dampen hydraulic surges that may occur if the BTF pumping facilities unexpectedly shutdown, such as following a power outage at the pump plant.
The Intermediate Forebay has no regulating gates controlling gravitational flow to the Main Tunnel. Instead, all flow control is done at the Intake Facilities. Therefore, no daily operational storage is necessary at IF beyond what is necessary to accommodate water surface changes at the downstream Byron Tract Forebay. The IF at bottom elevation -20 feet comprises of 28 acres, which is the smallest practical size to allow construction of the inlet and outlet structures and to provide sufficient reduction in velocity to capture sand-sized sediment not otherwise captured at the Intake Facilities.

**ES.2.4 Byron Tract Forebay**

The BTF provides the daily operational storage required to equalize and balance differences between the south Delta inflow and water exported by the SWP and CVP pumps. Depending on the operating water levels (maximum and minimum), the BTF has been preliminary sized for an operational storage capacity range of approximately 5,960 to 8,350 acre-feet (AF), with an approximate water storage surface area of nominally 810 acres, depending on depth.

Existing operating conditions at the exporting pumping plants would limit that the normal BTF operating range to approximately 10 feet (elevation +2.5 to +12.5 feet). This operating range results in a maximum of approximately 8,350 AF of potential active storage in BTF. The maximum water surface elevation (WSE) at BTF can be lowered to reduce the active storage in the forebay. For example, an operating range of +2.5 to +10 feet would result in approximately 5,960 acre-feet of active storage.

BTF is connected to a conveyance system that includes tunnels and open channels that lead to the existing export pumping stations.

**ES.2.5 Clifton Court Forebay**

The WaterFix BTO does not include any revisions to Clifton Court Forebay itself. Modifications will be made to some of the existing outlet channels from this forebay to ensure that both isolated North Delta, isolated south Delta, and dual-mode operation of the facilities can be achieved. In cases where the Banks PP is pumping water from the south Delta diversions, Clifton Court will be utilized within its current configuration which includes a gated entrance from the West Canal, the main body of water, and the Skinner fish facility.

**ES.2.6 Conveyance System to Existing Export Pumping Plants**

From BTF, two 40 feet ID tunnels and connection channels will convey flow to the existing export pumping plants. The two 40 feet ID tunnels are approximately 1.6 miles in length each and terminate at an outlet structure on the connection channels. The connection channel that will be located between the tunnel outlet structure and the Banks PP Intake Channel is approximately 0.5 miles in length and is designed to convey a maximum flow of 10,670 cfs. The connection channel between the tunnel outlet structure and the Jones PP Intake Channel is approximately 0.9 miles in length and is designed to convey a maximum flow of 4,600 cfs.

**ES.2.7 Head of Old River – Operable Barrier**

An Operable Barrier with control gates at the Head of Old River will reduce migration of San Joaquin River watershed salmonids into the South Delta through the Old River. The barrier is located where the San Joaquin River and Old River diverge. The barrier is approximately 210 feet long and 30 feet wide. It consists of five independent 125 foot bottom-hinged gates, with fish passage structure, boat lock with gates at each end, control building, boat lock operator’s building, and communications antenna, as well as floating and pile-supported warning signs, water level recorders, and navigation lights.

**ES.3 Other Design Considerations**

**ES.3.1 Flood Protection Considerations**

The WaterFix BTO is engineered to withstand flood water levels from the following potential sources:
EXECUTIVE SUMMARY

- 200-year return flood event in the Sacramento, Mokelumne, San Joaquin Rivers, or adjacent sloughs
- Inundation of floodplain from a 200-year return flood event with levee breach
- Wind-induced waves
- Mean high-water tides
- Sea Level Rise (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.

The estimated flood levels to be used in the design for each conveyance option facility are currently being developed. All flood protection levels must be confirmed and refined during subsequent study.

**ES.3.2 Seismic Considerations**

The WaterFix BTO facilities are critical to transporting water from north of the Delta to south of the Delta. Long delays could have significant negative impact on human life and the California economy. The facilities will be designed as described in ASCE 7 for critical facilities. Facilities that require extended time frames for repair/replacement (e.g. large pumps or tunnel structures) will be designed with the highest seismic standard to prevent prolonged delays.

DWR’s Water Resources Engineering Memorandum No. 70 (WREM-70) prescribes minimum Seismic Loading Criteria for the State Water Project, and provides different seismic loading criteria based on criticality of a facility. The WREM-70 allows flexibility so that the project engineer can judge which criteria to use.

The seismic loading selected for each facility is based on

- Consequences of failure,
- Criticality of the structure for water delivery, and
- Downtime and cost for the repair of the facility.

Generally, higher seismic loading criteria are recommended for structures that have greater

- Consequences of failure,
- Importance for water delivery,
- Downtimes, and/or
- Costs for repair.

Conversely, lower seismic loading criteria are recommended for structures that are not critical to water delivery and/or can be repaired in a reasonable time frame. For instance, canals can be repaired within a reasonable time frame compared to tunnels or the large pumps of a pumping plant.

Project specific seismic criteria are recommended to satisfy the needs and operation requirements. Some of the seismic criteria are slightly modified from the criteria recommended in WREM-70. Section 4 summarizes the seismic loading criteria recommended for the facilities.

**ES.3.3 Hydraulic Calculations**

Hydraulic modeling of the WaterFix BTO system was employed to establish the system head curves to support Intake Facilities, North Tunnel inside diameter, Intermediate Forebay elevations, and pump selection to determine pressures within the system (DWR, 2014).

**ES.3.4 Instrumentation and Controls**

Each WaterFix BTO facility site will include control and monitoring equipment. A Supervisory Control and Data Acquisition (SCADA) system provides for local and remote automatic and manual control and monitoring.
Currently, the communications system is planned to be implemented using some combination of fiber optic cable system(s), microwave radio, and/or leased telecommunications lines. The communications system will connect to the Delta Field Division Operations and Maintenance (O&M) Center at the south end of the project and the Joint Operations Center in Sacramento at the north end of the project.

**ES.3.5 Electrical Load and Supply**

Reliable supply of electrical power is required during construction of WaterFix BTO tunnels, and for WaterFix BTO operation. The peak intake pumping demand during operation of the system is estimated at approximately 60 megavolt-amperes (MVA). The electrical power demand for the construction of the North and Main Tunnel systems is estimated at 242 MVA peak.

It is anticipated that the utility interconnection facilities needed to connect the project to the electrical grid and the electrical power needed for most of the conveyance facilities (the largest electrical demand will be for operation of the TBMs) will be procured in time to support construction and operation of the facilities. However, it is possible that utility grid power will not be available in time to support critical path activities, particularly shaft pad construction and shaft sinking work that will precede the tunnel construction work. Therefore, the interim use of onsite generators as the power source for shaft sinking activities is anticipated. As soon as construction of the utility grid power is completed, electricity from the interim onsite generators will no longer be used and a tie-in to the utility grid will occur. On-site generators will not be used to power the TBMs.

The high voltage power supply required to operate the TBMs will be provided by Sacramento Municipal Utility District (SMUD), and Western Area Power Administration (WAPA). SMUD will provide temporary power supplies for TBMs in the northern portion of the project, roughly from the Intermediate Forebay to the three intakes, with new power transmission lines and equipment originating at its Franklin substation. WAPA will provide temporary TBM power from its Tracy substation up to Bouldin Island shaft locations, with power drops along the tunnel alignment. WAPA will also provide permanent operational power to the Byron Tract Forebay pumping plant.

There are multiple interconnection options available to the electrical grid and supply for both the operation and construction electrical power. Preliminary studies show some reinforcements and upgrades to the existing transmission grid will be needed to accommodate the large construction power requirements for this project. Because the service construction locations are spread over a distance of more than 40 miles, interconnection to more than one electric utility transmission service provider is possible and would need to be closely coordinated with the utilities.

**ES.4 Construction Considerations**

**ES.4.1 Borrow Sites**

Borrow materials will be required for forebay and overflow containment area embankments at the Intermediate Forebay, Byron Tract Forebay, Intake Facility site fill, tunnel shaft site fill pads, and other features. The primary borrow material needed will be soil suitable for use as engineered embankment fill, but rock, gravel, and sand will also be required.

At this point in project development, sufficient geotechnical information is not available to fully assess the suitability of borrow areas near the WaterFix BTO alignment to determine if adequate quantities of borrow material are actually available. However, several potential borrow sites are specifically identified in this CER that may be able to meet all, or some, of the borrow requirements at the various facility sites. These are shown in CER Volume 3 (Map Book). Also, several commercial borrow sites are available in the general vicinity of the project alignment and could be used. Additional explorations, land ownership considerations, and engineering analyses are needed to better define the actual borrow sites and associated borrow quantities that will be used for the work. Borrow material can be transported over land by truck or earth moving equipment and over water by barge.
ES.4.2 Excavated Material Disposal

Large volumes (approximately 25 million cy) of reusable tunnel material (RTM), consisting of saturated soils mixed with bio-degradable polymers, will be generated by tunneling operations. Large volumes (approximately 4.5 million cy) of excavated material is also expected to be removed from connection channels downstream of BTF that connect to both the Banks PP and Jones PP Intake Channels. Smaller quantities of excess excavated materials are expected at other facility sites, including about 1.9 million cy at the Intermediate Forebay, approximately 1.6 million cy at each intake site, and approximately 1 million cy from launch and reception shafts. These excavated materials may be used for engineered fill material, or will be stockpiled in the locations shown on Volume 2: Concept Drawings.

Much of the area surrounding the proposed alignment consists of low-lying floodplain developed as agricultural land. Depending on the properties of the soils, some predominantly organic soils may be deposited on portions of this land without adversely affecting its agricultural use. RTM will be temporarily spoiled at designated sites adjacent to the tunnel construction work areas and then transported to the opportune reuse location. Possible reuses include strengthening levees, raising subsiding Delta islands, and restoring natural habitats, among other uses. Excess excavated material from BTF areas will be disposed of in an adjacent common disposal/borrow area on Byron Tract, northwest of BTF. Unsuitable excess excavated material at the northern WaterFix BTO facility sites (Intake Facilities, North Tunnels, and Intermediate Forebay) will be disposed in the designated RTM disposal area near or at the Intermediate Forebay. Suitable materials may be reused for the fill pads of the Intake Facilities.

ES.4.3 Approach to Construction

In general terms, a conceptual schedule has been developed to provide guidance on potential length and sequencing of design and construction activities. An overall duration of approximately 15 years is estimated for project implementation from the beginning of preliminary engineering to the completion and commissioning of the system. Details of construction scheduling and overall approach to construction are more closely detailed in Section 24 (Construction Methods, Sequencing, Constructability and Schedule). A refined and detailed construction schedule will be developed during the preliminary engineering phase of the program.

For the purposes of this CER, and considering an estimated construction schedule, using industry average tunneling rates for the anticipated ground conditions, and including contracting considerations, 10 to 12 TBMs may be in simultaneous operation along the conveyance alignment at the peak of tunnel construction activities.

The lead time required to design, build, and transport new TBMs is approximately one year, with an additional six months provided in the project schedule to allow for fabrication at the site and a small delay contingency. The number and size of TBMs required for this project in the timescale proposed may be impacted by availability of specialty equipment and personnel with adequate experience to use the equipment to the standard required. Further study is required into how current and likely market capacity will dictate schedule and procurement strategies. See Table ES-3, located at this end of this section, for a summary of the WaterFix BTO schedule.

ES.4.4 Impact to Existing Facilities

This section discusses existing features that will be modified by WaterFix BTO construction. The final EIR/EIS addresses impacts to existing facilities due to construction traffic and construction activities in the vicinity.

ES.4.4.1 Highways

Highway 160 (SR 160), a state highway, will be modified by WaterFix BTO construction activities at each of the three Intake Facility sites. During the initial portion of the construction phase, which includes widening and raising the levee crest at each intake site, the highway will be relocated from its current alignment along the top of the river levee to a new alignment established on the top of the widened levee approximately 220 feet farther inland from the river.

Other state highways, plus county and private roads, will be impacted by construction traffic. Temporary improvements of these roads may be needed prior to the start of construction. Restoration of these roads may be
necessary after the work is complete. No other significant relocations have been identified. These impacts are discussed in the final EIR/EIS.

**ES.4.4.2 Levees**

Significant modifications to Project levees at each of the three intake sites will be required by the WaterFix BTO. All work on the Sacramento River levees will be conducted in accordance with Central Valley Flood Protection Board (CVFPB) and U.S. Army Corps of Engineers (USACE) requirements. In some locations along the conveyance alignment, other levee roads will be utilized to access work sites. In these locations, project planners and designers will closely coordinate with the affected reclamation districts to ensure that construction activities do not negatively impact levees and levee roads. The new facilities will interface with the Sacramento River levee regulated by the CVFPB and USACE at each intake site as follows:

- The levee will be widened on the land-side to increase the crest width, facilitate intake construction, provide a pad for the new facilities, and accommodate the Highway 160 realignment.
- An on-bank intake structure will be constructed partially in the levee.
- A series of gravity flow collector box conduits will carry flow through the levee prism to the land-side facilities.
- A cutoff wall will be installed in the levee section to help control seepage during construction and long-term operations.

**ES.5 Construction Schedules**

The construction schedule for the WaterFix BTO components is shown on Table ES-3.

*Table ES-3: Projected Construction Schedule for WaterFix BTO Facility Components*

<table>
<thead>
<tr>
<th>Facility Component</th>
<th>Month</th>
<th>Year</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Procurement</strong></td>
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</tr>
<tr>
<td>Pumps</td>
<td>January</td>
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<td>824</td>
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<tr>
<td><strong>Construction</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Site Prep, Roads, Barges, Utilities</td>
<td>April</td>
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<td>500</td>
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<tr>
<td>Temporary Power</td>
<td>May</td>
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<tr>
<td>Intakes</td>
<td>August</td>
<td>5</td>
<td>2207</td>
</tr>
<tr>
<td>North tunnels</td>
<td>July</td>
<td>4</td>
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<tr>
<td>Intermediate Forebay</td>
<td>July</td>
<td>8</td>
<td>1662</td>
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<tr>
<td>Main Tunnels</td>
<td>January</td>
<td>3</td>
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<tr>
<td>Pumping Plants</td>
<td>October</td>
<td>3</td>
<td>2905</td>
</tr>
<tr>
<td>Byron Tract Forebay</td>
<td>January</td>
<td>3</td>
<td>895</td>
</tr>
<tr>
<td>South Tunnels</td>
<td>July</td>
<td>6</td>
<td>700</td>
</tr>
<tr>
<td>Connection Channel to Export Facilities</td>
<td>March</td>
<td>8</td>
<td>550</td>
</tr>
<tr>
<td>Permanent Power</td>
<td>Feb</td>
<td>9</td>
<td>255</td>
</tr>
<tr>
<td>Start Up Commissioning</td>
<td>October</td>
<td>13</td>
<td>250</td>
</tr>
</tbody>
</table>

Some duration includes float days since some activities rely on the completion of other
SECTION 1.0
Introduction

1.1 Program Overview

This document describes the conceptual engineering details that make up the California WaterFix Byron Tract Option (WaterFix BTO) facilities as part of the 2018 Supplemental EIR/EIS for the program. The WaterFix BTO facilities provide an isolated method for conveying water from North Delta diversions to the Banks and Jones export pumps in the South Delta.

The material herein describes the conceptual engineering of conveyance facilities for the WaterFix BTO and is consistent with the 2018 Supplemental EIR/EIS on this same subject. The conceptual engineering report represents a 5-10% complete engineering design effort on the included facilities.

1.2 Purpose and Scope of Conveyance Option

The WaterFix BTO will convey up to 9,000 cubic feet per second (cfs) of Sacramento River water through an underground conveyance system from North Delta Diversions, across the Delta to export pumping plants in the South Delta. The purpose of the overall system is to gain a sustainable and reliable water supply capable of withstanding earthquakes and negative climate conditions.

The WaterFix BTO includes the following components:

- Three North Delta Intake Facilities:
  - Screened on-bank intake structures and,
  - Appurtenant facilities
- North tunnels
- Intermediate Forebay
- Dual Main Tunnels
- BTF Pumping Plant
- Byron Tract Forebay
- South Tunnels
- Connection Channels
- Head of Old River Control Gates – Operable Barrier

The WaterFix BTO is designed to:

- Be isolated from existing rivers and sloughs when conveying water from the Sacramento River to the export pumping plants.
- Divert water via fish screened intakes only from the Sacramento River in the North Delta.
- Deliver fish-free water to both exporting pumping plant approach canals downstream of their respective fish collection facilities.
- Protect conveyance facilities against damage from a 200-year flood event and sea level rise as a result of climate change.
- Have a design level of ground motion for project facilities from a seismic event in accordance with Section 4 of this CER.
1.3 Report Organization

Volume 1 of this report consists of the sections listed below, with figures (graphics) and tables presented within the text:

- Section 2.0: Background information.
- Section 3.0: Overview of the proposed alignment.
- Section 4.0: Geologic and seismic considerations.
- Section 5.0: Operational description of the existing facilities and the WaterFix BTO.
- Section 6.0: Preliminary Hydraulics of the entire proposed alignment.
- Sections 7.0 - 20.0: Individual facilities within the WaterFix BTO.
- Sections 21.0 - 24.0: Temporary construction operations and facilities.
- Section 25.0: Coordination with the Dual Conveyance Facility and the EIR/EIS.
- Section 26.0: Permits necessary for the work.
- Section 27.0: Architectural Considerations for WaterFix BTO
- Section 28.0: References for the works used in creating this report.

- Appendix A: Geologic and seismic information.
- Appendix B: Conceptual level construction sequencing of Intake Facilities.
- Appendix C: WaterFix BTO Conceptual Construction Schedule.
- Appendix D: Surge Analysis Technical Memorandum.
- Appendix E: Pipe Materials.
- Appendix F: Pipe Floatation Analysis.
- Appendix G: Summary of Tunneling Contractor Workshops
- Appendix H: Hydraulic Analysis Technical Memorandum
- Appendix I: Basis of Conceptual Design for Tunnel Lining Systems
- Appendix J: Alternative Designs for Tunnel Lining Systems
- Appendix K: Potential Water Leakage Rates from Tunnels Technical Memorandum
- Appendix L: DWR Seismic Design Criteria
- Appendix M: Seismic Review of Tunnel Liner Performance

Volumes 2 and 3 (bound separately) of this report contain the following:

- Volume 2: Conceptual Drawings of WaterFix BTO facilities.
- Volume 3: Map Book further detailing facility locations relative to other geographic information related to WaterFix BTO.
SECTION 2.0
Background

2.1 General
Currently, the State Water Project (SWP) and Central Valley Project (CVP) divert water from the Sacramento and San Joaquin Rivers for use by cities and farms in the Central Valley, San Francisco Bay Area, and Southern California. The current method for conveying water to the SWP and the CVP is solely through the South Delta Diversions which take place near the Clifton Court Forebay.

The SWP and CVP facilities include reservoirs on the Sacramento and the San Joaquin River systems, with the rivers themselves being used as conveyance channels. The Delta Cross Channel (DCC), near Walnut Grove, controls the flow of Sacramento River water into the eastern Delta. The water is conveyed by internal Delta channels through the Central Delta to the pumping and fish screening/collection facilities of the SWP and CVP in the South Delta, near the town of Tracy. Maximum pumping capacity of the current SWP and CVP facilities is 10,670 cfs installed capacity and 4,600 cfs, respectively.

2.2 History of Conveyance Option
The current configuration of the WaterFix, as presented in this CER, is the result of several optimization efforts that have taken place over time. The initial conveyance configuration, the Pipeline/Tunnel Option (PTO), was documented in the March 10, 2010 CER (DWR, 2010a) and the subsequent October 2010 Addendum (DWR, 2010b). The original PTO was a 15,000 cfs conveyance facility that consisted of five river intakes, pumping plants at each intake, an Intermediate Pumping Plant (IPP), and other appurtenant facilities. The PTO was later modified to the Modified Pipeline Tunnel Option (MPTO) in 2015 when the maximum design flow of the program was reduced from 15,000 cfs to 9,000 cfs. Additional changes from the PTO at that time included the reduction in the number of screened intakes from five to three, the elimination of the Intermediate Pumping Plant (IPP), increasing the size of the north and main tunnels to provide for gravity flow of the water from the intakes to the Clifton Court Forebay pumping plant. The July 2015 CER documented the MPTO facility configuration included in the 2016 Final EIR/EIS for the program.

The current WaterFix BTO effort represents a further optimization to the MPTO. The main differences between the 2015 MPTO and current WaterFix BTO include consolidating the pumps at a new Byron Tract Forebay (BTF), eliminating modifications to the Clifton Court Forebay and revising certain facility locations to avoid wetlands and sensitive habitat. This WaterFix BTO CER will be utilized in a subsequent 2018 Supplemental EIR/EIS on the program.

This report incorporates the most current recommendations for intake locations, Intermediate Forebay sizing and location, tunnel alignment optimization, consolidated pumping plants, and BTF design. Preferred tunnel alignment and forebay modifications could change pending further environmental and engineering reviews.
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SECTION 3.0
Overview of Conveyance Option

This section is an overview of the WaterFix BTO tunnel conveyance system and general facility locations. Included is a summary of geologic, seismic, and flood protection factors which will be further defined for future design activities.

3.1 Proposed Alignment and Key Components

The WaterFix BTO reaches are shown in Figure 3-1.

The proposed alignment of the conveyance facilities is approximately 44 miles long, extending from the North Delta Intake Facilities to the conveyance project’s pumping plants. As shown in the figure, the alignment includes three river Intake Facilities with on-bank intake structures and sedimentation basins; the North Tunnels which consist of three reaches totaling approximately 14 miles that connect the Intake Facilities to the Intermediate Forebay (IF); two parallel Main Tunnels, each approximately 30 miles long; an IF; a pumping plant located adjacent to a operational storage forebay on Bryon Tract, called Byron Tract Forebay (BTF). At the outlet of BTF, two 40’ inner diameter (ID) tunnels will carry water to an outlet structure, where the water will then be diverted to both the Banks and Jones Pumping Plant Intake Channels via two open channels.

Two North Tunnel segments, one originating at Intake No. 2 and terminating at a junction structure near Intake No. 3 and the other originating at Intake No. 5 and terminating at the IF, are single-bore, 28-foot ID tunnels. The North Tunnel connecting the junction structure near Intake No. 3 to the IF is a single-bore, 40-foot ID tunnel. The diameters of these tunnels are preliminary and will be optimized during the preliminary design phase of the project once geotechnical and topographical information is received.

The Main Tunnels are twin-bore, 40-foot ID tunnels. Each tunnel will have features that will be capable of isolating specific sections for dewatering prior to maintenance and inspections. The diameters of these tunnels are preliminary and will be optimized during the final design phase of the project once geotechnical and topographical information is received.

The pumping plant located at the terminus of the Main Tunnels will serve to lift water into an operational storage forebay – BTF. From BTF, two 40-foot ID tunnels will carry water to an outlet structure located south of Byron Highway. Downstream of the outlet structure, water will be diverted to both Banks and Jones Intake Pumping Plans via two open channels. The two open channels will be sized for the maximum export capacity of each respective downstream pumping plant, which for Banks is 10,670 cfs and for Jones is 4,600 cfs. Flow through each open channel will be metered downstream of the outlet structure.

The components of the conveyance system are described below.

3.2 Conveyance Reach Descriptions

Based on proposed tunnel drive configuration, WaterFix BTO is divided into a total of eight separate tunnel segments or “reaches” as shown in Figure 3-1 and described below. There are also two additional reaches comprised of open channels, which are both located directly upstream of each export pumping plant intake channel.

3.2.1 Reach 1: Intake No. 2 to Intake No. 3

Reach 1 starts at Intake No. 2 and ends with the junction structure at Intake No. 3. Intake No. 2 is on the east side of the Sacramento River, approximately 1 mile north of Hood and approximately 1.5 miles west of Interstate 5 (I-5). Water is diverted from the Sacramento River into Intake No. 2 and then flows toward the southeast through a 28-foot ID tunnel. This 28-foot ID tunnel extends approximately 2 miles from the tunnel access shaft to the junction structure at Intake No. 3.
3.2.2 Reach 2: Intake No. 3 to Intermediate Forebay
Reach 2 begins at Intake No. 3 and ends at the IF inlet shaft. Intake No. 3 is on the east side of the Sacramento River, 1.5 miles north of town of Hood and approximately 3 miles west of I-5. This facility conveys the water directly into the junction structure. From the junction structure, a 40-foot ID tunnel extends approximately 6.8 miles south to the inlet shaft at the IF on Glanville Tract.

3.2.3 Reach 3: Intake No. 5 to Intermediate Forebay
Reach 3 begins at Intake No. 5 and ends at the IF inlet shaft. Intake No. 5 is on the east side of the Sacramento River, 1.5 miles south of town of Hood, and approximately 3 miles west of I-5. This facility conveys water into a 28-foot ID tunnel that extends approximately 4.8 miles south to an IF inlet shaft at the IF on Glanville Tract.

3.2.4 Reach 4: Intermediate Forebay to Staten Island
Reach 4 starts on the Glanville Tract adjacent to the IF and is comprised of approximately 5.6 miles of the twin 40’ ID bore tunnels from the IF to Staten Island. Reach 4 ends at the construction shafts on Staten Island, approximately 3 miles southeast of the community of Walnut Grove and just east of the Sacramento River.

3.2.5 Reach 5: Staten Island to Bouldin Island
In Reach 5, the Main Tunnels extend approximately 7.2 miles nearly due south from the Staten Island construction shafts to construction shafts north of Potato Slough and south of Highway 12 (SR-12) on Bouldin Island.

3.2.6 Reach 6: Bouldin Island to Bacon Island
In Reach 6, the Main Tunnels extend approximately 8.9 miles southwest beneath Venice Island and nearly due south beneath Mandeville and Bacon Islands from the Bouldin Island shafts to reception shafts about 2.4 miles south of the Old River Connection Slough on Bacon Island.

3.2.7 Reach 7: Bacon Island to Byron Tract Forebay (BTF)
Reach 7, approximately 8.3 miles long, is the final section of the Main Tunnels. Drive shafts will be located at the BTFPP site, just outside the northwest corner of the existing CCF, on the opposite side of the Italian Slough from CCF. The drive shafts are located on Byron Tract.

3.2.8 Reach 8: BTF Outlet Structure to South Tunnel Outlet Structure
Reach 8 includes the two 40’ ID South Tunnels that originate at the outlet of BTF and terminate at an outlet structure south of Byron Highway. Each tunnel is approximately 1.6 miles long.

3.2.9 Reach 9: South Tunnel Structure to Connection with State Water Project Export Pumping Plants
Reach 9 is an open channel that originates at the outlet structure south of Byron Highway and terminates at a connection point with the Banks Pumping Plant Intake Channel. This channel will be approximately 0.5 miles long and will be sized to carry the maximum installed capacity of Banks Pumping Plant (10,670 cfs).

3.2.10 Reach 10: South Tunnel Structure to Connection with Central Valley Project Export Pumping Plants
Reach 10 is an open channel that originates at the outlet structure south of Byron Highway and terminates at a connection point with the Jones Pumping Plant Intake Channel. This channel will be approximately 0.9 miles long and will be sized to carry the maximum export capacity of Jones Pumping Plant (4,600 cfs).
Figure 3-1  Location of WaterFix BTO Reaches
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SECTION 4.0
Geologic and Seismic Considerations

4.1 Geologic Considerations

This section provides a general overview of the geology that exists in the Sacramento-San Joaquin River Delta. The Delta is 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 (refer to Appendix A, Figure A-1 and Figure A-2), approximately 435 miles long and up to 62 miles 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 mega-annum) through recent sediments up to about 5.5 miles 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, the relative sea level was approximately 300 feet lower than present-day sea level, and the location of the present-day Delta formed part of an 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 SLR was rapid, outstripping 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 SLR balanced by vertical marsh growth through biomass accumulation and sediment deposition (DWR, 2010a).

4.1.1 Project Area Geologic Units

Geologic units 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. 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. For further discussion, consult Appendix A (Geology and Seismicity).

4.1.1.1 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 and peat, and sands.

4.1.1.2 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 geologic unit is mapped only on broad natural levees and crevasses 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).

4.1.1.3 Floodplain Deposits (Holocene)
Atwater (1982) mapped these 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 since been 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.

4.1.1.4 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.

4.1.1.5 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 deposits (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.

4.1.1.6 Dune Sands (Pleistocene to Holocene)
These deposits consist of poorly graded 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 late 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.

4.1.1.7 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.

4.1.1.8 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.
4.1.2 Geological and Geotechnical Information Collected from Recent Investigations and Deep Seismic Survey

Most of the historically available geotechnical data collected in the Delta were generally limited to the Holocene units described above and Upper Pleistocene units that are present at shallow depths of less than approximately 100 feet. From May 2009 to the present time, approximately 236 borings and cone penetrometer test (CPT) soundings have been advanced at the intakes and forebays and along the various conveyance alignments. The subsurface exploration depth varied from 37 feet up to 520 feet below the existing ground surface, with 95 percent of the explorations reaching depths between 100 to 200 feet. The suspension P-S velocity logging method was used to collect compression and shear wave velocities to a maximum depth of approximately 500 feet in five borings located in the northern, central, and southern portions of the various conveyance alignments. In addition, boring diameter, normal resistivity, single point resistance, spontaneous potential, and natural gamma were obtained. Additional borings are planned.

The CPTs were generally paired with adjacent borings. Shear wave velocity was measured at 5-foot intervals in many of the CPT soundings to provide the variation in shear wave velocity with depth within the upper 200 feet below the ground surface. The CPTs also provided an approximate correlation to the undrained shear strength of fine-grained soil and the internal friction angle of sands.

Figures 4-2 through 4-6 present the subsurface conditions encountered in a number of the more recent investigations. The exploratory borings from these investigations are situated primarily along the 2010 PTO alignment, which is similar to the WaterFix BTO alignment except for adjustments made to route the tunnels to the BTF and through the relocated Bouldin Island shafts and the North Tunnel ventilation shafts. The WaterFix BTO alignment is on the same side of CCF as the 2010 PTO alignment but terminates at the BTF, and is easterly of the previous alignment between the Bouldin Island shafts and the North Tunnel ventilation shafts.

In addition to the recent borings, a DWR seismic stability evaluation for the Delta levees was performed that required accelerometers to be installed in deep borings (DWR, 1993). This study included seven 300- to 500-foot-deep boring locations with down-hole suspension P- and shear-wave velocity logs. The seismic survey locations of interest to the WaterFix BTO cover the areas of Staten Island, Bacon Island, and Clifton Court Forebay (CCF). Geotechnical data, such as split-spoon sampling and limited laboratory testing, were also conducted at the survey locations. These data, together with those collected from the project investigations, provide a conceptual level geological and geotechnical characterization of the Delta substratum for the WaterFix BTO. Additional borings, CPTs and other geophysical explorations and tests are planned for preliminary and final design.

In general, the Holocene deposits of soft mineral/organic soils and peaty material of the floodplain deposits and tidal marshes were encountered up to 60 feet below ground surface (bgs) within the Delta. The Holocene materials are generally characterized as organic soil or very soft to medium stiff silty clay with medium dense silty sand and poorly graded sand (Figures 4-2 through 4-6).

The deeper alluvium of probable Upper and Middle Pleistocene age (11,700 to 781,000 years before AD 2000) are generally characterized by dense to very dense silty sand, poorly graded sand, and very stiff to hard silty clay and clayey silt. Deeper clayey soils are very stiff to hard. A few isolated outcrops of eolian sands, identified as being part of the upper member of the Modesto formation, are present near the town of Hood and on Staten Island. These sands are loose to medium dense near the surface, becoming dense to very dense with depth. Drill hole DCA-DH-004 was drilled on one of these outcrops on Staten Island (See Figure 4-3).

Tephra, comprised of volcanic ash and pumice, was identified in several borings north from Intake 3 south to Bouldin Island. With the help of the USGS Tephrochronology Laboratory of Menlo Park, California, these tephra samples were identified as three separate events ranging in age from 575,000 to 200,000 years before present. The tephra at Intake 3 was determined to be about 575,000 years old and was encountered at an elevation of about -120 feet. Tephra encountered on the northwest side of Staten Island was determined to be about 400,000 years old at an elevation of about -187 feet, and tephra encountered on the south end of Staten Island has been preliminarily determined to be about 200,000 years old and was encountered at an elevation of about -140 feet. This 200,000 year old tephra was also encountered between Bouldin and Venice Island at an elevation of
4.0 GEOLOGIC AND SEISMIC HAZARDS

4.1 Geologic Hazards

About 166 feet. Based on this data, there is an apparent thickening of the Upper Pleistocene and Holocene sediments towards the middle of the Delta.

Because of the alluvial nature of the depositional environment at the proposed tunnel grade, lateral and vertical changes from silty clay to clayey silt to silty sand, and fine- to coarse-grained sand, should be anticipated over short distances.

4.1.3 Collection of Additional Geological and Geotechnical Information

As part of the program’s preliminary and final design phase, a series of geotechnical investigations will be performed along the tunnel alignments and at locations proposed for other conveyance facilities. The proposed exploration is designed as a two-part program (Phases 2a and 2b) to collect geotechnical data. The two-part program will allow refinement of the second part of the program to respond to findings from the first part. The exploration will include field and laboratory testing of soil samples. Phase 2a exploration will focus on collecting data to support preliminary engineering and Phase 2b exploration will support final design, and planning for procurement and construction-related activities. The land-based portion of the proposed Phase 2a and 2b exploration will occur at approximately 1,430 locations. The overwater portion of the proposed Phase 2a and 2b exploration will occur at approximately 100 locations.

The field tests will consist of auger and mud-rotary drilling with soil sampling using a standard penetration test (split spoon sampler) and Shelby tubes; cone penetration testing, geophysical testing; pressure meter testing; installation of piezometers and groundwater extraction wells; dissolved gas sampling; aquifer testing; and excavation of test pits. The field exploration program will evaluate soil characteristics and collect samples for laboratory testing. Laboratory tests will include soil index properties, strength, compressibility, permeability, and specialty testing to support tunnel boring machine selection and performance specification.

4.2 Seismic Hazard Design Considerations

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 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, such as those in the Coast Ranges-Sierran Block boundary zone. For further discussion of seismic hazards, refer to Appendix A: The Delta Risk Management Study (DRMS) (DWR 2007). The seismic hazards portions of the study will be updated during preliminary engineering phase of the WaterFix Program. Findings from this update will be applied to the design of the WaterFix facilities.

4.2.1 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 (Appendix A, Figure A-5). 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) 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 & Associates, Inc. and URS Corporation (URS) for DRMS (DWR, 2007).

“Blind” faults beneath the Delta and the West Tracy and Vernalis faults, part of the Coast Ranges-Sierran Block (Wong et al., 1988), are of particular significance to the assessment of seismic hazards in the Delta. The potential Delta seismic sources include the Northern Midland zone, the Southern Midland fault, the Thornton Arch zone, the West Tracy fault, and the Montezuma Hills source zone (Appendix A, Figure A-5). As is the case for many “blind” faults, the characterization of the Delta seismic sources is highly uncertain because of very limited available data. What is known about these sources primarily has come from subsurface seismic data.
4.2.1.1 Probabilistic Seismic Hazard Analyses

In the DRMS study, URS performed a Probabilistic Seismic Hazard Analysis (PSHA) 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, 2007). The PSHA approach uses the elements of the Deterministic Analysis and adds an assessment of the likelihood that ground motions will occur during a specified time period. The probability (frequency of occurrence) of different magnitude earthquakes on each significant seismic source and inherent uncertainties are directly accounted for in the analysis. The results of a PSHA are used to select the site ground motions based on the acceptable probability of exceedance during the service life of the structure for a given return period. Time-dependent seismic hazard results were computed at six sites in the Delta for the years of 2005, 2050, 2100, and 2200. Time-dependent probabilistic ground shaking hazard maps for 500-year return periods were developed for the Delta area (Appendix A, Figure A-7). The map is for peak ground (horizontal) acceleration (PGA) and a stiff soil site condition, Site Class D as defined in American Society of Civil Engineers (ASCE) 7, Chapter 20. An important point is that these maps are for a uniform site condition, so site response effects are not apparent on these maps.

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 (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 percent, and 10 percent in 50 years (return periods of 2,475 years, 975 years, and 475 years, respectively). The maps are for a firm rock site condition (National Earthquake Hazards Reduction Program site class B/C), making a direct comparison with the firm soil results of the DRMS study not possible. The USGS values for a 500-year return period in the project area 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, 2007). The difference can be attributed to site amplification of the soil versus the USGS firm rock ground motions. The DRMS earthquake ground motions were also compared to an earlier DWR study and to a 2000 California Bay-Delta Authority (CALFED) Bay-Delta Program study. The results for the 200-year return period event were found to be very comparable.

In the DRMS study, the seismic hazards for 500-year and 1,000-year return period PGAs are provided for six different site-specific locations in the Delta in the form of seismic hazard curves for the mean, median, and 85th percentile (mean plus 1 standard deviation) ground motions. A seismic hazard PGA contour map is also provided. The probabilistic 500- and 1,000-year PGA design values for WaterFix BTO facility locations were determined by taking the nearest site-specific PGA values from the seismic hazard curves presented in the DRMS study and multiplying them by the ratio of the PGA at the facility location to the PGA at the site-specific location. This is an approximate method, but sufficient for conceptual level design.

The resulting preliminary probabilistic seismic ground motions are summarized in Table 4-1 for the different project facility locations in order of north to south:

<table>
<thead>
<tr>
<th>Location</th>
<th>500-year PGA</th>
<th>500-year PGA</th>
<th>1,000-year PGA</th>
<th>1,000-year PGA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mean</td>
<td>85th %</td>
<td>mean</td>
<td>85th %</td>
</tr>
<tr>
<td>Intake No. 2</td>
<td>0.23</td>
<td>0.25</td>
<td>0.27</td>
<td>0.30</td>
</tr>
<tr>
<td>Intake No. 3/Junction</td>
<td>0.23</td>
<td>0.25</td>
<td>0.27</td>
<td>0.30</td>
</tr>
<tr>
<td>Structure</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intake No. 5</td>
<td>0.24</td>
<td>0.26</td>
<td>0.29</td>
<td>0.32</td>
</tr>
<tr>
<td>Intermediate Forebay</td>
<td>0.24</td>
<td>0.26</td>
<td>0.29</td>
<td>0.31</td>
</tr>
<tr>
<td>Staten Island Reception</td>
<td>0.27</td>
<td>0.29</td>
<td>0.32</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Shafts
Bouldin Island Drive Shafts 0.31 0.33 0.37 0.40
Bacon Island Reception Shafts 0.35 0.40 0.45 0.51
Clifton Court Forebay (Byron Tract Forebay) 0.40 0.45 0.50 0.57

* Stiff Soil, Site Class D was assumed for each location.

Notes:
% = percent(ile)
PGA = peak ground acceleration

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

The preliminary probabilistic ground motions provided above represent the PGA at the ground surface, underlain by stiff soil conditions. These ground motions should be confirmed and verified during preliminary and final design based upon facility locations and site-specific subsurface exploration, testing, and ground motion analyses. For ground motions at depth, it can be assumed that the ground motions generally decrease with depth. The attenuation of the ground motion with depth can be determined through site-specific dynamic site response analyses to account for subsurface conditions and site geometry. At 100-foot depth, the horizontal acceleration is estimated to be 70 percent of the ground surface motion (Federal Highway Administration [FHWA], 2009). The proposed depths of the tunnels are between 100 to 200 feet. For the conceptual level design, and in the absence of more rigorous analyses, a value of approximately one-half of the surface PGA was assumed for structural analyses of the buried tunnel linings.

4.2.1.2 Deterministic Seismic Hazard Analyses for Forebay Locations

The IF and CCF embankments will be under the jurisdiction of DSOD, based on the embankment height and water storage volumes exceeding the conditions for a low hazard, non-jurisdictional dam. Per current DSOD guidelines, the design seismic ground motion should be based on a deterministic analysis of nearby fault sources. The deterministic seismic hazard analysis (DSHA) approach uses the known seismic sources that can affect the site along with the available historical seismic and geological data to generate discrete, single-valued events or models of ground motion at the site. Typically, one or more earthquakes are specified by magnitude and location with respect to the site. Usually, the earthquakes are assumed to occur at the source closest to the site. The site ground motions are estimated deterministically, given the magnitude, source-to-site distance and site conditions. DSHA is the standard process used to estimate the seismic ground motion parameters for the Maximum Credible Earthquake (MCE) described further in this section and is typically conducted using a site-specific analysis. The hazard level for the deterministic analysis is dependent upon the consequences of failure of the dam. Based on the estimated hazard level (moderate, bordering on high) for the forebay embankments, the appropriate statistical level of acceleration for deterministic seismic hazard analyses is between the 50th and 84th percentile, or between the mean and 1 standard deviation above the mean values from attenuation relationships. The actual value used between the DSOD-required statistical range is dependent upon the recurrence interval that is reasonable for the project. A maximum average annual return period of 1,000 years is being used as a ceiling for the forebay deterministic values.

For the deterministic seismic hazard analysis at the forebay locations, PGA values were estimated from the occurrences of earthquakes on the crustal faults near the forebays. For the crustal faults, the next generation attenuation (NGA) attenuation relationships, as developed by Abramson and Silva (2008), Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008), were used to estimate the PGA values. The deterministic PGA values reported herein are the average of the four attenuation relationships. For faults of similar magnitude, only the nearest fault was analyzed, unless the fault type mechanism was different and warranted an evaluation.

Intermediate Forebay: A summary of the considered nearby active faults and preliminary deterministic PGA values resulting from attenuation using the NGA relationships for controlling faults is provided in Table 4-2 for the
IF. Faults with the same or lower magnitude than other faults closer to the project facilities were not analyzed. These PGA values should be confirmed and verified during preliminary and final design.

**Table 4-2:** Summary of Active Faults Surrounding Intermediate Forebay and Deterministic Ground Motions

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Distance to Fault Surface Trace from Project (kilometers and direction)</th>
<th>Characteristic Magnitude&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Slip Rate (mm/year)&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Deterministic Median PGA (g)</th>
<th>Deterministic 84th % PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thornton Arch Zone</td>
<td>0</td>
<td>6.5</td>
<td>0.2</td>
<td>0.36</td>
<td>0.57</td>
</tr>
<tr>
<td>Northern Midland Zone</td>
<td>16 west</td>
<td>6.5</td>
<td>1</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Montezuma Hills Zone</td>
<td>22 southwest</td>
<td>6.5</td>
<td>0.5</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Pittsburg-Kirby Hills Fault</td>
<td>38 northwest</td>
<td>6.7</td>
<td>0.7</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Coast Ranges-Sierran Block</td>
<td>43 west</td>
<td>6.8</td>
<td>2.0</td>
<td>0.14</td>
<td>0.23</td>
</tr>
<tr>
<td>Foothills Fault Zone</td>
<td>52 east</td>
<td>7.0</td>
<td>0.8</td>
<td>0.10</td>
<td>0.17</td>
</tr>
<tr>
<td>Greenville Fault</td>
<td>56 southwest</td>
<td>6.9</td>
<td>6</td>
<td>0.08</td>
<td>0.13</td>
</tr>
<tr>
<td>South Hayward Fault</td>
<td>80 southwest</td>
<td>7.3</td>
<td>9</td>
<td>0.08</td>
<td>0.13</td>
</tr>
<tr>
<td>San Andreas Fault</td>
<td>109 southwest</td>
<td>7.9</td>
<td>24</td>
<td>0.08</td>
<td>0.13</td>
</tr>
</tbody>
</table>

<sup>a</sup> Characteristic magnitudes and slip rates are based on maximum values from the DRMS report (DWR, 2007).

Notes:
- % = percent(ile)
- g = measurement of peak ground acceleration
- mm/year = millimeter(s) per year
- PGA = peak ground acceleration

The largest estimated site acceleration for the IF is from possible active blind faults beneath the Delta.

From Table 4-2, the maximum deterministic mean PGA at the IF is 0.36 g, while the maximum 84th percentile PGA is 0.57 g. From Table 4-1, the 1,000-year 85th percentile probabilistic PGA of 0.31 is lower than the 50th percentile deterministic ground motions. The 84th percentile deterministic ground motions will be used for the conceptual design PGA at the IF.

**Byron Tract Forebay:** A summary of nearby active faults and deterministic PGA values is provided in Table 4-3 for the Byron Tract Forebay. All distances provided in Table 4-3 are from the existing CCF to each specific fault.

The West Tracy fault passes through the BTF area; however, the slip rate and seismic recurrence rate for the West Tracy fault is low. This explains why the probabilistic values presented in Table 4-1 are lower than the deterministic values shown in Table 4-3 for recurrence intervals of less than 1,000 years. The probabilistic 85th percentile, 1,000-year PGA for the CCF (BTF) is 0.57 g, which is bracketed by the 50th and 84th percentile maximum deterministic ground motions of 0.47 g and 0.75 g (as shown in Table 4-3). Therefore, the probabilistic 85th percentile was used as the ground motion for the conceptual design at the CCF (BTF).
### Table 4-3: Summary of Active Faults Surrounding North and South Clifton Court Forebay and Deterministic Ground Motions

<table>
<thead>
<tr>
<th>Fault Name</th>
<th>Distance to Fault Surface Trace from Project (kilometers and direction)</th>
<th>Characteristic Magnitude(a)</th>
<th>Slip Rate (mm/year)(a)</th>
<th>Deterministic Median PGA (g)</th>
<th>Deterministic 84th % PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>West Tracy Fault</td>
<td>0</td>
<td>6.75</td>
<td>0.5</td>
<td>0.47</td>
<td>0.75</td>
</tr>
<tr>
<td>Southern Midland Fault</td>
<td>5 northwest</td>
<td>6.6</td>
<td>1</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Midway/Black Butte Faults</td>
<td>7 southwest</td>
<td>6.75</td>
<td>1</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Vernalis Fault</td>
<td>8 southeast</td>
<td>6.75</td>
<td>0.5</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Greenville Fault</td>
<td>16 southwest</td>
<td>6.9</td>
<td>6</td>
<td>0.21</td>
<td>0.35</td>
</tr>
<tr>
<td>Montezuma Hills Zone</td>
<td>17 northwest</td>
<td>6.5</td>
<td>0.5</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Mt. Diablo – South Fault</td>
<td>25 west</td>
<td>6.7</td>
<td>5</td>
<td>0.27</td>
<td>0.44</td>
</tr>
<tr>
<td>Calaveras Fault</td>
<td>35 southwest</td>
<td>6.9</td>
<td>20</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>Concord/Green Valley Fault</td>
<td>38 northwest</td>
<td>6.7</td>
<td>5</td>
<td>Not analyzed</td>
<td>Not analyzed</td>
</tr>
<tr>
<td>South Hayward Fault</td>
<td>45 southwest</td>
<td>7.3</td>
<td>9</td>
<td>0.13</td>
<td>0.21</td>
</tr>
<tr>
<td>Foothills Fault Zone</td>
<td>73 East</td>
<td>7.0</td>
<td>0.8</td>
<td>0.08</td>
<td>0.13</td>
</tr>
<tr>
<td>San Andreas Fault</td>
<td>76 southwest</td>
<td>7.9</td>
<td>24</td>
<td>0.11</td>
<td>0.19</td>
</tr>
</tbody>
</table>

\(a\) Characteristic magnitudes and slip rates are based on maximum values from the DRMS report (DWR, 2007).

Notes:

- \(\%\) = percent(ile)
- \(g\) = measurement of peak ground acceleration
- \(\text{mm/year}\) = millimeter(s) per year
- \(\text{PGA}\) = peak ground acceleration

#### 4.2.1.3 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 seismic sources described previously, 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, 2007) indicates that the Southern Midland fault may offset the contact between Holocene peat deposits and the underlying sandy deposits by approximately 2 to 4 meters. 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. The risk of surface rupture occurring in the Delta is therefore low.

#### 4.2.1.4 Seismic Design Criteria

For the purposes of developing seismic design criteria, the WaterFix facilities have been assigned a “critical” facility classification. Long delays in water delivery from the north to the south of the Delta could have a significant negative impact on human life and the California economy. The facilities will be designed as described in ASCE 7, which is often cited for the design of critical facilities. ASCE 7 does not specifically address the design of underground facilities like tunnels and shafts. Additionally, facilities that require extended time frames for repair/replacement (e.g. large pumps or tunnel structures) shall be designed with stringent seismic standard to prevent prolonged delays.
DWR’s Water Resources Engineering Memorandum No. 70 (WREM-70) prescribes minimum Seismic Loading Criteria for the State Water Project, and provides specific seismic loading criteria based on criticality of a facility. The WREM-70 allows flexibility so that the project team can assess which criteria is most appropriate to use for a specific project facility. SWP facilities, located throughout California, vary in purpose and criticality. Designing all the facilities to the same seismic standard would not be feasible or cost effective. Therefore, the WREM-70 provides a wide range of seismic loading criteria for various types of facilities.

The seismic loading selected for a facility is based on

- Consequences of failure,
- Criticality of the structure for water delivery, and
- Downtime and cost for the repair of the facility

Generally, higher seismic loading criteria are recommended for structures that have greater

- consequences of failure,
- importance for water delivery,
- downtimes, and/or
- costs for repair

Conversely, lower seismic loading criteria are recommended for structures that are not critical to water delivery and/or can be repaired in a reasonable time frame. For instance, canals can be repaired within a reasonable time frame compared to tunnels or the large pumps of a pumping plant.

Project specific seismic criteria are recommended to satisfy the needs and operation requirements of the WaterFix BTO facilities. Building upon the recommendations of WREM-70, DWR developed draft design standards for the WaterFix facilities. The seismic design portion of these criteria are included in Appendix L. Some of the seismic criteria presented here are slightly modified from the criteria recommended in WREM-70. Table 4-4 summarizes the seismic loading criteria recommended for the WaterFix facilities.

The upper-bound envelope, 84th percentile ground motion from the controlling fault and 975-year average annual return period event, is recommended for dams and reservoirs. Generally, the 84th percentile ground motion would control the design envelope; however, near major active faults the probabilistic ground motion with a 975-year average annual return period can do so. This criterion is recommended for all (including non-jurisdictional) dams and reservoirs.

The conveyance facilities, including pipelines, and tunnels, recommended seismic loading criteria are recommended to have a seismic loading criteria appropriate for a seismic event with a 2,475 year return period. Based on anticipated longer repair time, a higher seismic standard is recommended for pipelines and tunnels than for canals. Repair costs associated with tunnels and pipelines is also greater than repair costs for canals.

Bridges facilities are considered critical for emergency response purposes. Heavy equipment will need to be transported during an emergency. Consistent with off-site highways, seismic design loads for bridges shall follow Caltrans seismic design criteria.

Other facilities including buildings, surface structures, utility crossings, and other non-structural components such as electrical mechanical equipment are critical for water delivery. These types of facilities shall be designed using ASCE 7 with appropriate importance factors and modification factors to the maximum considered earthquake (MCE).

The criteria discussed above are sufficient to guide the conceptual engineering efforts conducted to date. Detailed seismic hazard analyses will be conducted in the early stages of preliminary design of WaterFix facilities. Based on the results of this analysis, the final seismic design criteria for the facilities will be established.
Table 4-4: Recommended Seismic Loading Criteria for the Facilities (Chapter 2 of Design Standards Volume II Design Criteria, June 2013)

<table>
<thead>
<tr>
<th>Facility Type</th>
<th>Seismic Loading</th>
<th>Selection Basis</th>
<th>Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dams, Reservoirs and Forebays</td>
<td>Greater of 84th Percentile and 975-year Return Period (RP)</td>
<td>Modified DSOD method</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Canals and Embankment</td>
<td>975-year RP</td>
<td>Damage consequence and ease of repair</td>
<td>PSHA</td>
</tr>
<tr>
<td>Pipelines</td>
<td>Greater of 84th Percentile and 2475-yr RP</td>
<td>Modified FEMA Function Class IV and difficult to repair</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Tunnels (includes inlets and outlets)</td>
<td>Greater of 84th Percentile and 2475-yr RP</td>
<td>Consideration of criticality and difficulty and cost of repair</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Bridges</td>
<td>Caltrans Seismic Design Spectrum</td>
<td>Caltrans Seismic Design Criteria (SDC 1.6)</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Building and Hydraulic Structures, Utility Overcrossings, and Mechanical and Electrical Equipment’s</td>
<td>Seismic Response Based on ASCE 7 MCE</td>
<td>Modified ASCE 7 Design Spectral Response to Approx. 975-yr RP – Collapse Prevention Performance Level</td>
<td>PSHA with DSHA Cap</td>
</tr>
</tbody>
</table>

Notes:
Seismic load shall be greater or equal to a deterministic spectrum from a 6.5 moment magnitude strike slip fault earthquake at a distance of 12 km
PSHA = Probabilistic Seismic Hazard Analysis
DSHA = Deterministic Seismic Hazard Analysis

4.2.1.5 Tunnel Performance and Seismic Hazard Analyses

Based on past observations of well-designed/constructed tunnels after earthquakes, lined tunnel systems using reinforced concrete have provided reliable seismic performance. Using modern Tunnel Boring Machines (TBMs) as proposed for the current project, the high-quality high-strength concrete segmental liner will provide a high level of seismic resilience. Given the critical nature of water conveyance during and after a major seismic event, the tunnels will be designed to withstand stresses and accommodate ground displacements associated with Maximum Considered Earthquakes (MCE).

MCE is defined as the ground motions associated with an extreme event which corresponds to the 84-percentile of a Deterministic Earthquake event or a 2,475-year return period of a Probabilistic Earthquake (2% probability of exceedance in 50 years), whichever is higher. Under the performance criteria of MCE, the concrete segmental liner will be structurally intact without any collapse (see Appendix M: Seismic Review of Liner Performance). Uncontrolled discharge of water or ground loss induced by the tunnels will not occur because of the high-strength segmental liner that will resist the thrust, flexural and shear stresses associated with the maximum earthquakes. The gasketed joints are pre-compressed during tunnel construction to assure water sealing capability and to account for future seismic movements. During a major seismic event, the gaskets will accommodate the maximum ground movements while they will not relax to cause any uncontrolled leakage. Minor cosmetic spalling or hairline cracks may occur after the extreme event, but the tunnels will survive the earthquake and continue to convey water. The tunnel(s) can be inspected and repaired to bring the liner to the original conditions. Further studies to optimize the segmental liner performance will be conducted during the preliminary design phase of the project. These studies will include, but are not limited to: hydraulics analyses, site specific
geologic and geotechnical investigations, seismic hazard analyses, gasket performance, structural design of segments and annular grouting.

4.2.1.6 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 borings contains loose sands with blow count values less than 15. When saturated, these foundation 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 Delta levees that have loose, saturated sand in their foundations, and are composed of silty sand, may liquefy during future moderate to strong shaking, resulting in levee failure (DWR, 2008b).

A preliminary assessment of the potential for liquefaction occurring at the proposed WaterFix BTO intake facility locations and in the vicinity of the general project alignment was evaluated using the data obtained from recent borings and CPT soundings. The liquefaction analyses were performed in general accordance with procedures that were developed by a consensus of the participants of the National Center for Earthquake Engineering Research workshops (Youd et al., 2001). The potential for liquefaction is estimated by calculating the estimated cyclic stress ratio induced by the design ground motion and compared with the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio. The risk of liquefaction is considered significant where the ratio of cyclic resistance ratio to cyclic stress ratio, or factor of safety, is less than 1.0.

At each project facility, the borings and/or CPT soundings that were observed to have the most critical conditions for liquefaction, based on the presence of sand and silt materials with either low blow counts or low cone resistance, were evaluated.

Final design liquefaction analyses should be performed when final seismic design criteria for the WaterFix BTO facilities have been adopted and design-level site-specific geotechnical exploration and testing have been completed.

**Intakes:** The risk of liquefaction at two of the three intake locations (Intakes No. 2 and 5) was preliminarily identified as high for a significant portion of the soils above elevation -65 feet. It should be noted that the nearest subsurface information available at the intake locations was from borings conducted from over the water adjacent to the intake sites. Additional exploration is currently proposed at the intake locations over land, which could encounter significantly different conditions.

**Intermediate Forebay:** No site specific subsurface information was available for the Glanville Tract IF. Based on information from a soil boring (DCE-DH-003) and a CPT (DCE-CPT-009) completed in the year 2009 and located about one mile and a half from the IF, it appears that the risk of liquefaction would be low. However, historical borings completed in the year 1966 and located about a mile from the IF show the presence of sandy materials susceptible to liquefaction at depths of 12 to 15 feet and 30 to 35 feet below ground surface.

**Byron Tract Forebay:** Available subsurface information indicates that the potential for liquefaction exists along all sides of the existing Clifton Court Forebay. For the purpose of the conceptual design, it is assumed that this analysis is valid for the area of the BTF. As more subsurface data is collected, additional liquefaction analyses should be performed to evaluate embankment stability and to determine potential mitigation measures.

**North Tunnels:** For the North Tunnels (north of the Intermediate Forebay), the liquefaction results from the intakes and the IF were judged to be representative, in the absence of additional data. Based on preliminary geotechnical investigations and analysis, the North Tunnels appear to be founded below the elevation where liquefaction has been identified at these locations. Liquefaction-induced settlement of pad fill at the intake tunnel shafts, and the tunnel junction structures near each of the intakes can be expected. This condition will be further investigated in the future preliminary design phase of the program once additional geotechnical information is obtained, and a more definitive seismic hazard analysis is completed.
Main Tunnels: Based on preliminary geotechnical investigations, the Main Tunnels appear to be founded below the elevation where liquefaction has been identified. Extensive liquefaction down to depths of 40 to 60 feet below ground surface is predicted along the Main Tunnel alignment, especially in areas with soft and loose soils. Consequently, there is the potential for liquefaction-induced settlement of the Main Tunnel drive shafts and reception shafts, and working pad fills. This potential condition is persistent along the Main Tunnel alignment through the central delta region. The critical component to be analyzed in the future preliminary design seismic analyses will be in structural connections between the shafts and the tunnels, as well as the soil-structure interaction of the shafts in potentially liquefiable ground conditions. As in the North Tunnel region, this condition will be further investigated in the future preliminary design phase of the program once additional geotechnical information is obtained, and a more definitive seismic hazard analysis is completed. These future analyses will determine if ground improvements near the shafts are required due to the potential for soil liquefaction in a seismic event, as well as the most effective method to design and construct the connections between the tunnels and the shafts.

4.3 Flood Protection Considerations

The conveyance facilities are considered to be critical lifeline facilities for the State of California. The proposed WaterFix BTO contains facilities within the central portion of the Delta. 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 mentioned above (DWR, 2009).

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.

4.3.1 Approach

The recommended flood protection criteria for the conceptual and preliminary engineering phases are the 200-year flood event WSE, including SLR, with an additional allowance for the 3-foot standard FB and the computed wind-wave run-up. For the intakes, both river and land side flood levels are considered.

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

- 200-year return flood event in the adjacent sloughs or rivers.
- Inundation of floodplain from a 200-year return flood event with levee breach.
- Wind-induced waves.
- Mean higher high water tides.
- 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.

Estimated flood levels are used in the design for each conveyance option. These assumptions will be confirmed and possibly refined during subsequent design phases. Table 4-5 indicates flood level conditions assumed for this CER for each facility of the WaterFix BTO. Flood protection elevations were not available for the specific shaft locations between the IF and the BTF. For the purposes of this CER, it was assumed that flood levels between the IF and the BTF will be the same as the Pearson District flood protection elevation of 32.2 feet. The Pearson District flood level was also used for all northern facilities, except the intakes as noted in Table 4-5.
### Table 4-5: Flood Protection Elevations for WaterFix BTO Facilities

<table>
<thead>
<tr>
<th>WaterFix BTO Facility</th>
<th>200-Year WSE with SLR</th>
<th>200-Year WSE with SLR, Wave Run-up and Freeboard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake No. 2 (all facilities at this site)</td>
<td>31.4&lt;sup&gt;a&lt;/sup&gt;, 24.2&lt;sup&gt;b&lt;/sup&gt;</td>
<td>34.4&lt;sup&gt;a, c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Intake No. 3/Junction Structure (all facilities at this site)</td>
<td>30.4&lt;sup&gt;a&lt;/sup&gt;, 24.2&lt;sup&gt;b&lt;/sup&gt;</td>
<td>33.4&lt;sup&gt;a, c&lt;/sup&gt;</td>
</tr>
<tr>
<td>Intake No. 5 (all facilities at this site)</td>
<td>28.4&lt;sup&gt;a&lt;/sup&gt;, 24.2&lt;sup&gt;b&lt;/sup&gt;</td>
<td>32.2&lt;sup&gt;a, c&lt;/sup&gt;</td>
</tr>
<tr>
<td>IF and Structures (includes drive shafts for north and Main Tunnels)</td>
<td>24.2&lt;sup&gt;b&lt;/sup&gt;</td>
<td>32.2&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>Staten Island Reception Shafts</td>
<td>24.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
<td>32.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
</tr>
<tr>
<td>Bouldin Island Shafts</td>
<td>24.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
<td>32.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
</tr>
<tr>
<td>Bacon Island Reception Shafts</td>
<td>24.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
<td>32.2&lt;sup&gt;b, d&lt;/sup&gt;</td>
</tr>
<tr>
<td>BTF and Structures (includes Main Tunnel shafts at northeast corner of BTF, and pump plant)</td>
<td>16.5&lt;sup&gt;b&lt;/sup&gt;</td>
<td>24.5&lt;sup&gt;e, b&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup> 200-year in-river flood level with SLR at each intake provided by DWR (2012). Intake sites must consider both river and land side flood levels for levee and pad heights. Wave run-up is considered zero for river flood levels. The river flood level controls for Intakes No. 2 and 3 and the land side flood level controls for Intake No. 5.

<sup>b</sup> Source: Mineart et al., 2009

<sup>c</sup> Refer to Section 7.0, Intakes and Sedimentation Facilities for criteria to select structures and levee heights.

<sup>d</sup> All shaft locations between IF and BTF must have flood elevation requirements confirmed during subsequent study.

**Notes:**

- **BTF** = Byron Tract Forebay
- **E** = East
- **IF** = Intermediate Forebay
- **SLR** = sea level rise
- **W** = West
- **WSE** = water surface elevation

All elevations are NAVD88
The conveyance option described in this report is based on input from DWR, USACE, and other sources as obtained and evaluated by the DHCCP. Six potential flooding scenarios were considered:

- 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 Leveses* (DWR, 2009), which states that the physical top of a 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.

SLR values were based on the recommendation of the Delta Vision Blue Ribbon Task Force 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 gage location. For conceptual engineering, both high and low water levels are required for the layout of the project facilities. For flood levels, 18 inches of SLR is applied above computed flood levels, as described in the various flood level reference documents. For low water levels, facilities must be conceptualized without the effect of SLR so they can provide required functionality before the effects of SLR are realized.
Figure 4-1: Completed Geotechnical Explorations (2009-2012)
Figure 4-3: Graphic Boring Logs, 2 of 5
Figure 4-4: Graphic Boring Logs, 3 of 5

NOTES:
1. For definition of Unified Soil Classification System abbreviations, consult ASTM D2487.
2. Consult geotechnical data report for Pipeline/Tunnel Option (draft April 2013) for complete boring logs and subsurface data.
NOTES:
1. For definition of Unified Soil Classification System abbreviations, consult ASTM D2487.
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Figure 4-5: Graphic Boring Logs, 4 of 5

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April 1, 2015

California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta River Restoration & Conformance Programs
NOTES:
1. For definition of Unified Soil Classification System abbreviations, consult ASTM D2487.
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Final Draft
April 1, 2015

Figure 4-6: Graphic Boring Logs, 5 of 5
5.0 Conveyance System Operations

This section briefly describes the anticipated future operation of the existing SWP and CVP Delta export facilities taking into account the proposed WaterFix BTO facilities.

5.1 Existing Facilities and Operations

The current method for conveying water to the SWP and the CVP export pumping plants (the Banks PP and Jones PP, respectively) is solely by through-Delta conveyance which results in what is referred to as South Delta diversions. The source of water for the existing export pumps is unregulated Delta inflow and re-diversion of stored water released from Shasta (Reclamation), Oroville (DWR), and Folsom (Reclamation) Reservoirs. Releases from these reservoirs into the Sacramento River are managed through the terms of the Coordinated Operations Agreement between DWR and Reclamation within regulatory requirements. The daily allocation of water available in the Delta for export via the SWP and CVP Delta pumping facilities is jointly determined by DWR and Reclamation by estimating flows necessary to comply with the various environmental and water quality standards, as well as other constraints.

The SWP and CVP 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.

5.1.1 Datum Corrections

As described below, different vertical datums are used for reporting water elevations at the SWP and the CVP. DHCCP has standardized to a single datum with correction factors to convert to the standard DHCCP datum.

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). The conversion between NGVD29 and NAVD88 is +2.36 feet at the Banks PP. However, DWR survey results indicate a conversion of approximately +3.10 feet from Banks PP elevations to the NAVD88 datum. Table 5-1 summarizes CCF water elevation information.

The CVP Delta export facilities consist of the Tracy Fish Collection Facility and the Jones PP. The conversion between the Jones PP site-specific datum and NAVD88 is approximately -0.43 feet.

5.1.2 Byron Bethany Irrigation District Delta Export Facilities

The Byron Bethany Irrigation District (BBID) diverts flow from the Banks PP intake channel upstream of Banks PP. Portions of the SWP Delta export facilities have the responsibility to deliver flow to the BBID pumps.

5.1.3 State Water Project Delta Export Facilities

The SWP Delta export facilities consist of Clifton Court Forebay, Skinner Delta Fish Protection Facility (Skinner Fish Facility), Banks PP, and Banks PP Intake Channel, which connects CCF to the Banks Pumping Plant.

5.1.3.1 Clifton Court Forebay

The existing diversions into CCF are restricted to a peak instantaneous flow of 12,000 cfs; a daily maximum of 13,870 AF; and a maximum 13,250 AF per day averaged over any 3-day period from mid-March through June 30 and from October 1 through mid-December. These daily maximum values may be increased by up to 1/3 of the San Joaquin River flow as measured at Vernalis from mid-December through mid-March and by 990 AF per day from July 1 through September 30.

CCF operation is linked to the Banks PP operation. 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.
The CCF maximum storage is 28,653 AF at the normal maximum water surface elevation (WSE); the minimum storage is 13,965 AF at the minimum WSE (DWR, 1974). For future operations, and unless engineering improvements are made to the perimeter embankment around CCF, the maximum operating WSE has been reduced by 1 foot. Table 5-1 summarizes CCF water elevation information.

Table 5-1: Existing Clifton Court Forebay Operational Water Elevations (Measured at Clifton Court Forebay Inlet Gates)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Data in NGVD29</th>
<th>Data in NAVD88</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Datum</td>
<td>NGVD29(^a)</td>
<td>NAVD88 (NGVD29 + 3.10 feet(^b))</td>
</tr>
<tr>
<td>Normal Minimum Water Surface Elevation(^c)</td>
<td>-2 feet</td>
<td>+1.1 feet</td>
</tr>
<tr>
<td>Normal Maximum Water Surface Elevation(^d)</td>
<td>+4 feet</td>
<td>+7.1 feet</td>
</tr>
<tr>
<td>Absolute Maximum Water Surface Elevation(^a,d)</td>
<td>+5 feet</td>
<td>+8.1 feet</td>
</tr>
</tbody>
</table>

\(^a\) Used by pumping plant operators.  
\(^b\) DWR survey results at Banks PP, February 2009, assumed to also apply at CCF.  
\(^c\) Source: DWR, 1974 (Bulletin No. 200, November 1974, DWR California State Water Project).  
\(^d\) Standard Operating Orders PC200.7-A and 600.22.

Abbreviations:  
CCF = Clifton Court Forebay  
DWR = Department of Water Resources  
NAVD88 = North American Vertical Datum of 1988  
NGVD29 = National Geodetic Vertical Datum of 1929

5.1.3.2 Skinner Fish Facility

The Skinner Fish Facility is located on the Banks Pumping Plant Intake Channel between CCF and Banks PP. Under peak pumping operations, headloss across the louver screens is reported to be 1.5 feet. The Skinner Fish Facility has a maximum operating elevation of +5 feet (NGVD29) before overtopping the louver screens.

5.1.3.3 Banks Pumping Plant

The Banks Pumping Plant has an installed capacity of 10,670 cfs and presently takes water from CCF via the Banks Pumping Plant Intake Channel downstream of the Skinner Facility. The pump plant consists of 11 pumps: nine large pumps (five at 1,130 cfs each, four at 1,067 cfs each) and two smaller pumps (375 cfs capacity each). There are flow meters on each of the five discharge lines.

The existing Banks PP is designed to operate with a WSE just upstream of the pumping plant ranging from -0.9 to +8.1 feet (NAVD88). However, the maximum operating WSE has been restricted through the standing orders for CCF.

Banks PP is generally operated on an on-peak/off-peak schedule. The pumping schedule maximizes pumping of the SWP’s export allocation from CCF into Bethany Reservoir and SWP conveyance system downstream during the off-peak hours and spreads out additional pumping at a reduced rate during on-peak hours if conditions dictate.

Table 5-2 presents a summary of Banks PP operational water elevation information.
Table 5-2: Operational Water Surface Elevations at Banks Pumping Plant

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Data in NGVD29</th>
<th>Data in NAVD88</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Datum</td>
<td>NGVD29[^a]</td>
<td>NAVD88 (NGVD29 + 3.10 feet)[^b]</td>
</tr>
</tbody>
</table>

**Banks PP Intake Channel (aka California Aqueduct) Immediately Upstream of Banks PP Trash Racks**

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Data in NGVD29</th>
<th>Data in NAVD88</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lowest Water Surface Elevation for Pump Operation[^a, c]</td>
<td>-4 feet (-3.5 feet)[^d]</td>
<td>-0.9 feet (-0.4 feet)</td>
</tr>
<tr>
<td>Typical Range in Water Surface Elevation under Priority 1 Gate Operations[^a]</td>
<td>-2 to + 0 feet</td>
<td>+ 1.1 feet to + 3.1 feet</td>
</tr>
<tr>
<td>Historical Typical Range in Water Surface Elevation[^a]</td>
<td>-2 to + 2 feet</td>
<td>+ 1.1 feet to + 5.1 feet</td>
</tr>
<tr>
<td>Maximum Design Water Surface Elevation under Normal Operations[^a, f]</td>
<td>+5 feet</td>
<td>+8.1 feet</td>
</tr>
</tbody>
</table>

[^a]: According to pumping plant operators.
[^b]: DWR survey results, February 2008.
[^c]: Dictated by pump submergence requirements to prevent cavitation.
[^d]: BBID pumps in approach channel limited to -3.5 feet prior to cavitation.
[^e]: As reported by DWR
[^g]: Upstream of fish louver screens at Skinner Delta Fish Protection Facility.

Notes:

BBID = Byron Bethany Irrigation District  
DWR = Department of Water Resources  
NAVD88 = North American Vertical Datum of 1988  
NGVD29 = National Geodetic Vertical Datum of 1929

5.1.4 Central Valley Project Delta Export Facilities

The CVP Delta export facilities consist of the Tracy Fish Collection Facility and the Jones PP.

5.1.4.1 Tracy Fish Collection Facility

The Tracy Fish Collection Facility is located off Old River at the head of Jones PP Intake Channel. The facility intercepts fish using louver screens, and the fish are then collected into tanker trucks and relocated away from pump intakes.

5.1.4.2 Jones Pumping Plant

The Jones Pumping Plant is at the end of a 2.5-mile-long, tidally-influenced, unlined intake channel that begins at the Tracy Fish Collection Facility. In contrast to the SWP facilities, the Jones PP has no forebay. The Jones PP is owned by Reclamation, but operated by the San Luis and Delta-Mendota Water Authority (SLDMWA). The Jones PP lifts water into the Delta-Mendota Canal (DMC).

The Jones PP has an original capacity of 5,100 cfs and a refurbished maximum of 5,630 cfs, but it is constrained by downstream canal configuration to a maximum of 4,600 cfs. The CVP pumping plant includes 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. 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 operational limitation of approximately 4,300 cfs immediately upstream of the O’Neill Forebay. There are approximately 100 turnouts...
along the DMC between the Jones PP and the O’Neill Forebay. When these turnouts are not in operation, the Jones PP peak output is limited to 4,300 cfs.

Generally, the Jones PP 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 Jones PP power tariff (which is different from the Banks PP power tariff) provides no economic advantage for the Jones PP to operate according to an on-peak/off-peak schedule. Typically, between two and five pumps are operated at one time at Jones PP, with changes in numbers of pumps in operation generally scheduled several days in advance.

Table 5-3 presents a summary of Jones PP operational water elevation information.

Table 5-3: Operational Water Surface Elevations at Jones Pumping Plant

<table>
<thead>
<tr>
<th>Vertical Datum</th>
<th>Site Data(^d)</th>
<th>Data in NAVD88</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Operating Criteria</td>
<td>-1 feet to +10 feet</td>
<td>-1.43 feet to +9.57 feet</td>
</tr>
<tr>
<td>Lowest Water Surface Elevation for Operation(^c)</td>
<td>-2 feet</td>
<td>-2.43 feet</td>
</tr>
<tr>
<td>Typical Range in Upstream Water Surface Elevation Measured Immediately Upstream of Trash Racks at Inlet to Pumping Plant(^d)</td>
<td>0 feet to +4 feet</td>
<td>-0.43 feet to +3.57 feet</td>
</tr>
<tr>
<td>Highest Water Surface Elevation under Typical Operation(^d)</td>
<td>+6 feet</td>
<td>+5.57 feet</td>
</tr>
<tr>
<td>Highest Water Surface Elevation for Pump Operation</td>
<td>+10 feet</td>
<td>+9.57 feet</td>
</tr>
</tbody>
</table>

\(^d\) The site-specific datum translation value at the Jones PP of -0.43 feet to NAVD88 is derived from DWR survey results undertaken for the DHCCP team (February 2009).
\(^b\) Source: Reclamation, 1951
\(^c\) Dictated by pump submergence requirements to prevent cavitation.
\(^d\) As reported by SLDMWA.

Notes:

DHCCP = Delta Habitat Conservation and Conveyance Program
NAVD88 = North American Vertical Datum of 1988
SLDMWA = San Luis and Delta-Mendota Water Authority
5.1.5 **Comparison between State Water Project and Central Valley Project Delta Export Facilities**

The primary differences between the SWP and CVP Delta export facilities are summarized in Table 5-4.

**Table 5-4: Comparison between Delta Export Facilities for the State Water Project and Central Valley Project**

<table>
<thead>
<tr>
<th>Factor</th>
<th>State Water Project</th>
<th>Central Valley Project</th>
</tr>
</thead>
<tbody>
<tr>
<td>Owner</td>
<td>DWR</td>
<td>Reclamation</td>
</tr>
<tr>
<td>Operator</td>
<td>DWR</td>
<td>SLMWA</td>
</tr>
<tr>
<td>Pumping Plant</td>
<td>Banks PP</td>
<td>Jones PP</td>
</tr>
<tr>
<td>Installed Capacity</td>
<td>Nominal 10,670 cfs (11 units)</td>
<td>Nominal 4,600 cfs (6 units)</td>
</tr>
<tr>
<td>Pump Sizing</td>
<td>5 at 1,130 cfs, 4 at 1,067 cfs, 2 at 375 cfs</td>
<td>1 at 1,000 cfs, 2 at 990 cfs, 1 at 950 cfs, 2 at 850 cfs (^a)</td>
</tr>
<tr>
<td>Pumping Regime</td>
<td>Operated on an on-peak/off-peak schedule. Typically, periods of non-operation during on-peak hours.</td>
<td>24/7 at constant rate when water is available for export.</td>
</tr>
<tr>
<td>Flow Variation</td>
<td>Output varies throughout the day; flow changes as often as once per hour. Output scheduled several days in advance.</td>
<td>Limited changes in number of pumps in operation, often constant throughout 24-hour period, any changes scheduled several days in advance.</td>
</tr>
<tr>
<td>Fish Screens</td>
<td>Skinner Delta Fish Protection Facility</td>
<td>Tracy Fish Collection Facility</td>
</tr>
<tr>
<td>Forebay</td>
<td>CCF, 28,653 AF</td>
<td>None, tidal channel</td>
</tr>
<tr>
<td>Forebay Water Level Control</td>
<td>Tidally influenced, partial control using radial gates at CCF inlet.</td>
<td>None, tidal</td>
</tr>
</tbody>
</table>

**Elevations Converted to NAVD88**

- Typical Range in Upstream Water Surface Elevation: +1.1 feet to +3.1 feet, -0.43 feet to +3.57 feet
- Lowest Water Surface Elevation for Pump Operation\(^b\): -0.9 feet for Banks PP, -0.4 feet for BBID pumps
- Highest Water Surface Elevation under Typical Operation: +5.1 feet, +5.57 feet
- Highest Water Surface Elevation for Overall Existing Operation: +10.6 feet, +9.57 feet

\(^a\) As reported by SLMWA.

\(^b\) Dictated by pump submergence requirements to prevent cavitation.

**Notes:**

- AF = acre-feet
- Banks PP = Harvey O. Banks Pumping Plant
- BBID = Byron Bethany Irrigation District
- cfs = cubic feet per second
- CCF = Clifton Court Forebay
- DWR = Department of Water Resources
- NAVD88 = North American Vertical Datum of 1988
- SLMWA = San Luis and Delta-Mendota Water Authority
- 24/7 = 24 hours per day, 7 days per week
5.1.6 Use of Existing Delta Export Facilities with the WaterFix BTO

In the current operation, the SWP draws water from the Clifton Court Forebay (CCF) and CVP draws its supply from the Old River. The proposed operation of the new WaterFix BTO will have three operation scenarios: isolated south Delta operation, isolated north Delta operation, and a potential dual operation scenario.

5.1.6.1 Isolated South Delta Operation

The isolated south Delta operation is the same as the current operation of the existing SWP and CVP facilities. The SWP will continue to divert its supply into the CCF from West Canal, whereas the CVP diverts its flows from Old River. During the isolated south Delta operations, both Skinner and Tracy Fish Facilities will remain operational. No diversions will be taken from the north Delta intake facilities.

5.1.6.2 Isolated North Delta Operation

For the isolated north Delta operation, both Banks PP and Jones PP draw water from BTF via the new conveyance facilities and the north Delta intakes. Only BTF is used in this scenario, and the CCF intake and Tracy Fish Facility gates will remain closed. The export water supply will be routed from the north Delta intakes to the BTF, and then conveyed through the BTF outlet conveyance system to the Banks and Jones Pumping Plant Intake Channels. The Tracy and Skinner Fish Facilities will not be in operation under the isolated north Delta operation scenario. For this scenario, the operating range of water surface elevations within the BTF will need to be compatible with the operating ranges of both export pumping plants.

5.1.6.3 Potential Dual Operation with WaterFix BTO

Under a potential dual operation scenario, both north Delta intakes and south Delta diversion facilities would be simultaneously used to meet the SWP and CVP demands. The inflow into BTF would be provided by the diversions from the north Delta intakes and the inflows into CCF would be from south Delta intakes. The SWP demands would be met by the flows from both BTF and CCF simultaneously, with the flows from the CCF passing through the Skinner Fish Facility. The CVP demands would be met by flows from the BTF and Old River simultaneously, with the diverted flow from Old River passing through the Tracy Fish Facility.

It can also be expected that the diversions from the north and south Delta intakes will be subjected to a number of constraints, including tidal cycles, water surface elevation at south Delta sloughs, flows at Sacramento River and its tributaries, water quality at key locations, season of the year, and gate operation cycles (Priorities). The diversion constraints, however, will be different for north Delta intakes and south Delta diversion facilities. It is anticipated that these constraints will be considered in determining flow distribution between the north and south Delta intakes under a potential dual source operational scheme. For example, on the delivery side, the SWP prefers to operate during off-peak hours, when electricity rates are lower, whereas the Jones PP, which is part of the CVP, must operate continuously throughout the day. Because of the power need and downstream storage availability, the pumping schedule at Jones PP is prepared several days in advance, and there is a minimum demand that must be met during each hour of the day.

Simultaneous operation of both the BTF and CCF is planned to be part of this conveyance system. Under the dual source operation scenario, control gates will control flow out of BTF, CCF and the Old River to meet target deliveries at both Banks and Jones PP’s. The flow control system should be such that it maximizes the diversion opportunities from both north and south Delta intakes. In order to add this operational capability to the conveyance system, control structures will need to be capable of throttling flow from all three sources. The open channels that feeds Banks and Jones PP downstream of the Skinner Fish Facility and downstream of the Tracy Fish Facility must maintain a lower WSE from all three sources to maintain flow control of all the throttling gates at each source. The control scheme will require flow meters, WSE transmitters, and a sophisticated SCADA system controls. The outlet structures from both BTF and CCF should also be able to account for the flow delivery to both pumping facilities and should be operable for all flow conditions, such as high, medium and low flows.

The location and design of structures capable of operating in a dual source scenario needs to be explored in further design phases of the project. Such work will refine the configuration of facilities necessary to achieve this mode of operation.
Table 5-5 summarizes the recommended elevations for both gates and their operations.

**Table 5-5: Comparison of Water Surface Elevations for WaterFix BTO Operation Scenarios**

<table>
<thead>
<tr>
<th>Forebay in Use</th>
<th>Factors for WSE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isolated South Delta Operation</td>
<td></td>
</tr>
<tr>
<td>CCF</td>
<td>For SWP</td>
</tr>
<tr>
<td></td>
<td>Historical minimum WSE = 1.1 ft</td>
</tr>
<tr>
<td></td>
<td>Typical minimum operating elevation = 1.1 ft</td>
</tr>
<tr>
<td></td>
<td>Typical maximum operating elevation = 3.1 ft</td>
</tr>
<tr>
<td></td>
<td>Normal maximum operating WSE = 8.1 ft</td>
</tr>
<tr>
<td></td>
<td>For CVP</td>
</tr>
<tr>
<td></td>
<td>Design minimum operating elevation = -1.43 ft</td>
</tr>
<tr>
<td></td>
<td>Design maximum operating elevation = 9.57 ft</td>
</tr>
<tr>
<td></td>
<td>Lowest WSE for pump operation = -2.43 ft</td>
</tr>
<tr>
<td>Isolated North Delta Operation</td>
<td></td>
</tr>
<tr>
<td>BTF</td>
<td>For SWP and CVP</td>
</tr>
<tr>
<td></td>
<td>Low WSE at BTF = 2.5 ft</td>
</tr>
<tr>
<td></td>
<td>High operating WSE = 12.5 ft</td>
</tr>
<tr>
<td>Dual Operation (simultaneous use of north Delta intakes and south Delta facilities)*</td>
<td>For SWP</td>
</tr>
<tr>
<td></td>
<td>Historical minimum WSE = 1.1 ft</td>
</tr>
<tr>
<td></td>
<td>Typical minimum operating elevation = 1.1 ft</td>
</tr>
<tr>
<td></td>
<td>Typical maximum operating elevation = 3.1 ft</td>
</tr>
<tr>
<td></td>
<td>Normal maximum operating WSE = 8.1 ft</td>
</tr>
<tr>
<td></td>
<td>For CVP</td>
</tr>
<tr>
<td></td>
<td>Design minimum operating elevation = -1.43 ft</td>
</tr>
<tr>
<td></td>
<td>Design maximum operating elevation = 9.57 ft</td>
</tr>
<tr>
<td></td>
<td>Lowest WSE for pump operation = -2.43 ft</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>BTF</td>
<td>For SWP and CVP</td>
</tr>
<tr>
<td></td>
<td>Low WSE at BTF = 2.5</td>
</tr>
<tr>
<td></td>
<td>High WSE at BTF = 12.5</td>
</tr>
</tbody>
</table>

*Design WSE’s for a potential dual source operational scenario will be refined during a later design phase.

### 5.2 Sacramento River Modes of Operation

The WaterFix BTO will operate under two primary river modes of operation: low level and normal pumping.

#### 5.2.1 Low Level Pumping mode

Low Level Pumping mode is a constant low level pumping of up to 300 cfs from each intake depending on the flow in the Sacramento River. The mode of operation will allow for movement of water through the system to prevent stagnation and sediment deposition during periods of flow restricted from north Delta.

#### 5.2.2 Normal Operations mode

The overall system operation is based on moving the daily water allocation for each of the SWP and CVP export demands, rather than maintaining a constant flow rate throughout the day.

Operation of the north Delta intake facilities, Byron Tract Forebay Pumping Plant (BTF PP), and the export pumping plants will be synchronized for the required flow rate. Minor adjustments in duration of pumping operations will be made to restore BTF water elevation to a common agreed-upon level at the start of each daily cycle.

The WaterFix BTO facilities have two normal river operating modes, depending on the season of the year, flow, and tidal cycle within the Sacramento River:

- **Intermittent Diversion from Sacramento River:** At lower river flows, preliminarily determined by WaterFix BTO operational criteria to be below 20,000 cfs.
• Continuous Diversion from Sacramento River: At higher river flows, preliminarily determined by WaterFix BTO operational criteria to be greater than 20,000 cfs.

5.2.2.1 Intermittent Diversion from Sacramento River
The Intake Facilities and BTF PP are to operate according to the tidal cycle to maximize the daily volume of water, independent of the power tariff structure. Since the primary objective is to capture water when available, and since Jones PP operates 24 hours per day, on-peak pumping at the BTF PP will be required. Under certain circumstances, these pumps could be non-operational during two 6-hour periods centered on high tide.

Jones PP will continue to operate 24 hours per day, 7 days per week; Banks PP will preferentially operate during off-peak hours. However, a consequence of intermittent diversion is that under certain conditions, it may be necessary to operate one or more Banks PP pumps during on-peak periods to obtain the SWP’s daily water allocation. Further analysis is required.

5.2.2.2 Continuous Diversion from Sacramento River
Similar to the facilities used for the intermittent diversion operating mode, the Intake Facilities and BTF PP operate continuously to maximize the daily volume of water diverted from the river, independent of the power tariff structure. The ability to operate the system using off-peak pumping alone at SWP export facilities is a function of the capacity of the BTF. Assuming that Jones PP is operating 24/7, the available storage in the BTF plus WaterFix BTO flows has to be sufficient to sustain both the Jones and Banks PPs during off-peak power periods. During on-peak hours, Banks PP would be off and the WaterFix BTO’s continuous rate would be higher than the Jones PP rate. Therefore, the WaterFix BTO system would restore the operating volume within BTF to a common agreed-upon level at the start of each daily cycle.

5.3 Concept of Operations
5.3.1 Operating Assumptions
The preliminary concept of operation for the WaterFix BTO has the following assumptions:

• Operate safely and reliably, complying with all applicable regulations, including all long-term Delta operating rules developed by the WaterFix BTO
• Maintain BTF water levels within the efficient operating bands of all pump sets at both Banks and Jones PPs.
• Divert the SWP and CVPs combined daily water allocation (i.e., “X” AF of water per day) with a flow rate based on operating duration through the north Delta intakes, the existing south Delta intakes, or both.
• Minimize impacts to the established operational methodology and control philosophy of both the SWP and CVP downstream of their respective existing export pumping plants.

5.3.2 Overall Operation of System Components
The system will have the primary components as described below:

• Intake facilities
• Tunnel systems (North Tunnels and Main Tunnels)
• Intermediate Forebay (IF)
• Byron Tract Forebay Pumping Plant (BTF PP)
• Byron Tract Forebay (BTF)
• BTF Outlet Conveyance System (comprised of South Tunnels, open channels to State Water Project and Central Valley Project Export Facilities and control structures)
5.3.3 Intake Facilities Operations

The preliminary concept for the WaterFix BTO includes three 3,000-cfs intake facilities along the Sacramento River, with a combined total capacity of 9,000 cfs. Each intake facility will consist of a screened intake, sedimentation basin, and influent shafts for the North Tunnels. DWR-Operations will develop the daily schedule for intake operation in advance and in coordination with Reclamation. Operations will select the target flow rate to be withdrawn from the Sacramento River and the individual intake facility (or facilities) to be in service at any time.

The daily schedule will be developed taking into account the factors listed in Table 5-6.

Table 5-6: Daily Operational Considerations for Withdrawal from Sacramento River

<table>
<thead>
<tr>
<th>Factor</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Target Export from Delta</td>
<td>SWP, BBID, and CVP Delta allocations. Ongoing coordination between DWR and Reclamation.</td>
</tr>
<tr>
<td>Biological</td>
<td>Possible or confirmed presence of species listed under the Federal Endangered Species Act and the California Endangered Species Act local to a given intake. Influence of biological factors on Delta.</td>
</tr>
<tr>
<td>Hydrological</td>
<td>Limitations on volume available for export based on flow rate within Sacramento River per BDCP. Limitations on permissible time during which withdrawal is allowable based on flow rate within Sacramento River (possible ebb tide pumping) per BDCP. Ongoing coordination with releases from Shasta, Oroville, and Folsom Dams. High flood levels in Sacramento River.</td>
</tr>
<tr>
<td>Intake Location</td>
<td>Priority for withdrawal from upstream intake locations first.</td>
</tr>
<tr>
<td>System Mode of Operation</td>
<td>System capacity downstream. System storage capacity downstream. System limitations (e.g., maintenance). Water Right Decision D-1641 (State Water Resources Control Board, 1999) and subsequent amendments.</td>
</tr>
<tr>
<td>Integration with Other Scheme Components</td>
<td>Coordinated with storage requirements and pumping schedules for Banks PP, BBID, and Jones PP.</td>
</tr>
<tr>
<td>Energy Usage</td>
<td>Consideration of power tariff structures. Ongoing coordination with electrical utility provider.</td>
</tr>
<tr>
<td>Seasonal</td>
<td>Compliance with seasonal restrictions for withdrawal from Sacramento River.</td>
</tr>
<tr>
<td>Water Quality</td>
<td>Water quality monitoring (turbidity, chemicals) local to given intake. Water quality concerns elsewhere in Delta (such as salinity).</td>
</tr>
<tr>
<td>Maintenance</td>
<td>Maintenance schedules for intake facilities. Consideration of rotation between intake facilities during prolonged periods of lower flows. Local sediment buildup.</td>
</tr>
</tbody>
</table>

Notes:

BBID = Byron Bethany Irrigation District
CVP = Central Valley Project
DWR = Department of Water Resources
PP = Pumping Plant
Reclamation = United States Bureau of Reclamation
SWP = State Water Project

The WaterFix BTO is expected to operate under 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 in-stream bypass flow represents the available flow for diversion into the WaterFix BTO 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 3,000 cfs at each of the three intake locations can be diverted; at other times, much less can be diverted, or no water at all. The California WaterFix 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 the BTF PP are understood to be specific to the tidal state at the intake locations.

When the available flow for diversion is significantly less than the maximum 9,000 cfs for a significant period of time, one or more of the three intake facilities and associated tunnels would operate under the Low Level Pumping mode described earlier (see Paragraph 5.2.1). Periodic use of these intakes will be required to exercise the equipment and maintain operational functionality. It is possible that the northern-most intake will be used for diversion in preference to the more southern intakes for various reasons.

Intermittent diversion addresses periods when tidal influences reduce river velocity such that sweeping flows at the intakes might be too low for diversions to occur. During these periods, usually associated with high tides, when Sacramento River flows are below a given threshold, it may be necessary to limit or stop withdrawals from the river; potentially, about 20,000 cfs flow diversions may be limited or halted. This figure could vary with the advancement of the operation studies based on refined project components. This concept, and associated river flows, must be further refined by specific regulatory criteria coupled with operations and river hydraulic modeling. For this CER, two 6-hour periods of no diversions (in a 24 hour day) at the intakes are considered to account for the intermittent diversion concept.

### 5.3.4 Intermediate Forebay Operations

The conceptual design of the WaterFix BTO includes a forebay designated as Intermediate Forebay (IF) that is located at Glanville tract, downstream of the Intake facilities and upstream of the BTF PP. The IF provides an atmospheric break in the deep tunnel system and buffer volume for the upstream intake sites and the downstream BTF PP. This buffer provides make-up water and storage volume to mitigate transients generated as a result of planned or unplanned adjustments of system pumping rates. The IF also facilitates isolating segments of the tunnel system, while maintaining operational flexibility. Thus each tunnel, into and out of IF, can be hydraulically isolated for maintenance, while maintaining partial system capacity. Finally, the IF provides a hydraulic surge dissipation capacity to dampen surge waves that may occur if the pumps at BTF shutdown in a sudden manner, such as would occur if there is a power outage at the BTF PP.

The IF is planned to be a pass-through facility between the intake sites and the Main Tunnels and is not intended to provide hydraulic controls or long term storage during normal, steady state operation. The IF will also function to further remove sediment not captured by the intake sedimentation basins. The hydraulic grade line and WSE at the IF will be dependent on the downstream WSE in the BTF PP wet-wells. No flow regulating gates will be provided in the IF; however, isolation gates will be provided on each tunnel exit / entrance into and out of IF. The IF will be sized as small as practical to accommodate the operational flow fluctuations and potential system upset conditions (power failure, unintended gate closures, etc.). More details are provided in Section 14.

Two tunnels provide flow into the IF. The combined flows from Intake No. 2 and 3 will be conveyed from the Junction Structure to the IF via a single 40-foot diameter tunnel. A 28-foot diameter tunnel conveys flow from Intake No. 5 directly to the IF. Water exits the south end of the IF via the dual Main Tunnels. These tunnels are fed by gravity from the IF and convey flow to the BTF PP.

### 5.3.5 Tunnel System Operations

The tunnel system is configured to operate under gravity flow conditions from the intakes to the BTF PP. The tunnels will be continuously flowing full of water when in operation. Isolation facilities will be provided at the upstream and downstream end of each tunnel. For operation and maintenance purposes, intermediate access shafts will be provided at roughly four mile increments along the Main Tunnel alignment. These locations will correspond with TBM launching and retrieval shafts that are established during construction.
5.3.6 Byron Tract Forebay Pump Plant (BTF PP) Operations

BTF PP will lift water from the Main Tunnels and discharge into BTF via two pumping plants. The pumping plants are equally sized and will have a combined capacity of 9,000 cfs.

Each pumping plant has an associated gravity flow system that is designed to provide operational flexibility by allowing water to gravity flow into BTF when it’s available in the system through the use of adjustable weir gates within the pump plant shaft. The weir gates are always set at slightly higher elevation than BTF water surface elevation whenever any of the pumps are in operation.

If hydraulic conditions are appropriate for gravity flow deliveries from the intakes to the BTF, pump operations are halted and the weir gates are lowered to allow water flow from the pump station shaft into BTF. Under such favorable hydraulic conditions, the amount of water that will gravity flow through the system will depend on the hydraulic differential of the water surface elevation between the pump plant shaft and BTF. The weir gates will also serve as system surge protection by allowing water to flow above the weir gates and out into BTF when the facilities experience a hydraulic surge.

Each pumping plant shaft will house six pumps (total of 12 pumps). Both pumping plants, when in combined operation, are capable of delivering the maximum design capacity of 9,000 cfs. Pump station details can be found in Section 8. In order to provide a wider operating range of flows, several of the pumps (or all of the pumps) can be equipped with variable frequency drives (VFD). VFD’s will allow adjustment of the pump speed to accommodate a wider range of flows from approximately 500 cfs up to 9,000 cfs. Flow will be measured on each individual pump discharge with calibrated ultrasonic flow meters.

The final number of pumps, the pump speed control systems, and level of automation condition will be finalized during preliminary design. The SCADA system will allow the operator to confirm that the pumping system is working in concert with the Sacramento River intake gates and assure that the intake sites allow delivery of the required pumping capacity to the tunnel conveyance system without violating the intake operating criteria. The pump plant will also incorporate an overflow weir that will be used as surge protection in the event of an operational upset, such as power failure.

5.3.7 Byron Tract Forebay Operations

The BTF is designed to provide daily operational storage to equalize and balance differences between inflow from the north Delta intakes and water exported by the Banks and Jones pumping plants. Under normal operating conditions, this leads to situations where inflow to BTF will exceed the outflow to the export facilities and vice versa. The daily amount of mismatch between inflow and outflow dictates the storage volume required at BTF.

The required daily operating storage is dictated by two operational situations:

- Inflow volume to BTF exceeds the operating capacity of the export pumping plants: This will happen when the WaterFix BTO is delivering at capacity and the export pumping plants are not pumping or are pumping at some lower rate.

- Export pumping volume from BTF exceeds the inflow volume: This higher export pumping rate typically occurs when Banks PP is running at a high capacity during the off-peak time of day.

During maximum export pumping operations, the Banks PP is operated at 10,670 cfs during the off-peak period and the Jones PP is operated at 4,600 cfs continuously. On a daily basis the combined volume of the export from both plants is about 19,500 AF. This volume exceeds the intake conveyance system’s maximum delivery capacity of approximately 17,800 AF, assuming that the north Delta intake facilities are diverting 9,000 cfs continuously for 24 hours. Thus, under these operating conditions, the export pumping plants can pump all of the water the BTF can supply. However, potential timing difference between export pumping and intake flows will require the use of daily storage to maximize river withdrawals while allowing the Banks PP to operate off-peak. These situations are described below for intermittent and continuous operating modes.
5.3.7.1 Intermittent Mode Daily Storage

To divert all available water during intermittent operations, the north Delta intake facilities must divert water during favorable river flows and tidal cycles. Diversions must be reduced or shut down during certain low river flows and unfavorable tidal conditions (high tides), regardless of the on-peak or off-peak energy situation. If the favorable intake diversion period occurs at the on-peak period at Banks PP, the water needs to be stored until the Banks PP returns to service during the off-peak interval. The worst case storage scenario occurs when the Jones PP is also off during the on-peak period and the north Delta intakes are diverting at the maximum rate. In this case, 4,500 AF of storage is needed for 6 hours.

5.3.7.2 Continuous Mode Daily Storage

If the north Delta Intake Facilities operates at full capacity for 24 hours, the volume of water delivered daily to the BTF is about 17,800 AF. If the Banks PP and the Jones PP are operated for an assumed 12-hour off-peak cycle at their maximum capacities of 10,670 cfs and 4,600 cfs, respectively, the exported volume from BTF is about 14,900 AF. If Jones PP is operated at full capacity and Banks pumping plants operates during off-peak hours, the daily exported volume would be about 19,500 AF. This volume exceeds the daily volume of the diversion capacity of the north Delta intake facilities, so, under maximum export conditions, either Banks PP or Jones PP cannot be run at full capacity during the isolated north Delta operation, assuming BTF is at the low water level initially.

In order for the Jones PP to operate at full capacity of 4,600 cfs for 24 hours per day, BTF would need to provide 9,100 AF of storage and Banks PP could only be operated at 8,800 cfs capacity during off-peak periods. Table 5-7 shows the “Jones Full Flow” scenario.

In order for Banks PP to operate at full capacity during off-peak hours, BTF would need to provide a minimum of 5,900 AF of storage, and Jones PP could only be operated at 3,100 cfs capacity during on-peak hours as shown in Table 5-7 for the “Banks Full Flow Off-peak” scenario. Similarly, if Jones PP were operated at 3,850 cfs continuously (on- and off-peak), Banks PP could operate at full capacity with 5,150 AF available BTF storage.

In order for both pumping plants to operate at full capacity during off-peak hours, the BTF storage would have to be augmented with flow from CCF (for Banks PP) or Old River (for Jones PP). In actual practice, Banks PP is not expected to operate at full capacity during off-peak hours, whereas the Jones PP is expected to operate at full capacity continuously. If Banks PP is operated at 9,000 cfs capacity during off-peak hours, BTF would need to provide a maximum of 4,500 AF of storage; and Jones PP could only be operated continuously at 4,500 cfs, as shown in Table 5-7 for the “Banks 9,000 cfs Off-peak” scenario.

Table 5-7: BTF Storage Balance Scenarios During Isolated North Delta Operations

<table>
<thead>
<tr>
<th></th>
<th>Jones Full Flow</th>
<th>Banks Full Flow Off-peak</th>
<th>Banks 9,000 cfs Off-peak</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>On-peak 12 Hours</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WaterFix BTO Supply</td>
<td>9,000 cfs</td>
<td>9,000 cfs</td>
<td>9,000 cfs</td>
</tr>
<tr>
<td>Export Pumps Use:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Banks</td>
<td>0 cfs</td>
<td>0 cfs</td>
<td>0 cfs</td>
</tr>
<tr>
<td>Jones</td>
<td>4,600 cfs</td>
<td>3,100 cfs</td>
<td>4,500 cfs</td>
</tr>
<tr>
<td>To Storage</td>
<td>4,400 AF</td>
<td>5,900 AF</td>
<td>4,500 AF</td>
</tr>
<tr>
<td>Off-peak 12 Hours</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>WaterFix BTO Supply</td>
<td>9,000 cfs</td>
<td>9,000 cfs</td>
<td>9,000 cfs</td>
</tr>
<tr>
<td>Export Pumps Use:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Banks</td>
<td>8,800 cfs</td>
<td>10,670 cfs</td>
<td>9,000 cfs</td>
</tr>
<tr>
<td>Jones</td>
<td>4,600 cfs</td>
<td>4,600 cfs</td>
<td>4,500 cfs</td>
</tr>
<tr>
<td>From Storage</td>
<td>4,400 AF</td>
<td>5,900 AF</td>
<td>4,500 AF</td>
</tr>
</tbody>
</table>

Notes:
AF = acre-feet
\( cfs = \text{cubic feet per second} \)

WaterFix BTO supply = Storage flowing through BTF

The flow scenarios presented in Table 5-7 represent a few possibilities for export flow conditions from BTF only, and storage requirements for those scenarios. In practice, flow scenarios will vary widely based on export demands, initial water levels in BTF, and a number of other outside factors.

In summary, three factors must be considered when developing the operation criteria for the isolated BTF scenario:

- The north Delta intake facility maximum diversion capacity of 17,800 AF per day alone will not support maximum capacity, 24-hour operations at both Jones and Banks PPs.
- For Banks PP to operate above 8,800 cfs (off-peak), Jones PP must be operated at a lower pumping rate, regardless of BTF storage volume.
- A daily BTF storage volume between 4,400 AF and 5,900 AF will be required to support the continuous pumping mode, with the higher end representing unusual operating conditions.

## 5.4 Maintenance Operations

The WaterFix BTO has features to improve operational redundancy and reliability enabling maximized conveyance capacity during maintenance operations. These features are summarized in Table 5-8.

<table>
<thead>
<tr>
<th>Table 5-8:</th>
<th>System Reliability and Redundancy</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Element</strong></td>
<td><strong>Project Feature</strong></td>
</tr>
<tr>
<td>Overall</td>
<td>BTF, Skinner Delta Fish Protection Facility, and Tracy Fish Collection Facility maintained in serviceable condition as redundant overall backup. Critical elements located above flood elevation, including allowance for SLR due to climate change impacts.</td>
</tr>
<tr>
<td>Intake</td>
<td>Three north Delta intake locations. Multiple screen bays per intake; can isolate independently.</td>
</tr>
<tr>
<td>Sedimentation Basin</td>
<td>Each intake location with twin sedimentation basins. Can isolate twin basins independently.</td>
</tr>
<tr>
<td>North Delta Intakes to IF (North Tunnels)</td>
<td>Tunnels and tunnel system from each intake to IF; during infrequent tunnel maintenance events, two intakes will remain in service. Intake No. 2 and Intake No. 3 share the same tunnel between the Junction Structure and IF.</td>
</tr>
<tr>
<td>IF to BT (Main Tunnels)</td>
<td>Two parallel tunnels, each with means of isolation upstream and downstream; 4,500 cfs capacity each. Ability to shut down one tunnel during periods of lower flow to maintain velocity in active tunnel.</td>
</tr>
<tr>
<td>BTF Pumping Plants</td>
<td>Six pumps provided in each of the two pumping plants, including a spare for the largest pump (5 duty, 1 standby) with a total maximum capacity of 9,000 cfs when both pumping plants are in combined operation.</td>
</tr>
<tr>
<td>Forebays</td>
<td>BTF storage from north Delta intakes and CCF storage from West Canal. Allowance for buildup of sediment below minimum operating level.</td>
</tr>
<tr>
<td>Communication System</td>
<td>Redundant communication paths and equipment.</td>
</tr>
</tbody>
</table>
SECTION 5.0 CONVEYANCE SYSTEM OPERATIONS

5.5 Implications of WaterFix BTO on Current SWP and CVP Operations

The WaterFix BTO changes the way water is conveyed to both Banks and Jones pumping plants, as follows:

- Adding common north Delta Intake Facilities with integral fish screens located along the Sacramento River upstream of Banks and Jones PPs.
- Utilizing a common conveyance system serving BTF that would be connected to both Banks and Jones PP.
- Maintaining CCF as only connected to the Banks PP.
- Removing tidal influence on water levels upstream of both export pumping plants when diverting from BTF.

The WaterFix BTO will also change other conditions as follows:

- Receiving water from BTF will require a greater level of daily operational coordination between DWR and Reclamation
- Common scheduling of individual pump operations at both Banks and Jones PP will be needed to manage the WSEs and volumes in both BTF and CCF and associated conveyance facilities.

Notes:

- \( \text{cfs} = \) cubic feet per second
- \( \text{IF} = \) Intermediate Forebay
- \( \text{BTF} = \) Byron Tract Forebay
- \( \text{CCF} = \) Clifton Court Forebay
- \( \text{SLR} = \) sea level rise
- \( \text{VFD} = \) variable frequency drive
SECTION 6.0
Conveyance System Hydraulics

This section describes the conveyance system hydraulics for major system components.

6.1 Facility Capacity

The conceptual engineering configuration for the WaterFix BTO has an ultimate maximum capacity of 9,000 cfs. The system is supplied by three intakes located on the Sacramento River, each with a capacity of 3,000 cfs, and a 9,000 cfs capacity pumping plant located at Byron Tract Forebay.

6.2 Preliminary Hydraulic Analysis

This section presents a preliminary assessment of the proposed operating and hydraulic conditions throughout the WaterFix BTO system.

6.2.1 WaterFix BTO System Description

The total system headloss from the Sacramento River Intakes to BTF utilizing dual main tunnels with equal capacities of 4,500 cfs is 19 feet. Table 6-1 lists the physical attributes of the major components of the WaterFix BTO system. For additional details, refer to the Concept Drawings (Volume 2) of each Intake Facility, IF, tunnel alignments, Byron Tract Forebay Pumping Plant; and to Section 3.0, “Overview of Conveyance Option.”

<table>
<thead>
<tr>
<th>System Component</th>
<th>Diameter (feet)</th>
<th>Length (feet)</th>
<th>Width (feet)</th>
<th>WSE Maximum</th>
<th>WSE Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connection Channel from South Tunnel Outlet to Jones PP Intake Channel</td>
<td>N/A</td>
<td>4,800</td>
<td>126 (base)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Connection Channel from South Tunnel Outlet to Banks PP Intake Channel</td>
<td>N/A</td>
<td>2,800</td>
<td>226 (base)</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Two Parallel Tunnels from BTF Outlet to South Tunnel Outlet</td>
<td>40</td>
<td>8,378 (each)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Byron Tract Forebay</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>12.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Two Parallel Tunnels from IF to BTF PP</td>
<td>40</td>
<td>158,023 (each)</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Intermediate Forebay*</td>
<td>N/A</td>
<td>1,500</td>
<td>800</td>
<td>25</td>
<td>-7</td>
</tr>
<tr>
<td>Intake No. 2 Tunnel to Intake 3 Junction Shaft</td>
<td>28</td>
<td>10,510</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Intake No. 3 Tunnel to IF</td>
<td>40</td>
<td>35,642</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Intake No. 5 Tunnel to IF</td>
<td>28</td>
<td>25,180</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Sacramento River at Intake No.2</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>31.4</td>
<td>1.9</td>
</tr>
<tr>
<td>Sacramento River at Intake No.3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>30.4</td>
<td>1.6</td>
</tr>
<tr>
<td>Sacramento River at Intake No. 5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>28.4</td>
<td>0.7</td>
</tr>
</tbody>
</table>

* IF length and width are at maximum WSE
** When Sacramento River elevation is +1 ft
Notes: IF =Intermediate Forebay
N/A=not applicable
6.2.1.1 Intake Hydraulic Considerations

Major physical elements associated with Intakes 2, 3, and 5 include:

- Intake screens.
- Intake Collector Box Conduits.
- Sedimentation basins.
- Drop Structures.
- Discharge tunnels to IF.

Intake hydraulics are primarily affected by Sacramento River water levels, tunnel lengths, and gravity flow rates in the system. The historical Sacramento River stage near the intake locations has ranged from +1 feet to over +25 feet, but remains below +6 feet for nearly 50% of the time. The fish screens are designed such that the maximum diversion (3,000 cfs) is possible at the lowest river elevation, but the system is also flexible enough to take the maximum diversion over a wide range of river stages.

For all intakes, water is diverted through the fish screens and flows through the intake collector box conduits to the sedimentation basins. Water exiting Intake No. 2 will flow into a tunnel shaft drop structure. The flow from the tunnel shaft is taken to a junction structure via a 28-ft ID tunnel. At the junction structure, the flows from Intakes 2 and 3 are combined and conveyed through a single 40-ft ID tunnel to the IF. The flow from intake 5 is taken directly to the IF using a single 28 ft ID tunnel. Flow will be controlled at each intake using sluice gates and flowmeters located within the intake collector box conduits.

6.2.1.2 Intermediate Forebay Hydraulic Considerations

The IF is mainly a “pass-through” facility. Water levels are primarily affected by influent flow, and outflows. The water surface elevation in the IF will vary between the +25 feet and minimum elevation of -20 feet.

6.2.1.3 Main Tunnel Hydraulic Considerations

Main Tunnel hydraulics is primarily affected by gravity flow from the IF to the suction side Pump Shaft and pumping operations into BTF. Each tunnel will include a vertical drop shaft at the upstream end and will terminate at the pump shaft at the downstream end. It is assumed that vertical shafts at the IF outlets (Main Tunnel inlets) will be designed accordingly to meet all hydraulic requirements for flow deliveries and to avoid adverse hydraulic processes, such as air entrainment or vortex formation within the shaft.

6.2.1.4 Byron Tract Forebay Hydraulic Considerations

The BTF provides the daily operational storage required to equalize and balance differences between the north Delta inflow and water exported by the existing SWP and CVP pumps. The BTF has been initially sized for an operational storage capacity range of approximately 5,960 to 8,350 acre-feet. The operating range of BTF was calculated to range from 2.5 to 12.5 feet. In order to deliver the maximum capacity required by both Banks PP and Jones PP and to meet the minimum and maximum operating range, the higher WSE from either pump station will dictate.

**North Delta Operation:** The maximum diversion capacity of the north Delta intake facilities is 9,000 cfs. The WSE at BTF should be such that the export pumping plants are operating in their preferred operating range. In order to continue the existing operations, the WSE in the BTF will be between 2.5 feet and 12.5 feet.

**South Delta Operation:** The WSE of CCF will vary depending upon the diversion schedule and export to Banks PP. For isolated south Delta operations, the demand of the Jones pumps will be met through Old River diversion facility. Since the flow is not contributed by the north Delta Intake Facilities, the WSE of BTF will have no impact.

6.2.1.5 South Tunnels Hydraulic Considerations

Flow that leaves BTF would be conveyed via two 40-ft ID tunnels (South Tunnels) that terminate at an outlet structure south of Byron Highway. The outlet structure is connected to two connection channels that deliver flows to Banks PP and Jones PP respectively. For physical characteristics of proposed connection channels see Section 10.0 (Connection Channels).
6.2.1.6 Connection Channels Hydraulic Considerations

Connection Channel between South Tunnel Outlet Structure and Banks PP Intake Channel: The connection channel between the South Tunnel Outlet Structure and the Banks PP Intake Channel is sized for a flow of 10,670 cfs. Flow would be conveyed into the connection channel based on the current pumping demand at the Banks PP.

Connection Channel between Outlet Structure and Jones PP Intake Channel: The connection channel between the Outlet Structure and the Jones PP Intake Channel is sized for a flow of 4,600 cfs. Flow would be conveyed into the connection channel based on the current pumping demand at the Jones PP.

6.2.1.7 Control Structures

As part of the conveyance system upstream of the export pumping plants, it is proposed that there will be control structures added within the connection channels downstream of BTF, and within the Banks and Jones PP intake channels, downstream of the fish facilities. The control structures will be located as follows:

- In each connection channel downstream of outlet structure
- Within the Banks PP Intake Channel between the Skinner Fish Facility and Banks PP Connection Channel
- Within the Jones PP Intake Channel north of Byron Highway and downstream (south) of the Tracy Fish Facility

The conceptual locations for the control structures are shown in Volume 2; the size, configuration, and location of the structures should be evaluated against operational needs of the system in subsequent design phases of the project.

The control structures will house gates capable of isolating and controlling the upstream water flow from downstream facilities, and potentially throttling to control flow schemes within the conveyance system.

6.2.1.8 System Hydraulic Calculations

In order to determine conveyance system hydraulic performance for certain critical hydraulic conditions, the WaterFix BTO system was modeled using real-time hydraulic modeling software called Innovyze InfoWorks CS to model steady state and dynamic real-time conditions (see Appendix H for detailed hydraulic analysis). Results from this modeling effort will be considered in the overall evaluation of the conveyance configuration.

Steady state conditions were modeled for a total conveyance of 9,000 cfs from Intakes 2, 3, and 5 (3,000 cfs per intake). The steady state analysis was based on a low Sacramento River El. of +1 ft and a high El. of +10 ft. This boundary range, coupled with the system losses, determined key hydraulic grade elevations at Junction Shafts, Intermediate Forebays, Main Tunnel Shafts, and at the BTF Pumping Plant. The dynamic analysis was based on various historical delivery trends in the Delta system from (1974-1991) and demand patterns were determined and modeled.

A worse case condition of Intake 2 closing due to emergency conditions was modeled to determine any adverse effects to the system. It was determined that control schemes would be in place to control the flow from Intakes 3 and 5 to stay within the guidelines and criteria of not exceeding 3,000 cfs per intake and also not exceeding the allowable fish screen velocity of 0.2 fps even during this “upset” condition of Intake 2 closing. The results are similar for any of the three intakes undergoing an emergency closure.

The calculated system head loss is mostly driven by the friction losses through the tunnels. Pressurized pipe calculations represented the tunnels and used the Manning’s friction coefficient of n = 0.0145, which is a conservative design value for slightly rough internal surface conditions in segment-lined tunnels.

Calculated head losses and associated hydraulic grade lines (HGL) throughout the WaterFix BTO system are depicted in the hydraulic profiles in the Concept Drawings (Volume 2).
6.3 Surge Evaluation

Surge or hydraulic transients can occur during a sudden start-up or sudden closure of the pumps or valves. The most critical surge condition is often the result of an uncontrolled sudden pump shutdown caused by a general power failure. The surge results in a local sudden pressure drop propagated throughout the interconnected conduits and then a reverse flow of high pressure. The surge analysis was conducted to provide conceptual level facility sizing for surge control facilities at the BTF Pumping Plant.

A surge analysis was conducted to evaluate any adverse effects to the conveyance system and associated facilities. The surge analysis was based on the following key assumptions:

- Intakes 2, 3, and 5 in operation delivering 3,000 cfs each (Total System Flow = 9,000 cfs).
- Sacramento River El. +10 ft.
- Wave Speed “a” = 1700 fps for all tunnels.
- Average Intermediate Forebay floor el. = -20 ft.
- Surge based on a Pump Trip of 10 seconds.
- Surge overflow weir at Combined Pump Station = 14.6 ft.

There were no critical observations in any of the tunnels, and maximum surge HGL’s for various locations along the tunnel alignment are shown in the Surge Analysis Technical Memorandum Appendix D.

Three various IF sizes were also evaluated for sensitivity analysis. The three sizes evaluated are listed below (dimensions shown are at IF bottom el.-20 ft):

1. 500 ft W x 500 ft H
2. 800 ft W x 1000 ft H
3. 800 ft W x 1500 ft H

Three key hydraulic parameters were observed and are summarized in the Table 6-2 below. They include Peak Backflow to each Intake, Peak Backflow Velocity through the Sedimentation Basins, and Backflow Volume.

<table>
<thead>
<tr>
<th>Intake No.</th>
<th>IF = 500' x 500'</th>
<th>IF = 800' x 1,000'</th>
<th>IF = 800' x 1,500'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Backflow (cfs)</td>
<td>Peak Backflow Velocity (ft/s)</td>
<td>Backflow Volume (ac-ft)</td>
</tr>
<tr>
<td>2</td>
<td>37</td>
<td>0.010</td>
<td>2.4</td>
</tr>
<tr>
<td>3</td>
<td>166</td>
<td>0.046</td>
<td>11.3</td>
</tr>
<tr>
<td>5</td>
<td>217</td>
<td>0.058</td>
<td>12.8</td>
</tr>
</tbody>
</table>

Based on a pump trip of all 10 pumps, results show backflows to each intake will vary based on the transient waves being dampened out at the IF for each size evaluated. Sedimentation Basin velocities are shown, and upon initial inspection do not appear to be critical, but they should formally be compared to maximum allowable velocities to avoid sediment disturbance.

The surge analysis results show that any of the three IF footprint sizes can be designed and do not provide any fatal flaws.
6.3.1 Intermediate Forebay Size Evaluation

The IF included in the concept design will provide an atmospheric break in the system between the intake tunnels and the Main Tunnels. The IF is designed to be a pass-through facility without flow control, with only isolation drop gates for system maintenance and inspection. The IF will serve several functions that are essential to the system:

- Provide an efficient and constructible method to link the northern tunnels with the main tunnels.
- Provide a hydraulic buffer between the intakes and pump station in the event of an emergency or sudden pump shutdown.
- Provide access point to dewater the northern or the main tunnels into the forebay.
- Prevent flow from short circuiting and balance the flow split to the main tunnels.

The IF will be sized as small as practicable to optimize all the aforementioned functions.

To determine if short circuiting was an issue due to the unequal flow discharging from the two forebay inlets (6,000 cfs from Intakes 2 and 3, and 3,000 cfs from Intake 5), a computational fluid dynamics model (Fluent, Ansys Inc) was built to evaluate the flow distribution across the forebay and into the two 40 ft I.D. tunnels. Four different sizes were evaluated and all four showed equal distribution of roughly 50% - 50% into each 40 ft I.D. tunnel.

Computational Fluid Dynamics (CFD) plots for the four IF footprints are shown below in Figure 6-1. The plots in Figures 6-2 through 6-5 show velocity contours and flow distributions percentages into each 40 ft ID main tunnel.
Figure 6-1: Intermediate Forebay CFD Cases

<table>
<thead>
<tr>
<th>Case No.</th>
<th>L (ft)</th>
<th>W (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,500</td>
<td>800</td>
</tr>
<tr>
<td>2</td>
<td>1,000</td>
<td>800</td>
</tr>
<tr>
<td>3</td>
<td>500</td>
<td>800</td>
</tr>
<tr>
<td>4</td>
<td>500</td>
<td>500</td>
</tr>
</tbody>
</table>

Figure 6-2: Intermediate Forebay CFD Case 1

Velocity Contours (fps) at Water Surface
Case 1: 1500 ft X 800 ft
Figure 6-3: Intermediate Forebay CFD Case 2

Figure 6-4: Intermediate Forebay CFD Case 3
SECTION 6.0 CONVEYANCE SYSTEM HYDRAULICS

It can be concluded from the CFD results that any of the four IF sizes evaluated are able to distribute the flow equally to the forebay outlets that discharge directly into the 40 ft ID main tunnels. Velocities across the forebay are less than 2 fps and provide stable equalization of flow.

Providing a storage time or buffer time for the operation of the pumps due to an emergency event at any of the intakes is also an important feature. Based on the four footprints evaluated in the CFD analysis, the corresponding buffer time for each geometric configuration of the IF is calculated and shown below in Table 6-3.

Table 6-3: Intermediate Forebay Size Evaluation

<table>
<thead>
<tr>
<th>Case</th>
<th>Top of Forebay Embankment El. (ft)</th>
<th>Invert Elevation (ft)</th>
<th>L₁ at Invert Elevation (ft)</th>
<th>W₁ at Invert Elevation (ft)</th>
<th>L₂ at Top of Forebay (ft)</th>
<th>W₂ at Top of Forebay (ft)</th>
<th>Storage Time from Invert to El. 0 (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>32.2</td>
<td>-20</td>
<td>1,500</td>
<td>800</td>
<td>1,918</td>
<td>1,218</td>
<td>52</td>
</tr>
<tr>
<td>2</td>
<td>32.2</td>
<td>-20</td>
<td>1,000</td>
<td>800</td>
<td>1,418</td>
<td>1,218</td>
<td>35</td>
</tr>
<tr>
<td>3</td>
<td>32.2</td>
<td>-20</td>
<td>500</td>
<td>800</td>
<td>918</td>
<td>1,218</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>32.2</td>
<td>-20</td>
<td>500</td>
<td>500</td>
<td>918</td>
<td>918</td>
<td>13</td>
</tr>
</tbody>
</table>
Four main components can be compared when determining which IF size should be designed: surge effects, equal flow distribution, storage time (buffer for pump operations), and cost/constructability. For the WaterFix BTO concept, the largest footprint (800 ft Wide x 1500 ft High at invert Elevation -20 ft) is shown in the Concept Drawings (Volume 2) until further optimization of the above components can be evaluated fully.
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SECTION 7.0
Intakes and Sedimentation Facilities

7.1 Description and Site Plans

The Intake and Sedimentation Facilities are designed to divert up to 9,000 cfs from the Sacramento River. These facilities consist of Fish Screens, Sedimentation Basins, isolation gates, flow control gates and sediment drying lagoons. The three intakes sites along with sedimentation basin facilities are located at the following sites along the Sacramento River:

<table>
<thead>
<tr>
<th>Intake Location</th>
<th>River Mile</th>
</tr>
</thead>
<tbody>
<tr>
<td>WaterFix BTO Intake No. 2</td>
<td>41.1</td>
</tr>
<tr>
<td>WaterFix BTO Intake No. 3</td>
<td>39.4</td>
</tr>
<tr>
<td>WaterFix BTO Intake No. 5</td>
<td>36.8</td>
</tr>
</tbody>
</table>

These intake locations were selected by DWR through a consultation process with the FFTT by assessing various studies and previous CERs. Figure 7-1 shows the location of the selected sites. Intake numbering is consistent with the original All Tunnel Option (ATO) CER and PTO CER numbering system.

Each intake and sedimentation facility will be sized to divert up to 3,000 cfs. The intakes have on-bank fish screens as shown in the Concept Drawings (Volume 2). The various control gates will be utilized to ensure compliance with the approach velocity of 0.2 fps at the fish screens and the 3,000 cfs maximum flow per intake. The sedimentation basins will be designed to remove settleable solids before entering the conveyance system.

A conceptual rendering of the on-bank intake and sedimentation facilities is shown in Figure 7-2 and in the Concept Drawings (Volume 2).

The intake configurations depicted in this CER will be further refined and optimized in consultation with the FFTT and applicable resource agencies through a series of focused and collaborative technical studies.

7.2 Intake General Arrangement

The main components of the Intake Facilities include:

- On-bank intake screening structures that divert flow from the Sacramento River
- Collector box conduits that funnel the flow to the sedimentation basins.
- Flow meters and flow control sluice gates located on each box conduit assure limitations on approach velocities and flow balancing between the three intake facilities are achieved.
- Isolation drop gates that allow for maintenance of the flow meters and flow control sluice gates.
- A sedimentation system that includes sedimentation basins that provide for the settling and removal of sandy material and coarse silts prior to the flow entering the conveyance tunnels and a sedimentation outlet structure that connects the Intake Facilities to the North Conveyance Tunnels

These components are described in more detail below.
Figure 7-1  Potential Intake Locations
### 7.2.1 Intake Structures

Intakes will be on-bank structures with fish screens similar to the Sacramento River intakes owned by the Freeport Regional Water Authority (FRWA), Glenn-Colusa Irrigation District, and Tehama-Colusa Canal Authority. Each of the three WaterFix BTO sites will vary slightly in terms of bathymetric conditions and design river levels. All of the intakes are sized at the design WSE to provide approach velocities at the fish screen of less than or equal to 0.20 fps at an intake flow rate of 3,000 cfs. The approach velocity of less than or equal to 0.20 fps is recommended by the State and Federal fish agencies for protecting Delta Smelt. The design WSE for each site was established as the 99 percent exceedance (Sacramento River stage) elevation. The maximum design WSE was established as the 200-year flood elevation plus an 18-inch allowance for SLR. Table 7-1 provides information on intake WSE, and Table 7-2 includes intake conceptual design criteria.

**Table 7-1: Intake Site-Specific Water Surface Information**

<table>
<thead>
<tr>
<th>Intake Designation</th>
<th>Design WSE (feet, NAVD88)</th>
<th>Maximum Design WSE (feet, NAVD88)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake No. 2</td>
<td>1.9</td>
<td>31.4</td>
</tr>
<tr>
<td>Intake No. 3</td>
<td>1.6</td>
<td>30.4</td>
</tr>
<tr>
<td>Intake No. 5</td>
<td>0.7</td>
<td>28.4</td>
</tr>
</tbody>
</table>

**Notes:**

NAVD88 = North American Vertical Datum of 1988

WSE = water surface elevation

**Table 7-2: Intake Structure Conceptual Design Criteria**

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General</strong></td>
<td></td>
</tr>
<tr>
<td>Intake Technology</td>
<td>On-bank intake</td>
</tr>
<tr>
<td>Number of Intakes</td>
<td>3</td>
</tr>
<tr>
<td>Maximum Single Intake Structure Capacity</td>
<td>3,000 cfs</td>
</tr>
<tr>
<td>Design Minimum WSE (feet, NAVD88)</td>
<td>99 percent exceedance elevation (see Table 7-1 for specific elevations)</td>
</tr>
<tr>
<td><strong>Intake Hydraulic Criteria</strong></td>
<td></td>
</tr>
<tr>
<td>Screen Approach Velocity</td>
<td>0.20 fps</td>
</tr>
<tr>
<td>Screen Sweeping Velocity</td>
<td>≥0.20 fps (subject to verification)</td>
</tr>
<tr>
<td><strong>Screens</strong></td>
<td></td>
</tr>
<tr>
<td>Type</td>
<td>Fixed vertical flat-plate profile bar screen</td>
</tr>
<tr>
<td>Material</td>
<td>304 stainless steel</td>
</tr>
<tr>
<td>Screen Slot Opening Size</td>
<td>1.75 millimeters (0.069 inch)</td>
</tr>
<tr>
<td>Screen Porosity</td>
<td>27 percent</td>
</tr>
<tr>
<td><strong>General Arrangement</strong></td>
<td></td>
</tr>
<tr>
<td>Capacity of Screen Bay Group (six per intake)</td>
<td>500 cfs</td>
</tr>
<tr>
<td>Screen Bay Clear Width</td>
<td>15 feet</td>
</tr>
<tr>
<td>Auxiliary Slot for Solid Panels</td>
<td>Required for fish screen panel removal</td>
</tr>
</tbody>
</table>
### TABLE 7-2 (continued)

#### Intake Structure Conceptual Design Criteria

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structural (continued)</strong></td>
<td></td>
</tr>
<tr>
<td>Maximum Flow Control Baffle Head Differential</td>
<td>2 feet</td>
</tr>
<tr>
<td>Floor</td>
<td>Concrete floor supported on steel cased drilled piers</td>
</tr>
<tr>
<td>Breast Wall, Interior Piers and Back Wall</td>
<td>Reinforced concrete</td>
</tr>
<tr>
<td>Cofferdam Front and Center Walls</td>
<td>Steel piles</td>
</tr>
<tr>
<td>Intake Box Conduit Cofferdam Back Wall</td>
<td>Concrete diaphragm wall</td>
</tr>
<tr>
<td><strong>Fish Screen Cleaning System</strong></td>
<td></td>
</tr>
<tr>
<td>Motor</td>
<td>Gear motors with VFD</td>
</tr>
<tr>
<td>Number of Cleaning Systems</td>
<td>One per Screen Bay Group</td>
</tr>
<tr>
<td>Cleaning Assembly Speed Range</td>
<td>0.5 to 2 fps</td>
</tr>
<tr>
<td>Cycle Time</td>
<td>5 minutes, maximum</td>
</tr>
<tr>
<td><strong>Sediment Jetting System</strong></td>
<td></td>
</tr>
<tr>
<td>Pump Capacity</td>
<td>2,500 gpm</td>
</tr>
<tr>
<td>Number of Pumps</td>
<td>One per screen bay group</td>
</tr>
<tr>
<td>Pump Type</td>
<td>Vertical turbine</td>
</tr>
<tr>
<td>Pump Motor Horsepower</td>
<td>100</td>
</tr>
<tr>
<td>Nozzle Spray Pressure</td>
<td>20 psi</td>
</tr>
<tr>
<td>Flow per Nozzle</td>
<td>25 gpm (first row – nearest to the fish screen); 100 gpm (second, third, and forth rows)</td>
</tr>
<tr>
<td>Pipe Material</td>
<td>Stainless steel</td>
</tr>
<tr>
<td><strong>Flow Control Baffles</strong></td>
<td>Produce uniform velocity distribution (horizontal and vertical)</td>
</tr>
</tbody>
</table>

#### Notes:

- \( \geq \) = greater than or equal to
- cfs = cubic feet per second
- FB = freeboard
- fps = feet per second
- gpm = gallons per minute
- NAVD88 = North American Vertical Datum of 1988
- psi = pounds per square inch
- SLR = sea level rise
- WSE = water surface elevation
The fish screen system consists of screen panels and flow control baffles that form a barrier to prevent fish from being drawn into the intake and a traveling screen cleaning system. A log boom protects the screens and screen cleaning systems from impact by large floating debris. Screen panels are installed in the lowest portion of the intake structure face, and solid panels are stacked above them in guides extending above the deck of the structure. These panels and their respective installation guides are stainless steel.

The screen panels are arranged in groups, with each screen bay group providing sufficient screen area for 500 cfs of diversion. There are six separate screen bay groups per Intake Facility, all of which are hydraulically independent.

Each screen bay group has a dedicated traveling screen cleaning system. These screen cleaners are supported by a monorail and driven by an electric motor and cable system. Space must be provided for the six screen cleaners and their associated parking area and drive system.

Flow control baffles are behind each screen panel and are installed in guides to accommodate complete removal of the baffle assembly for maintenance. These flow control baffles are designed to evenly distribute the approach velocity to each screen such that it meets the guidelines developed by the FFTT. The flow control baffle guides also serve as guides for installing bulkhead gates (after removal of the flow control baffles) for maintenance of a screen bay group. The bulkhead gates are designed to permit dewatering a screen bay group under normal river conditions.

A sediment jetting system will be used to suspend sediment deposits on the floor of each bay and mitigate sediment accumulation (shoaling) on the front sill of the intake. The system, which consists of a vertical turbine pump and associated piping, control valves, and spray nozzles, will be designed to run periodically. The sediment jetting pump will pressurize water from the pipe manifold located behind the back wall of the intake structure and deliver it to the spray nozzles, which will spray the bay floor. The re-suspended sediment is then conveyed to the sedimentation basins through the intake piping. The piping configuration to mitigate shoaling will be evaluated during preliminary engineering and final design.

All fish screen bay groups are separated by piers with appropriate guides to allow for easy installation and removal of screen and solid panels as well as the flow control baffle system and bulkheads. These items are removed from the deck by either a mobile or gantry crane. Piers support the operating deck set with a FB of 18 inches above the 200-year flood level with SLR. The levee in the immediate area is raised to provide a FB of 3 feet above the 200-year flood level with SLR. Sheet pile training walls have a radius of 200 feet and are upstream and downstream of the intake structure, providing improved river hydraulics and vehicular access to the operating deck as well as transitioning the intake structure to the levee.

A common plenum area behind each screen bay group collects and funnels the flow towards intake collector box conduits located at the back of the intake structure. The intake box conduits include isolation drop gates that are to be closed during the periods of extremely high river stage. The isolation drop gates are manually installed using a side boom mobile crane. An emergency electrical power source (an engine-generator with a capacity of approximately 250 kilowatts) may be used to close the electrically actuated control gates during concurrent periods of high river stage and utility power outage. Gate closure is required by USACE and CVFPB. Site lighting at the intake facilities can also be provided by this emergency power source.

The configuration of the intake structure from the screen face to the intake box conduits on the back wall, including the flow baffles, will be evaluated in detail using computational fluid dynamics and physical modeling during the preliminary engineering phase. The goal is to configure the structure so that uniform flow through the fish screens is not impeded by hydraulic conditions induced by the intake structural configuration. Modeling may suggest additional structure width and flow shaping or training walls within the structure.

Each intake includes a modular floating dock along the downstream training wall of the intake structure and an access staircase to board the floating work platform.

It is proposed that Intake No. 3 also include maintenance buildings for all three intake sites, based on similar facilities that DWR owns and operates.
Site security is provided at each intake. Fencing secures each site and prevents public access to sensitive areas and those with potential hazards. Security camera systems and intrusion alarm systems are in the site areas, at major control structures, and at all buildings. There is credentialed entry through access control gates and secure doors to the sites and buildings.

7.2.3 Intake Collector Box Conduits

Each intake consists of 12 intake collector box conduits. The box conduits are sized at 12 ft x 12 ft to minimize hydraulic losses and provide typical operating velocities that exceed 2.0 fps keeping sediments in suspension. The intake collector box conduits extend through a widened levee section and terminate with a wing wall transition structure located in the sedimentation basins. The length of each box conduit is approximately 375 feet, which allows for construction of permanent relocation of Highway 160 as part of the initial construction sequencing (see Section 12.0, “Access to WaterFix Sites” and Section 15.0, “Levees,” for further discussion of construction sequencing related to the relocation of Highway 160 and widening of the existing levee). The box conduit outlets into the sedimentation basins have a flared configuration to enhance flow distribution into the sedimentation basins.

It was determined that the most effective way to ensure proper flow control/distribution among the three intakes was to install flow control slide gates just downstream of the intake plenum. To achieve this objective 8 ft x 8 ft slide gates were installed near the entrance to each 12 ft x 12 ft box conduit. The flow rate in each box conduit is controlled by an electrically actuated 8 ft x 8 ft sluice gate located downstream of the drop gate at the beginning of the channel. Flow measurement within each box conduit will be provided by a multipath ultrasonic flowmeter. Meter accuracy of plus or minus two percent of actual flow will be achieved under both open channel and pressurized flow conditions in the box conduits. Each flow control sluice gate will be modulated by its dedicated flowmeter, allowing for independent operation of each intake box conduit and maximum flexibility to vary flow within each fish screen bay and between each of the three Intake Facilities. Sluice gate positions will be calibrated at system start-up and proper gate positions will be regularly confirmed as a part of normal system operations. Drop gates provided at each end of the intake channels allow for dewatering of the box conduits, removal of any accumulated sediments, and maintenance and repair of the sluice gates and flowmeters.

Intake box conduit size, length and configuration will be confirmed during preliminary engineering and final design. The size of the box conduit openings may be optimized to balance hydraulic losses against a reduced conduit cross-sectional area that will improve conduit velocities and reduce the potential for silt accumulation. Following periods of low intake rates, intermittent operation of each intake conduit at full capacity will scour sediments through the intake conduits and into the sedimentation basin. A cross-sectional area no greater than 12 ft. x 12 ft. may allow the use of pre-cast concrete units, which have the potential to reduce installation times and provide a corresponding reduction in construction costs.

The intake collector box conduits are shown in the Concept Drawings (Volume 2).

7.3 Sedimentation System General Arrangement

The sedimentation system at each intake site consists of a sediment jetting system in the intake structure that will re-suspend accumulated sediments for transport to the intake collector box conduits; twin unlined-earthen sedimentation basins for sediment capture; hydraulic dredging equipment and sludge conveyance piping for annual removal of accumulated sediments in the earthen basins; and sediment drying lagoons for drying and consolidating prior to disposal. These components of the sedimentation system are described below. General design criteria for the sedimentation facilities are shown in Table 7-3.
### Table 7-3: Sediment Collection System Design Criteria

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General</strong></td>
<td></td>
</tr>
<tr>
<td>Maximum Single Intake Structure Capacity</td>
<td>3,000 cfs</td>
</tr>
<tr>
<td><strong>Sediment Basin Criteria</strong></td>
<td></td>
</tr>
<tr>
<td>Sedimentation Inflow (Assumes twin sedimentation basins per intake site)</td>
<td>1,500 cfs per side</td>
</tr>
<tr>
<td>Normal Settling Depth</td>
<td>20 feet (below design WSE)</td>
</tr>
<tr>
<td>Settling Depth at Design WSE</td>
<td>11.7 feet</td>
</tr>
<tr>
<td>Basin Size (triangular shape)</td>
<td>Width = 250 to 660 feet (floor dimensions; max width at intake channels; min width at outlet); length = 600 feet (sediment settling zone only)</td>
</tr>
<tr>
<td>Projected Annual Peak Year Sediment Accumulation (per intake site)</td>
<td>34,400 cy (calculated)</td>
</tr>
<tr>
<td>Sediment Storage Depth</td>
<td>3 to 5 feet below settling depth (see above)</td>
</tr>
<tr>
<td>Sedimentation Basin Side Slopes</td>
<td>Unlined earthen slopes, 2:1 side slopes (all or portions of basin side walls may be armored for scour or erosion control)</td>
</tr>
<tr>
<td>Particle Size Retention</td>
<td>Particle sizes larger than 0.075 mm at design flow conditions</td>
</tr>
<tr>
<td>Basin Floor</td>
<td>Earthen over soil-conditioned subgrade</td>
</tr>
<tr>
<td>Basin Top of Slope Elevation</td>
<td>200-year WSE with SLR plus 3 feet</td>
</tr>
<tr>
<td>Maintenance</td>
<td>Each basin may be isolated using drop gates at both the exits from the intake channels and at the inlets to the tunnel drop structure. Provision for dewatering the sedimentation basin is not included.</td>
</tr>
<tr>
<td>Sediment Removal</td>
<td>Use suction dredge to remove accumulated sediment</td>
</tr>
<tr>
<td>Dredging Frequency</td>
<td>Assumed once per year based on peak year sediment loading to project</td>
</tr>
<tr>
<td>Dredge Discharge</td>
<td>Discharge to sediment drying lagoons by way of piped connection points</td>
</tr>
</tbody>
</table>

### Sediment Drying Lagoons

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number Required</td>
<td>4 lagoons per intake site</td>
</tr>
<tr>
<td>Size</td>
<td>Bottom width = 160 feet; bottom length = 350 feet</td>
</tr>
<tr>
<td>Depth</td>
<td>15 feet</td>
</tr>
<tr>
<td>Volume</td>
<td>860,000 cubic feet (per drying lagoon)</td>
</tr>
<tr>
<td>Inflow Rate</td>
<td>5.57 cfs (2,500 gpm)</td>
</tr>
<tr>
<td>Sides/Bottom</td>
<td>Reinforced shotcrete/roller-compacted concrete</td>
</tr>
<tr>
<td>Side Slopes</td>
<td>1:1</td>
</tr>
<tr>
<td>Access for Maintenance</td>
<td>Equipment ramps into each lagoon</td>
</tr>
<tr>
<td>Dewatering</td>
<td>Integrated underdrain system and decant system</td>
</tr>
</tbody>
</table>

---

* Annual sediment load to five intakes is shown in Table 4 of the referenced memorandum (DWR, undated).

**Notes:**

- % = percent
- cfs = cubic feet per second
- gpm = gallons per minute
- SLR = sea level rise
- WSE = water surface elevation
7.3.1 Sediment Jetting System

There are two general areas of sediment management at the intake structure: sediment accumulation (shoaling) in the river immediately outside the intake structure, and sediment accumulation once the sediment passes through the fish screens.

The shoaling outside the structure could impede the progress of the fish screen cleaners and, in an extreme case, block the fish screens and reduce the diversion capacity. Mitigations for shoaling need detailed geomorphology studies and physical and numerical modeling that predict potential changes through time of bed form. Raising the sill invert of the fish screens is an obvious mitigation, but it would require lengthening the screens. Other mitigations include sediment jetting systems on the front sill of the fish screen as shown in the Concept Drawings (Volume 2) and/or routine operation of the fish screen cleaner. Dredging may be required for mitigation in extreme cases of shoaling, regardless of whether or not other mitigation measures are employed.

For this CER, it is assumed that the screen inverts are at least 3 feet above the river bottom, and there is sediment jetting inside the structure and routine operations of the screen cleaners. This has been effective with similar sediment loading conditions. More detailed study of this issue will be needed at subsequent stages of design.

Inside the intake structure, the sediment jetting system suspends the sediment for transport into the intake collector box conduits. This system has proven to be effective in moving sediment out of the intake structure in similar facilities (intakes owned by Reclamation District 108 and Sutter Mutual Water Company), but dewatering and excavation might be required periodically.

7.3.2 Sedimentation Basins

Each sedimentation basin is an unlined, earthen gravity settling basin, as noted above and shown in the Concept Drawings (Volume 2). The basin is divided by an earthen berm running the full length of the basin, with three fish screen bays connected by the intake conduits serving each half of the overall sedimentation basin. The side slopes and the interior dividing berm are constructed with a side slope ratio of 2 horizontal to 1 vertical (2:1). Based on the flow velocity through the basin, flatter side slopes or armoring of the slopes to address erosion or scouring is not warranted. However, as determined during preliminary design, some side slope areas adjacent to the intake conduit and within the outlet zone of the basin may be reinforced with revetments. While each half of the sedimentation basin may be isolated using the drop gates located in the intake conduits and at the entrance to the outlet structure, there are no plans to dewater the sedimentation basin on regular intervals. Instead, the basins will be cleaned while containing water, as described in later portions of this section.

The sedimentation basin is triangular in shape to accommodate the wider geometry and uniform spacing of the intake conduits compared to the narrowing of the basin at the outlet structure. The maximum width of each half basin, measured at the intake channel along the floor is 677 ft for Intakes 2 and 5, and 575 ft for Intake 3. Dimensions for each basin are shown in Table 7-4.

<table>
<thead>
<tr>
<th>Intake Designation</th>
<th>Basin Width Intake Channel (feet)</th>
<th>Basin Width Outlet Zone (feet)</th>
<th>Basin Length (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake No. 2</td>
<td>677</td>
<td>265</td>
<td>660</td>
</tr>
<tr>
<td>Intake No. 3</td>
<td>575</td>
<td>265</td>
<td>660</td>
</tr>
<tr>
<td>Intake No. 5</td>
<td>677</td>
<td>265</td>
<td>660</td>
</tr>
</tbody>
</table>

Notes:
Width measured at the floor elevation of each twin basin
The sedimentation basins are sized to (1) provide a sufficiently low transit time within the basin such that the targeted size range of settleable solids will be removed from the river diversions prior to the flow entering the tunnel intake structure; and (2) provide a sufficient volumetric area at the basin floor elevation for the accumulation on an annual basis of the anticipated peak year sediment loading condition. Given that the basin geometry is dictated by the spacing of the collector intake channels, as noted above, the length of the sedimentation basins were determined using Stoke’s Law to calculate the average settling duration and corresponding settling distance to retain particles sized 0.075 mm and larger within the sedimentation basin. At the minimum design WSE of 11.7 feet, the average flow-through velocity in the sedimentation basin is 0.47 ft/sec, and the calculated distance to settle out 0.075 mm sized particles is approximately 400 feet. This distance does not take into account non-uniform flow at both the entrance and exit facilities and a 100 foot buffer zone was added at each end of the basin for flow transitions.

With the above average flow-through velocity, some of the smaller sized particles (less than 0.075 mm) are also retained in the sedimentation basins before the flow enters the tunnels leading to the IF. Particles smaller than 0.002 mm are considered colloidal, and with a settling velocity of less than 0.0001 centimeter per second and a very low shear velocity requirement for re-suspension, these particles are considered non-settling and not collectible by a gravity system. Some 0.075-mm to 0.002-mm material will flow into the deep tunnel system where it is conveyed to downstream facilities. Some smaller sized particles will be captured in the IF, which operates with cross-sectional velocities similar to those in the sedimentation channels.

Sediment transport within the tunnel system is dependent on maintaining a minimum flow rate that will keep particles sized less than 0.075 mm in suspension. Based on the Hjulstrom Curve for Sediment Transport, which compares grain size and flow velocity to determine when a particle will theoretically undergo deposition, transportation (or suspension), and erosion (or re-suspension), particles sized less than 0.075 mm will remain in suspension over a fairly wide velocity ranging between 0.45 cm/sec and 22 cm/sec (0.015 ft/sec to 0.72 ft/sec); corresponding to a flow rate in a 40 foot diameter tunnel of just under 20 cfs up to nearly 900 cfs. If the tunnels are shut down for an extended duration allowing the particles to settling within the tunnel, then a velocity greater than 900 cfs is theoretically required to re-suspend the settled particles.

The long-term tunnel maintenance requirements due to potential sediment accumulation will be influenced by many factors, including actual sediment loads, sediment size distribution, performance and maintenance of the sedimentation basins, frequency of shutdowns, and actual operating rates and limiting flow velocities. It appears that some fine silt and clay sediments may accumulate in the tunnels based on anticipated operations (i.e. periodic shutdowns and maximum limiting operating flow rates and resulting velocities). Selection of planned operating conditions to maintain a minimum tunnel flow rate and periodic operation at high flow rates to scour and re-suspend sediments should minimize long-term maintenance concerns. Experience also indicates that sediment accumulation tends to build up over time at sharp vertical bends, such as the risers leading to the IF. The selected operation and maintenance program should be tailored to inspect these areas first as a predictor to a more extensive maintenance program covering of the full length of the tunnels.

Table 7-5 summarizes anticipated sediment diversion to each intake based on historic data.
Table 7-5: Sediment Loading to the Project

<table>
<thead>
<tr>
<th>Annual Sediment Diversion to Each Intake</th>
<th>Tons per Year</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Annual Sand-Sized (0.075 to 1.75 mm) Sediment Diversion Per Intake</strong></td>
<td></td>
</tr>
<tr>
<td>Peak Sediment Load Year (1978)</td>
<td>37,400&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>Average Year (1974 to 1991)</td>
<td>12,140&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td><strong>Total Annual Sediment Diverted to Each Intake</strong></td>
<td></td>
</tr>
<tr>
<td>Peak Sediment Load Year (1978)</td>
<td>121,507&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>Average Year (1974 to 1991)</td>
<td>39,493&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup> Source: DWR, undated
<sup>b</sup> Source: DWR, undated, plus FRWA particle size distribution for less than sand-sized material (see Table 7-6)

Sediment volumes were derived from modeling data using an 18-year period of available river flows. The design condition was selected as the peak sediment load year of 1978. Each intake site is estimated to divert a total of approximately 37,400 tons per year of sediment in the 1.75-mm to 0.075-mm size range. Based on the sediment distribution data of the recently constructed FRWA (Table 7-6), the particle size distribution for other than sand-sized material was interpolated.

Table 7-6: Sediment Sieve Size Distribution – Freeport Regional Water Project

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>0.250-0.500</th>
<th>0.125-0.250</th>
<th>0.062-0.125</th>
<th>0.031-0.062</th>
<th>0.016-0.031</th>
<th>0.008-0.016</th>
<th>0.004-0.008</th>
<th>0.002-0.004</th>
<th>&lt;0.002</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distribution (%) by weight</td>
<td>3</td>
<td>19</td>
<td>11</td>
<td>5.5</td>
<td>9</td>
<td>8.5</td>
<td>9</td>
<td>8</td>
<td>27</td>
</tr>
</tbody>
</table>

Note:
<=> less than

The sediment accumulations per year in both the sedimentation basins and IF are shown in Table 7-7 (calculated by Stoke’s Law from the sediment diversions listed above in Table 7-5). Because of the relatively small volumes of sediment captured in IF, average sediment diversions (rather than maximum annual) were used to evaluate years between dredging operations for that facility.

Table 7-7: Depth of Sediment Accumulation

<table>
<thead>
<tr>
<th>Sediment Accumulation Per Facility</th>
<th>Sediment Volume</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake Sedimentation Basins</td>
<td>1.5 feet per year (based on peak load year)</td>
</tr>
<tr>
<td></td>
<td>0.70 feet per year (based on average load year)</td>
</tr>
<tr>
<td>Intermediate Forebay (based on average year sediment diversion)</td>
<td></td>
</tr>
<tr>
<td>1 Year</td>
<td>0.08 feet</td>
</tr>
<tr>
<td>10 Years</td>
<td>0.8 feet</td>
</tr>
<tr>
<td>20 Years</td>
<td>1.6 feet</td>
</tr>
<tr>
<td>30 Years</td>
<td>2.4 feet</td>
</tr>
<tr>
<td>40 Years</td>
<td>3.2 feet</td>
</tr>
<tr>
<td>50 Years</td>
<td>4.1 feet</td>
</tr>
</tbody>
</table>
7.3.3 Solids Handling

At each intake site, a diesel engine powered barge-mounted suction dredge (approximately 375 horsepower) hydraulically dredges the sedimentation basins through a dedicated dredge discharge pipeline to drying lagoons. Annual dredging of the sedimentation basins is needed to maintain the basin efficiency and reduce the amount of settleable solids from being transported downstream. Less than 0.70 feet of sediment is expected to accumulate in an average year (assuming a uniform distribution of settled material occurs across the floor of the sedimentation basin), with maximum annual depth estimated at about 1.5 feet per sedimentation basin. Because the settleable material will not be deposited uniformly, and to account for the lower basin velocities at the intake channels due to basin geometry, the sediment storage depth is increased to 5 feet. This increased depth allows for a greater accumulation of sediment in the upstream portion of the basin and also provides a mixing zone between the settled material and the cross-sectional area of flow moving through the basin.

The dredged slurry is expected to consist of approximately 10 to 15 percent solids. Given the large difference in elevation between the sedimentation basin water surface and the drying lagoons and the length of connecting pipeline, production rates for commercially available 10-inch to 12-inch dredges are estimated to range from 75 to 125 cy per hour in course sands (1.0 mm) and 125 to 175 cy per hour in finer sands (0.1 mm). For conceptual engineering, a production rate of 125 cy per hour was assumed to size the drying lagoon. The dredge can either be stored at a dedicated facility within one of the sedimentation basins or dry stored at a dedicated location. A hoist mechanism would be installed to transfer the dredge between the twin sedimentation basins at each intake site.

7.3.4 Sediment Drying Lagoons

The dredged sediment needs to be dried before offsite disposal. At each intake site there are four sediment storage and drying lagoons.

Each drying lagoon has underdrains and a manually operated decant system. Flow from the underdrains and decant system are collected in an outlet structure and piped to the sedimentation basins. For ease of construction and maintenance, and to prevent seepage, the drying lagoon floor will be constructed with roller-compacted concrete, and the side slopes will be lined with reinforced shotcrete or roller-compacted concrete.

Using the maximum annual sediment loading estimate of about 34,400 cy of sediment per year for each intake facility, drying lagoon sizing calculations were based on the maximum sediment volume captured at each intake site at a maximum flow of 3,000 cfs. Based on the dredge production rates, it takes approximately 88 hours of continuous dredging to remove the worst-case annual sediment from one intake site and approximately 30 hours for an average year.

The drying lagoon size for maximum case sediment quantity is 160 feet wide bottom, 350 feet long, 15 feet deep, with 1:1 side slopes. The tops of the lagoons are located at a lower level with respect to the main intake at each site. The lagoons are still protected from the design flood condition. Safety provisions for emergency egress (ladders, hand-holds, or other devices) will be determined during the next phase of project development.

Two drying lagoons will be available per one sedimentation basin. A yearly rotation cycle of lead/lag operation is used, with one drying lagoon filling and one settling and dewatered by the underdrains and decant system. Each sedimentation basin can be dredged in one rotation. Approximately one-half of the worst-case total sediment can be stored in each drying lagoon each year per basin. Dried sediment to be removed from each drying lagoon is approximately 3.5 feet per year for a maximum sediment loading scenario and 1.1 feet during an average year. Access ramps into each drying lagoon are provided for a front-end loader to load accumulated solids into a truck for disposal.

7.3.5 Sedimentation Basin Outlet Structure

The vertical shafts that will be used for tunnel excavations at each of the intakes will be converted to outlet shafts once the tunnel is completed (as shown on Volume 2: Conceptual Drawings). The outlet shaft is centrally located between the two sedimentation basins at each of the intakes. Each outlet shaft consist of two sets of drop gates, each set will consist of four drop gates dedicated to each basin. Each set of drop gates will receive flow from each
sedimentation basin down to the tunnels. The outlet shaft elevation is set above the 200-year flood level with
SLR. The outlet gates will normally be open except, when the basin is being dredged or during the 200-year flood
to avoid large sediments collecting in the tunnels.

7.4 Electrical

The Intake and Sedimentation Facilities (Intakes No.2, No.3, and No.5) and the Junction Structure located at
Intake No.3 shall be fed from the Utility via two 480V, 3-phase incoming service feeders. At each intake, incoming
electrical service feeders shall be routed into the electrical building and feed the arc-resistant, main-tie-main-tie-
main configured switchgear, with a standby emergency generator as the backup. The switchgear will then
distribute power to all the associated loads. The switchgear will be located within the electrical building’s
electrical room.

In order to provide redundancy in the electrical system for the control inlet gates, a standby emergency generator
shall be connected to provide emergency power in the event of a loss of both utility services.

Working clearances will be provided per the National Electrical Code within the electrical building to allow for
front access to the switchgear. Cooling systems for the electrical building will maintain the room temperature as
required so that no de-rating of the electrical equipment is necessary during maximum outdoor ambient
temperature conditions.

At each intake site, a control room will be located within the site’s electrical building. This control room will house
equipment and instruments to monitor and control Intakes No.2, No.3, No.5, and the Junction Structure located
at Intake No.3 and communicating with the SCADA system.

7.5 Construction Considerations

Construction of intake facilities and sedimentation basins requires means, methods, and approaches unique to
marine and heavy civil construction. See Section 24 for detailed construction methodology.

The preliminary concept includes the intakes located at the river side toe of the existing levee on the Sacramento
River. The sedimentation basin is planned to be approximately 450 feet on the land-side of the existing levee
centerline.

The geologic condition for the area is primarily governed by alluvial floodplain deposits of Holocene sediments
and occasional sandy and silty soils of Riverbank Formation. The deposits are underlain by medium stiff silty and
clayey soils and dense to very dense silty sand of older alluvium of the Riverbank Formation.

The subsurface conditions adjacent to the river banks along this stretch of the Sacramento River may be generally
characterized by a surficial layer of soft to medium stiff, fine-grained soils to a depth of approximately 20 to 30
feet bgs and underlain by stratified stiff clay, clayey silt, and dense silty sand to the depth of soil borings (DWR,
1958). Based on construction experiences at the FRWA Intake, the levees are assumed to consist of loose to
medium dense sands with varying percentages of silt and clay fines.

7.5.1 Intake No. 2

Intake No. 2 is located within the Sacramento River at the river-side toe of the existing east bank levee at river
mile 41.1. Six borings were drilled at Intake No. 2: DCR2-DH-004, DCR2-DH-005, DCR2-DH-006, DCR2-DH-007,
DCR2-DH-008, and DCR2-DH-009. All six borings were in the Sacramento River channel. The in-river subsoil
conditions adjacent to Intake No. 2 are generally a 13-to 21-foot-deep layer of very loose to loose sand at the river
bottom, followed by a 3- to 9-foot-thick layer of very soft to very stiff silt. The silt layer was underlain by 29 to 32
feet of loose to medium dense granular material followed by 10 to 17 feet of dense to very dense granular
material. Below the granular stratum, the borings encountered a 40- to 53-foot-thick layer of hard mixed fines.
The five northern borings encountered a dense to very dense sandy layer below the fines. Borings DCR2-DH-004,
DCR2-DH-006, and DCR2-DH-007 terminated in this sandy layer at elevations ranging from 133.9 to 136.2 feet.
Borings DCR2-DH-005 and DCR2-DH-009 both terminated in fine-grained soil below the 15- to 23-foot-thick dense
to very dense sandy layer at approximate elevations of -152.9 and -154.3 feet. Boring DCR2-DH-008 did not encounter the dense to very dense sandy layer and terminated in a fines layer at an approximate elevation of -134.9 feet.

7.5.2 Intake No. 3

Intake No. 3 is located within the Sacramento River at the river-side toe of the existing east bank levee at river mile 39.4. Three borings were drilled at Intake No. 3: DCR3-DH-013, DCR3-DH-014, and DCR3-DH-015. All three borings were in the Sacramento River channel. The in-river subsoil conditions adjacent to Intake No. 3 are generally a 40-foot-deep layer of loose to medium dense sand, silty sand, gravel, and silt at the river bottom, followed by a 15-to 20-foot-thick layer of medium dense to dense sand and very stiff fines. This layer was underlain by 15 to 20 feet of dense to very dense sand. Below the dense to very dense sand, the borings encountered a 5 to 8-foot-thick layer of very stiff to hard fines underlain by a 3- to 5-foot-thick layer of very dense sand. A 4.5-foot-thick tephra layer was encountered beneath this very dense layer in bore hole DCR3-DH-013. The borings encountered a mixture of hard fines with some very dense sand layers to the depth of the borings. The borings terminated at approximate elevations ranging from -169.2 to -174.6 feet.

7.5.3 Intake No. 5

Intake No. 5 is located within the Sacramento River at the river-side toe of the existing east bank levee at river mile 36.8. Three borings, drilled in the Sacramento River Channel at Intake No. 5 (DCR-DH-004, DCR5-DH-013, and DCR5-DH-014), and one historic boring, PCA 2-1, on land approximately 650 feet inland from the Sacramento River at the intake site, were considered for this site. The in-river subsoil conditions are generally a 10- to 21-foot-deep layer of medium dense sand at the river bottom. The medium dense sand was underlain by a 25- to 35-foot-thick layer of soft to firm fines, followed by 10 feet of hard fines. The fines strata were underlain by 75 feet of very dense sand in which DCR-DH-004 was terminated at approximate elevation -160.3 feet. Boring PCA 2-1 was drilled to a depth of 49 feet. It was predominantly very soft to stiff sandy fines with a few thin layers of silty sand.

7.5.4 Potential for Liquefaction

Preliminary liquefaction analyses were performed for the conditions found in the Sacramento River borings adjacent to the three intake sites. These results need to be confirmed by additional site-specific subsurface exploration and additional geotechnical analyses.

Liquefaction causes loss of soil strength within the liquefied zone. This affects not only surface facilities such as pad fill side slopes, but also deep foundations that have significant lateral demands. As such, all identified zones of liquefaction beneath laterally loaded piles and settlement sensitive facilities must be improved. This ground improvement is assumed to be jet grouting within the footprint of the intake, pipe manifold, gravity collector pipes, maintenance facilities, sediment drying lagoons, and pump discharge piping. Other improvement techniques will be evaluated in future work.

For Intake No. 2, the analysis was based on the conditions encountered in boring DCR2-DH-008 and the design seismic event of \( M_w = 6.75 \) and \( \text{PGA} = 0.23 \) g. This analysis indicates liquefaction should be expected in the upper three layers of very loose to loose sand, very soft to very stiff silt, and loose to medium dense sand, to a depth of 54 feet below the river bottom (elevation -68 feet). The resulting settlement due to liquefaction is estimated to be 24 inches.

The liquefaction analysis at Intake No. 3 was based on the conditions encountered in boring DCR3-DH-013 and a design event of \( M_w = 6.75 \) and \( \text{PGA} = 0.23 \) g. This analysis indicates liquefaction should be expected in the upper layers of loose sands and firm fines, to a depth of 34.5 feet below river bottom (elevation -64 feet). The resulting settlement due to liquefaction is estimated to be 17 inches.

The liquefaction analysis at Intake No. 5 was based on the conditions encountered in boring DCR-DH-004 and a design event of \( M_w = 6.75 \) and \( \text{PGA} = 0.24 \) g. This analysis indicates liquefaction should be expected in the upper loose to medium dense sand layer, and the silt and sandy portions of the underlying soft to firm fines, to a depth of 41 feet below the river bottom (elevation -59 feet). The resulting settlement due to liquefaction is estimated to be 24 inches.
SECTION 7.0 INTAKES AND SEDIMENTATION FACILITIES

7.5.6 Slurry Cutoff Wall
A deep slurry cutoff wall will be installed to enhance future public protection from levee underseepage in accordance with USACE requirements and to reduce the groundwater inflow into deep excavations within the intake facility site pad. The cutoff wall will extend upstream and downstream of the proposed diaphragm walls within the center of the existing levee. In this way, if future levee improvement regulations are issued by USACE, disruption of water diversion activities can be avoided.

The slurry cutoff wall will extend around the perimeter of the Intake Facility. This perimeter cutoff wall will tie into short sections of diaphragm wall within the widened levee crest and will increase public flood protection during construction, especially if the sediment basin floods as a result of an unanticipated levee breach.

7.5.7 Typical Intake Foundation
The depth of the intake structure and typical soil conditions at Intake No. 3 are considered the controlling case for static design (liquefaction issues at Intakes No. 2 and 5 will control seismic design). The foundation system for Intake No. 3 is also used for Intakes No. 2 and 5.

Intake placement at the toe of the river side of the levee imparts significant lateral loads to the structure. At Intake No. 3, the soil height is approximately 45 feet during excavation and placement of the tremie slab within the intake cofferdam. Lateral earth pressures were estimated using braced earth pressure loading. A relatively rigid, braced, dual cofferdam system is anticipated for construction of the intake cofferdam and the intake header cofferdam (both located on the river-side of the levee crest). The land-side of the cofferdam is anticipated to be a thick reinforced concrete diaphragm wall with pipe penetrations incorporated into the reinforcement. The deeper excavation and the braced earth pressure condition controlled the structural foundation design, as the seismic condition benefited from the thick tremie slab and structural concrete intake floor.

Construction of the intakes will require a steel sheet pile cofferdam with internal bracing to enclose the planned area of the intake structure. The steel sheet piles will be designed to key into an underlying impervious layer, when present, for seepage cutoff. The enclosed area will be excavated to the level of the design subgrade using clam shell or long-reach backhoes before ground improvement and installation of foundation piers. The foundation construction will either be in-the-wet construction or conventional construction using traditional dewatering methods.

A fully-cased reinforced concrete drilled pier foundation resists the lateral loads. Drilled pier conceptual design is in general accordance with Federal Highway Administration (FHWA) design procedures. Lateral design was the controlling factor. Lateral foundation deficiencies are limited to less than 1 inch.

In-the-wet foundation construction would require the foundation to be drilled using a barge-mounted drilling rig positioned outside of the cofferdam (or a deck-mounted drilling rig per the permit requirement). Pier casings would be advanced during drilling for the full depth of the holes. Tremie concrete would be placed on the entire enclosed area within the cofferdam after ground improvement and the foundation drilling, reinforcement placement, and pier concrete placement are completed. The thickness of the tremie concrete would be commensurate with the design uplift pressure and the uplift capacity of the drilled piers. A 5-foot-thick slab has been used in the conceptual design. Once the tremie slab has cured sufficiently, unwatering of the cofferdam can then proceed to allow other construction activities to be carried out in the dry. Temporary uplift forces acting on the tremie slab will be resisted by the drilled pier foundation.

7.6 Maintenance Considerations
Maintenance is an integral part of a functional and reliable project. The goal is to increase efficiency, reliability, and safety and to ensure project objectives are met.

The intake facilities will require routine or periodic adjustment and flow tuning to ensure operations are consistent with design intentions. Facility maintenance, which is part of long-term asset management, includes activities such as painting, cleaning, repairs, and other routine tasks that ensure operation in accordance with design standards after construction and commissioning. Operation and maintenance consists of routine,
preventive, predictive, scheduled, and unscheduled maintenance to prevent equipment or facility deterioration or failure.

7.6.1 General Inspections
Routine visual inspections of the facilities are 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 downtime.

A deliberate monitoring program increases awareness of conditions that compromise operational performance and basic function. Inspections can be visual observations of facilities, underwater examinations, and dewatering for thorough analyses. Video and photographic inspections, together with thorough recording of observations, are necessary for operating and maintaining facilities. A dynamic inspection program is necessary for managing and extending the service life of infrastructure.

7.6.2 Sediment Removal
Sediment can bury intakes, reduce intake capability and force shutdowns for restoration of the intake. Engineering and design can inhibit these possibilities. Maintenance of the river intakes includes the following:

- Suction dredging around intake structure.
- Mechanical excavation around intake structures using track-mounted equipment and clamshell dragline after installing a floating turbidity control curtain.
- Annual hydraulic dredging of sedimentation basins using a barge mounted suction dredge.
- Dewatering of intake, conveyance piping, and sedimentation basin to remove sediment buildup.
- Annual removal of sediment in the sediment drying lagoons.

7.6.3 Debris Removal
Debris in the vicinity of the structure could compromise its function. After heavy-to-extreme hydrologic events, the structures should be visually inspected for debris. If large amount of debris has accumulated, the debris must be removed.

Intake screens, which remove debris from the surface of the water, are maintained by continuous traveling cleaning mechanisms, or other screen cleaning technology. Cleaning frequency depends on the debris load. Daily checks of intake screen cleaner functionality must be performed.

7.6.4 Dewatering Considerations
The intake plenum of a screen bay group can be dewatered by closing the slide gates on the back wall of the intake structure, installing bulkheads in guides at the front of the structure, and pumping out the water with a submersible pump. Entry provisions into the intake plenum are not shown in the Concept Drawings (Volume 2) at this stage of project development.

The intake collector box conduits can be dewatered by closing the gates on both sides of the flow control sluice gates and flowmeter and pumping out the water between the gates. Entry provision ventilation requirements for the intake conduits are shown in the Concept Drawings (Volume 2 near the control gates).

Provisions are not included to allow for dewatering the earthen sedimentation basins.

7.6.5 Biofouling
Biofouling, the accumulation of algae, freshwater sponge, Asian clams, mussels, and other biological organisms, can occlude the screens and jeopardize function. A key design provision for intake facilities is that all mechanical elements can be moved to the top surface for inspection, cleaning, and repairs. The intake facilities have top-side gantry crane systems for removal and insertion of screen panels, tuning baffle assemblies, and bulkheads.

All panels will require removal for pressure washing. Additionally, screen bay groups will require dewatering for inspection and assessment of biofoul growth rates.
With the invasion of Quagga and Zebra mussels into inland waters, screen and bay washing will increase. Coatings and other deterrents will be more thoroughly investigated during preliminary and final design.

Asian clams are a common biological fouling agent on the Sacramento River where sediment settles. Routine operation of the sediment control features is necessary. Asian clam infestation can be controlled at the sedimentation basins and drying lagoons through regular inspection and maintenance.

### 7.6.6 Corrosion

Although Sacramento River water is not considered corrosive other than in areas of scour, aerobic and galvanic corrosion at the Intake Facilities needs to be monitored because of the substantial amount of metalwork located at the facilities. Materials for the intake screens and baffles are expected to consist of plastics and austenitic stainless steels. Other systems are anticipated to be constructed of mild steel. Mild steel items can be provided with protective coatings to preserve the condition of those buried and submerged metals and thereby extend their service lives. Passive (galvanic) anode systems can also be used for submerged steel elements. Maintenance consists of repainting coated surfaces and replacing sacrificial (zinc) anodes at multi-year intervals.

Removing screen and tuning baffle elements for cleaning allows inspection of metalwork to assess corrosion rates. Metal items receiving coatings are prone to localized corrosion and will be subject to a routine inspection with forensic material testing and metallurgical analyses, similar to the American Water Works Association (AWWA) M42 and D100 standards for water storage reservoirs. Cast-in-place and precast concrete that is in contact with water can be made with Portland cement (Type I or Type II). Other properties of concrete, such as thickness of cover over reinforcing steel and mix design, should be in accordance with American Concrete Institute (ACI) 350.

The influence of SLR on intake site water quality must be considered. Chloride exposure could require more corrosion resistant materials.

### 7.6.7 Impact Repairs

A log boom system will be aligned within the river alongside the intake structure to protect the fish screens and fish screen cleaning systems from being damaged by large floating debris. Fish screens, solid panels, and the traveling screen cleaner mechanisms are most exposed to impact damage. The concrete structure housing these elements is not expected to suffer much impact damage. Spare parts for vulnerable portions of the intake structure should be available to minimize downtime should repairs be needed.

With the majority of working components being submerged, and with security provisions in place, vandalism damage is not expected to be significant.

### 7.6.8 Mechanical Equipment

Intake Facility systems involving power-driven and routinely moving parts are the screen cleaning systems, the slide gates at the individual intake screens, the sediment jetting pumps, the electric operators for the roller gates, and gantry crane hoist systems. Maintenance consists of lubricating bearings, continuity checking of limit/torque switches, and inspecting and replacing parts per manufacturer recommendations. Onsite vendor training and O&M procedures will be required.

### 7.6.9 Maintenance Equipment

Operation and maintenance equipment for the MPTO intake facilities include the following:

- A self-contained portable high pressure washer unit to clean fish screen and solid panels, concrete surfaces, and other surfaces.
- Submersible pumps for dewatering.
- A floating work platform for accessing, inspecting, and maintaining the river side of the facility.
- A hydraulic suction dredge to remove sediment captured in the sedimentation channels shared among facilities.
- A man basket or bridge inspection rig to safely access the front of the intake structure from the upper deck.
8.1 Site Location and General Description

The BTF Pumping Plant is located at the northeastern corner of the proposed the Byron Tract Forebay as seen in Figure 8-1. The BTF is sited at the northwest corner of the existing Clifton Court Forebay adjacent to the Italian Slough. This location was previously designated as a disposal site for re-usable tunnel material (RTM) and will serve as the terminus of the twin 40-foot tunnels and the location of the new combined pumping plant station.

![Diagram of Byron Tract Forebay Pumping Plant Location]

Figure 8-1: Byron Tract Forebay Pumping Plant Location

8.1.1 Proposed Site Layout

The combined pumping plant arrangement at this location is shown in the Concept Drawings (Volume 2). As shown in the drawings, the two Main Tunnels terminate at the twin pump plant shafts. The pump shafts provide multiple functions: 1) Provide for gravity flow when the system hydraulics allows via a spillway weir gates, 2) Provide surge protection via the spillway weir wall, 3) House the pumps and their controls. The gravity flow will bypass the pumps via three weir gates by allowing flow to discharge into the spillway and flow directly to BTF if hydraulic conditions permit (for reference, see Concept Drawings Volume 2). The pump shafts will house the pumps, pump wet wells, and pump intakes. The pumps discharge via siphon discharges into the basin within the BTF. The facilities will be designed to allow separate operation of the system components.

Final grade for the permanent pump station will be at EL. 30 ft., other facilities, including switchyard, electrical buildings, and other infrastructure, can be at a minimum EL. 25 ft. to provide protection from the 200-year flood level with sea level rise (EL. 16.5 ft.), wave run up (5 ft.), and additional freeboard (3.5 ft.). To accommodate surface drainage, the final surface will be sloped at a minimum of 1%. The combined pumping plant will encroach
past the existing levee road into the Forebay, requiring the redevelopment of the existing levee road. The site grade of El. 25 ft. will be established prior to construction of the shafts to provide flood protection during construction for the tunnels and pump stations.

Portions of the site outside the area of permanent facilities may be filled to a lower elevation (between El. 10 and 17 ft.) to provide for stable temporary construction facilities and equipment and material storage areas.

Ground improvement beneath all areas with permanent structures will include installation of vertical wick or sand drains to accelerate settlement and mitigate future liquefaction potential. It is anticipated that the majority of consolidation settlement due to the fill weight will occur as the fill is placed and within a period of several months.

8.1.2 Shaft Design and Construction

The pumping plant shafts are assumed to be constructed using slurry diaphragm walls as shown in the Concept Drawings (Volume 2) due to the large diameter and depth. Considerations for design of the diaphragm walls include:

- Slurry walls are assumed to be 6 ft. thick and to utilize high strength concrete to achieve stability for the full dewatered condition. Slurry wall length is conservatively estimated to extend well below the base slab elevation for base stability and uplift resistance.
- If possible, once the ground conditions are known, base stability in the dewatered condition may be achieved through the use of jet grouting or deep soil mixing to create an impervious layer and/or dewatering (de-pressure) of sand layers below the base. This will minimize or eliminate requirements for ‘top-down’ interior wall construction and tremie construction of the base slab.
- A final shaft concrete liner wall will be constructed inside the slurry wall. This finished interior wall thickness of 4 to 5 ft. is likely to be required for long-term stability and water tightness. A thicker portion of the interior wall will be required for structural support surrounding tunnel break-in/break-out openings.

It is important to note that the deep shafts will extend well below the depth of consolidation settlement and will not settle with the surrounding fill. Differential settlement between the shafts (and everything supported on them) and shallow foundation structures will be equal to the total fill settlement following shallow foundation construction. Although the amount of post-construction fill settlement is anticipated to be small, the final design is likely to require some ground improvement beneath the adjacent structures (surge overflow channel, siphon discharges, and spillway basins) or provisions for accommodating differential movement.

8.1.3 Pumping Plant Shaft Connections to Main Tunnels

As shown in the Concept Drawings (Volume 2), the current concept assumes that the pump station shafts will be used as the initial launch shafts for the tunnel. The pumping plant will have 164 ft. internal diameter shafts which are necessary for hydraulic operations. This size is more than the 113 ft. ID required for a typical TBM launch shaft and mining operations at other tunnel drive and reception shafts along the main 40 ft. tunnel alignment (see Section 11, “Tunnels”).

8.1.4 Potential for Gravity Flow

Each main tunnel (ID 40 ft.) terminates in a pump shaft at the northeastern corner of the Byron Tract Forebay. The gravity flow configuration is shown in the Concept Drawings (Volume 2). When hydraulic conditions allow, the system can flow by gravity from the Sacramento River intakes into BTF.

During periods when the Sacramento River stage is higher than BTF, it is possible to achieve gravity flow through the system. The Concept Drawings show four weir gates on the north side of each pump shaft. The weir gates can be lowered to release flow into an outlet channel that discharges into BTF. The weir gates would be used control flow, based on the head differential from the weir crest to the water surface elevation in the pump shaft. The feasibility of gravity flow through the system is dependent upon the final design operating levels of BTF and will be evaluated further during the preliminary and final design.

Figure 8-2 shows the achievable gravity flow capacities estimate through the tunnels at static head conditions developed between the Sacramento River water surface elevation at the intakes and the water surface elevation.
The normal operating water surface elevation at BTF is from 2.5 feet to 12.5 feet and an initial fill elevation of 0 feet. The Sacramento River water surface elevation was varied between 2.5 feet to 20 feet with low, normal and normal-high elevations depicted as reference. Under the above varying boundary conditions, the achievable gravity flow capacities meet the approach velocity of 0.2 feet per second or less at each of the three intakes.

8.1.5 Pump Plant Overview

The two pumping plants receive flow from the pump shafts and lift the water into BTF. Each pumping plant will have a design pumping capacity of 4,500 cfs, providing a total pumping capacity of 9,000 cfs.

Each main pumping plant will include an influent wet-well, main raw water duty pumps, pump motors, installed spare pump/motor, pump discharge siphons, surge protection weirs, intake isolation gates, a bridge crane, and associated appurtenances. An adjacent electrical building will house the main switchgear, motor starters, high and low voltage motor control centers (MCC), variable speed drives (VFD), and communication and control cabinets. The layout of the pumping plants and the electrical buildings are shown in the Concept Drawings (Volume 2). The following sections describe the Pumping Plant features.

8.1.5.1 Pumping Hydraulics

The system was evaluated to determine the expected range of hydraulic conditions. Table 8-1 presents design water levels that were used for development of the system curve envelope presented in Figure 8-3.
Table 8-1 Design Water Levels for the Range of Operating Conditions

<table>
<thead>
<tr>
<th>Pump Head Design Condition</th>
<th>Sacramento River WSE @ Intake #2</th>
<th>Byron Tract Forebay WSE</th>
<th>Static Head on Pumping System</th>
<th>Total Dynamic Head @ 9,000 CFS</th>
</tr>
</thead>
<tbody>
<tr>
<td>High Head</td>
<td>1.9</td>
<td>14.7</td>
<td>16.1</td>
<td>33.1</td>
</tr>
<tr>
<td>Design Condition</td>
<td>5.0</td>
<td>12.5</td>
<td>13.0</td>
<td>30.0</td>
</tr>
<tr>
<td>Normal Low Head</td>
<td>10.0</td>
<td>2.5</td>
<td>-5.0</td>
<td>12.0</td>
</tr>
<tr>
<td>Extreme Low Head</td>
<td>31.4</td>
<td>1.0</td>
<td>-26.4</td>
<td>-9.4</td>
</tr>
</tbody>
</table>

Note: WSE indicates Water Surface Elevation – NAVD88
All values are in feet.

Figure 8-3: System Curves for the Range of Operating Conditions

Based on the hydraulic information from Section 6 of this CER, the expected total dynamic head during operation at 9,000 cfs is 30 feet in the Design Head condition and 12 feet at the Normal Low Head condition. The High Head condition would result in a total dynamic head of 33 feet at 9,000 cfs, and the system will flow by gravity in excess of 9,000 cfs in the Extreme Low Head condition. Because the High Head and Extreme Low Head conditions are driven by unusually low and high water levels in the Sacramento River, the typical operating envelope is between the Design Head and Normal Low Head system curves. The Design Head and Normal Low Head operating conditions are significantly affected by the operating water levels of BTF, which may range from Elevation 14 feet at the Design Head to 1 foot at the Normal Low Head condition. The range of operating water levels in BTF will influence equipment selection and should be finalized prior to commencement of preliminary design.

8.1.5.2 Wet-Well and Pump Intake

The invert of the Pumping Plant wet-well is established through evaluation of the upstream tunnel system hydraulics using the following design parameters:

- Sacramento River WSE = 5.0 feet
- System Maximum Pumping Rate = 9,000 cfs
- Design Capacity of the Largest Installed Pump = 1,125 cfs
- Dynamic losses in tunnel system = 17 feet

Based on pump capacity and general size considerations it is expected for the suction bell of each large pump to be approximately 16 feet in diameter. The preliminary depth and size considerations for the wet-well assumes a 2.5 times pump bell diameter for the submergence requirement and 0.5 times the pump bell diameter for the bell to floor clearance to prevent the formation of surface vortices. Using these design parameters, with a normal river level of 5 feet, the invert of the wet-well has been set at Elevation -64 feet. NPSH is the sum of 40 feet of submergence plus atmospheric pressure (less the vapor pressure of water), resulting in approximately 70+ feet, which should be ample NPSH based on the available pump selections and operating ranges.

As shown in the Concept Drawings (Volume 2), the influent conduit enters the wet-well from the bottom at the center of the pumping plant shaft, rises vertically, and distributes radially to six vertical column discharge pumps around the perimeter of the shaft. The wet-well shaft will also be equipped with an overflow weir discharging to BTF to relieve potential hydraulic transient surges in the system. The elevation and size of the surge relief weir will be determined during preliminary design by a transient analysis simulating a power failure when pumping at the design capacity of 9,000 cfs.

Each pump bay will be equipped with a portable dewatering sump pump, including slide-rails for pump installation, an access hatch, and discharge piping routed to the BTF. In addition to dewatering a pump wet-well for maintenance, the sump pumps can also be used for dewatering the upper portion of the tunnel shafts when tunnel dewatering is necessary.

During the preliminary design phase of the project, the wet-well geometry should be evaluated using computer modeling and physical hydraulic model study in accordance with the Hydraulic Institute Pump Intake Design Standard (ANSI/HI 9.8-2012). In addition to a wet-well design, a formed suction inlet with dry-well alternative was evaluated during conceptual design, but the dry-well alternative was not as ideal as the wet-well due to additional losses from the formed inlet. If hydraulic issues arise during modeling of the wet-well configuration, the formed suction inlet with the pumps in a dry-well can be re-evaluated.

### 8.1.5.3 Discharge Configuration

Preliminary discharge piping size for the large pumps is 12 feet in diameter and the discharge piping for the small pumps is 8.5 feet in diameter. Each pump discharge will transition to a discharge siphon outside of the pumping plant shaft and will incorporate an ultrasonic flow meter. The siphon discharge option was selected due to savings from elimination of the pump isolation valve as well as minimizing power by reducing the operating total dynamic head through siphon recovery. The other options evaluated were a discharge valve with a submerged discharge and a free discharge above the water surface without a discharge valve.

The invert elevation at the crest of the siphon is designed to isolate the pumping plant from BTF when the pumps are not operating and prevent the pumps from freely rotating on high wet-well elevations or surges. The conceptual design is based on a maximum water level in BTF of 14 feet and provides 2 feet of freeboard, resulting in a siphon crest invert elevation of 16.0 feet. The invert should be adjusted, as necessary, to provide appropriate protection from backflow through the siphon, based on the final design operating elevations of BTF as well as maximum allowable wet-well elevations and surge.

The siphon discharge drops to an elevation of -1.0 foot, which is 2 feet below the minimum design water level in BTF. The siphon outlet should be submerged at all times to maintain siphon prime; the recommended minimum siphon submergence for this is equal to one velocity head, or approximately 1.3 feet. The siphon discharges to a concrete apron designed for energy recovery and prevention of erosion in BTF.

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2 The dynamic losses for establishing the bottom of the wet-well were calculated to be 17 feet at 9,000 cfs, see Section 6 of this CER for detailed System Hydraulic information.
For a short duration at start-up, the siphons will cause the pumps to experience a higher than normal operating head until the siphon is primed. A siphon-breaker valve at the top of the siphon will remain open to support evacuation of air during pump startup and will close after a pre-set and adjustable time to engage the siphoning (approximately 30 to 60 seconds). Guidance provided for siphon design in the US Army Corps of Engineers Pump Station Design Manual (EM 1110-2-3105) indicates that the priming velocity for siphons should exceed 7 feet per second (fps) to create and maintain siphon prime. Physical hydraulic modeling of the siphon discharge should take place during preliminary design to confirm the siphon geometry and to determine if a vacuum breaker valve is required.

At pump shutdown, the siphon breaker valve will be opened to allow air into the discharge siphon to break the siphoning, which will prevent reverse flow from BTF through the pumps and into the tunnel system. The siphon breaker valves are to be actuated butterfly valves with a backup power supply (battery, hydraulic or pneumatic) such that they can be operated in the event of a power failure. There will also be a redundant, manual actuation provision or an additional manually actuated siphon breaker valve.

The velocity in each pump discharge siphon will be approximately 10 fps at the design flow. If a variable frequency drive is utilized for pump control, the minimum pumping flow could be significantly less than rated and/or less than the minimum 7 fps required to maintain prime. This resulting lower velocity requires evaluation during preliminary design to ensure the siphon will remain primed throughout the normal pump operating range.

Flow measurement on each pump will be accomplished using an acoustic flow meter on the straight section of pipe between the pump discharge and the siphon. The acoustic flow meters will be flow tested, calibrated and certified by lab testing, and used during commissioning of the pumping plants as well as during operation for equipment trending.

Alternate discharge configurations were considered, such as discharging above the high water level with no siphon and discharging below the normal water level with a discharge isolation valve. During preliminary design, a cost-benefit analysis and design evaluation should be performed before finalizing the selection of the discharge siphon.

8.1.5.4 Pumps

Pump types considered for the BTF Pumping Plant application included end-suction volute pumps and vertical column discharge pumps. Although both pump designs can perform well for this application, the vertical column discharge pumps were selected as the appropriate design due to the smaller station footprint required. As shown in the Concept Drawings (Volume 2), the pumps are suspended from a fabricated support base under each motor on the main pump room floor. The floor slab structure supports both the motors and the entire pump and column assemblies. Due to the length of the pump column, intermediate supports may be necessary such that the natural frequencies of the pump, supports, and connecting structures do not conflict with the operating frequency of the pumps. A Finite Element Analysis of the system should be conducted during final design to determine the configuration of intermediate supports and mitigate the potential for natural frequency vibrations.

To provide the firm design capacity of 9,000 cfs, a total of 12 pumps will be provided in the two Pumping Plants. Eight of the pumps will have a design capacity of 1,125 cfs and four will have a design capacity of 563 cfs. Each pump will be a single-stage unit and will have a pull-out type design, facilitating removal of the rotating assembly without disconnecting the discharge piping or removing the pump column. The pump discharge is below the motor level and is connected directly into the discharge siphon. The facility overhead crane will have sufficient clearance to disassemble and remove the pump in 20 foot sections, or the entire pull-out pump column can be removed through a roof hatch using a large mobile crane.

A number of pump manufacturers were consulted during the conceptual design evaluation, including Andritz, Fairbanks-Nijhuis, Flowserve, Flygt-Xylem, and Patterson Pump. These manufacturers all have pull-out style vertical column discharge pump selections that can meet the project design requirements. For purposes of this conceptual engineering report, pump selections were obtained from Flygt-Xylem to illustrate the performance requirements.
The pumping equipment will be selected to ensure that the normal conditions of service are within the pump’s Preferred Operating Range (POR). For pumps with a specific speed greater than 4500, the Hydraulic Institute defines the POR as the operating flow range between 80 and 120 percent of the pump’s best efficiency point (BEP) flow. The specific speed for the raw water pumps is expected to be in the range from 7,000 to 10,000, which is within the mixed- and axial-flow type impeller regions. For mixed- and axial-flow type pumping equipment, it is important that the operating point is not to the left of the POR during normal, extended operation. Pumps in this specific speed range tend to have unstable hydraulics to the left of the POR, resulting in unbalanced and variable loading on the pump impeller. The system should also be designed such that the pumps are not required to operate to the right of the POR at the full design speed.

The equipment shall be designed with a suitable net positive suction head (NPSH) required, allowing the pumps to safely operate through the full range of design conditions, without resulting in damaging cavitation. In accordance with the Hydraulic Institute Standards for NPSH Margin in Vertical and Centrifugal Pumps (ANSI HI 9.6.1-2012), the NPSH margin ratio (NPSH available / NPSH required) shall be greater than 1.1 when operating within the full range of design conditions. For extra margin of safety the NPSH margin ratio shall be increased to 1.2. Table 8-2 summarizes the basic preliminary design criteria for the pumps and motors.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Concept Design Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pump Type</td>
<td>Vertical Column Discharge</td>
</tr>
<tr>
<td>Total Number of Pumps (both Pumping Plants)</td>
<td>12, including 2-spare, (equivalent to one large pump per pump plant)</td>
</tr>
<tr>
<td>Number of Large Pumps</td>
<td>8</td>
</tr>
<tr>
<td>Number of Small Pumps</td>
<td>4</td>
</tr>
<tr>
<td>Total Design Flow</td>
<td>9,000 cfs</td>
</tr>
<tr>
<td>Design Condition Capacity - Large Pumps</td>
<td>1,125 cfs</td>
</tr>
<tr>
<td>Design Condition Capacity - Small Pumps</td>
<td>563 cfs</td>
</tr>
<tr>
<td>Design Condition Total Dynamic Head - Large and Small Pumps</td>
<td>37 feet</td>
</tr>
<tr>
<td>High Head Condition (Maximum Priming Head)</td>
<td>40 feet</td>
</tr>
<tr>
<td>Low Head Pumping Condition (reduced speed)</td>
<td>~5 feet</td>
</tr>
<tr>
<td>Motor Power - Large Pumps</td>
<td>6,000 HP</td>
</tr>
<tr>
<td>Motor Power - Small Pumps</td>
<td>3,000 HP</td>
</tr>
<tr>
<td>Conceptual Selection Maximum Rotation Speed – Large Pumps</td>
<td>160 rpm</td>
</tr>
<tr>
<td>Conceptual Selection Maximum Rotation Speed – Small Pumps</td>
<td>176 rpm</td>
</tr>
<tr>
<td>Motor Enclosure</td>
<td>TEWAC</td>
</tr>
</tbody>
</table>

Note: For constant speed pumping, some low-head operating conditions require the discharge head to be artificially increased to prevent the pumps from operating beyond the pump’s POR.

Figure 8-4 shows the system performance with 1 to 8 large constant speed pumps. For clarity, the figure does not represent the curves for the small pumps. The 90% efficiency line represents the pump’s Best Efficiency Point (BEP), which is the preferred operating point both for maximizing efficiency and minimizing equipment maintenance. The efficiency lines on either side of BEP define the Preferred Operating Range (POR). The 84% efficiency line represents 80% of BEP (left side of the pump’s POR) and the 74% efficiency line represents 120% of BEP (right side of the pump’s POR). The normal operating conditions should be at a point on the system curve between the 84% and 74% efficiency lines.
As is illustrated on the constant speed pump curves (Figure 8-4), the large pumps will provide flow increments of approximately 1,000 cfs with each additional pump that is put into service (note that the increments are larger at low flows and smaller at high flows due to the steepening of the system curve). Inclusion of the small pumps in the operating scheme reduces the operating flow increments by approximately half.

When operating with 5, 6, 7, or 8 pumps, the operating envelope between the Normal Low Head system curve and the Design System Curve is within the pump’s POR. However, with four or fewer pumps operating, an increasing amount of the lower head portion of the typical operating envelope falls outside the POR. If constant speed operation is determined to be the preferred method of controlling the pumps, it will be necessary to artificially increase the head on the pump discharge in some scenarios when operating fewer than five pumps. Opening the vacuum breaker valves on the siphon discharge would serve to increase the discharge head during low head operating conditions by preventing the siphons from priming.
Table 8-3: Range of Flows for Variable Speed Operation at the Design Head and Normal Low-Head Operating Conditions

<table>
<thead>
<tr>
<th>Number of Pumps Operating</th>
<th>Range of Flows for the Design Head Operating Condition (cfs)</th>
<th>Range of Flows for the Normal Low-Head Operating Condition (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8 Large (7 Large/2 Small; 6 Large/4 Small)</td>
<td>5,500 to 9,000</td>
<td>~8,000 * to 9,000 **</td>
</tr>
<tr>
<td>7 Large (6 Large/2 Small; 5 Large/4 Small)</td>
<td>5,200 to 8,300</td>
<td>~8,000 * to 9,000 **</td>
</tr>
<tr>
<td>6 Large (5 Large/2 Small; 4 Large/4 Small)</td>
<td>5,000 to 7,500</td>
<td>~8,000 * to 9,000</td>
</tr>
<tr>
<td>5 Large (4 Large/2 Small; 3 Large/4 Small)</td>
<td>4,300 to 6,500</td>
<td>~5,000 * to 7,500</td>
</tr>
<tr>
<td>4 Large (3 Large/2 Small; 2 Large/4 Small)</td>
<td>3,500 to 5,500</td>
<td>Gravity Flow ***</td>
</tr>
<tr>
<td>3 Large (2 Large/2 Small; 1 Large/4 Small)</td>
<td>2,800 to 4,600</td>
<td>Gravity Flow ***</td>
</tr>
<tr>
<td>2 Large (1 Large/2 Small; 4 Small)</td>
<td>1,900 to 3,000</td>
<td>Gravity Flow ***</td>
</tr>
<tr>
<td>1 Large (2 Small)</td>
<td>1,000 to 1,400</td>
<td>Gravity Flow ***</td>
</tr>
<tr>
<td>1 Small</td>
<td>500 to 800</td>
<td>Gravity Flow ***</td>
</tr>
</tbody>
</table>

Notes:
* Although the right side of the POR does not indicate that pumping below ~ 6,000 cfs is acceptable, pump manufacturers have indicated that when operating below 80% speed, the equipment can operate to the right of the POR to a TDH as low as 0 feet without adverse effects. For operation in this range, an NPSH margin ratio of 1.2 must be achieved to remain within Hydraulic Institute Standard requirements.
** To prevent exceeding 9,000 cfs, the pump speed must be limited when operating below the Design Head system curve.
*** Condition is conducive to gravity flow or operation with more pumps at low speed.

Variable pitch blade pumps were investigated for their ability to allow for wider operation ranges without a VFD. Evaluation determined the additional mechanical equipment and maintenance of the variable pitch blade system was not as attractive as constant speed or VFD’s. VFD’s are further discussed in the energy analysis section and should be further investigated during preliminary design as the system constraints become further developed.

8.1.5.5 Energy Analysis – Variable Speed and Constant Speed Pumping

Pump operation in a variable speed mode would facilitate delivery of a wider range of flows without artificially increasing the total dynamic head and remaining within the pump’s POR. Figure 8-5 through Figure 8-8 show pump and system curves utilizing eight, six, three, and one large 1,125 cfs pump and Figure 8-9 presents the range of operation for one small 563 cfs pump. As with the constant speed scenario, the large and small pumps can be operated simultaneously, with two small pumps roughly delivering the same flow as one large pump.

![Figure 8-5: Pump and System Curves for 8 Large Variable Speed Pumps Operating](image-url)
Figure 8-6: Pump and System Curves for 6 Large Variable Speed Pumps Operating

Figure 8-7: Pump and System Curves for 3 Large Variable Speed Pumps Operating
Figure 8-8: Pump and System Curves for 1 Large Variable Speed Pump Operating

Figure 8-9: Pump and System Curves for 1 Small Variable Speed Pump Operating
The pump selections are designed to deliver 9,000 cfs when operating on the Design Head system curve with eight large pumps operating (or similarly, 9,000 cfs with combinations of large and small pumps, such as 7 large/2 small or 6 large/4 small). When the total dynamic head conditions are less than the Design Head, the pumps would operate at reduced speed, or fewer pumps would be used to prevent exceeding 9,000 cfs. For the Normal Low Head condition, eight large pumps would operate at approximately 84% speed to deliver 9,000 cfs.

Variable speed operation provides the flexibility to adjust the number of pumps and the pump operating speed to optimize the hydraulic efficiency of the equipment. For example, when operating on the Normal Low Head system curve with six large pumps at a flow of 8,500 cfs (100% speed), the hydraulic efficiency is approximately 85%. This same flow can be conveyed at the highest hydraulic efficiency when operating eight large pumps at 80% speed. Additionally, by operating eight pumps instead of six, the total dynamic head is lower because the velocities through each pump and discharge siphon is lower, reducing the hydraulic losses. By operation of the system in this manner, the variable speed pumping can significantly reduce energy usage for the low-head operating conditions.

Table 8-4 indicates the approximate range of flows for the variable speed equipment when operating along the Design Head system curve and the Normal Low-Head system curve.

<table>
<thead>
<tr>
<th>Flow (cfs)</th>
<th>9 Feet of Static Head</th>
<th>5 Feet of Static Head</th>
<th>1 Foot of Static Head</th>
<th>-3 Feet of Static Head</th>
<th>-7 Feet of Static Head</th>
</tr>
</thead>
<tbody>
<tr>
<td>9,000</td>
<td>103%</td>
<td>98%</td>
<td>97%</td>
<td>90%</td>
<td>88%</td>
</tr>
<tr>
<td>6,000</td>
<td>91%</td>
<td>87%</td>
<td>65%</td>
<td>47%</td>
<td>26%</td>
</tr>
<tr>
<td>3,000</td>
<td>74%</td>
<td>55%</td>
<td>31%</td>
<td>21%</td>
<td>Gravity Flow</td>
</tr>
<tr>
<td>Weighted Average</td>
<td>96%</td>
<td>90%</td>
<td>81%</td>
<td>68%</td>
<td>69%</td>
</tr>
</tbody>
</table>

Note: Weighted Average is the average per volume of water pumped; i.e., weighted toward energy use at higher flow.

For the constant speed 3,000 cfs operation at -3 feet of static head, it is assumed that the static head will have to be artificially increased to keep the pump within the POR.

An evaluation of energy usage reveals that the operating levels in BTF have a significant influence on whether variable speed or constant speed operation is more cost-effective. Other major influences on energy consumption include typical pumping rates and the duration of pumping. Because details of these parameters will not be available until later design phases, the energy analysis is presented to provide a comparison of the amount of energy required for the various operating scenarios. The following assumptions were used in the analysis:

- Variable Frequency Drive has an efficiency of 97.5%.
- Hydraulic efficiency is based on the pump selection from Flygt-Xylem presented in the Pumping Hydraulics Section of this document.
- The number of variable speed pumps operating in a given scenario is that which provides the greatest hydraulic efficiency.
- The added maintenance of a VFD was not considered in the comparison.

The ranges of operating conditions in the analysis are within the envelope between the Design Head condition and the Normal Low Head condition. For these operating conditions, the hydraulic static head between the Sacramento River and BTF ranges from -9.0 feet to +9.0 feet. To evaluate a representative range of expected hydraulic conditions, operating scenarios with static heads of 9, 5, 1, -3, and -7 feet were considered. Table 8-4 above summarizes the percentage of energy used for variable speed pumping as compared to constant speed energy requirements for each of these operating conditions.

The energy analysis reveals that for lower static head conditions, the energy reduction benefits of variable speed pumping are increased. At the Design Head condition (9 feet of static head at 9,000 cfs), the constant speed operation uses less energy than variable speed due to losses associated with the variable frequency drives. For the range of flows considered at 9 feet of static head, the variable speed pumps use nearly as much energy (96%)
as the constant speed pumps. When the system static head is -3 feet, however, variable speed operation uses only 68% of the energy that constant speed operation would use, providing significant energy savings.

The primary driver for the range of static heads in the system is the WSE of BTF and the anticipated large range from +1 to +14. If the normal WSE of BTF was fixed at a smaller range and the pumps rerated for the smaller range, or if the normal WSE of BTF is finalized at an elevation greater than 10 feet, constant speed pumps may be a better choice when equipment maintenance is taken into consideration. However, if the typical BTF WSE is in the range of 1 to 8 feet, there is potential that variable speed pumping will provide significant energy savings.

A life cycle cost evaluation comparing constant and variable speed pumping should be conducted during preliminary design when the range of water levels in BTF has been finalized and pump selections have been made for the final system hydraulics.

### 8.2 Construction Methodology

#### 8.2.1 Constructability Considerations

Constructability considerations include, but are not limited to:

- Mobilization and demobilization;
- Contract administration;
- Development of staging and storage areas and construction zones;
- Earthwork, including deep excavation and shoring and bracing;
- Deep ground improvement and dewatering;
- Diaphragm wall and tremie slab construction;
- Trenching and pipeline installation;
- Foundation preparation, stabilization, and footing and slab-on-grade installation;
- Conventional CIP concrete construction involving formwork, reinforcing, placement, and finishing;
- Metalwork fabrication, assembly, installation, and structural framing;
- Electrical equipment (to include MCCs and VFDs for large motor loads);
- Mechanical equipment installation to include large pumps, truck-mounted cranes, large valves (with associated mechanical and electrical controls), flow meters, and miscellaneous civil site and electrical work.

#### 8.2.2 Site Access

Site access during construction via roadways will be provided for vehicle traffic from the east from Byron Highway via Camino Diablo. Access from the south will be provided from the Byron Highway via Western Farms Ranch. Routing of the various access roads may be required during various construction phases.

Site access via waterways can be provided from West Canal and Old River from the San Joaquin River. Waterway constraints will include restrictive channel depths and a 75 ft. horizontal clearance for the Orwood Bascule Railroad Bridge on the Old River.

Power supply to the site can be provided from the west, either by tie-in to the Brentwood Substation (PG&E) or the Tracy Substation (WAPA). A potable water line will also be required to supply the site from the closest public supply main.

#### 8.2.3 Temporary Facilities

Temporary facilities required for construction will be installed during or directly following the site development phase and will consist of:

- Site controls (fencing);
- Owner and construction contractor trailer and parking areas;
- Storage and treatment facilities for construction process water and storm water runoff;
- Switchyard (substation) for construction site energy supply;
- Barge unloading facility along the shoreline of the West Canal; and
- Construction staging areas.

### 8.2.4 Construction Contract Coordination

Some aspects of the construction site infrastructure are shown on the Surge Shaft/Pumping Plant Plan in the Concept Drawings (Volume 2). During construction of the pump stations, the construction site will be divided along a line north of the permanent office facilities and permanent substation to allow separation between the tunnel contractor in the northern part of the site and the pump station contractor in the southern part of the site.

Up to four or more construction contracts are likely to be utilized to complete the required work for the construction phases described in this section: site preparation, shaft construction, TBM tunneling, and pump station construction. The first three contracts will require access to the entire construction site, with the TBM tunneling contract relinquishing the southern portion of the site to the pump station contractor once TBM tunneling support infrastructure is relocated to the surge shaft area. Thereafter, the pump station contractor will have a sufficient, distinct work area. There will likely be provisions in the contracts to allow sharing of some of the site infrastructure, such as the main site access roads, temporary power substation, and process water and storm water runoff storage and treatment systems.
The WaterFix BTO conveyance system relies on a tunnel system to convey water southward from the Sacramento River Intake Facilities to the IF, and from the IF to Byron Tract Forebay for delivery to SWP and CVP export pumping facilities. See Section 11.0, “Tunnels,” for a description of the tunnel system. Pipeline and concrete box conduit construction has been largely avoided due to the saturated ground conditions that would make construction problematic. Additionally, significant surface disruptions that would occur with open cut pipeline or box conduit construction. Construction of tunnels avoids both of these potential issues.

The WaterFix BTO uses box conduits within the Intake Facilities. See Section 7.0, “Intakes and Sedimentation Facilities,” and Section 15.0, “Levees,” for descriptions of these pipelines in the Intake Facilities.

9.1 Description of Facilities

Each intake facility will have a maximum capacity of 3,000 cfs. Sacramento River water will be drawn into the intake facility through collector box conduits that feed water to the sedimentation basins where solids are removed. The flow from the sedimentation basins will discharge to a basin outlet structure that serves as the inlet for the North Tunnels.

9.1.1 Intake Locations and Pipeline Alignments

The concept for the conveyance pipelines is shown in the Concept Drawings (Volume 2). Final plan length would be determined at each intake location based on site-specific conditions.

9.1.2 Pipe/Concrete Box Material Type Alternatives

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

- Circular CIP concrete pipe.
- Concrete cylinder pressure pipe (AWWA C300).
- Steel pipe (AWWA C200).
- Rectangular CIP concrete box.
- Arch CIP concrete conduit.

Appendix E, “Pipe Materials,” summarizes the construction techniques; delivery and installation requirements; and other characteristics associated with each pipe material and configuration. For the purposes of this CER, design parameters and quantities for circular concrete CIP conduit are presented. Evaluation of other pipeline material alternatives is continuing.

9.1.3 Pipe/Concrete Box Number and Size Selection

Alternative configurations of number of conduits and conduit size were previously evaluated. The purpose of the analysis was to identify the optimum number of conduits and the optimum pipe diameter to be constructed for each conduit type. The analysis included evaluation of hydraulics, ROW, energy consumption, construction impacts, and economic criteria.
9.1.4 Pipe Hydraulics and Pressure Criteria
The conveyance system hydraulics is discussed in Section 6.0, “Conveyance System Hydraulics.”

9.1.5 Pipe/Concrete Box Cover Depth and Flotation
Pipe cover depth must take into consideration several factors, including (1) farming and agriculture needs; (2) high groundwater and flotation; (3) location of existing utilities; and (4) live loads. 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 flotation. Preliminary data indicate that groundwater is within 1 to 2 feet of existing ground elevation. Appendix F, “Pipeline Flotation,” presents the pipeline flotation analysis.

The recommended minimum pipeline depth of cover is 10 feet. Meeting this design criterion may raise flotation issues for several of the conduit types. The following flotation prevention options require further investigation to identify a preferred method:

- 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 farming.
- Provide concrete collars.
- Anchor conduit to piles.

9.1.6 Other Construction Components

Materials of Construction: Alternative conduit materials and construction configurations are evaluated in Appendix E. Designing for a range of materials and configurations 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 with impressed current cathodic protection.
- Field-fabricated reinforced concrete cylinder pressure pipe (RCCP) (AWWA C300) or RCP (AWWA C302).
- CIP concrete options, including circular CIP, rectangular box conduit, and arch shaped conduit.

Valve Structures: Where conveyance pipelines are connected to tunnels, isolation valves will be provided at the terminus of the pipelines to be able to isolate the conveyance pipelines from the tunnels. All isolation valves will be butterfly-type valves that will be housed in a valve structure, with access provided for repairs and inspection.

Pipe Embedment: Pipe embedment requirements will depend on the pipeline material configuration and geotechnical conditions. Additional geotechnical explorations will be undertaken to 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 will be used as embedment and backfill for the conduits and exported for use as fill elsewhere on the project. Pipeline embedment will be imported where suitable materials are not available. All embedment will be placed and compacted around the pipeline as required for pipeline support and to minimize surface settlement.

Roadway Crossing: Roadway crossings will be constructed by open-cut trench construction methods. Local access will be maintained by detours or temporary pavement. Compliance with local governing agency requirements will be performed for each roadway crossing.
**Drainage Crossings:** All pipeline sections installed within or across substantial drainage courses will be designed with additional protection including additional cover where necessary, concrete encasement, or riprap at open-cut installations. A scour analysis will 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.

**Pipeline Dewatering Facilities:** Pipeline dewatering facilities will be installed as part of construction to (1) provide a dry, stable excavation bottom for placement of bedding, pipe material, and backfill; (2) dewater lenses of silts and sands encountered during excavation; and (3) dewater highly permeable prolific sand layers below the excavation. In addition, due to the high groundwater table, dewatering facilities may also be implemented in 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 9-1 summarizes the characteristics of dewatering alternatives being considered at this time. 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, will be evaluated. Geotechnical and groundwater quality investigations will be required to assess whether treatment of groundwater prior to discharge is required.

**Table 9-1: Summary of Pipeline Dewatering Alternatives**

<table>
<thead>
<tr>
<th>Description</th>
<th>Well Point</th>
<th>Deep Well</th>
</tr>
</thead>
<tbody>
<tr>
<td>Application Methodology</td>
<td>Dewater silts and sand above the bottom of planned excavation</td>
<td>Depressurization of sand underneath the bottom of planned excavation</td>
</tr>
<tr>
<td>Well Screen Diameter 2 to 4 inches 6 to 8 inches</td>
<td>Well Screen Diameter 2 to 4 inches 6 to 8 inches</td>
<td>Well Screen Diameter 2 to 4 inches 6 to 8 inches</td>
</tr>
<tr>
<td>Well Depth 20 feet 75 to 300 feet</td>
<td>Well Depth 20 feet 75 to 300 feet</td>
<td>Well Depth 20 feet 75 to 300 feet</td>
</tr>
<tr>
<td>Well Spacing</td>
<td>3 to 6 feet at top of trench 3 to 6 feet at intermediate bench</td>
<td>50 to 75 feet</td>
</tr>
<tr>
<td>Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute</td>
<td>Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute</td>
<td>Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute</td>
</tr>
</tbody>
</table>

**Access Openings:** Access openings will be provided at primary transition structures, including the pumping plant, the conduit to tunnel transition structures, the valve structures, the gravity bypass structure, the gravity bypass transition structure, and the IF structures. Access openings will be configured to facilitate internal inspections and maintenance of the conveyance system.

**Corrosion Protection:** Corrosion control will be evaluated, based on the following:

- Pipeline construction materials.
- 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 from it.
- 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 will be further assessed during subsequent engineering analysis.

9.2 Construction Methodology

Construction of the intake pipeline conveyance system will utilize typical open trench excavations for the majority of the alignment.

9.2.1 Trench Width

Trench widths will 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 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 form bracing needed for CIP concrete construction.

Other conduit material alternatives will have smaller spacing requirements. While dewatering equipment is expected to be required, quantities have been shown with and without dewatering facilities. Concrete quantities for the four other methods of construction being considered for the WaterFix BTO conveyance system are included in Appendix E. Additional ground area will be required for construction equipment, materials laydown areas, access, and dewatering equipment.

Where high groundwater is encountered along portions of the alignment, a groundwater collection and disposal system will 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.

9.2.2 Description of Construction Methods and Procedures

Except where crossing under a major waterway, intake conveyance pipelines will be installed via open cut. Excavation will 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 will be installed and operated continuously while the trench is open 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, riprap erosion protection may be required. All discharges will meet the requirements of the NPDES permit. Diesel powered equipment is 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 will 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 will be set aside during excavation and saved for reapplication when construction is complete. A portion of the spoil area will be set aside as a separate topsoil storage area.

The following measures will also be implemented as part of the construction process:
- Dust control measures during construction will conform to all federal, state, and local requirements. Sediment tracked onto public streets will be removed (such as by street sweeping) to prevent it from entering a watercourse and becoming a dust generation source.

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

- Dewatering facility terminations will have velocity dissipation facilities such as rock or grouted riprap to reduce velocity/energy and prevent scour.

After construction is complete, the alignment will be re-contoured as required and all disturbed areas will be seeded. Consideration will also be given to additional replacement or upgrades to drainage facilities.

Erosion control measure for slopes within the alignment will include surface roughening followed by seeding.

Paved areas disturbed by construction will be restored to their original condition prior to construction.

### 9.3 Maintenance Considerations

Maintenance of the conveyance pipelines is dependent on the materials of construction as summarized in Table 9-2.

*Table 9-2: Summary of Pipeline Maintenance Considerations*

<table>
<thead>
<tr>
<th>Material and Conduit</th>
<th>Configuration Maintenance Considerations</th>
</tr>
</thead>
</table>
| 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.  
                       | • Transition structure vent inspection and repairs  
                       | • Regular inspections along the line for signs of leakage or erosion of soil cover |

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

The WaterFix BTO includes two connection channels that will carry water from the tunnel outlet structure downstream of BTF to the Banks PP and Jones PP Intake Channels (see figure 10-1).

10.1 Outlet Structure to Banks Pumping Plant Intake Channel

The first proposed connection channel will span between the South Tunnel Outlet Structure and the Banks Pumping Plant Intake Channel and is sized for a maximum flow of 10,670 cfs. The connection at the Banks Pumping Plant Intake Channel will occur downstream of the existing Skinner Fish Facility.

The connection channel will originate at the outlet structure, located in the south of Byron Highway, where the outlet tunnels from BTF will terminate. The proposed channel is approximately 2,800 LF (0.53 miles) in length. The alignment of the channel is such that it will allow for an adequate transition to the existing Banks PP Intake Channel. The alignment will pass through local roadways, drainage and/or irrigation supply features, and will likely require the relocation of one or more 69kV transmission towers. All these items will need to be resolved in subsequent design phases of the project.

10.2 Outlet Structure to Jones Pumping Plant Intake Channel

The second proposed connection channel will span between the outlet structure and the Jones Pumping Plant Intake Channel and is sized for a maximum flow of 4,600 cfs. The connection to the east, at the Jones Pumping Plant Intake Channel, will be placed approximately 2,500 LF downstream of Byron Highway.

The alignment of the connection channel will travel roughly southeast between the outlet structure and the Jones Pumping Plant Intake Channel. The length of the alignment is approximately 4,800 LF (0.9 miles). The channel will be located primarily east of the 230kV and 69kV transmission lines in the area south of Byron Highway. The channel will cross local roadways and drainage features. All these items will need to be resolved in subsequent design phases of the project.

10.3 Channel Cross-Sections

The cross sections for the connection channels are shown in CER Volume 2 (sheet BTF-C-3087FB). These cross sections represent possible alternatives for the configuration of the connection channels. At this phase of the design effort, there is still information to be determined that will influence the design of the connection channels. Future geotechnical investigations in the area should yield information about the surrounding soil such that it can be determined where to place the channel invert (based on located depth of groundwater), the need for reinforcement within the concrete liner due to potential uplift, and the maximum and minimum side slope criteria. In addition, determination of operational schemes for the conveyance system south of BTF will influence how the channels are designed.
Figure 10-1: BTF Connection Channels
SECTION 11.0

Tunnels

The WaterFix BTO relies primarily on tunnels to convey water south from intakes along the Sacramento River to the Banks and Jones export pumping facilities near Tracy. Tunnel details, including planned alignment, length, depth, diameters, and lining requirements, continue to be evaluated as geotechnical data become available. The dimensions described herein are conceptual and were used in the hydraulic analyses for engineering the concept of the WaterFix BTO.

The following tunnels will be constructed:

- **North Tunnels:**
  - 28-foot ID tunnel between Intake 2 and Intake 3
  - 40-foot ID tunnel between Intake 3 (Junction Structure) and Intermediate Forebay
  - 28-foot ID tunnel between Intake 5 and Intermediate Forebay

- **Main Tunnels:** Two 40-foot ID tunnels from Intermediate Forebay to Byron Tract Forebay Pumping Plant

- **South Tunnels:** Two 40-foot ID tunnels from Byron Tract Forebay Outlet Structure to South Tunnel Outlet Structure

11.1 Description of Tunnel Reaches and Drives

Table 11-1 shows the configuration of each tunnel reach included in the WaterFix BTO and the direction of tunneling. These reaches are considered conceptual and preliminary.

Table 11-1: Tunnel Descriptions

<table>
<thead>
<tr>
<th>Reach</th>
<th>Start of TBM drive</th>
<th>Completion of TBM drive</th>
<th>Internal Finished Diameter, ID (feet)</th>
<th>Number of Tunnel Bores</th>
<th>Flow Capacity (cfs)</th>
<th>Each Tunnel Length (miles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Intake No. 2</td>
<td>Intake No. 3 w/Junction Structure</td>
<td>28</td>
<td>1</td>
<td>3,000</td>
<td>1.99</td>
</tr>
<tr>
<td>2</td>
<td>Intermediate Forebay</td>
<td>Intake No. 3 w/Junction Structure</td>
<td>40</td>
<td>1</td>
<td>6,000</td>
<td>6.74</td>
</tr>
<tr>
<td>3</td>
<td>Intermediate Forebay</td>
<td>Intake No. 5</td>
<td>28</td>
<td>1</td>
<td>3,000</td>
<td>4.77</td>
</tr>
<tr>
<td>4</td>
<td>Intermediate Forebay</td>
<td>Staten Island</td>
<td>40</td>
<td>2</td>
<td>9,000</td>
<td>5.61</td>
</tr>
<tr>
<td>5</td>
<td>Bouldin Island</td>
<td>Staten Island</td>
<td>40</td>
<td>2</td>
<td>9,000</td>
<td>7.21</td>
</tr>
<tr>
<td>6</td>
<td>Bouldin Island</td>
<td>Bacon Island</td>
<td>40</td>
<td>2</td>
<td>9,000</td>
<td>8.88</td>
</tr>
<tr>
<td>7</td>
<td>Byron Tract Forebay Pumping Plant</td>
<td>Bacon Island</td>
<td>40</td>
<td>2</td>
<td>9,000</td>
<td>8.23</td>
</tr>
<tr>
<td>8</td>
<td>Byron Tract Forebay Outlet Structure</td>
<td>South Tunnel Outlet Structure</td>
<td>40</td>
<td>2</td>
<td>15,270</td>
<td>1.58</td>
</tr>
</tbody>
</table>

\(^a\) Individual tunnel capacity listed. From the Intermediate Forebay to BTF capacity is through both tunnel bores.

Notes:

- cfs = cubic feet per second
- ID = inside diameter
The size and configuration of the WaterFix BTO tunnels have been developed by assessing the conveyance facilities as a complete and integrated system. The hydraulic and operational analyses of the system examine the simultaneous sizing of the intakes, tunnels, pump stations and forebays. Included in this analysis was an assessment of various tunnel sizes required to meet project objectives of achieving 9,000 cfs. The size of each tunnel reach takes into account the required hydraulic capacity, hydraulic losses and flow velocities to suspend sediment and minimize sediment buildup in the downstream end of the tunnels. Reaches 1 through 3 are considered as North Tunnels (north of Intermediate Forebay), Reaches 4 through 7 are designated as the Main Tunnels (between IF and BTF) and Reach 8 is designated as the South Tunnels (south of BTF). See Figure 11-1 for an overall map showing the various tunnel reaches.

It is anticipated that the tunnels may be subject to unbalanced internal hydraulic pressures due to high Sacramento River levels, as well as surge pressures due to pump failure. Based on concerns of unbalanced internal hydraulic pressures as described above, one of the critical design issues for the project becomes determining a feasible and cost-effective tunnel lining system that can withstand the external loads acting on the tunnel as well as internal water pressure.

Evaluations were completed to determine viable pressure tunnel lining alternatives (see Appendix I – Basis of Conceptual Design for Tunnel Lining Systems). Appendix I summarizes the results of the conceptual level structural evaluation of a tunnel lining system for limited internal pressure based on operating the tunnels under gravity conditions as an inverted siphon system. The recommendation in Appendix I of using a one-pass lining with bolted connections at the radial joints is the basis of the segmental liner design for the WaterFix tunnels, and is documented in this CER.

Other approaches to optimizing the tunnel lining system will be further evaluated in later phases of the project. The eventual solution will establish a robust lower-bound for the ground load on the lining (along with appropriate factors of safety), and then employ appropriate alternatives to resist the remaining net internal pressure. A number of potential alternative approaches were examined to accommodate these unbalanced internal hydraulic pressures (See Appendix J – Alternative Designs for Tunnel Lining Systems).

Appendix J identifies other tension-resisting lining alternatives in addition to the current design approach as shown in Appendix I of this CER. The following options considered in this appendix as potential cost effective alternatives to the current basis of design:

- Reliance on ground loads
- Use of dowels to resist tension
- Systematic secondary grouting
- Alternative non-ferrous joint connector
- Post-tensioning
- Ferrous push-fit solutions from Japan
- Castellated joint

All tunnels slope “downhill” continuously from north to south, to avoid the construction of siphons, which would otherwise be difficult to dewater and maintain. The preliminary tunnel invert elevations range from 122 to 135 feet below mean sea level (msl) for the North Tunnels and from 147 to 163 feet below msl for the Main Tunnels. The preliminary tunnel invert elevations are based on assumed ground conditions with liquefiable soil at the upper strata. Additional geotechnical investigation will be required during the preliminary and final design phases to finalize the tunnel profile. In addition, the proposed vertical alignment of the Main Tunnels conforms to a Port of Stockton restriction at the undercrossing of the San Joaquin River (SJR) and Stockton Deep Water Ship Channel. Upon further geotechnical exploration and analysis, the Main Tunnel profile will be optimized to determine the

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1 Unbalanced internal hydraulic pressure refers to the difference between internal and external hydrostatic pressures.
actual minimum required depth of cover at the SJR and Stockton Deep Water Ship Channel. The minimum cover will then be compared to the current restriction imposed by the Port of Stockton.

11.1.1 Reach 1
Reach 1 conveys water from the drive shaft at Intake No. 2 to the Junction Structure of Intake No. 3. A single bore 28-foot ID tunnel with a flow capacity of 3,000 cfs will be constructed from the drive shaft to the 113-foot ID Junction Structure at Intake No. 3.

11.1.2 Reach 2
Reach 2 conveys water from the Junction Structure of Intake No. 3 to IF. The reach is a single bore 40-foot ID tunnel with a flow capacity of 6,000 cfs. The tunnel is driven from a 113-foot ID drive shaft at IF and terminates at the Intake 3 Junction Structure.

11.1.3 Reach 3
Reach 3 conveys water from Intake No. 5 to IF. This reach is a single-bore tunnel with a 28-foot ID and a flow capacity of 3,000 cfs. The tunnel is driven from a 113-foot ID drive shaft at IF and terminates at a 100-foot ID reception shaft at Intake No. 5.

11.1.4 Reaches 4 to 7
Reaches 4 to 7 consist of two parallel, 40-foot ID tunnels to convey the flow (9,000 cfs) from IF to BTF. The tunnels are constructed from several main large-diameter shafts (100-foot and 113-foot ID) at Glanville Tract, Bouldin Island, Bacon Island, and BTF. Under the current plans, tunnels will be driven (launching and operation of TBM’s) from Glanville, Bouldin and BTF shaft sites. The Bacon Island and Staten Island shafts are currently planned as reception shafts, and at these locations the TBMs on reaches 4 to 7 will complete their tunnel drives. Once tunneling is completed, the TBMs will be removed from the tunnels at these two locations (see Figure 11-1).

11.1.5 Reach 8
Reach 8 consists of two parallel, 40-foot ID tunnels to convey the flow (15,000 cfs) from BTF to connection channels that will deliver flows to both Banks PP and Jones PP respectively. The tunnels are constructed from a large-diameter drive shaft at the BTF Outlet Structure. The reception shaft will be at the outlet structure on the connection channels leading to the Banks PP Intake Channel and Jones PP Intake Channel.

11.1.6 Inlet and Outlet
The tunnels are connected to the river intake facilities by a specialized transition and dropshaft structure shown on the Concept Drawings (Volume 2) and described in Section 7.0, “Intakes,” and Section 14.0, “Forebays.”

Tunnels can be isolated and dewatered separately for inspection and maintenance if required. The following provisions are provided for isolation and/or dewatering:

- North Tunnel bore for intake 2 and 3 are connected together to the IF and can be isolated as one. Intake 5 is independently connected to the IF and can be isolated separately. The two main tunnel are connected to each of the pumping plants independently from one another, and can be isolated separately.

- At the discharge to the forebays and the inlets of the Main Tunnels at the Intermediate Forebay, sets of drop gates are used to isolate each tunnel and allow its dewatering for inspection.

The IF serves as surge dampening and as a transition between the North Tunnels and the Main Tunnels.
Figure 11-1 Tunnel Reaches
11.1.7 Dewatering and Filling

Access at each shaft is provided to facilitate tunnel dewatering at each tunnel (the structure at Intake No. 2, 3, and 5, two North Tunnel shafts at the Intermediate Forebay, and the two Main Tunnel shafts at the BTF pump structures). The tunnel dewatering equipment will include submersible pumps, each with a design capacity of approximately 25 cfs, with submersible motors that will operate with either 460 volt or 4160 volts. Each pump will deliver the target capacity and operate through a range of heads from 135 feet to 200 feet. The submersible pumps will be lowered into place on rails until they seat on the bottom of the shaft, where they will be securing into place for operation.

The assemblies are installed at one or more shafts on the Main Tunnels to speed dewatering. These assemblies will allow for the installation of temporary pumping equipment to dewater the tunnels. Given the infrequent need for dewatering, no permanent pumping capability is provided, and tunnel dewatering will be performed with temporary pumps.

The pumps can also be used to fill from the Intermediate Forebay to the North Tunnels adjacent to the Intermediate Forebay and from the BTF to the Main Tunnels, provided at least one other North Tunnel or Main Tunnel is operational. In addition, air and vacuum relief valves are along the tunnel alignments to facilitate filling and draining of the tunnels.

At each location, water in one tunnel will be pumped out to the Intermediate Forebay, BTF, shafts in Intakes No. 2, 3 and 5, or adjacent Main Tunnel shaft when dewatering. Water from those same locations is allowed to feed back into the empty tunnel for filling. Additional details and provisions for initial filling of the tunnels will be developed as part of preliminary design.

Pump and system curves representing the range of operating conditions are presented in Figure 8-2. The low-head curve represents the hydraulic conditions after the shafts are partially dewatered to the wet-well invert. When operating in this condition, it will be necessary to throttle the discharge to prevent the pump from operating outside the Acceptable Operating Range. The system curve referenced as “Water Level at Tunnel Crown” represents the operating conditions when the water level drops below the crown of the tunnel near the Intermediate Forebay (the Main Tunnels slope upward from the pump shaft to the Intermediate Forebay). When the water level reaches this elevation, the pumps can be operated without valve throttling. The pumping rate for each pump would range from 34 cfs, when the water level is near the tunnel crown, to 24 cfs when the water level is near the tunnel invert. It is estimated that with continuous operation of 8 pumps, dewatering could occur in approximately 10 days.

11.1.8 Finished Facilities Security

Once tunneling at each shaft site is completed, the construction shafts will be retrofitted so that they serve a permanent future access points to the tunnels for inspection and maintenance purposes. Consequently, the rough opening of the working shaft will be reduced to approximately 20-40-foot diameter and this opening will be fitted with a removable cover/hatch. Such coverings on exposed finished shafts will be provided with mechanisms to securely lock the roof, and each of these sites will be fenced. At these shaft/access sites, there are no additional security features except as related to the other main facilities at the inlet and outlet ends of each tunnel system.

11.2 Construction Methodology

The final selection of tunneling methods and the number of construction sites will be determined according to ground conditions and construction schedule. Based on an understanding of current tunneling technologies and general tunnel construction/contract practices, the conceptual design described in this document highlights the anticipated tunnel construction reaches. These tunnel construction needs, both from the standpoint of temporary construction footprints and permanent facility footprints, have been included in the Final EIR/EIS for the WaterFix BTO.

See Section 24 Construction Methods, Sequencing, Constructability and Schedule for detailed information regarding construction methodology, sequencing, and schedule.
11.3 Safety

11.3.1 Levee Failure and Shaft/Tunnel Flooding

It is unlikely that the shafts and tunnels will be inundated by a breach of the levee because the shaft collars will be formed on elevated fill pads (except the shafts at IF). However, due to the low lying nature of the shaft sites, other supporting systems may be affected adversely by a flood. To address this possibility, secondary levees are planned around the perimeter of the elevated shaft pad construction areas at the shaft sites to create a protected area adjacent to the elevated pads. These secondary levees provide some level of flood protection, but are not high enough to protect against severe flooding. At IF, all four tunnel shafts are protected by a levee equal in height to the elevated fill pads at the other shaft sites.

To help mitigate flooding issues and provide life safety for workers, it is assumed that the emergency standby generators and other life-critical systems such as ventilation will be sited on the elevated pad, above severe flood inundation levels. Additional analyses are required during design of the tunnels to determine the correct elevations for shaft tops and temporary works at each shaft site, including the protective levee at the IF.

11.3.2 Tunnel Classification

All tunnels constructed in California are required by law to obtain a tunnel classification from the State of California Division of Mines and Tunnels.

There are active natural gas fields deep below the anticipated alignment for the tunnels. These gas fields are estimated to be more than 1,000 feet below the inverts of the tunnels. As these gas fields are in the vicinity of the proposed tunnel alignment, it is anticipated that the State of California Division of Occupational Safety and Health (Cal/OSHA) might classify the tunnels as “potentially gassy.” This classification requires high levels of precautions related to tunnel construction safety. Under this classification, the TBMIs are required to be equipped with gas monitoring equipment that automatically shut down the TBM if gas is detected. It is also likely that special access and egress requirements may be imposed by Cal/OSHA. Additionally, all equipment used in the tunnels needs to be intrinsically safe.

11.3.3 Tunnel Ventilation System

Tunnel ventilation requirements defined by Cal/OSHA, including the minimum amount of fresh air supply and its flow velocity, are presented in the California Code of Regulations. Cal/OSHA requires that at least 200 cubic feet per minute of fresh air per person working underground be provided. Additionally, a minimum air velocity of 60 feet per minute (fpm) is required to dilute any contaminated gas present within the tunnel. Certain activities within tunnels, such as welding, require higher minimum air velocities.

Contractors usually provide more air flow than required by Cal/OSHA (e.g., approximately 100 fpm) in order to avoid work interruptions due to gas concentrations building up beyond allowable thresholds. This practice reduces the risk of work shutdowns and mandatory tunnel evacuation until the contaminant is diluted by the ventilation system to a concentration below the safe limit.

Cal/OSHA also sets requirements for the ventilation hardware used. Systems including steel ducts and explosion-proof fans capable of reversing the direction of air travel are required. The contractor usually designs the ventilation system when developing the means and methods for the work.

11.3.4 Tunnel Interventions

Certain operations for maintenance and repair of the TBM require work to be performed under high water inflow, high groundwater pressures, unstable ground conditions, or any combination of these factors. Where these operations are pre-planned at a particular location, it might be possible to construct a safe haven (such as at a vent shaft) by ground improvement to maintain stable conditions with reduced water inflow and under atmospheric conditions. Once a TBM reaches these areas, pressures at the face of the TBM can be reduced without inducing ground loss or excessive deformations. This allows maintenance activities to be undertaken at atmospheric pressure, which is safer and less expensive than personnel working in hyperbaric conditions.
It is likely that there are occasions where these operations cannot take place at a predetermined location. The TBMs should be designed to maximize maintenance and repair work performed from the TBM interior. For events where interventions are needed from the exterior, the equipment and procedures in place shall ensure that the work is performed safely, with minimum delay.

### 11.3.5 Other Tunneling Issues

Tunneling operations have additional impacts that need to be considered, including traffic, noise, lighting, vibration, dust and air quality, and tunnel water treatment and disposal. For additional information, environmental commitments are identified in the Final EIR/EIS, Appendix 3B. Treatment and disposal of construction water from the tunnel requires permitting according to current NPDES and Regional Water Quality Control Board regulations.

### 11.3.6 Ground Improvement

Ground improvement may be required to facilitate construction of the permanent structures, tunnel, and shafts; facilitate groundwater control at the locations of the shafts; prevent development of undesired ground movements; and potentially provide predefined zones for TBM maintenance interventions. The types of ground improvement that should be considered during future design activities include jet grouting, permeation or compaction grouting, and ground freezing. Site-specific geotechnical investigations are needed to design the extent and type of ground improvement that may be required.

### 11.3.7 Behavior Under Seismic Events

The tunnels and shafts must be able to withstand the design ground motions while maintaining continuous operation of the system. All structural systems shall be considered as Essential Facilities per California Building Code, which means the key systems shall remain operational after the maximum considered earthquake. Section 4.0, “Geologic and Seismic Considerations,” describes the typical seismic responses/damages of tunnels are expected to be less than those for above-ground structures. Special design considerations will be implemented to address the shaft-tunnel connection to account for the differences in structural stiffness and ground strain between the two elements. Special design considerations will be utilized to address tunnel shafts penetrating through potentially liquefiable soil conditions.

The conceptual design of the segment liner considered ground strains associated with three types of deformation resulting from earthquake motions:

- Axial extension and compression due to seismic wave propagating along the tunnel.
- Bending due to wave action perpendicular to the tunnel.
- Ovaling due to shear waves propagating normal to the tunnel.

The preliminary and final design will further evaluate the seismic performance based on detailed geotechnical data.

### 11.4 Maintenance Considerations

Maintenance requirements for the tunnels have not yet been finalized. Some of the critical considerations in terms of maintenance include evaluating whether the tunnels need to be taken out of service for inspection and, if so, how frequently this is required. Typically, new water conveyance tunnels are inspected at least every 10 years for the first 50 years and more frequently thereafter. In addition, the equipment that the facility owner needs to put into the tunnel for maintenance needs to be assessed so that the size of the tunnel access structures can be set. Equipment requirements such as trolleys, boats, harnesses, camera equipment, communication equipment, and ventilation need to be assessed prior to finalizing shaft designs.

Maintenance activities are expected to include, at a minimum, periodic inspection of the tunnels by remotely operated vehicles and removal of sediment that accumulates in the tunnels. Additional study is required to determine the frequency and extent of sediment removal activities.
Note that permanent power for maintenance is not anticipated at the Main Tunnel shafts; therefore, portable power is required during maintenance operations.

11.5 Engineering Analysis

During preliminary design, engineering analyses are required to confirm the constructability and cost of the selected vertical and horizontal alignments, shaft locations, and construction methodology selected for the tunnel and shafts.

Recommended engineering analyses include (but are not limited to):

- Anticipated geotechnical conditions.
- Anticipated ground behavior.
- Evaluation of soil abrasiveness.
- Corrosion evaluation.
- Earth and groundwater loads on tunnel support.
- Groundwater treatment/improvement feasibility analyses.
- Seismic motions and deformation.
- Internal pressure loads on tunnel lining.
- Handling loads during segment erection and transportation.
- Gasket design based on contact pressure, gap width, and offset anticipated.
- Concrete segment joint design.
- Segment leakage analysis and design.
- Tunnel lining tension design with potential scaled testing.
- Tunnel infiltration/exfiltration analysis.
- Evaluation of need for secondary lining or membrane due to internal tunnel pressures.
- Tunnel to shaft connection during normal operation and seismic conditions.
- Lateral earth pressures for shaft design.
- Shaft bottom stability.
- Shaft area settlement calculations.
- Tunneling-induced settlement calculations.
- Operation/ventilation requirements for tunnel dewatering and filling.
- Resource availability.
- Access and enabling works requirements.
Access to WaterFix sites may require both land and water routes.

12.1 Road/Land Access

Haul routes and access roads consist of two types: all-weather access roads and existing public and/or private roads. The issue of dust abatement will need to be addressed in all construction areas at all times.

All-weather access roads will be required for year-round construction at all facilities, including concrete and steel structures, tunnel portals, tunnel shafts, forebays, pumping plants, intakes, as well as for access to delivery areas, borrow areas, and permanent tunnel muck and excess excavation spoil piles. All-weather roads typically are surfaced with a minimum 24 inches of gravel.

Existing public and/or private roads will be used, as needed, for year-round access to all of the construction areas.

Haul routes should maximize use of the state highway system where possible because these roadways are rated for truck traffic and will generally provide the most direct and easily maneuverable routes for large loads. Once construction traffic exits the state highways, it will transition to county roads. County roads typically have one lane of traffic in each direction with paved shoulders ranging from under 1 foot to over 3 feet on each side. State highways and county roads will typically carry construction traffic to within a mile of major project work sites.

Construction traffic will also require the use of private roads to access the work sites, including tunnel shaft sites and barge landings. Private roads will carry construction traffic from the nearest state highway or county road through private land to the shaft sites. The private road segments required for access are expected to range in length from under 0.25 mile to over 5 miles. The majority of private access roads identified for use are dirt or gravel access roads on agricultural land. Private levee roads may also be used for construction traffic. The condition of levee roads range from paved to gravel roads.

It is anticipated that the following roadways will be impacted by construction activities:

- Highway 160 (SR 160)
- Highway 12 (SR 12)
- Highway 4 (SR 4)
- County Route J4 (Byron Highway)

For detailed anticipated construction impacts to these roadways see Section 24.

12.2 Water Access and Landing Facilities

Barges may be required for delivery of equipment and materials, hauling fill material, and possibly for muck hauling. Overall barge activity may include barge landing construction, barge operations in the river, tug operations, and barge landing removal. The majority of barge trips will probably originate at the Port of Pittsburg or Stockton due to their centralized locations relative to the proposed alignment. If necessary, alternate departure points include the Ports of Sacramento and Rio Vista. However, current lock functionality issues may limit the practicality of using the Port of Sacramento. If necessary to facilitate construction at a particular site, barge routes and landing sites will be selected by the construction contractor. All use of barges and barge landings will be expected to comply with the following criteria:

- Existing environmental commitments in the Final EIR/EIS
- Maximize continuous waterway access between departure port and shaft site.
- Use of existing barge landings where possible.
• Minimum water depth of 6 feet.

Loading and offloading construction equipment and materials from barges in the Delta can be accomplished by the use of a barge landing or by pushing ramp barges up against levees and unloading directly onto the levee. Boat ramps are not desirable for barge activities because the gradual slope of such ramps cause barges to bottom out during the loading process.

Currently, there are no barge landings in the vicinity of the proposed intake sites or launch, retrieval, and intermediate shaft sites, and limited data shown that typical levees in the Bay Delta area are too narrow and/or not stable enough to support substantial or frequent loading and unloading operations. For these reasons, a loading/unloading facility will need to be developed along the waterways near the launch shaft sites to facilitate barge delivery of heavy TBM components, routine barge delivery of heavy tunnel lining segments or fill soils, or barge export of tunnel spoils. Precise barge sitings and dimensions are to be determined by DWR’s construction contractors. Barge landings may also be needed for safe haven access sites, if they are sited in areas where existing surface roads will not be adequate to transport the equipment needed for shaft construction.

Where possible, the least environmentally impactful barge landing will be constructed. Barge landings construction will comply with the Final EIR/EIS requirements. Improvements will be required and may include any or all of the following:

• Construction of a working pad on the land side of the levee to support cranes and/or barge unloading ramps or bridges, as well as to serve as a staging and unloading area.
• Construction of a backfilled sheet pile wall to serve as a marginal wharf where barges could be moored for loading and unloading.
• Construction of on-land or in-water mooring dolphins to secure the barges during loading and unloading.

Loading and unloading of the barges could be performed by one or more alternatives at each facility. These could include:

• Crane Barge
• Ramps
• Tracked or Fixed-Base Crane
• Conveyor.

In general, more extensive barge landing facilities may be required for loading and unloading large quantities of fill material or tunnel muck. Specific design of these facilities will be required. All such facilities that affect levees will be designed, constructed, and operated in full compliance with all permits and environmental regulations, as well as the requirements of the CVFPB, USACE, and the local district with jurisdiction of the specific levee.

It is most critical to remove all heavy equipment from the levee as soon as possible to avoid compromising the structural integrity of the levees. For this reason, it is recommended that a dirt road, including an earthen ramp, be constructed on the landside of all levee unloading points if an existing ramp is not within the immediate vicinity. This will allow for all heavy equipment and materials to be removed from the levee immediately after unloading. Spoils from shaft construction could be used to construct these ramps.

1,2 Portions of this section were adapted from the draft Construction Access technical memorandum prepared by CDM Smith for Metropolitan Water District of Southern California dated August 29, 2012 (CDM Smith, 2012).
SECTION 13.0  
Utility and Infrastructure Crossings

This section identifies utility crossings associated with the WaterFix BTO alignment. These include the power transmission lines; communication transmission lines; buried gas transmission lines; agricultural irrigation water deliveries and drainage ditches; and potable water delivery systems that require rerouting or relocation, plus natural gas wells located in the natural gas fields located along the alignment. Buildings and infrastructure within the right-of-way (ROW) are also addressed in this section. Road and rail infrastructure crossings are discussed in Section 12.0, “Access to WaterFix Sites.”

WaterFix BTO construction impacts existing and proposed facilities. The extent of these impacts is evaluated through the project environmental process. Other utilities, including smaller electrical power distribution lines and communication lines, require field verification and coordination with utility providers. These other utilities will be identified in subsequent phases of the project and are not expected to result in impacts related to environmental review.

13.1 Description of Utilities

13.1.1 Inventory of Affected Utilities

The utility crossings, oil and natural gas wells, and structures were identified by overlaying geographical layers on 2006 high-resolution aerial photos from DWR, Delta Risk Management Strategy (DRMS), U.S. Geological Survey (USGS), California Department of Conservation (DOC), Ventyx, National Hydrography Dataset, and Reclamation.

The WaterFix BTO alignment crosses or potentially interferes with:

- Overhead power/electrical transmission lines.
- Natural gas pipelines.
- Inactive and active natural gas and oil wells.
- East Bay Municipal Utility District’s (EBMUD) Mokelumne Aqueducts.
- Various structures.
- Agricultural delivery canals and drainage ditches.
- Local electrical distribution lines.
- Local telephone and communication lines.

Each crossing or interference requires avoidance or mitigation measures. Measures to avoid existing electrical transmission lines, crossing over or under existing pipelines (revising elevations where necessary), relocating or rerouting existing utilities, capping abandoned wells, and revising drainage routing will be developed at the preliminary design level. Final measures will be determined on a case-by-case basis and will incorporate applicable state, county, city, and local agency requirements.

13.1.2 Power Lines

The WaterFix BTO forebays and stockpiles, borrow and spoil areas conflict with several power/electrical transmission and distribution lines owned by PG&E, SMUD, and WAPA. Avoidance measures and/or possible relocation will be determined during preliminary design.
13.1.3 Natural Gas Pipelines
Most of the natural gas pipeline crossings are near the surface (less than 30 feet below grade) and within the tunnel portion of the alignment. Since the tunnels are located in excess of 100 feet below grade, direct conflicts are not anticipated. Such crossings will be closely coordinated with the pipeline owners.

13.1.4 Natural Gas Wells and Fields
The project’s alignment passes through areas of the Delta that are underlain with gas fields. Figure 13-1 shows the approximate location of known gas wells and gas fields within the Delta. These gas fields lay 1,200 to 9,000 feet bgs, and no direct conflicts with the fields and project facilities are anticipated.

Available information indicates the presence of active, inactive and abandoned natural gas and oil wells in the project footprint including the tunnel alignment, and borrow, spoil, or muck areas. Publically available information, including the Division of Oil, Gas, and Geothermal Resources (DOGGR) reports, were used to set the preliminary tunnel alignment to avoid active gas wells.

13.1.5 Mokelumne Aqueducts Crossing
EBMUD constructed the three pipelines of the Mokelumne Aqueduct (Nos. 1, 2 and 3) in 1929, 1949, and 1963, respectively. They cross principally above ground on saddle pipe supports through the Upper and Lower Jones Tract, Woodward Island, Orwood Tract, and Bixler Tract.

The WaterFix BTO conveyance tunnels will be constructed well below the aqueduct at the north end of Woodward Island, and no conflicts are anticipated. The design and construction of this crossing will be closely coordinated with EBMUD.

13.1.6 Miscellaneous Structures
Aerial and satellite photo evaluation has identified many locations where structures are within the surface-impacted ROW of the preliminary conveyance alignment. Many locations contain multiple structures. Field surveys will verify structure location and condition. The tunnels are well below these structures, and no conflicts are anticipated.

13.1.7 Agricultural Delivery and Drainage Ditches
Agricultural delivery and drainage canals were identified using the National Hydrography Dataset. Minimal conflicts are anticipated for the tunnel alignments.

It is anticipated that existing agricultural drainage and/or delivery canals will be impacted by surface construction of WaterFix facilities. These existing canals will need to be further studied in a later design phase in order to determine the best course of action for either relocating or abandoning the drainage or delivery systems. Surface work that impacts irrigation and drainage ditches will be closely coordinated with property owners and reclamation districts.

13.1.8 Water Codes Statutes
The following water code sections and related statutes apply to the facilities described in this section:

- Water § 259
- Water § 11590
- Water § 11592
13.2 Construction Methodology

The utility and infrastructure crossings within the conveyance tunnel sections are minimally impacted by construction. To avoid the potential conflicts with shaft construction and disposal areas, the utility and infrastructure relocation will be coordinated with local agencies and owners.

For more detailed information regarding anticipated construction impacts to utilities, see Section 24.
Figure 13-1: Known Gas Wells and Fields in the Delta Region
SECTION 14.0
Forebays

The WaterFix BTO includes the following facilities:

- **The Intermediate Forebay (IF)** is mainly a pass-through facility providing an atmospheric break at the inlet to the Main Tunnels from IF to Byron Tract Forebay (BTF). Total system hydraulic loss between each intake location and the inlet to the Main Tunnels is mostly friction loss changing with flow. The hydraulic break allows for independent operation of the North Tunnels and the Main Tunnels, enabling isolation of each tunnel for maintenance.

- **Byron Tract Forebay (BTF)** will receive water conveyed through the north Delta conveyance system and provide capacity for daily operational storage to equalize and balance differences between inflow from the north Delta intakes and water exported by Banks and Jones pumping plants. The combination of the tidal cycle diversions and the efficient operation of the pumping plant affect the operation of the intakes and tunnel delivery flow.

14.1 Intermediate Forebay

14.1.1 Requirements and Assumptions

**Hydraulic Connections**: The only source of water to the IF is from the Sacramento River at the north Delta intakes. The only outlets from IF are the Main Tunnels.

**Head Loss Allowance**: There is negligible head loss between the inlet and the outlet of IF, depending on the size and configuration of the IF. Further hydraulic modeling and analysis will be conducted during the preliminary engineering to optimize the configuration and size of the IF in conjunction with the operation of the pumping plant at BTF.

**Stone Lake National Wildlife Refuge (SLNWR)**: The IF is located just east of property owned by the SLNWR adjacent to Zacharias Island. A buffer zone of at least 100 feet will be maintained from the linear parcel owned by SLNWR to the edge of IF levee construction footprint.

**Forebay Need**: The IF provides a location to mix flows from all three intakes prior to hydraulically splitting the flow equally into the two Main Tunnels. The IF also provides a location to dampen hydraulic surges. It is a pass-through facility with no daily operational storage and no regulation of flow to the Main Tunnels. As currently conceived, the IF is the smallest practical size to provide sufficient velocity reduction to deposit sediment not removed at the intake sedimentation basins.

**Seepage**: A slurry cutoff trench beneath the IF levee embankment protects the foundation from underseepage and piping. The cutoff trench is anticipated to extend to an elevation of at least -50 feet. A drain at the toe of the outer embankment slope limits saturated conditions at the ground surface. Seepage and control measures will be further analyzed once more detailed geologic information and monitoring data are available.

**Electrical**: The IF shall be fed from the utility via two 4160V, 3-phase incoming service feeders. Each incoming service feeder shall be routed into the electrical building to feed an arc-resistant, main-tie-main configured switchgear. The switchgear will then distribute the 4160V to the major loads, including the dewatering pumps and the 4160V to 480V transformers. The switchgear will be located within the electrical building’s medium voltage electrical room.

**Excavated Material**: Limited soils information and subsurface data show that soils in the vicinity of IF consist of organic clay, lean and fat clay, silty sand, clayey sand, sandy silt and sand. The subsurface data indicate that the organic soils can be encountered occasionally and extend to an estimated depth of 5 to 6 feet. It is estimated that the majority of the materials excavated below the near surface organic soils for the IF are reusable for embankment construction if properly moisture-conditioned prior to placement.
**Dam Regulatory Authority:** The IF stores water at an elevation more than 6 feet higher than the surrounding land, and it will have a storage capacity larger than 50 AF. It is assumed that IF will be subject to the jurisdiction of DWR’s Division of Safety of Dams (DSOD). Determination of jurisdictional status will be made during preliminary design work. If the IF is deemed jurisdictional by DSOD, then an application for construction of the IF must be obtained from DSOD during design, and an inundation map must be submitted to the California Office of Emergency Services (OES) in accordance with the California Emergency Services Act, Government Code, Section 8589.5 prior to operation of the IF.

**Security:** Site security for IF facilities and site consists of fencing to secure each site and prevent public access to sensitive areas or those with potential hazards, such as structures, open water, or steep slopes. Security camera systems and intrusion alarm systems would be installed at the site and control structures. Admission to the site requires credentialed entry through access control gates.

**Emergency Spillway Design:** If the IF is determined to be a DSOD jurisdictional facility, then all requirements such as an Emergency Spillway or low level outlets will be designed accordingly during Preliminary Design.

### 14.1.2 General Conditions

**Location:** The IF is located southwest of Glanville Tract and just east of Pearson District. The bottom is -20 feet elevation, except locally at the IF inlet and outlet structures. The surface area is approximately 27.5 acres at elevation -20 feet (inside toe of embankment). It is approximately 36.6 acres at a 0 foot elevation (approximate WSE at 9,000 cfs flow with Sacramento River WSE +10 ft), which would provide an opportunity for gravity flow at that elevation. At the top of the forebay embankment, elevation +32.2 feet, the surface area is 53.6 acres. The water is delivered to the IF inlet structure via the North Tunnels from the Sacramento River intakes.

**Inlet Structure:** Incoming North Tunnels transition to vertical shafts that terminate at the IF inlet structure. The inlet is a reinforced concrete structure consisting of bridge deck, piers, and wingwalls. The inlet structure incorporates two multi-gated bays with drop gates (5 at each bay) to provide isolation of each North Tunnel vertical shaft. One inlet has a capacity of 6,000 cfs to accommodate the 40-foot diameter tunnel connecting Intakes No. 2 and No. 3. The other inlet has a capacity of 3,000 cfs to accommodate the 28-foot diameter tunnel connecting Intake No. 5 to IF. All flows subsequently pass through a multi-gated structure into the IF main pool. This structure includes drop gates installed in openings between intermediate piers. Gates can be operated from an operating deck at the maximum embankment elevation of +32.2 feet.

**Outlet Structure:** The outlet structure is at the south end of the IF and connects directly to the 40-foot ID Main Tunnels via shafts. The outlet is a reinforced concrete structure consisting of bridge deck, piers, wingwalls, and trash racks. Flows through drop gates will discharge to a transition structure that directs flow to the vertical outlet shafts. The apron at the outlet shafts is proposed at elevation -22.5 feet. This elevation will allow shaft submergence when the BTF is at its minimum operating level of +1.1 feet and the intakes are not operating. Drop gates within the IF outlet structure allow isolation of each vertical Main Tunnel shaft.

**Isolation:** Drop gates at the inlet structure isolate each North Tunnel shaft. Drop gates at the outlet structure provide similar isolation for the Main Tunnel shafts.

**Debris Racks:** Any floating debris in the IF are intercepted by trash racks installed upstream of the outlet shaft.

**Embankment Layout:** The IF initial construction includes the permanent eastern and western sides of the IF embankment and outer embankments at the northern and southern ends around the construction shafts area for the North Tunnels and the Main Tunnels. This embankment provides flood protection for the construction areas. When tunnel construction is complete, the remaining portion of the IF is constructed along with the inlet and outlet. The embankment is constructed of engineered fill. The inlet and outlet structures are constructed on improved ground.

**Embankment Design:** The embankment crest elevation for the IF is +32.2 feet, which includes considerations for flooding and Sea Level Rise (SLR). This elevation is based on the recommended design embankment flood protection level at Pearson District between Hood and Snodgrass Slough (DHCCP Team, 2009). The embankment cross-section consists of engineered fill placed on suitable foundation material at a 4H:1V slope on both the...
inboard and outboard sides of the embankment. The embankment slopes could be steepened, depending on 1) conditions encountered in future subsurface exploration beneath the IF, 2) the risk of liquefaction, and 3) whether the interior slopes are lined with an impermeable liner. The embankment crest is 32 feet wide, which consists of a 24-foot-wide, two-way maintenance access road with 4-foot shoulders on each side.

The upstream side (water side) of the new embankment includes stone slope protection (riprap). The riprap is placed over an appropriate filter layer and extends from the toe of the embankment to the crest. Other linings, such as permeable asphalt concrete, can be considered and evaluated during preliminary engineering.

**14.1.3 Intermediate Forebay Embankment Stability**

**Foundation Conditions:** The subsurface conditions for the IF are based on a historic DWR boring about 1,600 feet southwest of the proposed IF site on Pearson Tract. The generalized soil profile consists of soft organic clay and silt to a depth of about 22 feet and stiff to very stiff silt and clay with thin interbeds of fine sand to 30 feet. Actual soil conditions at the IF are likely different. A recent soil boring and a CPT about a mile from the IF on Glenville Tract indicate low potential for liquefaction. Detailed geotechnical information on this site will be detailed through the program’s overall geotechnical exploration effort, which will commence in the early stages of preliminary design.

**Method of Analysis:** The preliminary stability of IF embankments was evaluated using limit equilibrium slope stability methods. Analyses used the computer program SLOPE/W Version 7.17 by GEO-SLOPE International of Calgary, Canada. The critical shear surfaces were found interactively. Circular failure surfaces were evaluated using the Spencer method of slices.

For the case of rapid drawdown of the IF, the embankment was evaluated for stability in accordance with a three-stage analysis as described by Duncan, Wright, and Wong (1990).

Pseudo-static analyses were performed to model seismic loading conditions. The pseudo-static analyses subjected the two-dimensional sliding mass to a horizontal acceleration equal to an earthquake coefficient multiplied by the acceleration of gravity. The earthquake coefficient, or pseudo-static coefficient, was determined based on procedures presented in Blake et al. (2002), using a preliminary deterministic design PGA value of 0.32 g (see Section 4.0, “Geologic and Seismic Considerations”). The pseudo-static analyses provided a preliminary check, or screen, on whether further analyses should be performed. For conditions where the factor of safety for the pseudo-static analyses is less than 1, a displacement analyses is required to estimate the anticipated ground deformation that may occur during the design earthquake loading.

Stability analyses were also performed using post-seismic residual strength to evaluate stability of the embankments in the event of liquefaction within the shallow foundation materials. It was assumed that there would be minimal strength loss or pore-pressure buildup within the compacted earthfill embankments during an earthquake.

**Material Properties Used in the Stability Analyses:** The total stress shear strength parameters for the foundation clay soil were based on triaxial test results and undrained shear strength interpretations from the CPT soundings. The scattered sand soil layers were conservatively ignored in the preliminary stability analyses, except where the effect of liquefaction was considered between depths of 5 to 10 feet bgs. The effective stress shear strength parameters were based on typical values for the various soil conditions encountered, and also on correlations with plasticity index.

For liquefied soil conditions following an earthquake, an undrained strength of 400 pounds per square foot was estimated based on the correlation with Standard Penetration Test blow counts presented by Seed and Harder (1990).

**Conditions Evaluated:** The IF embankment slopes are inclined at 4H:1V, and were analyzed under the following conditions:

- Static, saturated, long-term conditions.
- Undrained conditions immediately following construction.
• Rapid drawdown conditions on the interior side of the embankment.
• Pseudo-static analysis.
• Post-earthquake conditions with liquefied conditions within the foundation soil.

**Stability Analyses Results:** The results of the preliminary stability analyses of IF embankment slopes of 4H:1V were determined to be acceptable in regards to slope stability under all the conditions analyzed. Long-term slope stability was determined to have a factor of safety of at least 1.5, per requirements of Reclamation's Design Standard 13. Final design confirmation stability analyses should be performed after the completion of site-specific geotechnical exploration and testing and the adoption of final seismic design criteria for the IF.

### 14.2 Byron Tract Forebay

#### 14.2.1 Requirements and Assumptions

**Hydraulic Connections:** The only source of water will be from the north Delta intakes via the two main tunnels. The only outlet from BTF are two 40 foot ID tunnels that outlet at the southern end of the forebay.

**Fish Protection:** Fish are screened from entering the north Delta conveyance system at the Sacramento River Intake facilities, and thus are excluded from BTF. The BTF connection channel tie-in to the Banks PP Intake Channel will be located downstream of the existing Skinner Fish Facility (upstream of the Banks PP), and the BTF connection channel tie-in to the Jones PP Intake Channel will be located downstream of the existing Tracy Fish Collection Facility (upstream of the Jones PP).

**Forebay Storage Need:** Jones PP requires a conveyance facility that permits continuous withdrawal of a maximum 4,600 cfs. With the potential for inflow from the tunnels to vary throughout a typical day, BTF is proposed to balance inflow with outflow to support the Jones PP. Similarly, BTF is necessary to enable the Banks PP to maximize its operation when electrical power rates are lowest (off-peak). Preliminary calculations indicate a minimum operational storage requirement between 4,400 and 5,900 AF (see Section 5.0, “Conveyance System Operations”).

**Active Storage Availability:** The area of the proposed BTF is located northwest of the existing Clifton Court Forebay. The water surface area of BTF is approximately 810 acres (Elevation +2.5 feet) at minimum pool elevation. As described in Table -5-5 (Section 5.0 – “Conveyance System Operations”), the operating range for the isolated north Delta would be +2.5 to +12.5 ft. This operating range is based on the surface area of BTF and hydraulics of the outlet conveyance system and would result in approximately 8,350 AF of potential active storage in BTF for the isolated north Delta operation which exceeds the storage requirement stated in Section 5. To match the minimum required operational storage as outlined in Section 5, the WSE operating range at BTF would be +2.5 to +10 ft. This would result in approximately 5,960 AF of operational storage. The operating range for a potential dual source operation scenario could have a higher maximum WSE than the north Delta operation, depending on the capability of downstream gate structures to control flow schemes entering the Banks PP Intake Channel, and thus would also increase the active storage capacity of BTF. Otherwise, the Banks PP may need to pump during a short portion of the on-peak pumping period.

**Dam Regulatory Authority:** It is assumed that the BTF will be subject to the jurisdiction of DWR DSOD because it stores water at an elevation more than 6 feet higher than the surrounding land.

**Security:** Site security for the BTF facilities and site consists of fencing to secure each site and prevent public access to sensitive areas or those with potential hazards, such as structures, open water, or steep slopes. Security camera systems and intrusion alarm systems are in the site areas, at major control structures, and at all buildings. Admission to the sites and buildings require credentialed entry through access control gates and secure doors, respectively.

#### 14.2.2 General Description

**Location:** BTF is proposed to be located at the northwest corner of the existing Clifton Court Forebay adjacent to the Italian Slough. The proposed site is largely classified as “agricultural” and contains no sensitive environmental...
areas. Additionally, the proposed site is anticipated to have relatively favorable geotechnical characteristics. Water is delivered to BTF via two vertical shafts at the northeast corner of the forebay. Water flow is controlled by VFD pumps installed inside the two vertical shafts. The pumps and control system are described in more detail in Section 8.0 “BTF Pumping Plant.”

**Bottom Elevation:** It is possible that the current ground contours within BTF will have little or no changes after construction. Current elevations within the proposed BTF range from -9 to +4 ft (MSL). BTF will require maintenance with regular harvesting, dredging, and cleaning. For additional forebay maintenance considerations, see Section 14.

**Connections to Pumping Plants:** There will be an outlet structure located at the south end of the forebay where two 40 foot ID tunnels will carry water to another outlet structure located south of Byron Highway. Each outlet tunnel will be approximately 8,378 feet long (1.59 miles). For the outlet structure south of Byron Highway, two canals will carry water both to Banks and Jones Pumping Plant Intake Channels. These connection channels will be approximately 2,800 feet (outlet structure to Banks PP Intake Channel) and 4,800 feet (outlet structure to Jones PP Intake Channel) in length.

Both connection channels will require further investigation in the next stage of engineering to verify the conceptual design characteristics.

**Isolation:** A control structure is proposed to be located within each connection channel to hydraulically isolate existing SWP and CVP facilities from BTF, and additional control structures will be installed within the existing Banks and Jones PP Intake Channels to isolate BTF from the export conveyance systems when needed. The pumping plants themselves can also be isolated from the approaches. The Banks PP currently uses stop logs at the plant to perform pump maintenance. The Jones PP uses stop logs and/or a “floating bulkhead” to perform maintenance on the pumps and trash racks.

The addition of control structures capable of isolation function at existing approach channels enables the system to export water using one of two proposed operation scenarios described in Section 5: Isolated North Delta Operation or Isolated South Delta Operation. In order to export water under the third operational scenario, Potential Dual Source Operation, the control structures would require throttling gates capable of controlling flow schemes between BTF and the two export facilities. This complex operational scenario should be extensively explored in the next stages of design.

**Trash Racks:** Currently, the fish protection facilities on the approach channels to the Banks PP and Jones PP intercept most of the floating debris brought in through the existing conveyance systems. At the pumping plants themselves, trash racks (approximately 4-inch bar spacing at the Banks PP; 2.5-inch at the Jones PP) protect the pumps. Thus, a higher debris load is not expected due to the BTF connections downstream of the fish protection facilities.

### 14.3 Construction Methodology

For the most part, construction of BTF can take place while the existing conveyance system to the Banks PP and Jones PP remain operational. However, short duration shutdowns may be required during construction to tie new gate structures to the existing pumping plant approaches.

Both the IF and BTF sites possess similar infrastructure complexities, foundation characteristics, and construction periods to complete. Significant temporary construction zones are required for staging and storage. Construction of the embankments might include special considerations related to levee connections and flood protection (refer to Section 15.0, “Levees”).

See Section 24 for details regarding construction methodology, forebay components, and sequencing.
14.4 Maintenance Considerations

14.4.1 Floating Materials
It can be expected that the new BTF will experience problems with pond weed, just as the existing CCF currently experiences. The pond weed requires regular harvesting to maintain flow and forebay capacity. IF might also experience some pond weed problems, but the shorter residence time and more frequent level fluctuations could limit weed growth.

Pond weed and other floating debris currently accumulate on the fish protection facility trash racks upstream of both Banks PP and Jones PP. Automatic trash raking equipment and disposal facilities are necessary at both pumping plants once flows bypass the existing trash racks at the fish facilities.

14.4.2 Sediment Handling
The majority of easily settled sediments are removed at the sedimentation basins at each intake facility (see Section 7.0, “Intakes and Sedimentation Facilities”). The IF provides additional opportunity to settle sediment. It is anticipated that sediments in IF accumulate at an average annual rate of less than 0.10 feet per year. For a 50-year period, sediments accumulate to a depth of approximately 4.1 feet, which is less than one-half the height of the overflow weir at the outlet of IF. Remaining sediments passing through IF eventually are carried through the Main Tunnels to BTF. Additional opportunity exists at BTF to settle finer sediments, but given the upstream sediment removal and the larger storage available at the forebay, sediment accumulation at BTF is expected to be minimal over a 50-year period.

14.4.3 Embankment Maintenance
Maintenance requirements for the forebay embankments 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 the land-sides of the forebay embankments. Large rodents, such as muskrat and beaver, have been known to undermine similarly constructed embankments, causing embankment failure. Riprap slope protection on the water-side of the embankments requires periodic maintenance to monitor and repair any sloughing. Access is provided along the side of the new approach channel.

14.4.4 Structure Maintenance
Maintenance of forebay structures, including roller gates, radial gates, bulkhead gates, and stop logs, will be determined in the next stage of engineering.

14.4.5 Spillway and Stilling Basin Maintenance
Maintenance requirements for the spillway include the removal and disposal of any debris blocking the spillway crest or outlet channels. Debris in the stilling basin will also have to be removed to ensure normal water flow through the spillway to the receiving area. Any water spilled into the IF spillway containment area is pumped back into IF using temporary pumps provided by operations staff.
SECTION 15.0

Levees

This section describes the anticipated work associated with the levees adjacent to the intakes on the Sacramento River and at the South Delta facilities operated by DWR and Reclamation adjacent to the BTF. A portion of these levees are Federal Flood Control Project levees under the jurisdiction of the U.S. Army Corps of Engineers (USACE) and CVFPB, and specific requirements are applicable to work that will modify these levees. Also within the Delta are many levees that are not Federal Flood Control Project levees and are not under the jurisdiction of USACE and CVFPB. All work in-and-around these levees will be constructed in accordance with DWR and local Reclamation District requirements. All work on these levees will be done utilizing conventional cofferdam construction.

15.1 Configuration of Facilities

Construction of the three Sacramento River intakes will require significant modification to USACE levees at these locations. The intake structures are partially within the levee on the river side, and the gravity collector box conduits pass through the levee to the land-side facilities, including sedimentation basins. The finished elevation of the pad matches the levee crest elevation, thus providing a thickened levee section at the intake sites. The completed system constitutes a modification to the current levee configuration and provides features needed to maintain levee function.

The existing levees at the intake sites area traditional trapezoidal embankments with a levee crest width of approximately 30 feet and side slopes of approximately 2.5:1 to 3:1. The top elevation of the existing levees is approximately elevation 30 feet at each site. Highway 160 runs on top of the levee in each location. The levee side slopes extend to existing ground level on the land-side and nearly to the bottom flowline (thalweg) of the river on the water-side.

CVFPB and USACE regulate all facilities and activities within and adjacent to the levee prism. Furthermore, Caltrans will have specific requirements for modifying sections of Highway 160 at each location. The regulated portion of the levee prism extends below the adjacent ground surface or river bottom to depths below all planned facilities. The CVFPB and USACE also regulate all facilities and activities within 10 feet of the intersection of the levee side slope with the adjacent ground surface (i.e., 10 feet beyond the toe of the levee).

New facilities interfacing with the levee at each intake site include the following:

- Widened levee on the land-side to increase the crest width, facilitate intake construction, provide a pad for the sediment handling, and accommodate the Highway 160 realignment.
- On-bank intake structure.
- Large gravity collector box conduits behind the intake structure leading through the levee prism to the landside facilities.
- Cutoff walls (slurry and reinforced concrete diaphragm type).

Levee widening is done by placing low permeability levee fill material (in accordance with USACE specifications) on the land-side of the levee. The material is compacted in lifts and keyed into the existing levee and ground. The levee is widened by about 250 feet at each site.

Sand layers beneath the the existing levees and adjacent area are potentially liquefiable as discussed in Section 4.0, “Geologic and Seismic Considerations,” and ground improvement is required beneath the intake structure, gravity collector box conduits and portions of the pad fill area to mitigate the risk of liquefaction-induced settlement. Ground improvement is assumed to be accomplished by Cement Deep Soil Mixing (CDSM), although there will be further evaluation of the specific technique during future design activities. The widened levee allows construction of the intake cofferdam, the associated diaphragm wall and diaphragm wall tie-ins, and the centerline levee cutoff walls within the existing levee prism while preserving a robust levee section during
construction. It also allows Highway 160 to be permanently relocated approximately 220-feet toward the land-side and provides room for truck access from the highway to the intake structure.

The on-bank intake structure is constructed on the river-side of the levee, where the front screen wall is near the toe of the levee slope in the river. The complete intake structure is comprised of six 500-cfs screen bay groups, each feeding two large gravity collector box conduits located equidistant along the back wall of the intake structure that collect flow from the length of the bay group and convey that flow through the widened levee to the land-side facilities. Refer to Section 7.0 (Intakes and Sedimentation Facilities), for more information regarding the specific configuration of the intake structure.

The elevation of the top of the intake structure is 18 inches above the 200-year flood level (including Sea Level Rise - SLR), while the finished levee at the structures is either 3 feet or 3 feet plus wave run-up (5 feet at the intakes) above the 200-year flood level. At the upstream and downstream ends of the intake structure, a sheet pile training wall transitions from the concrete structure into the river-side of the levee in a manner similar to the Freeport Regional Water Authority of the FRWA intake shown in Section 7.0, “Intakes and Sedimentation Facilities.” Riprap is placed on the levee-side slope upstream and downstream of the structure to prevent erosion from anomalies in the river created by the structure. Riprap is also placed along the face of the structure at the river bottom to resist scour.

The intake structure is constructed inside a cofferdam system installed within the levee prism on the river-side. The intake structure has a foundation that uses a combination of improved ground and steel-cased drilled piers. Sand layers beneath the existing levees are potentially liquefiable, and ground improvement is required to maintain lateral resistance provided by the deep foundation system. Ground improvement below the intake structure is assumed to be accomplished by CDSM.

The cofferdam extends from the back wall located approximately 10 feet into the levee crest to approximately 10 feet beyond the face of the intake structure in the river. The back wall of the cofferdam along the levee crest is a deep slurry diaphragm cutoff wall designed for dual duty as a structural component of the cofferdam and to minimize seepage through and under the levee at the facility site. The diaphragm wall extends along the levee crest upstream and downstream of the cofferdam and the fill pad for the sedimentation basin on the land-side, which will allow for a future tie-in with levee seepage cutoffs that are not part of this project. The other three sides of the cofferdam system are expected to be sheet pile walls.

The cofferdam includes a tremie concrete seal in the bottom to aid dewatering and constructability within the enclosed work area. Ground improvement and installation of drilled piers (beneath the intake) are within the cofferdam prior to placing the tremie seal. The temporary cofferdam and permanent intake structure components are designed for minimal displacement to maintain levee stability under earthquake loads.

From the land-side of the diaphragm wall, the large gravity collector box conduits pass through the levee prism to the sedimentation basins. These collector box conduits are expected to be constructed by open-cut methods after the intake portion of the cofferdam is backfilled. Other installation methods or re-sequencing of the construction of the box conduits between the intake cofferdam and the sediment basins might also be considered during final design. Backfill above the box conduits and reconstruction of the disturbed portion of the levee prism is accomplished using low-permeability levee material in accordance with USACE specifications.

In conjunction with the diaphragm wall described above, a slurry cutoff wall (soil, bentonite, and cement slurry) is constructed around the perimeter of the construction area for the land-side facilities. This slurry wall is tied into the diaphragm wall at the levee by short sections of diaphragm wall perpendicular to the levee. The slurry cutoff wall overlaps for approximately 150 feet along the diaphragm wall at the points of tie-in. The slurry wall is intended to help prevent river water from seeping through or under the levee during periods when deep excavations and associated dewatering are required on the land-side. By using the slurry wall in conjunction with the diaphragm wall, the open cut excavation portion of the work on the land-side is completely surrounded by cutoff walls. These walls minimize induced seepage from the river through the levee, both at the site and immediately adjacent to the site, and serve as long-term seepage control behind the levee.
See Section 24 for additional detail regarding sequence of levee construction, and temporary and long-term flood protection features. Additional detail can be found in Appendix B, “General Description of Conceptual Level Construction Sequencing at the CWF Intake Facilities”. Additional details of the pad fill arrangement can be found in Sections 7.0, “Intakes and Sedimentation Facilities,” and Section 14, “Forebays.”
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SECTION 16.0
Channel Enlargement Measurements

No channel enlargements are proposed for the WaterFix BTO.
Operable Barrier – Head of Old River

17.1 Physical/Structural Component

The WaterFix BTO project involves constructing a Head of Old River operable barrier with control gates to reduce migration of San Joaquin River watershed salmonids into the South Delta through the Old River. The barrier is located where the San Joaquin River and Old River diverge (Figure 17-1), at −121.328513 latitude, 37.808166 longitude.

The barrier (Figures 17-2) is approximately 210 feet long and 30 feet wide, with top elevation of 15 feet msl (NAVD 88). It consists of five independent 125-foot bottom-hinged gates, with a fish passage structure, boat lock with gates at each end, control building, boat lock operator’s building, and communications antenna, as well as floating and pile-supported warning signs, water level recorders, and navigation lights.

17.2 Operable Barrier Construction Methods

There are two potential methods of constructing the operable barrier: (1) Cofferdam construction in a dewatered construction area; and (2) In-the-Wet construction, which eliminates the time, material, and cost of constructing a cofferdam. All in-water work, including installation of containment systems, occurs between August 1 and November 30 to minimize effects on delta smelt and juvenile salmonids. All other construction is throughout the year from a barge or from the levee crown. Any work performed in the channel after November 30 has to be within a cofferdam, silt curtain or similar containment system. The operable barrier is adjacent to the existing temporary barrier, which will continue to be installed until the permanent barrier is operable. The construction window for in-channel activities would vary for each method, as outlined below.

17.2.1 Cofferdam Construction

The cofferdam construction method is in two phases and allows in-water work to continue through the winter. In the first phase, cofferdam construction begins in August and lasts approximately 35 days, and the dewatered area is the project construction site for half of the operable barrier into the adjacent levee. The cofferdam is either removed or cut off at the required invert depth. In the second phase, the remaining half of the operable barrier is constructed using the same methods and incorporated into the final barrier layout, with the cofferdam either removed or cut off. Depending upon weather and river flow conditions, construction within the cofferdam continues until early November or throughout the winter.

17.2.2 In-the-Wet Construction

This method involves working within the existing channel flow, with no levee relocation. The channel invert is excavated to grade using a sealed clamshell bucket excavator working off the levee or from a barge. H-piles are placed in the channel and within these piles, gravel and tremie concrete is placed for the foundation. Reinforced concrete structures are then either floated in or cast in place using prefabricated forms on top of the gravel, tremie concrete, and H-piles. Divers complete the connections between the concrete structures and the piles. This is all in-water work occurring between August 1 and November 30. Construction of all other components would take place from a barge or from the levee crown and would occur throughout the year.

17.3 Design and Construction Detail

The operable barrier is within the confines of the existing channel, with no levee relocation. To ensure the stability of the levee, a sheet pile retaining wall is installed in the levee where the operable barrier connects to it. Construction occurs in two phases. The first phase includes construction of half of the operable barrier, masonry control building, operator’s building, and boat lock. The control building houses the emergency generator, control panels for the control gates, circuit breakers, and storage area for operation and maintenance equipment. The
second phase includes construction of the second half of the operable barrier, the equipment storage area and the remaining fixtures, including a communications antenna and fish passage structure. The construction period is estimated to be up to 32 months, with a maximum construction crew of 80 people.

The complete operable barrier requires approximately 1,500 cubic yards of concrete. A permanent storage area, 180 by 60 feet (10,800 square feet), is for equipment and operator parking. Access is controlled with fencing and gates. A communications antenna and a propane tank for emergency power are in the plan.

The operable barrier has a boat lock for public use. A small masonry operator’s building adjacent to the control building houses the controls for the boat lock gates and has observation windows with an unimpaired view of the boat lock. The boat lock is 20 feet wide and 130 feet long and is constructed using sheet piles and 20-foot wide and 10-foot high bottom-hinged gates on each end. Each bottom-hinged gate weighs approximately 8 tons and is opened and closed using an air-inflated bladder. The invert of the lock is -8.0 feet msl, with the top of the lock wall at 15 feet. A valve system using a 36-inch valve balances water levels for transferring boats upstream and downstream. The lock has floating boat docks for temporary mooring, navigation signs and lights, warning signs, and video surveillance.

The fish passage structure is designed according to guidelines established by NOAA Fisheries, USFWS, and CDFW for several species, including salmon, steelhead and green sturgeon. The structure will be reinforced concrete and approximately 40 feet long and 10 feet wide. With historical maximum head differential across the gate at 4 feet, the fish passage structure has four sets of vertical slot baffles with a 1-foot-maximum head differential across each set. An equal head drop through each set of baffles is self-regulated, without mechanical adjustments and independent of upstream and downstream water surface elevations.

Construction workers and equipment access the project from the south on Cohen Road, a county road. A construction staging area (approximately 10,000 square feet) is on the south side of Old River, just outside the levee roads. For maintenance access, an existing private road north of the fish control gate will be improved with two miles of gravel to a minimum 16-foot width to accommodate cranes and loaded 10-wheel trucks. The road begins at the end of Undine Road to the San Joaquin River levee and continues southwest along the levee to the gate site.

For slope protection, approximately 11,000 square feet (450 linear feet) of riprap is placed near the operable barrier and on the channel bottom, with fine materials such as sand adjacent to the riprap, creating a smooth slope from the channel bottom to the gate sill.

### 17.4 Dredging

The site at the head of Old River would be dredged to construct the operable barrier.

#### 17.4.1 Gate Dredging

Dredging up to 150 feet upstream and 350 feet downstream from the site is necessary to clear the area for construction and placement of the fish control gate. In total, up to 1,500 cubic yards of material is dredged. Dredging occurs between August 1 and November 30, lasting approximately 15 days. A purchased 50,000-square-foot area adjacent to the operable barrier site is used as a runoff management basin for both initial and maintenance dredging (described below).

#### 17.4.2 Hydraulic Dredging Method

The hydraulic dredging method siphons water-sediment slurry (4 parts water for every 1 part sediment) from the bottom of a channel and deposits it into a settling pond to dry. Hydraulic dredging is used where there are large areas to be dredged, the concern for induced turbidity and harm to benthic vegetation is great, and there is ample area available for settling ponds. It is relatively expensive but allows options in disposal sites inexpensively using flexible piping extending from the settling pond to the dredge area.

Hydraulic dredges operate 24 hours a day, 7 days a week, until dredging is complete. A pipe lowered from the dredging barge into the bottom sediment siphons water-sediment slurry into flexible piping that can effectively
extend 3,000 feet up or down the channel. This piping is weighted down to avoid interference with boat navigation. When the flexible pipe reaches the levee, it is attached to a semi-permanent, stationary pipe braced to the waterside of the levee and extending across the top and down the landside of the levee into the primary basin of a settling pond. The stationary pipe ranges from 8 to 18 inches in diameter and requires a gravel ramp over it for vehicle and agricultural equipment crossing. Direct deposition of the material into settling ponds allows uninterrupted dredging to the capacity of the settling pond. Up to 5,000 cubic yards of material can also be transported to settling ponds by barges. Adjacent to the channels and constructed of local compacted soils, the settling ponds decant the sediment from the water so that the dried material can be beneficially re-used.

Settling ponds are divided into primary, secondary, and return basins. The primary and secondary basins settle sediments out of the dredged slurry. Water reaching the return basin has most of the suspended sediment settled out and is pumped back into the channel subject to USACE and RWQCB discharge requirements. It takes between 24 and 36 days for sediment to settle out of the water. As water moves from the primary to the secondary basins, the primary basin is available for more dredged material.

Absolute capacity of a single pond is determined by the rates of sediments settling, water pumping from the return basin, and dredging. The pond is reused or left to dry. Dried material is then harvested for use.

### 17.4.3 Sealed Clamshell Dredging Method

Clamshell dredging is from either a barge or from the top of a levee, depending on restrictions based on vegetation on channel banks and the width of the channel. Barge clamshell dredges are maneuvered within the channel by tugboat. From a barge, the bucket assembly, attached to a boom of 100 feet maximum, is lowered into the channel to collect sediments. It scoops a maximum of 5 cubic yards of 50% water-sediment slurry for deposit into either a runoff management basin adjacent to the channel or into a barge for transport to a basin located farther away. From atop the levee, the clamshell dredge puts the same load of dredged material into a runoff management basin.

A runoff management basin is typically rectangular and uses the levee as one of its walls. The remaining three walls, constructed of compacted local soil, are a maximum 6 feet high. Runoff management basins contain slurry and prevent drainage into agricultural ditches and channels. Depending on climate and thickness of spread, the slurry reaches 25% moisture content in 2 to 6 weeks. At 25% or less, it can be used for levee reinforcement or agricultural soil supplement.

The clamshell dredging method is more cost-efficient than the hydraulic method, but it can cause greater disruption to channel vegetation when the bucket scrapes sediments from the channel bottom. This method is used where space is limited for settling ponds, disruption to vegetation and other organisms is minimal, dredging area is small, there are channel islands, or temporary turbidity and sedimentation in the water is not an issue. Turbidity can be reduced through the practices of slowly lowering and raising the clamshell bucket and using a closed bucket.

### 17.4.4 Disposal of Dredged Materials

Either method allows the dried dredged material to be used in the south Delta. According to “Environmental Study of Dredged Materials Grant Line Canal” (DWR, 2000), physical measurements, chemical analyses, and other tests indicated dried dredged material would be suitable for most uses, including levee stabilization, upland or agricultural applications. Gross sediment contamination was not present, and any contaminating constituents such as heavy metals were found at levels well below applicable regulatory limits. Dredged material impact on farmland was confined to particular areas within Grant Line and Fabian Bell Canals, so this will continue to be monitored.

Approximately 5% of all the dredged material will be used for 1-foot deep landside levee reinforcement. Levee areas with vegetation would not be reinforced to avoid negative impacts on sensitive vegetation and wildlife. The remaining 95% of the dredged material would be spread 1 foot deep over agricultural land to improve the quality of the existing soil.
17.5 Maintenance

The operable barrier will be owned, operated, and maintained by DWR. Maintenance of the bottom-hinged gates is every 5 to 10 years. Annual maintenance of the motors, compressors, and control systems requires a service truck. Maintenance dredging with a sealed clamshell dredge around the bottom-hinged gates to clear out sediment deposits will be performed as needed. Depending on the rate of sedimentation, need of maintenance dredging is estimated as every 3 to 5 years, removing no more than 25% of the original dredged amount. Maintenance dredging will be done between August 1 and November 30 and will not last longer than 30 days. Spoils will be dried in the areas adjacent to the operable barrier site. A detailed dredging plan on specific maintenance dredging activities will be developed.
Figure 17-1  Fish Control Gate Location
Figure 17-2: Head of Old River Gate, Artist’s Rendering (Looking Northward)
SECTION 18.0
Controls and Communications

18.1 Communications

A Supervisory Control and Data Acquisition (SCADA) system will be used for the main communications, control, and monitoring of the WaterFix BTO facilities. The new system will be used with the existing system to offer redundant paths for reliability.

Administrative and operational sites of the WaterFix BTO that communicate using the SCADA system include:

- Intake structures (Sites 2, 3, and 5)
- Sedimentation basins (Sites 2, 3, and 5)
- IF inlet and outlet structures
- BTF Pumping Plant
- Channel Control Structures
- Main electrical substation (location to be determined)
- Tunnel shaft sites selected for instrumentation
- DWR, Reclamation, SWP, and CVP operation facilities.

Each facility at these sites includes control and monitoring equipment that communicates with the Joint Operations Center (JOC) in Sacramento, as well as with the Area Control Center (location to be determined).

Currently, SCADA is to be implemented using some combination of fiber optic cable system(s), microwave radio, and/or leased telecommunications lines. This communications system connects to the Delta Field Division O&M Center at the south end of the project and to DWR headquarters (1416 9th Street, Sacramento) at the north end of the project. The existing communications system will also be expanded to provide project communications to the JOC and the Area Control Center (location to be determined).

Communications is provided from the northernmost facilities to the southernmost facilities and to the DWR headquarters via fiber optic cable buried in conduit, microwave radio, or leased lines from a telecommunications provider. The leased lines from a telecommunications provider(s) will be the primary communications media between facilities, with microwave and fiber as the secondary communications media (see the network architecture diagram for proposed primary and secondary communications media between facilities). The feasibility of the fiber optic communications media will be assessed during subsequent design phases.

Considerations for the use of fiber optic cable vs. microwave include:

- If fiber optic cable in conduit is used for this segment, the conduit route will run adjacent to roads, highways, railroads, utilities, or other easements.
- If microwave radio is used for this segment, parabolic antennas will be on the roof of DWR headquarters and all the rest of the facilities. The antenna at DWR headquarters will be mounted directly on the building roof, adjacent to existing antennas. The antenna at each of the other facilities will be installed on a new antenna-mounting structure at each site. A radio propagation study will determine the structure height. Preliminary investigation indicates the structure will be approximately 50 feet higher than the finished grade at the site. The feasibility of a microwave radio link will be determined after determining availability of frequencies that can be licensed by the Federal Communications Commission, performing a path propagation study, and investigating if any new building construction is planned in the intended path.

A buried fiber optic conduit is proposed to run from the south end of the new tunnel at the BTF pumping plant to the Delta Field Division O&M Center. The conduit will run adjacent to BTF, along the approach canal to Banks PP,
then to the Administration Building at the Delta Field Division O&M Center. The fiber optic conduit between the intake facilities and BTF pumping plant could be buried in the tunnel ROW or installed inside the tunnel itself. The location will be determined during preliminary engineering and final design.

A global positioning satellite (GPS)-based time clock is at the BTF pumping plant to support the control system and provide time-reference data. It requires a small dish antenna mounted on the roof or nearby the pumping plant. In addition, satellite-based clocks will be used to support communications.

SCADA will be further defined during preliminary engineering and final design. Strategies to prevent hacking or other unintended access by third parties will be incorporated into the design to provide a secure system. Redundant communications pathways using telephone lines, radio or microwave signals, and other technologies will be considered. Reliability features, such as uninterruptible power supplies and backup computer systems, will also be considered.

18.2 Control

All equipment is operated in local and/or remote control modes. In the local mode, the equipment is manually controlled at the equipment or from a nearby MCC, switchgear, VFD, local control panel, valve actuator, or hand station. When equipment is in local mode, all remote-mode control of the equipment is disabled. In the remote mode, equipment is controlled through a PLC based on automatic control strategies, with commands issued from any authorized plant control system workstation located at the facility, or commands issued from the workstations at the Area Control Center or Project Operations Center (within the JOC) through the SCADA system.

The control mode is selectable, where applicable, based on local/remote switches located at the field equipment or local control panels. Selector switch position feedback is wired to a PLC, allowing an operator using the operator workstation to know if control from the operator workstation is active. Some non-process equipment (e.g., sump pumps and HVAC equipment) are provided with local manual controls only.

18.3 Maintenance Considerations

The existing SWP is operated with a common controls and communications system. To maintain the common operational platform, SCADA is an extension of the existing system, and the extended system will be maintained in the same manner as the existing system.

18.4 System Control Description & Design Criteria

The proposed Bay Delta Conveyance System is comprised of three Intake Fish Screen Facilities and associated Sedimentation Basins, Tunnels, a Junction Structure, the Intermediate Forebay, and numerous supporting facilities. Water from the Sacramento River will gravity flow through fish screens into the sedimentation basins, then into deep tunnels that lead to the Intermediate Forebay. From the Forebay, the water flows by gravity through two 40-foot diameter tunnels to the Byron Tract Forebay Pumping Plant wet well, where it is pumped to Byron Tract Forebay. Under certain hydraulic conditions, water can bypass the pumps and gravity flow to Byron Tract Forebay.

18.4.1 Intake Fish Screens

Water entering the Intake Fish Screens is limited to an approach velocity of 0.2 ft/sec and a maximum flow of 3,000 cfs per intake. From the fish screens, the flows are sent via box conduits to an unlined earthen sedimentation basin. An earthen berm runs the length of and divides the basin into two equal halves. Flow control is provided by 8-foot x 8-foot control gates and an influent flow meter located in the box conduits. The control gates will modulate flow based on the flow meter output to maintain a velocity through the fish screens not to exceed 0.2 ft/sec and through each sedimentation basin not to exceed 1,500 cfs. The isolation gates are provided at each end of the box conduits for cleaning/maintenance and to isolate the intake from the river during floods.


18.4.2 Junction Structure

Intakes 2 and 3 flow into a common Junction Structure that has 8 isolation gates and a level transmitter. The flows from these two intakes are combined and conveyed to the Intermediate Forebay. Flow from Intake 5 bypasses the Junction Structure and flows directly to the Intermediate Forebay.

18.4.2.1 Intermediate Forebay

The Intermediate Forebay serves four main purposes: surge protection; storage buffer between the Intakes/River and the pump station; tie the northern tunnels with the southern tunnels; and balance flows at the outlet between the two southern tunnels. As a storage buffer, the forebay allows the pumps to have time to ramp up and down while delivering flow to the pump suction while the intake gates are adjusted to provide steady flow from the Sacramento River.

18.4.2.2 Byron Tract Forebay Pumping Plants

The Byron Tract Pumping Plants are two pump stations with wet well structures. Each pumping plant/wet well structure contains 4 high flow pumps (1,125 cfs) and 2 low flow pumps (563 cfs). Each pump is equipped with a siphon discharge, which releases the pumped water to Byron Tract Forebay. Since the pump discharge is a siphon design, the pump wet well must not exceed elevation +16 feet. A level sensor at the wet well will automatically close all intake isolation gates to keep the pumps from reverse rotation (turbine).

18.4.3 System Operation

The system will have 3 modes of operation: Gravity Flow, Pumped Flow and System Shutdown. The modes will operate as follows:

- **Gravity Flow** – In this mode, given certain hydraulic conditions (including Sacramento River elevation and Byron Tract Forebay elevation), the water can flow by gravity to Byron Tract Forebay. The ability to operate in gravity flow mode is dependent upon the final BTF operating levels. To initiate gravity flow mode, the operator will set the target flow at an operator workstation/HMI and initiate the system operations. The control gates at the intake sites will be adjusted to provide flow balance between intake sites and maintain the target flows in the sedimentation basins (see the discussion of intake control gates, below). Flows and levels will be monitored and the weir control gates in each pump shaft will be adjusted to maintain the target flow.

- **Pumped Flow** – Prior to initiating pumped flow, the operators will determine the number of pumps to operate (and at what speed, if VFDs are used), using an optimized system efficiency chart. The number of intakes and sedimentation basins to be operational will also be established before initiating start-up. The operators will manually start a single pump and operate it at full speed to prime the siphon. The siphon-breaker valve will close after a pre-set and adjustable time, and the siphon will prime. For variable speed operation, the pump will then be adjusted to the target speed. When positive flow is detected through the system at the intake site, the pre-determined number of intake control gates will be opened (additional discussion of the intake gate operations is presented below). The operator will then repeat the pump start-up sequences, maintaining a pre-determined and adjustable amount of time between each pump start (expected to be on the order of 5 minutes) until the desired quantity of pumps are operating. The system flowmeters will be monitored to confirm maintenance of the target flow. If variable frequency drives are incorporated, the pump speed will automatically adjust to maintain the target flow through the system.

A secondary safety control and permissive condition will be through level control. In this secondary control mode, the system will continuously monitor the Intermediate Forebay and River elevations to verify that sufficient elevation/flow is available for the flow control to be maintained.

- **System Shutdown** – Operators will manually shut down each pump, allowing a pre-determined time between each shutdown (expected to be on the order of 5 minutes). At each pump shutdown, the vacuum breaker valve will open to break siphon prime. The flow control gates at the intake sites will be managed as discussed in the following paragraphs.
The flow control gates at the 3 intake sites can all be closed at system shutdown or the gates at 2 of the 3 intake sites can be closed. Leaving gates open at more than one intake site when the system is not conveying flow to BTF would cause water to continuously circulate into one intake and out another, which should be avoided.

If the control gates at one of the sites remain open, the water level in the system will rise and fall with the changes in river level. This approach facilitates start-up of the Gravity or Pumped operation mode without equalizing the water surface between the river and the tunnel system. For this scenario, the pre-determined number of intakes and control gates will be opened upon initiating equipment start-up. The number of intakes and control gates will be selected such that the through-screen velocity at the intakes does not exceed 0.2 fps.

If the flow control gates at all of the intake sites are closed prior to system start-up, it will be necessary to confirm that the water levels between the river and the tunnel system are equalized prior to opening the flow control gates. The following 3 scenarios will need to be addressed to equalize the water level in the system prior to initiation of either gravity or pumped modes:

- **River Level Higher than Tunnel System** – When the water level in the river is higher than the tunnel system, the flow control gates will be partially opened prior to system startup to equalize the system. The gates will be modulated based on the flow measurements to maintain less than 500 cfs through each sedimentation basin. All of the control gates to be put into service can be modulated to rapidly equalize the system. Upon water level equalization within a pre-determined tolerance, the system start-up process can be initiated.

- **Tunnel System Level Higher than River Level** - When the water level in the tunnel system is higher than the river level, the flow control gates will remain closed while the first pump is started to draw down the water level in the tunnel system. Depending upon the differential between the tunnel and the river, more than one pump may be utilized to equalize the system. Upon equalization of the system water level within a pre-determined tolerance, the flow control gates will be opened and the system startup will proceed until the desired numbers of pumps are operational.

- **River and Tunnel System Water Level Equal within Pre-Determined Tolerance** – When the water level between the river and the tunnel system is equal within a pre-determined tolerance, the control gates at one of the intake sites will be opened and pump initiation will proceed according to the start-up protocol when the gates are left open at one intake site.

The flow at each intake site will be continuously monitored to provide information for modulation/throttling of the flow control gates. The flow control gates will limit the flow through a single sedimentation basin to 1,500 cfs to prevent exceeding the through-screen velocity criteria of 0.2 fps. Modulation of the flow control gates in this manner will limit the total flow from a single intake site to 3,000 cfs.

### 18.4.4 System Monitoring

The control system shall have several critical monitoring points to verify that the water levels and flows are sufficient to maintain pumping capacity. The critical monitoring points shall be:

- Elevation of the Sacramento River (at each Intake)
- Intake 2, 3 and 5 individual influent flow meters
- Intake 2, 3 and 5 gate positions
- Intermediate Forebay elevation level
- Main 40-foot tunnel flow meter
- Pump station discharge flow meter
- Byron Tract Forebay elevation.
- Connection Channel flow meters
For hydraulic surges, the control system shall monitor the level at several locations. The critical level monitoring points shall be:

- Level at Junction Structure
- Level at Intermediate Forebay
- Level at Byron Tract Forebay.

18.4.5 Failure Analysis and System Stress

Two extreme failure scenarios were considered and studied:

- Failure at one of the intakes where the flow of one intake is suddenly reduced to zero. In this scenario, the Intake Flow Control Gates at the other two intakes shall start closing to maintain the flow below 0.2 fps. In conjunction, the pumping shall be ramped down to stabilize the flow.

- Power failure where all pumps are tripped OFF. In this scenario, all gate positions are maintained, and surge levels and pressures are monitored.
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SECTION 19.0

Power Supply and Grid Connections

Electrical power is required for the construction and operation of the conveyance system, and electrical transmission corridors are required to transport that electrical power to both permanent and temporary (construction) sites.

19.1 Power Demand

The total peak construction electrical load is approximately 242 MVA. The peak intake pumping demand during operation of the system is estimated at approximately 60 megavolt-amperes (MVA). The construction electrical power demand for the main dual-bore tunnel system includes four dual-bore drive shafts (47+ MVA each), two intermediate shaft sites (2.2 MVA each), and a reception shaft (3.4 MVA). For the North Tunnel system between IF and Intakes No. 2, 3, and 5, the construction electrical power demand is at the IF drive shafts (12 MVA and 23.7 MVA for two single-bore drives of 28 and 40 feet diameter tunnels respectively), the Intake No. 2 drive shaft (10.5 MVA), two intermediate/vent shafts (1.1 MVA each), and a junction structure (2.0 MVA).

The tunnel alignment and loads for both the North Tunnels and Main Tunnels (and various shaft locations) are illustrated in Figure 19-1. Table 19-1 summarizes the peak construction power electrical loads.

<table>
<thead>
<tr>
<th>WaterFix BTO Component</th>
<th>MVA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Tunnel Drive Shaft Reach 4 (IF - Staten)</td>
<td>47.7</td>
</tr>
<tr>
<td>Main Tunnel Drive Shaft Reach 5 (Bouldin - Staten)</td>
<td>41.8</td>
</tr>
<tr>
<td>Main Tunnel Drive Shaft Reach 6 (Bouldin - Bacon)</td>
<td>47.1</td>
</tr>
<tr>
<td>Main Tunnel Drive Shaft Reach 7 (BTF - Bacon)</td>
<td>47.1</td>
</tr>
<tr>
<td>Main Intermediate/Vent Shaft E/W (Mandeville)</td>
<td>2.2</td>
</tr>
<tr>
<td>Main Intermediate/vent Shaft E/W (Victoria)</td>
<td>2.2</td>
</tr>
<tr>
<td>Main Reception Shaft (Bacon)</td>
<td>3.4</td>
</tr>
<tr>
<td>North Tunnel Drive Shaft Reach 1 (Intake No.2 – Junction Structure)</td>
<td>10.5</td>
</tr>
<tr>
<td>North Tunnel Drive Shaft Reach 2 (IF – Junction Structure)</td>
<td>23.7</td>
</tr>
<tr>
<td>North Tunnel Drive Shaft Reach 3 (IF - Intake No. 5)</td>
<td>12.0</td>
</tr>
<tr>
<td>North Tunnels (29’ and 20’)Intermediate/Vent Shafts</td>
<td>2.2</td>
</tr>
<tr>
<td>North Tunnel Junction Structure</td>
<td>2.0</td>
</tr>
<tr>
<td>South Tunnel Drive Shaft Reach 9 (BTF Outlet - South Tunnel Outlet)</td>
<td>TBD</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>242.0</strong></td>
</tr>
</tbody>
</table>

Notes: Refer to Section 11, Figure 11-1 for Reach designations.

BTF/IF = Byron Tract Forebay /Intermediate Forebay
E/W = East Tunnel Bore/West Tunnel Bore
MVA = megavolt-amperes
Figure 19-1: Tunnel Construction Power Requirements
19.2 Power Supply and Transmission Corridor Options

Two electric utility transmission service providers will provide transmission interconnection and services to deliver electrical power to the project: SMUD and WAPA. There are multiple interconnection options available to connect the project to the electrical grid and supply both the operation and construction electrical power.

Preliminary studies show that some reinforcements and upgrades to the existing transmission grid would be needed to accommodate the large construction power requirements for this project. Because the service construction locations are spread over a distance of more than 40 miles, interconnection to more than one electric utility transmission service provider is possible and needs to be closely coordinated with the utilities. Figure 19-2 shows all of the electrical transmission corridors currently under consideration for interim construction and permanent electrical power. Future system impact studies by the utility providers will determine which corridors will be used for the project. The system impact studies will also evaluate the viability of using some of the temporary construction transmission connections for permanent operations power and any upgrades or expansion required to the utility providers’ existing infrastructure to meet the additional electrical loads to their system(s).

It is anticipated that the utility interconnection facilities needed to connect the project to the electrical grid and the electrical power needed for almost all of the conveyance facilities (the largest electrical demand is for operating the TBM) would be procured in time to support construction and operation of the facilities. However, it is possible that utility interconnection facilities and power might not be available in time to support critical path activities, particularly shaft pad construction and shaft sinking work that precedes the tunnel construction work. Therefore, the interim use of onsite generation as the power source for shaft sinking activities is possible. After construction of the temporary (or permanent, in some cases) utility interconnection facilities and procurement of power is completed, electricity from the interim onsite generators would no longer be used. As noted in Table 19-1, the electrical power demand for the dual tunnel shafts ranges from 2.2 to 3.4 MVA per site.

DWR SWP Power and Risk Office (PARO) leads the process of identifying, evaluating, and establishing the electrical interconnection of this project to the California electric grid. PARO also leads the process of planning and obtaining the power needed to construct the project. The power needed for long-term operation of the project would be based on the power portfolios of CVP and SWP in proportion to their participation in the project. The energy needed for CVP’s portion of the project would be from existing CVP hydroelectric generation in accordance with the federal statutes that created the CVP. The energy needed for SWP’s portion of the project would be in accordance with DWR’s procurement practices and the 2012 Greenhouse Gas Reduction Plan (GHGRP).

The SWP GHGRP is a roadmap for reducing the SWP’s power portfolio greenhouse gas (GHG) emissions to 50 percent below 1990 levels by 2020 and to 80 percent below 1990 levels by 2050. The SWP portfolio includes a mix of the SWP’s large and small hydroelectric generation, long-term power purchase contracts for renewable energy generation, and power purchase contracts for energy generation, including generation from state-of-the-art combined cycle natural gas power plants and the California Independent System Operator’s short-term power market. The proportion of renewable energy in the SWP portfolio is updated periodically to ensure the GHGRP is followed and GHG reduction targets are met. The main power supply and transmission corridor options are summarized in the following sections.
Figure 19-2  Electric Power Grid Connection Plan
19.2.1 Preliminary Interconnection Options

WAPA and SMUD have existing transmission facilities near the project area that offer one or more interconnection options. Preliminary facilities are described below. Although each option offers different points of interconnection, there are also some common features in them. For example, it is anticipated that for each option:

- A new temporary substation would be constructed at each of the drive/launch shaft locations.
- Lower voltage sub-transmission lines would be used to power intermediate and reception shaft sites between the main drive shafts.
- A new substation would be constructed near the IF to support temporary construction load.

To serve permanent loads at the pumping plant located by the Byron Tract area, a new transmission line would be extended from the existing Tracy Substation to a new substation by the pumping plant area, where electrical power would be provided to the tunnel shaft areas for construction purposes and to appropriate bus voltage for utilization by pumps. To the extent possible, this temporary power for tunnel construction will be repurposed as permanent power for pumping plant operation. In addition, whenever such facilities built to serve construction are repurposed to serve permanent operation, an evaluation would be done to appropriately resize these facility ratings where needed. Further description of the electrical system at individual permanent facilities can be found in their corresponding sections.

For operation of the three intake facilities located by the Sacramento River and of the intermediate forebay facilities, existing distribution lines would be used wherever practical, which minimizes ROW issues associated with new higher voltage lines. However, if existing distribution lines cannot support the intake operation, there may be a need for a new 69 kV transmission line to serve intake operation. As such, electrical power would be transformed from 69 kV to 480V service, or appropriate equipment terminal voltage, for distribution and use for gate operation, lighting, and auxiliary equipment at the adjacent structures. These newly constructed transmission lines, distribution lines and substations would be owned either by the utility or the project.

Existing SMUD and WAPA transmission facilities near the northeast (IF) and south (BTF) ends of the project provide potential points of interconnection for the project. At the north end, a new transmission line at 69kV would extend from a SMUD substation to new substations at the IF, to serve both the North Tunnel and Main Tunnel construction loads, and to a new substation north toward the intakes as needed to support construction sites along the northern tunnels. At the south end, the project would connect to an existing WAPA 230 kV substation south of the existing CCF. From this substation, a new transmission line would extend north toward the pumping plant to new substations to serve both temporary construction and permanent loads. From the new substation, a new transmission line would continue to extend north toward and along the main conveyance system alignment to Bouldin Island to support construction at sites north of BTF. Lower voltage lines would be used to power intermediate and reception shaft sites between the main drive shafts.

19.3 Construction Methodology

Construction duration of both the single-bore and dual-bore tunnels is estimated to be in the range of 6 to 9 years, which does not include the construction and removal of new transmission lines and system upgrades such as reconductoring. Construction power demand for this time period is greater than the permanent demand. The EIR/EIS, ROW, and land acquisition processes required by this project are all critical path items. Power and utilities work is done to minimize environmental footprint by co-locating transmission corridors and using existing power infrastructure where practical.

The methods associated with the construction of electrical transmission lines cannot be fully evaluated until a utility provider or providers and transmission corridors are selected. To minimize disruption along some the corridors, the transmission towers might be transported to their locations by helicopter. It is anticipated that the construction issues related to electrical transmission corridor construction will be evaluated in more detail by the EIR/EIS team after completion of additional studies.
19.4 Grid Interconnection Reliability

The electrical power supply for the construction of this project will be obtained through more than one interconnection point in order to reduce the amount of loading at any one localized area. As is the case for all load served off the transmission grid, electrical power supply to the project is at some risk from potential transmission line failures or service interruptions, as well as local electric utility outages. The cost of a redundant power or transmission system to improve reliability must be weighed against both the likelihood of an interruption or an outage and the consequences of such events.

As an example, for 230 kV transmission lines, the typical outage frequency (North American Electric Reliability Corporation 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 circuit and at one point of interconnection to the utility grid, the project could expect to lose its power supply for duration of 1.35 days once every 7 years. If all of the construction power at approximately 242 MVA is to be served through one point of interconnection on a single circuit, and if this peak simultaneous load occurs for approximately 4 years during construction of the main tunnel, the probability of an outage during this period would be 18.5 hours of construction downtime.

To reduce the chances of losing electrical power to the permanent project facilities, a more reliable configuration could include serving the project with a second circuit interconnecting to the grid at either the same interconnection point as the first circuit and sharing the same route or at a different point, extending to a second utility substation along a different route than the first transmission line (to mitigate the chances of a single disaster damaging both lines). However, given the availability of existing high voltage lines and substations, a second, independent circuit or interconnection would likely more than double the cost. While the reliability of water delivery is important, a short term disruption in transmission service that is within industry reliability standards should not jeopardize the overall long-term quality of water delivered through the project and the protection of species and habitat in the Delta. Thus, a single line interconnection may be sufficient to meet project needs unless system impact studies indicate a decrease in reliability substantially greater than discussed above or operational studies indicate otherwise. The current design concept provided for only a single line interconnection to the WaterFix BTO pump plants. Provisions of a second incoming electrical service may be studied in future engineering efforts.
Clifton Court Forebay (CCF), the existing SWP forebay facility, receives water from the Delta and supplies it through the Skinner Fish Facility on the Intake Channel to the SWP Banks PP. The proposed WaterFix BTO does not affect the operation of this existing fish facility.
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SECTION 21.0

Borrow Sites

This section identifies general areas that are suitable 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 which can be practically mined and from which the material can be readily transported to the construction sites. Borrow materials are required for forebay embankments at IF and the Byron Tract Forebay, site fill at Intake Facilities, fill pads at tunnel shaft sites, in-river rock slope protection (RSP), and haul roads. The primary borrow material needs to be soil suitable for engineered embankment fill. Rock, gravel, and sand are also required.

At this point, there is insufficient geotechnical information to fully assess the suitability of borrow areas near the WaterFix BTO alignment. However, several potential borrow sites are specifically identified that may be able to meet all, or some, of the borrow requirements at the various facility sites. Additional explorations, land ownership considerations, and engineering analyses are needed to better define the actual borrow sites and associated borrow quantities that will be used for the work. Depending on the actual sites ultimately developed for borrow material, the method of transport will vary. Possible transportation of borrow material is via land by truck or earth moving equipment and via water by barge.

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 will continue until initiation of construction activities. The initial search was guided by the following assumed criteria, similar to that for other intake and conveyance facilities adjacent to the Sacramento River:

- Borrow material can have between 20 and 80 percent fines (i.e., material passing a #200 sieve).
- Borrow material should have a plasticity index of 8 or more and a liquid limit less than 50 (ASTM International [ASTM] D4318).
- Borrow material for non-water holding fills can have other characteristics as determined by the design engineer.
- Borrow material should not require post-excavation processing (other than moisture conditioning).
- 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 sites.
- 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 material produced.
- Borrow areas should be selected to minimize the impact or encroachment on existing surface and subsurface development and environmentally sensitive areas as much as possible.
- The total amount of borrow material for engineered fill is approximately 19 million cy (bank yards), based on the associated number of intakes, size of forebays, and conveyance requirements. The total amount includes approximately 2 million cy for the tunnel shaft pads, 7.8 million cy for the BTF embankments, 2 million cy for the IF embankments, and 6 million cy at the three intake sites (approximately 2 million cy each), and 1 million cy at the Byron Tract Pumping Plant site (Note: For reference purposes, the multiplier to convert “bank yards” to “truck yards” is 1.3, and the multiplier to convert “bank yards” to “yards compacted in place” is 0.75 (0.85 for RTM)).
Based on these criteria, the project area and the surrounding area were screened to identify potential sites. 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 the regional geologic map series published by the California Geological Survey (Map No. 1A Sacramento Quadrangle [1981] and Map No. 5A San Francisco – San Jose Quadrangle [1991]).

### 21.1.2 Potential Borrow Sources

The soils in the area of the proposed alignment are characterized by floodplain deposits consisting of clayey soils with various amounts of sand, silt, and peat. Potential target areas were identified based on soil properties and practicality of mining and transporting to project sites, using soil maps of the area developed for DRMS. Areas dominated by highly plastic or organic soils (e.g., those soils within the Unified Soil Classification System groups CH, MH, OH, PT, and OG) were eliminated from consideration as a source of engineered fill (Office of Surface Mining, 1998). For 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 these criteria are potentially suitable for engineered fill. Target soil types will be further refined as more information becomes available, including geotechnical data.

Target soils must not:

- Require extensive dewatering operations.
- Require land farming to reduce the moisture content of the existing soils.
- Significantly impact existing drainage patterns, including engineered drainage.
- Significantly impact existing development or infrastructure.
- Significantly impact cultural and environmental resources.

Some areas have been excluded from further consideration as potential engineered fill borrow areas. These areas include locations:

- More than 10 miles from the alignment.
- Within 100 feet of existing residential or commercial development.
- Within 100 feet of a military installation.
- Within 100 feet of existing roads, railroads, levees, and utilities that can be identified on current aerial photography of the project area.

As more information becomes available, distances will be refined, and specific potential borrow areas will be identified. The two most significant unknown variables are the groundwater level at potential borrow sites and the specific volume of borrow material required at various construction locations. Both of these factors need to be known before specific borrow locations and excavation configurations can be evaluated. Hazardous materials and environmental working conditions in or adjacent to sites have not been evaluated.

Imported durable rock is needed for in-river rock slope protection (RSP). Both RSP and bedding materials are needed. The maximum size of RSP is 400 pounds, and the maximum diameter of bedding rock is 4 inches. Crushed rock is needed for all-weather haul roads and for work pads at construction sites to enable all-weather construction activity. Crushed rock and/or aggregate are required for a variety of construction applications, including concrete. The most cost-effective source of crushed rock and aggregate is probably existing commercial operations able to barge the material as close as possible to the location where it is needed.

Table 21-1 characterizes the geological unit outcroppings in the general vicinity of the project that can possibly provide suitable source material for engineered fill.

Specific borrow areas are located near Intake 2, near IF on the east end of Glanville Tract (for fills at the intakes and IF), and BTF. These sites will need further analyses to determine if they have adequate quantities of suitable
borrow material. The site near Intake No. 2 probably cannot produce the volume required for all the facilities on the north end of the WaterFix BTO. The other identified borrow sites are expected to be sufficient.

Spoils disposal sites have been identified in the Concept Drawings (CER Volume 2). For additional information see Section 22.0, “Spoils Disposal Sites”. Additional geotechnical information will be required to determine if any portion of the borrow requirements can be met by first removing soils from these disposal areas before depositing spoils. To prevent the creation of new wetlands, timing of dewatering and excavation of dewatered borrow pits will be coordinated with the placement of spoil in the borrow excavation.
**Table 21-1: Summary of Potential Borrow Source Characteristics**

<table>
<thead>
<tr>
<th>Unit Name</th>
<th>Symbol</th>
<th>Age</th>
<th>Description</th>
<th>General Location</th>
<th>Potential Borrow Material</th>
<th>Suitability for Borrow</th>
<th>Rippability</th>
<th>Construction Considerations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yuba River Gold Fields</td>
<td>YGF</td>
<td>Modern</td>
<td>Well-graded gravel.</td>
<td>East of Yuba City.</td>
<td>Gravel</td>
<td>High</td>
<td>High</td>
<td>Unit consists of washed river rock, which may not be suitable for many applications.</td>
</tr>
<tr>
<td>Floodplain Basin Deposits</td>
<td>Qb</td>
<td>Holocene</td>
<td>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.</td>
<td>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.</td>
<td>Silt and clay</td>
<td>Variable</td>
<td>High</td>
<td>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.</td>
</tr>
<tr>
<td>Modesto Formation (alluvium)</td>
<td>Qm</td>
<td>Late Pleistocene</td>
<td>Gravely sand, silt, and clay.</td>
<td>Alluvial deposits in the center of the Sacramento and San Joaquin Valleys.</td>
<td>Sand, silt, gravel, and clay</td>
<td>Medium</td>
<td>High</td>
<td>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.</td>
</tr>
<tr>
<td>Montezuma Formation (poorly consolidated, clayey sand)</td>
<td>Qmz</td>
<td>Early Pleistocene</td>
<td>Poorly stratified clayey sand and pebbly sand.</td>
<td>Montezuma Hills, southwest of Rio Vista.</td>
<td>Clay and sand</td>
<td>High</td>
<td>High</td>
<td>The Montezuma Hills property is currently owned by an environmental land trust. A 500-kV line transects the property, which also overlies the producing Rio Vista gas field. Numerous producing gas wells and collection piping that would need to be addressed. Purchase of alternative property would be required.</td>
</tr>
</tbody>
</table>
### Summary of Potential Borrow Source Characteristics

<table>
<thead>
<tr>
<th>Unit Name</th>
<th>Symbol</th>
<th>Age</th>
<th>Description</th>
<th>General Location</th>
<th>Potential Borrow Material</th>
<th>Suitability for Borrow</th>
<th>Rippability</th>
<th>Construction Considerations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Turlock Lake Alluvium</td>
<td>Qtl</td>
<td>Early Pleistocene</td>
<td>Sandstone, siltstone, and conglomerate derived mainly from Sierran granitic and metamorphic rocks; non-marine. Also includes Corcoran Clay.</td>
<td>Eastern edge of the Sacramento and San Joaquin Valleys.</td>
<td>Sand, silt, gravel, and clay</td>
<td>Medium</td>
<td>Medium</td>
<td>Would require excavation of a large surface area. The unit is thin and located in areas with little relief. The available property is generally not developed. Existing railroad lines border the northern portion of the property.</td>
</tr>
<tr>
<td>San Pablo Group (marine sediments)</td>
<td>Msp</td>
<td>Late Miocene</td>
<td>Sandstone, mudstone, siltstone, and shale with minor quantities of tuff.</td>
<td>Southwestern border of Sacramento and San Joaquin Delta area.</td>
<td>Sand and silt</td>
<td>Low</td>
<td>Low</td>
<td>A substantial amount of processing may be required to achieve the desired grain size distribution.</td>
</tr>
<tr>
<td>Upper Cretaceous Marine Sedimentary Rocks</td>
<td>Ku</td>
<td>Late Cretaceous</td>
<td>Sandstone and shale.</td>
<td>West of CCF.</td>
<td>Sand</td>
<td>Low</td>
<td>Low</td>
<td>A substantial amount of processing may be required to achieve the desired grain size distribution.</td>
</tr>
<tr>
<td>Panoche Formation</td>
<td>Kp</td>
<td>Late Cretaceous</td>
<td>Sandstone, shale, siltstone, conglomerate lenses; marine.</td>
<td>West and Southwest of CCF.</td>
<td>Sand, silt, and gravel</td>
<td>Low</td>
<td>Low</td>
<td>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.</td>
</tr>
<tr>
<td>Franciscan Complex (melange)</td>
<td>Kjf</td>
<td>Late Cretaceous to Jurassic</td>
<td>Melange, greenstone, sandstone, shale, conglomerate, metagraywacke, limestone, chert, serpentinized ultramafic rock.</td>
<td>Coastal Ranges west of Interstate 5 and south of Interstate 580.</td>
<td>Sand and gravel</td>
<td>Low</td>
<td>Low</td>
<td>Mineral composition, degree of lithification, and grain-size distribution make this unit unsatisfactory for use as engineered fill, rock slope protection, or crushed rock.</td>
</tr>
</tbody>
</table>
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Spoils Disposal Sites

Significant thicknesses of non-supportive or organic soils must be removed in the course of forebay, pumping plant, and shaft construction. Large volumes (approximately 25 million cy) of re-usable tunnel material (RTM), consisting of saturated soils mixed with bio-degradable polymers, are generated by tunneling operations. Large volumes (approximately 4.5 million cy) of excavated material is also expected to be removed from connection channels downstream of BTF that connect to both the Banks PP and Jones PP Intake Channels. Smaller quantities of excess excavated materials are expected at other facility sites, including about 1.9 million cy at IF, approximately 1.6 million cy at each intake site, and approximately 1 million cy from the launch and reception shafts.

Organic materials will be stockpiled for placement over completed disposal areas. Soils that are unsuitable for reuse as restoration material, flood fight material, and engineered fill need to be disposed. These materials will be characterized and disposed appropriately. RTM will be handled and stockpiled in accordance with the requirements of the final EIR/EIS including the 2017 final MMRP.

The presence of hazardous materials or environmental working conditions in or adjacent to potential spoil disposal sites will need to be evaluated. Hazardous materials excavated during construction needs to be segregated from other construction spoils and properly handled in accordance with state regulatory requirements.

22.1 Description and Site Plan

Much of the area surrounding the alignment consists of low-lying floodplain developed as agricultural land. Depending on the properties of the spoils, some predominantly organic soils can be deposited on portions of this area without adversely affecting agricultural use.

Any RTM unsuitable for reuse will be disposed at sites adjacent to the tunnel construction work areas, eliminating the need for extensive hauling and allowing the use of conveyors and off-road equipment to move and place RTM. RTM disposal areas are shown on the Concept Drawings (Volume 2).

RTM at the BTF Pumping Plant shaft, South Tunnel RTM, and excess excavated material at the downstream connection channels at BTF area will be disposed in disposal/borrow areas on Byron Tract, both north and southwest of BTF. Off-road earth moving and hauling equipment will move the material from the channels to the disposal areas.

Unsuitable excess excavated material at the northern sites (IF, North Tunnels, and intakes) will be disposed in the designated RTM disposal areas at Glanville Tract near IF. If construction schedules allow, some suitable materials will be used for the intake fill pads. Unsuitable excess excavated material can also be disposed in the borrow/disposal site northeast of Intake No. 2 on the Glanville Tract east of the IF. At that site, the footprint of borrow areas will be filled with excavated material.

Disposal area potential was based on the potential disruption to existing development, infrastructure, drainage patterns, and cultural and environmental resources. After elimination of sites based on the impracticality of transport, the following restrictions were applied:

- Spoils are placed in project borrow areas whenever possible.
- Spoils are placed in designated spoil areas adjacent to the various work sites whenever feasible.
- Spoil areas are as close as possible to the work sites.
- Spoil areas are not within 100 feet of existing residential or commercial development.
- Spoil areas are not within 100 feet of a military facility.
Spoil areas are not within 100 feet of existing roads, rail lines, or infrastructure.

Spoil areas include a 150-foot buffer between fill areas and existing levees.

As more information about the nature and volumes of soil generated becomes available, distances and sizes of disposal areas will be refined and identified.

Preliminary disposal sites, selected using the criteria and quantities described, are shown on the Concept Drawings - Volume 2.

### 22.2 Construction Methodology

Conventional earthmoving equipment, such as bulldozers, graders, and conveyors, will be used to move and place the spoil. Spoil placed in disposal areas on low lying agricultural lands needs to be placed in 12-inch lifts, with nominal compaction to control potential subsurface failure of the existing floodplain soil. Spoil maximum height is expected to be 6-10 feet above preconstruction grade for all spoil sites and 10-15 feet above preconstruction grade for BTF sites with side slopes of 5H:1V or flatter. Spoils placed in areas with stronger foundation soils can be of greater depths and use steeper side slopes. After final grading of spoil, the area will be restored based on site-specific conditions per project restoration guidelines. See Section 11.0, “Tunnels,” for a description of the RTM handling procedure and limitations regarding the sequence for placing spoils, especially near existing levees. Depending on the location of the spoil disposal site and the quantity of material to be removed from the project site, material transport may take place by barge operations or by California legal transport trucks on state, county or local roadways.
SECTION 23.0
Stockpiles and Staging Areas

This section describes stockpiles and staging areas associated with construction activities.

23.1 Stockpiles

Stockpiles may be used in the construction areas of the project to store materials for later use. 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 and materials storage, and for temporary field offices for construction.

Materials to be stockpiled may include:

- Strippings from various excavations, facility work, staging areas, borrow areas, and disposal areas for possible reuse in landscaping or as topsoil replacement for agricultural areas.
- Tunnel muck that is stockpiled temporarily and is slated for reuse after treatment as needed.
- Peat spoils for possible use on agricultural land; 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).
- Aggregates or soil materials to be used for concrete, rolled compacted concrete, soil cement, or other processed materials of construction.
- Other materials being stockpiled on a temporary basis prior to hauling to permanent stockpile areas.

Areas designated as disposal areas, borrow areas, or construction staging areas may include stockpiles and may also be used to condition materials for later use.

Areas designated for tunnel muck disposal will be stripped of topsoil prior to placement of the muck. Stripped topsoil will be stockpiled and re-spread over these areas after the tunnel muck is placed. Topsoil stripping and stockpiles will be staged for consistency with muck placement sequencing.

Site clearing and grubbing, work area limits, and site access to stockpile locations will be developed, along with the applicable security provisions such as security fences, gates, and/or cameras. Silt fencing and straw bale dikes may be installed, as needed, to address drainage issues, and dust abatement and other environmental concerns relating to stockpiles will also need to be addressed.

23.2 Laydown and Construction Staging Areas

Laydown and construction staging areas will be needed for all elements of the proposed construction. Sufficient space will be required adjacent to the project sites to allow contractors to set up field offices, provide employee parking, and stage the materials and equipment needed for the work.

Each construction site will also include some combination of required processing operations, including concrete batch plants, pug mills, soil mixing facilities, and cement storage. Batch plants will be established at specific sites (see Volume 3 – Map Book), along with fine and course aggregate stockpiles, to produce concrete needed for the work. Pug mills will be provided for roller compacted concrete and other processed soil materials used at the various sites. Soil mixing facilities may be needed for some aspects of muck disposal and for ground improvement activities. Cement and required admixtures will be stored at each site as needed to support concrete, slurry walls, ground improvement, soil mixing, and other similar needs.

All soil and concrete processing facilities, as well as all materials storage, will be established with suitable grading and best practices to minimize surface water and local area impacts. Also, all storage and processing areas will be properly contained if required for environmental and regulatory compliance.
Material stockpiles and handling areas are expected to be used to support the concrete and soil processing features described above. These stockpiles will be used at the staging areas for all tunnel, forebay, and intake construction contracts. Contracts with significant earthwork elements, such as the intake, forebays and pumping plants, will also require earth material processing at each site. Specifically, the following construction and construction staging areas are anticipated:

- The full area enclosed by the relocated Highway 160 and river will be used for construction and construction staging at the intake sites.
- The full area enclosed by the forebay and the tunnel muck disposal area will be used for construction and construction staging at the IF site.
- The full area enclosed by the surge shafts and pumping plants and associated muck disposal areas.
- Construction and construction staging areas for the tunnel shafts and associated muck disposal areas as shown on the Concept Drawings.
- The area enclosed by the footprint of the BTF and the adjacent borrow/disposal site will be used for construction and construction staging.

Construction and construction staging for power supplies and other support features, such as access roads, will be developed in greater detail during preliminary engineering and final design.

Laydown and staging areas for all construction sites will need to be accessible in all weather conditions for as long as such construction is ongoing. Laydown and staging areas may require security fences, gates, and/or cameras.

### 23.3 Temporary and Permanent Footprint

Construction of the WaterFix BTO components will result in temporary construction and permanent facility footprints. Table 23-1 summarizes the projected footprint acreage for each WaterFix BTO component and a project total. Major assumptions involved in the generation of construction footprints include the following:

- Disposal areas for tunnel muck, as well as unsuitable and excess excavated material, are included with the “During Construction Acreage” because these areas are expected to be reclaimed for other uses on a permanent basis.
- Borrow areas currently identified for the WaterFix BTO are included with the “During Construction Acreage.”
- Offsite borrow areas cannot be estimated at this time and are not included.
- Offsite materials and equipment staging, fabrication, and storage sites cannot be estimated at this time and are not included.
- Small, miscellaneous minor access roads and small barge landing acreages are not included because they are expected to be within the accuracy limits of the larger acreages presented.
- No footprint areas are included for the subsurface portions of the tunneled sections of the WaterFix BTO because the tunnels will not result in surface impacts in these areas.
- No footprint areas are included for the electrical transmission and distribution system because the exact location and impact area for these facilities is not known at this time. Ongoing system impact studies are expected to further define these areas.
- No footprint areas are included for the fiber optic communication system because the exact location and impact area for these facilities is not known at this time. If the fiber optic lines are placed within the tunnels and radio or microwave communications are used between the project area and operations centers in Sacramento, no additional impact area would result. If surface-installed fiber optic conduit systems are used, the impact area would be consistent with those of the electrical system in the project area and would mostly use existing transportation and utility corridors for connections to the remote operations centers.
### Table 23-1: Projected Construction and As-Constructed Footprint for WaterFix BTO Facility Components

<table>
<thead>
<tr>
<th>Facility Component</th>
<th>(Temporary) During Construction Acreage(^a)</th>
<th>(Permanent) As-constructed Acreage(^b)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Intake Facilities</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Intake No. 2</td>
<td>37 acres</td>
<td>167 acres</td>
</tr>
<tr>
<td>Intake No. 3</td>
<td>60 acres</td>
<td>98 acres</td>
</tr>
<tr>
<td>Intake No. 5</td>
<td>55 acres</td>
<td>101 acres</td>
</tr>
<tr>
<td>Intake Subtotal</td>
<td>152 acres</td>
<td>366 acres</td>
</tr>
<tr>
<td><strong>North Tunnels(^c)</strong></td>
<td>323 acres</td>
<td>237 acres</td>
</tr>
<tr>
<td><strong>Intermediate Forebay</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IF (Including Overflow Containment Area)</td>
<td>8 acres</td>
<td>243 acres</td>
</tr>
<tr>
<td>Main Substation</td>
<td>31 acres</td>
<td>0 acres</td>
</tr>
<tr>
<td>IF Reusable Tunnel Material Disposal Area</td>
<td>0 acres</td>
<td>330 acres</td>
</tr>
<tr>
<td><strong>Intermediate Forebay Subtotal</strong></td>
<td>39 acres</td>
<td>648 acres</td>
</tr>
<tr>
<td><strong>Main Tunnels(^c),(^e)</strong></td>
<td>276 acres</td>
<td>2,749 acres</td>
</tr>
<tr>
<td><strong>Byron Tract Forebay(^d)</strong></td>
<td>90 acres</td>
<td>1,870 acres</td>
</tr>
<tr>
<td><strong>South Tunnels(^f)</strong></td>
<td>60 acres</td>
<td>140 acres</td>
</tr>
<tr>
<td><strong>Overall WaterFix BTO Project</strong></td>
<td>940 acres</td>
<td>6,010 acres</td>
</tr>
</tbody>
</table>

\(a\) Includes borrow material, and control structures work area  
\(b\) Includes re-usable tunnel material  
\(c\) Refer to Section 11 for a detailed breakdown of Main Tunnel sites  
\(d\) Includes Byron Tract Forebay Pumping Plant  
\(e\) Permanent acreages include tunnel subsurface impacts  
\(f\) Includes impact acreage for all conveyance facilities downstream of BTF (connection channel, outlet structure and control structures)

**Notes:**  

BTF = Byron Tract Forebay  
IF = Intermediate Forebay
SECTION 24.0

Construction Methods, Sequencing, Constructability and Schedule

24.1 Overview

This section presents a conceptual overview of construction methods, construction sequences, and other factors that may affect construction. This section was prepared to assist in the environmental planning effort to determine construction related impacts. The construction items discussed in this section will be further evaluated during preliminary and final engineering design phases. In general, the project will include the following types of construction packages:

1. Intake Facilities and Sedimentation Basins
2. Box Culverts
3. Tunnels
4. Forebays
5. Levees
6. Channels
7. Roads
8. Utilities and Infrastructure Crossings
9. Operable Barriers
10. Controls and Communications
11. Power Supply and Grid Connections

24.2 Project Constraints

There are physical, legal, environmental, and operational considerations which must be addressed during construction:

• Department of Fish and Wildlife streambed alteration agreements (1600 permits) typically limit construction between June 1 and October 31 (fish windows). This will affect all work on the river side of the levees not inside cofferdams or landside of levees. For this project, fish windows have been established each year for construction activities.
• Positive levee breach protection must be in place prior to any facility construction adjacent to river levees.
• The vertical outlet shaft which feeds the tunnels located at the end of each Intake Sedimentation Basin, IF Inlet and Outlet shafts, and Pump Plant shaft will first be used to launch or recover tunnel boring machines (TBM’s), reusable tunnel material (RTM) removal and/or for other tunnel operations. This requires tunneling activities to be finished prior to final completion of the facilities.
• Adverse geotechnical conditions include peat soils, high water table, compressible soils, and possible liquefaction.
• Multiple tunnel, intake, and forebay contracts will require a large labor pool including a number of specialists. The availability of this labor may require special wide-spread recruiting and training.
• Presence of existing utilities can conflict with sheet piling, bracing, and excavations.
• Presence of existing buildings and other structures must be considered.
• Work adjacent to the railroad must be considered.
The alignment passes near residential and agricultural areas and near utilities, railroads, aqueducts, culturally sensitive lands, and navigable and recreational waterways. Accordingly, selection of construction methods must consider:

- Safety and traffic impacts
- Impacts to river hydrology and groundwater (including flood season exclusionary periods)
- Impacts to archeological and cultural sites
- Fish and wildlife protection (including exclusionary periods for species protection)
- Traffic restrictions and times of use for local roadways
- Construction hours and nighttime work hour restrictions
- Local noise ordinances
- Federal, state, and local air quality regulations
- Land use
- Utility availability and interruptions
- Presence of weak and compressible soils (i.e., low-bearing capacity and susceptible to significant settlements)
- Site accessibility
- Quantity and type of material deliveries
- Safe storage and use of hazardous construction materials, including fuel
- Availability of staging or laydown areas
- Availability of new technology
- Coordination amongst multiple, simultaneous construction packages

24.3 Common Activities for all Construction Types

24.3.1 Permitting and Plan Preparation

Design and Construction Authority (DCA) will ensure that all major permits are in-place prior to commencement of any construction activity. (See Section 26, “Permits” for additional information).

24.3.2 Mobilization

During mobilization, the contractors bring staff, materials, and equipment to the construction sites. During this time, the contractors set up their work areas to expedite construction activities; locate offices, warehouse, staging, or laydown areas; secure temporary power; and configure traffic patterns to move labor, materials, and equipment in and out of the sites.

24.3.3 Site Work

Site work consists of clearing & grubbing, constructing site work pads, and defining and building construction access roads. Before site work commences, the contractor will implement erosion and sediment controls in accordance with a project specific SWPPP as required by the General Permit. Although plans are to utilize the existing levee roads, local roads, bridges, and highways during construction to the greatest extent possible, some new roads and bridges may be constructed to expedite construction activities and to minimize impact to existing commuters and the environment. Maintaining access roads and environmental controls will require enforcement of Best Management Practices (BMPs).
After mobilization and preliminary site work, construction will continue at each site. While general construction methodologies have been discussed in the preceding sections, the following discussion focuses on factors that govern the constructability of the key project components.

24.3.4 Access Routes
The size, weight, and volume of construction-related traffic require that the access routes to each site are analyzed to determine what improvements may be required. Although contractors are likely to make the most out of the existing levee roads, bridges, highways and waterways during construction, some roads and bridges might need to be altered, widened, strengthened, replaced, or newly built to expedite construction activities and minimize impact to the traveling public and the environment. Maintaining access roads and environmental controls requires compliance with appropriate.

24.3.5 Ground Improvement
Due to the soft ground conditions expected at the construction sites, it is necessary to improve existing sites to support heavy construction equipment, switchyards, transformers, concrete and grout plants, cranes and hoists, TBMs, and water treatment plants.

In addition to load-carrying capacity considerations, soft ground conditions may be subject to consolidation settlement caused by the placement of fill to create construction pads at each shaft site. Preliminary estimates suggest 8 to 10 feet of consolidation settlement can be expected from the placement of shaft pad area fills over very soft materials. This settlement is anticipated to be relatively slow to occur as the excess pore water pressure drains from the soft organic clays found at some sites. The estimated time for 90 percent of the consolidation to occur is approximately 50 years, which is not feasible for the project. Therefore, the conceptual design anticipates pre-loading the existing pad and placement of vertical wick drains, spaced at 5 foot intervals to a depth of 60 feet, to speed the consolidation through vertical relief of the excess pore water pressure in the compressible soils. Estimates suggest that all but approximately 12 inches of the total settlement will occur within 1 year following final pad placement if wick drains are installed beneath the pad fill. If it is required to limit settlement of the pad fills to less than this, overbuilding of the fill could be incorporated into the site preparation work. The additional fill height could be removed during final site preparation.

Although this work could be performed during facility construction, it is advantageous to commence these operations early so that the overall project schedule can be improved. The goal is to construct the access road onto the site from the nearest highway, improve the load-carrying capacity of the site within the facility work area, install wick drains and necessary drainage blankets, and form the permanent elevated platform for the facility.

Where pre-loading embankments are not practical and for deep soil improvement such as under shafts and tunnel safety zones, ground improvement methods may include jet grouting, stone or sand columns, dynamic compaction, deep soil mixing, etc.

24.3.6 Site Security and Public Safety
Continuous site security and public safety is required during construction. Appropriate fencing, access control, and non-working-hour patrols are expected to be provided for the main shaft work sites, including temporary staging and materials processing areas. RTM disposal sites are not fenced. All work sites are provided with suitable barriers, lighting, signage, and fencing to protect the public from the work. Traffic controls are provided to minimize disruption and meet safety requirements.

24.3.7 Concrete Batch Plants, Pug Mills, and Cement Storage
Each construction site consists of a combination of processing operations including concrete batch plants, pug mills, soil mixing facilities, and cement storage. Along with fine and course aggregate storage, batch plants are established at a site to produce needed concrete. Pug mills are provided for Roller Compacted Concrete (RCC) and other processed soil materials used at the various sites. Soil mixing facilities are needed for some of the muck disposal and for ground improvement activities. Cement and required admixtures are stored at each site to
support concrete, slurry walls, ground improvement, soil mixing, and other similar needs. All processing facilities and materials storage sites are established with suitable grading and best practices to minimize surface water and local area impacts. All storage and processing areas will be properly contained as required for environmental and regulatory compliance.

24.4 Typical Construction Methodology

The following typical methodology applies to most conventional construction activities and facilities:

- Temporary power for construction is required at all work sites.
- The ground beneath most structures will be improved where required (refer to the Concept Drawings in Volume 2 for limits of ground improvement).
- Foundation piles are driven to required depths beneath structures.
- The principal concrete structures are typically constructed within an open dewatered excavation that has sloped walls or excavation supports. The structure is conventionally built using reinforced concrete slabs and formed reinforced concrete walls.
- Alternatively, the concrete structures can be constructed using diaphragm walls, where only the internal portion of the structures must be dewatered and excavated. The diaphragm wall methodology is shown in the Concept Drawings (Volume 2).
- The fill pads require large quantities of low-permeability material, fill material, and typically are constructed using bulldozers, compactors, scrapers, and graders in compacted lifts (e.g., 12-inch lifts) to design specifications.
- Temporary power for construction is required at all work sites.

24.5 Intake Facilities and Sedimentation Basins

All Intake and Sedimentation Facility sites have similar infrastructure complexities, foundation characteristics, and construction periods to complete. Significant temporary construction zones are required for staging and storage. Construction of the intakes includes typical marine construction plus special considerations related to levee penetrations and flood protection (refer to Section 15.0, "Levees"). Construction of intake facilities requires means, methods, and approaches unique to marine and heavy civil construction. This section describes the construction approaches and types of construction for the intake and sedimentation facilities.

24.5.1 Construction Constraints

- Adequate environmental protection for fish and other species during the construction phase must be maintained.
- Work on the river side of the levee is restricted by flood control issues each winter season from November 1 to April 15. Waivers are required to work in the river during this period.
- California Department of Fish and Wildlife streambed alteration agreements (1600 permits) typically limit construction within natural waterways between July and October (fish windows). This will affect all work on the river side of the levees not inside the cofferdam. For this project, fish windows have been established from June 1 to October 31 each year.
- Positive levee breach protection must be in place prior to any Intake Facility Construction.
- Levee instability due to water seepage and soil erosion along the box conduits after construction must be monitored and prevented.
- Construction near levees requires approval and permitting from USACE, state, county, local agencies and others.
Flood control must be maintained.

- The river must remain navigable during construction.
- The soil or geologic condition at the intake location dictates the depth of the excavation, as well as the type of foundation used.
- Highway Route 160 passes through the site. The highway must be rerouted such that all construction activity must protect public-use safety of Route 160.
- The vertical outlet shafts to the conveyance tunnel each Intake Sedimentation Basin will first be used as recovery shafts for removing the TBM and other tunnel equipment. This requires tunnel completion prior to completion of the basin outlet.
- Adverse geotechnical considerations include peat soils, high water table, compressible soils, and possible liquefaction
- All materials to be delivered by truck over existing highway network.
- Potentially small construction windows due to permit requirements could cause scheduling concerns.
- Staging and storage area and construction zone prep (5 to 10 acres per each intake structure).

Refer to Section 15.0, “Levees” for additional information regarding work in the levee.

### 24.5.2 Major Construction Elements and Sequence of Construction

Presented in Table 24-1 are major construction elements and sequence of construction for completing the Intakes and Sedimentation Basins, including temporary and long-term measures to protect the levees. The elements and sequence presented is one possible approach and may be revised to fit actual construction contracting and implementation.

**Table 24-1 Construction Elements for Intake Facilities**

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Contractor Mobilization and initial activities include: clear &amp; grub Intake and Sedimentation area inside Levee, construction of temporary contractor plant including offices, shops, laydown, batch plant, and establish environmental protection including sediment ponds, diversion ditches, silt fences, and other required facilities. Prepare borrow areas and excess material disposal areas. Provide facilities to treat construction water.</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Construct a slurry cutoff wall on the landside of the existing levee. The location of the cutoff wall shall be parallel to and outside the landside prism of the existing levee. The cutoff wall should extend to a depth below and a distance upstream and downstream beyond the length of the relocated portion of Highway 160 and the portion of the new intake structure’s box culverts beneath the relocated potion of Highway 160 to minimize potential seepage through and under the existing levee.</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Construct a well point dewatering system to allow for excavation and placement of new fill slopes for the relocation of Highway 160 and the portion of Box culverts going under the highway embankment.</td>
</tr>
<tr>
<td>4</td>
<td>1,2,3</td>
<td>Excavate and remove peat soils to suitable subgrade beneath the area for the Widened Levee, the Box Culverts and relocated Highway 160.</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Construct soil improvements beneath widened levee area, under the box culverts, and the fill slopes required for the permanent relocation of Highway 160. Possible soil improvement techniques may include cement deep soil mixing, dynamic compaction, jet grouting, stone columns, surcharge preloading, etc. to be evaluated during project detail design.</td>
</tr>
</tbody>
</table>
**Table 24-1 Construction Elements for Intake Facilities**

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>1,5</td>
<td>Construct new fill slopes for the new permanent alignment of Highway 160 and the widened levee. Construct portions of the intake box conduits that traverse the fill beneath the new permanent alignment for Highway 160.</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Construct the roadway for the relocated part of Highway 160 and reroute traffic.</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>Prepare foundation for embankment around perimeter of Sedimentation Basin and drying lagoons. Will require well point dewatering and removal of unsuitable soils.</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>Place slurry wall around perimeter of sedimentation basin and drying lagoons and install deep wells such that sediment basing can be dewatered and excavated in the dry.</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>Construct embankment around perimeter of the sediment basin and outlet structure. Embankment must be constructed prior to completion of tunnels to the outlet shaft, and to a height to protect the tunnel from the 200-year flood plus allowance for sea rise. The embankment may be constructed from suitable materials excavated from the Sediment Basin or other project works. Embankment material may require drying on contractor provided material drying beds. Tie embankment into the Highway 160 embankment.</td>
</tr>
<tr>
<td>11</td>
<td>7</td>
<td>Construct a diaphragm wall within the existing levee crest forming the back wall of the planned new intake structure and extending below the intake to a depth embedding the wall in dense clays. Extend the diaphragm wall 150 feet beyond the length of the intake structure to minimize potential seepage though and under the existing levee. Provide cutouts for extending the intake box conduits through the diaphragm wall. Tie the Diaphragm wall into the slurry wall constructed in 9 above.</td>
</tr>
<tr>
<td>12</td>
<td>11</td>
<td>Construct sheet pile portion of Intake cofferdam (river side of existing levee) and training walls for intake structure driving sheet piles to depths required to achieve hydraulic cutoff. Tie the cofferdam into the diaphragm wall constructed in “11” above to provide an enclosed area for construction of the intake structure. This work must be done within the “Fish Window” restraint.</td>
</tr>
<tr>
<td>13</td>
<td>11,12</td>
<td>Excavate and fill within the cofferdam to provide a level working surface above the anticipated river high water level.</td>
</tr>
<tr>
<td>14</td>
<td>13</td>
<td>Construct soil improvements below the intake structure using jet grouting or other approved means.</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>Excavate within the cofferdam. This will be “wet” excavation maintaining equal water level inside and outside the cofferdam.</td>
</tr>
<tr>
<td>16</td>
<td>15</td>
<td>Construct drilled casings and reinforced concrete piers beneath the intake structure. An alternate using displacement piles may be specified. Piles will be constructed to a specified depth to assure stability and resist uplift. Considering the elevations of the Intake structure, piers at depths up to 90 feet below the intake invert will be required. Drilling such deep piers will have major cost and schedule impacts to intake construction.</td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>Install tremie concrete seal within cofferdam.</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>Dewater, simultaneously installing internal bracing during dewatering. Operations will include underwater construction, such as tremie slab placement and sheet pile trimming.</td>
</tr>
<tr>
<td>19</td>
<td>18</td>
<td>Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed metal installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>20</td>
<td>19</td>
<td>Metalwork fabrication, machining, assembly, and installation (stainless steel fish screen panels, embeds, flow control baffles, bulkheads, traveling brush screen cleaning system, gantry crane mechanical hoist system, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.). Gates and valves must be installed, tested and closed prior to completing box culverts through segment wall.</td>
</tr>
</tbody>
</table>
Table 24-1 Construction Elements for Intake Facilities

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>14</td>
<td>Construct soil improvements between the cofferdam diaphragm wall and any unimproved soils up to the slurry cutoff wall installed with Highway 160 relocation.</td>
</tr>
<tr>
<td>22</td>
<td>20,21</td>
<td>Construct remaining length of intake box conduits between the intake structure and the previously placed box conduits constructed with the relocation of Highway 160. The intake structure must be complete prior to any excavation in the levee. Depending on success of soil improvements, box culverts may require pile support. Use sheet pile to open cut levee to box invert. Construct fills over box culverts to finish grade between intake structure and Highway 160.</td>
</tr>
<tr>
<td>23</td>
<td>20</td>
<td>Complete construction on water side of intake, including pile cut-off, rip-rap, and floating service platforms. Underwater construction with divers will be required.</td>
</tr>
<tr>
<td>24</td>
<td>10</td>
<td>Construct soil improvements beneath landside facilities susceptible to settlement. Soil improvements could include jet grouting, sur-loading and wick drains, dynamic compaction, stone or sand columns or other approved methods.</td>
</tr>
<tr>
<td>25</td>
<td>10</td>
<td>Complete excavation of Sediment Basins and Sediment Drying Basins and placing embankments. The pad fill requires large quantities of fill material. It is constructed of low-permeability material using bulldozers, compactors, scrapers, and graders in compacted lifts (e.g., 12-inch lifts) to design specifications. Suitable fill material may come from sediment basin excavation or from borrow.</td>
</tr>
<tr>
<td>26</td>
<td>24</td>
<td>Drive piles for intake structures. The soil or geologic conditions at the sedimentation basin location dictates the depth of the excavation as well as the type of foundation used. Pipe piles are assumed for concrete structures as shown in the Concept Drawings (Volume 2).</td>
</tr>
<tr>
<td>27</td>
<td>26</td>
<td>Complete all structures possible prior to removing tunnel excavation equipment. The principal concrete structures are constructed conventionally using formed reinforced concrete walls and slabs installed after dewatered open excavation with sloped walls or excavation supports.</td>
</tr>
<tr>
<td>28</td>
<td>10</td>
<td>Remove tunnel excavation and lining equipment using Sediment Basin outlet shaft.</td>
</tr>
<tr>
<td>29</td>
<td>28</td>
<td>Complete outlet structure concrete and install gates and operators.</td>
</tr>
<tr>
<td>30</td>
<td>25</td>
<td>Construct RCC and AC lining of basins, and complete valve boxes, valves, piping and gates.</td>
</tr>
<tr>
<td>31</td>
<td>30,29,23</td>
<td>Complete work and commission project. Including miscellaneous civil site work (e.g., fencing, gates, access roadways and ramps, log booms and debris deflectors, hydroseeding, landscaping, etc.) and miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, intake and site power supplies, flow/level/turbidity/limit/torque instrumentation, utility service, etc.).</td>
</tr>
</tbody>
</table>

24.6 Box Culverts

Well-planned construction of the Box Culverts from the Intake to the Sedimentation Basin will be critical to Levee protection and overall project success. The following is intended to emphasize the criticality of this work.

24.6.1 General

The Box Culverts are part of the Intake Construction. Box culverts are used to convey water from the Intake structure to the Sedimentation Basin. There are 12 culverts per intake, each 12 ft. x 12 ft. x approximately 390 feet long. The 12 box culverts run parallel constructed in pairs. The distance between paired culverts is 50 ft. to 65 ft. The distance between culvert pairs varies from 125 ft. to 175 ft. depending on intake.

Each culvert will be constructed in two parts. First, the culvert will be constructed under Highway 160 running from the slurry wall constructed for Highway 160 construction to the sedimentation basin.

The second part will be constructed after completing of the intake structure and will connect the part 1 box culvert to the intake structure.
24.6.2 Ground Improvement/Foundation

Prior to excavation, ground improvement will be constructed using jet grouting, stone columns, dynamic compaction, or other methods to be determined during detail engineering. Placing foundation piles under the box culverts will be evaluated during detail engineering and may be directed.

24.6.3 Excavation

Excavation may either be typical open trench excavation or sheet pile shored excavation. It is anticipated the first part of excavation will be open cut, noting that excavation depths may be as much as 35 ft. With culverts spaced at 50 ft. and 125 ft. most of the material will be excavated requiring substantial backfill. Depending on availability of backfill, contractors may choose to shore excavations and pour slabs and walls against the sheet pile.

The second part of excavation will be through the existing levee, reaching depths up to 50 ft. In order to protect against levee breech, this work cannot take place until after the intake structure, Highway 160 and the sedimentation embankments are complete. It is anticipated the levees will be excavated down to a level allowing excavation for the culverts to be no more than 30 ft. The work area will be bounded on one side by the Intake and the other by Highway 160, making access to the culverts difficult. Efficient excavation will be in sheet pile shored excavations to facilitate completion of each culvert allowing access to the next culvert.

Open cut trench widths will 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 5 ft. below the existing ground surface. Dewatering will be required for each culvert. The slurry wall and segment wall constructed for Highway 160 and Intake, respective plus ground improvements will create effective water cut-off. Well points and/or deep wells will effectively remove ground water. Dewatering will have to continue until fill is placed over the top of the box culvert to a depth to prevent uplift.

24.6.4 Concrete Structure

The box culvert can be constructed of either cast-in-place concrete or pre-cast elements. The size of the box culverts has been designed as 12 ft. x 12 ft. to facilitate precast sections. While precast sections may shorten construction time, and the time excavations are open, the confined work area between Highway 160 and the widened levee embankment will be difficult to traverse with precast elements, and my limit construction of one pair of box culverts at a time. On the other hand, cast in place concrete may require less access planning, and allow construction of multiple culverts simultaneously.

After construction is complete, the work area will be re-contoured as required and all disturbed areas will be seeded. Consideration will also be given to additional replacement or upgrades to drainage facilities.

Erosion control measure for slopes within the alignment will include surface roughening followed by seeding.

Paved areas disturbed by construction will be restored to their original condition prior to construction.

24.7 Tunnel Construction

TBM Tunneling includes preparing the site to accept, assemble, and support either earth pressure balance (EPB) TBMs or slurry pressure balance (SPB) TBMs during tunnel excavation as well as assembly of the TBM itself. Associated activities will include construction of process water treatment and slurry treatment (SPB) plants, modification of power facilities to support the TBM drive, construction of facilities to support the TBM operations, delivery of tunnel muck trains, acceptance and staging of all TBM and tunnel excavation equipment, and assembly the TBM and associated equipment in the shaft. The selected site has sufficient area available for all of these activities.

Activities associated with excavating the tunnel and installing tunnel lining include delivery and storage of pre-cast tunnel lining segments; grout and ground conditioning materials; process and ventilation pipe; muck train rails; conveyance of these materials into the shaft; processing of TBM slurry (SPB) or removal of tunnel spoil from the shaft (EPB); and the transport, conditioning, and placement of tunnel spoil on-site. The TBM contractor will
perform ground improvement as necessary for launching of the TBMs from the shafts. See Section 11, “Tunnels,” for additional tunneling information.

Tunnel lining segments, adequate for a two to three-day supply, will be delivered by barge or truck to a storage area on the construction fill pad, delivered to the shaft via truck or special carrier vehicle, and lowered into the shaft by crane. Tunnel excavation spoil will be removed from each shaft using a crane or vertical conveyor and placed in a temporary stockpile adjacent to the shaft, then transported for disposal by truck. Alternatively, the spoil could be placed directly into a hopper feeding a conveyor belt system for muck transfer either directly off-site for disposal or to a temporary muck storage area for subsequent truck haul to disposal sites.

Completion of the tunneling phase includes removal of the tunneling-related equipment from the shaft and site; lining system completion and installation of any internal tunnel components; connecting the tunnels through the shaft; installation of permanent ventilation and maintenance access; and the completion of any structures or backfilling within the pump shafts. Dewatering pumps for these shafts are likely to be installed later during the pump station construction phase.

24.7.1 Tunnel Contractor Outreach Efforts

Considering the magnitude and complexity of the tunnel portions of the project as well as schedule and budget goals, numerous workshops were held with tunneling contractors to identify, evaluate and recommend tunnel-related strategies. The topics discussed included:

- Scope, size and complexity of potential tunnel contract packages
- Alternatives for tunnel project delivery and contracting methods
- Tunnel risk identification and risk management strategies
- Strategies and actions pertaining to tunnels that should be considered to meet program goals

One-day workshops were conducted with each major tunneling contractor which provided sufficient time for extensive discussion on the proposed tunneling scope of work. The attendees were provided with background information on studies conducted to date, current planning-level assumptions, and previous technical findings as a basis for participation in these workshops. The information shared by these industry contractors is considered to be critical to construction planning decisions.

A summary description of the Tunnel Contractors’ recommendations and observations on the contracting packages and strategies, preliminary risks, and critical actions based on the conceptual project definition can be found in appendix G. The recommendations represent various contractor opinions and will be further evaluated during preliminary and final design phases. The recommendations are based on several key criteria, such as the delivery schedule, basic tunnel configurations and alignment, and other assumptions discussed at the workshops.

24.7.2 Tunnel Construction Considerations

To construct the tunnels efficiently and safely the following items will need to be considered by the contractor; pressurized-face TBM tunneling is assumed (see Section 11, “Tunnels”).

- Shafts for tunnel access are expected to be constructed using diaphragm wall methods and excavated in the wet. The bottom of the shaft will be closed with placement of a tremie seal to prevent bottom instability and shaft uplift.
- High groundwater level and subsurface conditions require construction methods that do not require extensive dewatering in the tunnel and shaft construction.
- Water encountered during tunneling requires treatment and discharge permits prior to disposal
- The tunnels are expected to be classified as “potentially gassy,” which require specific safety measures and equipment during construction.
- Unforeseen geological conditions may cause construction delays.
- Extensive geotechnical data is required along the proposed alignment for the TBM design and tunnel construction.
- Shaft construction in weaker soils (such as peat) at depth will require ground treatment to provide for excavation stability and launching of TBMs.
- Advanced contracts to place fill at the launch and retrieval shaft sites is being considered to preload the sites and allow settlement to occur before shaft construction is initiated.
- Removing soft clay soils from a deep vertical shaft can pose difficulties.
- The interface between tunnel contracts for various reaches could become problematic unless responsibilities between contractors are clearly defined and well-coordinated.
- The anticipated number of TBMs and personnel required to operate these machines needs to be considered in the overall project schedule.
- The energy requirements for tunnel construction are significant (over 200 MW). Early contracts might need interim power generated onsite for early critical path activities to mitigate potential delays.

### 24.7.3 Tunnel Construction Methodology

This section summarizes the assumed construction techniques and the sequence of activities specific to tunnels. The final tunneling methods and the number of construction sites will be determined according to geotechnical conditions, current engineering practices and construction schedule. Based on current tunneling technologies and authorized project delivery method, the conceptual design designates tunnel construction reaches and sites for project planning purposes.

The compatibility of the tunneling excavation method with anticipated ground conditions is critical in minimizing risk, optimizing tunnel advancement rate, and designing the tunnel support system. Currently, geotechnical information is very limited, with insufficient boring locations along the planned tunnel alignment. Once adequate geotechnical investigations have been performed, preliminary design evaluations will refine the recommendations for tunnel excavation and support design.

To prepare for the startup of the tunnels, pre-tunneling construction work will be required including access roads, ground improvement, power supply, and site development.

#### 24.7.3.1 Ground Improvement

Ground improvement at shaft locations will initially include wick drains and pre-loading with embankments. Jet grouting or other ground improvement methods may be used to consolidate soil under the shafts and in the TBM approach and exit sides of the shaft.

#### 24.7.3.2 Power

The shaft construction (slurry wall) equipment and support facilities are expected to be operated initially by portable construction power if utility grid power cannot be brought to each shaft site by the time the work will begin. If utility grid power can be provided to the individual shaft locations, the use of onsite portable power for shaft construction activities is eliminated, and electricity from the utility grid is used at these sites.

The TBMs and supporting systems require large, high voltage power supplies. Power for the tunneling activities is expected to be provided through overhead high voltage transmission lines from major power supply network(s). No major onsite generation is expected. Back up smaller local generation is provided for critical systems in the event of power outage. Because power supply reliability is critical to construction operations, the power transmission towers and substations are placed on pile-supported structures at elevations to avoid flooding (except at IF and BT, where they will be protected by new berms).

Due to the lead time for the installation of a major overhead line, it is expected that planned activities are coordinated with the utility companies, with sufficient lead time to allow construction of power delivery systems
prior to tunnel construction. See Section 19, “Power Supply and Grid Connections,” for an overview of power supply options.

24.7.3.3 Mobilization
During mobilization, the contractors will finish the site development. This will include site grading and installation of offices, warehouses, storage and laydown areas, move in and erection of the batch plant, etc. The RTM site will be prepared, and the tunnel spoil removal system installed. Equipment such as cranes, compressors, electrical switch gear, water treatment facilities, loaders, dozers, etc. will be brought to the site.

24.7.4 Shaft Construction
24.7.4.1 Arrangements
Shafts are required along the proposed tunnel alignment to facilitate construction, operation, and maintenance of the water conveyance system. During the construction phase, shafts are used to launch the TBMs to initiate tunnel mining, support their operation, retrieve the TBMs on completion of the tunnel drive, and provide access for TBM repairs. After construction, the permanent shafts are finished to a much smaller diameter (approximately 20 feet) and will provide ventilation, facilitate tunnel dewatering/filling, and provide operation and maintenance access.

The Bouldin Island shafts are launch shafts to support the construction of the tunnels of Reaches No. 5 and 6 and will be used as a permanent shaft after construction. The Intake 3 Junction Structure shaft, the Staten Island and Bacon Island shafts are reception shafts for adjacent tunnel reaches. The Staten Island and Bacon Island shafts will be used as permanent shafts after construction.

If major TBM repairs are needed, contractors will be able to access their equipment from the surface using construction access shafts. These shafts, which are temporary shafts, will be used by the tunnel contractors for TBM repair during construction. These shafts will be located along the tunnel alignment. Once tunneling is complete, the upper portion of the construction access shafts will be removed down to approximately 10 ft. below the ground surface and backfilled to pre-construction conditions. The areas identified for contractor access are shown in Volume 2 and Volume 3.

Safe haven sites have been identified along the tunnel alignments and are identified in Volume 2 and Volume 3. A safe haven does not include a shaft. It is only an area reserved for the tunnel contractor to do deep grouting on the tunnel alignment, if necessary.

The tunnels are assumed to be spaced at a minimum 150 feet centerline to centerline. Shaft pairs will be staggered along the tunnel alignments to maintain a minimum clearance of one shaft diameter between the outside of the two shafts.

All shafts are circular in plan to provide a more efficient structure with thinner walls and less obstructive bracing.

Anticipated shaft and tunnel configurations for the North Tunnels are as follows:

- The shafts are assumed to be 100-foot ID for drive of a 28-foot ID tunnel and 113-foot ID for the Junction Structure at Intake No. 3.
- The finished sizes of the North Tunnel shafts at IF are 28 and 40 feet ID for Reach Nos. 2 and 3, respectively, to accommodate hydraulic functionality and access for maintenance and repair.
- The finished sizes of the North Tunnel shafts at Intake Nos. 2 and 5 match the adjacent finished tunnel ID (28 feet ID) and provide access for maintenance and repair.
- The finished size of the shaft for the Junction Structure at Intake No. 3 is 40 feet ID to accommodate the required control structure.
The North Tunnel intermediate construction access shafts are assumed to be 85 feet and 75 feet ID for Reach Nos. 2 and 3, respectively, to retain adequate structural integrity after the formation of the tunnel eyes.

It should be noted that the North Tunnel shafts and Junction Structure are subject to water surface elevation of the Sacramento River, and therefore must be designed to accommodate internal water pressure, even when completed at elevation 32.2 ft.

For the Main Tunnels, the temporary size of the shafts during construction is set by size of the tunnel to be constructed. These shafts are assumed to be 113 feet ID for launch or 100 feet ID for reception of a 40-foot ID tunnel.

The construction access shafts are assumed to be 85 feet ID during construction to retain adequate structural integrity after the formation of the tunnel eyes. The construction access shafts will be utilized during construction and backfilled to preconstruction condition as discussed above.

The temporary construction works are not required to design for extreme flood events with recurrence intervals of 200 years. However, the finished permanent pad for the tunnel shafts needs to be protected against flooding due to failure of levees and to meet hydraulic requirements. Based on the information provided in Section 11, the finished shaft area pad elevations are approximately 32 to 34 feet msl, except for BT, which is approximately 25 feet msl. These elevations include 8 feet of additional elevation to account for wind-wave run-up (5 feet) and freeboard (3 feet) as defined in Section 3, “Overview of Conveyance Option.” Because the tunnel shafts are constructed within the footprint of the shaft area pads (except at IF), and because the permanent shafts and pads must be protected from 200-year flood events, it is assumed that the final shaft area pad elevations are developed during advance work at each shaft construction site and not raised at a later date, with the exception of the shafts at the IF. The shafts at the IF are constructed at near existing site grades and final site grades are developed in conjunction with final IF inlet and outlet facilities.

The final elevations of the finished permanent pads will be evaluated during preliminary design and might be set lower than the elevations provided in Table 3-4. The permanent elevated pad perimeters are assumed to extend to 75 feet from the outside of the shafts to facilitate heavy equipment access for maintenance and inspection. As the existing ground elevations are significantly lower than the final planned elevations, the pad fills slope down to the adjacent existing site grades at an inclination of between 3 horizontal to 1 vertical (3H to 1V) to 5H to 1V.

Because these permanent pads represent significant earthwork, their footprints have been minimized, and all temporary ancillary facilities for construction are anticipated to be located outside the raised pad within the defined work areas. Footprints of these pads may be further reduced in preliminary design. These site facilities are assumed to be constructed at grade and protected by existing secondary levees as indicated.

Fill quantities required for the tunnel shaft area site pads and RTM disposal quantities generated at these sites are listed in Table 24-2.
Table 24-2: Tunnel Shaft Site Pad Fill Area and RTM Disposal Acreage for North and Main Tunnels

<table>
<thead>
<tr>
<th>Site</th>
<th>Work Area Site Pad Fill Volume (cy)</th>
<th>In-Place RTM Volume (cy)</th>
<th>RTM Disposal and Top Soil Storage Area⁽¹,²⁾ Site (acres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>North Tunnel Shaft - Intake No. 2 (RTM and launch shaft)</td>
<td>-</td>
<td>430,000</td>
<td>52</td>
</tr>
<tr>
<td>North Tunnel Shaft – Intake No. 3 (Junction Structure\Reception Shaft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>North Tunnel Shaft – Intake No. 5 (Reception Shaft)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>IF : North Tunnel and Main Tunnel (RTM and Launch shafts)</td>
<td>-</td>
<td>7,315,000</td>
<td>330</td>
</tr>
<tr>
<td>Staten Island Reception Shafts</td>
<td>429,000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Bouldin Island Launch Shafts</td>
<td>640,000</td>
<td>11,356,000</td>
<td>775</td>
</tr>
<tr>
<td>Bacon Island Reception Shafts</td>
<td>141,000</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Byron Tract Forebay (RTM and Launch Shaft, and BTF Connection Channels)</td>
<td>900,000⁽³⁾</td>
<td>11,555,000</td>
<td>760</td>
</tr>
<tr>
<td>Totals</td>
<td>2,110,000</td>
<td>30,656,000</td>
<td>-</td>
</tr>
</tbody>
</table>

¹ Assumed 1 foot of topsoil stripped and stored.
² Disposal areas filled 6 to 10 feet high for all sites except for BTF and Glanville Tract, which will be 10 to 15 feet high.
³ Fill pad volume is for BTF launch shaft.

Based on the current geotechnical data, it is assumed that a slurry wall system is required to construct the temporary shaft lining. Since the shafts will be constructed down from the permanent pad level, heavy construction equipment for the slurry walls will be supported on various depths of saturated peat below the pad. Special consideration to mitigate ground water during shaft construction shall be designed. Ground freezing or other support system may be evaluated upon further study during preliminary design, when additional geotechnical data is available.

Liquefaction of shaft sites is possible during the design seismic event. Liquefaction might result in settlement of the pad fill and surrounding ground. Further studies should be conducted to determine if the placement of the pad fill and in-situ soil densification to strengthen the existing soils are required to increase their resistance to liquefaction.

24.7.4.2 Junction Shaft
At Intake No. 3 of the North Tunnels, the Junction Structure combines 3,000 cfs from Intake No. 2 with 3,000 cfs at Intake No. 3 to deliver 6,000 cfs to IF. The preliminary shaft size is 113 feet ID, with final size to be determined during preliminary design.

24.7.4.3 Temporary Lining
Soil and groundwater conditions and the depths of the shafts require specialized shaft construction methods to provide stable ground support, to avoid ground loss and settlement of the ground around the shafts, and to prevent invert blowouts. The shafts will be designed to withstand external ground loads and groundwater to maintain stability and to provide sufficient structural support during TBM launch and reception. Temporary depressurization of ground water may be required during the construction of the shaft.

Potential shaft construction methods include overlapping concrete caisson walls, panel walls, jet-grout column walls, secant piles walls, slurry walls, precast sunken caissons, ground freezing and other technologies. For the purposes of this CER, it is assumed that slurry wall or cutter soil mixing techniques are used to construct the shafts; with the slurry walls formed using a series of interlocking reinforced concrete panels. The anticipated
panel size is estimated to be 8 feet long and 5 feet thick. The toe of the diaphragm walls extends below the invert level of the tunnel. A concrete base slab will seal the shaft bottom from external ground water.

Slurry wall reinforcement is typically deformed steel bars of 60 ksi or higher, but in the areas where TBMs enter and exit, a special break-in/break-out section needs to be constructed as an integral part of the shaft. Fiberglass reinforcement can be used at the point of entry/exit. Due to the width and length of cages required, it is expected that bars are delivered to the site and the cages fabricated at the shafts. When the concrete is placed in the slurry wall panels, a controlled continuous concrete placement is required. After the panels are formed, the material within the shaft can be excavated. Depending on the ground conditions and contractors’ selected techniques, excavation within the slurry wall shaft can be carried out using dry or submerged methods. For this CER, it is assumed that shafts are excavated in the wet.

24.7.4.4 Base
The shaft is excavated to approximately 30 to 50 feet below the invert level of the tunnel, and a concrete base is placed underwater using tremie techniques. It is expected that this is a reinforced mass concrete plug to withstand ground water pressure, with optional relief wells to relieve uplift pressure during tunnel construction.

24.7.4.5 Tunnel Eyes
The launch and reception of the TBMs require that large openings be created in the slurry walls. To maintain structural stability, it is necessary to provide additional structural support. This is expected to be provided by the construction of a reinforced concrete buttress or frame structure within the shaft. A thrust ramp support frame anchored to the shaft will be constructed inside the shaft to launch the TBM. Ground treatment of a zone outside the shaft slurry wall at each tunnel eye is required to stabilize the soils and reduce water and soils inflows to the shaft.

24.7.4.6 Tunnel Excavation Methods
The tunnels are constructed using closed-face pressurized soft ground TBMs in alluvial soils (soft ground) at depths greater than 100 feet with high groundwater pressures and earth pressures. Pressurized face mechanized TBMs include earth pressure balance (EPB) machines and slurry pressure balance (SPB) machines.

EPB machines hold the excavated tunnel spoils in a pressurized chamber behind the cutter head. This chamber 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 particularly suitable for tunneling in silts and clays. In granular materials, conditioners are added to plasticize the excavated material in the pressurized chamber. The modern additives are environmentally compatible and biodegradable, and include water, surfactant foam, polymers, bentonite, or combinations thereof. If dense gravels or cohesion-less (sandy) materials are encountered, the use of EPB machine could be problematic to maintain a constant pressure using the in-situ materials. Other mining methods (such as slurry TBM) should be considered under such ground conditions.

Slurry machines use a highly viscous fluid pressurized in the chamber in front of the cutter head to counterbalance 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. Soils containing a high percentage of fines and clays put excessive demand on the slurry pumping system. A slurry system will likely have a higher power demand than a similar EPB system.

The tunneling methodology for the BT would likely be a closed-face EPB TBM, based on the initial study of limited geotechnical data. The TBM shield supports the excavation until the precast segmental liner is erected at the end of the shield. Proper use of the EPB allows precise control of the amount of material removal at the face, greatly reducing the potential of surface settlement.
It is assumed that the tunnels are driven from the shafts without forward starter launch chambers or tail tunnels. The ground outside of the shaft on the tunnel alignment is improved to facilitate TBM drives (and receptions) through stable ground with low water inflow.

The RTM-handling system is likely to consist of continuous conveyor belts and a screw auger to transport the RTM to the ground surface. The RTM handling then consists of stockpiling RTM at the ground surface, with dewatering or drying of the RTM, and subsequent transfer of the solids to disposal areas. Transfer to disposal areas might be handled by conveyor, wheeled haul equipment, barges, or a combination of these methods.

24.7.4.7 Tunnel Support

Based on early project research and planning, a single-pass tunnel liner system is chosen to balance water conveyance requirements, project schedule, and construction cost. Coupled with modern TBM technologies in the anticipated ground conditions, the tunnel liner system will consist of precast concrete segmental liner with bolted-gasketed joints. The segmental liner will be designed to support external earth pressures; groundwater pressures; internal operating pressures; seismic loads; and construction loads due to handling, erection, and thrusting of the TBM. The segments are bolted together at the circumferential and longitudinal joints. The finished ring formed by the segments is smaller than the excavated tunnel cylinder, so the annular space between the segmental ring and the ground will be backfill-grouted to provide full contact for support. The backfill grout is typically injected through the tail shield of the TBM, which provides full circumferential liner support to ensure successful performance of the tunnel system. This lining system also minimizes impact to groundwater during construction and operation, as all concrete joints are sealed using high performance gaskets.

To minimize ground effects of one tunnel on an adjacent tunnel during parallel tunnels construction, the clear distance between adjacent tunnel bores is assumed to be two tunnel diameters (or 150 feet tunnel center to center). This is a conservative assumption because of insufficient geotechnical data to justify a closer spacing at the current study phase. For the 40-foot ID tunnels, it is anticipated that a 9-piece ring configuration would be used, with segment thickness of 20 inches minimum. The segments (7,000 psi minimum compressive strength) will be cast and steam-cured in concrete segment plants under strict quality control measures and delivered to the tunneling sites. Reinforcement will consist of both high strength steel reinforcement (up to 80,000 psi) and steel fiber for permanent ground loads and construction handling loads. Steel reinforcement will increase segment strength and durability and provide crack control.

Under the single-pass liner design, a typical joint between segments will be composed of gasket material to seal against water seepage and alignment bolts for tunnels subject to compression load only. Given the hydraulic grade line and ground cover of the tunnels, net tension where the internal pressure exceeds the external pressure (soil and water) is expected. If the segment ring is subjected to internal tension, special positive connections across the joint and tension reinforcement are necessary to transfer the tensile force throughout the segments. In general, however, a bolted-gasketed tunnel liner system is designed for compressive ring forces and is seldom subject to net tension. It is important that testing and analysis are conducted during preliminary and final design phases to optimize the tunnel liner system to resist the tension force.

In addition to strength requirements, leakage control through the liner is essential to ensure liner performance. Excessive leakage through the liner could lead to potential soil erosion, hydraulic fracturing and loss of liner support. Water leakage from the tunnel to the surrounding area also translates to economic loss. The leakage can be mitigated by a properly selected high-performance gasket, concrete mix design of long-term durability, supplemental concrete admixtures to increase water tightness, and uniformly-distributed reinforcement and steel fibers for crack control. It is anticipated that a PVC T-lock liner will not be required, and the PVC liner could complicate the tunnel construction and long-term operation. Once detailed geotechnical data is available, the segment liner will be designed to limit water leakage by considering surrounding soil-liner interaction and soil permeability.

For the net internal pressure design of the liner during conceptual phase, the external ground water pressure is assumed to be at elevation 0.0 (MSL) along most of the alignment. Occasionally, lower ground water elevation
may occur due to local conditions. The exact ground water elevations will be determined along the alignment during preliminary design following geotechnical exploration.

The combined pumping plant is located at BTF, with control gates at each river intake. Using results from a preliminary hydraulics study that considers both steady state and surge conditions (see Appendix D), the maximum HGL elevations are summarized below. System hydraulics will be further refined and analyzed during preliminary and final design, and the tunnel liner will be designed for all applicable load cases based on results of accepted hydraulics and geotechnical criteria.

- Static shut-in condition (for all tunnels): HGL = 15 feet of net internal pressure
- Surge condition (North Tunnels @ +0.5 feet): HGL = 15 + 0.5 = 15.5 feet of net internal pressure
- Surge condition (Main Tunnels @ +5 feet): HGL = 15 + 5.0 = 20.0 feet of net internal pressure

Given the net internal pressures, several studies (Jacobs Associates, 2012; CH2M Hill 2014) were conducted to provide alternative tension-resisting elements in the tunnel liner. Such alternatives include effective ground overburden, high strength bolts, shear dowels, post-tensioning system, ferrous push-fit connectors, and proprietary joint connectors. A more detailed evaluation regarding the alternative join anchorage system is included in Appendix I and J.

The conceptual joint design utilizes a high strength bolt connection system as shown in the Concept Drawings (Volume 2), based on past performance in tunneling projects such as the San Diego Bay Outfall Tunnels. Once detailed geotechnical data is available, the following alternatives will be considered (separately or in combination) in the preliminary and final design phases:
- Effective ground overburden to resist internal pressure.
- High strength tension bolting for high tension load case.
- Shear dowels for light to moderate tension load cases.
- Other mechanical lock-fit connections (if applicable).

The tunnel liner system will be designed for all the following load cases to ensure reliable performance during the minimum 100-year design life of the system:
- Full external ground load and external ground water pressure.
- Net internal pressure (difference between internal hydraulic pressure and external ground water pressure). Ground overburden to counteract the internal pressure is ignored at this conceptual phase but will be considered during preliminary and final design once detailed geotechnical data is available.
- Earthquake design – Finite element model on ground-tunnel interaction based on Maximum Considered Earthquake (MCE) events.
- Segment handling loads such as lifting, hosting, TBM pushing.
- Leakage control based on acceptable performance criteria.
- Construction, lifting and transportation loading

24.7.4.8 Precast Segment Plant and Yard

Multiple precast segment plants will be required to produce tunnel segments for this program. The size of each plant is dependent on the total number of segments required and the schedule for production, but it is likely that plants will require approximately 10 acres for offices, materials storage, concrete batch plant, and casting facilities. Segment storage space is dependent on contractor means and methods which include tunneling schedule, on-site segment storage, segment production schedule, segment delivery schedule and transportation method (barge, truck or rail) and segment fabrication facility storage space. The total acreage required could be several times the space required for the batch plant and casting facility.
The current assumption for the segment casting facility is that it will not be located at the tunnel construction site and that tunnel segments will be delivered from off-site facilities. It is also assumed that only limited storage of segments is onsite to reduce the size of the working site required.

### 24.7.4.9 Logistics

The TBM consists of a front shield section, plus additional trailing gantries carrying support equipment. Although the shield is transported in pieces, the size and weight of the pieces are still substantial. Provisions have been made for the delivery of the TBM cutterhead, front shield, and trailing gear by barges on the nearby waterways or by roadway. Final delivery decisions will be made by the tunnel contractors.

### 24.7.4.10 Excavated Material Disposal

The conceptual estimated volume of RTM to be disposed from the tunnels and shafts is approximately 30.7 million, assuming bulking factor of 1.1. The excavated material is saturated and might be plasticized with earth pressure balance (EPB) foam or soil conditioner during excavation (which does not preclude its reuse if biodegradable additives are employed). This RTM is spread on-site for draining and drying prior to placement in the spoils areas. Disposal of the decant liquids requires permitting in accordance with current National Pollutant Discharge Elimination System (NPDES) and Regional Water Quality Control Board regulations.

Temporary and permanent waste stockpiles and spoil areas require drainage and erosion control measures. The disposal method for excavated material is currently assumed to be raising the ground level adjacent to the construction sites within the footprint of the designated RTM disposal zones. It is anticipated that the existing 6 to 24 inches of topsoil (12 inches is assumed) is stripped and stockpiled and used for RTM pile cover following the placement of the tunnel spoils.

The maximum anticipated height of tunnel spoil placement is currently planned at 6 to 10 feet for all sites except the BTF and Glanville Tract, which are planned at 10 to 15 feet. RTM disposal areas have been identified to allow placement of anticipated volumes of spoil to no greater than this height. A preliminary analysis of settlement resulting from placement of up to 10 feet of this material indicates that long term settlement of up to 5 feet could be expected, depending on the specific site geological condition. Furthermore, the analysis indicates that the RTM placement would not cause settlement of the adjacent levees, provided it was not placed within approximately 150 feet of an existing levee. Additional geotechnical analyses should be performed to confirm these preliminary findings upon completion of supplemental site-specific geotechnical exploration and testing.

### 24.7.5 Conceptual Tunnel Construction Sequencing

#### 24.7.5.1 Shaft Sequencing

The number and size of slurry wall shafts to be constructed for this project in the timescale proposed is significant and might be impacted by availability of specialty equipment and personnel with adequate experience to use the equipment to the standard required for these deep shafts. Further study is required into current and likely market capacity for slurry wall construction. No detailed evaluation of these constraints has been conducted, so no adjustment has been incorporated into the conceptual construction schedule for this constraint.

It is expected that the adjacent shafts on each tunnel launch or reception site will be operated by one contractor and are constructed concurrently to make efficient use of the bentonite processing plant and site concrete batching facilities.

Typical sequencing for shaft construction once the shaft working pad area has been completed, including substantial completion of anticipated ground settlements are shown in the table below:
Table 24-3 Construction Elements for Tunnels

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Preliminary Work</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Mobilization of slurry wall contractor</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Piling for guide walls</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>Excavate and support for guide walls</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Construct concrete guide walls</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>Construct diaphragm walls</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Excavate soil from interior of shaft in the wet.</td>
</tr>
<tr>
<td>8</td>
<td>7</td>
<td>Place rebar and pour tremie slab in the base of the shaft</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>Dewater shaft and pour working slab</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>Form and pour TBM breakout walls</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>Form and pour TBM thrust rings in launch shafts</td>
</tr>
<tr>
<td>12</td>
<td>10,11</td>
<td>Begin TBM assembly or disassembly</td>
</tr>
</tbody>
</table>

24.7.5.2 Tunneling Sequencing

North Tunnel Reach Nos. 2 and 3 are assumed to be driven from the IF to the reception shafts at Intake Site No. 3 (Junction Structure) and at Intake Site No. 5, respectively, to limit conflicts at these intake sites and reduce temporary tunneling infrastructure requirements. North Tunnel Reach No. 1 is driven from the shaft at Intake No. 2 to the Junction Structure at Intake No. 3.

The direction of drive of the Main Tunnels was selected so that the Bacon Island and Staten Island shafts are only reception shafts due to their relative remoteness from a highway.

Based on the required construction schedule and using industry average tunneling rates for the anticipated ground conditions, including contracting considerations, 12 TBMs are assumed to be required for the full program. The lead time required from procurement (NTP) to design, build, and ship new machines on-site is approximately 1.5 years, with an additional 5 months provided in the schedule to account for fabrication at the site and a small delay contingency.

The number and size of TBMs required for this project in the timescale proposed is significant and might be impacted by availability of specialty equipment and personnel with adequate experience to use the equipment to the standard required. Further study is required into current and likely market capacity as this will affect the schedule and procurement strategy.
Table 24-4: Tunnel-boring Machines

<table>
<thead>
<tr>
<th>Reach</th>
<th>Number of Tunnel Bores</th>
<th>Outside Cut Diameter (feet)</th>
<th>Number of TBMs Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 and 3 (North Tunnels)</td>
<td>Single</td>
<td>31</td>
<td>2^a</td>
</tr>
<tr>
<td>2 (North Tunnels)</td>
<td>Single</td>
<td>44</td>
<td>1</td>
</tr>
<tr>
<td>4 through 7 (Main Tunnels)</td>
<td>Dual</td>
<td>44</td>
<td>8</td>
</tr>
<tr>
<td>8 (South Tunnels)</td>
<td>Dual</td>
<td>44</td>
<td>1</td>
</tr>
</tbody>
</table>

^a If driven concurrently, two TBMs assumed for the conceptual tunnel construction schedule

Note:

TBMM = tunnel-boring machine

The launch of a TBM from a shaft into water-bearing soft ground requires pre-excavation ground treatment near the shaft to maintain stability and water exclusion. The TBM is launched with limited backup equipment. Then, once sufficient tunnel has been constructed, the trailing equipment gantries can be erected. Once the gantries have been installed and the shaft logistics are rearranged, tunneling becomes more efficient and progress is anticipated to be faster after initial startup.

The tunneling advance rate depends on factors related to the actual ground conditions encountered, such as need for ground improvement; soil abrasiveness; presence of gravel, cobbles, and boulders; groundwater pressures; and variability of ground conditions, as well as logistical factors (machine utilization, capacity of the RTM conveyance system, and number of working shifts).

For the program cost estimate, a TBM advance base rate of 37.9 feet per day was assumed for the 40ft ID tunnels before adding delay times and 52 feet per day for the 28ft ID tunnels before adding delay times. The production cycle of a soft ground TBM involves an excavation cycle, a support ring erection cycle, a supply cycle and a backfill cycle. The excavation cycle and the support erection cycle are linear and cannot happen simultaneously. During the excavation cycle, the machine pushes against the previously installed support ring to advance forward. Once the machine has moved forward the length of one support ring the excavation stops. The thrust jacks are retracted, and the next support ring is erected. The cycle starts over again.

The supply cycle and the backfill cycles can happen concurrently with the excavation cycle or the segment ring erection cycle. Normally, the backfill cycle or filling the annulus between the soil and the support ring is performed during the excavation cycle due to congestion in the area where the support ring is being erected.

The base advance rates include time for normal maintenance including face cutter change times. This is frequently done at a fixed time every day. An allowance based on the contractor’s history is included for unplanned repairs in the entire tunneling equipment chain, underground or on the surface. The time for the advancement of the utilities, air, water, electrical, rail system and conveyor system is included in the base production rates.

Delay times that reduce the overall base production include such activities as initial learning curve, installation of the trailing gear during the initial tunnel drive, entry and exits from the various shafts, major TBM repairs at the safe haven sites and other intervention zones, etc.

When two TBMs are being driven for a particular reach, it might be required that a minimum distance be specified to stagger the drives. This will reduce the potential ground support loss due to proximity of adjacent tunneling operations. Additionally, due to the number of TBMs driving or operating concurrently on this project, it is expected that there are constraints imposed by resource availability (labor, equipment, and materials) between the contracts. No detailed evaluation of these constraints has been conducted, so no adjustment has been incorporated into the conceptual construction schedule for this constraint.
Further constraints exist because a TBM might not be able to enter a reception shaft to complete its drive if another TBM using the same reception shaft has not been completely removed. These situations create physical, contractual, and health and safety interfaces that must be addressed during future project development. Proper sequencing of the various contracts should minimize this problem.

### 24.8 Forebays

#### 24.8.1 General Construction Methodology

- The IF may be excavated in the dry using open-cut methods by excavators and haul trucks or scrapers. Due to high water table, a slurry wall around the perimeter along with dewatering inside the slurry walls may be needed during the construction period.
- Excavation for the BTF will primarily be under the embankment foundations and will be done with excavators and truck or scrapers.
- A 50-ft. deep slurry trench will be constructed in the dike foundations using slurry trench excavators (chain belt excavator or grabs). A bentonite water slurry will keep excavation open as the excavator advances. Excavated soil (or suitable borrow) will be mixed with bentonite and put back in the trench following the excavator advance. Slurry de-sanders, bentonite-soil mixing units, retention ponds, and support equipment will be required.
- The soil embankment is constructed using bulldozers, compactors, scrapers, excavators, loaders, haul trucks and graders in compacted lifts (e.g., 12-inch lifts) to design specifications. It may be necessary to dry otherwise suitable fill material using 2 to 5 acre drying beds.
- Inlet, Outlet, gates and control structures are constructed inside cofferdams.
- New bypass channels are used for passing Banks PP and Jones PP flows around the new control structures constructed in their existing approach channels. Control structures are built after water is flowing through diversions or structures are built half at a time to allow water to continue to flow down existing channels.
- The Jones PP and Banks PP must be maintained fully operational during construction, except for a few short durations (less than 1 day) preplanned outages.
- The soil or geologic conditions in the forebay location dictates the depth of the excavation, as well as the type of foundation used.
- Suitable ground improvement methodologies are to be determined during preliminary engineering and final design.
- Forebay areas are shared with tunnel construction. Suitable construction sequencing and contractor site sharing provisions must be developed.
- Suitable engineered fill materials are needed for embankment construction. Possible borrow areas have been identified and are shown on drawings.
- Tying into the existing aqueduct system requires completion of construction with minimum interruption.
- Large areas have been identified and for permanent storage of spoils. These areas are shown on the drawings.
- Access between the Skinner Facility and Banks PP must be maintained during construction of the channel between the Banks Approach Channel and California Aqueduct.
- The Byron Tract Forebay is laid out in such a way that most of the facilities can be constructed while the existing conveyance system to the Banks PP and Jones PP remain operational. However, short duration shutdowns (less than a day) may be required during construction to tie new control structures to the existing pumping plant approaches and when the new approach canals are connected to the existing approaches.
24.8.2 IF Embankment Construction Areas

The ground surface elevation at the IF site averages elevation 0 feet. The IF interior ground is fairly uniform and is will be graded to a -20 elevation. The outer perimeter of the IF embankment, with a top elevation of 32.2 feet, is constructed around the inlet and outlet shaft sites to provide a work area protected from flooding. The new embankments for IF are constructed by excavating 1.9 million CY for the forebay and under the embankment foundations down to suitable material and then installing the slurry cutoff wall. After the cutoff wall is completed, the embankments are constructed of 2.1 million CY of compacted fill to the desired height. Dewatering is required for excavation operations. Much of the excavated material is expected to be high in organics and unsuitable for use in embankment construction and requires disposal.

Once the embankment is complete around the IF, including the work areas for the Inlet and outlet shafts, the shafts are completed. These shafts first will be launch shafts for associated tunnels leading north to the intakes and South to the BTF. Tunneling operations will be from inside the IF protected by the levee from the 200-year flood. The IF cannot be completed until tunnel operations are complete.

After the tunnels are constructed, the new embankments for IF are completed by placing compacted fill inside the previously constructed perimeter embankment, excavating the remaining embankment foundations down to suitable material, and dewatering. Embankments are constructed of compacted fill to the desired height.

24.8.3 BTF Embankment Construction

The BTF is created by constructing a levee around the perimeter creating a 1,000+ acre impoundment. The invert will be at natural ground varying elevation from -9.0 to +4.0 feet. The levee is constructed around the pump stations and associated facilities in its NW corner, and the pad area is filled to provide a protected pump station work area at elevation +25.0.

The new embankments for the BTF is constructed by excavating the embankment foundations down to suitable material, dewatering, and installing the slurry cutoff wall. Approximately 1.9 CY of excavation is hauled to spoil and 7.7 million CY of earth fill is required for embankment. The required embankment material is either imported from BTF Channel excavations or borrowed from within the limits of the forebay to the extent possible or from borrow sites (see Section 21, “Borrow Sites”). Dewatering and moisture conditioning of onsite soils is required. Drying bed for conditioning fill material will be constructed inside the forebay.

Construct a BTF spillway concurrent with embankment construction. Spillway construction starts with excavation to subgrade. Foundation fills are constructed in lifts where possible under footings and slabs. Footings and slabs are constructed first where possible. Place walls after footings and slabs. RCC material is placed in-between walls. Any remaining concrete is placed after the RCC is complete and fills are made under remaining slab locations. Backfill against walls in lifts to finished elevations.

24.8.4 New Approach Channel to Banks and Jones Pumping Plants

The new approach channel system connecting BTF to the Banks PP and Jones PP is constructed using a concrete lined open cut channel section. Natural ground elevation varies from +20.0 to +30.0 for most of the canal length. The canal invert is at elevation -10.0. The canal has been routed to provide space for open cut construction. Ground water is expected to be at elevation -5.0 to -15.0. It is expected that a significant portion of the excavated material will be sandy, clayey sand, and may be suitable for BTF levee embankment construction and requires minimum disposal (see Section 22, “Spoils Disposal Sites”). However, material excavated from the new Connection Channel to Jones PP would have to be hauled across the California Aqueduct. A temporary construction bridge is suggested for this purpose, plus hauling concrete and other materials from the batch plant, RR siding, and laydown areas.

24.8.5 Construction Methodology:

- Excavation may mostly be in the dry using excavators, scrapers, trucks, and conveyors. Some excavation below El. 0 may require dewatering with well points or sumps.
- The canals will be concrete lined using slip form pavers.
• It is expected much of the material will be suitable for Byron Tract Levee embankment. Drying on drying beds may be required.

24.8.6 IF and BTF Structures
The IF and BTF inlet structures, outlet structures and emergency spillway structure are constructed using conventional reinforced-concrete methods on pipe pile foundations. The IF structures and BTF structures are constructed within sites surrounded by new earthen embankments. Some groundwater control may be required at these locations. The shallow foundations beneath adjacent concrete structures are improved using jet grouting to elevation -50.0 feet within the footprint of the structure and beyond the structures by a distance of approximately 25 feet.

24.8.7 Banks and Jones Intake Channel Control Structures
The new control structures in the existing Banks and Jones PP approach channels are constructed using conventional reinforced-concrete methods. The structures are constructed in sheet pile coffer dams with tremie seals and dewatering typical of cofferdam construction. The approach channel sections are constructed with braced sheet pile walls and a tremie concrete bottom slab. The shallow foundations beneath the concrete structures are improved using jet grouting to elevation -50.0 feet within the footprint of the structure and beyond the structures by a distance of approximately 25 feet. A temporary bypass channel is constructed around each structure to allow normal operations to continue during construction.

24.8.8 Major Construction Elements and Sequence of Construction
Presented in Tables 24-5 and 24-6 are major construction elements for completing the IF and BTF construction. The elements and sequence presented is one possible approach and may be revised to fit actual construction contracting and implementation.

Table 24-5 Major IF Construction Elements and Sequence of Construction

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Site development preparing the site for construction, including necessary grading and mitigation measures such as import of soil for building pads and constructing environmental protection including detention pond berms; setting up the construction trailer area; and temporary or permanent relocation of power lines and establishing a power substation or on-site power generation to support construction. It will also include off-site work, including road and bridge improvements required for site construction access and providing a potable water supply as necessary to support operations. Site development will include barge offloading facilities, batch plant, and craft workshops.</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Clear &amp; Grub the IF footprint. Work will include demolition of any existing facilities, re-routing or constructing new irrigation, power lines, communication line, farm roads and other facilities or utilities to minimize any disruption to properties adjacent to the Forebay. Work will also include constructing drainage, settlement ponds, site roads, and environmental protection facilities such as silt fences.</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Excavate the unsuitable material below the levee foundation. Haul the remainder to designated spoil areas. Excavation equipment includes scrapers, excavators, bulldozers, off-road and on-road trucks as deemed appropriate.</td>
</tr>
</tbody>
</table>
Table 24-5 Major IF Construction Elements and Sequence of Construction

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>3</td>
<td>Construct Slurry Trench around the Perimeter of the IF. Construct Slurry Trench around the Perimeter of the BTF. It is expected that most of the foundation of the BTF will be above the water table. The slurry trench will be constructed using long stick excavators, chain excavators or hydro-mills to open the slurry trench. A bentonite slurry will be used to keep the trench open during excavation. The trench once opened to full depth for a length of 100 feet or less will continuously be backfilled with a slurry of bentonite/cement mixed with excavated materials to construct an impervious “wall”. Slurry will be mixed in slurry mixing plants. Excavation slurry will be temporarily stored in retention ponds as the trenching/backfill cycle continuously move forward. It will be necessary to complete the slurry wall and dewater the IF area prior to excavating the IF to El. -20. Dewatering may be done using trenches, sump pumps, well point and/or deep wells.</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Excavate the IF. Usable material may be hauled direct to levee embankment, via drying beds if necessary. Use the same equipment noted in “4” above</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
<td>Complete construction of the IF Levee including embankment around the Inlet and Outlet Shaft sites. The levee embankment will be constructed from excavated suitable material or from approved borrow. It is assumed a substantial quantity of fill material will have over optimum moisture; therefor drying bed will be constructed and used to dry material by diskng and turning prior to placing on fill. The IF levee is constructed to El 32.2 which will protect shaft and tunnel operations against the 200-year flood. Embankment operations use similar equipment as excavation operations, but also include self-loading scrapers to haul from drying beds to embankment and include compaction equipment, rollers, motor graders, and water trucks or water pulls to place material in lifts until finish heights are reached.</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Rip rap slopes using excavators, loaders and trucks as required.</td>
</tr>
<tr>
<td>8</td>
<td>4</td>
<td>Construct ground improvement under the inlet/outlet structure and under shafts and in tunnel breakout zones. Ground improvement under the shaft and in the break-out zone will be by jet grouting. Ground improvement under the intake structure and approach channel will be by jet grouting, dynamic compaction, stone or sand columns, or other methods being considered. NOTE THAT INLET AND OUTLET SHAFTS WILL BE USED TO LAUNCH TBM’S AND FOR MUCK REMOVAL DURING TUNNEL CONSTRUCTION. THEREFOR INTAKE AND OUTLET STRUCTURES CAN NOT BE COMPLETED UNTIL AFTER THE TUNNELS ARE COMPLETED.</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>Construct segment walls for the Inlet and outlet shafts. This operation will use hydro-mills for excavation large cranes for installing rebar, slurry mixing plants, concrete batch plants and concrete pumps. Retention ponds for slurry and construction water will be required. Sufficient area for these operations will be provided within the forebay.</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>Excavate Inlet and Outlet shafts and pour tremie slab and working slab.</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>Form breakout and form and pour thrust rings and launch shaft walls.</td>
</tr>
<tr>
<td>12</td>
<td>11</td>
<td>Assemble TBM and drive tunnel from launch shaft to recovery Shafts (Section 11, Tunnels).</td>
</tr>
<tr>
<td>13</td>
<td>12</td>
<td>Turn Launch Shafts over to IF structures contractor</td>
</tr>
<tr>
<td>14</td>
<td>13</td>
<td>Form, rebar, and Pour Shaft concrete around tunnel elbow</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>Form, rebar, and pour interior shaft walls to intake/outlet invert grade and backfill the annular ring between interior wall and slurry wall. Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>16</td>
<td>15</td>
<td>Excavate as required and drill foundation piles for inlet and outlet structures, fill with concrete. Will require removing segment wall to El. -24. May require well points or deep wells for dewatering.</td>
</tr>
<tr>
<td>Activity</td>
<td>Predecessor</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>Form, rebar, and pour Inlet/Outlet walls and slabs. Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>Install gates and MEP. Metalwork fabrication, machining, assembly, and installation (roller gates, embeds, bulkheads, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.). Miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, inlet/outlet and site power supplies, flow/level/turbidity/limit/torque instrumentation, utility service, etc.).</td>
</tr>
<tr>
<td>19</td>
<td>17</td>
<td>Form, rebar, and pour approach channel slabs and walls, and backfill</td>
</tr>
<tr>
<td>20</td>
<td>18,19</td>
<td>Final completion and IF start-up</td>
</tr>
</tbody>
</table>
## Table 24-6 Major BTF Construction Elements and Sequence of Construction

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Site development preparing the site for the BTF and approach channel construction, including necessary grading and mitigation measures such as import of soil for building pads and constructing environmental protection including detention pond berms; setting up the construction trailer area; and temporary or permanent relocation of power lines and establishing a power substation or on-site power generation to support construction. It will also include off-site work, including road and bridge improvements required for site construction access and providing a potable water supply as necessary to support operations. Barge unloading facilities were included with the Pump Plant section and will be co-used by this operation.</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Clear &amp; Grub the BTF levee footprint. Begin in the area of the pump plant construction pad. Work will include demolition of any existing facilities, re-routing or constructing new irrigation, power lines, communication line, farm roads and other facilities or utilities to minimize any disruption to properties adjacent to the Forebay. Work will also include constructing drainage, settlement ponds, site roads, and environmental protection facilities such as silt fences.</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Excavate the unsuitable material below the North levee foundation/pump plant area and from the surface of the BTF discharge channel and haul to designated disposal areas. Excavation equipment includes scrapers, excavators, bulldozers, off-road and on-road trucks as deemed appropriate.</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>Import fill from the BTF discharge channel or designated borrow and construct the pump plant construction pad (Section 8).</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Turn the Pump Plant pad over to the Pump Plant Constructors</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>Excavate the unsuitable material below the remainder of the levee foundation and from the surface of the new California Aqueduct and Delta Mendota channels, and haul to designated disposal areas. Excavation equipment includes scrapers, excavators, bulldozers, off-road and on-road trucks as deemed appropriate.</td>
</tr>
<tr>
<td>7</td>
<td>3,6</td>
<td>Construct Slurry Trench around the Perimeter of the BTF. It is expected that most of the foundation of the BTF will be above the water table. The slurry trench will be constructed using long stick excavators, chain excavators or hydro-mills to open the slurry trench. A bentonite slurry will be used to keep the trench open during excavation. The trench once opened to full depth for a length of 100 feet or less will continuously be backfilled with a slurry of bentonite/cement mixed with excavated materials to construct an impervious “wall”. Slurry will be mixed in slurry mixing plants. Excavation slurry will be temporarily stored in retention ponds as the trenching/backfill cycle continuously move forward.</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>Construct temporary bridge over the California Aqueduct to facilitate hauling suitable fill material from the Delta Mendota Channel to the BYT. The bridge may be constructed of steel piling supporting steel bridge girders and concrete plank decking. It must be designed to carry the heaviest earthwork truck or scraper anticipated.</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>Construct haul and access roads connecting BTF construction, channel construction, siphon construction, control structure construction, batch plant, laydown and office areas. Include grade separations as required over Byron highway/railroad.</td>
</tr>
<tr>
<td>10</td>
<td>7,8,9</td>
<td>Complete construction of the BTF Levee embankment and channel excavation. The levee embankment will be constructed from excavated suitable material from the California Aqueduct and Delta Mendota Channel excavation or from approved borrow, preferably from inside the BTF. Unsuitable channel excavation will be hauled to disposal. It is assumed a substantial quantity of fill material will have over optimum moisture; therefore drying beds will be constructed and used to dry material by disking and turning prior to placing on fill. Embankment operations use similar equipment as excavation operations, but also may include self-loading scrapers to haul from drying beds to embankment and include compaction equipment, rollers, motor graders, and water trucks or water pulls to place material in lifts until finish heights are reached.</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>Rip rap slopes using excavators, loaders and trucks as required.</td>
</tr>
<tr>
<td>Activity</td>
<td>Predecessor</td>
<td>Description</td>
</tr>
<tr>
<td>----------</td>
<td>-------------</td>
<td>-------------</td>
</tr>
<tr>
<td>12</td>
<td>10</td>
<td>Place concrete lining on Channel slopes and invert over approved bedding material. Place bedding using light dozers. Use slip form concrete pavers. Deliver concrete from a central batch plant with transit mix trucks.</td>
</tr>
<tr>
<td>13</td>
<td>6,8</td>
<td>Construct ground improvement under the BTF outlet and siphon. Ground improvement under the structures will be by jet grouting, dynamic compaction, stone or sand columns, or other methods being considered.</td>
</tr>
<tr>
<td>14</td>
<td>13</td>
<td>Sheet pile supported excavation for the BTF outlet and siphon. Dewater as necessary with sump pumps, well points, and/or deep wells. The bottom of excavation for the outlet will be +/- EL -20 (OG at EL 0) and the bottom of the siphon excavation will be EL -30.0 (OG at EL +25). It is expected that open cut for the for the siphon to EL 0 would be practical, with tied back sheet pile below EL. 0. Excavate with long stick excavators and haul to disposal areas.</td>
</tr>
<tr>
<td>15</td>
<td>14</td>
<td>Drill pipe pile in structure foundations and backfill with concrete. Use 100+ ton cranes with pile leads.</td>
</tr>
<tr>
<td>16</td>
<td>15</td>
<td>Form, rebar, and pour inlet and siphon walls and slabs. Use standard concrete construction methods and procedures. Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>Install gates and MEP. Metalwork fabrication, machining, assembly, and installation (roller gates, embeds, bulkheads, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.). Miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, inlet/outlet and site power supplies, flow/level/turbidity/limit/torque instrumentation, utility service, etc.).</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>Form, rebar, and pour approach channel slabs and walls, and backfill. Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>19</td>
<td>9</td>
<td>Excavate diversion channels around new Jones and Banks Control Structures. NOTE: AS AN ALTERNATIVE TO DIVERSION CHANNELS, CONTROL STRUCTURES MAY BE CONSTRUCTED IN TWO PARTS, DRIVING PILE TO CONSTRUCT HALF THE STRUCTURE PER PART. PART ONE WOULD DIVERT AROUND THE NEW CONSTRUCTION AREA. PART TWO WOULD DIVERT THROUGH THE PART ONE STRUCTURE. ENGINEERING WOULD HAVE TO CONFIRM ADEQUATE WATER FLOW DURING PART TWO.</td>
</tr>
<tr>
<td>20</td>
<td>19</td>
<td>Excavate as required. Will require excavation to EL ~ 32.0 for Banks Control Structure and -22.0 for Jones Control Structure. Excavate inside tied-back sheet pile retaining walls driven from OG approximately EL. 0. Pile lengths over 60 ft. will be required for Banks and 50 feet for Jones. Use 100+ ton crane and pile leads.</td>
</tr>
<tr>
<td>21</td>
<td>20</td>
<td>Drill pipe pile in structure foundations and backfill with concrete. Use 100+ ton cranes with pile leads.</td>
</tr>
<tr>
<td>22</td>
<td>21</td>
<td>Form, rebar, and pour control structure walls and slabs. Use standard concrete construction methods and procedures. Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).</td>
</tr>
<tr>
<td>23</td>
<td>22</td>
<td>Install gates control building and MEP. Metalwork fabrication, machining, assembly, and installation (roller gates, embeds, bulkheads, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.). Miscellaneous electrical (conduit and conductors, cathodic protection, yard and overhead lighting, inlet/outlet and site power supplies, flow/level/turbidity/limit/torque instrumentation, utility service, etc.).</td>
</tr>
<tr>
<td>24</td>
<td>23</td>
<td>Form, rebar, and pour approach channel slabs and walls, and remove sheet pile and backfill</td>
</tr>
</tbody>
</table>
24.9 Levee Construction

This section includes a brief description of the configuration of the facilities in the levee, sequence of construction associated with the levee, and temporary and long-term flood protection features. Additional detail can be found in Appendix B, “General Description of Conceptual Level Construction Sequencing at the California WaterFix Intake Facilities”. Additional details of the pad fill arrangement can be found in Sections 7, “Intakes and Sedimentation Facilities,” and Section 15, “Forebays.”

24.9.1 Sequence of Construction at the Levee

A summary of the construction sequence for activities associated with the levee is presented in Table 15-1. For additional information regarding construction sequencing concepts at the levee, including diagrams, refer to Appendix B, “General description of Conceptual Level Construction Sequencing of California WaterFix Intake Facilities.”

Table 24-6 Major BTF Construction Elements and Sequence of Construction

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>13,18,24</td>
<td>Canal and levee finish out. Include riprap, gravel surfacing, topsoil, planting, erosion control, fencing, and removal of contractor’s construction plant, haul roads and temporary bridges and traffic separations.</td>
</tr>
<tr>
<td>26</td>
<td>24</td>
<td>Final inspection, punch list, and start-up.</td>
</tr>
</tbody>
</table>

Table 24-7 Construction Elements for Levees

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Construct a slurry cutoff wall landside of the existing levee prism for the relocation of Highway 160.</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Construct a well point dewatering system to allow for excavation and placement of new fill slopes for the relocation of Highway 160.</td>
</tr>
<tr>
<td>3</td>
<td>1,2</td>
<td>Excavate and remove peat soils to suitable subgrade beneath the area for the widened levee and relocated Highway 160.</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>Construct soil improvements beneath widened levee area and the fill slopes required for the permanent relocation of Highway 160.</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Construct new fill slopes for the new permanent alignment of Highway 160. Construct portions of the intake box conduits that traverse the fill beneath the new permanent alignment for Highway 160.</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>Construct the roadway for the relocated segment of Highway 160 and route traffic to new permanent alignment on widened levee section.</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Construct a diaphragm wall within the existing levee crest located along and outside of the back wall of the planned new intake structure. Extend the diaphragm wall beyond the length of the intake structure to minimize potential seepage though and under the existing levee.</td>
</tr>
<tr>
<td>8</td>
<td>6</td>
<td>Construct sheet pile portion of cofferdam (river side of existing levee) and training walls for intake structure. Tie the cofferdam into the diaphragm wall constructed on the landside of the levee crest to provide an enclosed area for construction of the intake structure.</td>
</tr>
</tbody>
</table>
24.0 CONSTRUCTION METHODS, SEQUENCING, CONSTRUCTABILITY AND SCHEDULE

24.9.2 Temporary and Long-Term Flood Protection Features

In addition to levee stability and seepage control, the facilities described in this section provide both temporary and permanent flood protection at the intake sites. Temporary flood protection during construction is by a combination of cofferdams, slurry cutoff walls, diaphragm walls, structures, and elevated pad fills for the landside facilities. All of these features, except the landside structures and elevated pads, require penetration of the existing levee prism.

24.9.3 Temporary Flood Protection Features

During construction, the widened and raised levee, the diaphragm wall, and the sheet pile cofferdam provide positive blockage to flood flows on the river-side of the levee. The widened levee provides additional levee prism stability, and the height is expected to be 3 to 4 feet higher than adjacent undisturbed portions of the levee. The cofferdam itself provides positive seepage control via the sheet pile and diaphragm cutoff walls and is designed to resist lateral displacement that might cause levee instability. The elevated pad fills behind the intake structure and the perimeter slurry cutoff wall provide an additional layer of positive flood control for areas surrounding the site.

The cofferdam walls and three positive closure devices provide direct flow barriers for the levee penetrations between the river-side to the land-side of the levee. These include the front sheet pile wall and back diaphragm wall, as well as the gates on the back wall of the intake structure, and the isolation gates within the collector box conduits.

The front wall of the cofferdam is the primary flood protection barrier. Under normal circumstances, no flow passes by the front wall. However, the front wall of the cofferdam is not constructed to full levee height; therefore, during a severe flood, it could be overtopped, and the cofferdam would fill with water.

The diaphragm wall is the back wall of the cofferdam. The diaphragm wall is deep and extends beyond the work area in length to prevent seepage that could cause levee instability. Also, the diaphragm wall is full levee height, so this wall could not be overtopped by any flood that is contained within the levee system. The diaphragm wall also provides a barrier between the river-side and land-side of the levee that is not penetrated for connections to the box conduits until after the back portion of the intake cofferdam is fully backfilled.
Even in the unlikely event that water were somehow to flow through the levee into the landside work area, it should be fully contained inside the earthen sedimentation basins. The elevated pad fills for the sedimentation basins are constructed using low-permeable fill material installed to a level equal to the top of the new levee to provide full containment.

24.9.4 Long-Term Flood Protection Features

Long-term flood protection is provided by multiple means at the completed facilities. These features include an improved and stable levee at the intake sites, under-seepage protection, full containment of any water allowed to flow through the gravity collector box conduits to the landside of the facility, and positive closure devices on the gravity collector box conduits to restrict flow from the river side to the landside.

The widened levee provides a finished levee prism with materials and dimensions that exceed those of the existing levees and meet or exceed the current requirements of the CVFPB and USACE.

The diaphragm wall and slurry cutoff walls at the site provide a positive barrier to seepage at the new levee and from induced seepage through adjacent levees due to activities at the site.

The sedimentation basins, including the fills placed to develop the basins, are installed to a level equal to the top of the new levee and provides full flood containment.

Positive closure gates are provided within the collector box conduits to isolate river flows. If these gates are closed, a positive barrier to flow from the river to the landside facilities is established.

24.10 Byron Tract Pumping Plants

24.10.1 Constructability Considerations

Major construction phases at this site will include site development, shaft construction, TBM tunneling, and pump station construction. Contracting strategies may result in one or more prime contractors being involved in each construction phase. The site development will need to take place before shaft construction and tunneling, and each of these subsequent phases will require large portions of the site to accommodate equipment, material storage, and operations. Pumping plant construction may begin after the start-up phase of TBM tunneling, with careful contract coordination regarding site access. Tunnel construction will include milestone provisions to allow coordinated parallel activities between tunnel contractor and pump plant contractor. The goal is to identify and manage the critical path of each contract in order to minimize the overall construction schedule duration. Major activities include but are not limited to:

- Mobilization and demobilization;
- Contract administration;
- Development of staging and storage areas and construction zones;
- Earthwork, including deep excavation and shoring and bracing;
- Deep ground improvement and dewatering;
- Diaphragm wall and tremie slab construction;
- Trenching and pipeline installation;
- Foundation preparation, stabilization, and footing and slab-on-grade installation;
- Conventional cast- in-place concrete construction involving formwork, reinforcing, placement, and finishing;
- Metalwork fabrication, assembly, installation, and structural framing;
- Electrical equipment (to include Motor Control Center and Variable Frequency Drive for large motor loads);
• Mechanical equipment installation to include large pumps, truck-mounted cranes, large valves (with associated mechanical and electrical controls), flow meters, and miscellaneous civil site and electrical work;

24.10.2 Construction Constraints

• The pump plant is co-located in the same shaft used to launch the TBM for Reach 7 tunnel. The pump plant cannot be constructed until after completion of the tunnel. An alternate concept is to construct a second small diameter service/RTM shaft near the pump plant. Once the TBM has tunneled through the service shaft, RTM handling facilities can be relocated to the service shaft and pump plant construction can begin. This alternate concept along with others will be explored further during preliminary design.

• The Pump Plant construction area is located where natural ground occurs at an elevation of “0” to El. -6.0 feet. The area requires fill to be placed to elevation 20.5 ft. prior to beginning pump plant construction. The sequence of operation would have the fill placed as part of the contract to construct the Byron Tract Forebay and Canal Excavation.

• Byron Highway and railroad must be kept in operation. Construction access from the highway and railroad will require dedicated turnout facilities. In addition, it is recommended grade separations (overpasses) be constructed to separate onsite construction traffic from public traffic and railroad.

• Several high voltage power transmission lines in the area must be kept operational.

• Adverse geotechnical considerations including peat, saturated soils, and significant settlement.

24.10.3 Site Access

• Site access via railroad may be an option. This will require separate agreement with the railroad and construction of a siding and possibly a spur line to site.

• Power supply to the site can be provided by Tracy Substation (WAPA).

• Potable water may be supplied from an onsite water treatment plant or from a potable water line to supply the site from the closest public supply main.

24.10.4 Temporary Facilities

Temporary facilities required for construction will be installed during or directly following site development and will consist of:

• Site controls (fencing);

• Owner and construction contractor office trailer and parking areas;

• Storage and treatment facilities for construction process water and storm water runoff;

• Switchyard (substation) for construction site energy supply; and

• Construction staging areas.

24.10.5 Construction Contract Coordination

Some aspects of the construction site infrastructure are shown on the Surge Shaft/Pumping Plant Plan in the Concept Drawings (Volume 2). During construction of the pump stations, the construction site will be divided to allow separation between the tunnel contractor in the northwest part of the site and the pump station contractor in the southeast part of the site.

Several construction contracts are likely to be utilized to complete the required work for the construction activities described in this section: site preparation which may be part of the Byron Tract Forebay Construction, shaft construction, TBM tunneling, and pump station construction. The first three contracts will require access to the entire construction site, with the tunneling contract relinquishing the southeast portion of the site to the pump station contractor once TBM tunneling support infrastructure is relocated to an access/RTM shaft.
Thereafter, the pump station contractor will have a sufficient, distinct work area. There will be provisions in the contracts to allow sharing of some of the site infrastructure, such as the main site access roads, temporary power substation, and process water and storm water runoff storage and treatment systems.

### 24.10.6 Construction Activities

#### 24.10.6.1 Overview

Major construction activities at this site will include site development, shaft construction, TBM tunneling, and pump station construction.

Contracting strategies may result in one or more prime contractors being involved in each construction activity. The site development will need to take place before shaft construction and tunneling, and each of these subsequent activities will require large portions of the site for equipment, material storage, and operations. Pumping plant construction may begin after the start-up of TBM tunneling, with careful contract coordination regarding site access. Tunneling contract will include milestone provisions to allow coordinated parallel activities between tunnel contractor and pump plant contractor. The goal is to identify and coordinate the critical path of each contract in order to minimize overall construction schedule duration.

#### 24.10.6.2 Site Development

Site development includes preparing the site for construction, including completing necessary grading and mitigation measures (including import of soil for construction pad and detention pond berms); establishing the construction trailer area; and establishing a power substation or on-site power generation to support construction. It will also include off-site work, road and bridge improvements required for site construction access, and providing a potable water supply as necessary to support the shaft and tunnel operations.

**Ground Improvement:** During Site Development subgrade preparation and ground improvement will be required within the footprint area of the pumping plants. This will include stripping of vegetation and installation of instrumentation to monitor ground/soil deformations and pore pressures. In addition, vertical wick or sand drains will be installed to accelerate settlement and mitigate future liquefaction potential. A layer of free-draining sand will be placed over the ground surface after vertical drain installation to provide for lateral drainage. Peat and organic soil layers within significant loading influence zone of structural foundations and fill embankment will also be removed and replaced with engineered fill. If required for continuity with the slurry cutoff walls proposed for the new perimeter Byron Tract berms, ground improvement will include installation of a slurry trench cutoff to elevation -50 ft.

**Earthwork:** Peat and organic soils layers are expected to undergo significant consolidation settlement if they are left in-place to be subject to new loadings. To eliminate consolidation settlement in peat and organic soil layers, these organic deposits under the loading influence zone of new structures and fill embankments will be removed and replaced with engineered fill. Geotechnical drilling and test pits will be required to determine the extent of peat, which may range from approximately 10 to 15 ft., or extend from existing ground surface to about El. -15 to -20 ft. With the groundwater occurring just below the existing ground surface, construction dewatering is required to maintain stable and relatively dry excavation to depths of approximately 10 to 15 ft. At this time, construction dewatering is anticipated to consist of a combination of active dewatering systems (such as deep wells), passive dewatering systems (such as trench drains and sump pumps), and seepage barriers (such as slurry trench cutoff walls).

**Site Facilities:** Site development will include a wheel wash facility, paving of the main access roads to each fill pad, a working all weather crushed rock surface, construction access control (fencing, gates, etc.), and installation of erosion control measures for the entire construction area. An office trailer area and worker parking areas will be prepared.

#### 24.10.6.3 Shaft Construction

Shaft Construction includes the following:

- Dewatering as necessary to support shaft excavation and operation
• Jet grouting or other ground improvement as required to seal the bottom of the shaft and provide base stability
• Installation of the shaft walls
• Excavation and on-site disposal of excavated soil
• Installation of a reinforced-concrete shaft base slab
• Delivery of steel rebar, concrete, and other materials
• Mobilization and demobilization of all necessary equipment
• Construction of the access/muck shaft will be completed during this activity

Dewatering and Deep Ground Improvement: Partial dewatering of deep aquifer zones may be required during construction to improve shaft base stability prior to installation of the base slab. Dewatering wells and associated pump, piping, and discharge water treatment would be installed during this construction activity.

Ground improvement (assumed to be jet grouting) will be required within sand layers below the base of the deep shafts prior to slurry wall construction. Construction area requirements include drill rig operation within the ground improvement area, high-pressure pumps, batch plants and cement silos, a basin for storage of backflow suspension, jet grout spoil drying and handling areas, and miscellaneous storage and equipment areas.

Slurry Wall Construction: Slurry wall construction will have significant site area requirements, assumed to include:
• Operation of one or two hydro-mills per shaft
• Operation of two large cranes for rebar cage lifting and installation
• One or two rebar cage assembly areas
• Slurry processing facilities including slurry mixers, slurry storage tanks, and a slurry de-sander operation (plant and pits)
• Concrete delivery and pumping areas to provide continuous concrete supply during slurry panel construction and later base slab, and final lining
• Process water treatment within storm water treatment area
• Workshop and associated tool, equipment, and storage areas

Shaft Excavation, Base Slab, and Final Lining Construction: Shaft excavation will require multiple cranes and/or conveyors to remove soil from the shafts and associated spoil handling facilities. Construction of the base slab and interior lining will require rebar and formwork assembly and high volume concrete placement. All this work is anticipated to be completed within the area utilized for the slurry wall construction.

24.10.6.4 Pump Station Construction

Pump Station Construction includes all internal structural, mechanical, and electrical elements within the pumping plant/surge shafts and associated surface facilities including the surge discharge channels and outlet splash basins. During this activity, earthwork would be primarily limited to excavation for the surge channels, the outlet splash basin, and electrical tunnels connecting the substations to the electrical building and then to the pump station. This activity also includes completion of the switchyard (likely in coordination with the electrical power company), the electrical/control building, and all power and control equipment and connections, and start up and testing.

Ground improvement and pipe piles will be constructed below the splash basin and surge discharge channel. Pump plant construction, surge channel, and splash basin construction will then continue vertically to completion of the dome roof.
24.10.6.5 Site Completion
Site Completion includes completing the site to its final configuration, allowing for long-term tunnel access and maintenance. This will include final grading and installation of slope protection for the BTF embankment dam. Associated activities will include backfilling of detention ponds and final site grading, installation of permanent drainage and erosion control measures, and demobilization of all equipment and materials.

24.10.7 Major Construction Elements and Sequence of Construction
Presented in Table 24-8 are major construction elements for completing the Pump/Surge Station and Tunnel, and the sequence of their construction. The elements and sequence presented is one possible approach and may be revised to fit actual construction contracting and implementation.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>—</td>
<td>Site Development</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>Ground Improvement and Segment Wall Construction for Launch (Pump/Surge) Shaft</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>Excavate inside launch shaft segment walls and pour tremie slab</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>Pump Water from launch shat and pour work slab (foundation slab)</td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>Form breakout and form and pour thrust rings and launch shaft walls.</td>
</tr>
<tr>
<td>6</td>
<td>1,2</td>
<td>Ground Improvements and segment wall construction for the access/muck shaft located approximately 300 feet north of launch shaft.</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>Excavate access/muck shaft and pour tremie slab and working slab.</td>
</tr>
<tr>
<td>8</td>
<td>5,7</td>
<td>Assemble TBM and drive tunnel from launch shaft through access/muck shaft. Relocate muck extraction equipment to the access/muck shaft (see 7.2.2.4 above)</td>
</tr>
<tr>
<td>9</td>
<td>8</td>
<td>Turn Launch Shaft over to Pump Station contractor</td>
</tr>
<tr>
<td>10</td>
<td>9</td>
<td>Form, rebar, and Pour Shaft concrete around elbow up to El. -64.</td>
</tr>
<tr>
<td>11</td>
<td>10</td>
<td>Form, rebar, and Pour Pump Station/Surge Shaft walls below El. -5</td>
</tr>
<tr>
<td>12</td>
<td>11</td>
<td>Drive Pile and excavate for outlet splash basin and surge discharge channel construction. Will require removing segment wall to El. -5. Construct ground improvements and drive foundation piles.</td>
</tr>
<tr>
<td>13</td>
<td>12</td>
<td>Form, rebar, and pour Pump Plant walls above -5 and form and pour operating deck.</td>
</tr>
<tr>
<td>14</td>
<td>12</td>
<td>Install piping, pumps to splash basin, form and pour splash basin concrete, and backfill</td>
</tr>
<tr>
<td>15</td>
<td>12</td>
<td>Form, rebar, and pour surge channel slabs and walls, and backfill</td>
</tr>
<tr>
<td>16</td>
<td>13</td>
<td>Form, Rebar, and pour pump station walls above operating deck</td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>Install overhead cranes and erect pump station roof</td>
</tr>
<tr>
<td>18</td>
<td>17</td>
<td>Install pump station pumps, valves, gates, and associated MEP</td>
</tr>
<tr>
<td>19</td>
<td>9</td>
<td>Construct permanent pump station office, warehouse, and MCC/Electrical Building</td>
</tr>
<tr>
<td>20</td>
<td>18,19</td>
<td>Final Completion and station start-up</td>
</tr>
</tbody>
</table>
24.11 Channels

There are two open channels on the project, 1) 2,800 ft. channel connecting the South Tunnel Outlet Structure to existing Banks Pumping Plant Intake Channel designed for a flow of 10,670 cfs and 2) 4,815 ft. channel connecting the South Tunnel Outlet Structure to existing Jones Pumping Plant Intake Channel designed for a flow of 4,600 cfs. A minimum 100 ft. buffer will be provided between the outer edge of the channel embankments and any existing roadways, as needed.

24.11.1 Construction Considerations

- Work near Byron Highway will require coordination with Caltrans.
- Construction near or under the rail lines will require close coordination with SPRR. This item is especially important because this involves a long period of coordination with SPRR.
- Tie-in of new canal to Banks PP and Jones PP Inlet Channels will require temporary shutdowns and close coordination with the SWP and CVP.

24.11.2 Channel Construction

Typical activities involved in the construction of canals include the following:

- Mobilization
- Clearing and grubbing the site.
- Setting up exclusionary fencing.
- Relocation of any utilities that may interfere with the construction of the canals.
- Excavating canal and/or building up embankments. Based on the existing grade elevation and required depth of the canal, it is expected that most of the canal will be excavated and require limited buildup of embankment.
- Placement of concrete lining at the bottom and side slopes of the canal.
- Finish grading and paving
- Installation of security system

24.12 Roads

24.12.1 Highway 160 (SR 160)

- Highway 160 will be impacted by construction activities at each of the three intake sites. During the initial construction phase, which widens and raises the levee crest, the highway will be permanently relocated from its current alignment along the top of the river levee to a new alignment established on top of the widened levee aligned approximately 220 feet farther inland from the river. Turn pockets and other highway features will be built to allow access to the intake sites. The location of the new permanent highway alignment is shown on the Concept Drawings (Volume 2). See Section 15 “Levees,” for sequencing of the relocation work.

24.12.2 Highway 12 (SR 12)

- In San Joaquin County, Highway 12 is a principal arterial and has a significant role in the interregional movement of goods and services. It also is a vital link between the counties of the northern San Joaquin Valley and the counties north of the San Francisco Bay.
- Where Highway 12 crosses the tunnel alignment, a spread diamond (Type L-2) interchange is proposed for anticipated large volume traffic during project construction. The tunnel shafts and this section of Highway 12...
are located on Bouldin Island, with the Little Potato Slough Bridge to the east and the Mokelumne River Bridge to the west.

- The existing roadway, from the Sacramento/San Joaquin County Line to I-5, consists of a two-lane conventional highway on level terrain. This portion of Highway 12 experiences an accident rate with both injuries and fatalities exceeding the State average on similar highway segments.

- The spread diamond (Type L-2) interchange will accommodate the large trucks and equipment requiring access to the shafts. Initially, at-grade T-intersections and left turn pockets and trumpet interchanges (Type L-11 or L-12) were considered, but it was concluded that a spread diamond not only satisfies geometrics but also the potential for future crossroad expansion.

- The overcrossing has two 12-foot lanes with 8-foot shoulders and a Type 732 concrete barrier, with a 16-foot vertical clearance. It is assumed that the overcrossing structure will consist of a cast-in-place/pre-stressed (simple) box. Auxiliary lanes in both directions will be considered to help with truck merging and weaving movements.

- Approximately 35 acres of right of way are required to construct the proposed Interchanges, the majority of which is owned by Caltrans. The exact locations of the interchanges depend on the tunnel alignment and Caltrans’ input during final design.

## 24.13 Utilities and Infrastructure Crossings

### 24.13.1 General

The utility and infrastructure crossings within the conveyance tunnel sections are minimally impacted by construction. To avoid the potential conflicts with shaft construction and disposal areas, the utility and infrastructure relocation will be coordinated with local agencies and owners.

### 24.13.2 Power Lines

Relocating poles or towers identified in field surveys will be coordinated with the operating entity. New poles or towers will be erected and cable-pulled before tie-in to the existing systems. Either special borrow and fill patterns or relocations are required for the transmission lines through the affected tunnel reusable material/borrow/disposal area.

### 24.13.3 Natural Gas Pipelines

Relocating natural gas pipelines will be coordinated with the utility companies to determine if new pipelines are constructed by cut-and-cover methods, trenching, or placement on at-grade saddles.

Identifying precise location of pipelines within a tunnel section is necessary to avoid conflicts with shaft construction and disposal of tunnel reusable material.

### 24.13.4 Natural Gas Wells

Access to the active wells in the tunnel ROW and borrow, spoil, or tunnel reusable material areas will be abandoned to a depth below the tunnel in accordance with DOC Division of Oil, Gas and Geothermal Resources requirements.

Future studies will identify the minimum allowable distance between the wells and tunnel excavation. Abandoned wells will need testing to confirm they have been abandoned properly. Those not abandoned properly will be improved to meet California Department of Conservation (DOC) well abandonment requirements. The known gas wells and fields in the Delta Region are shown in Section 13, Figure 13-1.
### 24.13.5 Mokelumne Aqueducts
The main tunnels from the IF to BT are under the Mokelumne Aqueducts at the north end of Woodward Island. These crossings will be evaluated at the preliminary design level in conjunction with EBMUD.

### 24.13.6 Structures
Structures affected by construction are moved, demolished, rebuilt, or otherwise mitigated. Each structure will be evaluated at the preliminary design level and mitigated individually in the EIR/EIS.

### 24.13.7 Agricultural Delivery and Drainage Ditches
Agricultural water delivery and drainage ditches interrupted by the conveyance facilities will be mitigated. Evaluating the systems and their requirements for mitigation measures will be coordinated with local farmers, landowners, governing agencies, and surveys of the delivery and drainage systems. Planned measures might include, but are not limited to:
- New or modified irrigation pumping plants.
- Extended delivery pipes.
- New or modified drainage ditches.
- Combining existing delivery or drainage systems.
- New or modified drainage pumping plants.
- New or modified access roads.

Specific mitigation measures will be based on local conditions and needs. Several of the existing water delivery or drainage systems can be combined. The actual mitigation will be determined on a site-specific basis consistent with the farmer, landowner, or local agency requirements, as appropriate.

### 24.13.8 Construction Considerations
- The various construction approaches include trenching, pipe rerouting, power line relocation, and others.
- Utility service (e.g., water, sewer, power) interruptions must be minimal during construction.
- Resolve issues with:
  - Power transmission or distribution lines.
  - High pressure gas lines and wells.
  - Railroad and aqueducts.
  - Existing water and sewer lines.
  - Agriculture drainage and water supplies.
  - Oil product lines. Operable Barriers Construction

For operable barrier construction discussion see Section 17, “Operable Barriers.”

### 24.14 Controls and Communications
Control and communications equipment are installed inside buildings or in outside electrical panels installed adjacent to facilities and structures. Equipment security and protection from vandalism will be assessed during the design phase. Specific considerations include:
- **Intake Facilities:** Conduit with fiber optic cable running in the intake concrete walls and out into the tunnel shafts. Pull-boxes are used to transition from the conduit to the tunnel.
• **North Tunnels from Intakes to IF:** Two conduits with fiber optic cable could run inside the tunnel bore after the concrete lining is installed. The conduits are attached to the side walls of the lining and run to pull-boxes at the surface of each access shaft. Alternatively, a fiber optic conduit near the surface along the tunnel ROW, microwave, or leased telecommunication lines is used.

• **Main Tunnels from IF to Byron Tract Forebay Pumping Plant:** Two conduits with fiber optic cable could run inside each parallel tunnel bore after the concrete lining is installed. The conduits are attached to the side walls of the lining and run to pull-boxes at the surface of each access shaft. Alternatively, a fiber optic conduit near the surface along the tunnel ROW, microwave, or leased telecommunication lines is used.

• **IF:** A buried conduit with fiber optic cable is placed along the east and north sides of the IF, connecting between the North Tunnels and the Main Tunnels.

• **BTF:** A buried conduit with fiber optic cable is placed along the south side of the forebay, connecting between the tunnels, the forebay structures, and the existing south Delta facilities. A railroad and highway crossing of the new fiber optic cable is required.

### 24.15 Power Supply and Grid Connections

#### 24.15.1 Overview

Construction duration of both the single-bore and dual-bore tunnels is estimated to be in the range of 6 to 9 years, which does not include the construction and removal of new transmission lines and system upgrades such as reconductoring. Construction power demand for this time period is greater than the permanent demand. The EIR/EIS, ROW, and land acquisition processes required by this project are all critical path items. Power and utilities work is done to minimize environmental footprint by co-locating transmission corridors and using existing power infrastructure where practical.

The methods associated with the construction of electrical transmission lines cannot be fully evaluated until a utility provider or providers and transmission corridors are selected. To minimize disruption along some the corridors, the transmission towers might be transported to their locations by helicopter. It is anticipated that the construction issues related to electrical transmission corridor construction will be evaluated in more detail by the EIR/EIS team after completion of additional studies.

#### 24.15.2 Construction Methodology

- The general approach is to install high-voltage lines above ground with sufficient overhead clearance. Specialty power line contractors will use standard industry construction methods. Poles or towers will be protected from construction equipment and operations.

- Grid connections will be completed with full coordination and permitting by grid operators/owners.

- Generators and substation are placed on concrete slabs (permanent or temporary) or steel frame structures (temporary only) with suitable foundations and containment areas.

#### 24.15.3 Constructability Considerations

- Buried power line installations should be considered where feasible during preliminary engineering and final design.

- It is possible that utility grid power is not available in time to support critical path activities, particularly tunnel shaft pad construction and shaft sinking work that precedes the tunnel construction work. Therefore, the interim use of onsite generation as the power source for shaft sinking activities is anticipated. As soon as construction of the temporary (or permanent, in some cases) utility grid power is completed, electricity from the interim onsite generators is no longer used, and a tie-in into the utility grid occurs.

- Use of large diesel generators at construction sites must be limited to minimize carbon footprint and air quality impacts.
• The soil or geologic condition in the transmission towers/poles and substation locations dictate the depth of the excavation, as well as the type of foundation to use.
• New power towers or poles must be located without conflict to existing utilities. New power lines should be constructed adjacent to other utility corridors where feasible.
• Power lines and substations for both temporary and permanent power supply must be installed.
• Suitable flood protection must be provided for temporary facilities. Permanent facilities are installed above the flood levels dictated by California WaterFix design standards.
• Specialty contractors will use industry practice methods for construction.
• Duct banks can be constructed parallel to surface facilities, and radio towers can be used for communication.
• Installed utilities and communication lines must be protected during construction.
• Integrating the system is dependent on finalizing components at the appropriate time.

24.16 Spoils Disposal Sites and Borrow Areas

24.16.1 Borrow Sites

Conventional earthmoving equipment, such as bulldozers, loaders, scrapers for short hauls, and excavators and off-highway trucks for longer hauls, are used to excavate borrow material above the groundwater table. If there are insufficient quantities of borrow material above the groundwater table, temporary dewatering operations will be considered, based on the economics of hauling borrow material more than 10 miles to the construction site.

Where borrow material from below the water table is considered, dewatering by excavating parallel trenches spaced from 6 to 120 feet apart and drawing water down using pumps may be the most effective way. Water may have to be treated prior to release, or preferably sprinkled on adjacent agriculture areas.

Soil excavated from borrow material may have excess moisture above optimum. In these cases, drying beds may be constructed, usually 2 to 5 acres in size. Drying beds should be contoured to drain. Where possible and layer of sand or fine gravel is placed. Wet material is dumped on the beds, usually with end dump trucks, and spread with dozers pulling disks. Disking, turning, and re-disking continues until the material is within 2 to 4% over optimum. Ideally, material is removed with self-loading scrapers which will remove another 1 to 3% moisture and hauled to embankment from the drying bed. Scrapers can remove soil down to the sand bed, mixing the soil for a uniform fill material. A smooth wheel compactor should be kept at the ready to seal the top of the drying bed and the embankment if rain storm approach.

All borrow sites must be maintained in a drained condition using drain ditches, sumps and pumps, or other means of removing surface water. Silt fences and drainage used to assure sediments are not deposited in waterways. Storm water must be treated, and silts removed, usually with settlement ponds constructed near the borrow site.

Dust and noise control at borrow sites must be planned and well maintained.

Some potential borrow areas near the alignment have layers of suitable material overlain by unsuitable fill materials, but still provide an economically viable borrow site. At these locations, overburden removal, stockpiling, and replacing in the borrow site will be considered.

Borrow site development requires restoration of the site upon completion of use. Restoration includes the following:

• Drainage and erosion control.
• Slopes flattened for stability.
• Vegetation on excavated slopes.
• Removal of construction facilities such as settlement ponds, roads, and drainage and sediment control facilities.
When practical, topsoil at developed borrow sites will be removed, stockpiled, and replaced. Restoration of commercial borrow sites will be the responsibility of the operator. Depending on the location of the borrow site and the quantity of material to be moved to the project site, material transport may take place by barge operations or by California legal transport trucks on state, county or local roadways.

24.16.2 Spoil Disposal Sites

Conventional earthmoving equipment, such as bulldozers, graders, and conveyors, will be used to move and place the spoil. Spoil placed in disposal areas on low lying agricultural lands needs to be placed in 12-inch lifts, with nominal compaction to control potential subsurface failure of the existing floodplain soil. Smooth wheel compactors should be used to seal the spoil prior to rainstorms. Spoil maximum height is expected to be 6 feet above preconstruction grade for all spoil sites with side slopes of 5H:1V or flatter. Spoils placed in areas with stronger foundation soils can be of greater depths and use steeper side slopes.

Prior to depositing spoil at any site, it may be necessary to strip and stockpile topsoil, normally with scrapers. Any existing utilities or irrigation conveyance systems in the area must be rerouted. The area will need to be graded for drainage and berms constructed to contain any water that drains form the spoil and/or from storm water.

Water from the site will need to be collected and treated, at least in sediment ponds and possibly with water treatment plants to remove contaminants or minerals.

Dust and noise control measures will need to be planned and continuously performed.

Spoil area must be maintained well drained without unplanned water retention areas.

Some spoil may be deposited in borrow areas after suitable material is excavated.

After final grading of spoil, the area will be restored based on site-specific conditions per project restoration guidelines. This may require replacing stockpiled topsoil.

See Section 11, “Tunnels,” for a description of the RTM handling procedure and limitations regarding the sequence for placing spoils, especially near existing levees. Depending on the location of the spoil disposal site and the quantity of material to be removed from the project site, material transport may take place by barge operations or by California legal transport trucks on state, county or local roadways.

24.16.2.1 Constructability Considerations

- The distance from the work site must be minimized to maintain short haul distances.
- The soil or geologic conditions in the disposal location dictate the depth of the fill, as well as offsets to mitigate the effects of settlement.
- Soil and geologic conditions dictate the final location of suitable borrow areas.
- Spoils disposal in borrow areas should be considered if feasible for the finished site and within economic proximity of the work areas.
- Topsoil must be stockpiled and sequenced with disposal fill operations. Dust mitigation will be implemented along haul routes and on disposal sites.
- Control and treatment of water from spoil piles will be maintained.

24.17 Other Aspects Related to Construction

Addressing constructability issues and proposing efficient construction methods must be encouraged to allow for contractor flexibility and ingenuity. As design progresses, construction methods and constructability issues will be further refined and clarified, and selected alternatives documented. Construction contractors will be invited to participate in workshops to identify and propose alternatives for consideration.
Early identification of constructability concerns, risks, and issues will be addressed in the Preliminary Engineering Schedule. Those concerns, risks, and issues, plus cost, play a major role in identification of construction methods. The project schedule may require some operations to be rescheduled so overall project goals or contract interface requirements are met. In the same way, the cost effectiveness of one construction method over another may be a significant factor in selecting which method is used. Construction and constructability go hand in hand with schedule and cost estimates to assess how project features are built.

### 24.18 Construction Schedule and Phasing

A conceptual design and construction schedule was prepared and summarized in Appendix C. This schedule is intended to provide guidance for the overall duration of the project, including design, bidding, and construction, and is not associated with specific notice-to-proceed dates.

This schedule was developed using Primavera P6 scheduling software and provides a conceptual sequence of design, bidding, and construction activities that could be used to complete the facility components. It is based on the information available at the time and assumed number of design and bid packages. It only delineates one possible sequence of work. It is not meant to dictate contractor means and methods and does not encapsulate possible phasing activities that could shorten the overall schedule or accommodate unforeseen elements that drive critical path.

The final sequence of activities and duration of the schedule will depend upon the actual execution of the work, the contractor’s actual means and methods, definition and variation of the design, abnormal conditions, and other variable factors. Therefore, a final schedule should be expected to vary from the preliminary schedule presented in this section.

### 24.18.1 Assumed Design and Construction Packages

The design and construction packages listed below were used. These packages are conceptual and are only provided to illustrate sequencing of the work; they are not a recommendation for how to implement the work. The actual design and construction packages will be developed later in the implementation process.

#### 24.18.1.1 Design Packages

- Intakes, and Sedimentation Facilities, at Sites No. 2, 3, and 5.
- Main Tunnel and Preliminary Site and Access Development.
- North Tunnels and Preliminary Site and Access Development.
- Communications.
- Power Supply (from power provider up to and including Pumping Plant Substation).
- Intermediate Forebay Embankment and Structures.
- Clifton Court Pumping Plant Structures
- Expanded Clifton Court Forebay Embankment and Structures.

#### 24.18.1.2 Construction Package Sequencing

Careful sequencing is required for virtually all construction packages to accomplish the work in accordance with the schedule shown in Appendix C. Key sequencing considerations are described below:

- **Advance Works.** All tunnel shaft sites that need ground improvement for soil consolidation need to be constructed so enough time has elapsed before shaft construction begins.
- **Intake sites.** All, or the applicable portion of, the fill pad at the intake needs to be constructed to support the work associated with the reception shaft for the north tunnel at each intake site. It is assumed that work is
concurrent at the site, but the north tunnel work is completed early enough to allow the intake work to use that portion of the site later during the work schedule.

- **IF site.** IF construction has been divided into two construction packages. The first package involves preparation of the site and construction of the main earthen embankment. When the embankment portion of the work is complete, the first IF construction contractor shares the site with the North Tunnel and Main Tunnel contractors. The North Tunnel contractor constructs the drive shafts at the IF site and drive the tunnels from the inside of the forebay. The Main Tunnel contractor constructs reception shafts at the IF site. Then, a second finishing IF contract is used to tie into the tunnels, construct the inlet and outlet structures, and provide the final elements of a completed forebay and appurtenant features.

- **BTF Site.** The Pump Plant pad on the North side of the BTF must be constructed prior to sinking the tunnel Reach 7 launch shaft (pump shaft). The PP pad is also the north containment of the BTF and could be contracted separately or as part of a total BTF package.

- **Main Tunnels.** The main tunnel construction is divided into four reaches, each a separate contract. Some of these reaches require that one contractor use the same drive shafts of another tunnel contract. In some cases, two contracts use the same reception shafts. The logistics of the coordinated use of the shafts is expected to be managed mostly by construction sequencing, but some shared, or alternating, use of the shafts is required. In some cases, using a separate access/muck shaft at the shared launch shaft is being considered such that construction work at the primary shaft can continue. This may be the case at the Reach 7 launch/pump shaft, the IF launch/inlet-outlet shafts and Bouldin Island for Reaches 5 and 6.

- All of the shared work site, coordination, and associated construction sequencing required for the work will be worked out in detail during preliminary engineering and final design. The conceptual construction schedule summarized in Appendix C assumes the sequencing concepts identified here can be worked out for the various sites.

### 24.18.1.3 Primary Construction Packages

- Advanced works for ground improvement at the tunnel shaft sites
- Intakes, and Sedimentation Facilities, at Site No. 2
- Intakes, Sedimentation Facilities, No. 3
- Intakes, and Sedimentation Facilities, at Site No. 5
- North Tunnels (Reaches 1) enabling contract for preliminary site and access development
- North Tunnel (Reaches 1, 2 and 3)
- Main Tunnel (Reach 4) enabling contract for preliminary site and access development (includes north tunnel drive site)
- Main Tunnel (Reach 4)
- Main Tunnel (Reach 5) enabling contract for preliminary site and access development.
- Main Tunnel (Reach 5)
- Main Tunnel (Reach 6) enabling contract for preliminary site and access development.
- Main Tunnel (Reach 6).
- Main Tunnel (Reach 7) enabling contract for preliminary site and access development
- Main Tunnel (Reach 7) at north Byron Tract
- North BTF Pump Plant
- Intermediate Forebay Embankment
• Intermediate Forebay Structures
• Byron Tract Forebay Embankment and Structures
• Connection Channels
• Communications
• Power Supply (from power provider up to and including Pumping Plant Substations)

24.18.1.4 Byron Tract Option Implementation Schedule Summary
A summary of key schedule information follows.

• The conceptual schedule duration is just under 14 years from the beginning of preliminary design to the completion of start and commissioning.

• The schedule duration is controlled by the construction schedule for the main tunnels, including the advance enabling contracts and the finishing contracts for pumping plant structures.

• Startup and commissioning of the pumping plant are expected to be the last project activity before the system is ready to deliver water on a continuous basis. If the schedule shows a time gap between the completion of the work and the beginning of start-up and commissioning, then the gap is considered float at this stage of scheduling.

• The availability of labor, equipment, materials, and qualified contractors with sufficient experience and bonding capacity to conduct the work will require further evaluation and will be subject to market forces at the time of bid. Items such as TBM delivery and the availability of specialty subcontractors, such as those who construct diaphragm walls, are critical to achieving the schedule shown. Additional schedule time might be required once these factors are considered in greater detail during more advanced phases of the process.

• Design Activities (based on design packages listed above):
  • 4 to 5 years each including all phases (1.5 years for preliminary design, 1.5 to 2.5 years for final design, and 1.0 years for contract document preparation and final revisions).

• Bid/Award Duration:
  • 4-6 months each from advertise to notice to proceed.

• Construction Duration:
  • Varies by contract; see schedule in Appendix C

• Power supply contracts (design and construction packages) are not reflected in the schedule included in this CER. It is assumed that power supplies will be available about 6 months prior to Notice to Proceed. Also, it is assumed that design and construction schedules for the power supply will be developed by others.

• A more detailed schedule is included in Appendix C.
SECTION 25.0
Dual Conveyance Facility Considerations

The WaterFix BTO described in this CER is an isolated conveyance facility component of one of the Dual Conveyance with Pipeline/Tunnel alternatives in the BDCP EIR/EIS, and it is one alternative configuration of the North Delta intake and conveyance facilities described in the EIR/EIS. Please refer to the EIR/EIS for a description of the operations of the WaterFix BTO facilities described in this CER.
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Permits

Implementation of the WaterFix BTO as described in this CER will meet the requirements of various Federal and State regulations, laws, policies, and Acts. Certain regulations require issuance of permits prior to project implementation; other regulations require agency consultation but might not require issuance of any permits prior to project implementation.

Permitting requirements for the WaterFix BTO are included in the BDCP Permitting Handbook (draft version, Issue Date: August 2011 and subsequent updates). This handbook includes the following:

- Provides information on the major requirements for permitting and environmental review and consultation;
- Identifies the major permits or actions, the agency in charge, agency authority, permit implementing entity, permitting process, and other relevant information;
- Describes regulations and policies that are likely to apply to BDCP/WaterFix activities;
- Describes processes, presents background information, and outlines how the activities would be authorized;
- Lists the parties that would likely be involved in required authorizations; and
- Identifies opportunities to streamline permitting.

The Handbook does not include all the permits or actions that may be needed for the project. All permitting requirements applicable to the WaterFix BTO as described in this CER will be determined during the final design phase, based on the requirements listed in the BDCP Permitting Handbook.

Design and Construction Authority (DCA) will ensure that all major permits are in-place prior to commencement of any construction activity. A specific State requirement is the Storm Water Pollution Prevention Plan (SWPPP). Another major permitting effort is for in-river construction requiring USACE and other permits or approvals. Rights-of-Ways and easements must also be in place. Cal/OSHA requires the DCA to obtain multiple permits. Treatment and disposal of construction water requires a National Pollutant Discharge Elimination System (NPDES) permit and coordination with the Regional Water Quality Control Board (RWQCB). Permits are required for each fuel storage depot.

To minimize site impacts and comply with permit requirements, DCA and construction contractors are required to develop safety plans, site utilization plans, traffic control plans, hazardous material management and containment plans, and various work plans. In their planning, DCA will indicate anticipated activities requiring site use as described in the utilization plan. Depending on the extent of the available site area, as well as anticipated demands, DCA will set up staging areas that include their offices, shops, safety zones, utilities, material handling, etc., and area for contractor offices, shops, utilities, etc. Due to the magnitude of need and the schedule demands, DCA may consider having their own concrete batch plants at several work sites. Only after required plans are developed and approved, will the DCA allow contractor mobilization.
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SECTION 27.0

Architectural Considerations

The architectural considerations are divided into three sections: existing conditions, programming, and design guidelines. This section focuses primarily at the Byron Tract Forebay; however, architectural considerations will be taken into account for other facilities including the river intakes. First, the existing conditions describe how the Byron Tract Forebay site currently is laid out. Then, the programming section defines what is needed for the pumping facility to function. Finally, the design guidelines provide a uniform architectural concept for the sites and buildings along the new intake tunnels.

27.1 Existing Conditions

27.1.1 Tunnel Alignment

The tunnels will begin at the intake facilities along the Sacramento River. The tunnels follow a southerly alignment until reaching the Byron Tract Forebay, with the terminus located at the northeastern side of BTF. A second set of tunnels will convey water from BTF to the conveyance channels to the Banks PP and Jones PP.

27.1.2 Pumping Plant Site

27.1.2.1 Location

BTF is located between the Italian Slough to the east, Byron Highway to the west and south, and Highway 4 to the north. The Byron Highway allows views to the east of the BTF area. Currently, a majority of the area is under agricultural use and private ownership. The proposed Byron Tract Forebay pump plant location is at the northeast side of the forebay.

27.1.3 Other Infrastructure near the Forebay

Figure 27-1 below shows other monumental infrastructure projects currently located near the proposed Byron Tract Forebay. To the south, the Skinner Fish Protective Facility removes fish from the Clifton Court Forebay prior to reaching the Banks Pumping Plant. This facility is largely made up of concrete structures with a few metal outbuildings. Also to the south, the Harvey O. Banks Pumping Plant (see Figure 27-2), lifts the water 244 feet to the Bethany Reservoir, and is the head of the California Aqueduct system. The complex is made of simple concrete structures and can move water up to 10,670 cubic feet per second. The office building has more detail emphasis, with the concrete walls and roof framing the glazed entry. Further south-east is the C.W. “Bill” Jones Pumping Plant which provides water into the Central Valley region. Its pump plant is a concrete structure with rhythmic openings in its façade.
Figure 27-1: Vicinity Map including distances from prominent landmarks
27.1.4 Other Monumental Complexes

The California Aqueduct has monumental structures over the entire system to facilitate the transport of water to southern California. These structures include the aqueduct pumping plants, like Dos Amigos, Teernink, and Chrisman. The Edmonston Pumping Plant (see Figure 27-3) raises water almost 2,000 feet to the next section of the aqueduct. The building plays on the rhythmic repetition of the equipment (see Figure 27-7). The full-height glazing provides adequate daylighting for the interior.
27.2 Programming

27.2.1 Site

To contain the increased amount of water during a 200-year flood, the levee will be constructed at elevation 25-feet. Raising the island elevation level with the new levee provides flat space requirements for the facility buildings. A security fence around the facility perimeter will protect critical water infrastructure from tampering, adds safety to the equipment, and protects the public from hazardous operations. Roads need improvement from narrow, gravel lanes to paved infrastructure capable of supporting heavy loads. The site requires parking for up to 15 vehicles for employees and visitors.
## Byron Tract Forebay

### Pre-Design Program Matrix

<table>
<thead>
<tr>
<th>Dimensions (Sq Ft)</th>
<th>Amenities</th>
<th>Adjacencies</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pump Plant</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(2) Pumping Plants</td>
<td>65' height above finish floor</td>
<td>Minimum (1) staff per plant at all times</td>
<td>Electrical Building Control Room Locker Room</td>
</tr>
<tr>
<td></td>
<td>182'dia (26,000 sf)</td>
<td>Radial bridge crane Coiling overhead door</td>
<td></td>
</tr>
<tr>
<td><strong>Electrical Building</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Building per pump plant</td>
<td>100'x100' (10,000 sf)</td>
<td>MCCs</td>
<td>Pumping Plant</td>
</tr>
<tr>
<td>Control Room</td>
<td>25' x 15' (375 sf)</td>
<td>2 SCADA workstations 2 general workstations</td>
<td>Access to plant processes</td>
</tr>
<tr>
<td>IT/Server Room</td>
<td>10'x10' (1000 sf)</td>
<td>Server stacks Extra room cooling</td>
<td>Control Room</td>
</tr>
<tr>
<td><strong>Maintenance</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Entire Building</td>
<td>100' x 100' (10000 sf)</td>
<td>Pumping Plants</td>
<td>The maintenance shop needs</td>
</tr>
<tr>
<td><strong>Office/Admin and Storage</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Admin Area</td>
<td>10' x 12' (120 sf)</td>
<td>Workstation Window to Lobby</td>
<td>Lobby</td>
</tr>
<tr>
<td>Lobby</td>
<td>10'x10' (100 sf)</td>
<td>Waiting Chairs</td>
<td>Admin Area</td>
</tr>
<tr>
<td>Plant Manager's Office</td>
<td>10' x 16' (160 sf)</td>
<td>Workstation Small Meeting Table</td>
<td>Operators, meeting space</td>
</tr>
<tr>
<td>Maint. Office</td>
<td>10' x 10' (100 sf)</td>
<td>Workstation</td>
<td>Maintenance shop Maintenance Meeting Space</td>
</tr>
<tr>
<td>Mee ing Room</td>
<td>Hold up to 15 to 20 people</td>
<td>Chairs Projector</td>
<td></td>
</tr>
<tr>
<td>Copy / Print / Supply Room</td>
<td>6' x 12' (96 sf)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Break Room</td>
<td>20' x 12' (240 sf)</td>
<td>10 people</td>
<td></td>
</tr>
<tr>
<td>Main Conference room</td>
<td>18' x 16' (288 sf)</td>
<td>8-12 people</td>
<td></td>
</tr>
<tr>
<td>Restroom for vendors and visitors</td>
<td>8' x 10' (80 sf)</td>
<td>Toilet / sink</td>
<td></td>
</tr>
<tr>
<td>Mens Locker Room</td>
<td>22' x 12' (264 sf)</td>
<td>14 lockers / 2 showers 2 stalls / 2 urinal / 2 sinks</td>
<td></td>
</tr>
<tr>
<td>Women's Locker Room</td>
<td>18' x 12' (216 sf)</td>
<td>8 lockers / 1 shower 2 stalls / 2 sinks</td>
<td></td>
</tr>
<tr>
<td>Mud Room</td>
<td>10' x 10' (100 sf)</td>
<td>Allow for dirty gear, laundry pick-up</td>
<td>Between pumping plant and Surge tanks Extra surge gates and pump parts</td>
</tr>
<tr>
<td>Storage</td>
<td>100'x100' (10, 000 sf)</td>
<td>Bridge Crane</td>
<td></td>
</tr>
<tr>
<td><strong>Water Filtration Building</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water Filtration</td>
<td>Potable water for staff</td>
<td>Filtered water for pumps</td>
<td>Near East Canal</td>
</tr>
<tr>
<td>Storage Tanks</td>
<td>Verify gallon storage</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Building Support</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Janitor</td>
<td>10' x 6'</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mechanical Room</td>
<td>32' x 30' (960)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Electrical /Mechanical Room</td>
<td>25' x 12'</td>
<td>Per county Title 8 Zoning Code</td>
<td></td>
</tr>
<tr>
<td>Parking</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Figure 27-5: Building Program Matrix*
27.3 Design Guidelines

The design guidelines in the following section provide a uniform concept of the whole site and other facilities along the new intake tunnel. These design guidelines bring together ideas from codes, precedents, and input from DWR for the site, landscape, and buildings. The guidelines start with a general application and then advance toward a specific application in Byron Tract Forebay.

Applicable Codes:
- California Building Code (CBC)
- California Energy Code (CEC)
- NFPA 13
- NFPA72
- Title 8 Zoning in Contra Costa County

The preliminary architectural finishes and landscape plant materials described in this section are only recommendations and will be further developed and finalized upon DWR review during the preliminary and final design phases of the project.

27.3.1 Site

27.3.1.1 General Site

The general site guideline focuses the layout on a North-South axis. Regardless of site size, the arrangement follows symmetry along that axis. The site also provides parking for visitors and employees within the symmetrical framework. Although the locations are rural, the sites have need for buffers. For a barrier of sound and sight, trees offer a shield. A security fence provides a physical barrier for safety.

27.3.1.2 Byron Tract Forebay Site

The Byron Tract Forebay Pumping Plant centers on an axis that splits the site so that the pumping plants and the electrical buildings mirror each other. To accommodate the large truck traffic, the roads are graded and paved and create a roundabout roadway system for easy service access to buildings and equipment. Sidewalks along the roundabout provide safe pedestrian access between buildings. The main parking fits in the block between the two pump plants as well as other stalls throughout the facility for easy access to the buildings and equipment. The security fencing follows the boundaries of the site to add a safety barrier. Properly positioned lighting around the site adds another safety precaution without intruding on the privacy of nearby residences.
27.3.2 Landscape

27.3.2.1 General Landscape
The new landscaped areas will utilize native, low maintenance vegetation. Disturbed interior site areas will receive groundcover to reduce erosion and pollution. Along the exterior, trees and shrubs will be planted where possible. Vegetation will be planted in natural groupings, and not in a regimented order. The random order matches how the groves and plantings are around neighboring farmhouses and residential communities.

27.3.2.2 Byron Tract Forebay Landscaping
The Byron Tract Forebay landscaping follows the native, low-maintenance vegetation. When the area is raised up, the entire area needs to be landscaped where structures are not being built to avoid erosion and pollution. Some portions on the site interior will have grasses and low lying shrubs to reflect what was there before the changes.

27.3.3 Building

27.3.3.1 General Design Concept
The building design concepts draw from existing facilities along the California Aqueduct, while tying into relationships with the landscape. The buildings use simple forms and appropriate massing for the interior process functions. The main building theme is a sturdy structure that will withstand the pumping process happening within the building. A softer element is used to define the points where humans interact with the buildings or where interior equipment repetitions are expressed on the façade.

27.3.3.2 Design Objectives
Design objectives are as follows:
- Provide a functional design and material palette for a uniform facility along intake tunnel.
- Use durable, simple materials for the long-term performance of the facility with low maintenance.
- Reference historic and neighboring facilities within design motifs.
- Provide functional spaces that fill process and non-process requirements.

27.3.3.3 Design Guidelines
The main, rugged walls have aggregate stone exposed from sand-blasting. The softer elements are a smooth pre-cast concrete frame that extends from the aggregate wall. The frame infill is a concealed-fastener, low-maintenance metal panel. The wall extrusion references the Banks Pumping Station office as the concrete walls and roof extend past the entrance, as seen in Figure 27-2. The rhythmic repetition of the equipment within references the Edmonston Pumping Station in Figure 27-3.

The Office Building is an example of the Design Guidelines at work with the non-process buildings, see Figure 27-9. The core building material is the sturdy carbon cast wall. From the stark walls, a smooth pre-cast concrete extrudes to frame the main entrance. The extrusion not only frames the doorway, but highlights a focal point by
using metal on this portion of the walls. Architectural louvers also highlight areas within the buildings that are consistently occupied, while also hiding potential penetrations in the strong concrete façade.

The pump plants, which are rendered in Figure 27-7, are examples of how these guidelines work for the facility’s process buildings. The two pump plants utilize the stark, rugged main material. A secondary smooth concrete extrusion frames the cylinder where workers enter the structure. Within the frame, a metal panel adds a humanized scale to the rugged concrete monolith. The glazing at the worker line-of-sight has its frame entirely on the inside of the thermal barrier. The rhythm of the pump head repeats on the façade with a smooth concrete extrusion. Another framed in feature is the bi-fold door, which also acts as a shading device when opened. The bridge crane rotates around the top of the concrete drums, while a sloped roof appears to float above. The frosted clerestory channel glass allow light to penetrate the pump room and also hide the steel structure holding up the roof. See a graphic depiction of process buildings in Figure 27-10.
### 27.3.3.4 Material Matrix

The material matrix is defined below in Figure 27-13.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Color</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast in Place w/ sand-blasted exposed aggregate</td>
<td>Aggregate Dark Grey</td>
<td>Primary exterior material of Pump Plant</td>
</tr>
<tr>
<td>CarbonCast w/ sand-blasted exposed aggregate</td>
<td>Aggregate Dark Grey</td>
<td>Primary exterior material for rest of plant</td>
</tr>
<tr>
<td>Precast concrete</td>
<td>Natural Grey</td>
<td>Secondary material form used to extrude from the building for human form</td>
</tr>
<tr>
<td>Concealed Fastener Low-Maintenance Metal Panel</td>
<td>Natural Zinc</td>
<td>Infill from extruded precast fins</td>
</tr>
<tr>
<td>SSG Curtainwall</td>
<td>Dark Anodized</td>
<td>Provides resiliency, corrosion resistance, and maximum daylight</td>
</tr>
<tr>
<td>Louver</td>
<td>Color to match metal panel</td>
<td>Provides air movement through building and punctuates the rhythmic elements inside</td>
</tr>
<tr>
<td>Translucent Glazing</td>
<td>Frosted</td>
<td>Clerestory above cylinder for light into pumping plant</td>
</tr>
</tbody>
</table>

*Figure 27-8: Material Matrix*
SECTION 27: ARCHITECTURAL CONSIDERATIONS

1. PRECAST INSULATED CONCRETE PANELS
2. CONSEALD FASTENER METAL PANEL SYSTEM
3. MECHANICAL/ARCHITECTURAL LOUVERS
4. PRECAST CONCRETE SURROUND
5. SSG STOREFRONT SYSTEM
6. SIMPLE PLATE STEEL CANOPY AT ENTRIES

Figure 27-91: Graphic Depiction of Design Guidelines for Non-process Buildings

Note: Depicted materials are shown in typical colors and may vary
SECTION 28.0  
References  


CALFED Bay-Delta Program Water Management Planning Branch, 1999.  *Conceptual Analysis of Incised Canal Configuration, September.  Section 10.3.1*  

California Data Exchange Center (CDEC), 2009. Station Meta Data: Sacramento River at Hood (SRH).  


Campbell, K.W. and Y. Bozorgnia, 2008. NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 s to 10 s.  *The Professional Journal of the Earthquake Engineering Research Institute, Volume 24, No. 1, pp. 139-171.  February.*


Delta Vision Blue Ribbon Task Force. *See Section 3.5 re: SLR values*


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DWR, 2010b. Addendum to the Conceptual Engineering Report for the Isolated Conveyance Facility Pipeline/Tunnel Option (formerly All Tunnel Option). October 22.


United States Bureau of Reclamation (Reclamation), 1947. Central Valley Project – California, Delta-Mendota Canal, Sta. LR-10+00 to Sta. L-132+24 Profile and Sections, Drawing No. 214-D-13607.

Reclamation, 1951. Designers’ Operating Criteria, Tracy Pumping Plant, Central Valley Project, California. May.


Water Code Section § 259. Law governing condemnation of railroad, public utility or state agency property: When the department condemns the property of any common carrier railroad, other public utility, or state agency, or the appurtenances thereof, it shall be governed by Article 3 (commencing with Section 11590) of Chapter 6 of Part 3 of Division 6.

Water Code Section § 11590. Substitution of facilities; agreement: The department has no power to take or destroy the whole or any part of the line or plant of any common carrier railroad, other public utility, or state agency, or the appurtenances thereof, either in the construction of any dam, canal, or other works, or by including the same within the area of any reservoir, unless and until the department has provided and substituted for the facilities to be taken or destroyed new facilities of like character and at least equal in usefulness with suitable adjustment for any increase or decrease in the cost of operating and maintenance thereof, or unless and
until the taking or destruction has been permitted by agreement executed between the department and the common carrier, public utility, or state agency.

**Water Code Section § 11592. Public Utilities Commission; submission of controversies:** In the event the department and any common carrier railroad, other public utility, or state agency fail to agree as to the character or location of new facilities to be provided as required in this article, the character and location of the new facilities and any other controversy concerning requirements imposed by this chapter shall be submitted to and determined and decided by the Public Utilities Commission of the State.


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1 Portions of this section were adapted from the draft *Construction Access* technical memorandum prepared by CDM Smith for Metropolitan Water District of Southern California dated August 29, 2012 (CDM Smith, 2012). Section 23.2
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. (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 Belknapp, 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 Belknapp, 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 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 (CRSB). 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 CRSB 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 CRSB 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).

Thornton Arch Source Zone. 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).

Montezuma Hills Source Zone. 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.

West Tracy Fault. 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).

Vernalis Fault. 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).

**Black Butte and Midway Faults.** 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 $\leq 3:1$ 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 $M \geq 4.0$ events have occurred in the past 40 years in the Delta and no $M \geq 5.0$ 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.

**October 21, 1868.** 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.

**April 19 and 21, 1892.** 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.

**March 31, 1898.** 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.

April 18, 1906. 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.

May 2, 1983. 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.

April 24, 1984. 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 $7.5 million. In San Jose, cracks formed in some walls and plaster fell, many items were thrown from store shelves and some chimneys cracked. Very strong shaking ($\sim 1.3$ g) was measured at Coyote Dam during about 20 km south of the epicenter. This earthquake is thought to have been very similar to an earthquake that affected the area in 1911.

October 17, 1989. 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.

October 30, 2007. 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, based on the distribution of aftershocks, focal mechanisms, and moment tensor solutions (U.S. Geological

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 $M_\text{6}$ to $6\frac{1}{2}$. 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 $M_{6.5} \pm 0.3$.

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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MEMORANDUM

Delta Habitat Conservation and Conveyance Program (DHCCP)

Intake Facility Component Peer Review

B&V Project 186493

General Description of Conceptual Level Construction Sequencing at the DHCCP Intake Facilities

March 5, 2015 (Revised April 7, 2015)

To: Jay Arabshahi, Project Manager
Evelyn V. Ramos, Deputy Project Manager

Prepared By: Dave Chamberlain, P.E.

Reviewed By: Andrew Lazenby, P.E., Deputy Project Manager
S. Thomas Freeman, RG, EG, GeoPentech, Inc.
Douglas Wahl, PE, GeoPentech, Inc.

Submitted By: Steven N. Foellmi, P.E., Project Manager

Introduction
This memorandum is intended to provide a general description of the sequence of proposed work activities to construct the modified DHCCP Intake Facilities, illustrating how the major work activities (consisting of groundwater management, Highway 160 relocation, intake structure construction, and landside facilities construction) may be implemented. This document provides further clarification on the assumptions for these major work activities and GeoPentech’s opinion of the geotechnical conditions that may impact the design and construction of the Intake Facilities.

A Technical Memorandum prepared by CH2MHiIl (dated Aug 10, 2012) also addresses the Conceptual Level Construction Sequencing of DHCCP Intakes. This previously prepared TM is based on earlier concepts for the design of the DHCCP Intake Facilities. While much of the information presented in this earlier TM regarding construction methods, flood protection, and geotechnical conditions at each of the intake sites remains valid, the recent modifications to the DHCCP Intake Facilities warrants a re-evaluation of the sequence of construction as generally described in this memorandum.

Assumptions
Several key assumptions used to prepare the sequence of proposed major work activities are summarized below.

• The intake facility configuration is generally in accordance with the Final Draft Concept Rendering, dated April 1, 2015.
As a first order of work, Highway 160 is relocated to its permanent alignment and traffic is rerouted to the new highway.

The existing levee prism and the existing Highway 160 cannot be disturbed by any dewatering and construction activities until the Highway 160 relocation work is completed.

Soil improvements addressing potential surface settlement due to seismic induced liquefaction is required under the relocated portion of Highway 160, the intake structure, and the intake box conduits.

Limited geotechnical information is available on the subsurface conditions at each intake site. The suitability of any dewatering or foundation improvement system should be based on on-site actual geotechnical data obtained with the next phase of project development.

Structural calculations were not performed to determine the adequacy, size or bracing requirements for any alternative method for controlling groundwater seepage and temporary or permanent engineered excavations and fill slopes.

The intake structure cofferdam is constructed to provide a single basin and that adequate bracing can be provided without the need for an intermediate support wall.

Geotechnical assumptions are provided by GeoPentech Memorandum, dated March 13, 2015, attached hereto.

**Modified Conceptual Intake Facilities**

In general, the modified intake facilities include an intake structure located within the river side of the existing levee prism, box culvert collector channels that convey the river diversions from the intake structure beneath a widened levee section to earthen sedimentation basins, the sedimentation basins and on-site solids handling facilities, a tunnel outlet structure, and other appurtenant facilities. The work at each intake facility includes relocating a segment of Highway 160 from atop the existing levee to a new alignment that is approximately 250 feet landside of the existing highway. As a first order of work, construction of the relocated segment of Highway 160 must be completed before any work affecting the existing highway is initiated.

A single option for the sequencing of construction activities for the modified intake facilities is described as provided in the table below. The accompanying figure further depicts the sequencing activities. This sequencing is likely to be modified by the actual construction contractor based on final designs of the intake facilities, site conditions, and alternative construction techniques.
## Construction Sequencing Activities

<table>
<thead>
<tr>
<th>Phase</th>
<th>Activity</th>
<th>Predecessor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Groundwater Management</td>
<td>1</td>
<td>—</td>
<td>Construct a slurry cutoff wall on the landside of the existing levee. The location of the cutoff wall shall be outside of the landside prism of the existing levee and up to the toe of the new fill slope constructed for the relocation of Highway 160. The cutoff wall should extend to a depth below and a distance upstream and downstream beyond the length of the relocated portion of Highway 160 and the portion of the new intake structure's box culverts beneath the relocated portion of Highway 160 to minimize potential seepage through and under the existing levee.</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1</td>
<td>Construct a well point dewatering system or a site perimeter slurry wall that will allow for excavation and placement of the new fill slopes for the relocation of Highway 160.</td>
</tr>
<tr>
<td>Highway 160 Relocation</td>
<td>3</td>
<td>1,2</td>
<td>Excavate and remove peat soils to suitable subgrade beneath the area for the widened levee and relocated Highway 160 (approximately 5 feet depth). Area to be cleared extends from the slurry cutoff wall to approx. 270 feet landside of existing levee.</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>3</td>
<td>Construct soil improvements beneath all or portions of the widened levee area and the fill slopes required for the permanent relocation of Highway 160 to address potential settlement from added gravity loading and seismic induced liquefaction. Possible soil improvement techniques may include cement deep soil mixing, dynamic compaction, jet grouting, stone columns, surcharge preloading, etc.</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>4</td>
<td>Construct new fill slopes for the new permanent alignment of Highway 160, including the transitions beyond the intake facility site. Construct portions of the intake box conduits that traverse the fill beneath the new permanent alignment for Highway 160 (the length of box conduits to be constructed is approx. 200 feet, and includes the exit wingwalls in the sedimentation basins).</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>5</td>
<td>Construct the roadway for the relocated segment of Highway 160 and reroute traffic to new permanent alignment on widened levee section.</td>
</tr>
<tr>
<td>Intake Structure Construction</td>
<td>7</td>
<td>6</td>
<td>Construct a diaphragm wall within the existing levee crest located along and outside of the back wall of the planned new intake structure. Provide cutouts for extending the intake box conduits through the diaphragm wall. Extend the diaphragm was as needed (approx. 150 feet) beyond the length of the intake structure to minimize potential seepage though and under the existing levee to allow for the excavation of the box conduits between the intake structure and the relocated Highway 160.</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>6</td>
<td>Construct sheet pile portion of cofferdam (river side of existing levee) and training walls for intake structure. Tie the cofferdam into the diaphragm wall constructed on the landside of the levee crest to provide an enclosed area for construction of the intake structure.</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>7, 8</td>
<td>Excavate within the intake structure cofferdam and installed diaphragm wall. Install dewatering system within area enclosed by cofferdam and underwater cofferdam.</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>9</td>
<td>Construct drilled casings and reinforced concrete piers beneath the intake structure. Install tremie seal within cofferdam.</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>10</td>
<td>Construct the intake structure within the cofferdam system. Backfill between existing/improved levee and training walls on the upstream and downstream sides of the intake structure.</td>
</tr>
</tbody>
</table>
### Geotechnical Issues Related to Construction Sequencing

- The presence of peat in the upper portion of the soil profile would likely make the logistics and construction layout more difficult, and may affect what soil improvement techniques are employed.

- Due to the presence of groundwater and likely soil types to be encountered, certain soil improvement techniques may be difficult or infeasible. For instance, dynamic compaction or jet grouting may be very difficult or potentially ineffective depending on the actual soil conditions. Stone columns would introduce avenues for seepage and would likely be infeasible because of the adjacent levee. Soil improvement techniques should be selected based on geotechnical information obtained during investigation of each site performed with the next phase of project development.

- Constructing a portion of the box culverts within the Highway 160 fill and then connecting to subsequent sections as well as the intake structures would require a plan to control settlements and differential settlements during the construction period. Presumably, the soil improvements constructed under the new roadway would address potential differential settlements in concert with the improvements and foundation design for the remaining structures.

- Overall, the proposed construction sequence should address site specific geotechnical concerns. However, careful design of the soil improvement scheme in coordination with the foundation design, as well as groundwater control for the structures, would be important to control costs. As an example, soil improvements under the intake structures may eliminate the need for a deep foundation. Further opportunities may be identified under subsequent peer reviews and value-engineering performed throughout the geotechnical investigation and design phases.
DHCCP INTAKE FACILITIES
CONSTRUCTION SEQUENCING FIGURE

Activity 9 thru 11
(9) Excavate / Dewater Cofferdam
(10) Construct Drilled Casings / Pour Tremie Concrete
(11) Construct Intake Structure

Activity 12 and 13
(12) Construct Soil Improvements
(13) Construct Box Conduits / Fill Slopes for Widened Levee

Activity 3 thru 6
(3) Remove Peat Soils / Excavate to Subgrade
(4) Construct Soil Improvements
(5) Construct New Fill Slopes / Box Conduits
(6) Re-route Traffic

Activity 14 thru 17
(14) Install Dewatering System
(15) Construct Soil Improvements
(16) Construct Landslide Facilities
(17) Commission Project

Activity 8
(8) Construct Sheet Pile Wall

Activity 7
(7) Construct Diaphragm Wall

Activity 1, 2
(1) Construct Slurry Wall
(2) Construct Well Point Dewatering
Memorandum

To: David Chamberlain – Black & Veatch

CC: Steve Foellmi – Black & Veatch

From: Sarkis Tatusian, Tom Freeman, & Douglas Wahl

Date: March 13, 2015

Re: Geotechnical Evaluation - B&V’s Modified DHCCP Intake Facilities Configuration for Metropolitan

This memorandum presents GeoPentech’s evaluation of the geotechnical conditions that may impact the proposed design and construction of the modified facility configuration proposed by Black & Veatch for the Metropolitan Water District of Southern California (Metropolitan) and the Delta Habitat Conservation and Conveyance Program’s (DHCCP) Intake Facilities along the Sacramento River. The following text and accompanying figures present a brief discussion of: 1) our understanding of B&V’s currently proposed configuration for Intake Facilities No. 2, 3, and 5; 2) the key assumptions we made in the course of our evaluation; and 3) a description of the general subsurface conditions at each proposed facility location, a listing of our identified primary geotechnical considerations, and our recommendations for further considerations.

Project Understanding

Based on our discussions with Black & Veatch, we understand that Metropolitan is evaluating various options for configuration of the intake facilities that will feed the proposed conveyance elements of the DHCCP. These proposed intakes are located on the eastern bank and levee of the Sacramento River in the northern part of the Sacramento-San Francisco Bay-San Joaquin Delta as shown on Figure 1. Black & Veatch has developed a set of conceptual-level designs for each facility as well as a proposed conceptual construction sequence that will be used to develop cost estimates for the construction of each facility. We also understand that a base estimate was previously prepared by another firm and that Black & Veatch intends to use their current effort as a basis of comparison with the previous estimate.

GeoPentech’s scope of work for this assignment included completing the following tasks:

- Review readily available relevant geologic and/or geotechnical data identifying the likely range of subsurface ground conditions at the three proposed intake facility locations.
- Review B&V’s proposed construction sequence relative to alternative currently industry accepted foundation alternatives. Provide guidance-level geotechnical/geological opinions of the key issues and constraints that would need to be evaluated for design and construction of the proposed facilities, considering the assumed ground conditions and B&V’s proposed construction sequence.
- Prepare this technical memorandum presenting the results of GeoPentech’s evaluation.
**Key Assumptions**

Because current information about, and available geotechnical data at the location of the three intake facilities is limited and variable, we had to make several assumptions to complete this assignment, including:

- The current ground surface elevation on the land-side of the existing river levee and Highway 160 at and around the three facility locations is at approximately +10-ft. MSL.
- Assume 5 feet of peat present at the surface of the intake locations.
- Assume generalized subsurface profile below the peat is as discussed below.
- Assume groundwater on land-side is within about 5-10 feet of the existing ground surface.
- That Cement Deep Soil Mixing (CDSM) will be used instead of jet grouting for general ground improvement or treatment under and around structural foundations. Due to the potential for clayey soils on the land-side of the proposed intake areas, jet grouting may not be as effective as CDSM, which can be used with both clayey or sandy soils as opposed to jet grouting which is more effective in only sand.
- Assume foundation settlement constraints are most critical beneath the facility’s box culverts and intake structure.
- Assume that CDSM ground improvements will generally extend to about elevation -40-ft. to -60-ft. MSL.
- Assume the CDSM area replacement ratio (ARR) will be 30% under and around facility embankments and elevated graded pad, and 50% under and around the intake structure and box culverts.

Reported costs per CY of treated material for CDSM on public projects vary considerably but generally fall between about $60-140/CY. A median value is about $80/CY, which is higher than most of the reported CalTrans projects. (FHWA-HRT-13-046) For comparison costs, we have assumed $80/CY.

**Description of General Subsurface Conditions**

GeoPentech reviewed the readily available geotechnical information in the vicinity of proposed Intake Facilities 2, 3, and 5, (Figure 1) to evaluate what geotechnical considerations may impact their design and construction. Figure 1 shows the location of the previously drilled geotechnical investigation borings. The soil description logs from some of these previously drilled borings were not available for this evaluation. The only boring logs that were available for our review were from the DWR’s investigation completed for the DHCCP between 2009 and 2012. Table 1 indicates the specific borings and CPTs whose logs were available and were reviewed.

<table>
<thead>
<tr>
<th>Exploration Name</th>
<th>Data Source</th>
<th>Type</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake 2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DCR2-DH-004</td>
<td>DHCCP GDR Rev0</td>
<td>Borehole</td>
<td>2010</td>
</tr>
<tr>
<td>DCR2-DH-005</td>
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<td>Borehole</td>
<td>2012</td>
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<td>DCR2-DH-006</td>
<td>DHCCP GDR Rev0</td>
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<td>DCR2-DH-007</td>
<td>DHCCP GDR Rev0</td>
<td>Borehole</td>
<td>2010</td>
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<tr>
<td>Exploration Name</td>
<td>Data Source</td>
<td>Type</td>
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</tr>
<tr>
<td>------------------</td>
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<tr>
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<td><strong>Intake 3</strong></td>
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<td>DCR3-DH-013</td>
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<td>DCR3-DH-005</td>
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<td>Borehole</td>
<td>2010</td>
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<tr>
<td>DCR3-DH-007</td>
<td>DHCCP GDR Rev0</td>
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<td>2010</td>
</tr>
<tr>
<td>DCN3-CPT-004</td>
<td>DHCCP GDR Rev0</td>
<td>CPT</td>
<td>2011</td>
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<tr>
<td><strong>Intake 5</strong></td>
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<tr>
<td>DCR5-DH-013</td>
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<td>DCN4-DH-009</td>
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<td>DCN4-CPT-009</td>
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<td>CPT</td>
<td>2011</td>
</tr>
</tbody>
</table>

In addition to the borings available for review, we incorporated information on the thickness of surficial organic material from the final DRMS study (URS 2009). Contours of thickness of organic material were digitized from a map provided in the DRMS study (see Figure 2) and are included on Figures 1, 3A, 4A, and 5A.

The following sections discuss the general subsurface conditions at Intake Facility 2, 3, & 5 based on the currently available geotechnical information, (borings and CPT logs), and our experience with the geology and geotechnical conditions in the area (URS, 2009; DWR 2013).

**Intake 2:**

The proposed location for Intake Facility 2 had available logs from six borings (DCR2-DH-004 through DCR2-DH-009) that were drilled by DWR in the river channel, adjacent to the existing levee opposite the proposed intake structure as shown in Figure 3A. Generally, the stratigraphy identified in the available six boring logs (Figure 3B) consists of very loose to moderately dense sands and gravels from approximately elevation -15 ft to about elevation -60 ft. The density of the sands increases from about elevation -60 ft to -80 ft. Stiff to hard, high plasticity clay layers were encountered at about elevation -80 ft to -90 ft and extends down to about elevation -120 ft to -130 ft.

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1 The legend for the figures reproduce herein from DWR (2013) is attached at the end of the memorandum.
Based on these boring logs the upper sands are very likely to be prone to liquefaction, lateral instability, seismically-induced settlement, and potentially have relatively high hydraulic conductivity and associated seepage. The lower clay layers would likely have relatively low hydraulic conductivity and cut off seepage effectively. Given that all six of these borings were drill in the existing Sacramento River channel, substantial variation may exist from what was encountered in these borings and what may exist beneath the existing ground surface land-ward from the river in the area of the intake facility. No peat or organic materials were identified in these borings, and the URS (2009) peat/organic surficial soils thickness map did not identify appreciable thickness of these extremely weak foundation soils in this area.

**Intake 3:**

The logs of six borings and one CPT (DCR3-DH-013 through DCR3-DH-015, DCR3-DH-003, DCR3-DH-005, DCR3-DH-007, and DCN3-CPT-004), which were located near the proposed Intake Facility 3, as shown in Figure 4A, were available for our review. Like the borings near Intake 2, as discussed above, all six borings were located in the Sacramento River channel with three directly opposite the proposed intake structure and three farther downstream in the river. The one CPT (DCN3-CPT-004) was located land-ward of the river towards the south end of the proposed intake facility. A few older borings, from previous investigations, whose logs were not available, are located nearby, with three appearing to be within the facility limits.

Generally, the stratigraphy identified in the six boring logs is more varied what was encountered in the borings near Intake 2. As shown on Figure 4B, the upper portion of the soil profile consists of very loose to dense sands and gravels with some interbedded soft to stiff silts and clays starting from approximately elevation -20-ft. to 30-ft. MSL down to about elevation -60-ft. MSL. The density of the sands and stiffness of the silts and clays increase below about elevation -50-ft. to -60-ft. MSL. Stiff to hard, low and high plasticity clays were encountered in the borings directly across from the proposed intake facility below about elevation -110-ft MSL extending down to the full depths of the borings. The borings further downstream indicate more variable conditions with interbedded sands and clays extending down to the full depths of the borings. The measurements from the one CPT (DCN3-CPT-004) indicate that relatively clayey soils extend down to an approximately elevation of -10-ft. MSL and are underlain by a 20-feet thick, moderately dense to dense sandy layer. The soils below the sandy layer appear to be interbedded sands, silts, and clays.

In general, the upper sands extending to approximately elevation -50-ft. to -60-ft. MSL are likely to be prone to liquefaction, lateral instability, seismically-induced settlement, and potentially have relatively high hydraulic conductivity and associated seepage. The lower clay layers encountered in the borings directly opposite the proposed intake structure will likely have relatively low hydraulic conductivity and would be less likely conduits for seepage into construction excavations. As was noted for Intake 2 given the alluvial depositional environment, substantial variation in the soil profile may exist land-ward from the river channel.

No peat or organic materials were identified near the ground surface in the six borings logs. However, the graphic log for DCR3-DH-005 indicates a 5-foot thick layer of peat from elevation -35-ft. to -40-ft. MSL, but the full log from this boring indicated that this layer was silty sand. Also, two borings (DCR3-DH-014 and DCR3-DH-015) had loose/soft material in the upper portion of the profile (about 4 and 7 ft thick respectively). However, samples of the soils were not retrieved from these borings to confirm their characteristics.

The URS (2009) surficial peat and organic soils map did not identify substantial thickness of surficial peat and organic soils within the proposed Intake 3 Facility area. Some minor thickness is shown within the zero-ft thickness contour indicated crossing through the site.
Intake 5:
The logs of Nine borings and two CPT (DCR5-DH-013 and DCR5-DH-014, DCR-DH-020, DCR4-DH-004, DCR4-DH-006, DCR4-DH-008, DCR-DH-004, DCN4-DH-009, DCN4-CPT-002, and DCN3-CPT-004), which were located near the proposed Intake Facility 5, as shown on Figure 5A, were available for our review. Eight of the nine borings were located in the Sacramento River channel, while one boring (DCN4-DH-009) and the two CPTs (DCN4-CPT-002, and DCN3-CPT-004) were located landward of the river but to the north outside the proposed facility as shown on Figure 5B\(^1\). A few older borings whose logs were not available were located nearby with two appearing to be within the facility limits.

The soil stratigraphy identified in the boring logs near Intake 5 (Figure 5B\(^1\)) appears to be even more variable than what was encountered in the borings and CPTs near Intakes 2 or 3. Similar to the other two intake locations, the upper portion of the soil profile near Intake 5 is generally less dense/stiff with blowcounts indicating a change in soil consistency around elevation -50-ft to -70-ft MSL. From elevation -15-ft to -20-ft MSL extending down to about elevation -50-ft to -60-ft MSL, the soil profile consist mainly of sand with some interbedded clays. Generally, from approximately elevation -60-ft to -80-ft MSL there is more clay with more sandy material further below. Soils below about elevation -50-ft to -70-ft MSL are logged as being generally dense/stiff to very dense/hard. However, note that as is typical in an alluvial/fluvial environment of deposition, the soil description of the individual layers identified in the available boring and CPT logs vary considerably from the overall trend in soil characteristics.

Based on our review and overall experience, general the upper sands extending to approximately elevation -50-ft to -60-ft MSL are likely to be prone to liquefaction, lateral instability, seismically-induced settlement, and potentially have relatively high hydraulic conductivity and associated seepage. The lower clay layers encountered in the borings directly opposite the proposed intake structure at approximately elevations -50-ft to -80-ft will likely have relatively low hydraulic conductivities and will probably be less likely conduits for seepage into construction excavations. As was noted for Intake 2 and Intake 3, given the alluvial/fluvial depositional environment, substantial variation may exist from the logs of the boring that were drill in the river channel. It should also be noted that DCN4-DH-009 indicates sands in the upper portion of the profile extending to a depth of about elevation -30-ft MSL have relatively high blowcounts. In general, these borings exhibited higher spatial variability than the borings near Intake 2.

No peat or organic materials were identified in the borings near Intake 5. One boring (DCR4-DH-008) had some very soft/loose material in the upper 4 ½ feet but no samples were retrieved from this layer so that laboratory classification test could be performed to assess whether or not these weaker soils were peat or organic. Note that the URS (2009) peat and organic surface soil thickness map identified the area of the proposed Intake Facility 5 as having minor thickness of organic soils.

**Generalized Subsurface Soil Profiles:**

For the purpose of developing quantities of the different soil types to assist B & V in their cost estimates, the generalized subsurface profiles shown in Figure 6 can be used as summarized below:

- **Intake 2:**
  5 feet of soft organic soils/peat overlying 65 feet of loose to moderately dense sand, overlying 20 feet of dense sand, overlying 40 feet of stiff moderate to high plasticity clay.

- **Intake 3:**
  5 feet of soft organic soils/peat overlying 30 feet of loose to moderately dense sand, overlying 35 feet of interbedded loose/med. stiff sands and clays, overlying 50 feet of
interbedded dense/stiff sands and clays, overlying 30 feet of stiff moderate to high plasticity clay.

- **Intake 5:**
  5 feet of soft organic soils/peat overlying 65 feet of loose to moderately dense sand, overlying 60 feet of loose to moderately dense sand, overlying 85 feet of stiff/dense to hard/very dense interbedded clays and sands.

**Primary Geotechnical Considerations**

Based on the review of the subsurface conditions near the proposed intake facilities, several important potential geotechnical issues should be considered with respect to the proposed design and construction sequencing for each facility.

- Although the presence of peat or organic soils was not noted in the boring logs that were reviewed, the presence or absence of these weak soils, especially at the surface or shallower depths needs verification. The presence of peat/organic soils in the upper portion of the profile will likely make logistics and construction layout more difficult and affect what soil improvement techniques are employed. Peat or significant amounts of organic soils extending below the proposed facilities would likely need to be removed or treated in place.

- Some soil improvement techniques may be difficult or infeasible. Deep dynamic compaction (DDC) or jet grouting may be difficult or potentially ineffective depending on the actual soil conditions on the land-side of each site. These two techniques are more effective and efficient if used in sands. More clayey deposits would tend to reduce the effectiveness of both techniques. In addition the presence of the shallow groundwater will increase costs for applying the DDC technique regardless on the soil type it is used in because the surface will have to be prepared with some sort of bridging layer to prevent excessive surface disturbances by the utilized equipment. Stone columns would introduce avenues for seepage and would likely be infeasible because of the adjacent levee and high groundwater level. Compaction grouting may be effective for liquefaction mitigation in relatively consistent sands, but less effective in more silty or clayey sands or interbedded sand and silty/clayey layers. Cement Deep Soil Mixing (CDSM) is amenable to a variety of conditions that may exist at each site and would likely be the preferred approach. Soil improvement techniques must be selected based on geotechnical information obtained during investigation at the site. A comparison of potential ground improvement techniques is included in Table 2.

- Soil improvement would have to be carefully QA/QC’d during construction and concurrent testing done to verify the improvement. This would depend on the type of improvement selected. For example, jet grouting or soil mixing may require some excavation of test columns in a test section and any of the techniques would likely require pre- and post-improvement measurements (i.e. SPT, CPT, etc.).

- Constructing the box culverts within the SR-160 fill first and then connecting to subsequent sections as well as the intake structures later would require plans to control total and differential settlements during the construction period. Soil improvements completed before the fill for the new roadway is place in concert with the foundation improvements for the remaining structures would help address this issue.

- Careful design of the soil improvement schemes in coordination with the foundation designs as well as groundwater control measure for the construction and/or operation of the Intake Facilities’ structures is important to control costs. There are likely a variety of opportunities
where value-engineering in concert with completing thorough geotechnical investigations/analyses will enhance the facility designs and construction.

Examples include:

1. Potentially using varying area replacement ratios (ARR) for jet grouting or CDSM under different elements depending on the facility performance criteria. Higher ARR (40-50% for CDSM) could be used under critical areas whereas, lower ARR (20-30% for CDSM) could be used under other lower performance criteria areas.

2. Combining some ground improvement elements with seepage control objectives, i.e. significant CDSM or jet grouting may limit groundwater seepage in areas near the river and perhaps alleviate the need for sheetpile cutoffs.

3. If some areas of the sites have sufficiently clean sandy material and the groundwater table is lower, DDC could work for a portion of each site which would reduce costs. Local dewatering might be used to create conditions where DDC would be effective. A combination of several methods could be used tailored to each site.

**Recommendations**

Based on our evaluation, we have the following recommendations:

- Ground improvement under the entire set of facility embankments and embankments for the relocated SR 160, may be cost prohibitive. Each facility has around 2 million ft$^2$ of area that would have to be improved in this case (see Figure 7). Rough cost ranges included in Table 2 would result in relatively large costs (on the order of $50-150M per facility). Considering the base estimate totals for jet grouting were about $5-12M, it is likely that this level of improvement would result in a relatively large increase in costs compared with the base estimate.

As a comparison, improving under only select elements such as the culverts or the intake structures would reduce the cost considerably but would not mitigate potential seismically-induced settlements and other foundation issues. Accordingly, if the performance criteria allows some deflection in the facility structures while maintaining operations, in order to control costs GeoPentech would recommend only targeted ground improvement of those critical facilities with zero tolerance for deflection. However, note that this may require further design evaluation and operational contingencies in order to account for the risk associated with allowing some facility deformation.

- Because very little peat/organic material has been logged in the available borings in the vicinity, and regional mapping by URS notes that little if any peat is anticipated in the areas near the 3 proposed intake locations, deep removals and over-excavations are relatively unlikely. However this statement is based on our judgment using using extremely limited land-side geotechnical data. We have assumed 5 feet of peat/organic material at the top of the soil profile will have to be removed.

- Given the predominantly sandy nature of the soils in the upper portions of nearly all the borings logs reviewed, it should be anticipated that groundwater seepage rates could be relatively high and groundwater control may be difficult. Based on the need for dewatering around the entire facility, until more detailed relevant groundwater data is available from each site, dewatering wells extending to 50-foot depth and spaced at about 50 feet apart around the site should be assumed for cost estimating purposes.
Alternatively the site could be surrounded by a slurry wall which would reduce seepage and allow passive methods of groundwater control within the excavated areas. In order to effectively cutoff the groundwater, the slurry wall should extend to at least elevation -80-ft. MSL at Intake 2, at least elevation -60-ft. MSL at intake 3, and intake 5. Note that these elevations would depend on actual site conditions.

We also believe that the slurry/sheetpile cutoff wall between the intake structure and the newly relocated SR 160 should extend at least 100 feet beyond the opposing line of dewatering wells (see Figure 7). Note that if the performance criteria do not require ground improvement under the entire length of embankments, we believe relatively limited removal of the upper soils would be required to be able to start placing new embankment fill. Hence, the slurry/sheetpile wall would not have to extend the entire length of the new relocated portion of SR-160. Also, if a slurry wall extends around the entire site, this wall could simply be tied into the rest of the slurry wall system.

It is possible that a variety of ground improvement methods employed at the same site would be more cost effective than simply choosing one approach. For example, combining local dewatering with DDC may be an effective way to trim the ground improvement costs substantially. However, until site-specific information is available, such value-engineering assumptions are difficult to justify. We recommend that field data be collected as early in the process as possible to allow such design refinement to be conducted before the facility design criteria, design concepts and construction options are finalized.

Key References


# Table 2 – Comparison of Potential Ground Improvement Methods

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<th></th>
<th>Advantages</th>
<th>Disadvantages</th>
<th>Costs*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jet Grouting</td>
<td>• Typically lower replacement ratio than mixing (5%-20%)</td>
<td>• Less effective and consistent in clayey soils</td>
<td>Relatively Expensive $80-450/CY</td>
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<tr>
<td></td>
<td>• No spoils created</td>
<td>• QA/QC difficult and improvement difficult to verify</td>
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<tr>
<td></td>
<td></td>
<td>• Typically used for specific local areas, not wide area treatment</td>
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<tr>
<td>Displacement/Compaction Grouting</td>
<td>• Relatively small equipment required</td>
<td>• Could require secondary and tertiary treatment for very low blowcount sand</td>
<td>Wide range of costs depending on site-specific conditions $35-230/CY</td>
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<tr>
<td></td>
<td>• Lower replacement ratio than mixing (3-20%)</td>
<td>• May not be effective for clayey soils</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Minimal disturbance to surface</td>
<td>• Silty or clayey sands may require additional treatment</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• No spoils created</td>
<td>• Some problems for shallow areas with little confinement</td>
<td></td>
</tr>
<tr>
<td>DDC</td>
<td>• Very low cost per cubic yard of treated volume compared to other methods under favorable conditions</td>
<td>• Shallow groundwater may require bridging surface layer</td>
<td>Very inexpensive for favorable conditions $1-4/SF</td>
</tr>
<tr>
<td></td>
<td>• No spoils created</td>
<td>• Generally limited to about 35 feet of depth. Could work for Intake 3, likely not for Intakes 2 and 5 based on the deeper loose sands</td>
<td></td>
</tr>
<tr>
<td>Stone Columns</td>
<td>• Effective for mitigating liquefaction susceptibility</td>
<td>• Introduces new seepage paths. Infeasible where groundwater flow must be controlled</td>
<td>Relatively inexpensive where feasible $15-50/LF</td>
</tr>
<tr>
<td></td>
<td>• Can provides additional bearing support</td>
<td>• Requires large amount of import rock</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• No spoils created</td>
<td>• Less effective for improving clayey material</td>
<td></td>
</tr>
<tr>
<td>CDSM</td>
<td>• Can be used to improve both sandy and clayey soils</td>
<td>• Higher replacement ratio (20-50%) required than compaction grouting</td>
<td>Moderate level costs $50-150/CY</td>
</tr>
<tr>
<td></td>
<td>• Depth limitation only based on equipment size</td>
<td>• Creates spoils that have to be handled/disposed</td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Relatively easy to QA/QC and verify improvement</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Can provide additional bearing support</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Spoils often make excellent backfill material</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Note that the cost ranges included here are intended for comparison purposes only.*
Legend

Thickness
- 0 - 5 feet
- 5 - 10 feet
- 10 - 15 feet
- 15 - 20 feet
- 20 - 25 feet
- 25 - 30 feet
- 30 - 35 feet
- 35 - 40 feet
- > 40 feet

Note: Represents thickness of organic materials near levee toe.

Source: Final DRMS Study, URS, 2009. Figure 6-34.
EXPLANATION

- Previous Investigation Location (logs unavailable)
- 2009-2012 DHCCP Completed Investigation (Borings and CPTs)
- Contours of Organic Material Thickness [ft] (URS 2009, see Figure 2)
- Logged/Interpreted Thickness of Organic Material [ft] (boring/cpt)

Notes:
- NAD83 State Plane III (ft)
- Aerial Imagery from NAIP
- Facility plan from Conceptual Engineering Report Dual Conveyance Facility Intake No. 2 Site Plan, Sheet CCO-C-1011T, transmitted by Black and Veatch March 9, 2015.
- Boring locations and names from DHCCP GDR Rev0 and DVR Geodatabase v101.

GeoPentech

Project No.: 12040A - TO3  Project: DHCCP Intake Facility Construction Sequecing  Date: Mar 2015  Figure 4A
Generalized Subsurface Profiles - Proposed Intake Locations

Project No. 15016A    Date: Mar 2015    Project: DHCCP Intake Facility Construction Sequencing    Fig 6
**Dewatering Wells:**
- Length: ~3,650 ft
- Spacing: ~50 ft
- Depth: ~50 ft
- Total Number of Wells: ~73

**Slurry/Sheetpile Cutoff Wall:**
- Wall Extent: ~100 ft beyond dewatering well extents
- Wall Length: 2,850 ft
- Sheetpile Depth: ~50 ft
- Total SF of Slurry/Sheetpile Wall: 142,500 ft²

**PROPOSED INTAKE #3**

**Ground improvement under embankments and culverts:**
- Area: ~1,966,750 ft²
- ARR: 30%
- Depth of treatment: ~70 ft
- Total volume of CDSTM: 1,529,694 CY

**Culvert Pile Alternative:**
- 12 x 12 ft Culverts: 12
- Culvert Length: 365 ft
- Pile Lengths: 70 ft
- Pile Spacing: 10 ft x 2 rows
- Total Potential Pile Footage: 5,390 ft

**Culvert CDSTM Alternative:**
- 12 x 12 ft Culverts: 12
- Culvert Length: 365 ft
- Improvement Width: 22 ft
- Improvement Depth: 70 ft
- ARR: 50%
- Total Volume of Potential CDSTM: 131,796 CY

**Ground treatment under intake structure only:**
- Area: 40 ft x 1,667 ft = 66,680 ft²
- ARR: 50%
- Depth of treatment: ~70 ft
- Total volume of CDSTM: 86,437 CY
- Number of Uplift Anchors (1/3 of CDSTM Shafts): ~1,150
### Consistency of Cohesive Soils (AASHTO)

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Pocket Penetrometer (tsf)</th>
<th>Torvane (tsf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>&lt; 0.25</td>
<td>&lt; 0.12</td>
</tr>
<tr>
<td>Soft</td>
<td>0.25 – 0.50</td>
<td>0.12 – 0.25</td>
</tr>
<tr>
<td>Med um St ff</td>
<td>0.50 – 1.0</td>
<td>0.25 – 0.50</td>
</tr>
<tr>
<td>St ff</td>
<td>1.0 – 2.0</td>
<td>0.50 – 1.0</td>
</tr>
<tr>
<td>Very St ff</td>
<td>2.0 – 4.0</td>
<td>1.0 – 2.0</td>
</tr>
<tr>
<td>Hard</td>
<td>&gt; 4.0</td>
<td>&gt; 2.0</td>
</tr>
</tbody>
</table>

### Apparent Density of Cohesionless Soils (ASTM 6066)

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>SPT N(60) - Value (blows / foot)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0</td>
</tr>
<tr>
<td>Loose</td>
<td>5</td>
</tr>
<tr>
<td>Med um Dense</td>
<td>31</td>
</tr>
<tr>
<td>Dense</td>
<td>50</td>
</tr>
<tr>
<td>Very Dense</td>
<td>&gt; 50</td>
</tr>
</tbody>
</table>

### Moisture (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry</td>
<td>Absence of moisture, dusty, dry to the touch</td>
</tr>
<tr>
<td>Mo st</td>
<td>Damp but not very wet, hard</td>
</tr>
<tr>
<td>Wet</td>
<td>Very wet, free water, usuas y s m b e w ater lab e</td>
</tr>
</tbody>
</table>

### Soil Particle Size (ASTM D 2488)

<table>
<thead>
<tr>
<th>Size</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>3/4 inch to 3 inches</td>
</tr>
<tr>
<td>Fine</td>
<td>No. 4 S eve to 3/4 inch</td>
</tr>
<tr>
<td>Med um</td>
<td>No. 10 S eve to No. 4 S eve</td>
</tr>
<tr>
<td>Fine</td>
<td>No. 200 S eve to No. 40 S eve</td>
</tr>
<tr>
<td>S t and C a y</td>
<td>Pass ng No. 200 S eve</td>
</tr>
</tbody>
</table>

### Description of Organics

<table>
<thead>
<tr>
<th>Organic Content</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>50 to 100%</td>
<td>“Peat” (Hum. f cast on factor of H1 to H3)</td>
</tr>
<tr>
<td>15 to 50%</td>
<td>“Hem c” (Hum. f cast on factor of H4 to H6)</td>
</tr>
<tr>
<td>5 to 15%</td>
<td>“Sap c” (Hum. f cast on factor of H7 to H10)</td>
</tr>
<tr>
<td>10% or less</td>
<td>“Organ c” (Soil name)</td>
</tr>
</tbody>
</table>

### Plasticity of Fine-Grained Soils (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonast c</td>
<td>A 1/8 inch thread cannot be ro ed at any water content.</td>
</tr>
<tr>
<td>Low</td>
<td>The thread can bare y be ro ed, and the ump cannot be formed when dr er th the past c m t.</td>
</tr>
<tr>
<td>Med um</td>
<td>The thread is easy to ro s and not much time is required to reach the past c m t; t cannot be roed ed after reach ng the past c m t. The ump crumb es when dr er than the past c m t.</td>
</tr>
<tr>
<td>H gh</td>
<td>It takes cons derable tme ro ng and knead ng to reach the past c m t. The thread can be ro ed severa tmes after reach ng the past c m t. The ump can be formed w hout crumb ng when dr er than the past c m t.</td>
</tr>
</tbody>
</table>

### Dry Strength of Fine-Grained Soils (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>Dry spec men crumb es nto powder w th mere pressure of hand ng.</td>
</tr>
<tr>
<td>Low</td>
<td>Dry spec men crumb es nto powder w th some fnger pressure.</td>
</tr>
<tr>
<td>Med um</td>
<td>Dry spec men breaks nto p eces or crumb es w th cons derab e fnger pressure.</td>
</tr>
<tr>
<td>H gh</td>
<td>Dry spec men cannot be broken w th fnger pressure; w break nto p eces between thumb and hard surface.</td>
</tr>
<tr>
<td>Very H gh</td>
<td>Dry spec men cannot be broken between thumb and a hard surface.</td>
</tr>
</tbody>
</table>

### Dilatancy of Fine-Grained Soils (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>No v s b e change n the spec men.</td>
</tr>
<tr>
<td>S ow</td>
<td>Water appears s ow y on the surface of the spec men dur ng shak ng and does not d sappear, or d sappears s ow y, upon squee ng.</td>
</tr>
<tr>
<td>Rap d</td>
<td>Water appears qu c k y on the surface of the spec men dur ng shak ng and d sappears qu c k y upon squee ng.</td>
</tr>
</tbody>
</table>

### Toughness of Fine-Grained Soils (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>On y s ght pressure s requ ed to ro the thread near the past c m t. The thread and the ump are weak and soft.</td>
</tr>
<tr>
<td>Med um</td>
<td>Med um pressure s requ ed to ro the thread near the past c m t. The thread and the ump have med um st finess.</td>
</tr>
<tr>
<td>H gh</td>
<td>Cons derab e pressure s requ ed to ro the thread to near the past c m t. The thread and the ump have very h gh st finess.</td>
</tr>
</tbody>
</table>

### Cementation (ASTM D 2488)

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak</td>
<td>Crumb es or breaks w th hand ng or fnger pressure.</td>
</tr>
<tr>
<td>Moderate</td>
<td>Crumb es or breaks w th cons derab e fnger pressure.</td>
</tr>
<tr>
<td>Strong</td>
<td>W/ not crumb b e or break w th fnger pressure.</td>
</tr>
</tbody>
</table>
Appendix C

WaterFix BTO Conceptual Construction Schedule
## CALIFORNIA WATERFIX 9000 CFS CONSTRUCTION SCHEDULE

<table>
<thead>
<tr>
<th>Activity ID</th>
<th>Activity Name</th>
<th>Original Duration</th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>8013</td>
<td>BT Dike Embankment, S, W, &amp; N Sides</td>
<td>529</td>
<td>01-Aug-23</td>
<td>11-Nov-24</td>
</tr>
<tr>
<td>8015</td>
<td>BT Dike Foundation, E Side Along Italian S</td>
<td>140</td>
<td>13-Oct-23</td>
<td>15-May-24</td>
</tr>
<tr>
<td>8016</td>
<td>BT Dike Embankment, E Side Along Italian S</td>
<td>227</td>
<td>26-May-24</td>
<td>02-Apr-25</td>
</tr>
<tr>
<td>8034</td>
<td>Canal From Golf Aqueduct To Delta-Bidwell</td>
<td>341</td>
<td>05-Dec-23</td>
<td>09-Apr-25</td>
</tr>
<tr>
<td>8038</td>
<td>Canal S Side Drain &amp; Irrigation Ditches Along ROW</td>
<td>167</td>
<td>11-Sep-24</td>
<td>07-May-25</td>
</tr>
<tr>
<td>8040</td>
<td>Canal &amp; Dike SWPPP</td>
<td>742</td>
<td>17-Jan-22</td>
<td>31-May-28</td>
</tr>
<tr>
<td>8042</td>
<td>Canal &amp; Dike Finale Out</td>
<td>129</td>
<td>16-Nov-24</td>
<td>29-May-25</td>
</tr>
<tr>
<td>8101</td>
<td>Outlet Works Epicentre, Byron Tract Forebay</td>
<td>74</td>
<td>24-Aug-23</td>
<td>06-Dec-23</td>
</tr>
<tr>
<td>8103</td>
<td>Outlet Works Epicentre, Byron Tract Forebay</td>
<td>369</td>
<td>11-Dec-23</td>
<td>09-Jul-25</td>
</tr>
<tr>
<td>8121</td>
<td>Banks Channel Control Structure Excavation</td>
<td>103</td>
<td>06-Feb-23</td>
<td>06-Jul-23</td>
</tr>
<tr>
<td>8123</td>
<td>Banks Channel Control Structure Concrete</td>
<td>249</td>
<td>07-Jul-23</td>
<td>31-Jul-23</td>
</tr>
<tr>
<td>8131</td>
<td>Jones Channel Control Structure Excavation</td>
<td>72</td>
<td>07-Jul-23</td>
<td>25-Oct-23</td>
</tr>
<tr>
<td>8133</td>
<td>Jones Channel Control Structure Concrete &amp;</td>
<td>249</td>
<td>27-Oct-23</td>
<td>23-Oct-24</td>
</tr>
<tr>
<td>8140</td>
<td>Old River Fish Control Structure Excavation</td>
<td>51</td>
<td>12-Feb-24</td>
<td>25-Apr-24</td>
</tr>
<tr>
<td>8141</td>
<td>Old River Fish Control Structure Concrete</td>
<td>204</td>
<td>26-Jan-24</td>
<td>15-Jul-25</td>
</tr>
<tr>
<td>8145</td>
<td>New Spillway Excavation</td>
<td>72</td>
<td>27-Oct-23</td>
<td>06-Feb-24</td>
</tr>
<tr>
<td>8146</td>
<td>New Spillway Concrete</td>
<td>102</td>
<td>01-Apr-24</td>
<td>30-Jan-25</td>
</tr>
</tbody>
</table>

### BT COMBINED PUMP PLANT & SURGE SHAFT

#### Common Activities
<table>
<thead>
<tr>
<th>Activity ID</th>
<th>Activity Name</th>
<th>Original Duration</th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>L50.0002</td>
<td>Surface Contractor Mobilization</td>
<td>88</td>
<td>07-Apr-31</td>
<td>04-Aug-31</td>
</tr>
<tr>
<td>L50.0003</td>
<td>Surface Contractor Site Staff</td>
<td>579</td>
<td>07-Apr-31</td>
<td>23-Jun-33</td>
</tr>
<tr>
<td>L50.0004</td>
<td>RT Underground Excavation Support</td>
<td>2373</td>
<td>05-May-23</td>
<td>23-Jun-33</td>
</tr>
<tr>
<td>L50.0300</td>
<td>Prone &amp; Stiles - Surge Gates &amp; Operators</td>
<td>310</td>
<td>01-Jul-31</td>
<td>05-Sep-31</td>
</tr>
<tr>
<td>L50.0400</td>
<td>Procure &amp; Deliver - Pumps &amp; Valves</td>
<td>824</td>
<td>25-Jul-31</td>
<td>35-Sep-31</td>
</tr>
</tbody>
</table>

#### Site Work
<table>
<thead>
<tr>
<th>Activity ID</th>
<th>Activity Name</th>
<th>Original Duration</th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>L51.0000</td>
<td>Temp Facilities/Offices/Shops/Roads/Utilities++</td>
<td>111</td>
<td>04-Jan-22</td>
<td>09-Jun-22</td>
</tr>
<tr>
<td>L51.0003</td>
<td>C/Dam Construction &amp; Wall/Water/Old River</td>
<td>72</td>
<td>25-Jan-22</td>
<td>05-May-23</td>
</tr>
<tr>
<td>L51.0004</td>
<td>EW 32 Acre Site Initial Earthwork</td>
<td>120</td>
<td>26-Dec-22</td>
<td>30-Jun-23</td>
</tr>
<tr>
<td>L51.0005</td>
<td>EW 32 Acre Site Fill 1 to 2 to H &amp; I RipRap</td>
<td>80</td>
<td>11-Nov-31</td>
<td>15-Mar-23</td>
</tr>
<tr>
<td>L51.0006</td>
<td>EW 32 Acre Site Final Site Work</td>
<td>90</td>
<td>11-Nov-31</td>
<td>15-Mar-23</td>
</tr>
<tr>
<td>L51.0007</td>
<td>Substation &amp; Electrical Distribution</td>
<td>100</td>
<td>03-Jul-23</td>
<td>29-Nov-23</td>
</tr>
</tbody>
</table>

#### Pump Shafts
<table>
<thead>
<tr>
<th>Activity ID</th>
<th>Activity Name</th>
<th>Original Duration</th>
<th>Start</th>
<th>Finish</th>
</tr>
</thead>
<tbody>
<tr>
<td>L52.0000</td>
<td>PS West Outer Wall above -1 to</td>
<td>100</td>
<td>23-Apr-23</td>
<td>19-Sep-23</td>
</tr>
<tr>
<td>L52.0001</td>
<td>PS West Operating Deck</td>
<td>60</td>
<td>11-Aug-23</td>
<td>02-Dec-23</td>
</tr>
<tr>
<td>L52.0002</td>
<td>PS West Pumps &amp; Operators</td>
<td>110</td>
<td>26-Sep-23</td>
<td>25-Feb-23</td>
</tr>
<tr>
<td>L52.0003</td>
<td>PS West Piping, Pumps to Spillway</td>
<td>78</td>
<td>11-Nov-31</td>
<td>28-Feb-23</td>
</tr>
<tr>
<td>L52.0004</td>
<td>PS East Outer Wall above -1 to</td>
<td>100</td>
<td>03-Jan-23</td>
<td>22-May-30</td>
</tr>
<tr>
<td>L52.0005</td>
<td>PS East Operating Deck</td>
<td>80</td>
<td>18-Apr-30</td>
<td>07-Aug-30</td>
</tr>
<tr>
<td>L52.0006</td>
<td>PS East Pumps &amp; Operators</td>
<td>110</td>
<td>25-Sep-31</td>
<td>25-Feb-32</td>
</tr>
<tr>
<td>L52.0007</td>
<td>PS East Piping, Pumps to Spillway</td>
<td>78</td>
<td>11-Nov-31</td>
<td>28-Feb-23</td>
</tr>
<tr>
<td>L52.0008</td>
<td>BT Pump Shaft Construction Plant</td>
<td>0</td>
<td>03-Jan-30</td>
<td>09-Jun-30</td>
</tr>
</tbody>
</table>

*Page 2 of 10*
### California WaterFix 9000 CFS Construction Schedule

<table>
<thead>
<tr>
<th>Activity ID</th>
<th>Activity Name</th>
<th>Original Start</th>
<th>Original Finish</th>
<th>Milestone</th>
<th>Remaining Work</th>
<th>Remaining Level of Effort</th>
<th>Actual Work</th>
<th>Critical Remaining Work</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>LS1.0500</td>
<td>West Pump Plant Concrete Walls</td>
<td>06-Dec-29</td>
<td>10-Apr-27</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS1.0505</td>
<td>East Pump Plant Concrete Walls</td>
<td>06-Aug-30</td>
<td>16-Dec-30</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS1.0510</td>
<td>West Overhead Cranes</td>
<td>14-Apr-27</td>
<td>07-Jun-27</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS1.0515</td>
<td>East Overhead Cranes</td>
<td>19-Dec-30</td>
<td>06-Feb-31</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS1.0520</td>
<td>West Pump Plant Roof Structure</td>
<td>06-Jun-27</td>
<td>06-Sep-27</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LS1.0825</td>
<td>East Pump Plant Roof Structure</td>
<td>08-Feb-31</td>
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## CALIFORNIA WATERFIX 9000 CFS CONSTRUCTION SCHEDULE

### INTAKES

#### Common Activities

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# California Waterfix 9000 CFS Construction Schedule

## Reach 7

### General
- **B1001**: NTW Reach 7
- **B1010**: Tunnel Work Consistent With Surface Activities
- **B1100**: Order & Manufacture East TBM
- **B1160**: Order & Manufacture West TBM
- **B1360**: Mud Disposal CONVEYOR SET UP
- **B2010**: Operate Mud Disposal Area
- **B2030**: Operate Tunnel Water Treatment Plant
- **B2040**: Induced Tunnel & Shaft
- **B2090**: Grouting & Sbm Mud Disposal Area
- **B2095**: Final Clean & Cleanup Mud Disposal Area
- **B2099**: Earn Hire Mud Disposal Area
- **B2100**: Turn Over To East Pump Plant Cover
- **B2150**: Turn Over To West Pump Plant Gear

### Launch Shaft (Pump/Surge Shaft)
- **B1020**: Setup East Pump Plant
- **B1030**: East Launch Shaft Stair Wall Installation
- **B1040**: Extrude East Launch Pump Shaft
- **B1050**: Tie Repar Truss Slab
- **B1080**: Setup & place trende slab
- **B1070**: Pump Work From Shaft
- **B1060**: Cure Time Truss Slab
- **B1080**: Tie Repar Work Slab
- **B1090**: Setup & Place Work Slab
- **B1110**: Tie Repar Thrust Ring
- **B1120**: Set Expansion Forms & Assemble Wall Forms
- **B1130**: Form Throat Ring
- **B1160**: Plume Ring
- **B1200**: E Pump Plant St & Strip Elbow forms
- **B1210**: Tie Tie
- **B1220**: Place west wall shaft Concrete
- **B1230**: Pump Plant Wall Rebar Below EL-2
- **B1300**: Assemble Pump Plant Wall Forms
- **B1370**: Set & Strip Pump Plant Wall Forms
- **B1380**: Place Pump Wall Concrete
- **B1510**: Setup Bow Pump Plant Shaft
- **B1520**: West Launch Shaft Bury Wall Installation

### Activity Details

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- 2022
- 2023
- 2024
- 2025
- 2026
- 2027
- 2028
- 2029
- 2030
- 2031
- 2032
- 2033
- 2034
- 2035
- 2036
- 2037
- 2038

### Status

- Remaining Level of Effort
- Remaining Work
- Milestones

**Actual Work**
- Critical Remaining Work

**Summary**

Page 0 of 10

Task Type: All Activities

03-Jul-18

(c) Primavera Systems, Inc.
## CALIFORNIA WATERFIX 9000 CFS CONSTRUCTION SCHEDULE

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### Intervention Grout Zones

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### Tunnel Excavate & Support

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*Page 8 of 10, TASK Filter: All Activities*

03-Jul-18

(c) Primavera Systems, Inc
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# California WaterFix 9000 CFS Construction Schedule

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## Launch Shaft

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Legend:
- Remaining Level of Effort
- Remaining Work
- Milestone
- Actual Work
- Critical Remaining Work
- Summary
### CALIFORNIA WATERFIX 9000 CFS CONSTRUCTION SCHEDULE

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#### Intervention Grout Zones

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#### Tunnel Excavate & Support

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### REACH 1 & 2

#### General

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Page 10 of 10  TASK: All Activities

(c) Primavera Systems, Inc  03-Jul-18
## CALIFORNIA WATERFIX 9000 CFS CONSTRUCTION SCHEDULE

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### Intermediate Shafts & Safe Havens

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### Tunnel Excavate & Support

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(c) Primavera Systems, Inc
1. Background

Under Task Order 5, a surge evaluation of the DHCCP Bay Delta Conveyance System (system) for a pump trip event was performed. The evaluation used a pre-event pumping rate of 9,000 cubic feet per second (cfs). The corresponding transient response within the system is a complex combination of both pressure segments (ex., tunnels) and free water surface segments (ex, pump station and surge shafts/overflow weirs, vent shafts along the tunnel alignment, Intermediate Forebay (IF), and sediment basins) open to atmospheric pressure.

Because of the many free surface aspects in the system, the InfoWorks CS model previously developed by AECOM for non-surge related hydraulic analysis was adapted and used to conduct the surge evaluation. Adaptations to the InfoWorks CS model included modified simulation parameters to better approximate sharp shocks (water hammer and pressure wave velocities) that are more characteristic of surge events in closed pipe reaches of the system.

1.1 Adverse Hydraulic Transient Condition Considered

Pressure transients, also often referred to as surge, or water hammer, can occur whenever flow conditions change within a water transmission system. Examples of such conditions include a sudden flow change resulting from rapid closure of a valve or from loss of power to pumps. For the vast majority of these transients, the impacts are not significant and specific control facilities are not necessary for protection. However, in extreme cases, pressure transients can result in damage to the conveyance system, and/or flooding damage.

One of the more critical conditions is the transient impacts associated with a total power failure (i.e., a pump trip) during peak delivery rates. This is the situation that has been analyzed herein.

Without surge mitigation, when the pumps at the Clifton Court Pump Station (CCPS) suddenly lose power and have no provision for overflow in a closed system, the water within the CCPS shaft is
rapidly brought to rest by the impulse of the higher pressure developed at the face of the pump impellers. As soon as the first, adjacent volume of water is brought to rest, the same action is applied to the next upstream segment of fluid bringing it also to rest. In this manner, a pulse wave of high pressure travels upstream at some sonic wave speed “a” and at a sufficient pressure to bring the fluid to rest. With the pressure increase, the tunnel expands slightly and the kinetic energy is converted to elastic energy in the pipe.

When this pressure wave reaches the IF - the boundary condition, the fluid in the tunnel is under the extra head required to stop the flow. At this point, the elastic energy in the pipe is lost as the pressure is suddenly released to the IF. With the lost pressure, the tunnel contracts releasing the stored energy and reversing the flow. This reflection process is repeated until the action of friction, the imperfect elasticity of fluid, and the tunnel wall dampens out the pressure waves – eventually bringing the fluid to rest at the constant river elevation.

This process is represented by the sketch presented below, which was taken from Figure 5-3 in Wylie’s book on transient in systems (Wylie, 1993).

While the above represents a theoretical condition, in actuality - for the DHCCP system, the compression (i.e., pressure) wave traveling upstream does not bring the fluid to rest because there is an overflow relief at the surge shaft weirs and - as a result, the magnitude of the potential surge is lessened.

1.2 Simulations

Once the InfoWorks CS model was adapted for surge evaluations (specific adaptations are discussed in Section 2 below), model runs were performed in order to evaluate system response to surge events under operational parameters determined by MWD. These runs can be grouped into the following two types of investigations.

- A set of runs to investigate the influence of IF size (bottom footprint) on the transient response: 500’x500’, 800’x1,000’, and 800’x1,500’.

- A series of both steady state and transient runs to investigate what happens at the intake screens when Intake 2 is off-line.

2. Technical Approach

As mentioned previously, the transient response within the system from a pump trip event is a complex combination of both pressure and free surface elements. Because of the free surface aspects in this system, the InfoWorks CS model was used. However, the simulation parameters
within the InfoWorks CS model were modified to better approximate a sharp shock to the system, similar to a water hammer event.

InfoWorks CS solves the open channel form of the 1-dimensional St. Venant equations of fluid flow. When pressurized (or surcharged) conditions occur, a numerical approach called the Preissman Slot is used to permit the continued use of the open channel formulation of the St. Venant equations.

The numerical approach using a Preissmann Slot was developed by Cunge and Wegner, 1964. The Preissmann Slot is a fictitious slot along the top of the tunnel extending upward. The purpose of the slot is to provide a cross-sectional area whose top slot width is very small for all flow depths greater than the top of the tunnel. The small top width is required to produce the proper celerity for pressurized flow as computed from the gravity wave celerity equation. By using the slot width and the acoustic wave speed (fps) in the gravity wave celerity equation, the slot width can be solved for as a function of acoustic wave speed.

\[ b = 0.25 g \pi \left( \frac{d}{a} \right)^2 \]  
Eqn. 1

Where \( a \) is the acoustic wave speed, \( b \) is the slot width, and \( d \) is the tunnel diameter (ft). From an inspection of this relationship, it can be observed that as the acoustic wave speed increases, the slot width becomes smaller.

In InfoWorks CS, the simulation (SIM) parameters associated with control of the slot width are:

- BSLMIN, which is the minimum slot width allowed. This is important to specify because the slot width is automatically calculated by the model based upon specification of the Celerity Ratio (CELRAT), which in turn is a function of tunnel diameter. Model default is 0.001m.
- CELRAT, which is the Celerity Ratio. Model default is 10.

What this means is that if the user does not specify these SIM parameters, the model will automatically calculate the slot width as approximately 2% of the tunnel width. This corresponds to a Celerity Ratio of 10. For a 40-foot diameter tunnel, the default slot width in InfoWorks CS would therefore be 9.6-inches. Using Eqn. 1 above the associated acoustic wave speed would be 225 fps. The potential effect of such a low wave speed can be to dampen out shorter periods of pressure wave reflections and the potential over-damping of pressure spikes (i.e., increases in hydraulic grade line (HGL)).

As part of a sensitivity investigation, the Celerity Ratio was modified to consider to faster acoustic wave speeds: 700 and 1700 fps. For these two cases, the InfoWorks CS SIM parameters were modified as follows:

- 700 fps – BSLMIN = 0.083 ft and CELRAT = 31.
- 1700 fps – BSLMIN = 0.014 ft and CELRAT = 76.

This sensitivity investigation was carried out for a Sacramento River EL 10.0, 9,000 cfs, 500’x500’ Intermediate Forebay (IF), and a pump trip of 10 seconds. Figure 1 through Figure 4 plot model results at various noted locations. Each plot has each of the following three runs shown: red line (default SIM settings), green line (CELRAT = 31) and blue line (CELRAT = 76). Figure 5 shows the results for the CELRAT = 76 case only, but shows the HGL on the upstream (blue line) and
downstream side (green line) of the Intermediate Forebay (IF). This plot shows the degree of IF dampening on the pressure waves.

After a review of the sensitivity results and a discussion with MWD staff, it was concluded that using a CELRAT of 76 (i.e., acoustic wave speed = 1,700 fps) was a good match with calculated acoustic wave speeds for the tunnel.
Figure 1 – HGL (ft) at the Staten Island Vent Shaft (located downstream of the IF)

Figure 2 – Same as Figure 1, enlarged to show the first 40 minutes
Figure 3 – HGL (ft) at the CC Surge Shaft

Figure 4 – Flow (cfs) in the Surge Bypass Channel
2.1 Software Used

InfoWorks CS is a well-established and widely used numerical model which dynamically solves the one-dimensional St. Venant equations of fluid flow. The development of the InfoWorks CS model for DHCCP Bay Delta Conveyance System is documented in TMs 1 and 2 under this Hydraulic Operation Equalization Study task. Technical documentation of InfoWorks CS can be found at www.innovyze.com.

2.2 Validation of InfoWorks CS Model

For a validation of the InfoWorks CS model, H2OSurge version 10.0 (SP3, Update #4) was used to perform a theoretical simulation, which was then compared to InfoWorks CS results. H2OSurge is developed by Innovyze to assist engineers in evaluating water hammer for water distribution systems, pump stations and transmission mains. H2OSurge solves the basic equations of fluid mechanics for the transient flow of an incompressible fluid using the wave characteristic method. Technical documentation of H2OSurge can be found at www.innovyze.com.

The use of InfoWorks CS to evaluate surge conditions through a modification of the slot width is not a new approach. In fact, since the Preissmann Slot was first developed (Cunge and Wegner, 1964), the size of the slot width has been adjusted to better match various conditions. This is perhaps most recently documented in a study by Karen E. Ridgway (CHI 2010 conference) titled “Evaluating Force Main Transients with SWMM5 and Other Programs”.

In this paper, various model’s ability to correctly simulate water hammer associated with pump failure were investigated. The programs evaluated included: SWMM5 by the United States Environmental
The study found that, with careful adjustment to the slot width, the InfoWorks CS accurately simulated the resulting water hammer behavior from pump failure. Specifically, it was found that the slot width needed to be adjusted by setting the CELRAT to 80 in the InfoWorks CS model. This is very close to the CELRAT of 76 calculated during the sensitivity investigation discussed previously (in Section 2), which was based upon a calculated acoustic wave speed of 1,700 fps for the DHCCP Bay Delta Conveyance System tunnels.

In addition to a literature comparison, the modified (i.e., CELRAT of 76) InfoWorks CS model was compared against results form H2OSurge on a theoretical problem. In order to perform such a model to model validation, a physical problem similar to the DHCCP Bay Delta Conveyance System was developed where the free surface and storage aspects of the DHCCP Bay Delta Conveyance System were removed. This is important because H2OSurge assumes a pressurized system and this model to model comparison is aimed at a validation of the modified InfoWorks CS model’s ability to capture maximum pressure spikes in a long tunnel system.

The theoretical problem test involved a single 40-foot tunnel segment running directly from the CCPS to the Sacramento River. The total length of tunnel was set to 184,000 feet. The river was set to an EL 10.0 and the CCPS shaft had an invert of EL -163.00 feet and a top elevation sufficient to fully contain the transient. The IF was not included nor was an overflow weir at the CCPS surge shafts. A pump trip “spin down” time of 10 seconds was used with a total pumping rate of 4,500 cfs immediately prior to the pump trip.

Figure 6 shows a water level history plot at the surge shaft located at the CCPS. There are several features on this plot which are called out on the figure by the following numbered call-outs.

1. Steady state hydraulics prior to the pump trip. Sacramento River EL 10.0 and a delivery rate of 4,500 cfs.

2. Pump trip. Pumps at CCPS spin down over a 10 second period. This period corresponds to an increase in system head (i.e., water surface elevation) required to stop the flow.

3. A period where a first reflection of the pressure wave has not returned from reflecting off the river. Flow at the CCPS has come to rest.

4. Associated with the return of the 1st compression wave reflected off the upstream river.

Figure 6 shows good correlation between H2OSurge and InfoWorks CS in the principal objective of capturing of the maximum pressure spike, or rise in water surface elevation at the CCPS surge shaft. In general, the results from H2OSurge are sharper than InfoWorks CS. This is due to the fundamental differences in the two model’s numerics but also because InfoWorks CS does not treat the upstream boundary condition as a rigid reservoir. These two differences result in a more dampened response. Upon the return of the 1st compression wave (call-out 4), H2OSurge results show a deeper down surge. Inspection of these results suggests this is related to how the upstream (river) or downstream (pumps) were specified in InfoWorks CS for this validation run. However, because the principal objective of testing the ability of InfoWorks CS to capture the maximum pressure spike, no further investigation was made.
Figure 6 – InfoWorks CS vs. H2OSurge Comparison
2.3 Assumptions

2.3.1 Layout and Profile

TM 3 builds upon draft of TM 2 (rev0) submitted to MWD in September 2014. In regards to the physical layout and profile, Run ID TM2_Ex. (TM 2, Table 1) was used. No other changes were made, except as follows:

- Intake River Elevation set at EL 10.0.
- IF floor elevation set at a constant EL -17.0 and sized as shown below in Figure 7.
- New surge shaft layouts located just upstream of the CCPS were provided as shown in Figure 8.
  - Fixed surge overflow weirs set at EL 14.6. Top of shaft ceiling set at EL 30.0.
  - Gravity gates were assumed to be closed. This would result in the highest water surface elevation over the surge weirs.

![Figure 7 - New IF Layout](image)

\[\text{Known: Elevation, height, slope, length L1, width W1} \]
\[\text{Unknown: length L2, width W2}\]

\[\text{Figure 7 - New IF Layout}\]

\[\text{Provided to AECOM by MWD.}\]
Figure 8 - New Surge Shaft Layout1
2.3.2 **Roughness**

As used in TM 1, a Manning roughness value of 0.0145 had been established during previous hydraulic investigations by MWD and others. This value is adequately conservative for the purposes of planning.

2.3.3 **Sonic Velocities**

The magnitude of the transients is dependent on the speed at which pressure waves propagate in the system. In turn, the speed of these pressure waves is dependent on the diameter, material type, and ground conditions of the tunnel. For this analysis, a uniform wave speed of 1,700 fps was used throughout the entire system. Small variations in wave speed (on the order of 10 to 15 percent) normally do not have a major impact on transient results.

2.3.4 **Pump Characteristics**

For the purposes of this analysis, the pumps were assumed to spin-down in 10 seconds\(^2\) after a pump trip. A total pumping capacity of 9,000 cfs was used.

### 3. Results

3.1 **General System Performance**

Prior to evaluating specific operational conditions and questions listed previously (Section 1.2), the overall system was evaluated over a range of conditions in order to test for adverse conditions resulting from a pump trip event. These conditions looked at a range of river water levels (i.e., EL 15.0 and EL 31.4) and also peak pumping capacities of the system.

3.1.1 **Travel Time of Wave**

When performing a transient analysis it is worthwhile to calculate a travel time of a pressure wave in the system. As was presented in Section 2.3.3, the sonic velocity (or wave speed) was estimated at 1,700 fps. Functionally, this means that when a sudden change in the velocity at the CCPS occurs (i.e., pump trip event), this information travels upstream at a speed of 1,700 fps. It is therefore possible to predict when a reflection off an upstream boundary with sufficient mass (i.e., Sacramento River) will occur. This is calculated by \(2L/a\), where \(L\) is the distance traveled (ft) and \(a\) is the wave speed.

For the DHCCP system, the return pressure wave will vary somewhat as reflection points are at different distances – each associated with an intake location. Considering this, one can expect a significant return wave from the Sacramento River within about 216 to 244 seconds after the start of a pump trip event. That means that during this period of time, when flow stoppage is being transmitted upstream, the flow entering the surge shafts will be reduced but not completely stopped because the water level will only build until it finds relief at the surge shaft weirs.

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\(^2\) Provided to AECOM by MWD.
3.1.2 Findings

1. After a sudden pump trip, complete stoppage of the flow does not occur because the surge shaft weirs allow some forward moving flow to continue. While this results in overflow to CCF it will be less than the delivery demand from the pumps of 9,000 cfs and actually limits the typical head build-up that would otherwise be required to stop the flow. In effect, the surge shaft weirs act as a large shock absorber to the system – gradually slowing the forward flow and limiting the degree of head imbalance that would otherwise occur between the CCPS and the upstream IF and Sacramento River.

For a short period of time, before the IF recovers to the river elevation, the water level in the surge shafts is higher than the IF and reverse flow occurs between the surge shaft and the IF. The timing is such that the IF level rises slightly above the river elevation for a brief period of time (on the order of 10-20 minutes). This results in a small reverse flow to the river at Intakes 5 and 3.

2. The characteristic response observed does suggest that reverse flows into the Sacramento River are a possibility during conditions when a head imbalance occurs. A head imbalance will occur when the water level at the surge shaft weirs (EL 14.6) is equal or higher than the Sacramento River water elevation.

During conditions where the Sacramento River water elevations are much higher than EL 14.6 little, or no, reverse flow will occur. However, in conditions where the Sacramento River water surface elevations are lower than EL 14.6 measurable reverse flow will occur. This condition creates a scenario that as flow stoppage occurs at the CCPS, the water level quickly rises to an elevation somewhat greater than EL 14.6. When the compression wave returns, a head imbalance has developed and flows will reverse back up the system towards the Sacramento River. While this condition does not pose a surge related risk to the CCPS or CCF, it does potentially create back flow through the intake screens into the river during periods of river levels below EL 14.6 unless checking gates or other control measures are used to prevent the backflow.

3.2 Testing the Influence of Various IF Sizes on the Transient Response

After the initial testing discussed in Section 3.1 above, a set of runs were performed to investigate the influence of IF size on the transient response: 500’x500’, 800’x1,000’, and 800’x1,500’. In particular, it was of interest to further evaluate the reverse backflow conditions at the Intakes and document the degree and nature of this backflow condition.

Two physical parameters of interest were extracted from the InfoWorks CS model: the volume of backflow during a pump trip event and the maximum velocity through the sediment basins. The later parameter (maximum velocity) was looked at in order to understand the likelihood of sediment re-entrainment. Table 1 summarizes these parameters for each of the three runs of increased IF footprint.

Results show backflow occurring principally at Intakes 3 and 5. Peak velocities through the sedimentation basins are all extremely low, thus the likelihood of sediment re-entrainment is negligible.
Table 1 – Physical Parameters During a Backflow Event at the Intakes

<table>
<thead>
<tr>
<th>Intake No.</th>
<th>IF = 500' x 500'</th>
<th>IF = 800' x 1,000'</th>
<th>IF = 800' x 1,500'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Peak Backflow (cfs)</td>
<td>Peak Backflow Velocity (ft/s)</td>
<td>Backflow Volume (ac-ft)</td>
</tr>
<tr>
<td>2</td>
<td>37</td>
<td>0.010</td>
<td>2.4</td>
</tr>
<tr>
<td>3</td>
<td>166</td>
<td>0.046</td>
<td>11.3</td>
</tr>
<tr>
<td>5</td>
<td>217</td>
<td>0.058</td>
<td>12.8</td>
</tr>
</tbody>
</table>

For each of the three runs, a series of flow and water level plots were generated in order to document the impact of the various IF sizes. In the figures, the three line colors represent the following: Blue → 500’x500’ IF, Green → 800’x1,000’ IF, and Red → 800’x1,500’ IF.

Figure 9 and Figure 10 show the HGL on either side of the IF. Results show an observable dampening effect from the IF.

Figure 11 and Figure 12 are plots at the reception shaft located at Intake 5. Figure 11 is the flow history and Figure 12 shows the velocities through the sediment basins corresponding to the values presented in Table 1.

Figure 13 and Figure 14 are HGL plots at the CCPS surge shaft and in the IF respectively.

While results initially showed an increasing reduction in parameters as the IF size was increased, the results from an IF size 800’x1,500’ (see Table 1 above) indicated a reverse in such a trend. Upon further investigation of this finding, it was noted that the system response at IF sizes 800’x1,000’ and 800’x1,500’ were influenced by their filling timing as it related to the initial set of larger transient waves that occur within the first few minutes after a pump trip event. The relationship between this IF filling time and the pressure waves influences the peak parameters listed in Table 1.

Based upon these findings it cannot be said that increasing the IF size from 500’x500’ will yield any different performance results related to the backflow conditions at Intakes 3 and 5.
Figure 9 – HGL (ft) at STA 3150+70 Vent Shaft (located Upstream of the IF)

Figure 10 – HGL (ft) at Staten Island Vent Shaft (located Downstream of the IF)
Figure 11 – Flow (cfs) History at Intake 5 Reception Shaft

Figure 12 – Velocity (fps) History in an Intake 5 Sediment Basin
Figure 13 – HGL (ft) at CCPS Surge Shaft

Figure 14 – HGL (ft) in IF
3.3 Evaluation of System Response with Intake 2 Off-line

During the course of TM 3 investigations, it became of interest to better understand the conditions at the intake screens during a period when Intake 2 was off-line, IF size 500’x500’, and a total pumping capacity of 9,000 cfs was needed. Two main questions were evaluated:

- How much flow can be withdrawn without exceeding the 0.2 fps rule at the intake screens at Intakes 3 and 5? Results are summarized in Table 2 and indicate that a full 9,000 cfs cannot be achieved without exceeding the 0.2 fps rule at the intake screens.

- Next, recognizing that 9,000 cfs could not be achieved, how many more screens would be required (in lineal feet) at both Intakes 3 and 5 to achieve a full 9,000 cfs? Results are summarized in Table 3 and present the additional screen length required to achieve the full 9,000 cfs without exceeding the 0.2 fps rule at the intake screens.

<table>
<thead>
<tr>
<th>Intake</th>
<th>Max allowable flow without exceeding 0.2 fps rule (cfs)</th>
<th>As-Designed Screen Length (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>3,580</td>
<td>1,028</td>
</tr>
<tr>
<td>5</td>
<td>3,590</td>
<td>1,402</td>
</tr>
</tbody>
</table>

Table 2 – Maximum Flow Withdrawal (Intake 2 Off-line)

<table>
<thead>
<tr>
<th>Intake</th>
<th>Max allowable flow without exceeding 0.2 fps rule (cfs)</th>
<th>Additional Screen Length Required to Maintain 4,500 cfs through each intake (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>4,500</td>
<td>265</td>
</tr>
<tr>
<td>5</td>
<td>4,500</td>
<td>359</td>
</tr>
</tbody>
</table>

Table 3 – Required Additional Screen Length (Intake 2 Off-line)

Subsequent to the steady state InfoWorks CS runs made to investigate the questions from an Intake 2 off-line condition, two conditions were evaluated under the transient pump trip event. In the figures, the two line colors represent the following: Blue \(\rightarrow\) as-designed screen lengths and Green \(\rightarrow\) increased screen lengths at Intakes 3 and 5.

Figure 15 and Figure 16 show the HGL on either side of the IF.

Figure 17 and Figure 18 are plots at the reception shaft located at Intake 5. Figure 17 is the flow history and Figure 18 shows the velocities through the sediment basins.
Figure 19 and Figure 20 are HGL plots at the CCPS surge shaft and in the IF respectively.

On these figures, the Blue ➞ as-designed screen lengths results show large oscillations of the HGL in the vicinity of the CCPS preceding the pump trip event. This is because the InfoWorks CS model is attempting to pull 9,000 cfs but pumps are having to cycle on and off because the system can only provide approximately 7,200 cfs without exceeding the 0.2 fps rule at the intake screens. Actual pump cycles and drawdown amounts will likely be different than the pumps simulated in the InfoWorks CS model.
Figure 15 – HGL (ft) at STA 3150+70 Vent Shaft (located **Upstream** of the IF)

Figure 16 – HGL (ft) at Staten Island Vent Shaft (located **Downstream** of the IF)
Figure 17 – Flow (cfs) History at Intake 5 Reception Shaft

Figure 18 – Velocity (fps) History in an Intake 5 Sediment Basin
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

<table>
<thead>
<tr>
<th>Pipe Type</th>
<th>Date</th>
<th>Size (ft)</th>
<th>Material Type</th>
<th>Length</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel Pipe</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Conveyance Pipe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Central Arizona Project</td>
<td>1990s</td>
<td>21</td>
<td>Steel</td>
<td>1.7 miles</td>
<td>Arizona</td>
</tr>
<tr>
<td>Pacific Corp Swt 2 Project</td>
<td>--</td>
<td>11.5-16</td>
<td>Steel plates</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td><strong>Penstock</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ghazi Barotha</td>
<td>--</td>
<td>5 @ 34.75</td>
<td>Steel-lined penstocks</td>
<td>100 meters</td>
<td>--</td>
</tr>
<tr>
<td>Goulley River Penstock</td>
<td>1990s</td>
<td>10-17</td>
<td>Steel</td>
<td>350 ft</td>
<td>West Virginia</td>
</tr>
<tr>
<td><strong>Concrete Cylinder Pipe</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pre-stressed concrete cylinder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Central Arizona Project</td>
<td>1976</td>
<td>21</td>
<td>PCCP fabricated near site</td>
<td>4.5 miles</td>
<td>Arizona</td>
</tr>
<tr>
<td>San Onofre Project</td>
<td>1979</td>
<td>10-18</td>
<td>PCP non-cylinder</td>
<td>3.3 miles</td>
<td>California</td>
</tr>
<tr>
<td>MWD Castaic Project</td>
<td>1971</td>
<td>16.75</td>
<td>PCCP fabricated near site</td>
<td>5.9 miles</td>
<td>California</td>
</tr>
<tr>
<td>USBR Navajo Project</td>
<td>1975</td>
<td>15.75-17.5</td>
<td>PCCP fabricated near site</td>
<td>4.4 miles</td>
<td>New Mexico</td>
</tr>
<tr>
<td>Great man-made River</td>
<td>Phase 1 mid 1980s</td>
<td>2 @ 13</td>
<td>PCCP fabricated near site</td>
<td>490 &amp; 510 miles</td>
<td>Libya</td>
</tr>
<tr>
<td>China South north</td>
<td>--</td>
<td>10</td>
<td>PCCP</td>
<td></td>
<td>China</td>
</tr>
<tr>
<td><strong>Cast-in-Place Concrete Pipe</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arch</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>LACSD Joint Outfall</td>
<td>1973</td>
<td>12</td>
<td>Concrete</td>
<td>3.8 miles</td>
<td>California</td>
</tr>
<tr>
<td>Circular</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tracy pump Plant d/c Lines</td>
<td>1947</td>
<td>15</td>
<td>Concrete</td>
<td>0.4 miles</td>
<td>California</td>
</tr>
<tr>
<td>Cast-in-Place Box Culvert</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roundhill Reservoir</td>
<td>--</td>
<td>2 @ 19.6 x 19.6</td>
<td>Cast-in-place concrete</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Batang Padang Hydro</td>
<td>--</td>
<td>40 ft x 40 ft</td>
<td>Cast-in-place concrete</td>
<td>200-300</td>
<td>--</td>
</tr>
<tr>
<td>KUMPP Proposed</td>
<td></td>
<td>4 &amp; 13’ x 13’</td>
<td>Cast-in-place concrete</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td><strong>Other</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FRP</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Jubail Cooling water bypass</td>
<td>--</td>
<td>13.1</td>
<td>FRP</td>
<td>2 km</td>
<td>--</td>
</tr>
<tr>
<td>Tunnels</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mangla</td>
<td>--</td>
<td>5 @ 26’</td>
<td>Steel grouted annular space</td>
<td>1800 ft each</td>
<td>--</td>
</tr>
<tr>
<td>Shaxi Wanjizsha Yellow River Diversion</td>
<td>--</td>
<td>18.3</td>
<td>--</td>
<td>-- China</td>
<td></td>
</tr>
<tr>
<td>Dokan dam</td>
<td>--</td>
<td>2 @ 36' and 39'</td>
<td>concrete lined tunnels 3'thick</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Bombay outfalls under “sea”</td>
<td>--</td>
<td>11.4</td>
<td>--</td>
<td>7 km</td>
<td>India</td>
</tr>
</tbody>
</table>

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 downstream 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**

<table>
<thead>
<tr>
<th>Option</th>
<th>General Depth-to-Groundwater</th>
</tr>
</thead>
<tbody>
<tr>
<td>ICF-West</td>
<td>Varies from a few feet to 10 feet bgs</td>
</tr>
<tr>
<td>ICF-East and Dual Conveyance</td>
<td>Varies from a few feet to 6 to 8 feet bgs</td>
</tr>
<tr>
<td>Through Delta</td>
<td>Varies from a few feet to 6 to 8 feet bgs</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Table 3: Conveyance Conduit Materials Design Requirements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Criteria</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>Flow Velocity Range, fps</td>
</tr>
<tr>
<td>Pressure, psi</td>
</tr>
<tr>
<td>Depth of Cover, ft soil</td>
</tr>
<tr>
<td>Groundwater</td>
</tr>
<tr>
<td>Design Life, yrs</td>
</tr>
<tr>
<td>Equivalent diameter, ft</td>
</tr>
</tbody>
</table>

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).
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>
<thead>
<tr>
<th>Route Classification</th>
<th>Width (ft)</th>
<th>Length (ft)</th>
<th>Height (ft)</th>
<th>Speed (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multi-lane freeway or express way with 12' lanes, 4' shoulders</td>
<td>17' if 3 or more lanes. 16' if 2 lanes</td>
<td>Unlimited</td>
<td>No maximum</td>
<td>Below 30 mph below posted maximum speed limit</td>
</tr>
<tr>
<td>Substandard freeway or 2 lane road with 12' lanes and 0' to 4' shoulders</td>
<td>15'</td>
<td>Unlimited on controlled access roadways. 135' for others</td>
<td>No maximum</td>
<td>Below 30 mph below posted maximum speed limit</td>
</tr>
<tr>
<td>Two-lane road. 11' lanes. 0 – any shoulder</td>
<td>15'</td>
<td>135</td>
<td>No maximum</td>
<td>Below 30 mph below posted maximum speed limit</td>
</tr>
<tr>
<td>Two-lane road. 10' lanes. 0 – any shoulder</td>
<td>15'</td>
<td>135</td>
<td>No maximum</td>
<td>Below 30 mph below posted maximum speed limit</td>
</tr>
<tr>
<td>Two-lane road. Less than 10' lanes</td>
<td>15'</td>
<td>135</td>
<td>No maximum</td>
<td>Below 30 mph below posted maximum speed limit</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th></th>
<th>Length of Section (ft)</th>
<th>Weight of Section (lbs)</th>
<th>Sections per Truck</th>
<th>Number of Truck Trips per Mile⁽¹⁾</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel (w/CML&amp;C)</td>
<td>13</td>
<td>39,400</td>
<td>1</td>
<td>406</td>
</tr>
<tr>
<td>Steel (no CML&amp;C)</td>
<td>20</td>
<td>41,500</td>
<td>1</td>
<td>264</td>
</tr>
<tr>
<td>RCCP</td>
<td>4</td>
<td>43,500</td>
<td>1</td>
<td>1,320</td>
</tr>
</tbody>
</table>

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  
RCCP = reinforced concrete cylinder pipe  
ft = feet  
yd³ = cubic yard  
lbs = pounds

Table 6: CIP Circular Pipe and CIP Box Culvert Transport

<table>
<thead>
<tr>
<th></th>
<th>Total Volume of Concrete per Mile (ft³)</th>
<th>Number of Trucks per Mile (10 yd³ per truck max)⁽¹⁾</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 CIP Circular Pipes</td>
<td>766,716</td>
<td>2,840</td>
</tr>
<tr>
<td>2 CIP Arches with Common Base Slab</td>
<td>1,571,011</td>
<td>5,819</td>
</tr>
<tr>
<td>2 CIP Arches with Separate Base Slab</td>
<td>1,119,096</td>
<td>4,145</td>
</tr>
<tr>
<td>CIP Box Culvert (2 w/common wall)</td>
<td>925,613</td>
<td>3,428</td>
</tr>
</tbody>
</table>

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  
ft³ = cubic feet  
yd³ = 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 fabricated.
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.
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.
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

<table>
<thead>
<tr>
<th>Inside Diameter after Lining (feet)</th>
<th>Wall Thickness (inches)</th>
<th>Lining Type</th>
<th>Coating type</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>1.0</td>
<td>Field-installed cement mortar</td>
<td>Field-installed coal tar AWWA C203</td>
<td>Impressed current protection system</td>
</tr>
</tbody>
</table>

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.
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.
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

<table>
<thead>
<tr>
<th>ID After Lining (ft)</th>
<th>Pressure Limits AWWA C300 (psi)</th>
<th>Pressure Limits AWWA C302 (psi)</th>
<th>Pressure Limits AWWA C303^[1] (psi)</th>
<th>C300 Maximum Earth Cover Depth (ft)</th>
<th>C302 Maximum Earth Cover Depth (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16</td>
<td>250</td>
<td>55</td>
<td>400</td>
<td>&gt;20</td>
<td>&gt;20</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Option</th>
<th>Wall Thickness (inches)</th>
<th>Base thickness (inches)</th>
<th>Top thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Circular</td>
<td>16</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>Rectangular</td>
<td>Exterior Walls = 20</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>Interior Wall = 18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arch</td>
<td>16</td>
<td>20</td>
<td>--</td>
</tr>
</tbody>
</table>

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.
# APPENDIX E – PIPE MATERIALS

## Table 9-1: Conveyance Line Summaries

<table>
<thead>
<tr>
<th>Material</th>
<th>Shop</th>
<th>Fabrication</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stee (AWWA C200)</td>
<td>Circu</td>
<td>Shop or Field</td>
<td>• Shop fabrication usually results in better weld quality.</td>
<td>• Having pipe results in more opportunities for damage, such as excessive deflections.</td>
</tr>
<tr>
<td>In situ cement mortar lined</td>
<td>ar</td>
<td>Field</td>
<td>• Lighter weight compared to other options.</td>
<td>• Flexible pipe requires more embedment compaction and greater quality control of the embedment.</td>
</tr>
<tr>
<td>Coal tar enamel coated (We ded steel plates are so included in this option evaluation)</td>
<td></td>
<td></td>
<td>• 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.</td>
<td>• Will likely require cement mortar ining once installed and that ining is less dense and more susceptible to variations in thickness compared with factory-installed cement mortar ining.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Two separate pipes are better from a failure standpoint compared to rectangular conduit with a common wall.</td>
<td>• Will require impressed current cathodic protection.</td>
</tr>
<tr>
<td>Concrete Pressure Pipe (AWWA C300 &amp; AWWA C302)</td>
<td>Circu</td>
<td>Shop or Field (field is more likely)</td>
<td>• Does not require impressed current cathodic protection.</td>
<td>• Heavier than steel pipe.</td>
</tr>
<tr>
<td></td>
<td>ar</td>
<td></td>
<td>• Requires less embedment compaction.</td>
<td>• Hand-applied cement mortar at joints can be susceptible to cracking. Requires periodic inspections and repairs.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• 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.</td>
<td>• Few suppliers have indicated they can fabricate concrete pressure pipe greater than 12-foot-diameter.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Two separate pipes are better from a failure standpoint compared to rectangular conduit with a common wall.</td>
<td>• Will likely require field fabrication shop requiring up to 50 acres.</td>
</tr>
<tr>
<td>Cast-in-Place</td>
<td>Circu</td>
<td>In Trench</td>
<td>• Thinner wall thickness compared to other cast-in-place options.</td>
<td>• Bedding preparation under pipe haunches will be more difficult compared to other options.</td>
</tr>
<tr>
<td></td>
<td>ar</td>
<td></td>
<td>• More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options.</td>
<td>• Curved formwork and reinforcement requires more labor, higher installation costs.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Two separate pipes are better from a failure standpoint compared to rectangular conduit with a common wall.</td>
<td>• More difficult to support formwork bracing compared to other cast-in-place options that have a flat slab at the base.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>• 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.</td>
</tr>
</tbody>
</table>

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**Revs on:** 0
## APPENDIX E – PIPE MATERIALS

<table>
<thead>
<tr>
<th>Material</th>
<th>Shape</th>
<th>Fabrication</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast-In-Pace</td>
<td>Rectauar</td>
<td>In Trench</td>
<td>• More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options.</td>
<td>• Requires more time before the trench can be &quot;closed up&quot; compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures.</td>
</tr>
<tr>
<td>Concrete</td>
<td>Arch</td>
<td>In Trench</td>
<td>• More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options.</td>
<td>• Concrete joints more susceptible to leakage.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Formwork is more costly and labor intensive compared with cast-in-place circular conduits.</td>
<td>• Not as favorable from a failure isolation standpoint compared to two separate circular conduits.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Embedment materials and embedment compaction not required.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Bottom slab can be paced and cured first providing surface for formwork building up.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• Utilizes common wa construction, making conveyance system more compact and the narrowest trench width.</td>
<td></td>
</tr>
</tbody>
</table>

| Cast-In-Pace      | Arch   | In Trench  | • More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. | • 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          |        |             | • Formwork is more costly and labor intensive compared with cast-in-place circular conduits. | • Concrete joints more susceptible to leakage.                                                   |
|                   |        |             | • Embedment materials and embedment compaction not required.                | • Arch formwork more labor intensive compared to rectangular formwork.                             |
|                   |        |             | • Bottom slab can be paced 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. |                                                                                                   |
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/ft³
• Unit Weight of Steel = 490 lb/ft³
• 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 yd³

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
Pipe 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.

\[
\text{Floatation} = \text{Weight of Conduit} + \text{Weight of Soil} - \text{Buoyancy}
\]

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.

\[
\text{Safety Factor} = \frac{\text{Weight of Conduit} + \text{Weight of Soil}}{\text{Buoyancy}}
\]

Table 1 presents the assumptions used in the floatation calculations.

<table>
<thead>
<tr>
<th>Description</th>
<th>Units</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight concrete (W_{\text{Conc}})</td>
<td>lb/cf</td>
<td>150</td>
</tr>
<tr>
<td>Unit weight water (W_{\text{H2O}})</td>
<td>lb/cf</td>
<td>62.4</td>
</tr>
<tr>
<td>Unit weight non-saturated soil (W_{\text{Soil}})</td>
<td>lb/cf</td>
<td>110</td>
</tr>
<tr>
<td>Unit weight steel (W_{\text{Steel}})</td>
<td>lb/cf</td>
<td>490</td>
</tr>
<tr>
<td>Minimum Safety Factor</td>
<td>N/A</td>
<td>1.1</td>
</tr>
</tbody>
</table>

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.

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.

\[
\text{Weight of Soil (lb/ft)} = \text{OD} \times \text{D} \times W_{\text{Soil}}
\]

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.

\[
\text{Weight of Pipe (lb/ft)} = \left(\frac{\text{OD}}{2}\right)^2 \pi - \left(\frac{\text{ID}}{2}\right)^2 \pi \times W_{\text{Conc}}
\]
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.

\[
Buoyancy (\text{lb/ft}) = ((\text{OD}/2)^2\pi + \text{OD} \times \text{Cover Depth}) \times W_{\text{H}_{2}\text{O}}
\]

### 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)**

<table>
<thead>
<tr>
<th>Depth of Cover (ft)</th>
<th>Pipe Weight lb/ft (x1000)</th>
<th>Soil Weight lb/ft (x1000)</th>
<th>Buoyancy lb/ft (x1000)</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>10.9</td>
<td>0.0</td>
<td>17.1</td>
<td>0.6</td>
</tr>
<tr>
<td>2</td>
<td>10.9</td>
<td>4.1</td>
<td>19.4</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>10.9</td>
<td>8.2</td>
<td>21.7</td>
<td>0.9</td>
</tr>
<tr>
<td>6</td>
<td>10.9</td>
<td>12.3</td>
<td>24.1</td>
<td>1.0</td>
</tr>
<tr>
<td>8</td>
<td>10.9</td>
<td>16.4</td>
<td>26.4</td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>10.9</td>
<td>20.5</td>
<td>28.7</td>
<td>1.1</td>
</tr>
<tr>
<td>12</td>
<td>10.9</td>
<td>24.6</td>
<td>31.1</td>
<td>1.1</td>
</tr>
<tr>
<td>14</td>
<td>10.9</td>
<td>28.7</td>
<td>33.4</td>
<td>1.2</td>
</tr>
</tbody>
</table>

\( \text{ft} = \text{foot} \)  
\( \text{lb/ft} = \text{pound per foot} \)

**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.

![Pipeline Floatation Diagram](image)

**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.

\[
\text{Weight of Pipe (lb/ft)} = (OD_{\text{Coating}}/2)^2 \pi - (OD_{\text{Steel}}/2)^2 \pi \cdot W_{\text{Concrete}} + (OD_{\text{Steel}}/2)^2 \pi - (ID_{\text{Steel}}/2)^2 \pi \cdot W_{\text{Steel}} + (ID_{\text{Concrete}}/2)^2 \pi - (ID_{\text{Lining}}/2)^2 \pi \cdot W_{\text{Concrete}}
\]

### 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)**

<table>
<thead>
<tr>
<th>Depth of Cover (ft)</th>
<th>Pipe Weight lb/ft (x1000)</th>
<th>Soil Weight lb/ft (x1000)</th>
<th>Buoyancy lb/ft (x1000)</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.4</td>
<td>0.0</td>
<td>12.8</td>
<td>0.2</td>
</tr>
<tr>
<td>2</td>
<td>2.4</td>
<td>3.6</td>
<td>14.8</td>
<td>0.4</td>
</tr>
<tr>
<td>4</td>
<td>2.4</td>
<td>7.1</td>
<td>16.8</td>
<td>0.6</td>
</tr>
<tr>
<td>6</td>
<td>2.4</td>
<td>10.7</td>
<td>18.9</td>
<td>0.7</td>
</tr>
<tr>
<td>8</td>
<td>2.4</td>
<td>14.2</td>
<td>20.9</td>
<td>0.8</td>
</tr>
<tr>
<td>10</td>
<td>2.4</td>
<td>17.8</td>
<td>22.9</td>
<td>0.9</td>
</tr>
<tr>
<td>12</td>
<td>2.4</td>
<td>21.3</td>
<td>24.9</td>
<td>1.0</td>
</tr>
<tr>
<td>14</td>
<td>2.4</td>
<td>24.9</td>
<td>26.9</td>
<td>1.0</td>
</tr>
<tr>
<td>16</td>
<td>2.4</td>
<td>28.5</td>
<td>28.9</td>
<td>1.1</td>
</tr>
<tr>
<td>18</td>
<td>2.4</td>
<td>32.0</td>
<td>31.0</td>
<td>1.1</td>
</tr>
<tr>
<td>20</td>
<td>2.4</td>
<td>35.6</td>
<td>33.0</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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 flotation 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 flotation 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)

<table>
<thead>
<tr>
<th>Diameter to Thickness (D/t) ratio</th>
<th>Steel thickness, inch</th>
</tr>
</thead>
<tbody>
<tr>
<td>160</td>
<td>1.200</td>
</tr>
<tr>
<td>200</td>
<td>0.960</td>
</tr>
<tr>
<td>240</td>
<td>0.800</td>
</tr>
</tbody>
</table>
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.

![Pipeline Floatation Chart](image)

**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**

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of arches</td>
<td>2</td>
</tr>
<tr>
<td>Arch height, feet</td>
<td>16</td>
</tr>
<tr>
<td>Arch width, feet</td>
<td>16</td>
</tr>
<tr>
<td>Arch thickness, inches</td>
<td>18</td>
</tr>
<tr>
<td>Distance between arches, feet</td>
<td>18</td>
</tr>
<tr>
<td>Distance from arch to edge of slab, feet</td>
<td>5</td>
</tr>
<tr>
<td>Slab width, feet</td>
<td>66</td>
</tr>
<tr>
<td>Slab thickness, inches</td>
<td>20</td>
</tr>
</tbody>
</table>
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.

\[
\text{Weight of Soil (lb/ft)} = w_S \times D \times W_{\text{Soil}}
\]

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.

\[
\text{Weight of Conduit (lb/ft)} = \left(2 \times \frac{1}{2} (\frac{OD}{2})^2 \pi \cdot (\frac{ID}{2})^2 \pi + w_S \times t_S \right) \times W_{\text{Conc}}
\]

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.

\[
\text{Buoyancy (lb/ft)} = \left(2 \times \frac{1}{2} (\frac{OD}{2})^2 \pi + w_S \times t_S + \text{Cover Depth} \times OD \right) \times W_{\text{H2O}}
\]

### 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

<table>
<thead>
<tr>
<th>Depth of Cover (ft)</th>
<th>Conduit Weight lb/ft (x1000)</th>
<th>Soil Weight lb/ft (x1000)</th>
<th>Buoyancy lb/ft (x1000)</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>28.9</td>
<td>0.0</td>
<td>24.6</td>
<td>1.2</td>
</tr>
<tr>
<td>2</td>
<td>28.9</td>
<td>14.5</td>
<td>32.8</td>
<td>1.3</td>
</tr>
<tr>
<td>4</td>
<td>28.9</td>
<td>29.0</td>
<td>41.0</td>
<td>1.4</td>
</tr>
<tr>
<td>6</td>
<td>28.9</td>
<td>43.6</td>
<td>49.3</td>
<td>1.5</td>
</tr>
<tr>
<td>8</td>
<td>28.9</td>
<td>58.1</td>
<td>57.5</td>
<td>1.5</td>
</tr>
<tr>
<td>10</td>
<td>28.9</td>
<td>72.6</td>
<td>65.7</td>
<td>1.5</td>
</tr>
<tr>
<td>12</td>
<td>28.9</td>
<td>87.1</td>
<td>74.0</td>
<td>1.6</td>
</tr>
<tr>
<td>14</td>
<td>28.9</td>
<td>101.6</td>
<td>82.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

ft = foot
lb/ft = pound per foot

ARCHES FLOATATION
Safety Factor v Cover Depth

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

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of boxes</td>
<td>2</td>
</tr>
<tr>
<td>Box height, feet</td>
<td>20</td>
</tr>
<tr>
<td>Box width, feet</td>
<td>10</td>
</tr>
<tr>
<td>Wall thickness, inch</td>
<td>20</td>
</tr>
<tr>
<td>Top slab thickness, inch</td>
<td>18</td>
</tr>
<tr>
<td>Bottom slab thickness, inch</td>
<td>20</td>
</tr>
<tr>
<td>Total conduit width, feet</td>
<td>25</td>
</tr>
<tr>
<td>Total conduit height, feet</td>
<td>23</td>
</tr>
</tbody>
</table>

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/ft³ specified in Table 1, resulting in lb/ft of soil.

\[ Weight \ of \ Soil \ (lb/ft) = wc \times D \times W_{Soil} \]
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.

\[ \text{Weight of Conduit (lb/ft) = (wc x hc – 2 x (bh x bw)) x WConc} \]

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.

\[ \text{Buoyancy (lb/ft) = (wc x hc + Cover Depth x wc) x WH20} \]

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**

<table>
<thead>
<tr>
<th>Depth of Cover (ft)</th>
<th>Conduit Weight lb/ft (x1000)</th>
<th>Soil Weight lb/ft (x1000)</th>
<th>Buoyancy lb/ft (x1000)</th>
<th>Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>26.3</td>
<td>0.0</td>
<td>35.9</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>26.3</td>
<td>5.5</td>
<td>39.0</td>
<td>0.8</td>
</tr>
<tr>
<td>4</td>
<td>26.3</td>
<td>10.9</td>
<td>42.1</td>
<td>0.9</td>
</tr>
<tr>
<td>6</td>
<td>26.3</td>
<td>16.4</td>
<td>45.2</td>
<td>0.9</td>
</tr>
<tr>
<td>8</td>
<td>26.3</td>
<td>21.9</td>
<td>48.3</td>
<td>1.0</td>
</tr>
<tr>
<td>10</td>
<td>26.3</td>
<td>27.3</td>
<td>51.4</td>
<td>1.0</td>
</tr>
<tr>
<td>12</td>
<td>26.3</td>
<td>32.8</td>
<td>54.5</td>
<td>1.1</td>
</tr>
<tr>
<td>14</td>
<td>26.3</td>
<td>38.2</td>
<td>57.6</td>
<td>1.1</td>
</tr>
<tr>
<td>16</td>
<td>26.3</td>
<td>43.7</td>
<td>60.7</td>
<td>1.2</td>
</tr>
<tr>
<td>18</td>
<td>26.3</td>
<td>49.2</td>
<td>63.8</td>
<td>1.2</td>
</tr>
</tbody>
</table>

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 flotation 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 flotation sensitivity analysis resulted in the following conclusions.

1. Concrete thickness variations affect flotation 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>
<thead>
<tr>
<th>Depth of Cover (feet)</th>
<th>Required Thickness to Prevent Floatation (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>32</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
</tr>
<tr>
<td>10</td>
<td>16</td>
</tr>
</tbody>
</table>
Table 10: Required Depth of Cover

<table>
<thead>
<tr>
<th>Conduit</th>
<th>Minimum Depth of Cover Required (feet)</th>
<th>Farming Depth (feet)</th>
<th>Total Depth of Cover (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Pipe</td>
<td>10</td>
<td>6</td>
<td>16</td>
</tr>
<tr>
<td>Steel Pipe</td>
<td>16</td>
<td>6</td>
<td>22</td>
</tr>
<tr>
<td>Concrete Arches</td>
<td>0</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>Concrete Boxes</td>
<td>12</td>
<td>6</td>
<td>18</td>
</tr>
</tbody>
</table>

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
Appendix G

Summary of Tunneling Contractor Workshops
<table>
<thead>
<tr>
<th>Key Project Issues &amp; Discussion</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Contractor</strong></td>
</tr>
<tr>
<td><strong>Tunnel and Shaft Protection</strong></td>
</tr>
<tr>
<td><strong>Tunnel drive length per reach and access to TBM along tunnel</strong></td>
</tr>
<tr>
<td><strong>Owner-furnished tunnel liner segment for the contract/ Shaft construction</strong></td>
</tr>
<tr>
<td><strong>Contract delivery alternations</strong></td>
</tr>
<tr>
<td><strong>Contractor/ Shaft construction</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>A</th>
<th>Prefer to have two contractors with each one building one leg of the tunnel.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Like the idea of one contractor sinking 4 shafts and turning it over.</td>
</tr>
<tr>
<td></td>
<td>Prefer two contracts as one contract will get less competition.</td>
</tr>
<tr>
<td></td>
<td>Prefer for owner to procure bargeing contract.</td>
</tr>
<tr>
<td></td>
<td>Consider intervention zones at one mile intervals as overall, they are fine with two miles.</td>
</tr>
<tr>
<td></td>
<td>A mile driven for EBF TBM is acceptable. Use of the mid-drive access shaft location for major maintenance on the cutter head of shield brushes is a great feature.</td>
</tr>
<tr>
<td></td>
<td>Building a pad is the best option, as opposed to erecting barrier walls.</td>
</tr>
<tr>
<td></td>
<td>Suggested owner should pro-construct flood protection.</td>
</tr>
<tr>
<td></td>
<td>Suggested doing a high/meallower risk analysis for equipment to be protected.</td>
</tr>
<tr>
<td></td>
<td>Mound with ramp is a clever idea that cuts down on how many acres you need (about 3 acres for surface area).</td>
</tr>
<tr>
<td></td>
<td>Building a pad is the best option, as opposed to erecting barrier walls.</td>
</tr>
<tr>
<td></td>
<td>Suggested owner should pro-construct flood protection.</td>
</tr>
<tr>
<td></td>
<td>Don’t need to have owner-furnished tunnel segments.</td>
</tr>
<tr>
<td></td>
<td>Suggested segments be designed a bit thicker (add 2 to 3 in) than actually needed by strict calculations so that all of the rebars and other tab/fib materials and concrete consolidations could be done correctly.</td>
</tr>
<tr>
<td></td>
<td>Agree that segments would probably be barge.</td>
</tr>
<tr>
<td></td>
<td>Don’t see an issue in equipment (TBM) availability.</td>
</tr>
<tr>
<td></td>
<td>The market is available and so are laborers.</td>
</tr>
<tr>
<td></td>
<td>Considers a 6 month delay as adequate.</td>
</tr>
<tr>
<td></td>
<td>Sureties will be the actual entity that “prequalifies” firms and it’s to participate in the work, since they won’t bond firms or it’s that they don’t have confidence in.</td>
</tr>
<tr>
<td></td>
<td>Prefer design bid build.</td>
</tr>
<tr>
<td></td>
<td>This project isn’t schematically complicated so other alternatives are not as beneficial.</td>
</tr>
<tr>
<td></td>
<td>Prefer contractor to provide foundation of their own design.</td>
</tr>
<tr>
<td></td>
<td>This is most likely an intervention project.</td>
</tr>
<tr>
<td></td>
<td>Use of the mid-allowance for the shafts.</td>
</tr>
<tr>
<td></td>
<td>Considers a 6 month delay as adequate.</td>
</tr>
<tr>
<td></td>
<td>Don’t see an issue in equipment (TBM) availability.</td>
</tr>
<tr>
<td></td>
<td>The market is available and so are laborers.</td>
</tr>
<tr>
<td></td>
<td>Considers a 6 month delay as adequate.</td>
</tr>
<tr>
<td></td>
<td>Sureties will be the actual entity that “prequalifies” firms and it’s to participate in the work, since they won’t bond firms or it’s that they don’t have confidence in.</td>
</tr>
<tr>
<td></td>
<td>Prefer design bid build.</td>
</tr>
<tr>
<td></td>
<td>This project isn’t schematically complicated so other alternatives are not as beneficial.</td>
</tr>
<tr>
<td></td>
<td>Prefer contractor to provide foundation of their own design.</td>
</tr>
<tr>
<td></td>
<td>This is most likely an intervention project.</td>
</tr>
<tr>
<td></td>
<td>Use of the mid-allowance for the shafts.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B</th>
<th>Prefer a single contract.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A $2.5B contract is not too big to handle.</td>
</tr>
<tr>
<td></td>
<td>Did not like the idea of two contractors building one leg of the tunnel.</td>
</tr>
<tr>
<td></td>
<td>If there are shared contracts contractor is concerned about managing flooding issues.</td>
</tr>
<tr>
<td></td>
<td>If owner is considering two contracts then contractor strongly suggests setting up demarcation lines between the contracts.</td>
</tr>
<tr>
<td></td>
<td>Concerned about pressure issues between mile markers 2 and 4.</td>
</tr>
<tr>
<td></td>
<td>Recommended intervention zones should be located at one mile intervals.</td>
</tr>
<tr>
<td></td>
<td>Contractor would like to impact their cutter head every 1/2 mile.</td>
</tr>
<tr>
<td></td>
<td>Changing a few cutters under pressure is very risky. That is why owner needs to prescribe measures to make sure other contractors don’t cut corners, causing high pressure interventions.</td>
</tr>
<tr>
<td></td>
<td>Tunnel drive length of 8 miles is adequate.</td>
</tr>
<tr>
<td></td>
<td>Suggested another option (outside of putting a ski pad) would be to bring up a wall.</td>
</tr>
<tr>
<td></td>
<td>Recommended owner elevate pad ahead of time for a different contract.</td>
</tr>
<tr>
<td></td>
<td>Recommended owner to get a dirt-guy and have protection ready by contractor’s arrival (5).</td>
</tr>
<tr>
<td></td>
<td>Contractor would bid on a segment contract.</td>
</tr>
<tr>
<td></td>
<td>Don’t want to have design liability.</td>
</tr>
<tr>
<td></td>
<td>No preference to hosting owner-furnished segments or having contractor provide segments.</td>
</tr>
<tr>
<td></td>
<td>Identified a significant difference in wage rate between having the segments provided by a precast yard as a manufactured product vs setting up a plant just for this project; precast yard wages could be as low as $20 to $30/hr up to $50/hr for contractor wages.</td>
</tr>
<tr>
<td></td>
<td>Proving the segments may be a good opportunity to use an alliances contracting approach.</td>
</tr>
<tr>
<td></td>
<td>Suggested establishing a prequalification process for evaluating and selecting joint venture teams. Pay close attention to the prequalification process so that there are enough prequalified contractors there.</td>
</tr>
<tr>
<td></td>
<td>See no issue in operator and equipment availability.</td>
</tr>
<tr>
<td></td>
<td>6 month turn around on tunnel advertisement/bid may be a bit tight if you have to do a new proposal process for every single tunnel bid contract.</td>
</tr>
<tr>
<td></td>
<td>Prequal 3/4 teams, not individual contractors.</td>
</tr>
<tr>
<td></td>
<td>Did not see a benefit in using design-build project delivery on the tunnels.</td>
</tr>
<tr>
<td></td>
<td>Dewatering could be an issue as it would require a lot of power.</td>
</tr>
<tr>
<td></td>
<td>See no problem in IMDD bringing the primary power to the site and contractor in control of distributing.</td>
</tr>
<tr>
<td></td>
<td>To drive transport and vent shafts, contractors will need about 30MW of power at the job site. This needs to be provided in the form of a temporary generator, or have pole-line power brought in.</td>
</tr>
<tr>
<td></td>
<td>The current CEQA restriction on generator power at 300KW will not work.</td>
</tr>
<tr>
<td></td>
<td>Prefer CDP, they would have the smaller contractors sign up under the joint venture CDP.</td>
</tr>
<tr>
<td></td>
<td>Owner needs to make sure that the specification is clear.</td>
</tr>
<tr>
<td></td>
<td>Risk sharing should be clearly spelled out. Include partnering, dispute resolution, provide incentives to resolve problems.</td>
</tr>
<tr>
<td></td>
<td>CDP have more negatives but not really opposed to the idea; they’ve seen more problems with PLAs.</td>
</tr>
<tr>
<td></td>
<td>Prefer owner to procure the equipment.</td>
</tr>
<tr>
<td></td>
<td>See no issue in operator and equipment availability.</td>
</tr>
<tr>
<td></td>
<td>tunnels at site.</td>
</tr>
<tr>
<td></td>
<td>Recommend setting up shafts at north drive connecting tunnel to second shaft at south then allowing second contractor to go north from south shaft.</td>
</tr>
<tr>
<td></td>
<td>Would perform intervgas between every 1,000 ft to check their tools.</td>
</tr>
<tr>
<td></td>
<td>79 gas at 317 is required by CalOSHA for intervention.</td>
</tr>
<tr>
<td></td>
<td>Would like to see safe working performance space (i.e. toughness of cutters).</td>
</tr>
<tr>
<td></td>
<td>Owner will not pay for shaft but will provide access to the land. 40 foot per day is reasonable.</td>
</tr>
<tr>
<td></td>
<td>Prefer build-up of pad prior to contractor coming in.</td>
</tr>
<tr>
<td></td>
<td>Would need 7-10 acres per site for the pad if the contract required 2 contractors.</td>
</tr>
<tr>
<td></td>
<td>Owner’s design eng has set up a basic design for the shafts and ID about what can definitely NOT be done. Let contractors take it from there.</td>
</tr>
<tr>
<td></td>
<td>Suggested oval shape for the shaft 120 ft is max depth for slurry wall technology.</td>
</tr>
<tr>
<td></td>
<td>Would not have a problem with owner providing segment. Contractor didn’t seem in favor of taking the liability for segment design.</td>
</tr>
<tr>
<td></td>
<td>Owner needs to be prescriptive about segment design.</td>
</tr>
<tr>
<td></td>
<td>A 6 month turn around on tunnel bids is acceptable only if everything is already designed and contract specs from one bid to the next so that contractors can quickly prepare bids.</td>
</tr>
<tr>
<td></td>
<td>Schedule shown during workshop is NOT defendable if Design/Build is used. DB will take more time to execute than DDB.</td>
</tr>
<tr>
<td></td>
<td>Prefer design bid build (DBB).</td>
</tr>
<tr>
<td></td>
<td>Owner should conform (standardize) all the contract specs from one bid to the next so that contractors can quickly prepare bids.</td>
</tr>
<tr>
<td></td>
<td>Schedule shown during workshop is NOT defendable if Design/Build is used. DB will take more time to execute than DDB.</td>
</tr>
<tr>
<td></td>
<td>4.38MVA for the 20-hoover shell is low.</td>
</tr>
<tr>
<td></td>
<td>Would prefer to have 600amp on site initially.</td>
</tr>
<tr>
<td></td>
<td>Willing to provide information on power for ground freezing.</td>
</tr>
<tr>
<td></td>
<td>35% bonding is acceptable.</td>
</tr>
<tr>
<td></td>
<td>Contractor believes 50% of contract bonding makes sense.</td>
</tr>
<tr>
<td></td>
<td>Contractor is flexible on either OCP or CCP.</td>
</tr>
<tr>
<td></td>
<td>Equipment should be insured on builder insurance.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>C</th>
<th>Prefer separate contracts (smaller than 2.5 Billion).</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Prefer for one contractor to sink 4 shafts.</td>
</tr>
<tr>
<td></td>
<td>Concerned about issues in getting the mud up a vertical belt due to wetness (slurry) might need to turn to slurry and pump it out OR expending out water.</td>
</tr>
<tr>
<td></td>
<td>Recommend setting up shafts at north drive connecting tunnel to second shaft at south then allowing second contractor to go north from south shaft.</td>
</tr>
<tr>
<td></td>
<td>Performance specs (i.e. toughness of cutters).</td>
</tr>
<tr>
<td></td>
<td>Owner will not pay for shaft but will provide access to the land. 40 foot per day is reasonable.</td>
</tr>
<tr>
<td></td>
<td>Would perform intervgas between every 1,000 ft to check their tools.</td>
</tr>
<tr>
<td></td>
<td>Would need 7-10 acres per site for the pad if the contract required 2 contractors.</td>
</tr>
<tr>
<td></td>
<td>Owner’s design eng has set up a basic design for the shafts and ID about what can definitely NOT be done. Let contractors take it from there.</td>
</tr>
<tr>
<td></td>
<td>Suggested oval shape for the shaft 120 ft is max depth for slurry wall technology.</td>
</tr>
<tr>
<td></td>
<td>Would not have a problem with owner providing segment. Contractor didn’t seem in favor of taking the liability for segment design.</td>
</tr>
<tr>
<td></td>
<td>Owner needs to be prescriptive about segment design.</td>
</tr>
<tr>
<td></td>
<td>A 6 month turn around on tunnel bids is acceptable only if everything is already designed and contract specs from one bid to the next so that contractors can quickly prepare bids.</td>
</tr>
<tr>
<td></td>
<td>Schedule shown during workshop is NOT defendable if Design/Build is used. DB will take more time to execute than DDB.</td>
</tr>
<tr>
<td></td>
<td>Prefer design bid build (DBB).</td>
</tr>
<tr>
<td></td>
<td>Owner should conform (standardize) all the contract specs from one bid to the next so that contractors can quickly prepare bids.</td>
</tr>
<tr>
<td></td>
<td>Schedule shown during workshop is NOT defendable if Design/Build is used. DB will take more time to execute than DDB.</td>
</tr>
<tr>
<td></td>
<td>4.38MVA for the 20-hoover shell is low.</td>
</tr>
<tr>
<td></td>
<td>Would prefer to have 600amp on site initially.</td>
</tr>
<tr>
<td></td>
<td>Willing to provide information on power for ground freezing.</td>
</tr>
<tr>
<td></td>
<td>35% bonding is acceptable.</td>
</tr>
<tr>
<td></td>
<td>Contractor believes 50% of contract bonding makes sense.</td>
</tr>
<tr>
<td></td>
<td>Contractor is flexible on either OCP or CCP.</td>
</tr>
<tr>
<td></td>
<td>Equipment should be insured on builder insurance.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>D</th>
<th>Clay and sand will mix at the cutter head making it perfect material for EBF.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Suggested ground freezing, and talking to companies in the ground freezing industry.</td>
</tr>
<tr>
<td></td>
<td>Don’t see advantages in PLAs.</td>
</tr>
<tr>
<td></td>
<td>They already cover a lot of PLA in union agreement, so there is no added benefit with PLAs.</td>
</tr>
<tr>
<td></td>
<td>Would prefer for work sites to be &quot;ready to go&quot; when they start construction.</td>
</tr>
<tr>
<td></td>
<td>Would start work on shafts right away.</td>
</tr>
<tr>
<td>Contractor</td>
<td>Tunnel-drive length per reach and access to TBM along tunnel drive</td>
</tr>
<tr>
<td>------------</td>
<td>------------------------------------------------------------------</td>
</tr>
<tr>
<td>隧道</td>
<td>隧道和竖井保护</td>
</tr>
</tbody>
</table>

### Key Project Issues & Discussion

1. **Tunnel and Shaft Protection**
   - 每米开挖的安全是一个问题；考虑到安全在每个阶段的复杂性。
   - 推荐使用安全措施来设计隧道和衬砌层的施工。
   - 业主应考虑在隧道的建设中，确保施工方法和安全措施。

2. **Owner-furnished tunnel liner segment for the contractor/Shaft availability**
   - 合同商应考虑使用业主提供的衬砌段。
   - 模拟应考虑使用业主提供的衬砌段。
   - 业主应考虑在隧道的建设中，确保施工方法和安全措施。

3. **Contractor, operators, and equipment (TBM) availability**
   - 合同商应考虑使用业主提供的衬砌段。
   - 模拟应考虑使用业主提供的衬砌段。
   - 业主应考虑在隧道的建设中，确保施工方法和安全措施。
<table>
<thead>
<tr>
<th>Contractor</th>
<th>Tunnel drive length per reach and access to TBM along tunnel headings</th>
<th>Turned and Shaft Provision</th>
<th>Owner-furnished tunnel/liner segment for the contractor/Shaft construction in TBM availability</th>
<th>Contractor, operators, and equipment</th>
<th>Contract delivery alternatives</th>
<th>Power requirements for TBM and site operations</th>
<th>CDIP vs OCP/Banding</th>
<th>PLA's EBR, and Other Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boudin Island - Construction of four 40-foot TD tunnel headings</td>
<td>Need early contracts for site prep</td>
<td>They can set up on-site production. They have experience with this. If the site allows it and the cost is reasonable then it’s doable. They prefer whatever option is cheapest but still gives good quality.</td>
<td>Flexibility is necessary if there is onsite production.</td>
<td>There is to be a balance in risk-sharing. They’ve shoved away from jobs that want contractor to take all the risk.</td>
<td>Owner needs to get in touch with bonding companies ahead of RFP and RFI. They recommend talking to insurance companies.</td>
<td>They are neutral on which insurance to get, but owner probably won’t see savings from OCP.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>They think owner’s biggest challenge will be the bottom play</td>
<td>They think freezing makes sense but a lot of power is needed for such a long duration</td>
<td>There is no arrangement for maximum number of interventions.</td>
<td>Owner is going to need a lot coordination with the local union</td>
<td>There is to be a balance in risk-sharing. They’ve shoved away from jobs that want contractor to take all the risk.</td>
<td>They have the idea of a draft ODR/EBR and knowing how to advance.</td>
<td></td>
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<tr>
<td></td>
<td>If contractor via shafts, they see trouble in setting up and disassembling TBM</td>
<td></td>
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<th>PLA's EBR, and Other Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>They prefer having one contractor ask all 3 shafts to work together. A good separation between the NOS contractors would be 500-1,000 ft.</td>
<td>They think 80/90% of the work is about supplying the segments.</td>
<td>Owners are OK with owner supplying the segments.</td>
<td>Owners should streamline contracts, have similar language.</td>
<td>Owners need to be concerned with the package of the project and how it will be structured.</td>
<td>Owners need to be concerned with the package of the project and how it will be structured.</td>
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<tr>
<td></td>
<td>They like the idea of having a separate contractor for each shaft. It’s an opportunity for more small business involvement</td>
<td>They think 3-5 acres for the pad is about right</td>
<td>They would focus on protecting their electrical gear</td>
<td></td>
<td></td>
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</tbody>
</table>

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<th>PLA's EBR, and Other Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>It’s very ambitious to think two contractors will be able to work together.</td>
<td>They think safe havens every 2 miles is pushing it. There will be 12-15 interventions before you get to two miles.</td>
<td>They like the idea of early contracts for site prep work.</td>
<td>Owner should build the shafts. Contractor doesn’t want that responsibility. The price contract to owner would not be that much different. There is no drawback with respect to insurance on TBM design. They prefer costing on site.</td>
<td>Equipment shouldn’t be an issue.</td>
<td></td>
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</tbody>
</table>

APPENDIX G - Summary of Tunneling Contractor Comments
<table>
<thead>
<tr>
<th>Contractor</th>
<th>Tunnel drive length per reach and access to TBM along tunnel drive</th>
<th>Tunnel and Shaft Protection</th>
<th>Owner-furnished tunnel/liner segment for the contractor/Shaft construction</th>
<th>Key Project Issues &amp; Discussion</th>
<th>Contract delivery alternatives</th>
<th>Power requirements for TBM and site operations</th>
<th>CIP vs OCP / Bonding</th>
<th>PLA, DPR, and Other Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>They like the idea of a subcontractor handling the muck disposal. The muck contractor will have a lot of downtime though. They think owner-managed barge landing will be problematic for owner. They like the idea of having separate barges landing for each contractor. They are not very concerned about access. They feel they can work from inside the tunnel.</td>
<td>They like the idea of having early contracts for site prep work. They like the idea of having early contracts for site prep work. They like the idea of having early contracts for site prep work. They like the idea of having early contracts for site prep work. They like the idea of having early contracts for site prep work. They like the idea of having early contracts for site prep work.</td>
<td>Risk is higher with owner-provided segments because any issues would end up affecting all contractors. Contractors should be given the opportunity to handle this. It's a lot of lead for a single source manufacturer. Contractor accustomed to supplying their own segments. They don't like the idea of building a segment plant on-site. Material would still have to be imported either way.</td>
<td>They think 18 months is tight. They recommend 24 months instead. You need more cushion time especially for the starter tunnel. 6-month stagger between contracts is OK. You need standardization though. Main power availability will be challenge but they can train people</td>
<td>DB would take longer than DB. CM/OC not appropriate for this type of project. DB is the best approach.</td>
<td>If owner brings power to the site, the contractor should take care of the rest. About 1MW is needed to sink one shaft at a time; early work has to be done before contractor arrives.</td>
<td>They'd rather manage their own losses so they prefer CIP. Owner should take care of bond for clarity. Owner should contact bonding brokers for more information.</td>
<td>The high-speed rail had a symposium that was very helpful. There was open discussion. The contractor would like to see that type of format in the future. They also liked the one-on-one sessions with contractors.</td>
</tr>
</tbody>
</table>
Appendix H
Hydraulic Analysis Technical Memorandum
Technical Memorandum No. 1

To

Jay Arabshahi, MWD of Southern California
Paul Sotsaikich, MWD of Southern California

Subject

DHCCP Bay Delta Conveyance System
Hydraulic Operation Equalization Study

From

Keith Campbell, AECOM
Ryan Edison, AECOM

Date

August 22, 2014

Project Number

60322033

1. Background

Metropolitan Water District of Southern California (MWD) retained AECOM to perform hydraulic modeling of an alternative DHCCP Bay Delta Conveyance System configuration that utilizes gravity flow between the river intakes and a pump station located at the Clifton Court Forebay.

Figure 1 is a schematic showing the general arrangement of the major conveyance system as considered in this study. Figure 1 defines the existing as-proposed general arrangement referenced throughout this Technical Memorandum (TM). The existing as-proposed general arrangement was established from DWR drawings (Ref. 1), modified by MWD, and the Combined Pumping Plant Option Technical Memorandum for the Clifton Court Pump Station (CCPS) (Ref. 2).

At the upstream end of Figure 1, there are the three intakes on the Sacramento River. Not shown in Figure 1 at each intake there is a bank of screens followed by three sedimentation basins at each intake site. Each sedimentation basin has four roller gates resulting in a total of 12 roller gates per intake site. Following these roller gates, there is a single reception shaft where flows drop into the individual portions of the Upper Tunnels. Intakes 2 and 3 join together at the Junction Structure (JS). At the downstream end of the Upper Tunnels flow rises to a surface water body referred to as the Intermediate Forebay (IF). Flows then drop back down into the Lower Tunnels upon exiting the IF. The Lower Tunnels are twin 40-foot-diameter tunnels that convey flows to the CCPS.

1.1 Study Objective

The primary objective of this study is to determine the conveyance system hydraulic performance for certain critical hydraulic conditions and model run scenarios identified jointly by AECOM and MWD. These results from this modeling effort will be considered by MWD in its overall evaluation of the alternative conveyance configuration and efficacy.
2. Technical Approach

2.1 General Overview

In order to investigate the hydraulic performance of the existing as-proposed conveyance system and various derivatives, a numerical model capable of dynamically simulating various operational conditions is required. The ability to model both free surface and pressurized flows is also required. Given these general requirements, the InfoWorks CS model was selected. InfoWorks CS is a well-established and widely used numerical model which dynamically solves the one-dimensional St. Venant equations of fluid flow. Documentation of the InfoWorks CS model can be found at www.innovyze.com.

The first step in this study involved the construction of the conveyance system within the InfoWorks CS modeling platform. The following subsections of this TM document some of the key conveyance features and also some of the general modeling assumptions.

2.2 Loss Coefficients

2.2.1 Friction Loss

A Manning roughness value of 0.0145 had been established during previous hydraulic investigations by MWD and others. This value corresponds to a C value of 125, which seems adequately conservative for the purposes of planning.

2.2.2 Minor Losses

These losses account for the transitions between conveyance features (ex., entrance/exit losses) and for specific hydraulic features such as screens and gates. An effort was made to use appropriate...
values, but it is acknowledged that certain structures (e.g., JS) require a more detailed assessment using additional methods such as Computation Fluid Dynamics (CFD) in order to establish the correct headloss across the structure.

Combined minor loss values (i.e., KL values) are noted for each element on the InfoWorks Modeling Schematic attached to this TM.

2.3 Intake Screens

Each of the three intake locations have a bank of intake screens as detailed by DWR drawings (Ref. 1). In this study, the capacity of each intake’s screens is a function of river level, screen invert, and the number and size of the screen. Table 1 summarizes the key parameters for each of the three intakes as taken from the DWR drawings (Ref. 1).

Table 1 – Intake General Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Intake 2</th>
<th>Intake 3</th>
<th>Intake 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Screen Size (W x H)</td>
<td>15’-7” x 12’-6”</td>
<td>15’-7” x 17’-0”</td>
<td>15’-7” x 12’-6”</td>
</tr>
<tr>
<td>Number of Screens</td>
<td>90</td>
<td>66</td>
<td>90</td>
</tr>
<tr>
<td>Screen Invert(^1)</td>
<td>-10.0</td>
<td>-15.0</td>
<td>-11.0</td>
</tr>
<tr>
<td>Intake Pipe Invert</td>
<td>-9.0</td>
<td>-9.0</td>
<td>-10.0</td>
</tr>
</tbody>
</table>

\(^1\)Elevations shown are based on NAVD 88 units feet.

2.3.1 Fish Screen Velocity Limit

For the purposes of this study, it was understood that the intake screen approach velocity should not exceed 0.2 fps. This velocity was calculated using the gross wetted screen area. No screen fouling was considered, nor the effects of sweeping velocities parallel to the river channel.

2.3.2 Screen Headloss Calculations

In order to approximate the screen’s headloss within the InfoWorks CS model, as the screens themselves are not included directly as a model element, a minor headloss coefficient (KL) is calculated based upon the Kirschmer’s formula. This formula defines the headloss through a vertical screen as follows:

\[ h_s = K_s \left( \frac{t}{b} \right)^{\frac{4}{3}} \left( \frac{v^2}{2g} \right) \]

Where \( h_s \) is the loss of head (m); \( K_s \) is the screen loss coefficient of 2.42 for an assumed rectangular bar shape; \( t \) is the thickness of bars of 1.75 (mm), which was assumed; \( b \) is the clear spacing between bars of 1.75 (mm), taken from Ref. 1; and \( v \) is the screen approach velocity (m/s).
2.4 Intermediate Forebay (IF)

The IF is located downstream of the intakes and is an open air, free surface body of water where the Upper Tunnels are joined to the Lower Tunnels.

2.4.1 Function

The principal function of the IF is to provide a potentially, cost effective surface junction structure allowing for the connection of the Upper Tunnels to the Lower Tunnels. As a secondary function, the IF permits various operational control strategies. For example, the pumps at the CCPS could be controlled off the IF level.

2.4.2 Initial Layout

At the beginning of this study, modeling considered the use of an IF layout from MWD with a floor footprint of 2,200 feet long by 2,000 feet wide. The finished floor elevation was EL -9.0 and minimum and maximum operating water surface elevations of EL -1.8 and 20.0, respectively, were established. The top of embankment was EL 32.2. Between the two operating water surface elevations there is approximately 2,300 acre-feet of storage. At a constant CCPS pumping demand of 9,000 cfs, this storage amount equates to approximately 3 hours of supply.

2.4.3 Refined Layout

After shakedown testing of the InfoWorks CS model (discussed below), refinements to the IF were made by MWD. The refined layout has a floor footprint of 1,500 feet long by 800 feet (upstream) to 756 feet (downstream) wide. The finished floor elevation was allowed to vary and was established through InfoWorks CS simulations. The top of embankment was unchanged. Figure 2 shows a plan view of the refined IF layout as provided by MWD.

During shakedown testing, the InfoWorks CS model was used to help establish the role and function of the IF as part of the design. It became apparent that under a pump flow control scheme, the IF simply floats on the hydraulic grade line (HGL). However, because it is essentially a surface junction structure connecting the Upper and Lower Tunnels, the finished floor elevation becomes an important aspect of design. If the floor elevation is set too high at low river elevations and high pumping, then the required HGL cannot be realized as it passes through the floor of the IF. Essentially, the IF becomes a high point in the system acting to block, or restrict flow.

During a pump flow control scheme where pumps are staged on and off before adjustments to upstream control gates are made, the IF volume has little or no impact on hydraulic performance and essentially “floats” on the hydraulic grade line passing through the IF. However, when the IF level is used as the pump control scheme, then volume is a relevant variable. The volume of the IF was found to depend on the pump control scheme assumed – IF level or flow. An IF level pump control scheme, as defined by MWD, is detailed in Section 2.5.1 below. Under these operational conditions MWD set the size of the IF to roughly provide 1 to 3 feet of drawdown for controlling on/off staging of the CCPS pumps.
Figure 2 – Intermediate Forebay Refined Layout
2.5 Simulations

The evaluation of the conveyance system with the InfoWorks CS model evolved over the course of the study and was adapted based upon input from MWD on weekly calls and WEBEX presentations. In general, the simulations performed can be grouped into three testing categories: shakedown, normal operations and an emergency shutdown (Intake 2 closure), and alternatives to the modified as-proposed general arrangement.

Shakedown testing focused on developing a hydraulic understanding of the conveyance system’s critical design points, identifying design disconnects, refinement of the general arrangement, and developing the IF’s role and function within the conveyance system. The refinements to the existing as-proposed general arrangement (referred to in this study as the modified as-proposed general arrangement) during this shakedown testing were then tested under the following two operational conditions:

- Normal Operations – Total pumping demand of 9,000 cfs. Intake roller gates are adjusted as needed in order to achieve a flow balance of approximately 3,000 cfs per intake. Note: the “normal operations” referred to in this study are not the assumed actual normal operations of the system. In actuality, the CCPS demands change both in magnitude and duration throughout the day and season and depend on many factors such as regulated withdrawal amounts, river levels, and downstream demand.

- Emergency Shutdown – In order to stress test the IF under a pump control scheme that measures IF level, all 12 roller gates at Intake 2 are closed over a 20-minute period. Because the CCPS pumps are reading IF levels, there becomes a flow imbalance within the system with the CCPS pumps still operating under a total pumping demand of 9,000 cfs, but the supply from the Sacramento River drops to 6,000 cfs due to the emergency shutdown of Intake 2.

Once the testing of the modified as-proposed general arrangement was complete, three additional alternatives were tested as follows:

- Alternative 1 – The upper tunnel diameters were used, versus the roller gates per the modified as-proposed general arrangement, to provide a rough flow balance between the intakes at a total pumping demand of 9,000 cfs. The upper tunnel diameter between the JS and IF was unchanged – namely 40 feet (see Figure 1). The roller gates were then used to fine-tune the flow balance.

- Alternative 2 – Same as Alternative 1 but started by reducing the upper tunnel diameter between the JS and IF by 20% (i.e., 32 feet) and then adjusted the upper tunnel diameters to provide the rough flow balance between the intakes at a total pumping demand of 9,000 cfs.

- Alternative 3 – Same as Alternative 1 but the IF floor elevation was raised to EL -13.0 on the upstream end and EL -14.0 on the downstream end. This alternative focused on keeping a shallow IF as a deep IF may have constructability issues. The Upper Tunnel diameters were increased to achieve the 9,000 cfs at a river EL 1.0.
2.5.1 Emergency Shutdown (Intake 2 Closure) Operational Details

As discussed above, the intent of simulating an emergency shutdown at Intake 2 was to stress test IF under a CCPS pump control scheme that measures the IF level. In order to perform such a simulation within the InfoWorks CS model using real-time controls, additional operational conditions had to be defined. MWD provided the chronological specifics of such a control scheme as follows.

- Intake 2 roller gates close linearly over a 20-minute period.
- CCPS pump control measures IF level.
- Each of the six CCPS pumps deliver 1,500 cfs each.
- First CCPS pump starts shutting off when the IF level falls 0.5 feet.
- Second CCPS pump stats shutting off when the IF level falls another 0.5 feet.
- CCPS pumps shut down over a 15 minute period and then remain off.

2.5.2 InfoWorks CS Simulation Duration

In order to simplify the number of simulations, both the operational conditions (i.e., normal operations and emergency shutdown) were evaluated in a single 48 hour simulation. During the first 21 hours the system was brought to a steady-state, balanced-flow condition across all three intakes. The required roller gate throttling was established prior to the start of the simulation. InfoWorks CS simulations were made to help establish the amount of throttling by the intake roller gates in order to provide a balance of 3,000 cfs per intake under a total CCPS pumping demand of 9,000 cfs. This first 21 hours of simulation (once steady state is reached) is considered normal operations in this study.

At 21 hours, the roller gates at Intake 2 began to close over a 20 minute period. This is the start the emergency shutdown.
3. Results

As discussed previously, the evaluation of the conveyance system with the InfoWorks CS model evolved over the course of the study and was adapted based upon input from MWD on weekly calls and WEBEX presentations. For the purposes of reporting, the simulation results are grouped into three testing categories: shakedown, normal operations and emergency shutdown (Intake 2 closure), and alternatives to the modified as-proposed general arrangement.

3.1 Shakedown Testing

Shakedown testing involved an initial testing of the existing as-proposed conveyance system under the various pumping head conditions as detailed in Ref. 2 and summarized as follows for ease of reference:

- Low/Low Pumping Head – SR EL 31.4 and CCF EL 0.5 ft
- Normal Low Pumping Head – SR EL 15.0 and CCF EL 0.5 ft
- “Design Pumping Head” – SR EL 9.4 and CCF EL 8.5 ft
- “High Pumping Head” – SR EL 1.9 and CCF EL 8.5 ft

Where SR is the Sacramento River and CCF is the Clifton Court Forebay.

These shakedown tests also considered various demand patterns based upon diurnal flow data provided by MWD. The data provide reported a river stage and corresponding demand at each of the three intakes every 15 minutes. The period of record was from 9/30/1974 to 9/30/1991. This data was reviewed in order to develop typical, observed demand patterns from a seasonal basis. It was observed that in this dataset the demand always ramped up to 3,000 cfs at each intake and that this occurred during river stages as low as EL 0.2 to 0.6 feet and also during the low flow months of October and November. While these demand patterns were tested during the shakedown period, because of uncertainty in its overall quality and reasonableness they were abandoned in favor of a constant pumping demand of 9,000 cfs. This approach was well suited to the stress testing that became a significant focus of this study.

During these initial tests it was noted that the layout of the JS would need modification of the gate invert elevations – namely lowered below the design hydraulic grade line (HGL). The exact gate inverts can be optimized during design. For the purposes of this study, it was assumed that the JS gate inverts would be set to match the IF finished floor elevation.

At the end of shakedown testing, capacity curves (Figure 3) were developed for the hydraulic conveyance system as a function of river elevation. Figure 3 shows how the hydraulic zone of operation is bounded by various system hydraulic capacity constraints – identified on the figure by large black dots with white numbers and described below.
System hydraulic capacity is limited by the invert of the intake pipes. This is EL -9.0 for Intakes 2 & 3 and EL -10 for Intake 5.

System hydraulic capacity is limited by the 0.2 fps intake screen approach velocity limit discussed in Section 2.3.1 above.

System hydraulic capacity is limited by physical constraints within the conveyance system. For example, the IF finished floor elevation or intake pipe diameters.

Shakedown tests showed that the existing as-proposed IF finished floor elevation of EL -9.0 was restricting the overall system’s hydraulic conveyance at river elevations below EL 3.0. Figure 3 shows that hydraulic zone of operation is being bounded on the left side (i.e., lower river elevations) first by the Conveyance System Hydraulic Capacity Limits (dash gray line) rather than the 0.2-fps Intake Screen Limit (light blue line). Based upon this observation, the existing as-proposed was modified to better match the 0.2 fps Intake Screen Limit. A series of InfoWorks CS simulations were performed to increase the conveyance system’s hydraulic capacity. It was found that by lowering the IF floor elevation and increasing the 4-foot-diameter intake pipes to 6-foot-diameter, a better matching (green line) of the system hydraulic capacity limits was obtained.

Another observation from the capacity curves shown in Figure 3 is that a design point for the IF can be established around a river EL 1.0. This is a natural inflection point on the 0.2 fps Intake Screen Limit curve and designing a conveyance system to match this capacity curve provides a unified hydraulic design. As such, it was decided at the end of the shakedown testing to advance a modified as-proposed general arrangement, which would be used as a basis for all future InfoWorks CS simulations. Also, a river EL 1.0 was established as a critical design point for stress testing the IF under a CCPS pump control scheme that measures IF level.
Figure 3 – As-Proposed Hydraulic Conveyance System Capacity Curve

- Modified As-proposed
- Lower IF Floor EL
- 4’ dia. Intake Pipes increased to 6’ dia.

Legend:
- Conveyance System hydraulic capacity limits
- 0.7 fps Intake Screen Limit
- IF Design Point

Hydraulic Zone of Operation
3.2 Normal Operations and Emergency Shutdown (Intake 2 Closure)

Using the modified as-proposed general arrangement of the conveyance system developed out of the shakedown tests, two operational conditions were simulated as discussed in Section 2.5.1 – namely normal operations and emergency shutdown (Intake 2 Closure).

3.2.1 Result Presentation

Because the InfoWorks CS is a fully dynamic model, results can be viewed in a number of ways. For the purposes of this study, a summary table (Table 2) was developed along with both flow and water level history plots at the IF and CC surge shafts (Figure 4 through Figure 9). The summary table has a significant amount of information on it, but the general layout of the table presents the individual tests as individual rows and key parameters/values as columns. Clarification of each column in Table 2 is presented below. All simulations considered a full CCPS pumping demand of 9,000 cfs and all columns are associated with a 9,000 cfs condition.

- **Run ID** – A shorthand notation identifying the InfoWorks CS simulation presented.
- **Description** – Provides a brief description of what was simulated.
- **Intake River Elevation** – The fixed water level used in the InfoWorks CS model at the upstream boundary. Note while simulations used a fixed water level, the model can evaluate changes in the river level in order to simulate both tidal and seasonal fluctuations of the Sacramento River.
- **Upper Tunnel Diameters** – Provides the diameters (units in feet) for each of the main upper tunnel portions.
- **Intake Flow Rates (cfs) Unregulated by 0.2 fps Rule** – The steady state flows from each intake assuming no balancing of flows between the intakes using the roller gates located at the sedimentation tanks (gates remain fully open). Also, there is no trimming of flows with the roller gates in order to prevent an exceedance of the 0.2 fps intake screen approach velocity limit.
- **Percent Gate Opening of All 12 Gates to Balance Flows Regulated by the 0.2 fps Rule** – Represents the steady state flows that occur during normal operations where flows between the intakes are balanced to approximately 3,000 cfs per intake. The InfoWorks CS simulations used full gate closures (or fractions of full gate closures) to balance flows across the intakes, however, it is acknowledged that this function will be performed by closing all gates at a given intake by an equal amount. As such, a percent gate opening assuming all gates used to balance flows at the same time was calculated using a radial gate discharge relationship (i.e., \( Q = C_d A \sqrt{2gh} \)) to calculate a representative percent gate opening. \( C_d = 0.8 \) (typical est. for radial gates) and h is the corresponding headloss value shown in the "River --> Downstream Side of Roller Gates" column – described below. Relationship is for a submerged orifice condition - no overtopping. Gate dimensions assumed to be 12’ x 16’ (H x W). Two values are presented. The first value on the left is simply the percent gate opening of all 12 gates to balance flow. For example, 20% means all twelve roller gates are open 20% of the gate height. The second value is the steady-state flow after flow balancing has been achieved – a condition referred to as “normal operations.”
• **Flow All Intakes Open / Peak Flow If Intakes 3 & 5 Are Not Adjusted at the Same Time During Intake 2 Closing (ex. 3,000 cfs / 0 cfs)** – Two values are presented; both are flows from a given intake. The first value is a repeat of the normal operations steady, balanced flows discussed above. The second value is the momentary maximum flow value at Intakes 3 & 5 that occurs during an Emergency Shutdown (Intake 2 closure). This occurs because the IF level drops due to the assumed CCPS pumping control scheme, which reads IF level in 0.5-foot increments. As the IF level drops, Intakes 3 & 5 hydraulically “see” a greater available driving head and flows increase at these intakes until the CCPS pumps equalize the system and the roller gates are used to rebalance the remaining pump demand of 6,000 cfs between the remaining two intakes – namely 3,000 cfs per intake. Unless otherwise stated (i.e., Run ID Ex.2), there is no trimming of the roller gates at Intakes 3 & 5 during this momentary flow condition such that the 0.2-fps intake screen limit would be maintained.

• **Velocity through Fish Screens at Peak Flow During Emergency Intake 2 Closing (fps)** – Because the intake screens would have some ability to absorb some of the momentary flow spikes reported in the previous column, it is helpful to also report the actual velocity through the gross, wetted screen area. The calculation of this velocity is based upon all screens at a given intake being used with flow evenly distributed and no screen fouling. Velocity is calculated using the gross, wetted screen area at the corresponding river level reported in the “Intake River Elevation” column. Intake 2 reports a 0.00 fps velocity for all tests in order to demonstrate that this intake is being closed.

• **Steady State Intermediate Forebay Elevations (ft)** – Underneath this column heading are four additional sub-columns.
  - **Floor EL** – Two values are reported. The first value is the finished floor elevation of the IF at its upstream (US) end. The second value is the finished floor elevation of the IF at its downstream (DS) end.
  - **All Intakes Open (9,000 cfs)** – This is the IF water level during normal operations, with all intakes in operation and roller gates used to balance flows across the intake to approximately 3,000 cfs each.
  - **Only Intake 2 Closed (6,000 cfs)** – This is the IF water level after emergency shutdown (Intake 2 closure). Roller gates are not used to rebalance flows between Intakes 3 & 5.
  - **Pumping Drawdown (ft) / Min. IF Level (ft)** – Two values are reported. The first value is the temporary drawdown in the IF resulting from the flow imbalance caused by a pump control scheme that uses the IF level. During an emergency shutdown (Intake 2 closure), the CCPS pumps continue to draw 9,000 cfs until the IF level drops 0.5 feet. Because Intake 2 is closing the IF is not being supplied the full 9,000 cfs. This imbalance causes a drawdown in the IF. The second value is the minimum IF level observed during the emergency shutdown. It corresponds to the point of drawdown.

• **Steady State CC Surge Shafts WSEL (ft)** – Similar to the IF water surface elevations reported, two subcolumns are reported at the CC surge shafts.
  - **All Intakes Open (9,000 cfs)** – This is the IF water level during normal operations, with all intakes in operation and roller gates used to balance flows across the intakes to approximately 3,000 cfs each.
  - **Only Intake 2 Closed (6,000 cfs)** – This is the IF water level after emergency shutdown (Intake 2 closure). Roller gates are not used to rebalance flows between Intakes 3 & 5.
• **Steady State Headloss (ft) Values Prior to Intake 2 Closure (All intakes open at 9,000 cfs; gates used to balance flows between intakes)** – This series of columns provides a summary of the headlosses in the upper tunnel portion of the conveyance system.

  - **River → Downstream Side of Roller Gates** – Headloss is report for each of the three intakes and is taken from the river elevation to the downstream side of the roller gates.

  - **Across JS** – Headloss is reported across the JS from two paths – via Intakes 2 and 3. This was done to acknowledge the different paths of flow within the JS; therefore, such an arrangement can accommodate future refinements in the minor loss values from a CFD analysis.

  - **IF** – Headloss is reported for the entrance, which includes the losses through the forebay and for the exit.

  - **Tunnel Losses** – These are the friction losses in the upper tunnel segments from each of the three intakes.

In addition to the development of Table 2, flow and level history plots at the intakes and the CC surge shafts were developed for each of the Run IDs listed in Table 2. These plots are summarized in Figure 4 through Figure 9. Each figure contains two plots. Plot (a) is the flow history plot at each of the intake reception shafts. Plot (b) is the level history plot at both the IF and CC surge shafts. Both plots capture the normal operations and emergency shutdown (Intake 2 closure) conditions – as indicated by zone arrows underneath the x-axis. The x-axis denotes simulation hours, with each tick representing 3 hours. Values reported on these plots correspond with the values found in Table 2. For the level history plots, additional zone arrows are provided above the plot and denote additional information regarding the use of the roller gates to balance or trim flows.

### 3.2.2 Result Observations

The modified as-proposed general arrangement was simulated in three distinct InfoWorks CS tests — Table 2 Run IDs Ex., Ex.1, and Ex.2. Ex. and Ex.1 are identical except that Ex.1 performs the simulation at a higher river elevation – EL 10.0. Ex.1 was simulated in order to test the intake screen’s ability to absorb the momentary spikes in flow (created by the imbalance of a pump control scheme that uses IF level) at higher river levels where there is slightly more available wetted screen area than is available at a river EL 1.0. As a result, the screen velocities at Intake 3 exceeded the 0.2 fps intake screen limit during the momentary spike in flows (i.e., 0.23 fps).

Ex.2 was simulated in order to test if the roller gates at Intakes 3 & 5 could be trimmed during a closure of Intake 2 at a river EL 1.0 in such a manner as to maintain 3,000 cfs and thus not exceed the 0.2 fps intake screen limit. Ex.2 is essentially Ex.1 but roller gates are trimmed at Intake 3 during closure of Intake 2. By trimming the roller gates at Intake 3 the screen velocity was reduced from 0.22 to 0.19 fps, for Ex. and Ex.2 respectively.

### 3.3 Alternatives to the Modified As-Proposed General Arrangement

#### 3.3.1 Result Presentation

See Section 3.2.1.
3.3.2 Result Observations

Upon completion of testing the modified as-proposed general arrangement, three additional alternatives were tested related to sizing options of the Upper Tunnels. Alternative 1 tested a design where the Upper Tunnel diameters were used, versus the roller gates per the modified as-proposed general arrangement, to provide the flow balance between the intakes at a total pumping demand of 9,000 cfs. The Upper Tunnel diameter between the JS and IF was unchanged – namely 40 feet (see Figure 1). Alternative 2 is essentially the same as Alternative 1, but tunnel sizing started by reducing the Upper Tunnel diameter between the JS and IF by 20% (i.e., 32 feet) and then adjusting the remaining Upper Tunnel diameters to provide the flow balance between the intakes at a total pumping demand of 9,000 cfs. Alternative 3 is also essentially the same as Alternative 1, but the IF floor elevation was raised to EL -13.0 on the upstream end and EL -14.0 on the downstream end. This alternative focused on keeping a shallow IF as a deep IF may have constructability issues. The Upper Tunnel diameters were increased to achieve the 9,000 cfs at a river EL 1.0.

Alternative 1 results in an increase of the Intake 2 tunnel portion upstream of the JS from 28 to 30 feet, as this is the most hydraulically restrictive path due to the longest tunnel length. The tunnel portion from Intake 3 to the JS is reduced from 28 to 22 feet and the tunnel portion from Intake 5 to the IF is reduced from 28 to 25 feet. One-foot-diameter increments were assumed in the analysis.

Alternative 2 results in an increase of the Intake 2 tunnel portion upstream of the JS from 28 to 30 feet. The tunnel portion from Intake 3 to the JS is reduced from 28 to 22 feet and the tunnel portion from Intake 5 to the IF is reduced from 28 to 22 feet. One-foot-diameter increments were assumed in the analysis.

Alternative 3 results in an increase of the Intake 2 tunnel portion upstream of the JS from 28 to 40 feet. The tunnel portion from Intake 3 to the JS is increased from 28 to 30 feet and the tunnel portion from Intake 5 to the IF remained unchanged at 28 feet. One-foot-diameter increments were assumed in the analysis.

In each alternative, the intake roller gates were used to balance flows across the intakes during normal operations. No trimming of flows at Intakes 3 and 5 were performed during emergency shutdown (Intake 2 closure). As a result, the screen velocities at Intake 3 exceeded the 0.2 fps intake screen limit during the momentary spike in flows – namely 0.22, 0.26, and 0.24 fps for Alternatives 1, 2, and 3 respectively.
4. References

1. “Volume 2 – Conceptual Engineering Report Drawings, Modified Pipeline / Tunnel Option, Conceptual Engineering Report”; Delta Habitat Conservation and Conveyance Program (DHCCP); California Department of Water Resources (DWR); Final Draft Version; October 1, 2013.

2. “Combined Pumping Plant Option, Clifton Court Forebay Pump Station Concept, Delta Habitat Conservation & Conveyance Program”; Draft Final Technical Memorandum; Task Order No. 4; CDM Smith; February 10, 2014.
## Table 2 – Summary of Simulations

<table>
<thead>
<tr>
<th>Run ID</th>
<th>Description</th>
<th>Intake River Elevation</th>
<th>Upper Tunnel Diameters</th>
<th>Intake Flow Rates (CFS) Unregulated by 0.2 fps Rule</th>
<th>Percent Gate Opening(^1) of All 12 Gates to Balance Flows(^2) Regulated by 0.2 fps Rule</th>
<th>Flow(^3) All Intakes Open / Peak Flow? Intakes 3 &amp; 5 Are Not Adjusted at the Same Time During Intake 2 Closing (Ex. 3,000 CFS / CFS)</th>
<th>Fish Screen Approach Velocity(^4) at Peak Flow During Emergency Intake 2 Closing (FPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ex.</td>
<td>As-proposed design with new IF layout and an increase of intake piping from 4’ to 6’.</td>
<td>1.0</td>
<td>28</td>
<td>28</td>
<td>40</td>
<td>28</td>
<td>3,465</td>
</tr>
<tr>
<td>Ex.1</td>
<td>Same as Ex. but at a higher river EL.</td>
<td>10.0</td>
<td>28</td>
<td>28</td>
<td>40</td>
<td>28</td>
<td>2,427</td>
</tr>
<tr>
<td>Ex.2</td>
<td>Same as Ex. but gates at Intake 3 are trimmed during the emergency closing of Intake 2 in order to prevent the surging of peak flows (i.e., 3,602 CFS) at Intake 2 observed during the Ex. run ID.</td>
<td>1.0</td>
<td>28</td>
<td>28</td>
<td>40</td>
<td>28</td>
<td>2,485</td>
</tr>
<tr>
<td>Alt.1</td>
<td>Upper tunnel diameters adjusted to provide a better flow balance between the intakes at 9,000 cfs. Kept the upper tunnel between the JS and IF 10’.</td>
<td>1.0</td>
<td>30</td>
<td>22</td>
<td>40</td>
<td>23</td>
<td>2,504</td>
</tr>
<tr>
<td>Alt.2</td>
<td>Same as Alt.1 but first reduced the upper tunnel diameter between the JS and IF by 10% and then adjusted the upper tunnel diameters to provide a better flow balance between the intakes at 9,000 cfs.</td>
<td>1.0</td>
<td>30</td>
<td>22</td>
<td>32</td>
<td>22</td>
<td>2,952</td>
</tr>
<tr>
<td>Alt.3</td>
<td>Same as Alt.1 but IF floor elevation raised to EL - 15’ on the upstream end and EL - 14’ on the downstream end. Upper tunnel diameters increased to achieve 9,000 cfs at river EL 1.0.</td>
<td>1.0</td>
<td>40</td>
<td>30</td>
<td>40</td>
<td>28</td>
<td>2,831</td>
</tr>
</tbody>
</table>

**Notes**

1. NC = Not Calculated
2. Model assumed full gate closure. However, because actual operations will likely result in a trimming of all 12 gates equally, a radial gate discharge relationship (i.e., Q=CD^4*sgn(Tgh)) was used to calculate a representative percent gate opening. Cl=0.8 (typical est. for radial gates) and T is the corresponding headloss value shown in the “Inriver – Downstream Side of Roller Gate” column (relationship is for a submerged orifice condition - no overtopping, gate dimensions assumed to be 22’ x 16’ x 6’ x W).
3. Flow balance provided to within +/- 15 cfs of 9,000 cfs.
4. Pumping volume accounted for but allowance for a fireboard is not included, nor recommended for river EL 1.0 pulling 3,000 cfs.
5. Assumes pumps are controlled using a level set point at the IF for a given pump flow rate. This simulation a maximum drawdown in the IF caused by the closure of intake gates (upstream control) as the pumps react to the dropping level and stage off. In practice, pumps may be controlled based on flow control settings and shutdown of an intake would be accomplished by first reducing the pump flow rate to 8,000 cfs, closing the intake gates, and rebalancing the flow between the remaining two intake (downstream control).
6. Fish screen approach velocities calculated assuming all screens in operation, 2% fouling, and using weirs gross screen area.
7. Intake elevation immediately upstream of tunnel interface with junction structure.
<table>
<thead>
<tr>
<th>Run ID</th>
<th>Intake 2 Tunnel Interface</th>
<th>Intake 3 Tunnel Interface</th>
<th>Intake 2 Tunnel Interface</th>
<th>All Intakes Open (9,000 cfs)</th>
<th>Only Intake 2 Closed (6,000 cfs)</th>
<th>Steady State CC Surge Shutoff WSEL (FT)</th>
<th># Pumping Draftdown (FT)/Min. of Level (FT)</th>
<th># Pumping Draftdown (FT)/Min. of Level (FT)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>All Intakes Open (9,000 cfs)</td>
<td>Only Intake 2 Closed (6,000 cfs)</td>
<td>Only Intake 2 Closed (6,000 cfs)</td>
<td>All Intakes Open (9,000 cfs)</td>
<td>Only Intake 2 Closed (6,000 cfs)</td>
<td>All Intakes Open (9,000 cfs)</td>
<td>Only Intake 2 Closed (6,000 cfs)</td>
<td>All Intakes Open (9,000 cfs)</td>
</tr>
<tr>
<td>Ex.</td>
<td>-3.35</td>
<td>-3.27</td>
<td>-6.71</td>
<td>-15 FS / -19 FS</td>
<td>-10.6</td>
<td>-8.2</td>
<td>-1.8 / -12.4</td>
<td>-19.8</td>
</tr>
<tr>
<td>Ex.1</td>
<td>6.14</td>
<td>6.11</td>
<td>3.19</td>
<td>-15 FS / -15 FS</td>
<td>0.2</td>
<td>2.0</td>
<td>-1.5 / -1.27</td>
<td>0.99</td>
</tr>
<tr>
<td>Ex.2</td>
<td>-3.35</td>
<td>-3.27</td>
<td>-9.49</td>
<td>-15 FS / -15 FS</td>
<td>-10.6</td>
<td>-11.3</td>
<td>-2.0 / -12.6</td>
<td>-19.8</td>
</tr>
<tr>
<td>Alt.1</td>
<td>-2.38</td>
<td>-2.37</td>
<td>-5.98</td>
<td>-14 FS / -18 FS</td>
<td>-10.1</td>
<td>-7.3</td>
<td>-1.9 / -12.0</td>
<td>-19.1</td>
</tr>
<tr>
<td>Alt.2</td>
<td>-2.56</td>
<td>-2.52</td>
<td>-6.59</td>
<td>-22 FS / -26 FS</td>
<td>-18.7</td>
<td>-11.1</td>
<td>-1.3 / -20.0</td>
<td>-27.9</td>
</tr>
<tr>
<td>Alt.3</td>
<td>-1.04</td>
<td>-1.02</td>
<td>-4.11</td>
<td>-13 FS / -14 FS</td>
<td>-7.9</td>
<td>-3.8</td>
<td>-1.8 / -9.3</td>
<td>-17.1</td>
</tr>
</tbody>
</table>

**Notes:**
1. Model assumed full gate closure. However, because actual operations will likely result in a trimming of all 12 gates equally, a radial gate discharge relationship (i.e., Q=Cd*A^0.6) was used to calculate a representative percent gate opening. Cd=0.8 (typical est. for radial gates) and h is the corresponding head loss value shown in the "River -> Downstream Side of Roller Gates" column. Relationship is for a submerged orifice condition - no overtopping. Gate dimensions assumed to be 12" x 16" (1 ft x 1 ft).
2. Flow balance provided to within ±15 cfs of 3,000 cfs.
3. Pumping volume accounted for but allowance for a freeboard is not included, nor recommended for river FL,1.0 putting 3,000 cfs.
4. Assumes pumps are controlled using a level set point at the IF for a given pump flow rate. This simulates a maximum drawdown in the IF caused by the closure of intakes gates (upstream control) as the pumps react to the dropping level and stage off. In practice, pumps may be controlled based on flow control settings and shutdown of an intake would be accomplished by first reducing the pump flow rate to 6,000 cfs, closing the intake gates, and rebalancing the flow between the remaining two intakes (downstream control).
5. Fish screen approach velocities calculated assuming all screens in operation, 0.6 foot, and using water gross screen area.
6. WSEL elevation immediately upstream of tunnel interface with junction structure.
3,803 cfs at Intake 3, which exceeds 0.2 fps screen velocity limit unless trimmed along with Intake 2 closing.

All intakes balanced to deliver ~3,000 cfs

Intake 2 Closing over 20 min.

3,118 cfs at Intake 5

Intake 3

Intake 2 (CLOSED)

(a) Flow History at Intake Reception Shafts

MODEL START-UP
ROLLER GATES USED TO BALANCE FLOWS BETWEEN INTAKES
9,000 CFS
TRANSITIONAL
INTAKE 2 CLOSES 2 PUMPS SHUT-OFF
INTAKE 2 CLOSED NO TRIMMING OF FLOWS AT INTAKES 3 & 5
6,000 CFS

IF Level = -10.6

IF level drops -1.8

IF Level = -8.2

CC Surge Shaft Level = -12.2

Intake 2 Closing over 20 min

(b) Level History at IF and CC Surge Shafts

Figure 4 – Run ID Ex.
(a) Flow History at Intake Reception Shafts

Figure 5 – Run ID Ex.1

(b) Level History at IF and CC Surge Shafts
Figure 6 – Run ID Ex.2

(a) Flow History at Intake Reception Shafts

(b) Level History at IF and CC Surge Shafts
(a) Flow History at Intake Reception Shafts

3,629 cfs at Intake 3, which exceeds 0.2 fps screen velocity limit unless trimmed along with Intake 2 closing.

All intakes balanced to deliver ~3,000 cfs

Intake 2 Closing, over 20 min.

3,039 cfs at Intakes 5

Intake 2 (CLOSED)

(b) Level History at IF and CC Surge Shafts

MODEL START-UP
ROLLERS GATES USED TO BALANCE FLOWS BETWEEN INTAKES
INTAKE 2 CLOSES 2 PUMPS SHUT-OFF
INTAKE 2 CLOSED NO TRIMMING OF FLOWS AT INTAKES 3 & 5

9,000 CFS
TRANSITIONAL
6,000 CFS

IF Level drops -1.9

IF Level = -10.1

IF Level = -7.5

CC Surge Shaft Level = -19.3

CC Surge Shaft Level = -11.5

Intake 2 Closing over 20 min.

Simulation Time (HRS)

Normal Operating Condition

Emergency Shutdown Condition (Intake 2 Closure)

Figure 7 – Run ID Alt.1
(a) Flow History at Intake Reception Shafts

4,319 cfs at Intake 3, which exceeds 0.2 fps screen velocity limit unless trimmed along with Intake 2 closing.

3,038 cfs at Intakes 5,

Intake 2 Closing over 20 min.

Intake 3

Intake 5

Simulation Time (HRS)

(b) Level History at IF and CC Surge Shafts

All intakes balanced to deliver ~3,000 cfs

Normal Operating Condition

Emergency Shutdown Condition (Intake 2 Closure)

Model Start-Up

Roller Gates Used to Balance Flows Between Intakes

9,000 CFS

Intake 2 Closes 2 Pumps Shut-Off

Transitional

Intake 2 Closed No Trimming of Flows at Intakes 3 & 5

6,000 CFS

IF Level drops -1.3

IF Level = -11.1

CC Surge Shaft Level = -15.1

Intake 2 Closing over 20 min.

IF Level = -18.7

CC Surge Shaft Level = -27.9

Normal Operating Condition

Emergency Shutdown Condition (Intake 2 Closure)

Figure 8 – Run ID Alt.2
(a) Flow History at Intake Reception Shafts

3,951 cfs at Intake 3, which exceeds 0.2 fps screen velocity limit unless trimmed along with Intake 2 closing.

All intakes balanced to deliver ~3,000 cfs

Intake 2 Closing over 20 min.

Intake 3
Intake 5
Intake 2 (CLOSED)

0000 06:00 12:00 18:00 24:00
Simulation Time (HRS)

(b) Level History at IF and CC Surge Shafts

MODEL START-UP
ROLLER GATES USED TO BALANCE FLOWS BETWEEN INTAKES
9,000 CFS
INTAKE 2 CLOSES 2 PUMPS SHUT-OFF
TRANSITIONAL
IF level drops -1.6
IF Level = -7.9
INTAKE 2 CLOSED NO TRIMMING OF FLOWS AT INTAKES 3 & 5
6,000 CFS
IF Level = -5.6
CC Surge Shaft Level = -9.7
Intake 2 Closing over 20 min.

Simulation Time (HRS)

Normal Operating Condition
Emergency Shutdown Condition (Intake 2 Closure)

Figure 9 – Run ID Alt.3
Appendix I

Basis of Conceptual Design for Tunnel Lining Systems
PROJECT MEMORANDUM

To: Robb Bell, GE (Metropolitan Water District of Southern California)
cc: John Bednarski, PE; Sergio Valles, PE; Howard Lum, SE; Jay Arabshahi, PE; and John Shamma, PE (Metropolitan Water District of Southern California)
From: Steve Dubnewych, PE; Johannes Van Greunen, PE; Mike McKenna, PE; and Steve Klein, PE
Job No.: 4034.14
Date: July 10, 2012 (Revision 1)
Subject: Agreement 82817 Task Order 14 - Tunnel Lining Feasibility Study Project Memorandum No. 6 Conceptual Engineering Evaluations for 40-foot Inside Diameter Gravity Flow Tunnel

1 Introduction

1.1 Project Understanding

Currently, the State Water Project (SWP) and Central Valley Project (CVP) divert water from the Sacramento and San Joaquin rivers to the Central Valley, the San Francisco Bay Area, and Southern California. Several conveyance options are being considered by the Metropolitan Water District of Southern California (MWD) and the California Department of Water Resources (DWR) to improve conditions in the Delta as well as the reliability of water supply. The Delta Habitat Conservation and Conveyance Program pipeline/tunnel option (Delta Tunnel) is a concept being considered to convey water from the Sacramento River to pumping plants in the south Delta. The system will convey water around or under the Delta, allowing habitat restoration in the region and protecting the water supply should levee failure occur (Sanchez et al., 2011).

Our understanding of the proposed project is that may include intake tunnels, pumping stations, forebays, sedimentation basins, and two main conveyance tunnels. The alignment of the main conveyance tunnels is expected to be about 37 miles long. One of the alternatives being considered may consist of twin 40-foot inside diameter (ID) tunnels operating as a syphon under gravity flow conditions, and would be constructed in soft ground at an invert elevation ranging from approximately -165 to -185 feet (NAVD88). Groundwater is expected to be approximately 5 feet below the ground surface. The design flow rate is 15,000 cfs (7,500 cfs per tunnel) and the maximum internal design pressure head in the tunnel will vary from approximately 205 feet at the intake to 194 feet at the outlet relative to the tunnel invert. The maximum net differential...
internal water pressure is expected to be about 50 feet in the North Reach, reducing to 24 feet at the Byron Tract Forebay. Excavation of the tunnels will likely involve the use of closed-face pressurized tunnel boring machines (TBM) such as earth pressure balance or slurry machines. Some of these details may be modified as the design advances.

One of the critical design issues for the project is determining a feasible and cost-effective lining system for the tunnel that can withstand the external loads acting on the tunnel, but also the internal water pressure. This is a significant challenge for a large-diameter water conveyance tunnel in soft ground as soils are usually too weak to reliably resist the internal water pressure; therefore, the lining system must be designed for the full internal water pressure. In addition, soils are susceptible to piping, erosion, and hydraulic fracturing if there is excessive leakage from the tunnel. Consequently, an essentially watertight lining system that can resist the tensile stresses due to the water pressure, limit cracking and control leakage is required. The large tunnel size and relatively high internal pressures are additional factors that impose practical limitations on the types of tunnel linings that may be economically employed.

1.2 Scope of Work

The first phase of the work for this Task Order consisted of performing conceptual-level engineering evaluations for selected viable pressure tunnel lining alternatives as presented in the Tunnel Lining Feasibility Report dated May 2, 2012 prepared by JA.

This memorandum summarizes the results of the conceptual level structural evaluation of a tunnel lining for limited internal pressure based on operating the tunnels under gravity conditions as an inverted syphon system. Conceptual engineering analyses are based on tunnel lining Alternative A1 as discussed in the Tunnel Lining Feasibility Report. An assessment of the lining system and expected leakage is summarized herein. The information presented in this memorandum will serve as a basis for development of a conceptual cost estimate for the tunnel lining for the project.

2 Factors Influencing Long-Term Lining Operational Performance

This section describes the factors considered for the evaluation of the long-term operational performance of the lining, consisting of structural considerations, such as loading, conceptual design approach and crack control.

2.1 Conceptual Lining Design Parameters

For the conceptual structural design, the same geotechnical material properties indicated in the Delta Habitat Conservation and Conveyance Program Team Draft Report (DHCCP, 2010) were considered as part of this study. No additional geotechnical engineering analyses were conducted. It was assumed that the ground will not provide any support to counteract the internal pressure, because of the relatively low modulus of the ground. The loading effect of the
external ground load was ignored in the design and leakage calculations, but its potential contribution is discussed in Section 4.3 below.

For structural analysis and design, ACI 318, ACI 224, and Eurocode EC2 recommendations were considered to assess concrete crack widths for designing reinforcement. The 20-inch segmental lining thickness described above also provides sufficient space to accommodate the reinforcement required to carry the tension across the radial joints.

2.1.1 Load Cases

The principal load cases considered in the conceptual design of the tunnel lining are:

- Ground load and external water pressure (during construction and after dewatering)
- Ground load and external water pressure combined with working internal pressure
- Ground load and external water pressure combined with transient internal pressure
- Pressures due to grouting of the annulus
- Segment handling and storage
- TBM jacking loads
- Shrinkage and temperature
- Seismic demands

During final design, these load cases, and possibly others, will have to be evaluated in greater detail. However, for this conceptual evaluation, the focus is on developing a solution to support the differential internal water pressure through the use of reinforcing steel to accommodate the hoop tension and checking that basic structural requirements could be met. More detailed structural design evaluations will be required if this option is selected for preliminary design, including possibly additional research and testing of prototype linings or components of the linings.

2.1.2 Conceptual Design Approach for This Study

Excavation with a TBM and using a gasketed precast concrete segmental lining as the excavation support is considered the only realistic construction option for this tunnel given its size, length, and the expected ground conditions. The segmental lining thickness is based on our experience with tunnels of similar size, and only a preliminary precast concrete segmental lining evaluation was performed to confirm that a 20-inch-thick, 8,000 psi concrete lining will be adequate to support the loads and provide sufficient space for the details required to carry the tension across the radial joints. It was also assumed that the ground does not provide any support to counteract the internal pressure. Considering a worst case load case, the loading effect of the overburden ground load was ignored to calculate the hoop tensile reinforcing. However, the reliability of assuming some loading contribution from the ground must be evaluated and confirmed for the final design.
Because of the nature of the project, the principal structural element to ensure long-term performance is the hoop reinforcing steel embedded in concrete. To protect the reinforcing will require not only special attention to concrete quality, but also design for control of anticipated crack widths. For environmental structures exposed to an aggressive environment, both ACI 224 and the Eurocode recommends limiting average crack widths to 0.008 inch (0.2 mm). Where environmental conditions are not as severe, average crack widths of 0.012 to 0.016 inch (0.3 to 0.4 mm) is considered acceptable. The latter crack width limit is mainly to ensure that surface crack widths are of an acceptable size. Crack widths depend on concrete cover, reinforcing steel stress, bar size, and spacing and can be estimated with formulas, based on empirical research, provided by the referenced publications. Setting realistic allowable crack width limits commensurate with project objectives and anticipated environmental conditions is an important input parameter for the design.

3 Alternative A1: One-pass Precast Concrete Segmental Lining with Tension Reinforcement

Based on using a one-pass precast segmental lining, a conceptual design was prepared based on Alternative A1 in the Draft Tunnel Lining Feasibility Report (Jacobs Associates, 2012) with bar reinforcing to resist the internal pressure, as described below and shown on the preliminary drawings SK-401 and SK-402 (see Appendix A). The likely construction sequence and constructability issues are also discussed for this alternative. The watertightness of the lining is assessed and the potential leakage estimated in Section 4 below.

3.1 Conceptual Design

The conceptual design includes a 20-inch thick precast double gasketed segmental lining (8,000 psi concrete) as the initial and final support. The conceptual design of the lining for the five tunnel reaches is shown in Table 1. The hoop tension requires two layers of bars at 10-inch spacing when using a reinforcing tension stress of 20 ksi to control cracking. The required hoop tension reinforcing and bolt connection for the radial joints are summarized in Table 2. However, even with measures outlined in ACI 224, the cracked segmental lining would likely be relatively permeable under the internal pressures expected for this option of the Delta Tunnel. Therefore, to ensure long term watertight performance of the tunnels the interior surface of the tunnel lining would potentially need to be lined with an impervious barrier (e.g., T-Lock, Agru Sure Grip, or equivalent) for all of the five tunnel reaches currently being considered by DWR. See the further discussion of the effect of ground loading on the permeability of the lining and need for a membrane.

The challenge with this design is to connect the reinforcing across the radial joints. A wedge system similar to the one developed for the Metropolitan Area Outer Discharge Channel (MADOC) tunnel, or a bolted connection similar to the one used for the South Bay Ocean Outfall project would be appropriate for this option (refer to the Draft Tunnel Lining Feasibility Report (Jacobs Associates, 2012) for descriptions of these projects). In the circumferential joints, the use of dowels, similar to the ones made by Sofrasar, would be adequate to compress the
gasket and maintain longitudinal continuity, since both tensile strains due to the net internal pressure and anticipated seismic loads are small.

3.2 **Anticipated Construction Sequence**

The conceptual construction sequence for Alternative A1 would consist of the following:

1. The tunnel will be excavated by TBM and lined with the precast concrete segments.
2. The bolted connections across the radial joints will be made as the segment ring is erected.
3. The annulus will be grouted behind the tail shield, and a secondary grouting operation may be needed to ensure that any voids are completely filled.
4. All deep bolt pockets will subsequently be filled with concrete or mortar behind the excavation and off the critical path (the necessity for T-lock patching of bolt pockets will be determined during final design).

3.3 **Constructability Issues**

As for any one-pass lining alternative, ring build tolerances for Alternative A1 will need to be very tight to ensure proper alignment of bolting holes, but even more so to limit offsets of gaskets, which would result in greater long-term leakage. Double gaskets would likely be required to minimize exfiltration caused by misaligned segments and ring deformation.

If a system similar to the one implemented on the MAODC project were to be used, ring build tolerances would be even more critical. A bolted connection could be slotted or slightly oversized to allow for some give during bolt installation; however, a wedge assembly connection would require nearly perfect ring installations every time. This level of tolerance would result in a slower overall advance rate for the tunnel boring machines. All bolt pockets will have to be patched to protect them long term against corrosion and to minimize the impact that the deep pockets will have on system hydraulics.
Table 1: Conceptual Design of 40-foot ID Precast Segmental Lining

<table>
<thead>
<tr>
<th>Tunnel Diameter</th>
<th>40 feet ID</th>
<th>20 inch Segment thickness</th>
<th>Concrete cover</th>
<th>2 inches</th>
<th>Ground</th>
<th>Poisson’s</th>
<th>Ratio</th>
<th>0.3</th>
<th>600 ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alternative A1: One-pass Lining with Rebar and Bolts at Radial Joints</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Reach 1 Head (ft)</strong></td>
<td><strong>Hoop Tensile Force</strong></td>
<td><strong>Steel Yield Stress</strong></td>
<td><strong>Steel Design Stress</strong></td>
<td><strong>Required Steel Area</strong></td>
<td><strong>Bar Size/Bolt Dia</strong></td>
<td><strong>Bar Area</strong></td>
<td><strong>Bar Spacing</strong></td>
<td><strong>Steel Area</strong></td>
<td><strong>Steel Area</strong></td>
</tr>
<tr>
<td>5 ft segment</td>
<td>323.7</td>
<td>60</td>
<td>20.00</td>
<td>16.185</td>
<td>11</td>
<td>1.560</td>
<td>10</td>
<td>18.720</td>
<td>1.56%</td>
</tr>
<tr>
<td>3 Bolts (LF 1.4)</td>
<td>323.7</td>
<td>84.8</td>
<td>60.57</td>
<td>5.344</td>
<td>1.625</td>
<td>2.074</td>
<td>20</td>
<td>6.222</td>
<td>0.52%</td>
</tr>
<tr>
<td>Reach 2 Head (ft)</td>
<td>44.0</td>
<td>284.9</td>
<td>84.8</td>
<td>60.57</td>
<td>4.703</td>
<td>1.5</td>
<td>1.767</td>
<td>20</td>
<td>5.301</td>
</tr>
<tr>
<td>5 ft segment</td>
<td>300.0</td>
<td>84.8</td>
<td>60.57</td>
<td>4.703</td>
<td>1.5</td>
<td>1.767</td>
<td>20</td>
<td>5.301</td>
<td>0.44%</td>
</tr>
<tr>
<td>3 Bolts (LF 1.4)</td>
<td>220.1</td>
<td>84.8</td>
<td>60.57</td>
<td>3.634</td>
<td>1.375</td>
<td>1.485</td>
<td>20</td>
<td>4.455</td>
<td>0.37%</td>
</tr>
<tr>
<td>Reach 4 Head (ft)</td>
<td>34.0</td>
<td>181.3</td>
<td>84.8</td>
<td>60.57</td>
<td>2.993</td>
<td>1.25</td>
<td>1.227</td>
<td>20</td>
<td>3.682</td>
</tr>
<tr>
<td>5 ft segment</td>
<td>250.0</td>
<td>84.8</td>
<td>60.57</td>
<td>4.703</td>
<td>1.5</td>
<td>1.767</td>
<td>20</td>
<td>5.301</td>
<td>0.44%</td>
</tr>
</tbody>
</table>

Table 2: One-pass Lining Alternative A1: Preliminary Segmental Lining Hoop Reinforcing Requirements

<table>
<thead>
<tr>
<th>Reach No.</th>
<th>Net Differential Internal Water Pressure (feet)</th>
<th>Reinforcing Bar Size (hoop steel)</th>
<th>Bolt Size at Radial Joints (inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50 to 44</td>
<td>#11</td>
<td>1.625</td>
</tr>
<tr>
<td>2</td>
<td>44 to 40</td>
<td>#10</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>40 to 34</td>
<td>#10</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>34 to 28</td>
<td>#9</td>
<td>1.375</td>
</tr>
<tr>
<td>5</td>
<td>28 to 24</td>
<td>#8</td>
<td>1.25</td>
</tr>
</tbody>
</table>
4 Leakage Estimate

This section summarizes the preliminary leakage estimates for the conceptual tunnel lining Alternative A1. The tunnel would consist of twin 40-foot-ID tunnels constructed in soft ground at invert depths ranging from approximately 160 to 175 feet below the ground surface (bgs). Groundwater is expected to be at a depth of approximately 5 feet bgs. The maximum net differential internal water pressure is expected to be about 50 feet for Reach 1 reducing to a minimum of 24 feet for Reach 5 at the Byron Tract Forebay.

For a discussion of the background and calculations to estimate the leakage through the reinforced concrete lining and the gasketed joints of the precast segmental lining refer to the Tunnel Lining Feasibility Report ( Jacobs Associates, 2012).

4.1 Leakage through Reinforced Concrete

During the operation of the tunnel system, the net internal water pressure will cause tensile circumferential strain in the lining. As the pressure-induced tensile strain develops, radial (longitudinal) cracking of the lining will take place, causing permeability of the lining to increase. Because of the cracks, pressure tunnels lined with reinforced concrete are classified as semi-permeable linings. The combined effect of all the cracks in the lining determines its permeability characteristics. Table 3 summarizes the estimated leakage rate through reinforced concrete based on the analytical method by Fernandez, assuming the ground surrounding the tunnel has a modulus of elasticity and permeability of 600 kips per square foot (ksf) and 0.0000984 ft/sec (0.003 cm/s), respectively. The assumed permeability of the ground is roughly equivalent to the permeability of sand.

In addition to leakage rate, the head loss across the lining expressed in terms of percentage of the net internal water head is shown in Table 3. Only approximately 4% to 5% of the net internal water head is lost through the lining when the cracking of the lining is considered. The remaining water pressure would need to dissipate through the ground surrounding the tunnel.

<table>
<thead>
<tr>
<th>Reach No.</th>
<th>Net Internal Water Head</th>
<th>Crack Spacing</th>
<th>Crack Width</th>
<th>Lining Permeability</th>
<th>Head Loss Across Lining</th>
<th>Estimated Leakage Rate per Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50</td>
<td>14.2</td>
<td>0.0078</td>
<td>0.000058 (1.76E-03)</td>
<td>5.0</td>
<td>11.3</td>
</tr>
<tr>
<td>2</td>
<td>44</td>
<td>14.3</td>
<td>0.0083</td>
<td>0.000070 (2.13E-03)</td>
<td>4.2</td>
<td>10.1</td>
</tr>
<tr>
<td>3</td>
<td>40</td>
<td>14.3</td>
<td>0.0076</td>
<td>0.000053 (1.60E-03)</td>
<td>5.5</td>
<td>9.0</td>
</tr>
<tr>
<td>4</td>
<td>34</td>
<td>14.6</td>
<td>0.0081</td>
<td>0.000064 (1.94E-03)</td>
<td>4.6</td>
<td>7.7</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>15.1</td>
<td>0.0086</td>
<td>0.000073 (2.22E-03)</td>
<td>4.0</td>
<td>6.3</td>
</tr>
</tbody>
</table>
4.2 **Leakage through Gasketed Joints of the Segmental Lining**

In theory, leakage should not occur at or below the design water pressure for a well-designed gasket. However, imperfections such as offsets between erected segments, post-installation damage, debris trapped between the gaskets, and other factors could result in minor leakage.

The maximum net internal water pressure excluding transient pressure for the tunnels on this Project is approximately 1.5 bar. Because this pressure is lower than the available observed values, a leakage estimate based on the values available (2 to 3.5 bar) will be conservative. A reasonable range based on the data may be 0.35 to 2.0 L/m²/day. Applying these leakage rates to a segmentally lined tunnel with an inside diameter of 40 feet, the estimated leakage rate would range from approximately 0.0017 to 0.0095 cubic foot per second (cfs) per 1,000 linear feet of tunnel. These values are very small compared to the leakage because of cracking of the concrete shown in Table 3, and are therefore considered negligible for these calculations.

4.3 **Effect of External Ground Loading on Leakage**

Although the ground modulus is relatively low and the ground can only support a very small part of the internal pressure (and is therefore neglected in the conceptual analysis), the ground is expected to exert significant pressure on the lining long-term. To assess and illustrate the effects of the ground loading both the percentage of internal pressure transmitted to the ground and the average effective overburden pressure, have been plotted vs. the crack width for each tunnel reach in Figures 1 to 5. From these Figures it is clear that the percentage of internal pressure transmitted to the ground rises rapidly as crack widths increase. The average effective overburden pressure is taken as the average of the effective vertical pressure at the tunnel crown and the effective lateral pressure at springline based on a Ko of 0.5. For convenience the average effective overburden pressure has also been expressed in terms of equivalent excavated tunnel diameters.

Figure 1 indicates that in Reach 1 a ground loading of 41% of the effective overburden pressure or an equivalent ground load of 0.89 excavated tunnel diameters would prevent any cracking of the lining. Figure 5 indicates that in Reach 5 a ground loading of 21% of the effective overburden pressure or an equivalent ground load of 0.49 excavated tunnel diameters would prevent any cracking of the lining. To ensure a reasonable level of reliability a factor of safety of the order of 1.5 on these overburden pressure values would be required. For Reach 1 a ground loading of 62% of the effective overburden pressure or 1.34 excavated tunnel diameters would be needed to counteract the internal pressure. For Reach 5 a ground loading of 32% of the effective overburden pressure or 0.75 excavated tunnel diameters would be needed to counteract the internal pressure. To put this in perspective for example, Terzaghi (Deere et al., 1969) proposes a long-term ground loading varying from 0.62 to 1.38 diameters for circular tunnels in gravel and sand. Using the equivalent ground load of 0.62 diameter for Reach 5 results in a factor of safety against cracking of about 1.25 and approximately balances the net
internal pressure for that reach. However, it should also be kept in mind that if the tunnel is located in permeable ground, then leakage from the tunnel may initiate piping, erosion, and hydraulic fracturing, leading to excessive exfiltration and an unacceptable level of leakage.

To confirm the ground load that can be expected for this project additional analysis using numerical methods should be employed during the next design phase. Once additional geotechnical information becomes available the ground behavior can be modeled and various mitigation measures such as a second round of grouting can also be evaluated. In addition, the permeability of the ground plays a very significant role in the leakage calculations presented herein, because the cracked reinforced concrete lining is considered very permeable and leakage is controlled by the ground permeability. Additional geotechnical information is required not only about the ground properties regarding strength and stiffness, but also regarding ground permeability and the possible interconnected nature of any layers of sands and gravels that may be encountered along the alignment at tunnel depth.

![Figure 1: Reach 1 – Effective Overburden Pressure vs. Concrete Lining Crack Width](image-url)
Figure 2: Reach 2 – Effective Overburden Pressure vs. Concrete Lining Crack Width

Figure 3: Reach 3 – Effective Overburden Pressure vs. Concrete Lining Crack Width
Figure 4: Reach 4 – Effective Overburden Pressure vs. Concrete Lining Crack Width

Figure 5: Reach 5 - Effective Overburden Pressure vs. Concrete Lining Crack Width
5 Summary of Lining Evaluation

Based on our understanding of the project objectives and our experience with similar water conveyance tunnels, it appears that lining Alternative A1, the one-pass lining with bolted connections at the radial joints, could be the most cost-effective approach for the range of differential internal pressures to which the tunnel will be subjected.

A critical element in the development of a watertight one-pass lining system subject to internal pressures is the performance of the gaskets and long-term durability of the gasket system over the life of the project. Based on our experience with the Arrowhead project, we believe that gaskets can be designed to perform acceptably for the anticipated water pressures. However, additional testing of both short- and long-term performance of gaskets will likely be required to address gasket compression, creep, effects of cyclic loading, offset, and fabrication tolerances, all under hydrostatic pressure conditions.

Another critical issue that must be considered is whether the net internal pressure can be reliably balanced by some portion of the overburden load with an adequate factor of safety against leakage and potential ground erosion. This could make the use of a watertight membrane unnecessary because the width of the cracks in the lining would be substantially reduced resulting in limited leakage through the lining. This issue requires further in-depth study once more comprehensive geotechnical information is available. At this stage of the design and to ensure long term watertight performance of the tunnels, it is recommended to include the cost of lining the interior surface of the tunnel with an impervious barrier for all the tunnel reaches.
6 References

ACI 224 Control of Cracking in Concrete Structures

ACI 318 Building Code Requirements for Structural Concrete and Commentary


Eurocode 2: Design of Concrete Structures, 2004


Appendix A: Drawings


Drawing SK-402: Typical Segmental Lining Reinforcing
TUNNEL CROSS SECTION
SCALE NOT TO SCALE

1. CONCRETE 28-DAY STRENGTH 8000 psi.
2. SEGMENT RING WIDTH 9'-6".
3. BOLT/ANCHOR PLATE/REBAR X4 PER SEGMENT RING

METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA
FINAL TUNNEL LINING CONCEPTUAL ENGINEERING AND EVALUATION
ONE PASS SEGMENTAL LINING GRAVITY TUNNEL ALTERNATIVE A1:
REINFORCING WITH BOLTED CONNECTIONS AT RADIAL JOINTS
Appendix J

Alternative Designs for Tunnel Lining Systems
Conceptual Design of Tunnel Linings for the Resistance of Internal Pressure – BDCP MPTO/CCO

PREPARED FOR: Metropolitan Water District of Southern California
PREPARED BY: CH2M HILL
DATE: November 6, 2014
PROJECT NUMBER: 650647.03.31.02.01

1.0 Introduction

Subsequent to the conceptual arrangement presented in the draft Bay Delta Conservation Plan (BDCP) Environmental Impact Report/Environmental Impact Statement, CH2M HILL, working for the California Department of Water Resources (DWR), developed a concept that relocated the combined pumping facilities to the vicinity of Clifton Court Forebay (CCF). This revised concept was further evaluated by the Metropolitan Water District of Southern California (MWD), including conceptual development of the pumping facilities and revised hydraulic analyses. This revised concept significantly alters the anticipated internal tunnel pressures during operation. To address the revised pressure demand on the structural tunnel elements, a conceptual tunnel lining design is required.

2.0 Revised Facilities

CH2M HILL understands that the combined pumping plant would be located at the northeast corner of the Clifton Court Forebay, that a slightly enlarged Intermediate Forebay (IF) would be located approximately 5 miles south of Planned Intake No. 5 on the Sacramento River, and that the North Tunnels connecting the intakes to the IF were upsized to 28 and 40 feet in finished inside diameter.

The results of hydraulic analyses described in AECOM Technical Memorandum No. 2 indicate that a typical high Sacramento River stage of el. 15 feet could result in a static shut-in pressure of approximately 15 feet in excess of the assumed groundwater head of el. 0 feet in both the North Tunnels (between the intakes and the IF) and the Main Tunnels (between the IF and CCF). Further hydraulic analysis provided by MWD indicated that anticipated surge pressures during pump failure would be up to 0.5 foot higher than the static case in the North Tunnels and up to 5 feet higher in the Main Tunnels.

The purpose of this memorandum is to present a number of approaches available to accommodate unbalanced internal hydraulic pressures within the North and Main Tunnels, and to present a conceptual design approach for the 28- and 40-foot BDCP tunnels under internal pressures of up to 20 feet of head.

3.0 Mitigation Measures for Internal Pressure

As discussed previously, the lining may be subject to surge pressures that could, in the case of the 40-foot-diameter tunnel, be up to 20 feet greater than the external pressures. The following damage or degradation could occur if the design does not address this scenario:

- Damage to joint connection elements without adequate capacity for the loads, which could be brittle in nature for some connection types.
- Excessive joint opening and leakage into the annulus outside the tunnel. If this occurs frequently over the lifetime of the tunnel, uncontrolled leakage through the liner could compromise the lining support.
- Shear deformations across the joint could occur on opening. In an extreme case this could result in gasket lipping tolerances being exceeded, leading to the gasket becoming compromised.
In extreme cases the joint could open too much, resulting in significant escape of water, ground ingress, or both.

Therefore the design must mitigate the effects of any net internal pressures. The aim of this technical memorandum is to propose other tension-resisting alternatives additional to the bolted joint detail as shown in the Conceptual Engineering Report (MWD, 2014).

3.1 Baseline for Comparisons

With the exception of the solutions that require non-standard geometries, all the linings presented herein are based on the common geometrical configuration shown in Table 1, unless otherwise noted:

<table>
<thead>
<tr>
<th>TABLE 1</th>
<th>Baseline for Comparison of Options to Resist Internal Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tunnel Feature</strong></td>
<td><strong>Internal Diameter</strong></td>
</tr>
<tr>
<td>Thickness</td>
<td>22 inches</td>
</tr>
<tr>
<td>Length</td>
<td>6 feet</td>
</tr>
<tr>
<td>Segmentation</td>
<td>8 segments plus key</td>
</tr>
<tr>
<td></td>
<td>3 dowels/dowel positions per segment</td>
</tr>
<tr>
<td></td>
<td>1 dowel/dowel position on the key</td>
</tr>
<tr>
<td></td>
<td>25 dowel positions overall</td>
</tr>
</tbody>
</table>

Reinforcement: Conventionally reinforced
Concrete strength: 7000 psi
Sealing: Single EPDM gasket

Table 1 shows a typical configuration for these tunnel sizes. It will likely be altered at detailed design once the balance of the various requirements for the tunnel are better understood. Nevertheless, it provides a reasonable basis for comparing options at this stage.

The design net internal pressures (including maximum surge conditions) are:

- 20.0 feet net internal pressure for the 40-foot-diameter tunnel
- 15.5 feet net internal pressure for the 28-foot-diameter tunnel

The remainder of this section proposes different alternative tension-resisting options using the information listed above as a baseline.

3.2 Reliance on Ground Loads

While it is unlikely that full overburden will act on the lining, it is reasonable to expect that there will be some effective soil pressure acting on the lining. The tunnel boring machine will control inward displacements to an extent in order to control settlements, and the soil will not be self-supporting in the long term, if at all.

Based on experience from trying to determine the minimum pressure on the lining in the past, including those made in the analysis of minimum soil loads on the Blue Plains Tunnel (Harding et al., 2014), it is likely that at the buoyant weight of soil at least half a tunnel diameter above the tunnel will act on the crown. This equates to a resistance to a net internal pressure of 17 feet and 12 feet for the 40- and 28-foot-diameter tunnels, respectively. These pressures are slightly less than the design net pressures, so careful analysis would be required to ensure that the required pressures could be relied upon.
Observations

If the lower-bound soil pressure is less than required to resist the net internal pressure, it could still be relied upon to reduce the net pressure that other systems have to be designed for.

Experience with cohesive material indicates that while the effective soil pressures tend to be larger than those that might be expected in the granular material, these pressures will only be exerted in the medium term, after post-tunneling consolidation effects have occurred. This is dependent on the permeability of the ground and could occur several years after the completion of tunneling.

Expansion on the lining could result in increased soil loads being mobilized. This principle is often used in rock but is not suitable for monolithic linings in soil because the relative stiffness of the soil in relation to the lining is too low. However, in smaller linings only small openings in the joint are often required to mobilize the small resistance required to resist the tension. At such opening displacements the gaskets remain watertight and the risk of lipping occurring when the segments return to compression is low. However, at the proposed diameters the openings would be too large, so this option is not recommended. Only soil loads that arise on the lining prior to the application of the tension force should be considered.

It is important to note that successfully implementing this scheme will require both good quality ground investigation and a comprehensive sensitivity analysis. A robust site investigation is required to establish the ground characteristics along the alignment. However, even with good quality data on those characteristics, some uncertainty as to their behavior will remain. Furthermore, the analysis undertaken for Blue Plains Tunnel demonstrates that the tunnel boring machine (TBM) operation, and particularly the characteristics and behavior of the annulus between the TBM tail shield and the ground, also have a highly significant influence on the final ground pressure on the lining. It is difficult to specify TBMs and operation to control such behaviors to a high degree of accuracy, therefore, it is necessary to consider a wide range of operating conditions. Nevertheless, even given such a range of inputs, it has been possible to establish a lowest credible ground load suitable for use in design in the past.

It would be prudent to ensure that any calculated loads are actually secured. If a robust lower-bound estimate is employed in conjunction with a load factor of 1.0 or less then the ground load may not be enough. While load testing could be employed to verify that the ground load is being achieved, a mitigation measure (discussed subsequently) is recommended in case the required pressures are not observed.

Advantages

Relying on the effective soil loads is the least expensive option to construct as it requires no changes to the lining design provided sufficient pressure can be relied upon. Furthermore, the fact that the lining does not go into tension could result in less reinforcement being required.

No modifications to the lining design or special connection features are required.

Disadvantages

It is not straightforward to calculate a minimum design pressure because most analytical techniques used to establish the soil pressures on a lining are aimed at determining the upper-bound—or maximum—design pressures. Any reliance on soil pressures would require instrumentation to demonstrate that the minimum effective pressures are achieved. This is particularly the case in the cohesive material, where the total pressure on the lining immediately following construction could be less than the long-term external water pressure.

Difficulties of calculation notwithstanding, the ground load may not provide sufficient resistance to resist the internal tension forces in their entirety.
3.3 Use of Shear Dowels to Resist Tension

Dowels on the circumferential joint resist differential circumferential movements across the circumferential joint. This resistance can provide restraint against the segments opening in tension, as shown in Figures 1 and 2.

Figure 1. Dowels resisting tension: conventional 3 dowel arrangement

![Diagram of 3 dowel arrangement](image)

- 1. Opening created by segments moving apart circumferentially
- 2.1 Segments moving apart circumferentially resisted by a single dowel on each joint on this side
- 2.2 Segments moving apart circumferentially resisted by two dowels on each joint on this side
- 3. Path of tensile forces across opening

Figure 2. Dowels resisting tension: 4 dowel arrangement

![Diagram of 4 dowel arrangement](image)

- 1. Opening created by segments moving apart circumferentially
- 2. Segments moving apart circumferentially resisted by two dowels on each joint on both sides
- 3. Path of tensile forces across opening

The dowel system has the following features:

- The complete tension force from the ring is transferred across the circumferential joint at one or two dowel locations on one side and two on the other.
- The opening at the joint is the sum of the shear displacement of the dowels on either side and the combined tolerances on dowel location.
- For the 40-foot-diameter tunnel and a 15-foot differential working pressure, the tension across this joint is 526 kN (115 kips) on the one dowel side and half that value on the opposite side.

Few options exist that provide working load at this level. Anixter (formerly called Sofrasar) produces a dowel called the SOF-SHEAR 400 that can achieve the conceptual working load. However, two dowels would be required at each dowel location to provide the required resistance. Table 2 lists the key figures of the dowel system.

### TABLE 2

<table>
<thead>
<tr>
<th>Feature</th>
<th>40-foot-diameter Tunnel</th>
<th>28-foot-diameter Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pressure</td>
<td>20.0 feet net internal</td>
<td>15.5 feet net internal</td>
</tr>
<tr>
<td>Tension per ring</td>
<td>718 kN (161 kips)</td>
<td>394 kN (88 kips)</td>
</tr>
<tr>
<td>Dowel pairs on side</td>
<td>2</td>
<td>1</td>
</tr>
</tbody>
</table>
TABLE 2

Key Features of the Dowel System

<table>
<thead>
<tr>
<th>Feature</th>
<th>40-foot-diameter Tunnel</th>
<th>28-foot-diameter Tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear on dowel on side 1</td>
<td>179 kN (40 kips)</td>
<td>197 kN (44 kips)</td>
</tr>
<tr>
<td>Displacement of dowel pair on side 1</td>
<td>3.1 mm</td>
<td>3.4 mm</td>
</tr>
<tr>
<td>Dowel pairs on side 2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Shear on dowel on side 2</td>
<td>179 kN (40 kips)</td>
<td>98 kN (22 kips)</td>
</tr>
<tr>
<td>Displacement of dowel pair on side 2</td>
<td>3.1 mm</td>
<td>1.5 mm</td>
</tr>
<tr>
<td>Combined tolerances on dowel location</td>
<td>2.0 mm</td>
<td>1.0 mm</td>
</tr>
<tr>
<td>Total worst case opening</td>
<td>8.2 mm</td>
<td>5.9 mm</td>
</tr>
<tr>
<td>Required test pressure</td>
<td>1.2 bar</td>
<td>0.9 bar</td>
</tr>
<tr>
<td>Radial (outward) deformation</td>
<td>9.7 mm</td>
<td>7.1 mm</td>
</tr>
</tbody>
</table>

Observations

Correspondence with other dowel manufacturers has revealed that while they can achieve the required loads, the displacements associated with the loads are much larger than those for the SOF-SHEAR dowel. Therefore the SOF-SHEAR dowel is likely the only product being produced by a recognized segment fixture supplier that is suitable for application at such a high diameter in this particular configuration. However, other manufacturers are working to achieve these loads with lower deflection. At least one manufacturer is developing a product that has similar performance to the Anixter product and is likely to be able to bring something to market within the next couple of years. Therefore, the solution will probably not be dependent on one supplier.

The 40-foot-diameter tunnel would require two dowel locations on either side. The proposed configuration features four dowel locations per segment. If a normal key were installed, the tension would have to be resisted by only one dowel location, therefore, the key has to be the same size as normal segments. If this arrangement includes a stiff gasket, it may be possible to achieve up to 1 bar net internal pressure.

The beneficial effect of the dowel system was taken into account for the recent design of the Blue Plains Tunnel in Washington DC. The tunnel’s characteristics are shown in Table 3.

TABLE 3

Blue Plains Tunnel Characteristics

<table>
<thead>
<tr>
<th>Feature</th>
<th>23 feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Internal diameter</td>
<td></td>
</tr>
<tr>
<td>Ring length</td>
<td>6 feet</td>
</tr>
<tr>
<td>Ring arrangement</td>
<td>6 segments plus key</td>
</tr>
<tr>
<td>Dowel arrangement</td>
<td>3 dowels per segment</td>
</tr>
<tr>
<td>Net internal water pressure</td>
<td>30.0 feet net internal</td>
</tr>
<tr>
<td>Net internal water pressure after accounting for minimum external effective ground pressure</td>
<td>8.4 feet net internal</td>
</tr>
<tr>
<td>Tension per ring</td>
<td>160 kN (161 kips)</td>
</tr>
<tr>
<td>Dowel capacity</td>
<td>160 kN</td>
</tr>
</tbody>
</table>
The original analysis for the Blue Plains Tunnel allowed for load share between the radial joint bolts and the dowels, with the bolts taking most of the load. However, a long-term check was undertaken with the bolts omitted as they could not be expected to last the 100-year design life. Under this scenario, load share with the ground was allowed for the serviceability check (gasket opening), although most of the load was resisted by the dowels. In an ultimate limit state case the dowels were sufficient to resist the required load by themselves.

The Bay Delta project may be able to adopt a similar approach. However, given a likely higher reliance on the dowels, it is likely that a full-scale test of the system would be required to confirm the design.

The soil load (as discussed in Section 3.2) could significantly reduce the design load for the dowels. If such reductions could be achieved, the number of dowels could be reduced. Furthermore, alternative dowel types that exist on the market today could become suitable at lower dowel loads.

**Advantages**

Shear dowels assist with the ring build, and improve both build time and accuracy. Therefore, the only impacts on schedule are likely to be positive. Similarly, dowels also resist stepping and lipping movements between the segments during service, ensuring that significant lips between segments cannot occur due to repeated opening of the joint.

The anticipated levels of displacement will mobilize some resistance of the ground, providing modest increases in effective soil pressure acting on the lining. Based on experience examining the differences in the results from beam spring models on the Blue Plains Tunnel,\(^1\) and recognizing the difference in the size of the tunnel, the increase in ground loads would likely be of the order of 20%.

The steel dowels are fully embedded in plastic, providing reliable long-term durability. Additionally, dowels require no pockets on the inside face of the lining and, therefore, no pocket filling is required.

**Disadvantages**

The number and size of dowels is higher than would be expected for a conventional tunnel of this type and this will increase construction costs, probably of the order of $200 per foot for the 40-foot-diameter tunnel for the numbers presented in Table 2.

If higher pressures than designed for are encountered, the dowels would restrain further movement but the tunnel would lose its sealing capacity for the duration of the increased load. However, the load displacement plot for the dowels shows that while the capacity of the dowels is at least twice the working load, the linear portion of the plot only extends to the working loads. Therefore, if the quoted loads are exceeded at least some of the displacement will be unrecoverable, and this could impair the performance of the system to resist the normal operational pressure thereafter.

The lining will be subject to significantly increased tensile stresses in the vicinity of the joints of adjacent rings, which could require additional reinforcement.

### 3.4 Systematic Secondary Grouting

Systematic secondary grouting involves a second grouting activity that is employed at a distance back from the primary grouting (which is expected to be through the TBM tail shield). The application of the secondary grouting ensures a minimum effective soil pressure is mobilized. It would be performed at a minimum distance of 10 to 20 rings from the tail shield to ensure that the primary grout has achieved sufficient strength prior to the secondary grouting activity.

---

\(^1\) Result with shear springs were examined as part of the design process, but were neither reported nor presented in the actual calculations for Blue Plains Tunnel as the beneficial effect was not required to justify the design.
Grouting near the tail shield carries the risk of grout filling the annulus between the ground and the shield, grouting the machine in. By moving the activity further from the shield, higher grout pressures can be employed, which can increase the effectiveness of this measure.

Based on experience with instrumented rings, tail shield grouting can lock in pressures comparable to the grout pressures. However, it is not clear that achieving the full grout pressure can be relied upon in all ground conditions, therefore, higher grout pressures may be required in some ground types.

**Advantages**

This measure secures adequate load in the lining to ensure that the internal pressure can be resisted without the ring going into tension. No modifications to the lining design or special connection features are required.

The grouting process is a separate process from the ring build and, therefore, doesn’t impact build time (and hence TBM progress).

**Disadvantages**

Instances of grout pressures that are actually higher than full overburden pressure have been generated in linings in soft ground, albeit at much shallower depths than the North and Main tunnels. However, it is not clear that it will be easy to achieve high grout pressures in all ground conditions. The fact that compensation grouting activities (which have similar aims and processes) have been successfully used in a variety of soils from sands to clays suggests that while a grouting solution exists, it may prove difficult to achieve in practice. If relied upon with no alternative or backup, this uncertainty would constitute an unacceptable risk to the project.

The application of a grouting activity at every grout port requires a special gantry toward the back of the TBM, or a gantry that runs independently just behind the TBM backup train. A separate grout batching plant may also be required. The activity and additional equipment would add cost to the project.

While grout sockets are specified with hydrophilic gaskets and sealing caps, they often leak despite all reasonable measures being taken. The systematic grouting uses all or most of the grout sockets, so such leakage will be prevalent throughout the tunnel.

Similar to the ground loading, instrumentation is likely to be required to demonstrate that the applied grout pressures are sufficient to generate the required load in the lining.

Ground heave could result if very high grout pressures are employed, although it appears unlikely that sufficiently high pressures would be required.

### 3.5 Post-Tensioning

In this system of mitigation, the segments are provided with one or more ducts that fit together to provide continuous ducts around the ring. After erection, post tension cables are inserted and stressed to a defined load. Stressing usually occurs within a “pocket” in the lining that is grouted after completion of the post-tensioning operation, providing a smooth finish and providing corrosion protection to the anchorage area.

**Observations**

Post-tensioning of tunnel linings has a reasonably long history albeit not widespread use. The projects listed in Table 4 were outlined by Swanson (1981). These projects employed a cast-in situ lining, rather than a segmental lining, nevertheless they demonstrate the longevity of the solution. More recently, this technology has been applied in the Thun Tunnel in Switzerland (Kohler and Rupp, 2008).
TABLE 4  
Post-tensioned Tunnel Lining Projects (after Swanson, 1981, and Kohler and Rupp, 2008)

<table>
<thead>
<tr>
<th>Project</th>
<th>Date</th>
<th>Location</th>
<th>Lining Type</th>
<th>Internal Pressure</th>
<th>Length</th>
<th>Internal Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piastra Andonno</td>
<td>1974</td>
<td>Italy</td>
<td>In situ</td>
<td>260 ft</td>
<td>37,400 ft</td>
<td>10.8 ft</td>
</tr>
<tr>
<td>Taloro</td>
<td>1976</td>
<td>Italy</td>
<td>In situ</td>
<td>300 ft</td>
<td>1,620 ft</td>
<td>18.0 ft</td>
</tr>
<tr>
<td>Grimsel Head-race Tunnel</td>
<td>1977</td>
<td>Switzerland</td>
<td>In situ</td>
<td>250 ft</td>
<td>650 ft</td>
<td>22.3 ft</td>
</tr>
<tr>
<td>Grimsel Tail-race Tunnel</td>
<td>1977</td>
<td>Switzerland</td>
<td>In situ</td>
<td>460 ft</td>
<td>200 ft</td>
<td>22.3 ft</td>
</tr>
<tr>
<td>Thun Tunnel</td>
<td>2007</td>
<td>Switzerland</td>
<td>Segmental</td>
<td>Not specified</td>
<td>3,940 ft</td>
<td>17.7 ft</td>
</tr>
</tbody>
</table>

The installation of the post-tensioning system can be undertaken in the ring build area or further back. If the installation occurs in the shield, it is recommended that the stressing be undertaken further back. It is commonly observed that bolts on joints loosen as the ring passes out of the back of the tail shield due to movement of the ring. The equivalent phenomenon for a post-tensioned cable would be alleviation of load. Therefore, a final stressing at some distance back from the shield would be required.

Due to the requirement to roll the ring to follow the alignment, the post-tensioning pockets could be at any of a number of locations around the circle. Therefore, a special gantry would be required to provide access for the installation, stressing, and grouting exercises. Two gantries may be required if the grouting is performed separately.

Securing durability is problematic for post-tensioned structures and there have been instances of failure of post-tensioned concrete structures, mainly due to poor grout encapsulation. Given the extreme difficulty of accessing and inspecting the cables, it may be necessary to employ a three-lines-of-defense approach. This could employ a continuous duct installed inside the discontinuous ducts within the segments, along with coated cables. If the lining was also double-gasketed, with gaskets on both the inside and outside of the joint to prevent water ingress, then this would provide the three layers of protection.

Durability could be secured by using a glass reinforced polymer (GRP) product. However, it tends to be less flexible at the strengths required, making installation more difficult. Furthermore, creep characteristics would have to be very well understood to assure a 100-year design life.

Advantages

Post-tensioning has the benefit of providing a stiffer ring and increased moment resistance at low loads. Furthermore, the ring does not go into tension under internal pressures, and therefore it may be possible to reduce reinforcement levels.

Disadvantages

The following elements will add cost to the system:

- Material costs of ducts and post-tensioning equipment
- Cost of placing ducts in the segments
- Cost of installing, stressing, and grouting in post-tensioning cables
- Gantries and equipment for installing, tensioning, and grouting

While there is precedent for this solution, it has not been used for a tunnel of this size. It has also not been used for a tunnel with a 100-year design life. This second point is important as it may be difficult to secure a durable solution.
3.6 Alternative Joint Connector

CH2M HILL has been involved in the development of a non-ferrous alternative to steel bolts on the longitudinal bolt. It is a non-ferrous (non-corrosive) connector that spans the joint and provides mechanical restraint against its opening. It is installed as the segments are erected, does not have a negative impact on build time, and has the potential to improve build accuracy.

The technology is currently in the early stages of prototyping so it is not possible to reveal details at this time. However, details could be released once patents are filed by the manufacturer, which is expected to occur by December 2014. It is reasonable to expect that a fully tested system could be available in plenty of time for finalization of design.

Advantages

The connector provides direct tension resistance across the joint. It does not require any specialist equipment or installation activities.

Disadvantages

The system has not been used before in the U.S. and contractors will have a learning curve as they familiarize themselves with the technology. It is reasonable to expect that this impact will be limited to the first few weeks and months of tunneling, and therefore only a small impact on the overall tunnel will result.

3.7 Ferrous Push-fit Solutions from Japan

A number of ferrous solutions that provide tension resistance to linings have been developed. Many of these systems are either only available in Japan and a limited number of nearby countries. A number of potentially suitable systems have been identified.

The original developers were contacted by CH2M HILL for information. Many of the manufacturers do not operate outside Japan and will not provide support for their use in the U.S. market. Therefore, if any of the solutions were to be adopted, they would require development from concept through prototyping to implementation, so up-front investment will be required to prove the design including trials and testing.

We are still awaiting responses from some of the manufacturers. Patent issues have not been researched at this stage but would have to be considered if any of these concepts were to be carried forward.

Description of systems

Table 5 provides a description of the systems identified as potentially suitable. All images are taken from Koizumi (2000).
### TABLE 5

#### Description of Ferrous Push-fit Solutions

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>One Pass NM segments</strong></td>
<td>Machined steel pieces that fit together on insertion to provide a tight fit. They were originally conceived as part of hybrid segments comprising all-round steel structure with concrete infill. However, they could be employed in conventional reinforced concrete segments. Careful detailing could ensure that the required tensile capacity is provided with minimal impact on joint bearing surface, so the ability of the lining to take compression loads would not be compromised.</td>
</tr>
<tr>
<td><strong>Guide lock segment</strong></td>
<td>The guide lock system provides strong tensile connection across the joints. It consists of an anchored T-shaped piece that secures inside a C-shaped steel piece on the opposite segment. A key feature of this system is that it is employed in a hexagonal lining style system, where the segments are effectively offset from each other by half a ring. In this configuration half the segments of the ring may be placed and are available for thrusting off by the TBM rams. The remaining segments are then placed half a ring length closer to the face, and can be placed while the TBM is advancing and thrusting off the segments already placed. Thus continuous mining is possible without having to stop for ring erection. The system could be designed to take significant loads, as it runs the length of the joint. By designing the steel components to take compression as well as tension, problems with the reduced joint contact area could be avoided. Adapting this technology into a conventional segmentation would be difficult due to the high joint angles employed on the key.</td>
</tr>
<tr>
<td><strong>Honeycomb segment connector</strong></td>
<td>This system is envisaged with a true hexagonal lining. The “male” steel connection pieces secure into the “female” pieces once the adjacent segments have been placed. The connections assist build control/guidance and also provide a means for tension to be resisted. The machined steel pieces would be secured into the molds at the corners. As with the guide lock segments, continuous mining is possible with this arrangement.</td>
</tr>
</tbody>
</table>
Past uses of these systems are described in Table 6.

TABLE 6
Past Use of Solutions

<table>
<thead>
<tr>
<th>System</th>
<th>Project Name</th>
<th>Year</th>
<th>Internal Diameter</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>One Pass NM segments</td>
<td>Tokyo Metro Oedo Line</td>
<td>1999</td>
<td>22.3 ft</td>
<td>Rail</td>
</tr>
<tr>
<td></td>
<td>Kamidagawa No.7 Ring Line</td>
<td>2004</td>
<td>43.3 ft</td>
<td>Sewer storage</td>
</tr>
<tr>
<td>Honeycomb segment connector</td>
<td>Tokyo Central Circular Route</td>
<td>2003</td>
<td>38.7 ft</td>
<td>Road</td>
</tr>
<tr>
<td>Guide lock segment</td>
<td>Fukuoka Sewer</td>
<td>1998</td>
<td>11.0 ft</td>
<td>Sewer</td>
</tr>
</tbody>
</table>

Sources: Koizumi, 2000; Nippon Steel & Sumitomo Metal, 2014; and Okumura, 2014.

While other examples might exist, the study has focused on establishing use on at least one project. All the systems mentioned are currently marketed in Japan, but efforts to engage with suppliers have met with resistance, as the suppliers do not currently market outside Japan. The lack of engagement of the original developer would mean that if any of these systems were to be developed for use on the Bay Delta project then the development would start from scratch. Care would also be required to ensure that patents were not violated.

Advantages and Disadvantages

Table 7 lists the advantages and disadvantages of ferrous push fit solutions.

TABLE 7
Advantages and Disadvantages of Ferrous Push fit Solutions

<table>
<thead>
<tr>
<th>System</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>All systems</td>
<td>Tensile capacity can be achieved with very low levels of joint opening (probably less than 0.5 mm)</td>
<td>Connectors are steel and will require protection from the effects of corrosion. This could be mitigated by double gasketing or detailing with the ability to inject grout around the fittings. Systems have never been used in the U.S. so up-front investment will be required to prove the design including trials and testing. There will also be a learning curve during implementation. Likely to be much more expensive than standard lining connection systems.</td>
</tr>
<tr>
<td>One Pass NM segments</td>
<td>Relatively small steel pieces that could easily be fixed to the mold. Will assist with build accuracy. Will not significantly impact the load carrying capacity of the joints.</td>
<td>Technology is not supported in the U.S., so development would be required.</td>
</tr>
<tr>
<td>Guide lock segment</td>
<td>Will assist with build accuracy. Provide schedule benefits from continuous mining.</td>
<td>Awaiting feedback from developer. Lack of support and patenting issues possible.</td>
</tr>
<tr>
<td>Honeycomb segment connector</td>
<td>Will assist with build accuracy. Provide schedule benefits from continuous mining.</td>
<td>Technology is not supported in the U.S., so development would be required.</td>
</tr>
</tbody>
</table>
Overall, the One Pass NM segment system would be the easiest to implement as it fits with existing segment geometries. The other two options might be considered due to the schedule benefits they could provide but would require buy-in from the contractor to be successful.

### 3.8 Castellated Joint

This is another system that has been developed in Japan. It has been used on at least one project: Chiba Shibaura Water Treatment Plant Outfall, a 12.8-foot sewer tunnel constructed in 2001. This project was subjected to an internal pressure of 3 bar.

The solution consists of a specially profiled joint as shown in Figures 3 and 4.

**Figure 3. Typical Segment for ‘Ring lock’ Castellated System**

**Figure 4. Tensile Resisting Mechanism for ‘Ring lock’ Castellated System**

The ring acts to transfer the tension load into the adjacent rings via the bearing surfaces of the profiled joint. In principle, the mechanism is similar to the dowels, except that the compression of the concrete provides a much stiffer resistance.
If the bearing surfaces come into contact during installation they could be damaged or even impede accurate build. For this reason they are detailed with a small gap. To prevent these gaps resulting in joints opening under internal pressure cases, the segments are detailed with small recesses that permit the injection of high-strength grout into the space between the bearing surfaces. This ensures that the linings can be built to practical tolerances, and will suffer minimal joint opening under internal pressure.

**Observations**

As with the ferrous connection systems, the suppliers of this solution do not market it outside Japan. Patent issues would need to be managed carefully.

This system could be detailed to be similar to lining systems that are currently employed in the U.S. However, the profiled joint could create issues with the TBM ram pad interface. Ideally the protrusions would be located between the ram pads, but this means that the pad would span over the recess and bear on the concrete on either side. This is not a situation that is routinely designed for so it would be advisable to test the capacity of this arrangement. In a worst-case scenario, a slightly higher segment thickness may be required to provide the required capacity.

Careful design of the profiled joint will be required to ensure that the protrusions are adequately reinforced. Furthermore, careful detailing will be required to ensure that the protrusions or recesses are not prone to damage on installation.

**Advantages**

This system requires no special connection features and can readily provide at least 1 bar net outward pressure.

Substantial guidance for the design and execution of this method exists and is in the public domain (albeit in Japanese).

**Disadvantages**

The grouting activity would result in increased project costs.

Trialing of the grout system would be required to verify that the whole bearing area is reliably grouted.

Testing of TBM ram loads will probably be required to verify that the ring has adequate capacity.

In a worst-case scenario, segment thickness would have to increase to provide adequate bearing surface for TBM ram loads.

Stress concentrations could lead to increased reinforcement requirements as per the dowel solution.

### 3.9 Summary and Recommended Option

The overall advantages and disadvantages of the different systems are summarized in Table 8. All options have no impact on the build time unless otherwise noted.

**TABLE 8**

<table>
<thead>
<tr>
<th>System</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reliance on ground loads</td>
<td>Cost savings could be substantial if no other mitigations are required</td>
<td>Not straightforward to calculate.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>May not be sufficient to resist tension.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Requires detailed ground investigation to evaluate</td>
</tr>
</tbody>
</table>

May

Requires
ted
depth
to
evaluated

The

would

Testing

In

Stress

could

as

reinforcement

requirements

as

the

the
do
to

the

the
TABLE 8
Advantages and Disadvantages of Potential Solutions

<table>
<thead>
<tr>
<th>System</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowels to resist tension</td>
<td>Assists with build.</td>
<td>Higher number of dowels – cost impact</td>
</tr>
<tr>
<td></td>
<td>Will allow some mobilization of ground pressure</td>
<td>Plastic behavior at higher loads compromises system if encountered</td>
</tr>
<tr>
<td></td>
<td>No durability issues</td>
<td>Higher reinforcement may be required in some areas</td>
</tr>
<tr>
<td></td>
<td>No sockets on inside face</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Can be omitted if not required</td>
<td></td>
</tr>
<tr>
<td>Systematic secondary grouting</td>
<td>Ensures no tension in the ring</td>
<td>Additional costs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Minor increase in leakage</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Potential for heave needs to be managed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>No precedent for actively generating the pressure</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Locking in sufficient pressure may prove unachievable</td>
</tr>
<tr>
<td>Post-tensioning</td>
<td>Provides stiffer ring.</td>
<td>Increased costs from installation</td>
</tr>
<tr>
<td></td>
<td>May be possible to reduce reinforcement</td>
<td>Lack of precedent at this size</td>
</tr>
<tr>
<td></td>
<td>Can be omitted if not required</td>
<td>Durability could be difficult to secure</td>
</tr>
<tr>
<td>Alternative joint connector</td>
<td>Direct resistance across the joint</td>
<td>Not used before</td>
</tr>
<tr>
<td></td>
<td>No specialist equipment/activities</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Can be omitted if not required</td>
<td></td>
</tr>
<tr>
<td>Ferrous push-fit solutions</td>
<td>Resists tension with low/no joint opening</td>
<td>Expensive</td>
</tr>
<tr>
<td></td>
<td>Assist with build</td>
<td>Limited previous application and support</td>
</tr>
<tr>
<td></td>
<td>Some types allow continuous mining.</td>
<td>Potential patent issues</td>
</tr>
<tr>
<td>Castellated joint</td>
<td>Resists tension with low/no joint opening</td>
<td>Cost of grouting</td>
</tr>
<tr>
<td></td>
<td>Provides full resistance to pressure</td>
<td>Trialing required</td>
</tr>
<tr>
<td></td>
<td>Design and execution guidance is available</td>
<td>Testing for TBM ram loads required</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Could require increased thickness.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Additional reinforcement may be required</td>
</tr>
</tbody>
</table>

The above systems could all be used in isolation to achieve resistance to internal pressures. However, it is likely that the most effective solution will involve a combination of at least two of the solutions. The collection of detailed ground investigation data will be required for the detailed design. Therefore, it is recommended that these data be used to establish a robust lower-bound ground pressure in the first instance. This will provide a reduced net internal pressure for which one or more solutions may be employed to resist.

Of the alternatives, the ferrous systems (connectors and post-tensioning) are subject to durability risk, while the secondary grouting system has some uncertainties over its effectiveness in varied ground types. The dowels have the advantage of being familiar to U.S. contractors, so build issues are unlikely. They also have the advantage that they can be omitted to save cost if they prove not to be needed along any given reach of the tunnel. This is also true of the alternative connector.

The castellated joint option provides a stiffer connection than the dowels or alternative connector. This avoids unforeseen effects of repeatedly allowing the lining to expand (by up to 10 mm in the case of the dowels). The primary risks associated with this kind of lining is the risk of poor grouting on the bearing area.
and the risks associated with the TBM ramps acting on the edges with the recesses. These risks could be eliminated by some early design and testing work.

For this reason it is recommended that the castellated joint option be investigated prior to finalizing the design, with a backup option of ground loads and dowels or alternative joint connectors.

In the meantime, the other options should be investigated in more detail to see if they are feasible. This would provide the knowledge to write a robust performance specification if the contractor could see a more effective way of delivering options other than the preferred.

Finally, the options presented herein are not an exhaustive list of all possibilities and there is almost certainly scope for contractor’s innovation to provide a different solution. By providing a robust solution alongside a performance specification for contractor-designed alternatives, it may be possible to secure a better solution.

### 4.0 Conceptual Design

The shear dowel system for the main (40-foot) tunnel is shown in the attached drawings. It takes the base case described in Section 3.1 and applies the outline design for the dowel system as described in Section 3.3.

Where design parameters have not been available, moderately conservative assumptions have been made based on available data and engineering judgment based on past experience, including soil in situ stress parameters and stiffness, and TBM thrusts.

The general arrangement drawing shows the overall arrangement with the locations of the shear dowels. Erector cone recesses and bolt pockets and associated reinforcement detailing have been omitted from the reinforcement drawing for clarity.

### 5.0 Conclusions

This technical memorandum proposes other tension-resisting alternatives additional to the bolted joint detail as shown in the Conceptual Engineering Report (MWD, 2014). The following options have been presented:

- Reliance on ground loads
- Use of dowels to resist tension
- Systematic secondary grouting
- Alternative non-ferrous joint connector
- Post-tensioning
- Ferrous push-fit solutions from Japan
- Castellated joint

The advantages and disadvantages of each are presented. The recommended solution is to establish a robust lower-bound for the ground load on the lining (along with appropriate factors of safety), and then employ alternatives to resist the remaining net internal pressure.

Of the solutions considered, the use of shear dowels is recommended for incorporation into conceptual design as it is a technology that is familiar to U.S. tunnel contractors, permits fast and accurate ring erection, and requires no secondary activities in the tunnel. A conceptual design of the segmental lining with the shear dowel solution for the main (40ft) tunnel is presented in the attached drawings.

### 6.0 References


**ELEVATION ON CIRCUMFERENTIAL JOINT**

- **#6 Closed Ties Bursting Reinforcement**: 26 Each End
- **#6 U-Bars, 32 EA Each End**
- **#8 U-Bars at 6" Spacing**
- **#6 Bars, 32**
- **HOR. #8 Bars at 6" Spacing**
- **Bursting Reinforcement**: 26 Each End
- **#6 Bars, 32**

**PLAN ON INTRADOS**

**40' ID PRECAST TUNNEL LINING REINFORCEMENT**

[Diagram of Reinforcement Details]
1 Introduction

The California WaterFix program has been developed to stabilize water deliveries from the Sacramento – San Joaquin River Delta (Delta) area. The program utilizes a twin tunnel concept to meet the purpose and need of the program. A tunnel leakage study was performed in 2012, but since that time the design concept for the project has been changed, with the pumps being moved from the intakes to the terminus of the project, which significantly impacts the water pressure in the tunnels and consequently the potential leakage rate. Metropolitan Water District (MWD) has requested that Arup North America, Ltd. (Arup) evaluate the impact of design updates to the tunnel alignment, elevation and internal pressure on water seepage rates from the tunnel to the surrounding ground. MWD has authorized this evaluation in Task Order Number 5, signed on October 13th, 2016, as part of Agreement Number 143949. The scope of work includes the following:

- Review and compilation of estimated relevant external pressures (along tunnel alignment);
- Review and compilation of estimated internal tunnel pressures;
- Estimation of micro-cracking in the segmental lining and resulting permeability of the lining;
- Review of Jacobs Associates report dated July 10th 2012; and
- Estimate of water seepage rates from the tunnel to the surrounding ground.

The following information has formed the basis for this review:

2 Project Background

As part of the California WaterFix program, new twin tunnels and intakes will be constructed to stabilize water deliveries from the Delta area. The invert of the twin tunnels will be up to 175 feet below ground level (ft bgl) with a length of around 30.15 miles per tunnel and will be excavated using Tunnel Boring Machines (TBMs). The North Tunnels will transport water from the intakes to the intermediate forebay, and total 13.5 miles in length. Water will then be transported from the intermediate forebay via the Main Tunnels to Clifton Court Forebay. The tunnel construction will also be supported by the construction of fifteen shafts and a number of safe haven work areas (i.e. subsurface evacuation areas).\(^1\)

The tunnel alignment consists of:

- **The North Tunnels**, which include three separate tunnel lengths:
  - North Tunnel, Reach 1 – 1.99 miles of single 28-foot-ID tunnel between Intake No.2 and Intake No.3 (Stations \(^2\) 3350-3450)
  - North Tunnel, Reach 2 – 6.74 miles of single 40-foot-ID tunnel between Intake No.3 and the Intermediate Forebay (Stations 3000-3350).
  - North Tunnel, Reach 3 – 4.77 miles of single 28-foot-ID tunnel between Intake No.5 and the Intermediate Forebay (Stations 5000-5250).
- **The Main Tunnels** - 30.2 miles of twin 40-foot-ID tunnels between the Intermediate Forebay (Reach 4, Station 0) and Clifton Court Forebay Pumping Plant (Reach 7, Station 1592).

In 2012, Jacobs Associates produced estimates of tunnel leakage rates. At the time the Jacobs Associates report was produced, the program configuration consisted of pumping plants located at each of the three river intakes, which lifted water from the Sacramento River into the tunnel system. Tunnel diameters were sized based on this arrangement. This concept was subsequently revised such that water will now flow by gravity from the Sacramento River through the tunnel system to a pumping facility located near the project terminus at Clifton Court.

The revised design is analyzed in this memorandum which provides updated estimates of leakage relevant to the current design, which is shown in Figure 1. Figure 2 illustrates the differences between the previous and current designs. The key changes from 2012 to the current design with respect to the potential for leakage are the revised locations of the pumping facilities and the resulting lower internal pressure, the shallower tunnel depth and the change in the assumed elevation of groundwater from five feet below ground level to elevation 0ft. As a result, for the current design, there are significant lengths where the groundwater pressure is now higher than the internal tunnel pressure, and inflow rates have been estimated at these locations.

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\(^1\) Number of shafts is based on design progress as of July 1 2015.

\(^2\) Stationing along the alignment is provided in hundreds of feet, i.e. Station 1600 is 160,000 ft along the alignment.
Figure 1. Overall Hydraulic Plan and Profile Along the Alignment.
Figure 2. Sketches of a) Current Design, and b) Previous 2012 Design (Not to scale).
Technical Memorandum

237477 February 6, 2017

3 Hydrogeological Setting

The tunnel alignment lies within an agricultural area where groundwater is managed in order to maintain the groundwater level below crop root zones. For the purposes of this analysis, groundwater has been taken at an elevation of 0 ft, to be consistent with the CER\(^3\).

Based on the current proposed tunnel alignment, the depth of the tunnels is between about 123 and 175 ft bgl. Soils widely range through the alignment and include both low permeability clays as well as sands and gravels (which are likely to have much higher permeability). Geotechnical boreholes and Cone Penetrometer Testing (CPT) boreholes provide limited information on the types of materials to be encountered in the tunnel. Based on a review of the available data from the GDR and comparison with the current tunnel alignment drawings the following is noted:

- The current alignment closely follows the alignment which was considered in the GDR for the southern two-thirds of the Main Tunnels (Reaches 5, 6 and 7, between stations 600 and 1400) and thus existing geotechnical data is representative of subsurface conditions which will be encountered during tunneling.
- Between stations 0 and 600 (Reach 4) and between station 1400 (Reach 7) and the southern end of the Main Tunnels alignment (at around station 1592), the existing geotechnical data is offset significantly. Across these lengths the representativeness of the existing geotechnical data from the GDR is unclear.
- Geotechnical data associated with North Tunnels (north of the Intermediate Forebay) is focused on intake areas rather than the tunnel alignment.

In general, the current proposed tunnel alignment (where data is representative) will encounter mostly low permeability materials (clays and silts). However there will be lenses of sands and gravels encountered across the alignment. It is unclear from the available information how many of the sands and gravels have aquifer potential and how many are discontinuous lenses with no significance for water supply. However given the alluvial nature of the local geology, it may be expected that there will be some sands and gravels which act as aquifers through which groundwater flows.

\(^3\) “For the net internal pressure design of the liner during conceptual phase, the external ground water pressure is assumed to be at elevation 0.0 (MSL) along the majority of the alignment. Occasionally, lower ground water elevation may occur due to local conditions”. CER Volume 1, 11.2.6.
4 Estimation of Tunnel Inflow and Leakage Rates

Inflow and Leakage rates along the tunnel alignment are estimated as shown in Figure 3 through Figure 5 and Figure 6 (in Section 5). These estimates are based on the following assumptions:

- Ground surface along the alignment has been derived from the tunnel alignment drawings. Groundwater is assumed to be at elevation 0.0 ft (MSL) along the alignment to be consistent with the CER. In some cases, ground surface is below elevation 0.0 ft, so this assumption may result in a minor underestimate of leakage rates.

- The tunnel invert varies between about 123 and 175 ft bgl (down to -163 ft elevation) for the Main Tunnels, The proposed tunnel profile range is shown on Sheet Nos. CCO-C-1035TN to 1038TN (CER). Variation of the tunnel profile within this range has no noticeable impact on the calculated leakage rates.

- The internal pressure head in the tunnel in the updated design is lower than the previous design and varies from 120 ft to 154 ft relative to the tunnel invert. Internal tunnel pressures are reported along the alignment based on Sheet Nos. CCO-C-4005HH to 4007HH of CER.

- The maximum differential water pressure is expected to range from a minimum of -9 ft (where the external water pressure is greater than internal) to a maximum of +9 ft (where the internal water pressure is greater than external). The range of pressures along the alignment is evenly spread between these two values.

- The estimated inflow and leakage rates documented in this memorandum are the long term steady-state flow rates during operation of the entire tunnel system (both twin tunnels filled), and not irregular, transient, or periodic peak leakage rates.

- Leakage rates are estimated according to the framework proposed by Fernandez (1994). Nomogram 1 in the Fernandez paper is used to estimate leakage and pore pressure outside pressurized tunnel liners. A base case liner permeability of $10^{-6}$ cm/s ($10^{-8}$ m/s) was adopted in accordance with the assessment by Fernandez. This liner permeability was selected since liner strains ($\varepsilon_L$) along the alignment are less than $1.5 \times 10^{-4}$ at all locations and thus micro cracking is not likely to take place.

- To account for the potential variability of liner permeability due to leakage through the gaskets, a sensitivity analysis was undertaken to envelope a lower and upper range of leakage rates; where the upper estimate is assumed to be the base value. The liner permeability ranged between $10^{-7}$ cm/s, an order of magnitude lower than the base value and up to the base value. It should be noted, that with a precast concrete segmental lining manufactured in factory conditions, the concrete permeability is likely to be at the lower end of this sensitivity range, but leakage through the gaskets could make the overall permeability closer to the base value assumed. A review of typical inflow specifications for completed tunnels indicates that a permeability equivalent to $10^{-7}$ cm/s is the industry standard;

---

4 Differential water pressures given in feet of water. Negative differential water pressures indicate inflow to the tunnel would be expected to occur, while positive differential water pressures indicate leakage from the tunnel is expected.
even better performance can be achieved with careful attention to segment specification and erection.

- It is expected that segments under compression will exhibit slightly lower permeability than segments under a net tensile load where micro-cracking might occur. However, in the current design, the lining is expected to be under compression, and as noted above, micro-cracking is not anticipated.

- In addition, ground permeability was varied by an order of magnitude (both for higher and lower permeabilities) to capture ground uncertainties.

- Inflow rates have been estimated based on the method given by Goodman (1980) and endorsed by Fernandez (1994). Inflow potentially occurs at locations where the external water head exceeds the internal water pressure in the tunnel (i.e. negative differential water pressure within the tunnel).

- The modulus of elasticity of liner is calculated using ACI 318 ‘Building Code for Structural Concrete’ with an assumed concrete compressive strength of 7,000 psi.

The input parameters assumed in the calculations are presented in Table 1. The results of the sensitivity analysis are presented in Table 2 (in Section 5).

### Table 1. Input Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Value</th>
<th>Unit</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_m$</td>
<td>Ground Permeability (Assumed to be as sand)</td>
<td>$9.8 \times 10^{-5}$</td>
<td>ft/sec</td>
<td>Sensitivity analysis included an order of magnitude lower and greater than the assumed ground permeability.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>($2.8 \times 10^{-5}$)</td>
<td>(m/s)</td>
<td></td>
</tr>
<tr>
<td>$k_L$</td>
<td>Liner Permeability</td>
<td>$3.3 \times 10^{-8}$</td>
<td>ft/sec</td>
<td>Based on $e_\phi &lt; 1.5 \times 10^{-4}$ (Fernandez, 1994)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>($1.0 \times 10^{-8}$)</td>
<td>(m/s)</td>
<td></td>
</tr>
<tr>
<td>$r_p^O$</td>
<td>Tunnel outside radius</td>
<td>15.3, 21.7</td>
<td>ft</td>
<td>For internal diameters (ID) of 28 and 40’, respectively.</td>
</tr>
<tr>
<td>$r_p^i$</td>
<td>Tunnel inside radius</td>
<td>14, 20</td>
<td>ft</td>
<td>For ID of 28 and 40’, respectively.</td>
</tr>
<tr>
<td>$th$</td>
<td>Tunnel liner thickness</td>
<td>1.3, 1.7</td>
<td>ft</td>
<td>For ID of 28 and 40’, respectively.</td>
</tr>
<tr>
<td>$a$</td>
<td>Radius to liner center</td>
<td>14.7, 20.8</td>
<td>ft</td>
<td>For ID of 28 and 40’, respectively.</td>
</tr>
<tr>
<td>$E_m$</td>
<td>Modulus of Elasticity of Ground</td>
<td>4.5 ($6.48 \times 10^5$)</td>
<td>ksi</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>($6.9 \times 10^5$)</td>
<td>(lb/ft²)</td>
<td></td>
</tr>
<tr>
<td>$E_L$</td>
<td>Modulus of Elasticity of Liner</td>
<td>4.770 ($6.9 \times 10^8$)</td>
<td>ksi</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>($6.9 \times 10^8$)</td>
<td>(lb/ft²)</td>
<td></td>
</tr>
<tr>
<td>$\nu_m$</td>
<td>Poisson Ratio of Ground</td>
<td>0.30</td>
<td>unitless</td>
<td></td>
</tr>
<tr>
<td>$\nu_L$</td>
<td>Poisson Ratio of Liner</td>
<td>0.20</td>
<td>unitless</td>
<td></td>
</tr>
<tr>
<td>$\nu_w$</td>
<td>Specific Weight of Water</td>
<td>62.4</td>
<td>lbf/ft³</td>
<td></td>
</tr>
</tbody>
</table>

During construction, or if the tunnel is unwatered for future maintenance, the gaskets between the segmental lining will avoid the flow of any small particles into the tunnel. This situation is similar to a typical transportation tunnel, and there is a long track record of large diameter tunnels in similar ground conditions.
conditions that do not see around loss of ground. A typical specification is to require no visible water inflow above springline, and damp patches only (no flowing water) below springline.

Figure 3. Estimated Tunnel Inflow and Leakage Rates – North Tunnel Reach 1 and Reach 2.
Figure 4. Estimated Tunnel Inflow and Leakage Rates – North Tunnel Reach 3.
Figure 5. Estimates of Tunnel Inflow and Leakage Rates – Main Tunnels (Reaches 4, 5, 6, 7) per Tunnel.
Table 2. Estimated Total Inflow (-) and Leakage Rates (+) for Current Design

<table>
<thead>
<tr>
<th>Tunnel Length (feet)</th>
<th>Tunnel Configuration</th>
<th>Estimated Inflow (cfs)</th>
<th>Estimated Leakage (cfs)</th>
<th>Net Estimated Leakage Whole Tunnel (cfs)</th>
<th>Estimated Leakage Sensitivity Analysis** Whole Tunnel (cfs)</th>
<th>2012 Design Alternative A1 (cfs)* (for comparison)</th>
</tr>
</thead>
<tbody>
<tr>
<td>46,100</td>
<td>28’ &amp; 40’ ID North Tunnel Reaches 1 &amp; 2</td>
<td>0.00</td>
<td>+0.5</td>
<td>+0.5</td>
<td>+0.1 to +0.5</td>
<td>Not assessed</td>
</tr>
<tr>
<td>25,200</td>
<td>28’ ID North Tunnel Reach 3</td>
<td>0.00</td>
<td>+0.2</td>
<td>+0.2</td>
<td>0.0 to +0.2</td>
<td>Not assessed</td>
</tr>
<tr>
<td>2 x 159,200</td>
<td>Two 40’ ID Main Tunnels Reaches 4 to 7</td>
<td>-3.7</td>
<td>0.00</td>
<td>-3.7</td>
<td>-0.4 to -3.7</td>
<td>2827</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>-3.7</td>
<td>0.7</td>
<td>-3.0</td>
<td>-0.30 to -3.0</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
The lengths of tunnels where the internal water pressure is lower than the external groundwater pressure will have inflow, and where the internal water pressure is higher than the external pressure the tunnel will leak. So overall, for all the tunnels taken together (reaches 1 to 7), the sections subject to inflow will have a total inflow of -3.7 cfs, and the sections subject to leakage will have a total leakage of 0.7 cfs; overall the entire system has an inflow of 3 cfs.

*Alternative A1 refers to one-pass precast concrete segmental lining with tension reinforcement. The leakage rates have been calculated by taking an average of the estimated leakage rates from both the concrete liner and the gaskets, and applying over a 60.3 miles length for twin tunnels.

** The sensitivity analysis looked at concrete permeabilities in the range of $10^{-5}$ cm/s to $10^{-6}$ cm/s and ground permeabilities one order of magnitude above and below the base assumption.
Comparison with 2012 Leakage Estimates

A comparison between the assumptions and estimation of the leakage rates for the 2012 design study and the current analysis is presented in Table 3 and the resultant leakage rates are presented as a function of differential head in Figure 6.

Table 3. Comparison of assumptions between current and previous designs

<table>
<thead>
<tr>
<th></th>
<th>2012 Estimate</th>
<th>Current Estimate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Characterization</td>
<td>Sand</td>
<td>Sand</td>
</tr>
<tr>
<td>Tunnel Diameter</td>
<td>40 ft</td>
<td>40 ft and 28 ft</td>
</tr>
<tr>
<td>Tunnel Invert Depth Range below Ground Surface</td>
<td>160 ft to 175 ft</td>
<td>123 ft to 175 ft</td>
</tr>
<tr>
<td>Groundwater</td>
<td>5 bgs</td>
<td>0 bgs</td>
</tr>
<tr>
<td>Internal Design Pressure Head in the Tunnel</td>
<td>194 ft to 205 ft</td>
<td>120 ft to 154 ft</td>
</tr>
<tr>
<td>Maximum differential water pressure</td>
<td>50 ft</td>
<td>9 ft</td>
</tr>
<tr>
<td>Radial (Longitudinal) Cracking of the Lining</td>
<td>Liner strains ($\epsilon_{l}^{0}$) &gt;1.5x10^{-4}, so increased liner permeability was assumed, based on Fernandez (1994)</td>
<td>Liner strains ($\epsilon_{l}^{0}$)&lt;1.5x10^{-4}. Sensitivity analysis performed with regards to liner permeability to account for leakage through the gaskets and to capture various lining designs</td>
</tr>
<tr>
<td>Lining Permeability</td>
<td>5.3x10^{-5} to 7.3x10^{-5} ft/sec</td>
<td>3.28x10^{-8} ft/sec</td>
</tr>
<tr>
<td>Head Loss Across Lining</td>
<td>4 to 5.5 %</td>
<td>~1 %</td>
</tr>
<tr>
<td>Comparison of Leakage Rates</td>
<td>Lower leakage rates are now estimated, as would be expected due to the lower internal tunnel head pressures.</td>
<td></td>
</tr>
</tbody>
</table>
6 Conclusions

Leakage rates between the ground and the tunnel are sensitive to the differential pressure between the external groundwater system and the internal water pressure. Where the groundwater head is above the pressure head in the tunnel, inflow to the tunnel may occur. This is mainly in sections of the alignment which run beneath areas of relatively higher ground. Where the pressure head in the tunnel exceeds the external head (such as in in valley locations and more low lying topography) leakage may occur.

The permeability of the liner has a significant effect on the calculated leakage rates. Leakage rates are noticeably more sensitive to liner permeability than that of ground permeability. The elevation of the tunnel makes almost no difference to the leakage rates.
Technical Memorandum

February 6, 2017

Under the low differential pressures which will occur with the current alignment and design water pressures, radial cracking is not expected, and the liner permeability will be a function of the concrete, as well as the circumferential and radial joints.

The net load on the segmental lining is anticipated to be external, with the confining pressure of the soil and external groundwater providing a higher load than the internal water pressure. This will maintain the segmental lining in compression (avoiding higher permeability due to micro-cracking) and also keep the gaskets sealed.

The leakage rates estimated based on current design conditions are lower to those estimated for the previous 2012 design. The main differences between the two analyses is the current design internal pressure is less than the previous design (by between 65 to 126 ft. of water). With the current tunnel alignment, hydraulic grade line and an assumed water table elevation of 0.0 ft (MSL), the internal and external water pressures are essentially balanced, which results in essentially no overall leakage. The tunnel leakage rate should be further evaluated in the preliminary and final design stages upon availability of site-specific geotechnical and hydrogeological data.

7 Recommendations

These estimates have been made for average conditions, and are based on generalized values of hydrogeological parameters. The effect of transient or short term fluctuations or spatial variations in both internal and external water pressures should be considered, particularly in relation to the potential for cracking of the lining and also scour of sediment outside the tunnel.

To minimize leakage, there are a number of steps that can be taken, that correlate with good practices taken to limit inflows in segmentally lined tunnels:

- Specification of a high quality concrete, with low permeability. This will also meet the goal of having a concrete with high durability. Measures to achieve this would include use of pozzolans such as fly ash, specification of well graded aggregate, and a low water cement ratio. The permeability can be made a specific contract requirement and be tested for during segment production. This assessment has used $10^{-7}$ cm/s, which is typical but there are examples of projects where with special attention to the concrete $10^{-9}$ cm/s or even $10^{-10}$ cm/s have been specified. This is normally for durability reasons but the impact on leaks will be a reduction, although not as significant as the order in magnitude changes would suggest since most leakage will occur at the joints.

- Careful detailing of inserts, such as the plastic sockets used for grout holes. Leakage around these inserts is a common cause of water flow through a segmental lining. An example of good detailing would be the addition of hydrophilic seals around the outside of the inserts. This is normally considered in design and specification, but not to the extent of testing the components. For tunnels of this length this may be justified and would add assurance that leakage would be minimized.

- Specification (and enforcement) of tight build tolerances for the segmental lining. A well-built lining will have better alignment at gaskets and will avoid concrete damage, which will reduce leakage potential. This varies a lot between projects and contractors, and is likely to have the
biggest impact on leakage. Careful attention to build could reduce the leakage by an order of magnitude compared with this prediction.

- Specific quality control measures that can be taken in the field to ensure good segment build include dimensional checks on each ring assembled, to ensure that the tunnel diameter is within tolerance and that steps and lips between adjacent segments are minimized; and careful development and monitoring of the grout mix, pump pressure and injection volume to ensure complete filling of the annular void with a grout that provide stability to the lining and minimize displacement after erection.

- The gasket details and the segment connections will need to be designed to ensure the gaskets remain adequately compressed when the internal water pressure is applied. The gaskets rely on being compressed together to give a good seal, and a lining such as this with a net low loading will have to rely on a combination of a gasket that requires a low load to maintain closure, and connections that hold the segmental rings together over the life of the project. Adequate annular void grouting will also be essential to provide external restraint. While this is standard practice, this recommendation has been included because this is a fairly unique situation where the filling of the tunnel reduces the gasket compression.

8 References


# Technical Memorandum

237477    February 6, 2017

## DOCUMENT CHECKING

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<thead>
<tr>
<th>Name</th>
<th>Checked by</th>
<th>Approved by</th>
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<tbody>
<tr>
<td>Foteini Vasilikou; Michael Chendorain, PE</td>
<td>Michael Chendorain</td>
<td>Jon Hurt</td>
</tr>
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<thead>
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</tbody>
</table>

[Signature]

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Appendix L
DWR Seismic Design Criteria
Chapter 2: Seismic Design Criteria

Technical Recommendation

DWR, Reviewed By

Supervising Engineer  Date

MWD, Reviewed By

General Design Team Mgr  Date

MWD, Reviewed By

Design Unit Mgr  Date

Management Recommend

DWR, Recommend

Chief Engineer  Date

MWD Recommend

Chief Engineer  Date

Program Advisory Group Recommend

DWR, Recommend

Program Advisory Group  Date

MWD, Recommend

Program Advisory Group  Date
Chapter 2: Seismic Design Criteria

Technical Recommendation

DWR, Reviewed By

MWD, Reviewed By

Supervising Engineer Date

General Design Team Mgr Date

DWR, Reviewed By

MWD, Reviewed By

Principal Engineer Date

Design Unit Mgr Date

Management Recommend

DWR, Recommend

MWD Recommend

Chief Engineer Date

Chief Engineer Date

Program Advisory Group Recommend

DWR, Recommend

MWD, Recommend

Program Advisory Group Date

Program Advisory Group Date
# TABLE OF CONTENTS

**CHAPTER II**  
SEISMIC DESIGN GUIDELINES ........................................................................ II-1

**SECTION 1**  
GENERAL .................................................................................................. II-1

**SECTION 2**  
SEISMIC DESIGN CRITERIA ........................................................................ II-1

**SECTION 3**  
GROUND MOTION DEVELOPMENT ................................................................ II-7

3.1 Subsurface Characterization .................................................................. II-7

3.2 Identification and Characterization of Potential Seismogenic Sources II-8

3.3 Seismic Hazard Analysis Methodology ...................................................... II-8

3.3.1 PROBABILISTIC SEISMIC HAZARD .................................................. II-8

3.3.2 DESIGN GROUND MOTIONS .............................................................. II-9

3.3.3 ATTENUATION RELATIONSHIPS ....................................................... II-9

3.3.4 HORIZONTAL GROUND MOTION ..................................................... II-9

3.3.5 VERTICAL RESPONSE SPECTRA .................................................... II-9

3.3.6 NEAR-FIELD SOURCE EFFECTS ...................................................... II-10

3.3.7 SITE RESPONSE ANALYSIS ............................................................. II-10

3.3.8 DEVELOPMENT OF ACCELERATION TIME HISTORIES ............... II-11

3.3.8.1 Selection of Seed Time Histories .................................................. II-11

3.3.8.2 Scaling the Seed Time Histories to Design Parameters ............... II-12
CHAPTER II  SEISMIC DESIGN GUIDELINES

SECTION 1  GENERAL

The purpose of this chapter is to provide general guidelines regarding the evaluation of seismicity and the development of seismic design criteria for various facilities including pumping plants, embankments, levees, appurtenant works, tunnels, buildings, bridges and other project features. As the facilities are located in a seismically active area, site-specific seismic hazard assessments shall be performed for all the facilities. The evaluation of potential geologic and seismic hazards, including ground shaking, ground rupture, faulting, slope movement, liquefaction and its consequences such as ground settlements, lateral spreading, and flow failures, are discussed in Chapter 3 of Geotechnical Design Criteria.

The approach for evaluating the seismicity and associated ground motions for any feature of the project consists of:

1. Review existing data and information to identify contributing faulting and seismicity of any site.
2. Assess site conditions including surface and subsurface conditions to estimate ground motions.
3. Estimate appropriate ground shaking hazards and developing site-specific response spectra.
4. Develop site-specific acceleration time histories as necessary to evaluate seismic performance of critical facilities.

The following sections provide guidelines for assessing seismicity and for developing design ground motions for the facilities.

SECTION 2  SEISMIC DESIGN CRITERIA

The project is a new facility that transports water from north of the Delta to south of the
Delta. These facilities are not considered essential because they do not (1) directly supply water to people or for fire suppression, and (2) are not expected to be used in an emergency.

They are critical. Long delays in water delivery from the north to the south of the Delta could have a significant negative impact on human life and the California economy. The facilities shall be designed as described in ASCE 7 for critical facilities. Facilities that require extended time frames for repair/replacement (e.g. large pumps or tunnel structures) shall be designed with the highest seismic standard to prevent prolonged delays.

DWR’s Water Resources Engineering Memorandum No. 70 (WREM-70) prescribes minimum Seismic Loading Criteria for the State Water Project, and provides different seismic loading criteria based on criticality of a facility. The WREM-70 allows flexibility so that the project engineer can judge which criteria to use. SWP facilities, located throughout California, vary in purpose and criticality. Designing all the facilities to the same seismic standard would not be feasible or cost effective. Therefore, the WREM-70 provides a wide range of seismic loading criteria for various types of facilities.

The seismic loading selected for a facility is based on

- Consequences of failure,
- Criticality of the structure for water delivery, and
- Downtime and cost for the repair of the facility.

Generally, higher seismic loading criteria are recommended for structures that have greater
consequences of failure,
importance for water delivery,
downtimes, and/or
costs for repair.

Conversely, lower seismic loading criteria are recommended for structures that are not critical to water delivery and/or can be repaired in a reasonable time frame. For instance, canals can be repaired within a reasonable time frame compared to tunnels or the large pumps of a pumping plant.

Project specific seismic criteria are recommended to satisfy the needs and operation requirements. Some of the seismic criteria are slightly modified from the criteria recommended in WREM-70. Table II.1 summarizes the seismic loading criteria recommended for the facilities.

The upper-bound envelope, 84th percentile ground motion from the controlling fault and 975-year average annual return period event, is recommended for dams and reservoirs. Generally, the 84th percentile ground motion would control the design envelope; however, near major active faults the probabilistic ground motion with a 975-year average annual return period can do so. This criterion is recommended for all (including non-jurisdictional) dams and reservoirs.

The conveyance facilities, including canals, pipelines, and tunnels, recommended seismic loading criteria are presented in Table II.1. Based on anticipated longer repair time, a higher seismic standard is recommended for pipelines and tunnels than for canals. Repair costs associated with tunnels and pipelines is also greater than repair costs for canals.
Bridges facilities are considered critical for emergency response purposes. Heavy equipment will need to be transported during an emergency. Consistent with off-site highways, seismic design loads for bridges shall follow CalTrans seismic design criteria.

Other facilities including buildings, hydraulic structures, utility crossings, and other non-structural components such as electrical mechanical equipments are critical for water delivery. These types of facilities shall be designed using ASCE 7 with appropriate importance factors and modification factors to the maximum considered earthquake (MCE) as described in the Structural Chapter.
<table>
<thead>
<tr>
<th>Facility Type</th>
<th>Seismic Loading(^1)</th>
<th>Selection Basis</th>
<th>Analysis Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dams and Reservoirs</td>
<td>Greater of 84(^{th}) Percentile and 975-year Return Period (RP)</td>
<td>Modified DSOD Method</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Conveyance Facilities</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Canals</td>
<td>975-yr RP</td>
<td>Damage consequence and ease of repair</td>
<td>PSHA</td>
</tr>
<tr>
<td>Pipelines</td>
<td>Greater of 84(^{th}) Percentile and 2475-yr RP</td>
<td>Modified FEMA Function Class IV and difficult to repair</td>
<td>Envelope of DSHA and PSHA</td>
</tr>
<tr>
<td>Tunnels (Include inlets and outlets)</td>
<td>Operational Basis Seismic Check Based on 144-yr RP</td>
<td>Performance based approach to target elastic behavior and avoid outages and subterranean repairs.</td>
<td>PSHA</td>
</tr>
<tr>
<td>Bridges</td>
<td>CalTrans Seismic Design Spectrum</td>
<td>CalTrans Seismic Design Criteria (SDC ver. 1.6 Appendix B)</td>
<td>Envelope of DSHA, PSHA</td>
</tr>
<tr>
<td>Building and Hydraulic Structures, Utility Overcrossings, and Mechanical and Electrical Equipments</td>
<td>Seismic Response Based on ASCE 7 MCE</td>
<td>Modified ASCE 7 Design Spectral Response to Approx. 975-yr RP – Collapse Prevention Performance Level</td>
<td>PSHA with DSHA Cap</td>
</tr>
<tr>
<td></td>
<td>Operational Basis Seismic Check Based on 144-yr RP</td>
<td>Pseudo Performance Based Approach to target</td>
<td>PSHA</td>
</tr>
<tr>
<td>RP</td>
<td>Operational Performance Level</td>
<td></td>
<td></td>
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<td>----</td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

1. Seismic load shall be greater or equal to a deterministic spectrum from a 6.5 moment magnitude strike slip fault earthquake at a distance of 12 Km.
SECTION 3  \hspace{1cm} GROUND MOTION DEVELOPMENT

As indicated in the previous section, design ground motions shall be developed by conducting site-specific seismic hazard analyses. Depending on the facility requirements, seismic hazard analyses shall be conducted using both probabilistic and deterministic methods.

The following steps shall be considered when performing seismic hazard analyses for a given project site using either probabilistic or deterministic methods.

1. Characterize subsurface conditions based on the available subsurface information;
2. Identify potential seismic sources and fault parameters;
3. Perform probabilistic or deterministic seismic hazard analyses (PSHA or DSHA) to generate acceleration response spectra at the surface of a competent subsurface soil layer or bedrock surface;
4. Incorporate directivity effect from the faults that are close to the project site; and
5. Perform site response analyses to calculate response spectra at the ground surface if soft soils or liquefiable soils are present in the subsurface.

3.1 Subsurface Characterization

Engineers are to use the available information to characterize subsurface conditions and exercise engineering judgment as necessary to identify soft soil layers and liquefiable soils in the subsurface and foundation earth materials. To develop the seismic ground motions, the average shear wave velocity in the upper 100 feet ($V_{s30}$) is to be calculated using either measured or estimated shear wave velocity data at the site.

As the $V_{s30}$ value has significant influence in the ground motion prediction equations, it
is better that Vs30 data be obtained from direct measurement of shear wave velocities at a site. In cases where measured site specific Vs data is not available, the designer can use other properties such as blowcount, density, etc. to estimate the range of Vs30 for the analysis. If the Vs30 varies across a site, the seismic ground motions are to be developed to bracket the appropriate range of the Vs30 values.

3.2 Identification and Characterization of Potential Seismogenic Sources

All significant seismogenic sources within 200 km of the site are to be identified and characterized with respect to geometric and geologic conditions that affect the development of seismic source parameters for use in the development of the earthquake ground motions. The required parameters for seismic hazard analysis to develop design response spectra for a site, compiled or estimated from available data for each potential seismogenic source, are:

- Style of Faulting (e.g. strike-slip, reverse, normal);
- Closest source-to-site distance;
- Total fault length and estimated rupture length;
- Rupture width;
- Slip rate; and
- Maximum magnitude (Mmax).

3.3 Seismic Hazard Analysis Methodology

3.3.1 PROBABILISTIC SEISMIC HAZARD

Probabilistic seismic hazard analyses (PSHA) is to be performed using an industrially accepted computer program. The PSHA program employs the analytical procedure to compute seismic hazard originally developed by Cornell (1968).
3.3.2 DESIGN GROUND MOTIONS

Design ground motions is to be determined using the arithmetic average of multiple New Generation Attenuation (NGA) relationships from both probabilistic and deterministic seismic hazard analysis methods.

3.3.3 ATTENUATION RELATIONSHIPS

The following attenuation relationships are recommended for the determination of design accelerations:

- Chiou and Youngs (2008).
- Campbell and Bozorgnia (2008).
- Idriss (2008).

Other attenuation relationships are to be considered, as judged appropriate, depending on specific site conditions and fault mechanisms.

3.3.4 HORIZONTAL GROUND MOTION

The horizontal ground motion response spectral curves are to be provided for a 5 percent damping value for periods up to at least 10 seconds. The development of response spectra for longer periods and other damping ratios might be required for certain cases, depending on the type of facility and analysis requirements.

3.3.5 VERTICAL RESPONSE SPECTRA

When vertical response spectra are necessary, the methods recommended by Gulerce and Abrahamson (2011) are to be used to obtain the ratio between vertical and horizontal response spectra. This ratio is multiplied with the average horizontal
component to obtain vertical response spectra.

3.3.6 NEAR-FIELD SOURCE EFFECTS

For the faults that are located within 10 to 15 km from a site, the near-field source effects are to be considered (Abrahamson, 2000). The near-field source effects can be considered in two ways:

1. Probabilistic Method - by moving the hypocenter location along the fault when conducting a PSHA analysis or;
2. Deterministic Method - as described by Somerville et al. (1997) by locating the hypocenter on a fault such that the maximum near-field source effect is calculated.

The near-field effects are slightly on the conservative side when calculated using the Deterministic Method.

3.3.7 SITE RESPONSE ANALYSIS

Site response analysis is necessary if the foundation subsurface contains liquefiable soils or soft soils, and is untreated. In this case, design ground motions are to be developed at the surface of a competent underlying layer using abovementioned methods and propagated through the soils above the competent layer using site response analysis. The results from the site response analysis shall be used to develop design response spectra at the base of the foundation.

The site response analysis is to be conducted using equivalent linear or nonlinear soil models. Equivalent linear soil models are used for small intensity earthquakes so that the soils would experience only a limited nonlinear behavior and can be reasonably represented by equivalent linear models. For large intensity ground motions, the
nonlinear soil models incorporated in a finite element or finite difference program can be used to capture the nonlinear behavior of soils. The site response analysis is to consider the sensitivity of the soil response to the uncertainties in the soil properties, depth of soil model, and the input motion.

Acceleration time histories are necessary to perform the site response analysis. The procedure described in the following section can be used to develop design time histories.

3.3.8 DEVELOPMENT OF ACCELERATION TIME HISTORIES

Time history analysis is useful to evaluate the seismic performance of structures, both in the design of new structures and the evaluation of existing structures. This type of analysis requires the development of design acceleration time histories, which are to consider the following:

1. Several site-specific time histories are to be developed and used with both positive and negative polarity to account for the uncertainty and variability associated with ground motions. Generally, three to seven time histories are adequate to analyze a structure. If only three time histories are used, then the most severe response of the three time histories are to be used as the basis for design/evaluation. If seven time histories are utilized, then the average response to these time histories are to be used as the basis for design/evaluation;

2. To develop time histories for design analysis, seed time histories from past earthquakes or from synthetically developed earthquake records are to be utilized; and

3. Selected seed time histories are to be scaled or spectrally matched to the design spectra to obtain appropriate acceleration time histories to use in the design analysis.

3.3.8.1 Selection of Seed Time Histories

To develop acceleration time histories, seed time histories are to be selected either from
past recorded earthquakes or from available synthetic records. It is preferred that recorded
time histories from past earthquakes be used rather than synthetic time histories. However, in certain scenarios, recorded time histories are scarce and the use of synthetic time histories is necessary to generate the desired total number of time histories. When selecting seed time histories, the following criteria are to be considered:

4. The selected motion is to be compatible with project-specific parameters such as earthquake magnitude and site-to-source distance as determined from the hazard de-aggregation, rupture mechanism, site conditions, peak intensity measurements, and frequency content;

5. The Arias Intensity and duration of the selected motions are to be consistent with the estimated ground shaking at the site; and

6. The selected motion is to have appropriate directivity or near-field source effects as necessary for the site.

Recorded ground motions can be downloaded from the PEER (Pacific Earthquake Engineering Research) Center strong motion database web site (http://peer.berkeley.edu/smcat/).

3.3.8.2 Scaling the Seed Time Histories to Design Parameters

After selecting the recorded time histories, if the motion was not sampled at a time step of 0.005 seconds, re-sampling is necessary. Re-sampling involves calculating the Fast Fourier Transform (FFT) and then the Inverse FFT with interpolation at the desired sampling rate.

The time domain spectral matching procedure as coded in RSPMATCH written by Abrahamson (2003) or later version is recommended to be used for the facilities. The quality of the results are to be assessed by measuring the tolerance to which the
matched motions converge towards the target spectrum in the frequency domain, and how well the matched motions compare to the original motions in the time domain. In particular, the matched displacement and velocity time histories must look reasonable and reflect some of the predominant characteristics of the original motions. In addition, the spectrally matched time histories are to satisfy the design Arias Intensity for the design level.

The spectrally matched motions are to be baseline corrected prior to use in any analysis, including site response analysis.
Appendix M
Seismic Review of Tunnel Liner Performance
1 Task Order 6 Description

The California WaterFix program has been developed to stabilize water deliveries from the Sacramento – San Joaquin River Delta (Delta) area. The California WaterFix EIR/EIS relies exclusively on a twin tunnel concept to meet the purpose and need of the program. Arup North America, Ltd. (Arup) has been requested to evaluate the impact of using a 2,455-year earthquake for the design of the segmental tunnel lining. The 2015 CER presents two segmental lining designs – the current accepted design with additional tension connections across the joints, and Alternative 1 (located in Appendix I) which uses standard joint details. The Alternative 1 segmental lining drawings have been used as a basis for this report, since that arrangement has the lowest capacity and if the Alternative 1 design performed adequately in a seismic event, the current accepted design is likely to perform better.

This task order results from commentary by interested stakeholders during the review process that the tunnel concept which requires additional work to confirm the seismic performance of the tunnels.

1.1 Report Scope

The following sub-tasks are addressed in this report:

**Subtask 1:** Analyze the potential impact of "ovaling" on the tunnel rings during a seismic event and the potential for that deformation to open up the gasket joints and provide a pathway for leakage into/out of the tunnels.

**Subtask 2:** Investigate the difference between the radial and circumferential joint design to mitigate ground deformation impacts.

**Subtask 3:** Evaluate the potential impact to the current design concept for the California WaterFix Tunnel liner by upgrading the seismic event criteria from the 975-year return period to the 2,475-year return period.
Technical Memorandum

237477    July 6, 2018

Subtask 4: Review the critical design loads likely to be present for the California WaterFix tunnels to determine which loading (e.g. installation loads, seismic loads, handling loads, ground loads) governs, and compare different load cases compared to the seismic load case.

Subtask 5: Evaluate the potential for liquefaction of ground along the tunnel alignment at tunnel depth, considering the increase in the target return period. Perform assessments using the Boulanger and Idriss (2014) methodology.

Subtask 6: Analyze the anticipated ground motion attenuation with varying tunnel depth during a seismic event, reviewing the possibly variation of seismic motion with depth and whether the reductions vary with increased return period. This will be performed using the site response analysis in Subtask 5.

Subtask 7: Investigate the ability of backfill grouting to enhance tunnel durability during a seismic event, reviewing the strains developed from Subtask 1 and provide recommendations accordingly in terms of type of grout selection.

Subtask 8: Review current industry standards for seismic design of tunnels determined to be “Critical Infrastructure”. Discuss what magnitude of seismic event does the industry typically plan for in tunnel design, and which codes and/or guidelines are typically referenced. Also discuss the current experience and discussions with various agencies.
2 Analysis of Tunnel Ovaling Under RFQ (Subtask 1)

2.1 Subtask Scope

As described in more detail in Section 9, underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial, and curvature deformations. The ovaling/racking deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis, and is seen by the diameter of the circular tunnel being slightly reduced on one axis while being slightly increased on the other, as shown in Figure 2.1. This results in small rotations of the joint faces between tunnel lining segments, potentially impacting the watertightness of the tunnel lining.

Figure 2.1: Ovaling/Racking Deformation of Tunnel

This section summarizes the model used in this study to analyze the potential for a leakage pathway resulting from ovaling of the tunnel rings during a seismic event. The analysis has been completed using Oasys LS-DYNA general finite element software. The 3D plain strain numerical model is designed to focus on the 2 15 CER Alternative 1 segment design joint design (with no tension bolting between segments), focusing on the ring and joint behavior and the impacts to the design when stepping to the higher seismic design loading under the imposed Maximum Design Earthquake (MDE) 2,475-year event. Use of the Alternative 1 segment design (see Appendix I) is the most critical case; the use of tension bolting across the joint will only improve performance.

The LS-DYNA model contains the structural elements of the ring, the longitudinal joint layout, and gaskets, and the surrounding ground mass, allowing for soil structure interaction behavior to be captured. The model is not intended to function as full 3D Non-Linear Time History (NLTH) soil structure interaction analysis as described in Section 9. Such an analysis is appropriate during final design, as it requires extensive effort and computational time and should include non-linear soil properties and a set of at least of 11 spectrally matched ground motions applied at the base of a full multi layered soil profile, similar to the one assumed for the 1D SRA analysis described in Subtask 3.

It is also noted that modifications of any assumptions or parameters (i.e. material properties, ground motions, soil conditions, etc.) can alter the results and conclusions presented in this Section.
2.2 Model Overview and Assumptions

The seismic analysis to assess the segment performance has been performed in LS-DYNA using a 3D model considering both the tunnel and the surrounding soil. The model is set up in plain strain boundary conditions and ground motions are applied in the horizontal direction resulting 2D ovalization of the tunnel lining.

The model and tunnel geometry and key input assumptions are described below and illustrated in Figure 2.2 to Figure 2.7:

1) Tunnel geometry is as follows:
   - Internal Diameter: 40’-0’’ (12.2m)
   - Lining Thickness: 20’’ (500mm)
   - Number/Geometry of Segments: 9 equal segments (each with an arc of 40 degrees)

   The model represents a 3’-4” (1m) length of tunnel including a circumferential joint and 1’-8” (0.5m) of segments either side of the joint.

2) Single ground motion time history at 100 feet below ground surface was provided as an input (see Subtask 3) which is applied at the base of the model with Lysmer Dampers acting as a nonreflecting boundary. This ground motion is assumed to be constant for the tunnel and not vary with depth or soil properties.

3) Shear wave velocity of 770ft/ has been assumed for the dampers as per CER July 2015 geotechnical document and Subtask 3 site response analysis.

4) Model side boundaries are set to ensure similar soil response through the model (i.e. no elevation based amplification)

5) Water pressure elevation head assumed at 0 feet elevation, equal to external groundwater. The model is therefore an effective stress model assuming equal internal and external water pressures.

6) The tunnel is fully filled with water. On the basis that the internal and external water pressures are equal in the tunnel in the long term, no water pressures are included in the model. However, mass elements are added to the internal face of the tunnel segments to allow for inertial effects of water inside the tunnel.

7) Constant isotropic soil effective stress assumed in the ground and equal to approximately 36 psi (250kPa) assumed acting onto the tunnel. This value represents an idealization of the assumed worst case conditions along the alignment – i.e. maximum total ground stress coupled with maximum assumed internal and external water pressures along the tunnel.

8) The ratio between the vertical and horizontal ground stress is assumed to be equal ($K_0 = 1$). This assumption is commonly deployed for buried structures in over-consolidated soils. This
also allows simplification of the model output and focuses only on the ovalization resulting from the seismic motion.

9) The soil material is an elastic soil with an elastic modulus of 29 ksi (200MPa) based on geotechnical information given in CER July 2015 documentation (see also Subtask 3).

10) The concrete is an elastic material model assuming nominal concrete properties for 7000 psi (C50) concrete segments.

11) Segments and gaskets are modelled explicitly using solid elements. Due to the limited information provided in CER July 2015, the Datwyler M 385 73 type “Wesertunnel” gasket is assumed in terms of materials and acceptance criteria.

12) Due to the potential for segment joint slippage observed in early runs of the model, nominal 12in (300mm) long 2.4in (60mm) diameter plastic guide rods are placed within each joint to provide sliding resistance within the joint. Note the model is approximately 30 in (0.75m) long, so the guide rod is present along approximately 4% of the segment joint length.

13) The distance from the sides of the gaskets to the edge of the segments as presented in the 2015 CER resulted in high concentrated stresses at the concrete edges. The gasket locations have been adjusted to be approximately 1.65 inches (42mm) from the segment edges in line with standard practice.

14) Based on CER July 2015 drawings, a total of 36 circumferential dowels are considered between the adjacent rings. Due to the limited information provided, the doweled connections have been modelled using the tensile and shear performance of the Sof-Fix 110 connectors by Optimas.

15) To represent the actual installation approach, the segments have been brought together in place, with bearing surfaces at joints in full contact; it is assumed that no transfer of tension occurs across the joints.

Figure 2.2: LS-DYNA model overview.
Figure 2.3a: WaterFix tunnel geometry information as per CER July 2015 (Appendix I). Note: Alternative 1 is identical, but without the radial tension bolting shown in this figure.
Figure 2.3b: WaterFix tunnel geometry information as per CER July 2015 (Appendix I).
Figure 2.4: Overview of tunnel segment arrangement based on CER July 2015 (Appendix I).

Figure 2.5: Overview of tunnel joints as modelled. Gasket geometry not available in CER July 2015 (Appendix I).
Figure 2.6 Datwyler M 385 73 type “Wesertunnel” gasket parameters assumed in the numerical model. (1mm = 0.0395 in.; 1 bar = 14.5psi)
2.3 Ground Motion

The single ground motion considered in this analysis represents a 2,475-year return period at the southern end of the alignment at 100-foot depth (Ground Motion 1 Component 1 - See Subtask 3). This ground motion includes high spectra response at low periods which assumes a stiff tunnel structure with a dominant period less than 1 second.

A sensitivity check has been carried out on Ground Motion 6 Component 2 (See subtask 3) assuming a flexible tunnel structure with a dominant period greater than 2 seconds. See also Section 2.4.3.

Figure 2.8: shows the acceleration time history applied at the base on the model and Figure 2.9: shows the associated response spectrum.
Figure 2.8: Acceleration time history ($\text{m/s}^2 = 1 \text{ ft/s}^2$)

Figure 2.9: Response spectrum (5% damped) ($1 \text{ m/s}^2 = 3.281 \text{ ft/s}^2$)
2.4 Analysis Results

2.4.1 Tunnel Ovalization

Figure 2.10 and 2.11 illustrate the degree of potential ovalization that could occur during the seismic excitation as simulated in the LS-DYNA analysis for the chosen ground motion. The image from the analysis has a deformed shape that is magnified to highlight the ovalization shape. A peak ovalization of approximately 0.15% of the tunnel’s inner diameter is observed.

This ovalization modelled corresponds well with closed form solutions carried out in accordance with the methods described in Section 9.4 below which gives a transient maximum peak ovalization of 0.2%. This ovalization occurs at the time where the liner is subject to the maximum shear strain. Residual ovalization in observed to be minimal, with the joints losing following passage of the shear strains.

![Figure 2.10: Worst case tunnel deformed shape during seismic analysis (note: exaggerated scale)](image)

Figure 2.10: Worst case tunnel deformed shape during seismic analysis (note: exaggerated scale)
2.4.2 Tunnel Hoop Stresses

Figure 2.12 shows the tunnel lining maximum and minimum hoop stresses enveloped during the seismic excitation. Note that +ve = tension, -ve = compression in LS-DYNA.

The enveloped minimum hoop stress in the tunnel is shown in Figure 2.12(a). The average minimum is approximately -145psi (-1 N/mm²) with localized peaks of up to -1233 psi (-8.5 N/mm²) at the crown and invert during the ground motion. A majority of the tunnel does not see stresses exceed -218psi (-1.5 N/mm²). Approximately +580 psi (+4 N/mm²) peak tension is observed at the external face due to tunnel ovalization.

The enveloped minimum hoop stress in the tunnel is shown in Figure 2.12(b). The average stress in the tunnel is shown to be < -0.5 N/mm² (73 psi) with localized peaks of up to -870 psi (-6 N/mm²) at the same location as the minimum stress described above. Approximately +943 psi (+6.5 N/mm²) peak tension is observed in the segments, predominantly at the external face, during the ground motion at peak ovalization.

The stresses presented in Figure 2.12 can thus be summarized as follows:

\[
\begin{align*}
\sigma_{N_{\text{max}}} \text{ [peak compression]} & = -1233 \text{ psi (-8.5 N/mm}^2) \\
\sigma_{N_{\text{min}}} \text{ [peak tension]} & = +943 \text{ psi (+6.5 N/mm}^2) \\
\sigma_{N_{\text{avg}}} \text{ [average compression]} & = -145 \text{ psi (-1.0 N/mm}^2)
\end{align*}
\]

It is noted that these stresses are relatively low considering that the tunnel is to be constructed of reinforced concrete segments. As such segment structural performance is not expected to be compromised during the seismic motion.
Figure 2.12 a) Maximum and b) minimum hoop stress in the tunnel during seismic excitation. (1 N/m² = 145psi). +ve = tension, -ve = compression.

2.4.3 Gasket Performance

Gasket performance can be measured as a function of joint gap opening and offset which can be correlated to gasket water tightness as per Figure 2.6 above.

Due to the low ovalization experienced by the tunnel from to the chosen applied ground motion and input soil and tunnel properties, the joint opening observed during the motion in the tunnel simulation is considered negligible (a 40ft ID tunnel under 0.15% diametrical deformation results in a gap opening of approximately 0.5mm (0.02 in)). Following the ground motion, no residual gaps are observed. Gasket performance should therefore not be compromised either during or after the seismic event.
However, as is described further in Section 5, varying ground response and consideration of additional ground motions as well as different tunnel segment configuration and properties may produce additional ovalization that may result increased joint opening and higher joint demands and therefore reduced gasket performance.

A sensitivity model run with a different ground motion and soil response (Ground Motion 6 Component 2 – See Subtask 3) has been conducted. This additional ground motion assumes a flexible tunnel structure which corresponds to high spectral response at higher periods (> 2 secs). The sensitivity check has shown that temporary gap openings can increase to about 6-7mm, but minimal to no residual gaps remains after the motion passes, similar the base case described above.

It is therefore not currently anticipated that movements will compromise gasket performance, given the additional capacity that can be provided at critical locations with tension connections across the joints, additional dowel capacity between rings, and enhanced gasket performance. These assumptions should be further investigated during the design phase of the project.

**2.4.4 Longitudinal (Radial) Joint Performance**

The LS-DYNA model presented above is a 2D model focused on capturing radial joint behavior. For circumferential joint opening a manual calculation was carried out using Arup’s in-house spreadsheet using various methods described in Section 9.

The manual calculation’s inputs are in accordance with the ground motion and ground parameters assumed for the LS-DYNA model. The calculation assesses the free-field ground strains ($\epsilon_{\text{MDE}}$) imposed on the tunnel which is assumed to act as a continuous “pipe” as described in Section 9.4.

Axial and curvature deformations develop in a horizontal tunnel the seismic waves propagate either parallel or obliquely to the tunnel, resulting in zones of compression and tension. The tunnel gasket will deform under these strains, extending or shortening under this action.

This longitudinal tensile strain considers elongation of the gasket and the concurrent reduction in gasket pressure resisting water load. To ensure a conservative design, this case has been calculated towards the end of the tunnels life once 50% relaxation has occurred.

$L = \text{Length of Tunnel Segment} = 5 \text{ ft}$

$\Delta L = \text{Elongation of Gasket} (L \cdot \epsilon_{\text{MDE}}) = 0.08\text{ in} (2\text{mm})$

The calculation presented above demonstrates that the gap forming due to seismic loading results in gap opening is minimal. From experience on similar tunnel projects and from the gasket compression curve shown in Figure 2.6, it is therefore concluded that elongation due to seismic movement will not reduce the sealing capacity of the gaskets and cause leakage into the tunnel.

However, as is described further in Section 3, varying ground response and consideration of additional ground motions as well as different tunnel segment configuration and properties may produce additional tunnel longitudinal curvatures that may result increased joint opening and higher joint demands and therefore reduced gasket performance.
Nevertheless, given that additional capacity that can be provided at critical locations with tension connections across the circumferential joints with additional tensile dowel capacity between rings, and/or enhanced gaskets are available can be used to assure the proper gasket performance. These assumptions should be further investigated during the design phase of the project.
Circumferential and Radial Joint Design (Subtask 2)

3.1 Subtask Scope

The scope of this subtask is to investigate the circumferential and radial (longitudinal) joint design per the 2015 CER and provide input on how modifications to the joint design may enhance the performance of the joints, and therefore maintain the sealing capacity of the rings during a 2475-year seismic event. Circumferential joints are defined as the joint that is formed between sets of adjacent rings. Radial joints are defined as the joint that is formed between segments in the same ring.

3.2 Joint Geometry and Configuration

Sensitivity studies have been conducted looking at the gasket location relative to the inner and outer surfaces of the segment. If the gasket is located too close to these surfaces, our sensitivity studies have shown that crushing and spalling of concrete can be observed. Therefore, the results presented may vary based on the final design of gasket locations.

Several different joint details are currently being utilized in California and other highly seismic regions to address ground deformation issues during a seismic event. This section discusses common approaches to deal with joint design to accommodate the impacts a tunnel lining will experience during a seismic event. Typically, the seismic impacts on a tunnel lining are:

- a distortion, or ovalization, of the circular lining which is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis, causing deformations in the plane of the tunnel cross-section. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation.
- Compression/relaxation in the longitudinal direction of the tunnel induced by components of seismic waves that propagate along the longitudinal axis.

The approaches are described in the following sections and are all potential application to the WaterFix project to avoid joint damage or leakage. The exact solution, which may consist of one of these approaches, or a combination of several of them, will be developed through the detailed design solution.

3.2.1 Use of convex-convex versus flat-flat radial joints

The use of convex-convex radial joints has been adopted by several owners in locations where distortional movement of the tunnel may occur, including Singapore LTA (ground movement due to soft clays) and LA Metro (ground movement due to seismic events). A convex-convex joint consists of both joint faces in the radial joint having a large radius curvature so that the contact between the faces is the intersection of the two curved surfaces. This compares with a flat-flat joint, which consists of two flat surfaces. While a flat-flat joint has a higher capacity than a convex-convex joint if the segments are perfectly aligned, as the contact area is spread over a large area, as soon as distortional movement of the tunnel occurs and the joint faces rotate relative to each other, the contact area is limited to the edge.
of the joint face, resulting in a concentrated eccentric load. The concept behind the convex-convex joint is that the bearing surface remains in the middle section of the segment face when distortion occurs. This allows the load to be spread more evenly and avoids highly concentrated loads, that otherwise may cause damage, allowing the joints and gaskets to behave as designed.

![Diagram of joint types](image)

Figure 3.1: Different joint types under distortion comparative figure. Note that distortion and clearances to accommodate it are exaggerated. Gasket is not included in Convex-Convex joint type for clarity

### 3.2.2 Use of double versus single gaskets

Typically, segmental tunnel linings provide a high level of watertightness using high quality precast concrete and a single rubber (ethylene-propylene diene monomer, or EPDM) gasket. A single EDPM gasket, if designed and implemented properly, can readily meet the sealing demands of the projects, even under extremely onerous conditions as demonstrated on several recently completed projects with
large TBMs and under extremely high-pressure conditions. When adjacent segments are placed together, the gasket is compressed tight and provides a barrier to water ingress through the joint. The gasket design incorporates allowances for placement tolerances and relaxation of the gasket over time. The use of gaskets for watertightness has been proven over many decades of use and under high water pressures.

Double gaskets, where gaskets are located near both the intrados and extrados faces, have historically been used where there is concern over leakage into the tunnel due to high external water pressures (e.g. Elbe Tunnel) are present, where gasses (LA Metro) in the ground mandate a higher need for sealing and redundancy, or in seismic conditions where the joint may open during short-term distortion of the lining. The use of double gaskets is typically combined with the use of cross gaskets to compartmentalize the space between the inner and outer gaskets. If bolts are used in the segment joints, the heads of the bolts will also require seals.

![Double Gasket Diagram](image)

Figure 3.2: Single vs Double Gasket

### 3.2.3 Segment and Ring Connectors

Segment connectors are normally provided between segments in the same ring as well as from ring to ring. Traditionally, the connections were made using steel bolts with fiberglass reinforced plastic or aramid female embeds. While relatively simple, the bolts are limited in their shear resistance due to the need to increase the flute size in the segment body to account for construction tolerances, and therefore only the moderate increase in frictional shear resistance across the joints should be considered in resisting seismic action.

Starting in the early 2000s several projects began using high strength plastic connectors across the circumferential joints between rings. These first connectors did not have much pullout force, with the female sides often simply a cast negative space in the body of the concrete segment. Since then, significant advancements have been incorporated, including larger barbs (teeth) as well as integrally designed female embeds that provide significant pullout and shear capacity. These connectors are cast and fit with much tighter tolerances, meaning that less movement across the joint is needed to fully activate the dowels in shear. When building tunnel rings greater than 9m these shear connectors are
often supplemented with bolts. The use of a combination of bolts and circumferential connectors provides both the increase in frictional resistance as well as shear capacity. The connectors are often specified and are manufactured from virgin polymers. For the WaterFix tunnels, it is foreseen both circumferential bolts with shear connectors will be used in the ring to ring connections.

Segment to segment connectors continue to utilize bolts, however their need in the permanent connection is debatable, with several projects completed worldwide removing the bolts out after the backfill grout is fully cured. Accompanying the radial bolts, and also alone, guide rods can add additional shear resistance to the radial joints. Varying in range between 30mm up to 60mm and beyond, the rods act to reduce the segment to segment stepping and have a high shear capacity per unit length. It should be considered that the guide rods are normally manufactured from recycled LDPE products, and therefore have less fabrication tolerance than the circumferential connectors. Considering the size of the tunnel, it is expected that the WaterFix tunnels will use either guide rods, radial bolts, or a combination thereof to provide additional shear resistance across the radial joints.

3.2.4 Effect of skewed angles on radial joint performance

Radial Joints are often skewed (i.e. angled relative to the longitudinal axis of the tunnel) to avoid the gaskets on the radial face rubbing and potentially tearing as a segment is placed alongside one that is already installed. This does however influence the performance of the joints under seismic conditions. As the ring deforms diametrically, skewed joints tend to ‘ride up’ on opposing corners of the radial face (i.e. inside leading edge corner vs outside trailing edge corner or vice versa), and causes stress concentrations in these regions. If the radial joint is to be skewed, these stresses must be appropriately captured in analysis and designed to prevent local bursting or in extreme cases, failure of the corners throughout the thickness of the segment body. Alternatively, straight radial joints (aligned with the longitudinal axis of the tunnel) can be used if bolting is used rather than dowels on the circumferential joint, as this allows more flexibility in placing the segment. In additional, recent advances in gasket manufacturing have seen stronger, more resilient gaskets, and stronger adhesives or cast-in-gaskets to maintain the gasket in place during segment placement, and lubrication of the gasket can also be used to minimize this problem.

3.2.5 Circumferential Joints

The circumferential joints is typically defined as the joint that is formed between two sets of adjacent rings. Typically, segmental tunnel linings have some locked-in compressive stress across the circumferential joints due to the segments being placed behind a closed-face TBM. Compression/relaxation across the circumferential joint will cause variation in this compressive stress, but does not typically impact the joint design.

3.2.6 Project Examples

Depending on the project requirements, the design of the tunnel liner’s radial and circumferential joints may vary from one project to another. The following section includes a selection of ring arrangements and joint
design from select past projects, along with the current concept for the California WaterFix Tunnels are summarized in Table 3.1, with joint details provided in Appendix A.

Table 3.1: Typical Joint Details

<table>
<thead>
<tr>
<th>Project</th>
<th>Internal Diameter</th>
<th>Tunnel Use</th>
<th>Seismic Conditions</th>
<th>Radial Joint Design</th>
<th>Water Pressure</th>
<th>Gasket Design</th>
<th>Connection Design</th>
<th>Skewed radial joint?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lake Mead Intake No.3</td>
<td>20'-0''</td>
<td>Raw Water</td>
<td>Moderate Seismicity</td>
<td>Flat-Flat</td>
<td>18 bar</td>
<td>Single</td>
<td>Radial Bolt, Bolts between rings</td>
<td>Yes</td>
</tr>
<tr>
<td>Arrowhead tunnel</td>
<td>16'-0''</td>
<td>Raw Water</td>
<td>High Seismicity</td>
<td>Flat-Flat</td>
<td>20 bar</td>
<td>Single</td>
<td>Radial Bolt, Bolts between rings</td>
<td>Yes</td>
</tr>
<tr>
<td>Crenshaw Corridor</td>
<td>18'-2''</td>
<td>Metro Tunnel</td>
<td>High Seismicity</td>
<td>Convex-Convex</td>
<td>3 bar</td>
<td>Double</td>
<td>Radial Bolt, Dowels between rings</td>
<td>Yes</td>
</tr>
<tr>
<td>SF Central Subway</td>
<td>17'-2''</td>
<td>Metro Tunnel</td>
<td>High Seismicity</td>
<td>Flat-Flat</td>
<td>4 ar</td>
<td>Single</td>
<td>Radial Bolt, Dowels between rings, guide rods radially</td>
<td>Yes</td>
</tr>
<tr>
<td>TMCLK</td>
<td>51'-2''/40'-8''</td>
<td>Road Tunnel</td>
<td>Moderate Seismicity</td>
<td>Flat Flat</td>
<td>5 bar</td>
<td>Single</td>
<td>Guide rods radially, Shear cone between rings</td>
<td>No</td>
</tr>
<tr>
<td>LA Outfall</td>
<td>18'-0''</td>
<td>Effluent Water</td>
<td>High Seismicity</td>
<td>Flat-Flat (with packer)</td>
<td>9 bar</td>
<td>Single</td>
<td>Radial Bolt, Dowels between rings</td>
<td>Yes</td>
</tr>
<tr>
<td>California WaterFix Tunnel Current Concept</td>
<td>40'-0''</td>
<td>Raw Water</td>
<td>Moderate Seismicity</td>
<td>Flat-Flat</td>
<td>5 bar (estimate)</td>
<td>Double</td>
<td>Tension Connection (Current Accepted Design) or Radial Bolt (Alternative 1), Dowels between rings</td>
<td>No</td>
</tr>
</tbody>
</table>
4 Impact of 2,475-year Earthquake (Subtask 3)

4.1 Subtask Scope

The scope of this sub-task is to assess the potential impact to the current design concept for the WaterFix tunnel liner by upgrading the seismic event criteria from the 975-year return period to the 2,475-year return period.

4.2 Impact

InfraTerra reviewed the seismic hazard analyses and developed uniform hazard target spectra based on spectral acceleration parameters from the DRMS report scaled to Site Class B/C conditions. The target spectra were used to select eleven ground motions representative of the 975-year and 2,475-year hazards. Site response analyses were performed based on subsurface conditions near Clifton Court, and the results indicate the median peak accelerations at a depth of 100 ft are about 0.21 g and 0.24 g for 975-year and 2,475-year ground motions, respectively. This relatively small difference between events can be attributed to the general seismic hazard in this area, which is dominated by shallow, crustal earthquakes with magnitudes, Mw, ranging from about 7 to 7.3 at rupture distances, Rrup, between 10 and 30 km away. A liquefaction hazard analysis was also performed at the depth of the tunnel near Clifton Court. The results indicate the liquefaction hazard is very low.

The study focused on Clifton Court as the DRMS report showed that the seismic hazard was the highest at that location.
Critical Design Loads (Subtask 4)

5.1 Subtask Scope

This section covers the identification of the most-critical design loads that will need to be considered for design of the precast concrete segmental linings for the WaterFix tunnels. The design of the segments will be driven by a range of different loading conditions: manufacturing loads, handling loads, installation loads, grouting loads, ground and groundwater loads and seismic loads. Each of these load cases will impact one or more areas of the segment design which include overall capacity of the section; the radial and circumferential joints and detailing of the joint faces.

The following section describe the different loading conditions that will be experienced by the segments and how they impact the design of the segments.

5.2 Manufacturing and Transportation

The segments will be subject to a wide range of loading during the process of casting, stripping, lifting, transporting and stacking of the segments. The main impact of these loads is to cause bending moments around the longitudinal axis of the segment. These loads can be onerous as they can occur soon after concrete placement and initial cure in the molds when the concrete strength is still developing, resulting in a lower capacity of the segment to resist these flexural stresses.

These loads are typically accommodated by specifying and designing handling equipment and stacking supports that minimize the imposed stresses. For example, when the segments are lifted out of the mold, a vacuum lifting pad that covers most of the segment surface can be used to minimize the bending stress in the segment.

Following individual handling of the segments they are stacked into ring sets. Typically, large forklifts are used to move the complete stacks for initial storage. Under these conditions, the segments act primarily in flexure with no axial load. When factoring in the early age strength of the concrete and the bending moment arm between axial and tensile zones, this can drive the need to have a marginal amount of reinforcement simply for these handling loads. The methods of handling and stacking are normally controlled so that this is not a critical load.

5.3 Installation

After handling of the segments on-site and delivery to the heading, if a mechanical anchor is used, it will impose a single-point lifting case on the segments, which can be the highest bending moment encountered by the segments. When lifting the individual segments for erection, the loading can reverse. This load case can be minimized by using vacuum lifts which spread the pick-up point into an area distributed across the whole segment.

One of the most critical loads the segment will experience is the load from the thrust rams used to advance the TBM forward, pushing off the completed segment rings. This loading case is often the
loading case that determines the thickness of the tunnel lining. The ram loads are spread to the ram shoes, and result in several critical loads on the segment:

- Tensile stresses immediately under the ram shoes, due to the concentrated applied loads,
- Tensile bursting stress between the ram shoes, caused by the concentrated compressive loads imposed on the leading edge of the segment,
- Shear/bending caused by potential unevenness in the bearing of the segment on previous rings due to build tolerances (planarity between segments in the same ring).

The first two of these cases often are not critical in the design, however under very high thrust requirements, this can influence the thickness of the segment. For the third bullet point, the segments may be difficult to design to this case, and may result driving the thickness and quantity/arrangement of the reinforcement, and result in a critical loading case for the segment. Detailing of the joint geometry and specification of build tolerances are essential in avoiding this load scenario.

Careful detailing of the joint, including edge details, is important in this case to avoid concentration of the load to close to the corner resulting in spalling of the edge.

### 5.4 Backfill Grouting

Two grouting loads are imposed on the lining: primary backfill and check grouting. For the purposes of this report, it has been assumed all primary backfill injections are performed through the TBM tailshield. Once set, the strength of the grout is in the 150 to 300 psi range.

Under primary backfill grouting, the assumption is that the entire ring is grouted and the annulus uniformly filled. The segment rings should be checked under this transitional semi-fluid state, which can result in higher than normal bending moments, but should not produce a critical loading case.

Following annulus grouting, if there is any indication that voids within the annulus could be present, check grouting is performed by injecting through the segment and injection of a fluid grout into the void space. There are two loading conditions which are formed which require checking: single point pressures on the grout socket due to the injection; and the distributed pressure on the extrados of the tunnel lining over an approximate 1m x 1m area, or over a 60-degree arc of the ring. The external pressure placed on the finished ring can induce higher bending moments that the static ground loads alone, however these are transient and concentrated locally and often neglect the influence of the adjacent rings. As for primary grouting, the rings are checked under these conditions, but only where very high injections pressures are used, should not produce a critical loading case.

### 5.5 Empty Tunnel

After the lining is constructed but prior to operation it will be subject to external ground and groundwater loading. The overall hoop loads/ring bending are unlikely to be a critical load case in this situation given the uniformity of the high hydrostatic loads that will provide a high thrust and low bending moment in the lining. The radial joint loading is however likely to be a critical load case with
the hoop loads in the lining causing concentrated loading resulting in tensile bursting stresses near the joint. These loads are typically accommodated by the quantity and arrangement of the reinforcement and detailing of the joint geometry.

Should a seismic event occur during construction while the tunnel is empty, then the bending stresses in the main body of the segment will experience instantaneous increases as the tunnel ovalizes. This will likely be the most onerous bending load seen by the segment. Depending on the exact geometry of the segmental lining, this would be expected to require a similar level of reinforcement in the segment to the construction loading case. The earthquake would also cause localized loading at the joints, as tunnel distortion results in rotation of joints. This can be mitigated by the design of the joints, as previously discussed.

Careful detailing of the joint, including edge details, is also important in this case to avoid concentration of the load to close to the corner resulting in spalling of the edge.

5.6 Operational Tunnel

In the operational condition the tunnel is filled with water under a gravity flow operation so that both the net loading and the out-of-balance differential between the horizontal and vertical loads on the tunnel is reduced. Although there are many hydraulic conveyance/surge/shut-down/partial system out of service scenarios that require a comprehensive analysis, it is anticipated that this means the most critical design cases will occur prior to the tunnel being put into operation and that operation load case will be less onerous or equal to the construction case. The exception would be a major seismic event, which would produce high bending loads in the segment and concentrated loads at the joints. These would be of similar magnitude to those seen during construction from ram loading and radial joint loads from the empty tunnel case.
5.7 Summary

The table below describes the most critical loading case for each design area.

<table>
<thead>
<tr>
<th>Design Area</th>
<th>Critical Loading Case</th>
<th>Design Approach</th>
</tr>
</thead>
</table>
| Capacity of main section - bending around longitudinal axis | 1. Ground and groundwater loading AND/OR Seismic loading  
2. Early age stripping, handling, stacking and transportation | 1. Selection of concrete strength, section thickness and reinforcement  
2. Use of lifting devices, stacking configurations to maintain bending stresses within limits |
| Capacity of main section – bending around perpendicular axis/shear along longitudinal axis | Application of thrust ram loads to advance the TBM | Selection of ram locations and segment thickness to minimize stresses; Specification of manufacturing and installation tolerances to minimize concentrated loads; Reinforcement detailing to accommodate imposed stresses |
| Circumferential joint bearing/bursting | Application of thrust ram loads to advance the TBM | Selection of ram locations and segment thickness to minimize stresses; Specification of manufacturing and installation tolerances to minimize concentrated loads; Reinforcement detailing to accommodate imposed stresses |
| Radial joint bearing/bursting | Ground and groundwater loading AND/OR Seismic loading | Detailing (recess geometry) and location of gasket groove (distance from corner) to reduce shear/tensile stresses; reinforcement detailing to accommodate loading |
| Shearing of corners | 1. Ground and groundwater loading AND/OR Seismic loading  
2. Application of thrust ram loads to advance the TBM | Detailing (recess geometry) and location of gasket groove (distance from corner) to reduce shear/tensile stresses; reinforcement detailing to accommodate loading |
6 Liquefaction Analysis (Subtask 5)

6.1 Subtask Scope

This section covers the impact for liquefaction potential of the soils along the alignment at approximately the depth of the tunnels under the 2,475-year event.

6.2 Liquefaction Risk

A liquefaction hazard analysis was also performed at the depth of the tunnel near Clifton Court. The results indicate the liquefaction hazard is very low. This is in line with previous findings, and as the increase in seismic energy from the 975-year event to the 2,475-year event is small, not unexpected.

Further analysis will be required as the design develops and further information is received.
7 Ground Motion Attenuation (Subtask 6)

7.1 Subtask Scope

This section contains the analysis of the attenuation of the anticipated ground motion as a function of depth, and contains a comparative analysis of the reductions in ground motion between the 975 and 2,475 events.

7.2 Reported Attenuations

Chapter 3 of the current July 2015 CER estimates the attenuation of the PGA with depth below ground surface. The CER estimates that at 100ft below grade, horizontal acceleration will be 70% of the acceleration at ground surface.

The detailed analyses performed as part of this sub-task indicate that a 50% reduction in PGA at 100 ft depth is acceptable for use in design. However, it should be noted that attenuation is highly variable, and is very sensitive to position with the soil column and should not be assumed to be the case at any particular depth. Therefore the 70% value included in the CER provides an appropriate level of conservatism appropriate for the preliminary level of engineering analysis. Also note that as the surface materials are prone to liquefaction, the assessment of attenuation needs to be subject to more detailed analysis as would be appropriate for future design development.
8 Ability of Backfill Grouting to Enhance Tunnel Durability (Subtask 7)

8.1 Subtask Scope

This sub-task scope includes the evaluation of different methods of backfill grouting and whether different backfill materials or methods may enhance the performance of the tunnel under a seismic event. Additionally, the strains developed in the LS-Dyna modeling are addressed and their impact on the backfill grout performance.

8.2 Annulus and Check Grouting

Two common types of annulus grouting are used in the industry. The first uses a mortar base, consisting primarily of a well graded sand, fly ash, bentonite, retarder, and a minor amount of cement paste. This is commonly referred to as ‘inert mortar’ as it is not chemically active and can often be held in extended suspension for extended periods of time on the BM, and its heavy body requires pressure pumps to inject. When injected, it remains a low mobility semi-fluid and allows the ground stresses to arch around the lining, resulting in lower overall ground loads to be imposed on the lining. For the purposes of this report, it has been assumed all primary backfill injections are performed through the TBM tailshield.

The second type of grouting for this application is pressurized, two component tailskin grouting (also referred to as an A/B Mix). The primary component is a thin cement slurry, generally consisting of cement and water, a retarder (usually a sugar), and bentonite. The secondary component is sodium silicate. When combined, the sodium silicate quickly reacts with the cement paste and binds the water in a solid matrix. The structure of the backfill appears like a fine-grained soft stone with compressive strengths of 300 to 500 psi, but in general are not required to be stronger or more resistant than the surrounding soil. The two-component backfill provides rapid support to the lining and bedding within the ground mass. Under this condition, ground loads are immediately imposed onto the tunnel lining, and is assumed to result in less stress redistribution in the ground than the inert mortar mixes.

If the backfill grout is approximated as a non-structural continuous ring of unreinforced weak concrete, the strain levels in tunnel will be directly superimposed onto the backfill, which will then exceed its tensile capacity and induce inelastic behavior (cracking). The extent of this cracking would be likely be distributed around the ring, but would not cause loss of confinement from the surrounding ground mass.

It is recommended that a stiff, homogeneous grout is used in order to ensure that a stiff and uniform backfill is applied around the tunnel. In consideration of the two general types of backfill grout currently employed in the industry, the two-component mix design does appear to have an advantage over the inert mortar mixes as they are more uniform in nature and stiffer (i.e. higher apparent modulus) than the inert mortar mixes. The addition of bentonite, which is normally already present in the two-component mixes, to the backfill annulus will lend itself to some amount of autonomous healing, though the extent and capability in-situ has not been well studied.
9 Current Industry Standards and Guidelines for Seismic Assessment of Tunnels (Subtask 8)

9.1 Subtask Scope

This section briefly describes the current industry standards used for the design of tunnels. In particular, tunnels that are considered as ‘critical infrastructure’ and their further classification as such is discussed. Current experience and industry practice with other agencies is also covered.

9.2 General

The seismic design of tunnels is generally based on the intended operational performance level the tunnel system must achieve in a post-earthquake disaster situation. This requires Seismic Performance Objectives to be selected for the tunnel system based on the level of consequence of failure.

There is yet no specific Code of Practice for the seismic design of tunnels around the world to follow the Performance Objective approach. However, a combination of relevant Building Codes and Guides or Technical Papers is commonly used in all projects for which the seismic performance of the tunnel is considered an important load case.

A key industry standard that follows the Performance Objective approach and complies various standards and guides that is particularly applicable to seismic design of tunnels in California, is given in Metro Rail Design Criteria Section 5 (2014) Part B of Chapter 3. This section formulates a coherent framework for the design and analysis of tunnels to assess their seismic performance, by compiling applicable local and international guides and standard.

9.3 Performance Objectives

The guiding philosophy of earthquake design for the tunnels is to provide a high level of assurance that the overall system will continue operating during and after an Operating Design Earthquake (ODE). Operating procedures assume safe shut down and inspection before returning to operation. Further, the system design needs to provide a high level of assurance that public safety to be maintained during and after a Maximum Design Earthquake (MDE).

The American Lifelines Alliance (2005) which is applicable to buried Carrier Pipes, can be used to define the ODE and MDE requirements for tunnels.

The ODE is defined for an earthquake that can reasonably expected to occur at least one during the design life of the tunnel. In probabilistic seismic hazard analysis (PSHA) the ODE is typically considered to have a return period of 475 years (10% probability of exceedance in 50 years). This analysis is aimed at reducing economic risk exposure and providing continuing operations while experiencing little or no damage.
A qualitative performance statement would typically include the following:

“Alignment distortions should be minor. Damage should be minor. Tunnel shall accommodate the stresses due to deformation resulting from ground motions with little to no damage. Gaskets in tunnel at joints shall accommodate deformations resulting from the ground motions with no reduction in sealing performance. Minor inelastic deformation and damage acceptable in the segmental tunnel lining. Only minor cracking in the shaft walls. Gaskets at shaft connections and the connections themselves should accommodate stresses and deformations resulting from the ground motions.”

The MDE is defined as representing the maximum expected earthquake that can be experienced at the site. The analysis is aimed at avoiding structural collapse of the tunnel. In PSHA the MDE is typically considered to have a return period of 2,475 years (2% probability of exceedance in 50 years).

A qualitative performance statement would typically include the following:

“Alignment distortions should not impact the function of the system. Damage should be limited and not impair the load-resisting capability of the structural system. Function of the gravity pipeline. Tunnel shall accommodate the stresses due to deformation resulting from ground motions with only minor damage that can be accessed and repairable. Gaskets in tunnel at joints shall accommodate deformations resulting from the ground motions with only minor reductions in sealing. Significant inelastic deformation in the segmental tunnel lining is acceptable but shall not impair the load-resisting capability of the tunnel. Significant racking in the shaft walls is acceptable, but not impair the load-resisting capability or sealing capacity of the lining. Gaskets at shaft connections and the connections themselves should accommodate stresses and deformations resulting from the ground motions.”

The American Lifelines Alliance (2005) also sets out the following Pipe Function Classes based on the requirement for post-earthquake operation which are assumed to be applicable to tunnels. These classes are based on Chapter 11 of ASCE Standard 7-10 – Minimum Design Loads for Buildings and Other Structures.

Table 9.1: Pipe Function Classes, American Lifelines Alliance (2005)

<table>
<thead>
<tr>
<th>Pipe Function Class</th>
<th>Seismic Importance</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Very low to None</td>
<td>Pipelines that represent very low hazard to human life in the event of failure. Not needed for post earthquake system performance, response, or recovery. Widespread damage resulting in long restoration times (weeks or longer) will not materially harm the economic well being of the community.</td>
</tr>
<tr>
<td>II</td>
<td>Ordinary, normal</td>
<td>Normal and ordinary pipeline use, common pipelines in most water systems. All pipes not identified as Function I, III, or IV.</td>
</tr>
<tr>
<td>III</td>
<td>Critical</td>
<td>Critical pipelines serving large numbers of customers and present significant economic impact to the community or a substantial hazard to human life and property in the event of failure.</td>
</tr>
<tr>
<td>IV</td>
<td>Essential</td>
<td>Essential pipelines required for post-earthquake response and recovery and intended to remain functional and operational during and following a design earthquake.</td>
</tr>
</tbody>
</table>
These Pipe Function Classes are analogous to the Risk Categories as set out in the International Building Code (2012) Chapter 16 Section 1604.5 and the ASCE Standard 7-10 (Table 1.5-1), which again are based on the nature of occupancy or emergency response function.

With increased functional importance, the requirement for operational reliability increases. Industry guidelines consider tunnel reliability in relation to a probability of exceedance of a seismic event in a time period or a return period of a certain seismic event.

Table 9.2: Earthquake Hazard Return Period by Pipe Function Class, American Lifelines Alliance (2005)

<table>
<thead>
<tr>
<th>Pipe Function Class</th>
<th>Probability of Exceedance P in 50 years</th>
<th>Return Period $T$ (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>100%</td>
<td>Undefined</td>
</tr>
<tr>
<td>II</td>
<td>10%</td>
<td>475</td>
</tr>
<tr>
<td>III</td>
<td>5%</td>
<td>975</td>
</tr>
<tr>
<td>IV</td>
<td>2%</td>
<td>2,475</td>
</tr>
</tbody>
</table>

As addressed in this memorandum, the WaterFix tunnel is assumed to be classified as a Class IV structure and therefore requires a design that follows the MDE’s 2 475-year return period) described above.

9.4 Analysis Method

In accordance with Metro Rail Seismic Design Criteria (2014), the general procedure for seismic design of underground structures is based primarily on the ground deformation approach specified herein. During earthquakes, underground structures move together with the surrounding geologic media. Therefore, the structures are designed to accommodate the deformations imposed by the ground taking into consideration the effects of soil-structure interaction.

Underground tunnel structures undergo three primary modes of deformation during seismic shaking: ovaling/racking, axial, and curvature deformations. The ovaling/racking deformation is caused primarily by seismic waves propagating perpendicular to the tunnel longitudinal axis. Vertically propagating shear waves are generally considered the most critical type of waves for this mode of deformation as given by Wang (1993) and Hashash et al. (2001):
Curvature deformations are induced by components of seismic waves that propagate along the longitudinal axis, as shown in Figure 9.2:

For bored circular tunnels such as the WaterFix soft ground tunnel sections, using precast concrete segmental linings, design for reinforced concrete tunnel lining is carried out in accordance with the provisions of the California Building Code and Caltrans implemented AASHTO LRFD. Additional guidance in tunnel applications not generally covered in bridge and building codes is presented in FHWA-NHI-09-010 Report, “Technical Manual for Design and Construction of Road Tunnels”, Chapter 10. The design shall also comply with the requirements specified in Metro Rail Design Criteria Section 5 – Structural/Geotechnical.
The analysis of the tunnel due to imposed ground deformations due to either ODE or MDE earthquakes can be done in two principal methods:

1) **Closed Form Solutions:** The seismic ovaling loads for the lining of bored circular tunnels is defined in terms of change of tunnel diameter ($\Delta D_{EQ}$) caused by the vertically propagating shear waves of the MDE and ODE ground motions. $\Delta D_{EQ}$ can be considered as seismic ovaling deformation demand for the lining.

Further details of the close form solution procedure can be found in Metro Rail Seismic Design Criteria (2014) Chapter 3B7.1.1.

2) **2D or 3D Numerical Soil-Structure Interaction Analysis:** The bored tunnel cross-section subjected to ovaling deformation resulting from the surrounding ground. In general, this analysis can be done as a pseudo-static analysis in which displacements or displacement time histories are statically applied to the soil-structure system. However, recently and with the advancement of understanding and computing power, dynamic Non-Linear Time History Analysis (NLTHA) is used in which the ground is excited by a set of ground motions applied at the based on the model and acting on an embedded 2D or 3D structural model of the tunnel. The ground motions should be spectrally matched for both the ODE and MDE.


### 9.5 Design Checks

#### 9.5.1 Seismic Demands

The output from the closed form solutions or numerical analysis listed in Section 9.4 above should be used to extract axial and curvature deformation along the tunnel as well as radial ovalization for the tunnel cross section.

The seismic demands should include axial and shear forces and bending moments acting in the tunnel segments and the behavior of the tunnel joints due a combination of the various deformation modes experienced by the tunnel as described in Metro Rail Seismic Design Criteria (2014) Chapter 3B7.2.

#### 9.5.2 Stability Check

There are two levels of stability checks for the tunnel liner design based on the performance criteria for the ODE and MDE seismic ground motions.

Under the ODE there will be no to minimal damage to the lining segments, joints, and watertightness. The tunnel will be able to be put back in service after a post-earthquake inspection. Under the MDE the criteria are no collapse and being able to evacuate the tunnel safely immediately after the MDE.
Technical Memorandum

237477 July 6, 2018

Inelastic deformations and damage are allowed but any structural damage shall be controlled and limited to elements that are easily accessible and can be readily repaired. No collapse mechanisms are allowed.

For both ODE and MDE load cases, the combination of seismic demands calculated as described in Section 9.5.1 above should be used to check the section capacities relative to AASHTO LRFD as modified for tunnels in the AASHTO Technical Manual for Design and Construction of Road Tunnels (FHWA) (2009) Section 10.3.3.

The following stability criteria are specified in the Metro Rail Seismic Design Criteria (2014) Chapter 3B7.2:

a) For the ODE level ground motions, design the lining to respond essentially in an elastic manner with no ductility demand. Do not exceed a concrete compression strain of 0.001. Do not exceed a steel tensile strain of 0.002.

b) For the MDE level design, inelastic deformations are allowed, but kept to the acceptable levels. Concrete compression strain shall not exceed 0.02. Steel tensile strain shall not exceed 0.006.

For the MDE level design, the concrete strain may be allowed to exceed 0.002 but not to exceed 0.004 provided that the strain is predominantly in flexural mode.

c) For joints without being specifically designed as ductile connection, no uplift (zero tension) is allowed across the joint. Joint shear capacity should be checked. Shear friction approach using the compressive load across the joint may be used to check the joints shear capacity. Joints compressive and bearing capacity relative to unreinforced concrete should be checked, unless specifically designed bearing plates and confinement is provided.

d) If segment joints are specifically designed as ductile connection, the concrete and steel strain limits given above may apply to the joints. Connection so the steel crossing the joint shall be designed to develop 1.25 times the yield strength of the steel without brittle failure of the concrete at the anchors.

9.5.3 Additional checks

The following additional checks should be carried out in addition to the basic checks specified above:

1) Interfaces - Interfaces between the bore tunnel and the more massive structures shall be designed as flexible/expansion joints to accommodate the differential movements.

2) Geological variations - Abrupt changes in stiffness of geologic formations shall be accommodated by designing the structures in these formations for the static and seismic loads and deformations resulting from such variations. The focus in this case is on the longitudinal response of the tunnel to the differential free-field deformations between the soil (a soft formation) and the rock (a stiff formation).

Further details are provided in Metro Rail Seismic Design Criteria (2014) Chapters 3B7.3 and 3B7.4.
9.6 Key Industry Standards and Guides

The following key industry standards and guidelines should be used as minimum for assessing the seismic performance of the WaterFix Tunnel:


c) Metro Rail Design Criteria Section 5 Structural, Appendix Metro Supplemental Seismic Design Criteria (SSDC) – Chapter 3 Part B Metro SSDC for Underground Structures.


e) Seismic Analysis and Design of Retaining Walls Buried Structures (2008), Slopes, and Embankment, NCHRP Report 611 Transportation Research Board.


10 Conclusions and Recommendations

10.1 Conclusions

The studies in this report demonstrate that the impact of changing the seismic demand from a 975-year event to 2,475-year report can be accommodated with the tunnel segment designs currently being considered for the California WaterFix. The analysis examined the Alternative 1 design as presented in the 2015 CER, Appendix I, without tension connectors across the segment joints. The addition of guide rods or other means to provide shear resistance along the longitudinal joints should improve the seismic performance of the joints, and it is recommended these are evaluated during detailed design.

The time-history analysis runs completed demonstrate that under transient conditions, increases joint openings are possible, but rapidly fall to very minor levels with attenuation of seismic motions. Permanent deformations are on the order of 0.5mm (0.02 in) and should prevent any uncontrolled leakage into or out of the twin tunnels.

Under seismic conditions, catastrophic failure of the lining is highly unlikely due to the low ovalization and lining stresses observed during the analysis, maintaining service after such an event. However, detailed analysis focusing on the performance of the joints should be performed in the design phase to verify that any inelasticity or stress concentrations are within acceptable performance limits.

It should be noted that the analyses and considerations outlined in this report are in support of the concept presented in the CER 2015 report. Further detailed work will be required to develop the concepts into the design phase of the project.

10.2 Recommendation

As the project develops, further work and analyses will be required and as a minimum will need to include the following:

- Further Site Investigations will be needed. It is likely several phases of investigations will be required including additional deep boreholes, geophysics, and comprehensive lab testing to develop a full understanding of the non-linear behavior of the soil under static and dynamic conditions.
- Detailed Seismic Hazard Study. The DRMS report needs to be updated to include recent developments and understanding of the geologic environment. Detailed studies will need to be carried out at all points of significant infrastructure.
- Extensive Hydraulic studies, including Surge Analysis, to input into the tunnel design.
- Detailed Tunnel design looking to take the designs outlined in the concept report to full final design. Optimization of segments to take the operational and construction cases into consideration.
Appendix A Joint Details from Previous Projects

Lake Mead Intake No.3
Technical Memorandum

237477    July 6, 2018

Crenshaw Corridor

TYPICAL RADIAL BOLTING - SECTION DETAIL

DEVELOPED PLAN VIEW ON INTRADOS

DRAFT
Westside Extension
Eurasia Tunnel

Technical Memorandum

237477 July 6, 2018

Anup | F0.15 Page 43 of 46