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

DHCCP Team

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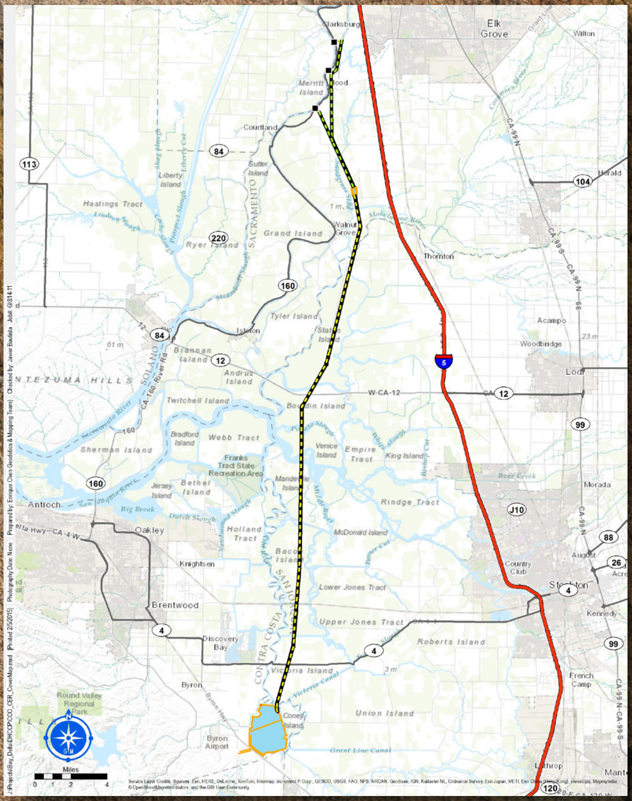

Delta Habitat Conservation & Conveyance Program (DHCCP)

Final Draft: July 1, 2015

1

Conceptual Engineering Report

Modified Pipeline / Tunnel Option – Clifton Court Forebay Pumping Plant
Volume 1 – Conceptual Engineering Report

901 P Street Suite 411 B, Sacramento, CA 95814

Conceptual Engineering Report

Dual Conveyance Facility Modified Pipeline/Tunnel Option- Clifton Court Forebay Pumping Plant (MPTO/CCO)

July 1, 2015

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DHCCP: 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

Information from the Draft ATO CER, the PTO CER and the MPTO 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 and the MPTO 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 or the MPTO 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 Delta Habitat Conservation and Conveyance Program (DHCCP) in order to assist the Lead Agencies in their decision making process. It is anticipated that this document would be used to support engineering and design as well as the Environmental Impact Statement (EIS) and Environmental Impact Report (EIR) for the Bay Delta Conservation Plan (BDCP). It is also expected that the Lead Agencies, the EIR/EIS consulting team, and other stakeholders in the BDCP process would recommend modifications to these facilities over the coming months, and such changes would be evaluated as needed to support the EIR/EIS.

This document is part of an iterative process of developing options that can be used as alternatives in the EIR/EIS. Therefore, all locations, dimensions, quantities, design concepts, construction techniques, and 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.

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Volume 2: Concept Drawings (Bound Separately)

Volume 3: Map Book (Bound Separately)

Acronyms and Abbreviations

ACI	American Concrete Institute
AF	acre-feet
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASTM	ASTM International (formerly American Society for Testing and Materials)
ATO	All Tunnel Option
ATS	Automatic Transfer Switch
AWWA	American Water Works Association

Banks PP	Harvey O. Banks Pumping Plant
BBID	Byron Bethany Irrigation District
BDCP	Bay Delta Conservation Plan
BEP	best efficiency flow capacity point
bgs	below ground surface
BMP	best management practice

Cal/OSHA	State of California Division of Occupational Safety and Health
CALFED	California Bay-Delta Authority
CCF	Clifton Court Forebay
CCO	Clifton Court Option
CER	conceptual engineering report
CFD	computational fluid dynamics
cfs	cubic feet per second
CIP	cast-in-place
CPT	cone penetrometer test
CVFPB	Central Valley Flood Protection Board
CVP	Central Valley Project
cy	cubic yard

DCC	Delta Cross Channel
Delta	Sacramento River and San Joaquin River Delta
DHCCP	Delta Habitat Conservation and Conveyance Program
DMC	Delta-Mendota Canal
DSOD	Division of Safety of Dams
DRMS	Delta Risk Management Strategy
DWR	California Department of Water Resources

E&W	East and West
EBMUD	East Bay Municipal Utility District
EIR	environmental impact report

EIS	environmental impact statement
EPB	earth pressure balance
FB	freeboard
FFTT	Fish Facilities Technical Team
fpm	feet per minute
fps	feet per second
FRWA	Freeport Regional Water Authority
g	measurement of peak ground acceleration
gpd	gallons per day
ghg	greenhouse gas
GHGRP	Greenhouse Gas Reduction Plan
gpm	gallons per minute
GPS	global positioning satellite
HI	Hydraulic Institute
HGL	hydraulic grade line
H:V	horizontal to vertical ratio
HMA	hot mix asphalt
HMI	Human Machine Interface
hp	horsepower
HPU	hydraulic power unit
HVAC	heating, ventilation, and air conditioning
ID	inside diameter
ICF	Isolated Conveyance Facility
IPP	Intermediate Pumping Plant
IF	Intermediate Forebay (Intermediate Pumping Plant Forebay)
JOC	Joint Operations Center
Jones PP	C.W. “Bill” Jones Pumping Plant
km	kilometer(s)
kV	kilovolt
LCI	Load commutated inverter
MCC	motor control center
MCE	Maximum Considered Earthquake
mm	millimeter(s)
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
SLDMWA	San Luis and Delta-Mendota Water Authority
SLR	sea level rise

SPB	Slurry pressure balance
SMUD	Sacramento Municipal Utility District
SPRR	Southern Pacific Railroad
SWP	State Water Project
SWPPP	Stormwater Pollution Prevention Plan
TBD	to be determined
TBM	tunnel-boring machine
TDH	total dynamic head
TEWAC	totally enclosed water-to-air-cooled
UPRR	Union Pacific Railroad
URS	URS Corporation
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
VFD	variable frequency drive
w day	working day
WAPA	Western Area Power Administration
WGCEP	Working Group on California Earthquake Probabilities
WSE	water surface elevation
%	percent(ile)
24/7	24 hours per day/7 days per week

Executive Summary

The Bay Delta Conservation Plan (BDCP) is an effort by federal and state agencies and other stakeholders that began in 2006 to stabilize water deliveries from the Delta while enhancing the Delta ecosystem. In July 2012, Governor Edmund G. Brown, Jr. and United States Secretary of the Interior Ken Salazar reaffirmed both the State and Federal commitment to the BDCP as a comprehensive solution to achieve the dual goals of a reliable water supply for California and a healthy California Bay Delta ecosystem that supports the State's economy.

A cornerstone of the BDCP strategy is BDCP Conservation Measure 1 (CM1), which focuses on the construction and operation of a dual-conveyance water delivery system that would modernize the heart of California's aging water supply network in a way that balances the needs of the Delta ecosystem and California's water supplies. The Delta Habitat Conservation and Conveyance Program (DHCCP), which was in place by 2008, was formed to manage a number of activities that support the BDCP in general and CM1 in particular, including engineering, real estate services, identification of habitat restoration opportunities, and preliminary designs for water conveyance facilities. As part of the DHCCP effort, conceptual engineering information was and continues to be needed to support the development of the environmental impact statement (EIS) and environmental impact report (EIR) required under the BDCP.

A significant part of the conceptual engineering effort needed to support BDCP CM1 involves facility and conveyance system design. The BDCP, however, and the design activities needed to support CM1, have evolved over the years, due primarily to additional engineering analyses, landowner concerns, and public comment:

1. All Tunnel Option (ATO): This was the original concept as described in the March 2010 Conceptual Engineering Report (CER). As the name indicates, this option relied primarily on tunnels to convey the water through the Delta system.
2. Pipeline Tunnel Option (PTO): This alternative, described in the October 2010 CER, included both pipelines and tunnels for conveyance purposes.
3. Modified Pipeline Tunnel Option (MPTO): The MPTO concept, which was the subject of the October 2013 CER, made significant changes to the earlier concepts, including reducing the number of intakes, increasing the size of the tunnels in the gravity-feed portion of the system, decreasing the size of the intermediate forebay, and eliminating an intermediate pumping plant.

Information regarding each of these alternatives is presented in separate CERs and supporting documents that are available on the Bay Delta Conservation Plan website.

This CER addresses the latest alternative, identified as the "Dual Conveyance Facility Modified Pipeline/Tunnel Option – Clifton Court Forebay Pumping Plant Option," or "MPTO/CCO." This latest option optimizes the earlier MPTO design concept to better utilize the Clifton Court Forebay, based on information obtained from engineering analyses that evaluated locating the pump plants at the Sacramento River vs. adjacent to Clifton Court Forebay. The CER provides new text and figures to reflect the changes to the conveyance facilities resulting from the optimization in alignment and features, including the following:

- Larger north tunnels for gravity feed system
- Reduction of the internal hydrostatic head within the tunnel system
- Optimized intermediate forebay
- Consolidated pumping plant at Clifton Court Forebay (CCF)
- Modification to the CCF
- Elimination of the pumping plants at the intakes.

This CER is a conceptual engineering effort. Facility locations, dimensions, and elevations (both topographic and facility) are approximate and subject to change during the preliminary engineering phase.

ES.1 Project Overview

The MPTO/CCO described in this CER is an isolated facility component of the Dual Conveyance with Pipeline/ Tunnel alternatives in the BDCP EIR/EIS, and it is one alternative configuration of the Sacramento River and San Joaquin River Delta (Delta) intake and conveyance facilities described in the EIR/EIS. The MPTO/CCO overall alignment is shown in Figure ES-1.

The MPTO/CCO will include the following:

- 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 provide for gravity flow delivery through dual Main Tunnels to the North Clifton Court Forebay.
- A Pumping Plant located at the northeast corner of Clifton Court Forebay (CCF).
- CCF will be divided into two parts: North Clifton Court Forebay (NCCF) and South Clifton Court Forebay (SCCF). These forebays are in the south Delta, near the existing State Water Project (SWP) Harvey O. Banks Pumping Plant (Banks PP) and federal Central Valley Project (CVP) C.W. “Bill” Jones Pumping Plant (Jones PP) approach canals and will provide storage and flow regulation. NCCF will receive the flow from the Intake Facilities; SCCF will function as a replacement of the current CCF. SCCF will consist of the southern portion of the existing CCF, with expansion to the south into Byron Tract 2.

ES.1.1 MPTO/CCO Assumptions

The facilities included in the MPTO/CCO assume the following:

- The MPTO/CCO delivers 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 MPTO/CCO must be able to deliver this flow rate more than 99 percent of the time.
- The MPTO/CCO is engineered to:
  - Transport water in conveyances isolated from existing rivers and sloughs.
  - Divert water through fish-screened intakes on the Sacramento River.
  - Deliver water to the SWP and CVP export pumping plant approach canals downstream of their respective fish collection facilities.
- Be protected against a 200-year flood event with the sea level rise (SLR) predicted from climate change.
- Use gravitational flow through the Main Tunnels.

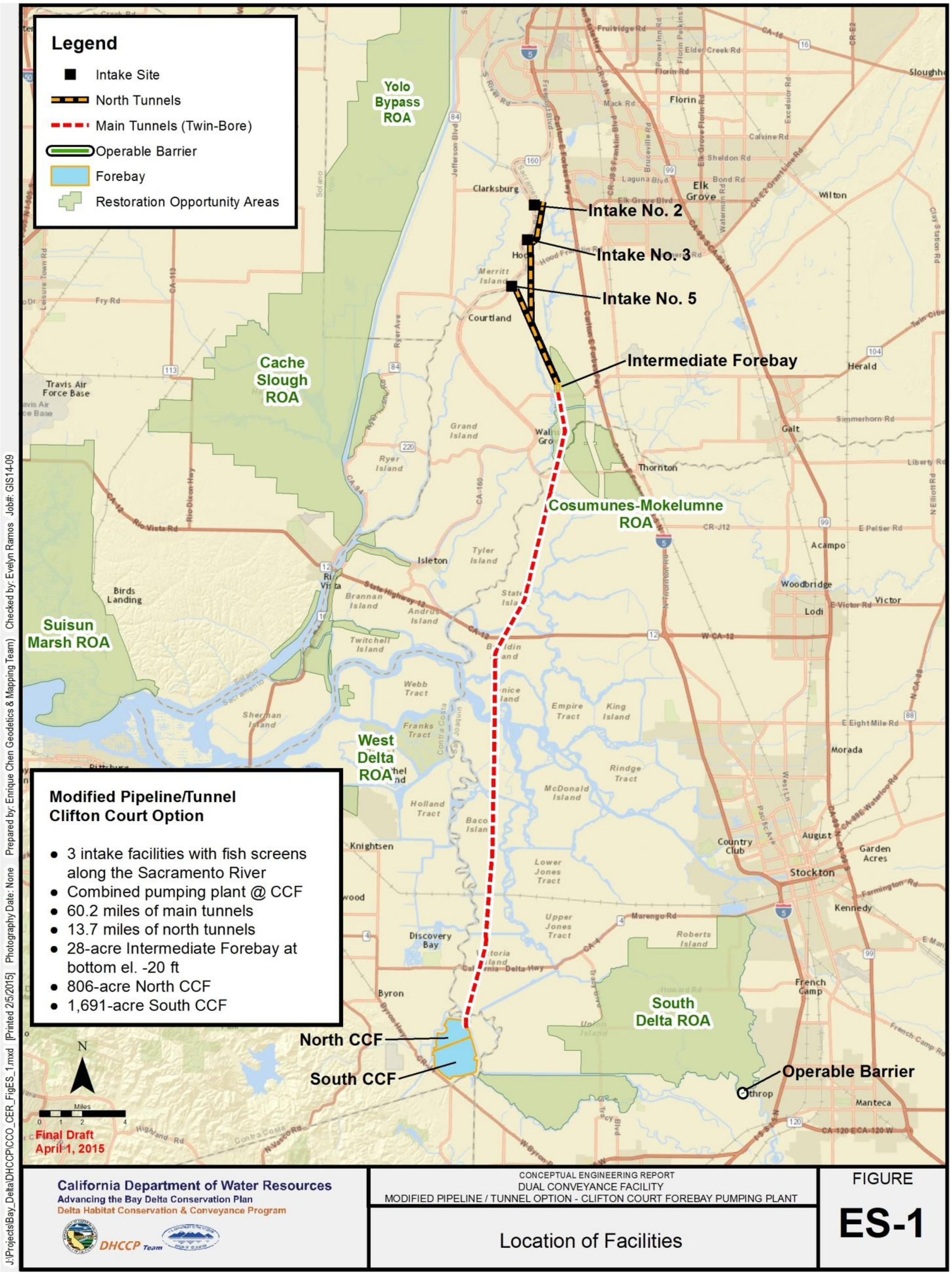
ES.1.2 Implications of MPTO/CCO on Current SWP and CVP Operations

The MPTO/CCO facilities conveying water to the SWP and CVP export pumping plants are as follows:

- Fish-screened Intake Facilities between Sacramento River Miles 36 and 42.
- Isolated conveyance system with an IF.
- Create NCCF from the northern part of the existing CCF.

The MPTO/CCO changes the following conveyance factors:

- Operating volume of the NCCF is significantly less than the existing CCF.



Modified Pipeline/Tunnel Clifton Court Option

- 3 intake facilities with fish screens along the Sacramento River
- Combined pumping plant @ CCF
- 60.2 miles of main tunnels
- 13.7 miles of north tunnels
- 28-acre Intermediate Forebay at bottom el. -20 ft
- 806-acre North CCF
- 1,691-acre South CCF

Figure ES- 1: Location of Facilities

ES.1.3 Implications of MPTO/CCO on Current California State Water Code

The MPTO/CCO 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 MPTO/CCO Component Descriptions

Table ES-1 summarizes the physical characteristics of the Intake Facilities, North Tunnels, IF, Main Tunnels, Clifton Court Pumping Plant (CCPP), and NCCF. Figure ES-2 illustrates a conveyance schematic of the MPTO/CCO, with references to sections within this CER where detailed facility information is presented.

Table ES-1: Summary of Modified Pipeline/Tunnel Clifton Court Option Physical Characteristics

Feature Description	Approximate Characteristics
Overall Project	
Conveyance Capacity	9,000 cfs
Overall Length of All Tunnels	73.9 miles
Intake Facilities	
Number of In-River-Screened Intakes	3
Flow Capacity at Each Intake	3,000 cfs
Clifton Court Pumping Plant	
Total Number of Pumps (both Pumping Plants)	12
8 Large Pumps, Capacity per Pump (cfs)	1,125
4 Small Pumps, Capacity per Pump (cfs)	563
Total Dynamic Head	37 feet
Estimated Pump Load (MW, both Pumping Plants)	36

Table ES-1 (CONTINUED): Summary of Modified Pipeline/Tunnel Clifton Court Option Physical Characteristics

Feature Description	**Approximate Characteristics**
North Tunnels from Intakes to Intermediate Forebay	
North Tunnel – Intake No. 2 (Connecting to a Junction Shaft near Intake No. 3)	
Maximum Flow	3,000 cfs
Tunnel Length	1.99 miles
Number of Tunnel Bores; Number of Shafts (total)	1; 1
Tunnel Finished Inside Diameter	28 feet
North Tunnel – Intake No. 3 (from Junction Shaft to IF)	
Maximum Flow Tunnel (Intake Flow)	6,000 cfs (3,000 cfs)
Tunnel Length	6.74 miles
Number of Tunnel Bores; Number of Shafts (total)	1; 3
Tunnel Finished Inside Diameter	40 feet
North Tunnel – Intake No. 5	
Maximum Flow	3,000 cfs
Tunnel Length	4.77 miles
Number of Tunnel Bores; Number of Shafts (total)	1; 3
Tunnel Finished Inside Diameter	28 feet
Intermediate Forebay	
Surface Area at River El. 10’, IF WSE at 0’, 9,000 cfs conveyance	37 acres
Main Tunnels	
Number of Tunnels	2; parallel
Tunnel Length (each)	30.1 miles
Number of Tunnel Bores; Number of Shafts (total per tunnel bore)	2; 9
Tunnel Finished Inside Diameter	40 feet
North Clifton Court Forebay	
Surface Area at Maximum Operation Level	806 acres
Active Storage	4,970 to 8,100 AF
South Clifton Court Forebay	
Surface Area at Maximum Operation Level	1,691 acres
Active Storage	12,050 AF

Notes:
AF = acre-feet
cfs = cubic feet per second
ft = feet
MVA = Mega Volt Ampere

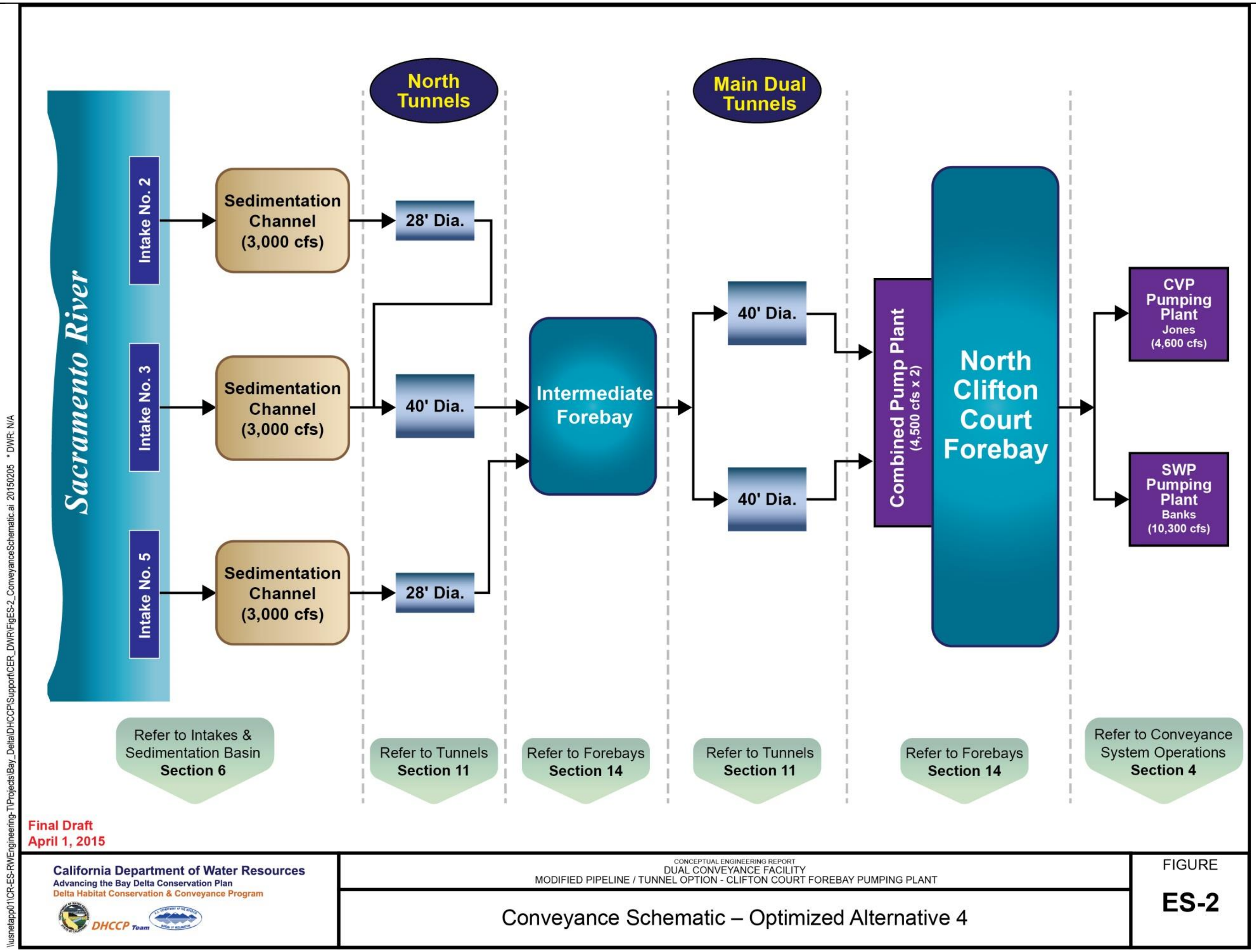


Figure ES- 2: Conveyance Schematic

ES.2.1 Intakes

The three 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). The Intake Facilities will be located along the Sacramento River, at sites selected in coordination with the FFTT. The MPTO/CCO intake locations were initially selected as the Proposed Project for conveyance by DWR in consultation with the FFTT through various studies and previous CERs associated with the EIR/EIS. Intake numbering is consistent with the earlier Pipeline/Tunnel Option (PTO) CER numbering system.

Each Intake Facility will consist of the following:

- A fish-screened intake structure that employs state-of-the-art on-bank fish screens.
- Twelve large gravity collector box conduits that will extend through the levee to convey flow to the sedimentation system.
- A sedimentation system consisting of gravity settling basin to capture sand-sized sediment and a drying lagoon for sediment drying and disposal.

The sedimentation basins will provide the transition from the collector box conduits to an outlet shaft that will discharge into a tunnel leading to the IF. A substation with transformers and switching equipment will be located on each site for electrical power supply.

ES.2.2 North and Main Tunnel Alignments

The proposed conveyance tunnels consist of the North Tunnels, which consist of three separate tunnel reaches totaling approximately 14 miles that connect the three Intake Facilities to IF, and two parallel Main Tunnels to the NCCF, each approximately 30 miles long.

The North Tunnels are two single-bore 28-foot and one single-bore 40-foot inside diameter (ID) tunnels. The Main Tunnels are twin-bore 40-foot inside diameter tunnels. The inlets and outlets will be equipped with isolation structures to allow the tunnels to be dewatered, maintained, and inspected.

ES.2.2.1 North and Main Tunnel Construction Considerations

The compatibility of the tunneling excavation method with anticipated ground conditions is a critical design and 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.

It is assumed that two contractors will construct all shafts and tunnels for the North Tunnels (Reaches 1 through 3). For the Main Tunnels, a different contractor for each of the four reaches will construct all drive and vent shafts and both tunnels (4 contractors total). The reception shafts will be constructed by the contractor of the adjoining reach, except for the Bacon Island and Staten Island shafts, which will be constructed by one of the two adjoining reach contractors. See Section 3.2, Figure 11-1, and Table 11-1, for more information on tunnel reaches.

Considering an estimated construction schedule (discussed in Section ES.4), using industry average tunneling rates for the anticipated ground conditions, and including contracting considerations, 10 to 11 tunnel-boring machines (TBM) may be required. The lead time required to design, build, and ship new machines will be approximately one year, with an additional six 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 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 MPTO/CCO schedule.

ES.2.3 Intermediate Forebay

The IF is on the Glanville Tract, east of the Pearson District and west of Interstate 5. The IF serves as an atmospheric break in the system from the inlet to the dual Main Tunnels. This break in the system allows the flows from each Intake to merge and distribute equally to each barrel of the Main Tunnels, improving operational stability in the Clifton Court pumping plant, and allows independent operation of each of the North Tunnels and the Main Tunnels.

The IF has no regulating gates controlling gravitational flow to the Main Tunnels; therefore, no daily operational storage is necessary at IF beyond that necessary to accommodate water surface changes at the downstream NCCF. The IF at bottom elevation -20 feet, at 28 acres, 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 North Clifton Court Forebay

The NCCF 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. Preliminary calculations indicate an operational storage capacity range of approximately 4,970 to 8,100 acre-feet (AF), with an approximate water storage surface area of nominally 806 acres, depending on depth.

Constraints on the exporting pumping plants fixed a normal forebay operating range of 7.0 feet (elevation +0.50 to +7.5 feet). This operating range results in approximately 4,970 AF of potential active storage in NCCF. Additional operating storage up to 8,100 AF can be obtained by operating NCCF at a range of up to 9.0 feet, which is within the efficient operating range of both NCCF and the export pumping plants.

NCCF is connected with new control structures and canals to the existing approach canals to the export pumping plants.

ES.2.5 South Clifton Court Forebay

The SCCF is designed to be hydraulically dependent on Delta waterways and retain the same operation criteria as the existing CCF. The SCCF will include part of Byron Tract Island located on the south side of the existing CCF. The SCCF will draw its supply through the West Canal using intake gates and it will deliver flow to Banks PP. SCCF has an approximate water storage surface area of nominally 1,691 acres, depending on depth.

Constraints on the exporting pumping plants limit the normal operating range to 7.0 feet (elevation +1.1 to +8.1 feet). This operating range results in approximately 12,050 AF of potential active storage in SCCF. Additional operating storage can be obtained if the operating range is increased, which appears feasible.

ES.3 Other Design Considerations

ES.3.1 Flood Protection Considerations

The MPTO/CCO is engineered to withstand flood water levels from the following potential sources:

- 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 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.

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 design level of ground motion for the Intake Facilities has a 10 percent chance of being exceeded in 50 years, while the design level for the tunnels has a 5 percent chance of being exceeded in 50 years. At the forebays, DWR Division of Safety of Dams (DSOD) criteria mandate the use of deterministically derived ground motions for design (between the mean and one standard deviation above the mean) that are derived from analysis of nearby fault sources using attenuation relationships.

ES.3.3 Hydraulic Calculations

Hydraulic modeling of the MPTO/CCO system was employed to establish the system head curves to support Intake Facilities, North Tunnel inside diameter, IF elevations and pump selection and to determine pressures within the system (DWR, 2014).

In addition, conceptual surge analysis studies were completed under a variety of conditions to verify IF water level fluctuations and to size pump shafts and the Clifton Court pumping plants.

ES.3.4 Instrumentation and Controls

Each MPTO/CCO 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

Electrical supply is required during construction and for MPTO/CCO operation. 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 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 temporary (or permanent, in some cases) 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.

Three electric utility transmission service providers could provide transmission interconnection and services to deliver electrical power to the project: Pacific Gas and Electric (PG&E), Sacramento Municipal Utility District (SMUD), and Western Area Power Administration (WAPA). 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.

DWR’s SWP Power and Risk Office (PARO) will lead the process of identifying, evaluating, and establishing the electrical interconnection of this project to the California electric grid. PARO will also lead the process of planning and obtaining the power needed to construct the project. The long-term power needed to operate the project will be provided from the power portfolios of CVP and SWP in proportion to their participation in the project.

ES.4 Construction Considerations

ES.4.1 Borrow Sites

Borrow materials will be required for forebay and overflow containment area embankments at IF, NCCF and SCCF, 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 MPTO/CCO 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 the Map Book (Volume 3). 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

Significant thicknesses of non-supportive or organic soils may be removed in the course of forebay, pumping plant, and shaft construction operations. Large volumes (approximately 30.7 million cubic yards (cy)) of re-usable tunnel material (RTM) consisting of saturated soils mixed with biodegradable polymers will be generated by tunneling operations. Large volumes (approximately 8 million cy) of dredged material are expected to be removed from NCCF and SCCF. Smaller quantities of excess excavated materials are expected at other MPTO/CCO facility sites. These soils, whether unsuitable for use as engineered fill or in excess of embankment or fill pad volumes, will need to be disposed.

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 NCCF and SCCF areas will be disposed of in an adjacent common disposal/borrow area on Byron Tract, northwest of NCCF, between Byron Highway and Italian Slough. Unsuitable excess excavated material at the northern MPTO/CCO facility sites (Intake Facilities, North Tunnels, and IF) will be disposed in the designated RTM disposal area near or at the IF. Suitable materials may be reused for the fill pads of the Intake Facilities.

ES.4.3 Construction Packages, Sequencing, and Schedule

The overall schedule for implementation of the MPTO/CCO will be determined during subsequent phases of this project. 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.

ES.4.4 Impact to Existing Facilities

ES.4.4.1 Highways

Highway 160 (SR 160), a state highway, will be impacted by 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.

Byron Highway, south west of the existing CCF, will both be rerouted to accommodate the construction of a proposed siphon connecting the new NCCF to the existing SWP approach channel. See Section 12.0 “Bridges-Road and Railroad” for more details.

Other state highways, plus county and private roads, will be impacted by construction traffic and might need temporary improvements or restoration after the work, but no other significant relocations have been identified.

ES.4.4.2 Levees

The new facilities will interface with the Sacramento River levee regulated by the Central Valley Flood Protection Board (CVFPB) and United States Army Corps of Engineers (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.

All work on the Sacramento River levees will be conducted in accordance with CVFPB and USACE requirements.

Some work will also be required on levees operated by DWR and the Bureau of Reclamation (Reclamation) in the south Delta, adjacent to the expanded Clifton Court Forebay. All work on the south Delta levees will be conducted in accordance with DWR and Reclamation requirements, as applicable.

ES.5 Temporary and Permanent Footprints

Construction of the MPTO/CCO components will result in temporary construction and permanent facility footprints. Some of the temporary acreages are assessed as permanent for the purpose of proposed mitigation. Table ES-2 summarizes the maximum projected footprint acreage for each MPTO/CCO component and a project total. Major assumptions involved in the generation of construction footprints are described in Section 23.0, “Stockpiles, Haul Routes, and Other Construction-Related Elements.”

Table ES- 2: Projected Construction and As-constructed Footprint for MPTO/CCO Facility Components

Facility Component	**(Temporary) During Construction Acreage**	**(Permanent) As-constructed Acreage**[a]
Intake Facilities	152	366
North Tunnels	323	237
Intermediate Forebay [b]	39	648
Main Tunnels	276	2,749
North and South Clifton Court Forebays	2,145	2,121
Overall MPTO/CCO Project	2,935	6,121

[a] Includes disposal areas for all RTM plus unsuitable and excess excavated material, as applicable. Some of the impacted acres are assessed as permanent for the purpose of proposed mitigation, although they could be temporary in nature.

[b] Includes main electrical substation area nearby.

CCF = Clifton Court Forebay

IF = Intermediate Forebay

ES.6 Construction Schedule

The construction schedule for the MPTO/CCO components is shown on Table ES-3:

Table ES- 3: Projected Construction Schedule for MPTO/CCO Facility Components

Facility Component	Start Date Month	Year[a]	Durations[b] Days
Procurement			
Pumps	January	Year 1	824
Construction			
Site Prep, Roads, Barges, Utilities	April	Year 1	191
Temporary Power	May	Year 1	504
Intakes	January	Year 4	2489
Pipelines and Transitions	August	Year 9	486
North Tunnels	July	Year 2	2,088
Intermediate Forebay	July	Year 6	1,346
Main Tunnels	January	Year 2	2,495
Pumping Plants	January	Year 2	2,111
North and South Clifton Court Forebays	January	Year 5	1,370
Permanent Power	December	Year 4	1990
Start Up Commissioning	April	Year 11	262

[a] Year 1 is currently representing 2018

[b] Some duration include float days since some activities relies on the completion of others.

SECTION 1.0

Introduction

1.1 Program Overview

This document describes the facilities that make up the Modified Pipeline/Tunnel Option (MPTO)/Clifton Court Option (CCO), referred to collectively as the MPTO/CCO. The MPTO/CCO is an isolated facility component of one of the Dual Conveyance with Pipeline/Tunnel alternatives in the Bay Delta Conservation Plan (BDCP) EIR/EIS, and it is one of the alternative configurations of the Delta intake and conveyance facilities described in the EIR/EIS.

The material herein describes the preliminary conceptual engineering of MPTO/CCO facilities currently under investigation.

1.2 Purpose and Scope of Conveyance Option

The preliminary concept for the MPTO/CCO is to convey up to 9,000 cubic feet per second (cfs) of Sacramento River water through an underground conveyance system across the Delta to pumping plants in the South Delta. The purpose of the overall system is to gain a sustainable and reliable water supply capable of withstanding earthquake and climate.

The MPTO/CCO includes the following components:

- Three North Delta Intake Facilities:
  - Screened on-bank intake structures.
  - Conveyance tunnels.
- An Intermediate Forebay (IF) to provide flow regulation by balancing flow from all three intakes.
- Dual Main Tunnels.
- Pumping Plants.
- The existing CCF divided and expanded to provide storage and flow regulation:
  - North Clifton Court Forebay will be receiving water from the Sacramento River.
  - South Clifton Court Forebay will be taking water from the South Delta.

These components are based on the following premises:

- The MPTO/CCO must be able to deliver up to 9,000 cfs at the low water level in the Sacramento River.
- The MPTO/CCO 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.
- MPTO/CCO facilities are protected against damage from a 200-year flood event and sea level rise as a result of climate change.
- Design level of ground motion for:
  - Intake Facilities has a 10 percent chance of being exceeded in 50 years.

  - North and Main Tunnels have a 5 percent chance of being exceeded in 50 years[1].
  - Forebays follow the DSOD criteria mandating the use of deterministically derived ground motions for design (between the mean and one standard deviation above the mean) that are derived from analysis of nearby fault sources using attenuation relationships.

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: Operational description of the existing facilities and the MPTO/CCO.
- Section 5.0: Preliminary Hydraulics of the entire proposed alignment.
- Sections 6.0 - 20.0: Individual facilities within the MPTO/CCO.
- 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 MPTO/CCO
- 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: Conceptual Construction Schedule.
- Appendix D: Surge Analysis Technical Memorandum.
- Appendix E: Pipe Materials.
- Appendix F: Pipe Floatation Analysis.
- Appendix G: Summary of Tunneling Contractor Comments
- Appendix H: Hydraulic Analysis Technical Memorandum
- Appendix I: Conceptual Design of Tunnel Linings
- Appendix J: Conceptual Engineering Evaluation of Tunnel Lining Systems

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

- Volume 2: Concept Drawings.
- Volume 3: Map Book further detailing facility locations relative to other geographic information.

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[1] These criteria are subject to further study and possible revision. Further detail is provided in Appendix A.

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 Delta.

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 salvage 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 10,300 cfs and 4,600 cfs, respectively, for a combined pumping capacity of approximately 15,000 cfs.

2.2 History of Conveyance Option

The Modified Pipeline/Tunnel Option (MPTO) is an optimized effort to the Pipeline/Tunnel Option (PTO) documented in the March 10, 2010 CER (DWR, 2010a) and the subsequent October 2010 Addendum (DWR, 2010b). The primary difference between the PTO and the MPTO is the reduction of the maximum design flow from 15,000 cfs to 9,000 cfs. Additional changes from the PTO include the reduction in the number of screened intakes from five to three and the elimination of the Intermediate Pumping Plant (IPP). The elimination of the IPP resulted in the need to increase the size of the main tunnels to allow gravity flow from the IF to the export pumping plants.

The current MPTO/CCO effort represents a further optimization to the Modified Pipeline/Tunnel Option (MPTO) documented in the October 1, 2013 CER (DWR, 2013). The main differences between the MPTO and MPTO/CCO include consolidating the pumps at Clifton Court Forebay (CCF), increasing the size of the North Tunnels, and decreasing the size of the Intermediate Forebay.

This report incorporates the most current recommendations for intake locations, IF sizing and location, tunnel alignment optimization, consolidated pumping plants, and CCF re-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 MPTO/CCO tunnel conveyance 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 MPTO/CCO reaches are shown in Figure 3-1.

The proposed alignment is approximately 44 miles long from the north Delta Intake Facilities to the exporting pumping plants. As shown in the figure, the alignment includes three Intake Facilities with in-river intake structures and sedimentation basins; the North Tunnels, which consist of three reaches totaling approximately 14 miles that connect the Intake Facilities to the IF; two parallel Main Tunnels, each approximately 30 miles long; an IF; a pumping plant located adjacent to Clifton Court Forebay; and the North Clifton Court and South Clifton Court Forebays.

The North Tunnels from Intake No. 2 to a junction structure near Intake No. 3 and from Intake No. 5 to IF are single-bore, 28-foot inside diameter (ID) tunnels. The North Tunnel connecting the junction structure near Intake No. 3 to the IF is a single-bore, 40-foot inside diameter (ID) tunnel. Due to the lack of geological and topographical information, the diameters of these tunnels are preliminary and will be optimized during the final design phase of the project.

The Main Tunnels are twin-bore, 40-foot ID tunnels. Each has isolation devices to confine sections and dewater for maintenance and inspection purposes.

3.2 Reach Descriptions

The MPTO/CCO is divided into a total of eight separate reaches as shown in Figure 3-1 and described below.

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, 1 mile south of Clarksburg 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 east 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 with the IF inlet shaft. Intake No. 3 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 the water directly into the junction structure. From the junction structure, a 40-foot ID tunnel extends approximately 6.74 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 with an 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 4.77 miles south to an IF inlet shaft at the IF on Glanville Tract.

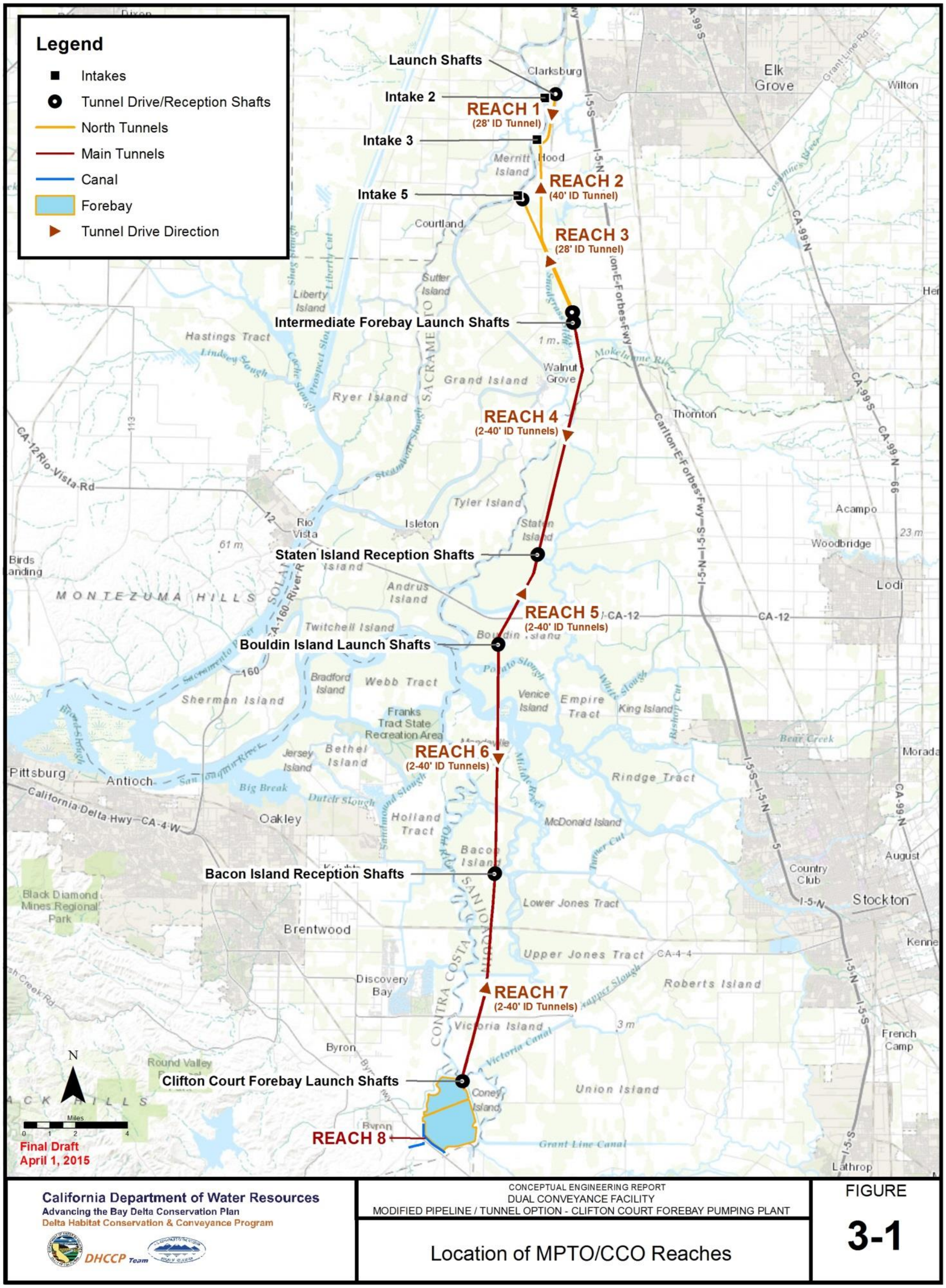

Figure 3-1 Location of MPTO/CCO Reaches

3.2.4

3.2.5 Reach 4: Intermediate Forebay to Staten Island

Reach 4 starts on the Glanville Tract adjacent to the IF and is comprised of the northernmost 9.17 miles of the twin bore 40-foot ID tunnels from the IF to Staten Island. Reach 4 ends at the construction shafts on Staten Island, approximately 2 miles southeast of the community of Walnut Grove and just east of the Sacramento River..

3.2.6 Reach 5: Staten Island to Bouldin Island

In Reach 5, the main tunnels extend approximately 3.83 miles nearly due south from the Staten Island construction shafts to construction shafts immediately north of Potato Slough and south of Highway 12 (SR-12) on Bouldin Island.

3.2.7 Reach 6: Bouldin Island to Bacon Island

In Reach 6, the main tunnels extend approximately 8.86 miles nearly due south from the Bouldin Island shafts to reception shafts about 1.8 miles south of the Old River Connection Slough on Bacon Island.

3.2.8 Reach 7: Bacon Island to North Clifton Court Forebay (NCCF)

Reach 7, approximately 8.29 miles long, is the final section of the main tunnels. Drive shafts are just outside the northeast corner of CCF at the southern portion of Byron Tract in Reclamation District 800.

3.2.9 Reach 8: North Clifton Court Forebay (NCCF) and Connection to State Water Project and Central Valley Project

Reach 8 starts at the outlet structure from NCCF and extends to both the Jones Pumping Plant intake channel and the Banks Pumping Plant intake channel. The conveyance portion is approximately 2.8 miles long and includes radial gate control structures, canals and siphons. Reach 8 also includes an emergency spillway for the NCCF, NCCF and SCCF embankment, and the divider embankment.

3.3 Geologic Design Considerations

3.3.1 Regional Geology

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).

3.3.2 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).

3.3.2.1 Artificial Levee Fill (Historical)

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

3.3.2.2 Alluvial Channel and Natural-levee Deposits (Holocene)

Alluvial channel and natural-levee deposits are characterized by loose, poorly graded, sandy to clayey silt and silty sands. These deposits are associated with active, historic, and prehistoric non-tidal channels. This 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).

3.3.2.3 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.

3.3.2.4 Flood-basin Deposits (Holocene)

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

3.3.2.5 Peat and Peaty Mud of Tidal Wetlands (Holocene)

This unit includes sediments deposited in tidal marsh at, or near, sea level. Delta peat and mud typically have low bulk density and include silt, clay, and peat with minor sand 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.

3.3.2.6 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.

3.3.2.7 Older Alluvium (Pleistocene)

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

3.3.2.8 Bedrock (Tertiary and Upper Cretaceous)

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

3.3.3 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 3-2 through 3-5 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 MPTO/CCO alignment except near the NCCF and between the Bouldin Island shafts and the North Tunnel ventilation shafts. The MPTO/CCO alignment is on the opposite side of NCCF from the 2010 PTO alignment and 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 MPTO/CCO 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 MPTO/CCO.

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 3-2 through 3-5).

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 3-4).

Tephra, comprised of vocanic 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 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.

3.4 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.

3.4.1 Seismic Sources

A model of the active and potentially active seismogenic faults in the greater San Francisco Bay region was developed as part of the Delta Risk Management Strategy (DRMS) study (DWR, 2007) (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.

3.4.1.1 Probabilistic Seismic Hazard Analyses

In the DRMS study, URS performed a Probabilistic Seismic Hazard Analysis study in which the annual probabilities of occurrence at selected times over the next 200 years (e.g., 2005, 2050, etc.) for plausible earthquake events were defined for all seismic sources that could impact the Delta (DWR, 2007). 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 MPTO/CCO 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 3-1 for the different project facility locations in order of north to south:

Table 3-1: Probabilistic Seismic Hazards for Modified Pipeline/Tunnel Clifton Court Option Facilities

Location[a]	500-year PGA mean	500-year PGA 85th %	1,000-year PGA mean	1,000-year PGA 85th %
Intake No. 2	0.23	0.25	0.27	0.30
Intake No. 3/Junction Structure	0.23	0.25	0.27	0.30
Intake No. 5	0.24	0.26	0.29	0.32
Intermediate Forebay	0.24	0.26	0.29	0.31
Staten Island Reception Shafts	0. 27	0.29	0.32	0.35
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 (North & South CCF)	0.40	0.45	0.50	0.57

[a] 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 bgs. 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 bgs. 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.

3.4.1.2 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 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 3-2 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 3-2: Summary of Active Faults Surrounding Intermediate Forebay and Deterministic Ground Motions

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

[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
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 3-2, the maximum deterministic mean PGA at the IF is 0.36 g, while the maximum 84th percentile PGA is 0.57 g. From Table 3-1, 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.

North and South Clifton Court Forebay. A summary of nearby active faults and deterministic PGA values is provided in Table 3-3 for the NCCF and the SCCF.

The West Tracy fault passes through the NCCF and SCCF 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 3-1 are lower than the deterministic values shown in Table 3-3 for recurrence intervals of less than 1,000 years. The probabilistic 85th percentile, 1,000-year PGA for the NCCF 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 3-3). Therefore, the probabilistic 85th percentile was used as the ground motion for the conceptual design at the NCCF.

Table 3-3: Summary of Active Faults Surrounding North and South Clifton Court Forebay and Deterministic Ground Motions

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

[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
mm/year = millimeter(s) per year
PGA = peak ground acceleration

3.4.1.3 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.

3.4.1.4 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 MPTO/CCO 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.

For purposes of the preliminary liquefaction analyses, a horizontal PGA corresponding to the probabilistic 85th percentile, 1,000-year ground motion was used for the forebay locations, and the probabilistic median 500-year ground motion was used for all other facility locations. An earthquake magnitude of M6.75 was assumed, as defined in Appendix A. The depth to groundwater that was observed at each boring or CPT location was assumed to be the water level at the time of the earthquake event.

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 as described above.

Final design liquefaction analyses should be performed when final seismic design criteria for the MPTO/CCO 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. The estimated ground settlement following the selected earthquake for analysis (probabilistic 500-year average annual return period) was estimated to be 24, 17, and 24 inches at Intakes No. 2, 3, and 5, respectively. 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.

North and South Clifton Court Forebays. Available subsurface information indicates that the potential for liquefaction exists along all sides of the expanded Clifton Court Forebay. Preliminary liquefaction analysis shows that the estimated ground settlement following the design earthquake at the forebay site to be 1 to 6 inches along the west and south sides, which, given the relatively flat embankment slopes, is not considered likely to result in failure of the embankment. 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, the liquefaction results from the intakes and the IF were judged to be representative, in the absence of additional data. The North Tunnel appears 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 junction structure near Intake No. 3 can be expected.

Main Tunnels. For the Main Tunnels, extensive liquefaction of the upper 40 to 60 feet is predicted in areas with soft and loose soils, and liquefaction-induced settlement of the Main Tunnel drive shafts and reception shafts working pad fills can be expected.

3.5 Flood Protection Considerations

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 3-4 indicates flood level conditions assumed for this CER for each facility of the MPTO/CCO. Flood protection elevations were not available for the specific shaft locations between the IF and the NCCF. For the purposes of this CER, it was assumed that flood levels between the IF and the NCCF 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 3-4.

Table 3-4: Flood Protection Elevations for Modified Pipeline/Tunnel Clifton Court Option Facilities

MPTO/CCO Facility	200-Year WSE with SLR	200-Year WSE with SLR, Wave Run-up and Freeboard
Intake No. 2 (all facilities at this site)	31.4[a], 24.2[b]	34.4[a, c]
Intake No. 3/Junction Structure (all facilities at this site)	30.4[a], 24.2[b]	33.4[a, c]
Intake No. 5 (all facilities at this site)	28.4[a], 24.2[b]	32.2[a, c]
IF and Structures (includes drive shafts for north and Main Tunnels)	24.2[b]	32.2[b]
Staten Island Reception Shafts	24.2[b, d]	32.2[b, d]
Bouldin Island Shafts	24.2[b, d]	32.2[b, d]
Bacon Island Reception Shafts	24.2[b, d]	32.2[b, d]
North and South CCF and Structures (includes Main Tunnel shafts at northeast corner of NCCF, and pump plant)	16.5[b]	24.5[b]

[a] 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.

[b] Source: Mineart et al., 2009

[c] Refer to Section 6.0, Intakes and Sedimentation Facilities for criteria to select structures and levee heights.

[d] All shaft locations between IF and NCCF must have flood elevation requirements confirmed during subsequent study.

Notes:

NCCF = North Clifton Court Forebay
E = East
IF = Intermediate Forebay
SLR = sea level rise
W = West
WSE = water surface elevation

All elevations are NAVD88; also see Section 4.2.1.

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 Levees* (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.

The conveyance facilities are considered to be critical lifeline facilities for the State of California. The proposed MPTO/CCO 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.

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.

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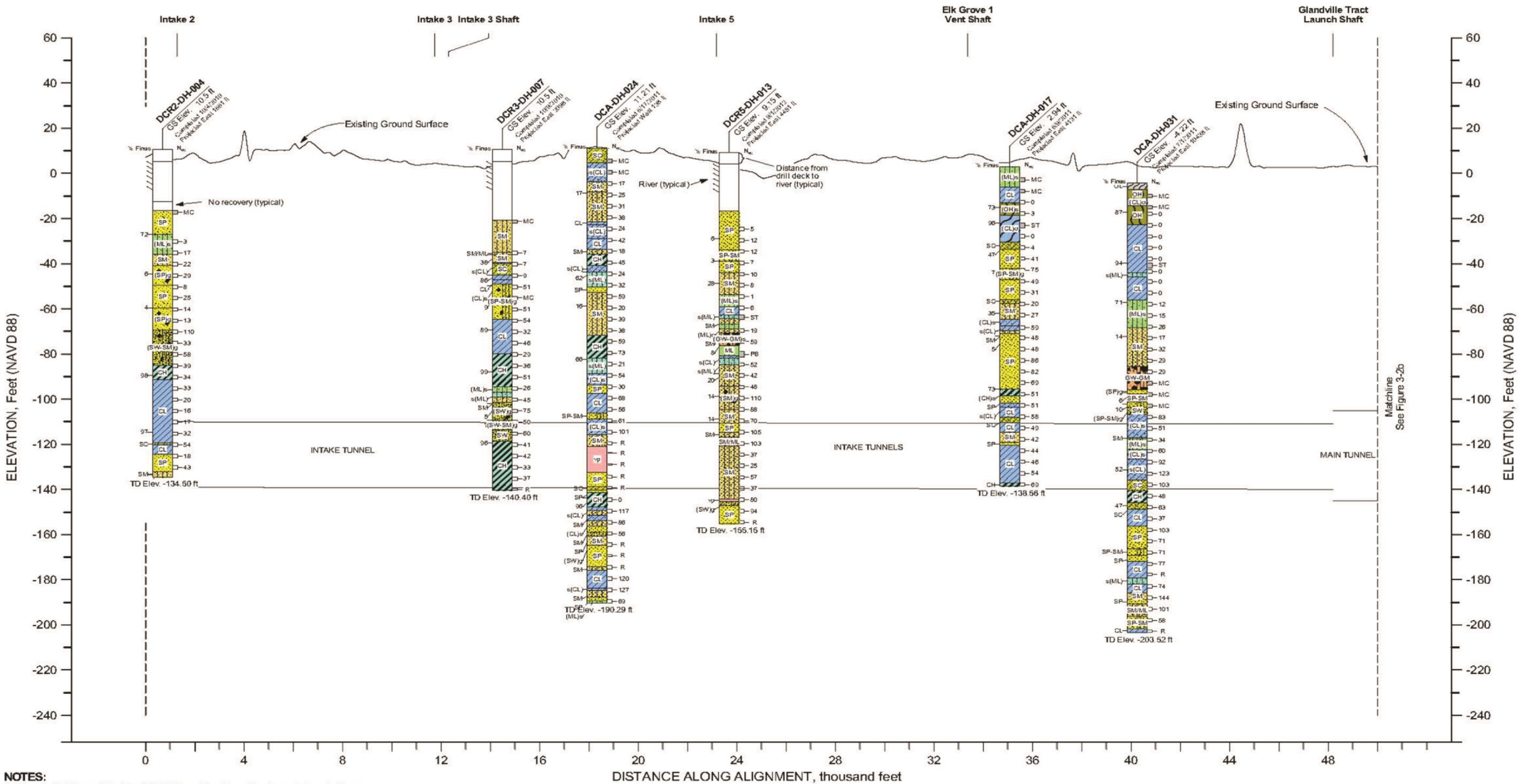
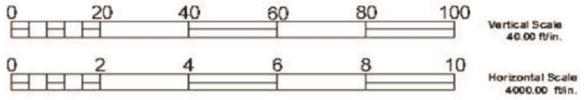


NOTES:

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.



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April 1, 2015

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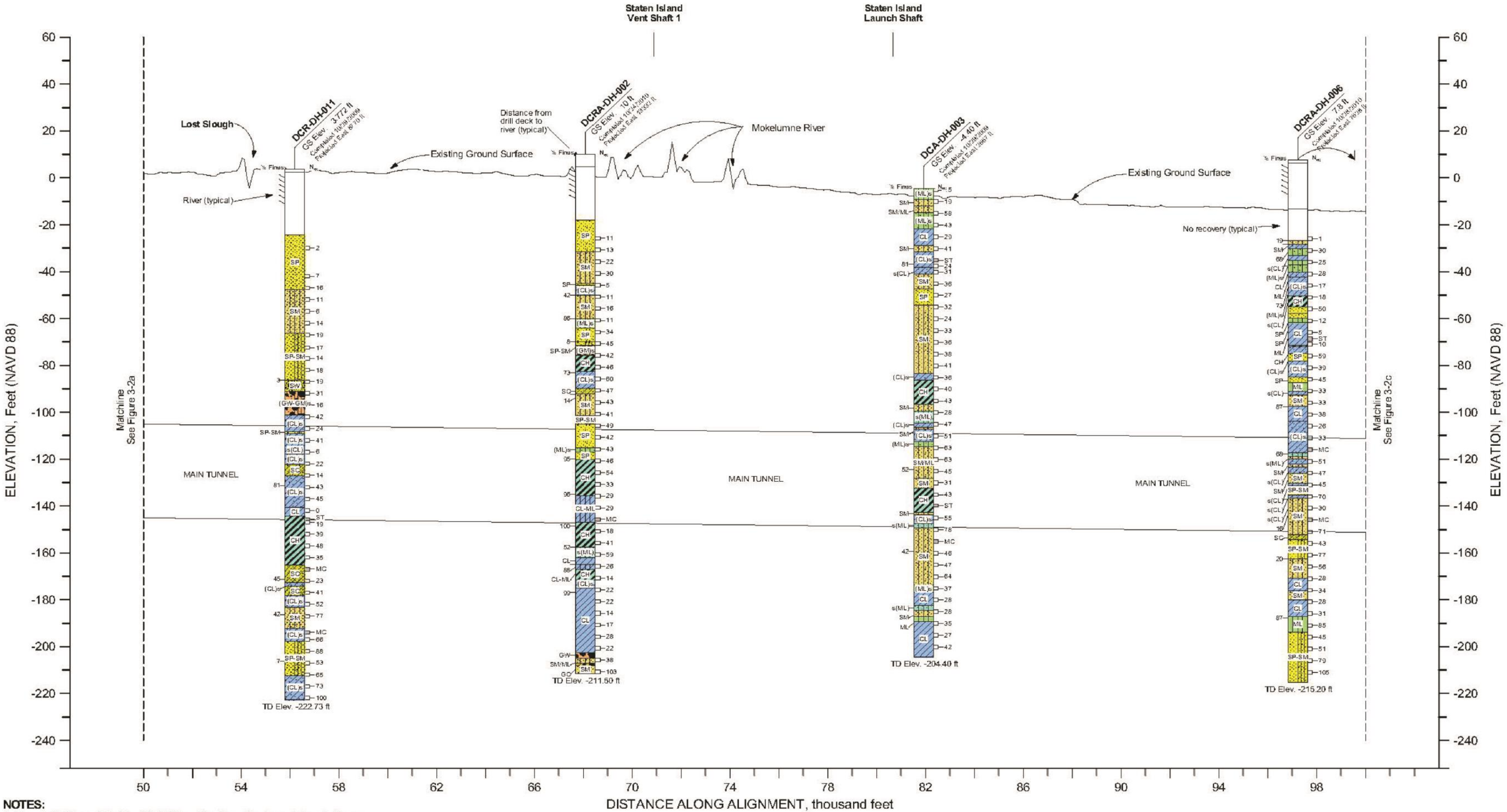
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program
DHCCP Team

CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

Graphic Boring Logs

FIGURE
3-2a

Figure 3-2: Graphic Boring Log, 1 of 5



NOTES:

1. For definition of Unified Soil Classification System abbreviations, consult ASTM D2487.
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0 20 40 60 80 100 Vertical Scale 40.00 ft/in.

0 2 4 6 8 10 Horizontal Scale 4000.00 ft/in.

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Graphic Boring Logs

FIGURE
3-2b

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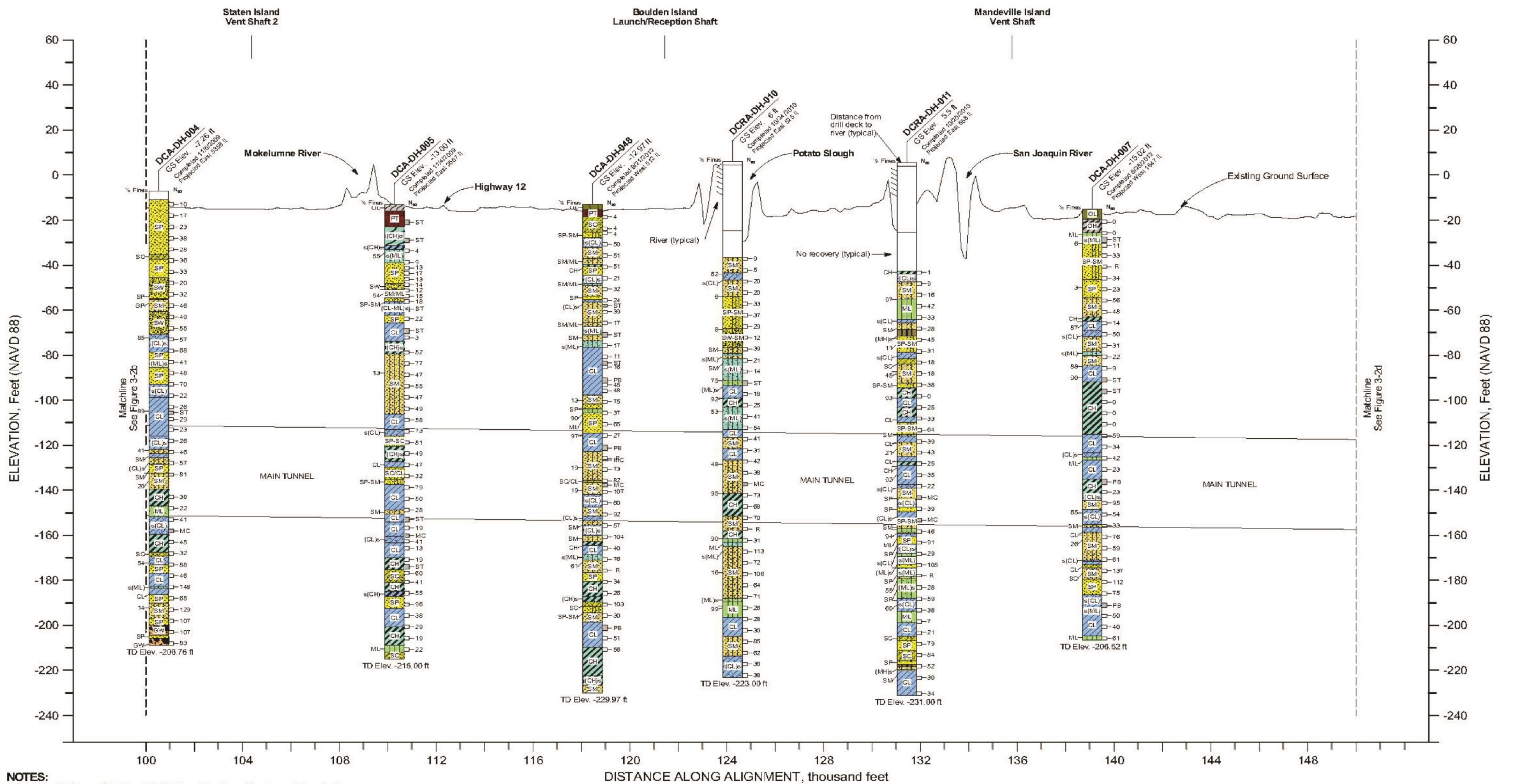
Figure 3-3: Graphic Boring Logs, 2 of 5



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.**

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Graphic Boring Logs

FIGURE
3-2c

Figure 3-4: Graphic Boring Logs, 3 of 5

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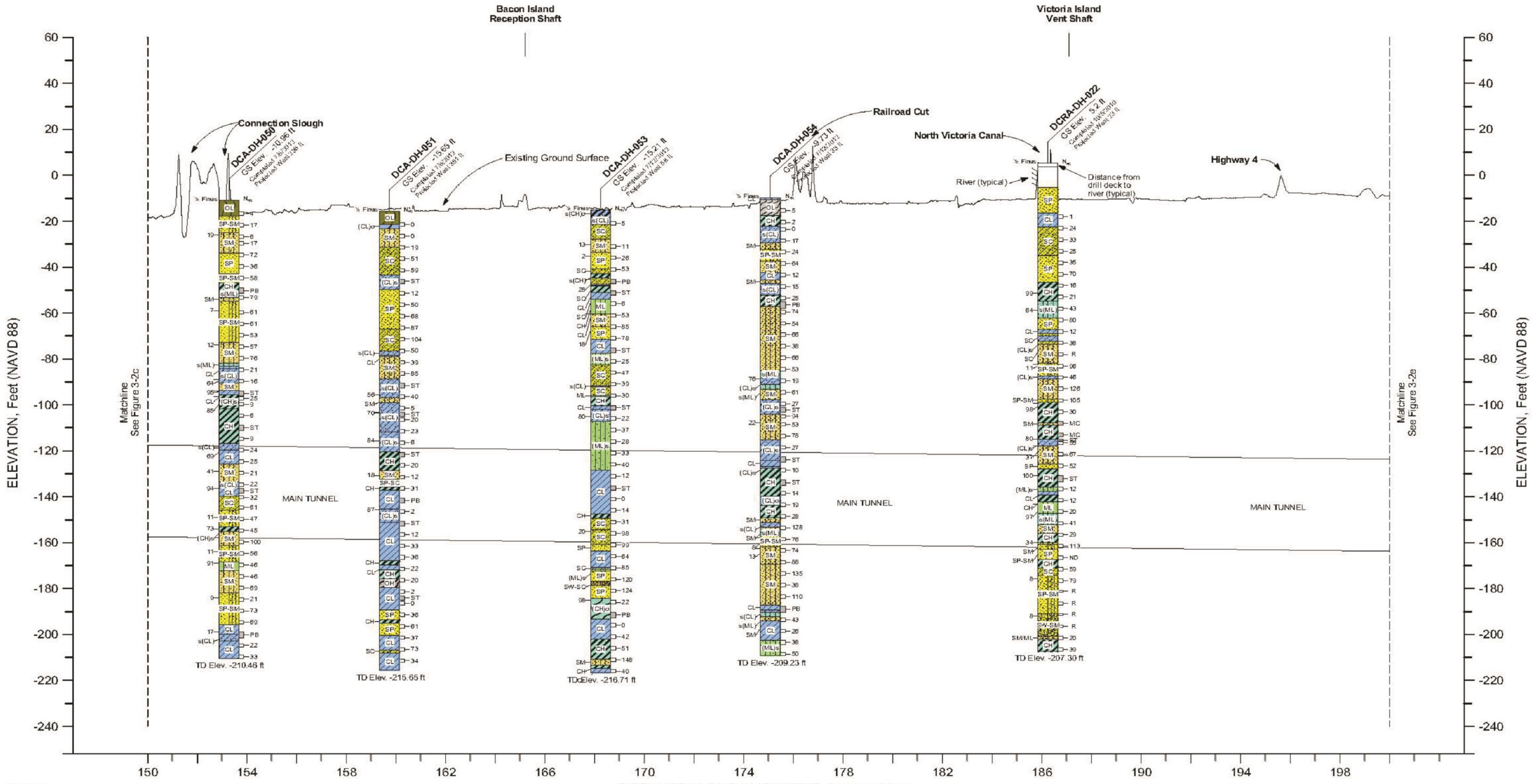

NOTES:

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2. Consult geotechnical data report for Pipeline/Tunnel Option (draft April 2013) for complete boring logs and subsurface data.

0 20 40 60 80 100 Vertical Scale 40.00 ft/in.

0 2 4 6 8 10 Horizontal Scale 4000.00 ft/in.

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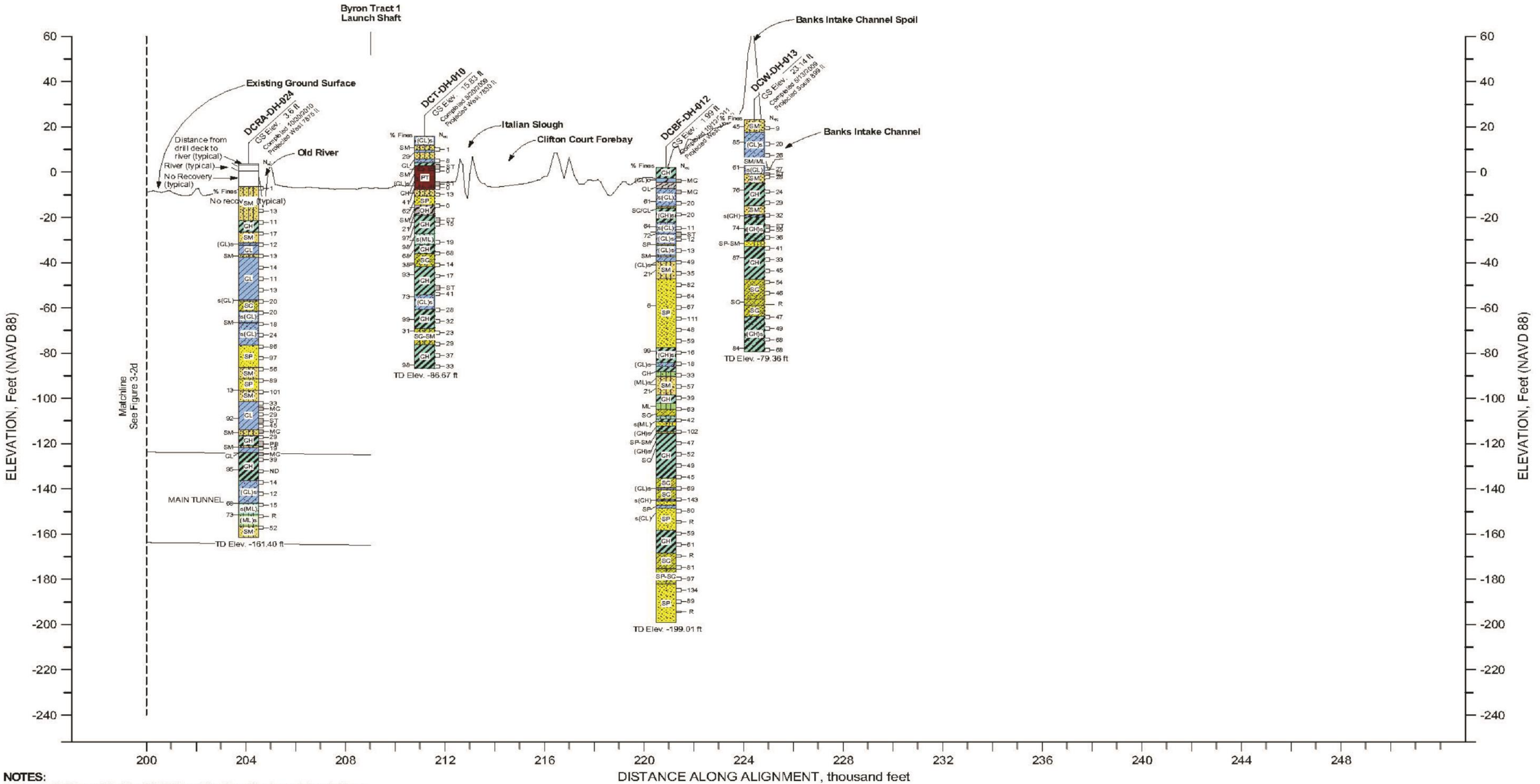
Graphic Boring Logs

FIGURE
3-2d

Figure 3-5: Graphic Boring Logs, 4 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.

0 20 40 60 80 100
Vertical Scale 40.00 ft/in.

0 2 4 6 8 10
Horizontal Scale 4000.00 ft/in.

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Graphic Boring Logs

FIGURE
3-2e

Figure 3-6: Graphic Boring Logs, 5 of 5

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SECTION 4.0

Conveyance System Operations

4.1 Conveyance System Operations

This section briefly describes operation of the existing SWP and CVP Delta export facilities and operational considerations and concepts for the proposed MPTO/CCO facilities.

4.2 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. 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.

4.2.1 Datum Corrections

As described below, different vertical datums are used for reporting 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 4-1 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.

4.2.2 Byron Bethany Irrigation District Delta Export Facilities

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

4.2.3 State Water Project Delta Export Facilities

The SWP Delta export facilities consist of the CCF, the Skinner Delta Fish Protection Facility (Skinner Fish Facility), and the Banks PP.

4.2.3.1 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 4-1 summarizes CCF water elevation information.

Table 4-1: Existing Clifton Court Forebay Operational Water Elevations (Measured at Clifton Court Forebay Inlet Gates)

Criteria	**Data in NGVD29**	**Data in NAVD88**
Vertical Datum	NGVD29[a]	NAVD88 (NGVD29 + 3.10 feet[b)])
Normal Minimum Water Surface Elevation[c]	-2 feet	+1.1 feet
Normal Maximum Water Surface Elevation [d]	+4 feet	+7.1 feet
Absolute Maximum Water Surface Elevation[a,d]	+5 feet	+8.1 feet

[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

4.2.3.2 Skinner Fish Facility

The Skinner Fish Facility is located on the California Aqueduct 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.

4.2.3.3 Banks Pumping Plant

The Banks PP has an installed capacity of 10,670 cfs and presently takes water from CCF via the California Aqueduct 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 4-2 presents a summary of Banks PP operational water elevation information.

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

Criteria	Data in NGVD29	Data in NAVD88
Vertical Datum	NGVD29[a]	NAVD88 (NGVD29 + 3.10 feet)[b]
California Aqueduct Intake Channel Immediately Upstream of Banks PP Trash Racks		
Lowest Water Surface Elevation for Pump Operation[a, c]	-4 feet (-3.5 feet)[d]	-0.9 feet (-0.4 feet)
Typical Range in Water Surface Elevation under Priority 1 Gate Operations[e]	-2 to + 0 feet	+ 1.1 feet to + 3.1 feet
Historical Typical Range in Water Surface Elevation[a]	-2 to + 2 feet	+ 1.1 feet to + 5.1 feet
Maximum Design Water Surface Elevation under Normal Operations[a, f]	+5 feet	+8.1 feet

[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.

[f] Source: DWR 1974 (Bulletin No. 200, November 1974, DWR California State Water Project).

[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

4.2.4 Central Valley Project Delta Export Facilities

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

4.2.4.1 Tracy Fish Collection Facility

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

4.2.4.2 Jones Pumping Plant

The Jones PP is at the end of a 2.5-mile-long, tidally-influenced, unlined intake canal 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 4-3 presents a summary of Jones PP operational water elevation information.

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

	Site Data[d]	**Data in NAVD88**
Vertical Datum	Site-specific, (centerline of pump discharge set at Elevation 0.00)[b]	Site -0.43 feet[a] = NAVD88
Design Operating Criteria	-1 feet to +10 feet	-1.43 feet to +9.57 feet
Lowest Water Surface Elevation for Operation[c]	-2 feet	-2.43 feet
Typical Range in Upstream Water Surface Elevation Measured Immediately Upstream of Trash Racks at Inlet to Pumping Plant[d]	0 feet to +4 feet	-0.43 feet to +3.57 feet
Highest Water Surface Elevation under Typical Operation[d]	+6 feet	+5.57 feet
Highest Water Surface Elevation for Pump Operation	+10 feet	+9.57 feet

[a] 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

4.2.5 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 4-4.

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

Factor	**State Water Project**	**Central Valley Project**
Owner	DWR	Reclamation
Operator	DWR	SLDMWA
Pumping Plant	Banks PP	Jones PP
Installed Capacity	Nominal 10,670 cfs (11 units)	Nominal 4,600 cfs (6 units)
Pump Sizing	5 at 1,130 cfs, 4 at 1,067 cfs, 2 at 375 cfs	1 at 1,000 cfs, 2 at 990 cfs, 1 at 950 cfs, 2 at 850 cfs[a] (Original capacity: 6 at 767 cfs)
Pumping Regime	Operated on an on-peak/off-peak schedule. Typically, periods of non-operation during on-peak hours.	24/7 at constant rate when water is available for export.
Flow Variation	Output varies throughout the day; flow changes as often as once per hour. Output scheduled several days in advance.	Limited changes in number of pumps in operation, often constant throughout 24-hour period, any changes scheduled several days in advance.
Fish Screens	Skinner Delta Fish Protection Facility	Tracy Fish Collection Facility
Forebay	CCF, 28,653 AF	None, tidal channel
Forebay Water Level Control	Tidally influenced, partial control using radial gates at CCF inlet.	None, tidal
Elevations Converted to NAVD88		
Typical Range in Upstream Water 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	-2.43 feet
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 SLDMWA.

[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
SLDMWA = San Luis and Delta-Mendota Water Authority
24/7 = 24 hours per day, 7 days per week

4.2.6 Use of Existing Delta Export Facilities with the MPTO/CCO

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 MPTO/CCO will have three operation scenarios: isolated south Delta operation, isolated north Delta operation, and dual operation.

4.2.6.1 Isolated South Delta Operation

The isolated south Delta operation is similar to the current operation of the existing SWP and CVP facilities, except the existing Clifton Court Forebay will be enlarged and separated into two cells, North Clifton Court Forebay (NCCF) and South Clifton Court Forebay (SCCF). The SWP will continue to divert its supply into the SCCF 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.

4.2.6.2 Isolated North Delta Operation

For the isolated north Delta operation, both Banks PP and Jones PP draw water from NCCF via the new conveyance facilities and the north Delta intakes. Only NCCF is used in this scenario, and the SCCF intake and Tracy Fish Facility gates remain closed. The flow from the north Delta intakes is taken from the NCCF and bypasses the Tracy and Skinner Fish Facilities. The operating range of water surface elevations within the NCCF will be compatible with the operating ranges of both export pumping plants.

4.2.6.3 Dual Operation with MPTO/CCO

Under dual operation, both north Delta intakes and south Delta diversion facilities are used to meet the SWP and CVP demands. The inflow into the NCCF is provided by the diversions from the north Delta intakes and the inflows into SCCF are from south Delta intakes. The SWP demands are met by the flows from both NCCF and SCCF, with the flows from the SCCF passing through the Skinner Fish Facility. The CVP demands are met by flows from the NCCF and Old River, with the diverted flow from Old River passing through the Tracy Fish Facility. This requires the simultaneous use of both NCCF and SCCF to meet the Banks and Jones pumping needs.

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. 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 NCCF and SCCF will be part of this conveyance system. The flow control system from NCCF should be such that it maximizes the diversion opportunities from both north and south Delta intakes. The outlet structures from both NCCF and SCCF 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.

To simplify the operation of the NCCF, two separate gates structures will be constructed at the outlet. The first outlet gate will have its sill at -12.0 ft. elevation, which is very close to the sill elevation of the existing CCF intake gates. This gate will be used during isolated north Delta operations. The sill elevation of the second gate will be kept at 5.6 ft. elevation and this gate will be used during dual operations while the first gate is kept closed. The sill elevation of the second gate corresponds to the historical maximum operating elevation of CCF (5.07 ft.) plus 0.5 ft. of freeboard. The flow from the NCCF will pass over a weir to the conveyance canal, which can used to measure the flow from north Delta intakes. The gates with lower sill elevations can still be used during dual operation, provided that the following conditions are simultaneously satisfied:

- At any time the WSE within the NCCF is higher than the WSE of the SCCF and Old River near Tracy Fish Facility.
- Enough driving head is available between NCCF and the intake channel leading to Banks PP to meet the target delivery to Banks PP.
- Enough driving head is available between NCCF and the intake channel leading to Jones PP to meet the target delivery to Jones PP.

If the above conditions are not simultaneously satisfied, the flow at NCCF needs to be regulated with gates with higher sill elevation, which will increase the pumping cost. Table 4-5 summarizes the recommended elevations for both gates and their operations.

A total of 1.5 ft of head loss is assumed when the NCCF is delivering the maximum flows to Jones Pumping Plant and 2.0 ft when delivering to Banks Pumping Plant. The actual headloss numbers will be refined during preliminary design.

Table 4-5: Comparison of Water Surface Elevations for MPTO Operation Scenarios

Operation Scenario	**CCF in Use**	**Factors for WSE**
Isolated South Delta Operation	SCCF	**For SWP** Historical minimum WSE = 1.1 ft Typical minimum operating elevation = 1.1 ft Typical maximum operating elevation = 3.1 ft Normal maximum operating WSE = 8.1 ft **For CVP** Design minimum operating elevation = -1.43 ft Design maximum operating elevation = 9.57 ft Lowest WSE for pump operation = -2.43 ft
Isolated North Delta Operation	NCCF	**For SWP and CVP** Minimum WSE at NCCF =1.1 ft Preferred operating range minimum = 3.1 ft Preferred operating range maximum = 7.07 ft Maximum operating WSE = 11.07 ft
Dual Operation (simultaneous use of north Delta Intakes and south Delta facilities)	NCCF	**For SWP and CVP** Minimum WSE at NCCF = 5.1 ft Maximum WSE at NCCF = 14.7 ft
	SCCF	**For SWP** Historical minimum WSE = 1.1 ft Typical minimum operating elevation = 1.1 ft Typical maximum operating elevation = 3.1 ft Normal maximum operating WSE =5.1 ft **For CVP** Design minimum operating elevation = -1.43 ft Design maximum operating elevation =5.1 ft Lowest WSE for pump operation = -2.43 ft

4.3 Sacramento River Modes of Operation

The MPTO/CCO will operate under two primary river modes of operation: low level and normal pumping.

4.3.1 Low Level Pumping mode

Low Level Pumping mode is a constant low level pumping of up to 300 cfs at 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 restricted north Delta pumping.

4.3.2 Normal Operations mode

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

Operation of the north Delta intake facilities, Clifton Court Forebay Pumping Plant (CCFPP), 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 NCCF water elevation to a common agreed-upon level at the start of each daily cycle.

The MPTO/CCO 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 BDCP to be below 20,000 cfs.
- Continuous Diversion from Sacramento River: At higher river flows, preliminarily determined by BDCP to be greater than 20,000 cfs.

4.3.2.1 Intermittent Diversion from Sacramento River

The Intake Facilities and CCFPP 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 CCFPP 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.

4.3.2.2 Continuous Diversion from Sacramento River

Similar to the facilities used for the intermittent diversion operating mode, the Intake Facilities and CCFPP 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 NCCF. Assuming that Jones PP is operating 24/7, the available storage in the SCCF and NCCF plus MPTO/CCO 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 MPTO/CCO's continuous rate would be higher than the Jones PP rate. Therefore, the MPTO/CCO system would restore the operating volume within NCCF to a common agreed-upon level at the start of each daily cycle.

4.4 Concept of Operations

4.4.1 Operating Assumptions

The preliminary concept of operation for the MPTO/CCO has the following assumptions:

- Operate safely and reliably, complying with all applicable regulations, including all long-term Delta operating rules developed by the BDCP.
- Maintain NCCF and SCCF 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.

4.4.2 Overall Operation of System Components

The system will have the primary components as described below:

- Intake facilities
- Tunnel systems (North Tunnels, Junction Structure, and Main Tunnels)
- Intermediate Forebay (IF)

- Clifton Court Forebay Pumping Plant (CCFPP)
- Dual CCF (NCCF and SCCF).

4.4.3 Intake Facilities Operations

The preliminary concept for the MPTO/CCO 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 4-6.

Table 4-6: Daily Operational Considerations for Withdrawal from Sacramento River

Factor	Comments
Target Export from Delta	SWP, BBID, and CVP Delta allocations. Ongoing coordination between DWR and Reclamation.
Biological	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.
Hydrological	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.
Intake Location	Priority for withdrawal from upstream intake locations first.
System Mode of Operation	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.
Integration with Other Scheme Components	Coordinated with storage requirements and pumping schedules for Banks PP, BBID, and Jones PP.
Energy Usage	Consideration of power tariff structures. Ongoing coordination with electrical utility provider.
Seasonal	Compliance with seasonal restrictions for withdrawal from Sacramento River.
Water Quality	Water quality monitoring (turbidity, chemicals) local to given intake. Water quality concerns elsewhere in Delta (such as salinity).
Maintenance	Maintenance schedules for intake facilities. Local sediment buildup. Consideration of rotation between intake facilities during prolonged periods of lower flows.

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 BDCP is expected to include long-term water operating rules for the Delta, including north Delta diversion bypass rules representing the minimum flow required to be maintained in the Sacramento River downstream of any diversion (intake) location. The difference between the flow in the Sacramento River upstream of any diversion point and the required in-stream bypass flow represents the available flow for diversion into the MPTO/CCO 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 BDCP long-term water operating rules are also expected to include the concept of intermittent diversion linked to the tidal cycle, when the flow within the Sacramento River is below a threshold value. The hours of operation of intermittent pumping at the CCFPP 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 4.3). 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 hr day) at the intakes are considered to account for the intermittent diversion concept.

4.4.4 Intermediate Forebay Operations

The preliminary concept of the MPTO/CCO includes a forebay designated as Intermediate Forebay (IF) that is located at Glanville tract, downstream of the Intake facilities and upstream of the CCFPP. The IF provides an atmospheric break in the deep tunnel system and buffer volume for the upstream intake sites and the downstream CCFPP. 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.

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 CCFPP 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.).

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 CCFPP.

4.4.5 Tunnel System Operations

The tunnel system will consist of individual 28-foot diameter tunnels from each of the intakes (North Tunnels); a Junction Structure that combines flow from Intake Nos. 2 and 3 into a single 40-foot diameter tunnel and conveys flow to the IF; and the main, dual tunnels from IF to NCCF. Flow from Intake No. 5 is conveyed directly to the IF. The tunnel system will be operated under gravity conditions from the intakes to the CCFPP with isolation facilities at the upstream and downstream end of each tunnel. For operation and maintenance purposes, intermediate access shafts will be provided at roughly the midpoint between tunnel launching and retrieval shafts.

4.4.6 Clifton Court Forebay Pump Plant (CCFPP) Operations

CCFPP will lift water from the Main Tunnels and discharge into NCCF via two pumping plants. Each pumping plant has an associated gravity flow system that is designed to provide operational flexibility by allowing water to gravity flow 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 NCCF water surface elevation whenever any of the pumps are in operation.

If gravity flow is available in the system, pump operations are halted and the weir gates are lowered to allow water flow from the pump station shaft into NCCF. The amount of water that will gravity flow through the system will depend on the hydraulics differential water surface elevation between the pump plant shaft and NCCF. The weir gates will also serve as system surge protection by allowing water to flow above the weir gates and out into NCCF when the facilities experience a hydraulic surge.

Each pumping plant shaft will house six pumps (total of 12 pumps), capable of delivering the design capacity of 9,000 cfs (4,500 cfs per pumping plant). 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.

4.4.7 North Clifton Court Forebay Operations

The NCCF 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 NCCF will exceed the outflow to the export facilities and vice versa. The daily amount of mismatch between inflow and outflow dictates the storage volume at NCCF.

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

- Inflow volume to NCCF exceeds the operating capacity of the export pumping plants: This will happen when the MPTO/CCO is delivering at capacity and the export pumping plants are not pumping or are pumping at some lower rate.
- Export pumping volume from NCCF 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 normal operation, the Banks PP is operated at 10,300 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 normal operating conditions, the export pumping plants can pump all of the water the NCCF can supply. However, the timing difference of export pumping and of intake flows requires 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.

4.4.7.1 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.

4.4.7.2 Continuous Mode Daily Storage

If the north Delta Intake Facilities operate at full capacity for 24 hours, the volume of water delivered daily to the NCCF 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 10,300 cfs and 4,600 cfs, respectively, the exported volume from NCCF 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 either Banks or Jones cannot be run at full capacity during the isolated north Delta operation.

In order for the Jones PP to operate at full capacity of 4,600 cfs for 24 hours per day, NCCF 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 4-7 shows the “Jones Full Flow” scenario.

In order for Banks PP to operate at full capacity during off-peak hours, NCCF would need to provide a maximum 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 4-7 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 NCCF storage.

In order for both pumping plants to operate at full capacity during off-peak hours, the NCCF storage would have to be augmented with flow from SCCF (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, NCCF 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 4-7 for the “Banks 9,000 cfs Off-peak” scenario.

Table 4-7: NCCF Storage Balance Scenarios During Isolated North Delta Operations

Jones Full Flow		**Banks Full Flow Off-peak**		**Banks 9,000 cfs Off-peak**	
• Jones Full Flow 24 hours • Banks Reduced Flow 12 hours		• Jones Reduced Flow 12 hours • Banks Full Flow 12 hours		• Jones Reduced Flow 24 hours • Banks 9,000 cfs Flow 12 hours	
		On-peak 12 Hours			
MPTO/CCO Supply	9,000 cfs	MPTO/CCO Supply	9,000 cfs	MPTO/CCO Supply	9,000 cfs
Export Pumps Use:		Export Pumps Use:		Export Pumps Use:	
Banks	0 cfs	Banks	0 cfs	Banks	0 cfs
Jones	4,600 cfs	Jones	3,100 cfs	Jones	4,500 cfs
To Storage	4,400 AF	To Storage	5,900 AF	To Storage	4,500 AF
		Off-peak 12 Hours			
MPTO/CCO Supply	9,000 cfs	MPTO/CCO Supply	9,000 cfs	MPTO/CCO Supply	9,000 cfs
Export Pumps Use:		Export Pumps Use:		Export Pumps Use:	
Banks	8,800 cfs	Banks	10,300 cfs	Banks	9,000 cfs
Jones	4,600 cfs	Jones	4,600 cfs	Jones	4,500 cfs
From Storage	4,400 AF	From Storage	5,900 AF	From Storage	4,500 AF

Notes:

AF = acre-feet

cfs = cubic feet per second

MPTO/CCO supply = Storage flowing through NCCF

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

- The north Delta intake facility maximum diversion capacity of 17,800 AF per day alone will not support full 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 NCCF storage volume.
- A daily NCCF 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.

4.5 Maintenance Operations

The MPTO/CCO has features to improve operational redundancy and reliability enabling maximized conveyance capacity during maintenance operations. These features are summarized in Table 4-8.

Table 4-8: System Reliability and Redundancy

Element	**Project Feature**
Overall	NCCF and SCCF, 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.
Intake	Three north Delta intake locations. Multiple screen bays per intake; can isolate independently.
Sedimentation Basin	Each intake location with twin sedimentation basins. Can isolate twin basins independently.
North Delta Intakes to IF (North Tunnels)	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.
IF to NCCF (Main Tunnels)	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.
CCF Pumping Plants	Two 4,500 cfs capacity pumping plants at CCF. Six pumps provided in each of the two pumping plants, including a spare for the largest pump; each plant with a design capacity of 4,500 cfs (5 duty, 1 standby).
Forebays	NCCF storage from north Delta intakes and SCCF storage from West Canal. Allowance for buildup of sediment below minimum operating level.
Communication System	Redundant communication paths and equipment.

Notes:

cfs = cubic feet per second
IF = Intermediate Forebay
NCCF = North Clifton Court Forebay
SCCF = South Clifton Court Forebay
SLR = sea level rise
VFD = variable frequency drive

4.6 Implications of Modified Pipeline/Tunnel Clifton Court Option on Current SWP and CVP Operations

The MPTO/CCO 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 NCCF that would be connected to both Banks and Jones PP.
- Having the SCCF only connected to the Banks PP.
- Removing tidal influence on water levels upstream of both export pumping plants when diverting from NCCF.

The MPTO/CCO will also change other conditions as follows:

- Receiving water from NCCF 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 the both NCCF and SCCF and associated conveyance facilities.

SECTION 5.0

Conveyance System Hydraulics

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

5.1 Facility Capacity

The preliminary concept for the MPTO/CCO has a 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 Clifton Court Forebay.

5.2 Preliminary Hydraulic Analysis

This section presents a preliminary assessment of the proposed operating and hydraulic conditions throughout the MPTO/CCO system.

5.2.1 MPTO/CCO System Description

Table 5-1 lists the physical attributes of the major components of the MPTO/CCO system. For additional details, refer to the Concept Drawings (Volume 2) of each Intake Facility, IF, tunnel alignments, Clifton Court Forebay Pumping Plant; and to Section 3.0, "Overview of Conveyance Option."

Table 5-1: Physical Attributes of MPTO/CCO Components

System Component	Diameter (feet)	Length (feet)	Width (feet)	WSE Maximum	WSE Minimum
North Clifton Court Forebay	N/A	N/A	N/A	14.7	1.1
South Clifton Court Forebay	N/A	N/A	N/A	5.1	1.1
Clifton Court Forebay Pumping Plant	N/A	N/A	N/A	14.7	1.1
Two Parallel Tunnels from IF to NCCF	40	156,620 (each)	N/A	N/A	N/A
Intermediate Forebay	N/A	1,500	800	25	-20
Intake No. 2 Tunnel to Intake 3 Junction Shaft	28	11,150	N/A	N/A	N/A
Intake No. 3 Tunnel to IF	40	36,207	N/A	N/A	N/A
Intake No. 5 Tunnel to IF	28	25,180	N/A	N/A	N/A
Sacramento River at Intake No.2	N/A	N/A	N/A	31.4	1.9
Sacramento River at Intake No.3	N/A	N/A	N/A	30.4	1.6
Sacramento River at Intake No. 5	N/A	N/A	N/A	28.4	0.7

* IF length and width are at maximum WSE

Notes: IF =Intermediate Forebay
N/A=not applicable

5.2.1.1 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 +2.8 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.

5.2.1.2 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 -20.0 feet.

5.2.1.3 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 NCCF. 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.

5.2.1.4 Divided Clifton Court Forebay Hydraulic Considerations

The existing CCF will be divided into two components: the NCCF and the SCCF. The SCCF is expanded to include area on the south side of the existing CCF. The NCCF receives flow from the north Delta intakes, and it will be used during isolated north Delta operation and dual operation as defined below (see also Section 4.2.7). The operation of the SCCF imitates the existing condition, and it will be used during dual and isolated south Delta operation. The water surface elevation on NCCF and SCCF depends primarily upon the operation mode.

North Delta Operation: The maximum diversion capacity of the north Delta intake facilities is 9,000 cfs. The WSE at NCCF should be such that the export pumping plants are operating in their preferred operating range. In order to continue the existing operations, the minimum WSE in the NCCF will be about 1.1 ft. This elevation assumes 2 ft of head loss between the Banks PP and the NCCF outlet.

South Delta Operation: The WSE of SCCF 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 NCCF will have no impact.

Dual Operation: For conceptual level design, two separate outflow gates are proposed at the outlet of NCCF. The first gate is used during isolated north Delta operation and will divert flow to both Banks PP and Jones PP. The sill elevation of this gate would be at around -12 ft. The second gate has its sill at 5.6 ft elevation. The second gate is used during dual operation, with the outflow regulated and taken to the exporting pumping plants. The second gate, which has the higher sill elevation, will remain closed during isolated north Delta operation, and both gates will be closed during isolated south Delta only operations. For this configuration, the WSE of the NCCF should be 5.6 ft or higher.

5.2.2 System Hydraulic Calculations

The MPTO/CCO 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). 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 CCF 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 MPTO/CCO system are depicted in the hydraulic profiles in the Concept Drawings (Volume 2).

5.3 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 CCF 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 5-2 below. They include Peak Backflow to each Intake, Peak Backflow Velocity through the Sedimentation Basins, and Backflow Volume.

Table 5-2: Surge Evaluation – Hydraulic Parameters

Intake No.	**IF = 500' x 500'**			**IF = 800' x 1,000'**			**IF = 800' x 1,500'**		
	Peak Backflow (cfs)	**Peak Backflow Velocity (ft/s)**	**Backflow Volume (ac-ft)**	**Peak Backflow (cfs)**	**Peak Backflow Velocity (ft/s)**	**Backflow Volume (ac-ft)**	**Peak Backflow (cfs)**	**Peak Backflow Velocity (ft/s)**	**Backflow Volume (ac-ft)**
2	37	0.010	2.4	22	0.006	1.7	39	0.011	3.1
3	166	0.046	11.3	104	0.029	8.3	97	0.027	9.7
5	217	0.058	12.8	137	0.037	10.3	122	0.032	10.8

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.

5.3.1 Intermediate Forebay Size Evaluation

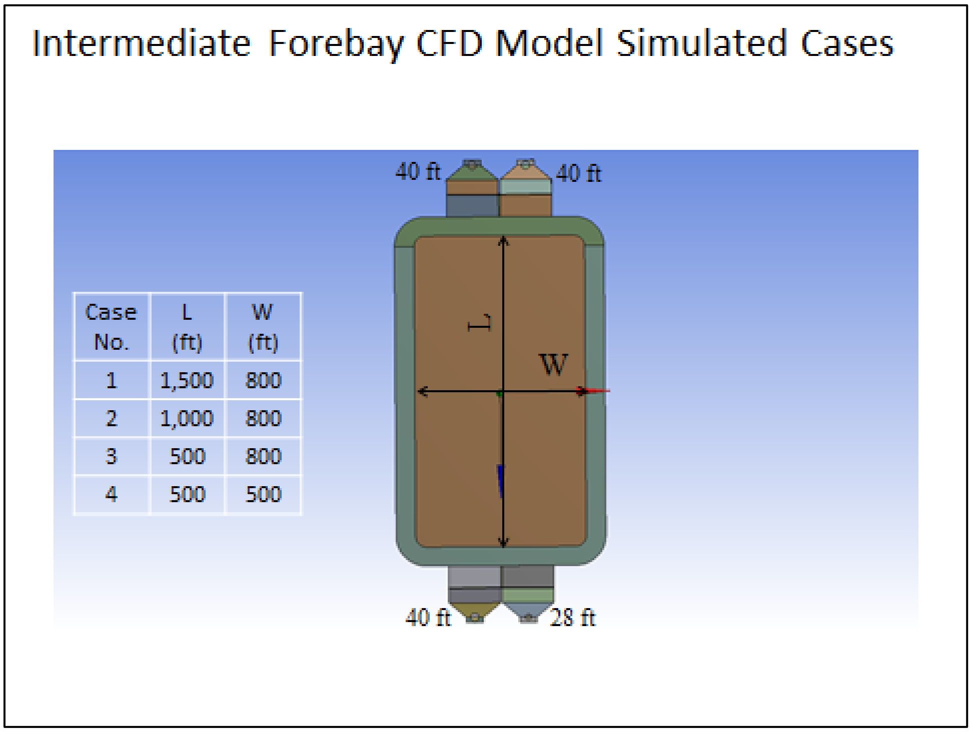
The IF included in the concept design will provide an atmospheric break in the system between the intake tunnels and the Main Tunnels. The IF is designed to be a pass-through facility without flow control, with only isolation drop gates for system maintenance and inspection. The IF will serve several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

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

The IF will be sized as small as practicable to accommodate 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.

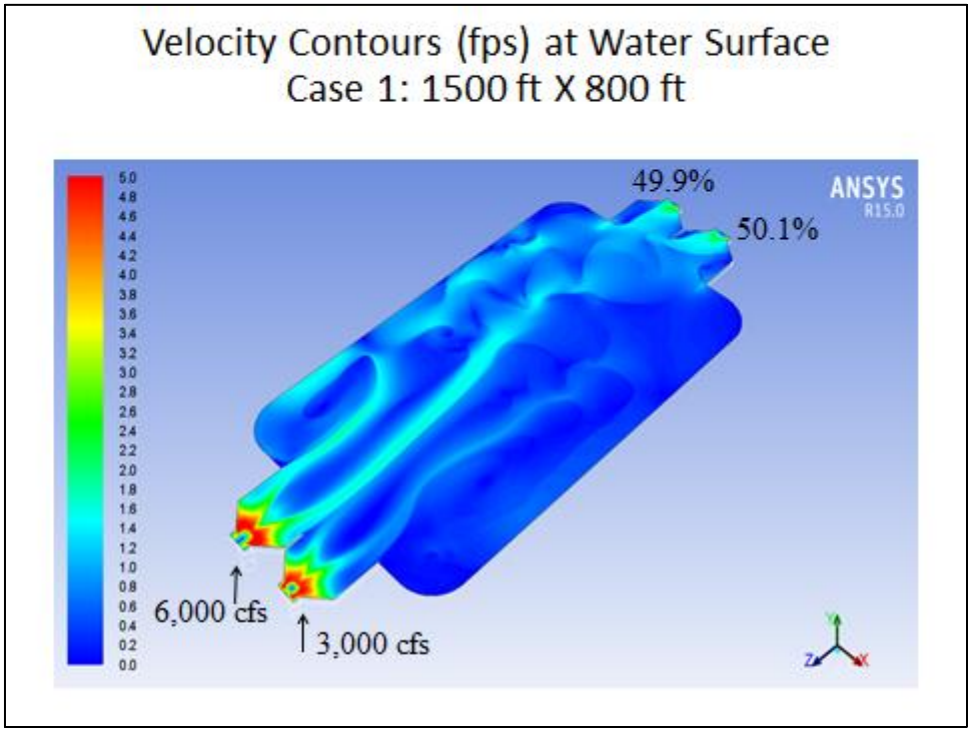
Computational Fluid Dynamics (CFD) plots for the four IF footprints are shown below in Figure 5-1. The plots in Figures 5-2 through 5-5 show velocity contours and flow distributions percentages into each 40 ft ID main tunnel.



Intermediate Forebay CFD Model Simulated Cases

Case No.	L (ft)	W (ft)
1	1,500	800
2	1,000	800
3	500	800
4	500	500

Figure 5-1: Intermediate Forebay CFD Cases



Velocity Contours (fps) at Water Surface
Case 1: 1500 ft X 800 ft

Figure 5-2: Intermediate Forebay CFD Case 1

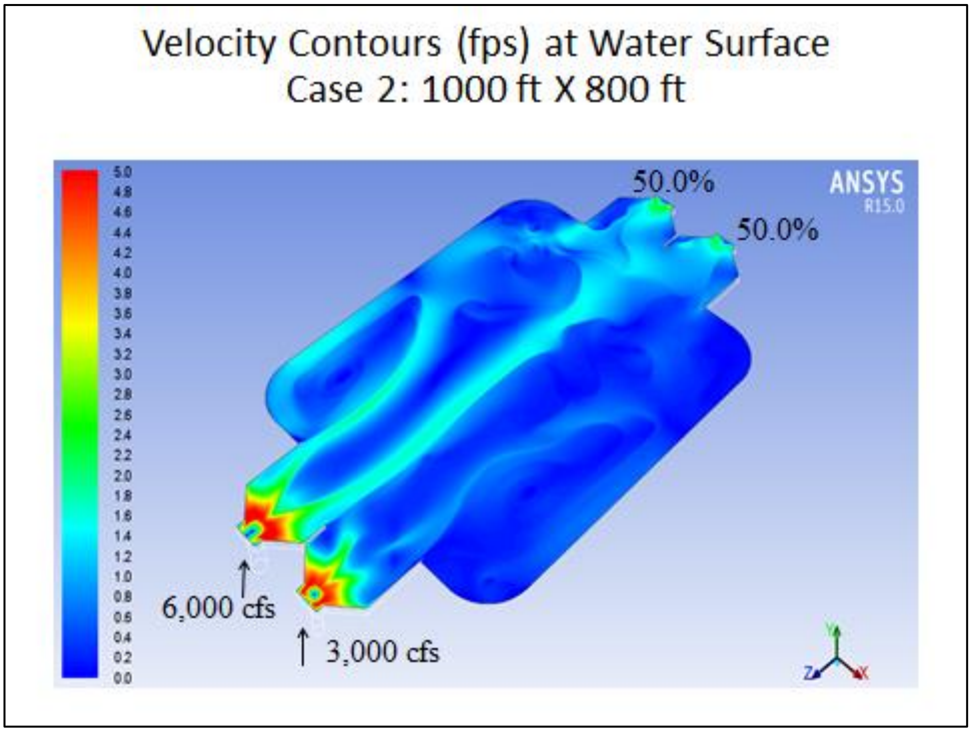


Figure 5-3: Intermediate Forebay CFD Case 2

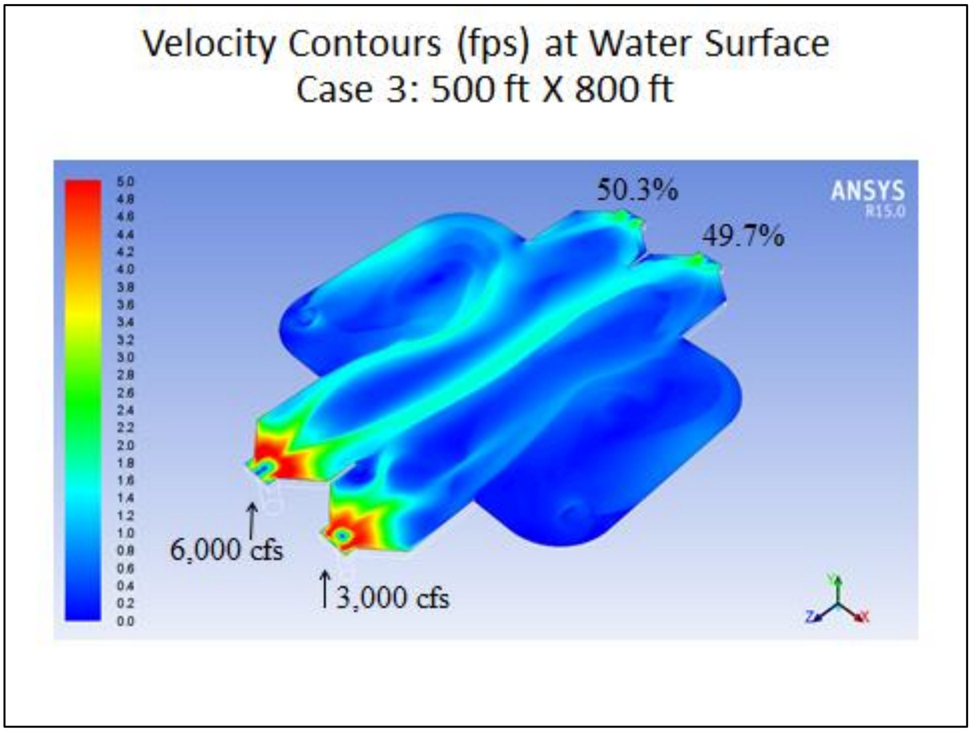


Figure 5-4: Intermediate Forebay CFD Case 3

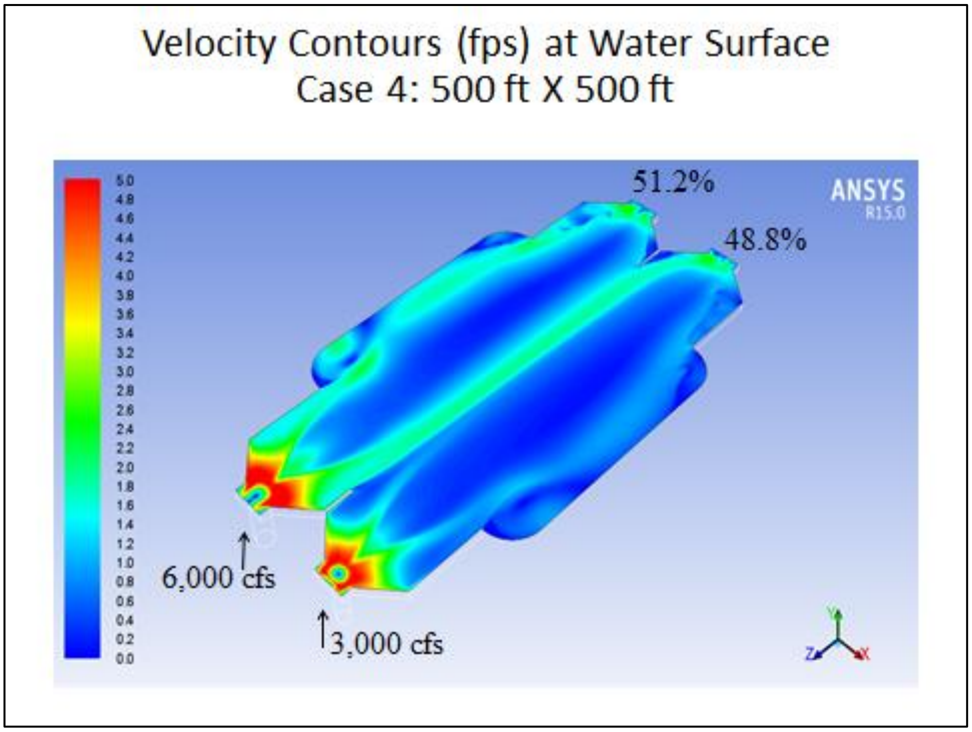


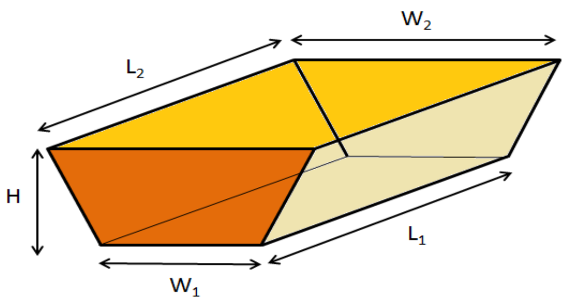
Figure 5-5: Intermediate Forebay CFD Case 4

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 geometry is calculated and shown below in Table 5-3.

Table 5-3: Intermediate Forebay Size Evaluation

Case	Top of Forebay Embankment El. (ft)	Invert Elevation (ft)	L_1 at Invert Elevation (ft)	W_1 at Invert Elevation (ft)	L_2 at Top of Forebay (ft)	W_2 at Top of Forebay (ft)	Storage Time from Invert to El. 0 (minutes)
1	32.2	-20	1,500	800	1,918	1,218	52
2	32.2	-20	1,000	800	1,418	1,218	35
3	32.2	-20	500	800	918	1,218	19
4	32.2	-20	500	500	918	918	13



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 MPTO/CCO concept, the largest footprint (800 ft W x 1500 ft H at invert Elevation -20 ft) is shown in the Concept Drawings (Volume 2) until further optimization of the above components can be evaluated fully.

SECTION 6.0

Intakes and Sedimentation Facilities

6.1 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:

MPTO/CCO Intake No. 2	River Mile 41.1
MPTO/CCO Intake No. 3	River Mile 39.4
MPTO/CCO Intake No. 5	River Mile 36.8

These intake locations were selected by DWR in consultation with the FFTT by assessing various studies and previous CERs associated with the EIR/EIS. Figure 6-1 shows the location of the selected sites. Intake numbering is consistent with the Draft 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 as proposed by DWR. The intakes have on-bank fish screens as seen 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 6-2 and in the Concept Drawings (Volume 2).

6.1.1 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 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.

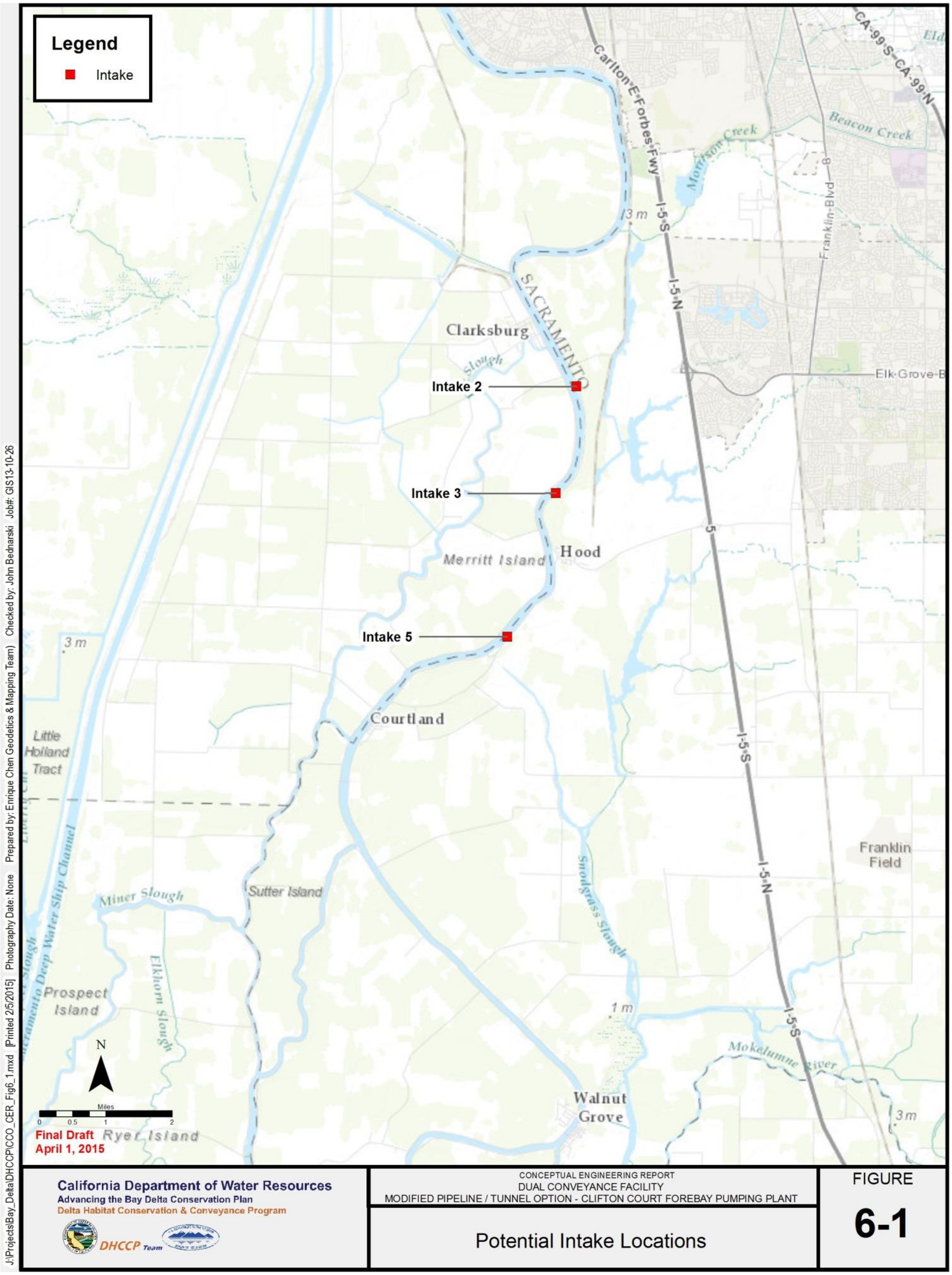


Figure 6-1 Potential Intake Locations

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Final Draft
April 1, 2015

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

CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

On-Bank Intake

FIGURE
6-2

Figure 6-2: On-Bank Intake

6.1.1.1 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 MPTO/CCO 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 6-1 provides information on intake WSE, and Table 6-2 includes intake conceptual design criteria.

Table 6-1: Intake Site-Specific Water Surface Information

Intake Designation	Design WSE (feet, NAVD88)	Maximum Design WSE (feet, NAVD88)
Intake No. 2	1.9	31.4
Intake No. 3	1.6	30.4
Intake No. 5	0.7	28.4

Notes:

NAVD88 = North American Vertical Datum of 1988

WSE = water surface elevation

Table 6-2: Intake Structure Conceptual Design Criteria

Criterion	Description
General	
Intake Technology	On-bank intake
Number of Intakes	3
Maximum Single Intake Structure Capacity	3,000 cfs
Design Minimum WSE (feet, NAVD88)	99 percent exceedance elevation (see Table 6-1 for specific elevations)
Intake Hydraulic Criteria	
Screen Approach Velocity	0.20 fps
Screen Sweeping Velocity	≥0.20 fps (subject to verification)
Screens	
Type	Fixed vertical flat-plate profile bar screen
Material	304 stainless steel
Screen Slot Opening Size	1.75 millimeters (0.069 inch)
Screen Porosity	27 percent
General Arrangement	
Capacity of Screen Bay Group (six per intake)	500 cfs
Screen Bay Clear Width	15 feet
Auxiliary Slot for Solid Panels	Required for fish screen panel removal

TABLE 6-2 (continued)
Intake Structure Conceptual Design Criteria

Criterion	Description
Structural (continued)	
Maximum Flow Control Baffle Head Differential	2 feet
Floor	Concrete floor supported on steel cased drilled piers
Breast Wall, Interior Piers and Back Wall	Reinforced concrete
Cofferdam Front and Center Walls	Sheet piles
Intake Box Conduit Cofferdam Back Wall	Concrete diaphragm wall
Fish Screen Cleaning System	
Motor	Gear motors with VFD
Number of Cleaning Systems	One per Screen Bay Group
Cleaning Assembly Speed Range	0.5 to 2 fps
Cycle Time	5 minutes, maximum
Sediment Jetting System	
Pump Capacity	2,500 gpm
Number of Pumps	One per screen bay group
Pump Type	Vertical turbine
Pump Motor Horsepower	100
Nozzle Spray Pressure	20 psi
Flow per Nozzle	25 gpm (first row – nearest to the fish screen); 100 gpm (second, third, and forth rows)
Pipe Material	Stainless steel
Flow Control Baffles	Produce uniform velocity distribution (horizontal and vertical)
Intake Collector Box Conduits	2 box conduit channels per screen bay group at 12 ft by 12 ft

Notes:

≥ = 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 (see Section 6.3.9).

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.

6.1.1.2 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, “Bridges – Road and Railroad,” 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).

6.1.2 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 6-3.

Table 6-3: Sediment Collection System Design Criteria

Criterion	Description
General	
Maximum Single Intake Structure Capacity	3,000 cfs
Sedimentation Basin Criteria	
Sedimentation Inflow (Assumes twin sedimentation basins per intake site)	1,500 cfs per side
Normal Settling Depth	20 feet (below design WSE)
Settling Depth at Design WSE	11.7 feet
Basin Size (triangular shape)	Width = 250 to 660 feet (floor dimensions; max width at intake channels; min width at outlet); length = 600 feet (sediment settling zone only)
Projected Annual Peak Year Sediment Accumulation (per intake site)	34,400 cy (calculated)
Sediment Storage Depth	3 to 5 feet below settling depth (see above)
Sedimentation Basin Side Slopes	Unlined earthen slopes, 2:1 side slopes (all or portions of basin side walls may be armored for scour or erosion control)
Basin Floor	Earthen over soil-conditioned subgrade
Basin Top of Slope Elevation	200-year WSE with SLR plus 3 feet
Maintenance	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.
Sediment Removal	Use suction dredge to remove accumulated sediment
Dredging Frequency	Assumed once per year based on peak year sediment loading to project
Dredge Discharge	Discharge to sediment drying lagoons by way of piped connection points
Sediment Drying Lagoons	
Number Required	4 lagoons per intake site
Size	Bottom width = 160 feet; bottom length = 350 feet
Depth	15 feet
Volume	860,000 cubic feet (per drying lagoon)
Inflow Rate	5.57 cfs (2,500 gpm)
Sides/Bottom	Reinforced shotcrete/roller-compacted concrete
Side Slopes	1:1
Access for Maintenance	Equipment ramps into each lagoon
Dewatering	Integrated underdrain system and decant system

[a] 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

6.1.2.1 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.

6.1.2.2 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 6-4.

Table 6-4: Sedimentation Basin Sizes

Intake Designation	Basin Width Intake Channel (feet)	Basin Width Outlet Zone (feet)	Basin Length (feet)
Intake No. 2	677	265	660
Intake No. 3	575	265	660
Intake No. 5	677	265	660

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 deep tunnels 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 shutdown 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 6-5 summarizes anticipated sediment diversion to each intake.

Table 6-5: Sediment Loading to the Project

Annual Sediment Diversion to Each Intake	Tons per Year
Annual Sand-Sized (0.075 to 1.75 mm) Sediment Diversion Per Intake	
Peak Sediment Load Year (1978)	37,400[a]
Average Year (1974 to 1991)	12,140[a]
Total Annual Sediment Diverted to Each Intake	
Peak Sediment Load Year (1978)	121,507[b]
Average Year (1974 to 1991)	39,493[b]

[a] Source: DWR, undated

[b] Source: DWR, undated, plus FRWA particle size distribution for less than sand sized material (see Table 6-6)

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 6-6), the particle size distribution for other than sand-sized material was interpolated.

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

Sieve Size (mm)	**0.250-0.500**	**0.125-0.250**	**0.062-0.125**	**0.031-0.062**	**0.016-0.031**	**0.008-0.016**	**0.004-0.008**	**0.002-0.004**	**<0.002**
Distribution (%) by weight	3	19	11	5.5	9	8.5	9	8	27

Note:

<= less than

The sediment accumulations per year in both the sedimentation basins and IF are shown in Table 6-7 (calculated by Stoke's Law from the sediment diversions listed above in Table 6-5). 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 6-7: Depth of Sediment Accumulation

Sediment Accumulation Per Facility	**Sediment Volume**
Intake Sedimentation Basins	1.5 feet per year (based on peak load year) 0.70 feet per year (based on average load year)
Intermediate Forebay (based on average year sediment diversion)	
1 Year	0.08 feet
10 Years	0.8 feet
20 Years	1.6 feet
30 Years	2.4 feet
40 Years	3.2 feet
50 Years	4.1 feet

6.1.2.3 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.

6.1.2.4 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.

6.1.2.5 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 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.

6.1.3 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.

6.2 Construction Methodology

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.

6.2.1 General Constructability Considerations

All Intake 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"). The sedimentation facilities need to resist uplift forces after construction is complete, which impacts the construction work.

Some of the major elements to be constructed:

- Driving sheet piles to depths required to achieve hydraulic cutoff. Considering the elevations of the Intake structure and associated piping, steel cased drilled 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, and alternate intake configurations will be considered to minimize such impacts.
- Installation of a diaphragm wall along the levee to serve as the back wall of the intake and manifold cofferdam system and to permit tie-in to slurry cutoff walls around the site and along the levee.
- Underwater construction, such as tremie slab placement and sheet pile trimming.
- Cofferdamming, shoring, and bracing.
- Site access and dewatering.

Construction elements for all intake facilities include:

- Staging and storage area and construction zone prep (5 to 10 acres per each intake structure, 20 to 40 acres total, including sedimentation basins).
- Diaphragm wall installation, including construction of pipe penetrations.
- Ground improvement (assumed to be jet grouting) beneath the intake and collector box conduits.
- Sheet pile cofferdamming, shoring, bracing, and hydraulic cutoff.
- General earthwork (e.g., excavation, spoil, backfill, levee construction).
- Dewatering wells, construction water treatment, return to watercourse.
- Installation of drilled piers, including drilling into place and structural fill.
- Foundation preparation and structural slab construction inside cofferdam.
- Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).

- Metalwork fabrication, machining, assembly, and installation (stainless steel fish screen panels, embeds, flow control baffles, bulkheads, traveling brush screen cleaning system, gantry crane mechanical hoist system, guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.).
- Erosion control (underwater placement of stone protection/geotextile).
- Miscellaneous civil site work (e.g., fencing, gates, access roadways and ramps, log booms and debris deflectors, hydroseeding, landscaping, etc.).
- 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.).

Diaphragm walls, installed to depths to achieve hydraulic cutoff, are planned for the entire perimeter construction of the sedimentation basins. The sedimentation basin could, on occasion, require dewatering for maintenance.

6.2.2 Intake Structure and Sediment Facilities Geotechnical

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.

6.2.2.1 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.

6.2.2.2 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 DCRA-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.

6.2.2.3 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.

6.2.2.4 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.

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 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 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 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.

Liquefaction also 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.

6.2.2.5 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.

6.2.2.6 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 deflections 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.

6.3 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.

6.3.1 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.

6.3.2 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.

For sedimentation system operation, see Sections 6.1.2.1 through 6.1.2.4.

6.3.3 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.

6.3.4 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.

6.3.5 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.

6.3.6 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.

6.3.7 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.

6.3.8 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.

6.3.9 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.

‘SECTION 7.0

CCF Pumping Plant

7.1 Site Description, Pumping Plant and Shafts

7.1.1 Site Description

7.1.1.1 Location

The CCF Pumping Plant is located at the northeastern corner of Clifton Court Forebay on a small island just south of Kings Island as shown in the Concept Drawings (Volume 2). This location is currently a “drain area” for the site, and the terrain consists of a low lying area (El. -2 to -6 [1]) between the existing Clifton Court Forebay embankment on the west (El. 17±), the West Canal levee on the east (El. 14±), and the Italian Slough levee on the north (El. 14±). This location will now serve as the terminus of the 40-foot tunnels and the location of the new combined pumping plant station.

Available site subsurface data is limited to an exploration boring (CCFI-6) drilled in 1994 near the center of the DWR-owned island to a depth of 99.5 ft. and another exploration boring (DCT-DH-010) drilled in 2009 from the top of the West Canal levee just south of the island to a depth of 102.5 ft. An additional boring (DCRA-DH-024) was drilled in 2010 from a barge in the Old River about ¾ of a mile east of the site to a depth of 165 ft (see Figure 7-1). These borings indicate the island is underlain by generally very soft to soft organic soils (peat, organic silt) to a depth of up to 15-20 feet below the existing ground surface, overlying very soft to soft clay overlying interlayers of soft to medium stiff clay and silt. Below about El. -50, the borings encountered layers of medium dense, silty to clean sand. Below about El. -70, interlayered dense sand and very still clay and silt were encountered. Limited subsurface data for this area has been collected over the years. There are several borings from the original CCF levee construction in the 1960s. Additional data was collected in the mid-1990s when DWR examined the area for a new inlet to CCF. Finally, some data was also collected in 2002 and 2010. Deeper geotechnical data is not available; however, interlayered dense and very stiff alluvial deposits are anticipated to extend to depths of at least several hundred feet beneath the site. Groundwater levels are at or near the ground surface elevation on the island. It is anticipated that the “soft” materials will be removed and replaced with engineered fill prior to conducting other site work. Dewatering may be required during this site work.

7.1.1.2 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, 2) Provide surge protection via the spillway, 3) House the pumps and their controls. The gravity flow will bypass the pumps via three weir gates by allowing flow to discharge directly to NCCF if hydraulic conditions permit (for reference, see Concept Drawings Volume 2). The pump shafts will house the pump wet well, pump intakes, and the pumps themselves. The pumps discharge via siphon discharges into a spillway basin within the NCCF. The facilities will be designed to allow separate operation of the system components.

Final grade for the permanent pump station facilities, including switchyard, electrical buildings, and other infrastructure, will be at a minimum EL. 25 to provide protection from the 200-year flood level with sea level rise (El. 16.5), wave run up (5 ft.), and additional freeboard (3.5 ft.). For 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 will be established prior to construction of the shafts to provide flood protection during construction for the tunnels and pump stations.

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[1] All elevations are in feet and based on NAVD88 datum.

Figure 7-1: Clifton Court Forebay Exploration Data

Portions of the site outside the area of permanent facilities may be filled to a lower elevation (between El. 10 and 17) 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.

7.1.1.3 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 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 sully 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 founded 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.

7.1.1.4 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 150-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 and mining operations at other tunnel drive and reception shafts along the main 40 ft. tunnel alignment (see Section 11, “Tunnels”).

7.1.2 Gravity Flow

Each main tunnel (ID 40 ft.) terminates in a pump shaft at the northeastern corner of the Clifton Court 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 CCF.

During periods when the Sacramento River stage is higher than NCCF, it is possible to achieve gravity flow through the system. The Concept Drawings show three weir gates on the north side of each pump shaft. The weir gates can be lowered to release flow into a conveyance channel that discharges into NCCF. The weir gates would also be used for flow measurement and flow control, 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 CCF and will be evaluated further during the preliminary and final design.

7.1.2.1 Tunnel Dewatering

Submersible dewatering pumps will provide tunnel dewatering. A single set of dewatering pumps can be moved to either pump shaft. To facilitate dewatering of a single tunnel within 2 weeks, it will be necessary to pump at an average rate of approximately 200 cfs. Tunnel dewatering will require a wide range of pumping heads, from a minimum of approximately 25 feet to a maximum head of nearly 200 feet. During initial dewatering, the pumping head will be low, but will increase rapidly as the shafts within the tunnel system are dewatered. When the water level reaches the crown of the Main Tunnel, the level will drop at a much slower rate. The wet-well dewatering sump pumps will be designed to dewater the upper portions of the tunnel system down to the wet-well invert (additional discussion of these pumps is provided in the pumping section –see Paragraph 7.1.3). To accommodate the wide range of heads, the dewatering pump discharge piping will be equipped with throttle valves to prevent the pumps from running off their curves.

The tunnel dewatering equipment will include eight submersible pumps, each with a design capacity of approximately 25 cfs, with submersible motors that will operate with either 460 volt or 4160 volts. A Flygt - Xylem Model CP 3351 with an 800 HP motor will deliver the target capacity and operate through a range of heads from 135 feet to 200 feet, while remaining within the manufacturer-defined Acceptable Operating Range. The submersible pumps will be lowered into place on rails until they seat on discharge elbows, where they will be securing into place for operation.

Pump and system curves representing the range of operating conditions are presented in Figure 7-2. 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.

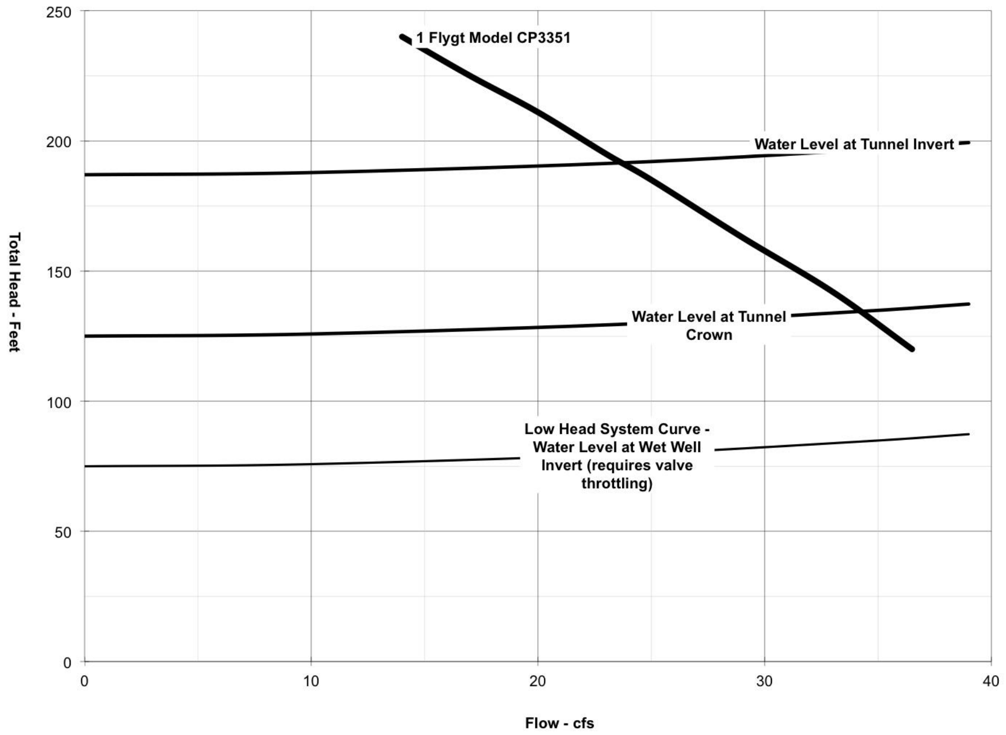

Figure 7-2: Dewatering Pump and System Curves – 1 Pump Operation (Flygt – Xylem Model CP 3351)

7.1.3 Pumping Plant

7.1.3.1 Overview

The two Pumping Plants receive flow from the pump shafts and lift the water into NCCF. 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), 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.

7.1.3.2 Pumping Hydraulics

The system was evaluated to determine the expected range of hydraulic conditions. Table 7-1 presents design water levels that were used for development of the system curve envelope presented in Figure 7-3.

Table 7-1 Design Water Levels for the Range of Operating Conditions

Pump Head Design Condition	Sacramento River WSE @ Intake #2	Clifton Court Forebay WSE	Static Head on Pumping System	Total Dynamic Head @ 9,000 CFS
High Head	1.9	14.0	12.1	33.1
Design Condition	5.0	14.0	9.0	30.0
Normal Low Head	10.0	1.0	- 9.0	12.0
Extreme Low Head	31.4	1.0	- 30.4	-9.4

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

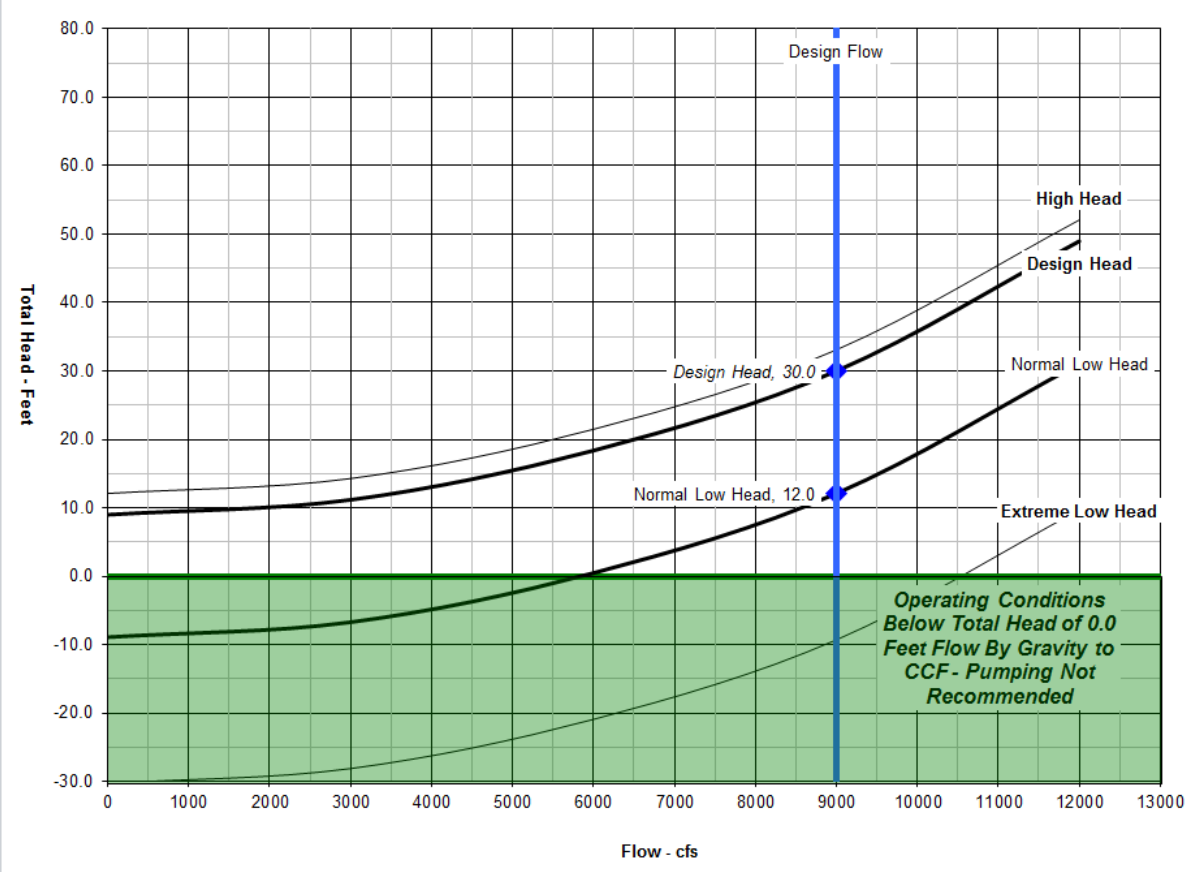

Figure 7-3: System Curves for the Range of Operating Conditions

Based on the hydraulic information from Section 5 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 NCCF, 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 NCCF will influence equipment selection and should be finalized prior to commencement of preliminary design.

7.1.3.3 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 = 21 feet[2]

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 NCCF 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 NCCF. 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.

7.1.3.4 Discharge Configuration

The discharge piping 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

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[2] The dynamic losses for establishing the bottom of the wet-well were calculated to be 21 feet at 9,000 cfs, see Section 5 of this CER for detailed System Hydraulic information.

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 NCCF when the pumps are not operating and prevent the pumps from free rotating on high wet-well elevations or surges. The conceptual design is based on a maximum water level in NCCF 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 NCCF 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 NCCF. 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 NCCF.

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 NCCF 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 needs to be evaluated 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 ultrasonic flow meter on the straight section of pipe between the pump discharge and the siphon. The ultrasonic 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.

7.1.3.5 Pumps

Pump types considered for the CCF 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 a 20-ft section, 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 is 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 7-2 summarizes the basic preliminary design criteria for the pumps and motors.

Table 7-2: Pump Conceptual Design Criteria

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

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 7-4 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.

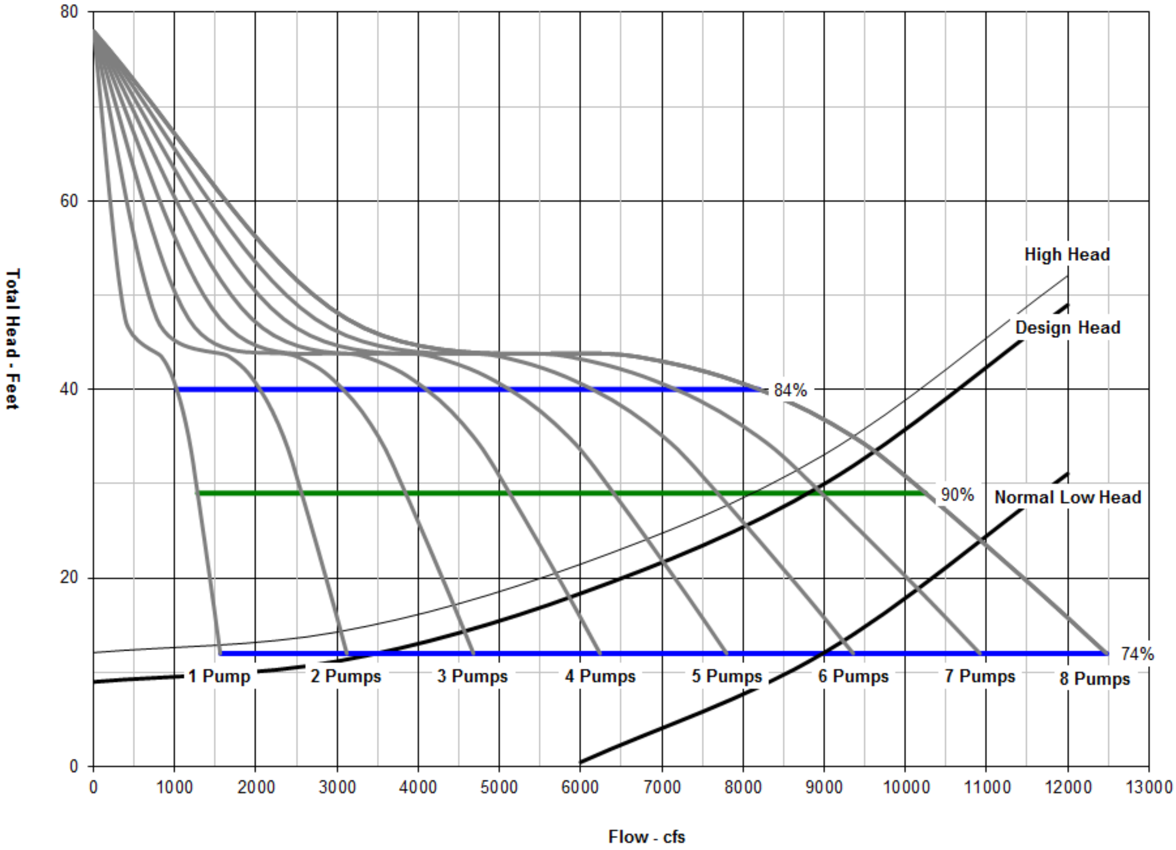

Figure 7-4: Pump and System Curves for 1 to 8 Large Constant Speed Pumps

As is illustrated on the constant speed pump curves (Figure 7-4), 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 7-3: Range of Flows for Variable Speed Operation at the Design Head and Normal Low-Head Operating Conditions

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

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.

7.1.3.6 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 7-5 through Figure 7-8 show pump and system curves utilizing eight, six, three, and one large 1,125 cfs pump and Figure 7-9 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.

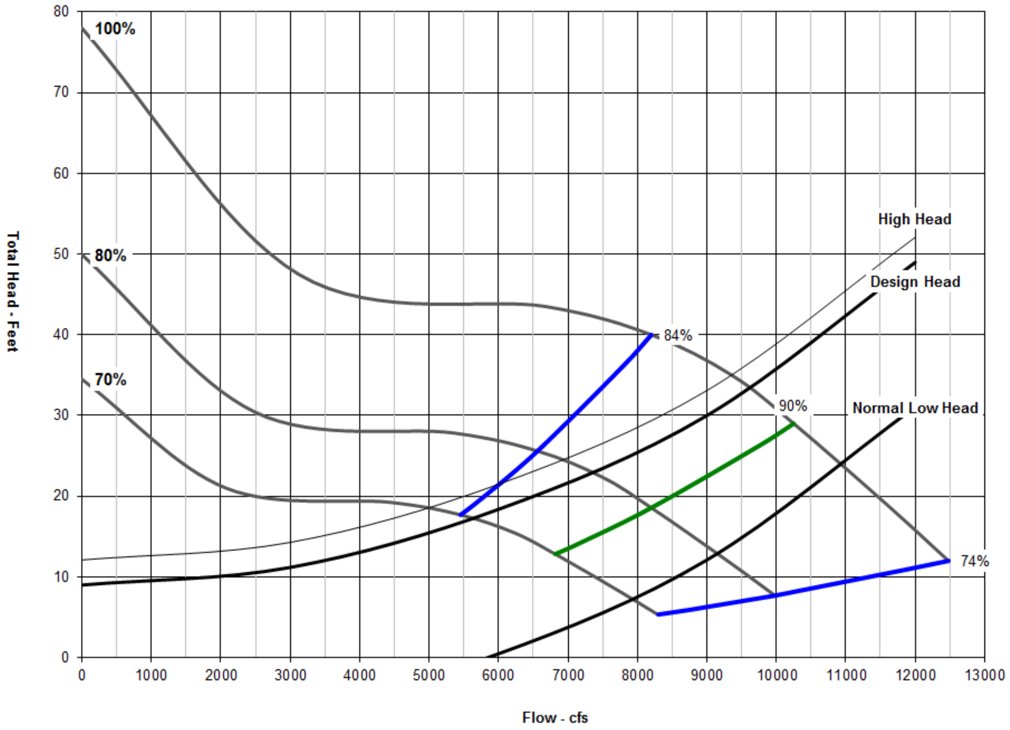

Figure 7-5: Pump and System Curves for 8 Large Variable Speed Pumps Operating

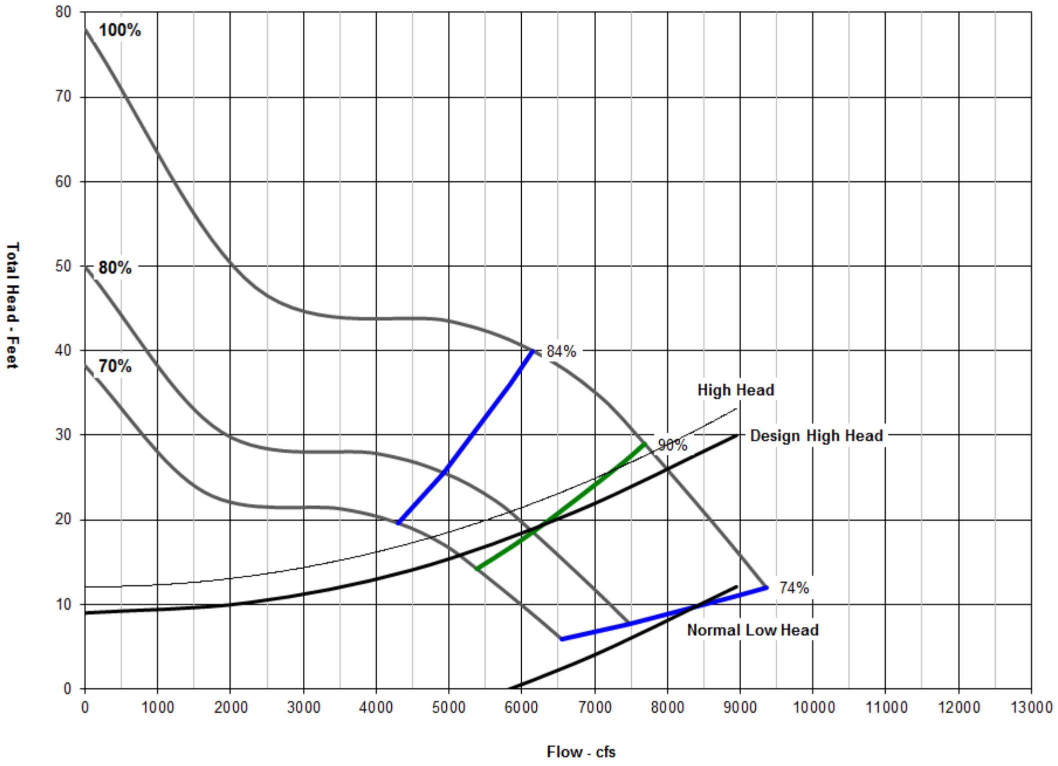

Figure 7-6: Pump and System Curves for 6 Large Variable Speed Pumps Operating

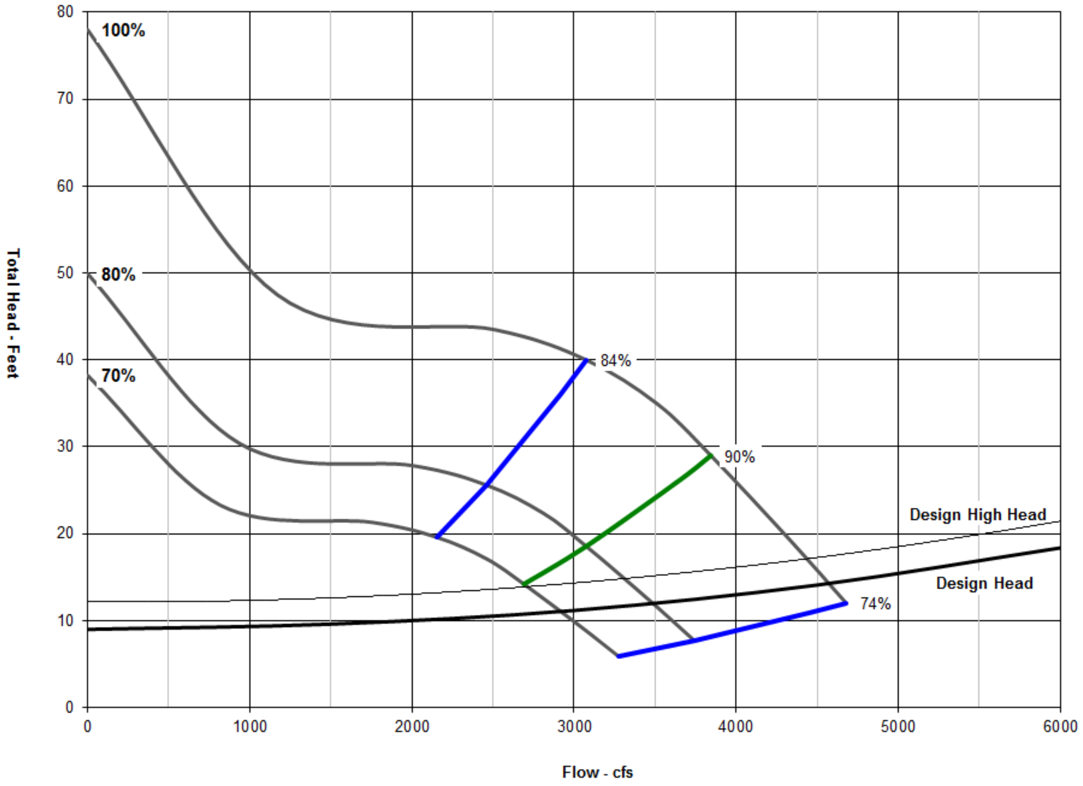

Figure 7-7: Pump and System Curves for 3 Large Variable Speed Pumps Operating

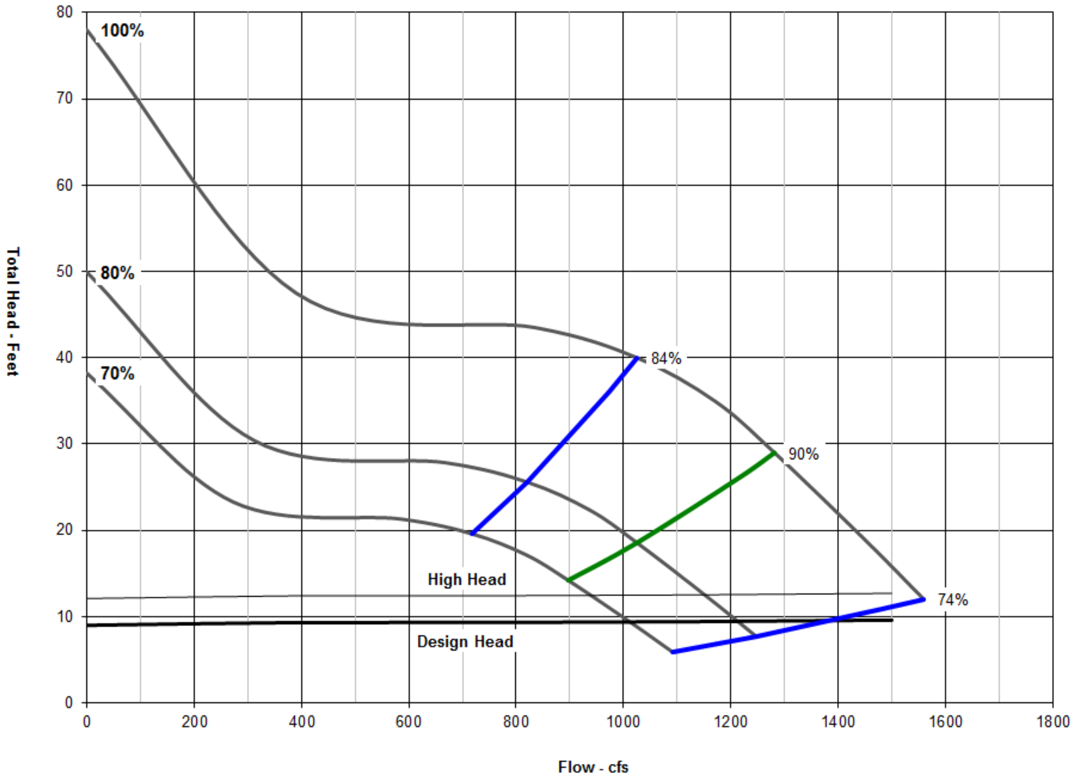

Figure 7-8: Pump and System Curves for 1 Large Variable Speed Pump Operating

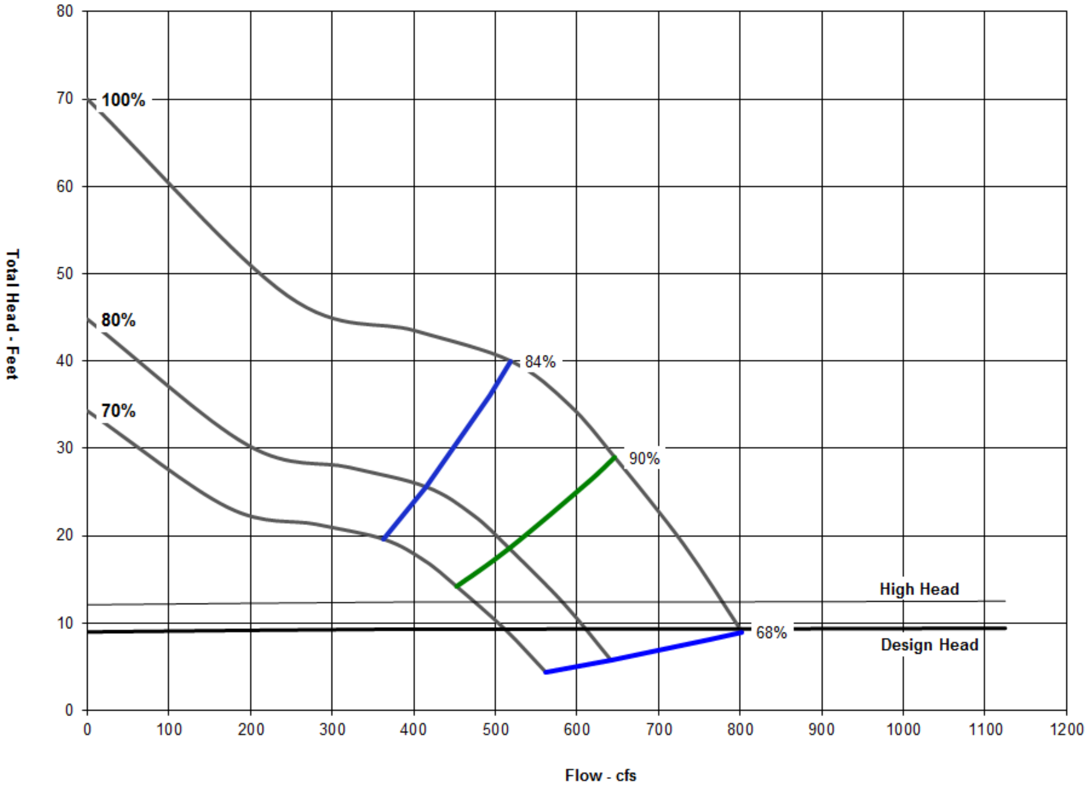

Figure 7-9: Pump and System Curves for 1 Small Variable Speed Pump Operating

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 7-4 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 7-4: Variable Speed Pumping Energy as a Percentage of Constant Speed Energy

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

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 NCCF 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 NCCF 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 7-4 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 NCCF and the anticipated large range from +1 to +14. If the normal WSE of NCCF was fixed at a smaller range and the pumps rerated for the smaller range, or if the normal WSE of NCCF 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 NCCF 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 NCCF has been finalized and pump selections have been made for the final system hydraulics.

7.1.3.7 Electrical

The Utility's 230kV transmission line and 230kV-115kV substation used during temporary tunnel construction will be repurposed and used to feed the long-term electrical needs of the Pumping Plant. Coordination with the Utility will need to take place regarding the 230kV-115kV substation because the construction-related loads will be greater than the permanent loads. If modifications to the substation are necessary upon completion of construction, these modifications will need to occur prior to repurposing the substation for the permanent installation. The feasibility of providing a backup Utility source shall be determined during the preliminary design phase.

The repurposed 230kV-115kV substation will feed a new substation that will step-down the voltage from 115kV to 13.8kV. The 13.8kV feeders will be routed into the electrical building's high voltage electrical room. 15kV arc-resistant, main-tie-main configured switchgear will then distribute the 13.8kV to the major loads, including the 6,000 HP pumps, the 3,000 HP pumps, the dewatering pumps, and the 13.8kV to 480V transformers.

Grounding methods and requirements will be further determined during preliminary design –specifically, the requirement for either a high-resistance or low-resistance ground at the new substation and the 15kV switchgear.

The current recommendation for VFD type if used is for a pulse width modulated (PWM) versus an over load commutated inverter (LCI) based on the current operating requirements. But this decision shall be further vetted during preliminary design.

The 480V feeders will be routed into the electrical building's low voltage electrical room to feed the low voltage motor control centers (MCCs). 480V arc-resistant, main-tie-main configured MCCs will then distribute power to all the minor loads. A standby emergency generator is proposed at the pumping plants for emergency operation of lighting, sump pumps, computer systems, and for other low-voltage needs at the Pumping Plants and Gravity / Isolation Shafts.

Working clearances will be provided per the National Electrical Code within the electrical building to allow for front and rear panel access to the switchgear and front access to each VFD and MCC. 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. Exhaust air from each VFD is vented directly outside through noise canceling filters.

Two (2) control rooms will be located within the electrical building. The overall control room will house the supervisory control and data acquisition (SCADA) system, which includes an HMI primary server, HMI secondary server, HMI historian, HMI domain controller, HMI engineering workstation, two (2) Operator View Clients, Ethernet switches, and two (2) PLCs. This overall control room shall be responsible for controlling and monitoring the entire process from the Intake Structure to the Pumping Plants and communicating with the Delta Field Division O&M Center, DWR Headquarters, and Joint Operations Center. The second control room shall only be responsible for controlling and monitoring the Pumping Plants and communicating with the SCADA system. A separate battery room with a separate ventilation system shall house the battery banks and shall be located within the electrical building.

Each motor is the totally enclosed, water-to-air cooled (TEWAC) type. The water-cooling system is sized for the required continuous flow of the connecting TEWAC heat exchanger and the motor thrust bearing oil reservoir. The motor cooling water system is anticipated to be a closed loop cooling water system consisting of a tank, heat exchangers, piping, and valves with a secondary raw water system. The secondary raw water system will pump water from the facility wet-well through the closed loop heat exchanger and discharge back to the wet-well. Each TEWAC closed loop cooling water system consists of a flow control station incorporating flow control valves, flow meters, flow and pressure indicating transmitters, and isolation valves.

7.1.3.8 Buildings

Each Pumping Plant will be housed in a building and have an associated dedicated Electrical Building. The main floor of the Pumping Plants will be the motor level at Elevation 25 feet, with the wet-well below. Overhead doors will provide access for large trucks to enter the main floor of the Pumping Plants.

Due to the preliminary layout of the pumps, the Pumping Plant buildings will be circular structures, each equipped with a bridge crane that rotates around the perimeter that will allow the entire main floor of the building to be accessed with the crane hook. Pump removal will be accomplished by first removing the motor with the bridge crane, followed by lifting the interior components of the pump out of the pump column. Due to height limitations, the pump will be removed in 2 or 3 sections for complete removal of the equipment. The pumps can be placed directly on service trucks inside the building for transport. Each pumping plant will have an equipment laydown and erection area to facilitate pump and motor removal and installation.

Access hatches in the main pumping room floor will facilitate wet-well access. In the unusual event that a pump column requires removal or maintenance, the hatches will facilitate access to the pump discharge head and any pump column supports located below the motor level.

Building mechanical system requirements will conform to the requirements of the State of California Title 24, the California Mechanical Code, and other applicable codes and will include heating, ventilation, and air conditioning (HVAC); plumbing; and fire protection systems.

Heat gain sources within the pumping plant building are a combination of the external weather conditions and the internal pieces of equipment and lighting. The primary heat gain inside each intake pumping plant facility will be from the TEWAC motors (maximum of 6 operating units: 4 at 6,000 HP and 2 at 3,000 HP). The primary heat gain inside each electrical building will be from VFDs, electrical switchgear and MCCs, and process instrumentation and controls equipment.

Each of the pump plant buildings is to be cooled using an evaporative cooling type system (with economizer components to use outdoor air for cooling and ventilation when feasible). Electric unit heaters or heating coils within the evaporative cooling units maintain the desired minimum temperatures within the facility. Where necessary, a packaged or central type air-cooled direct expansion refrigerant system is to be used.

Heat exchangers for the liquid cooled VFDs are along the electrical building exterior. Evaporative cooler units are along the exterior walls of the electrical buildings and pumping plants.

Two electrical buildings will be constructed, one for each pump plant. Each electrical building has a potable water supply for the restroom, service sink, and emergency shower and eyewash stations. The potable water supply system source is expected to be an onsite domestic well, but will be further developed during the final design phase. The restroom is compliant with the American Disabilities Act. Additional space can be made available to include shower facilities and storage lockers for Operations staff, if desired. The electrical buildings shall be kept at a constant low temperature by an HAVC system to protect the electrical equipment from overheating. The size of the system will be evaluated during the detailed design phase.

Fire suppression systems are in each pumping plant and electrical building in accordance with the requirements of the specific authority having jurisdiction. The types of fire suppression systems will be further evaluated during the detailed design phase of each facility.

A single control room will be constructed and will provide for either local or remote operation of the pumping facilities. The control room may be attached to one of the electrical buildings, or it may be a stand-alone building.

See Architectural Section of this report for additional information on buildings.

7.2 Construction Methodology

7.2.1 Constructability Considerations

7.2.1.1 General

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.

7.2.1.2 Site Access

Site access during construction via roadways will be provided for vehicle traffic from the east from Byron Highway via Clifton Court Road and the Italian Slough levee crest road or the NCCF embankment crest road. Access from the south will be provided from the Byron Highway via NCCF embankment crest road and West Canal levee crest road. The West Canal will be utilized for barge access once a barge unloading facility is established. It is assumed that the Italian Slough levee crest road will be kept open for public access to King Island. Re-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.

7.2.1.3 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.

7.2.1.4 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.

7.2.2 Construction Phases

7.2.2.1 Overview

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 for 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. 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.

7.2.2.2 Site Development

This phase includes preparing the site for construction, including completing necessary grading and mitigation measures (including import of soil for the temporary cofferdam, construction pad, and detention pond berms); setting up the construction trailer area; 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 the shaft and tunnel operations. For additional barge landing considerations, see Section 23.0.

Temporary Cofferdam and Detention Ponds. A temporary cofferdam will be required within Clifton Court Forebay followed by dewatering of the construction area to allow filling of the area surrounding the pumping plant shafts to El. 25. Detention ponds will need to be created at the beginning of this phase to handle storm water runoff during the entire construction phase. Some storm water treatment may be required to achieve discharge requirements during the site development phase.

Ground Improvement. Subgrade preparation and ground improvement will be required within the footprint area of the pumping plant fills. This will include stripping of vegetation and installation of instrumentation to monitor 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 NCCF berms, ground improvement will include installation of a slurry trench cutoff to El. -50.

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. On average, the thickness of peat and organic soil layers is assumed to range from approximately 10 to 15 feet, or extend from existing ground surface to about El. -20 to -25 feet. 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 feet. 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. In addition, the northeast corner of the construction fill pad will be prepared to use as a barge loading and unloading facility, for more detail descriptions, (see Section 23.3, Barge Traffic and Landing Facilities).

7.2.2.3 Shaft Construction

This phase 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 surge shaft overflow discharge channel will be completed during this phase and the site restored to a level surface prior to the TBM contractor’s work. Based on this assumption, the required deep excavations and shoring can be coordinated with the shaft construction excavation and shoring and not interfere with the tunneling or later construction phases.

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 phase.

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 crane mounted hydromills 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, final lining, and outlet channel construction;
- 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. Surge overflow channel construction may require drilling equipment for shoring installation as well as excavation, rebar and formwork assembly, and concrete placement. All this work is anticipated to be completed within the area utilized for the slurry wall construction.

7.2.2.4 TBM Tunneling

This phase 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.

TBM delivery is assumed via the barge loading/unloading facility at the West Canal, with transport of the TBM components to the shaft site using multi-axle carriers. Crawler cranes may also be used for transport of larger equipment.

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 (across West Canal or onto barges) or to a temporary muck storage area.

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.

7.2.2.5 Pump Station Construction

This phase includes all construction of the pumping stations, including all internal structural, mechanical, and electrical elements within the pumping plant shafts and associated surface facilities. During this phase, earthwork would be primarily limited to excavation for electrical tunnels connecting the substations to the electrical building and then to the pump station. This phase 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.

7.2.2.6 Site Completion

This stage includes completing the site to its final configuration, allowing for long-term tunnel access and maintenance. This will include removal of the temporary cofferdam in the NCCF, excavation of excess fill placed for construction within the NCCF, and final grading and installation of slope protection for the NCCF 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.

SECTION 8.0

Pipeline/Concrete Box Conveyance Systems

The MPTO/CCO 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 North Clifton Court Forebay for delivery to SWP and CVP export pumping facilities. See Section 11.0, "Tunnels," for a description of the tunnel system.

The MPTO/CCO uses box conduits within the Intake Facilities. See Section 6.0, "Intakes and Sedimentation Facilities," and Section 15.0, "Levees," for descriptions of these pipelines in the Intake Facilities.

8.1 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.

8.1.1 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.

8.1.2 Pipe/Concrete Box Material Type Alternatives

Pipeline or conduit alternatives have been investigated for application to the conveyance system for the MPTO/CCO 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.

8.1.3 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.

8.1.4 Pipe Hydraulics and Pressure Criteria

The conveyance system hydraulics is discussed in Section 5.0, “Conveyance System Hydraulics.”

8.1.5 Pipe/Concrete Box Cover Depth and Floatation

Pipe cover depth must take into consideration several factors, including (1) farming and agriculture needs; (2) high groundwater and floatation; (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 floatation. Preliminary data indicate that groundwater is within 1 to 2 feet of existing ground elevation. Appendix F, “Pipeline Floatation,” presents the pipeline floatation 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 floatation 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.

8.1.6 Other Construction Components

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

- In-situ cement-mortar lined, coal-tar epoxy coated steel pipe 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. Where installed in or across existing substantial drainage courses, the pipeline will be protected by 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 the 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 level of the groundwater table, dewatering facilities may also be considered post-construction for inspection, maintenance, or in the case of emergency. Two dewatering schemes are being considered: well point method and the deep well pump method.

Table 8-1 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 8-1: Summary of Pipeline Dewatering Alternatives

	Dewatering Alternatives	
Description	**Well Point**	**Deep Well**
Application Methodology	Dewater silts and sand above the bottom of planned excavation	Depressurization of sand underneath the bottom of planned excavation
Well Screen Diameter 2 to 4 inches 6 to 8 inches	Well Screen Diameter 2 to 4 inches 6 to 8 inches	Well Screen Diameter 2 to 4 inches 6 to 8 inches
Well Depth 20 feet 75 to 300 feet	Well Depth 20 feet 75 to 300 feet	Well Depth 20 feet 75 to 300 feet
Well Spacing	3 to 6 feet at top of trench 3 to 6 feet at intermediate bench	50 to 75 feet
Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute	Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute	Well Yield 1 to 10 gallons per minute 30 to 100 gallons per minute

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

8.2 Construction Methodology

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

8.2.1 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 MPTO/CCO 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.

8.2.2 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.

8.3 Maintenance Considerations

Maintenance of the conveyance pipelines is dependent on the materials of construction as summarized in Table 8-2.

Table 8-2: Summary of Pipeline Maintenance Considerations

Material and Conduit	**Configuration Maintenance Considerations**
Steel pipe	• Maintenance and operation of an impressed current cathodic protection system • Periodic internal inspections and repair of cement mortar lining.
RCCP or RCP	• Periodic internal inspections and repair of cement mortar lining at the joints • Periodic inspections of internal concrete • Repairs to concrete as needed, including sealing cracks and repairing spalling to prevent exposure of steel.
CIP	• Periodic inspections of internal concrete and joints • Repairs to concrete as needed including sealing cracks and repairing spalling toprevent 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

SECTION 9.0

Canals

The only new canals in the MPTO/CCO are the short bypass canal connections between NCCF and the approaches to Banks PP and Jones PP. The new canal to the Banks PP is approximately 2,000 feet long and connects to the existing Banks PP approach canal through a siphon under the Byron Highway and the SPRR. A 100-foot buffer will be provided between the toes of the new canal embankment and the extents of the existing traveled way (see Section 10.0, “Culvert Siphons - Shallow Crossings”). The new canal which connects the MPTO/CCO facilities to the existing Jones (CVP) canal tie-in is approximately 4,000 feet long. Both approach canals are included in the discussion of the NCCF (see Section 14.0, “Forebays”).

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SECTION 10.0

Culvert Siphons – Shallow Crossings

The MPTO/CCO uses inverted hydraulic siphons for shallow crossings under existing major waterways and roadways. These siphons are comprised of multiple-cell box culverts. Tunnels are used to cross below deeper waterways as described in Section 11.0.

10.1 Description, Locations and Site Plan

The MPTO/CCO includes culvert siphons at the following locations:

- New North Clifton Court Forebay (NCCF) Outlet Channel
- Byron Highway/Southern Pacific Railroad (SPRR)

The siphons will convey water from NCCF below the existing Clifton Court Forebay outlet channel and the Byron Highway/SPRR to the Banks PP intake channel. Siphons for the rail crossings are preferred over bridges because of the difference in grade between the planned canal water level and the existing railroad grade, with the canal water level being higher. Bridge approaches will require several miles in length to accommodate typical maximum grades associated with railroads.

The SPRR is a freight-only line that has been inactive since the early 1990's. Union Pacific, the current owners, has plans to reactivate this line sometime in the future with freight trains reaching speeds of 70 miles per hour (mph). Once operational, service must be maintained on this track, so a shoofly would need to be constructed to keep the track in service while the culvert siphon is constructed.

10.1.1 Proposed Culvert Size and Shape

The culvert size and shape was selected to optimize velocity and head loss. It is typical to size the culvert siphons to pass flows with a velocity range between 3 and 10 fps. Typically, culvert siphons are constructed with either circular or rectangular sections. Since the siphons required for this project are very large and are expected to be constructed within cofferdams or shored open-cut construction using reinforced concrete, the cells are rectangular. A typical siphon crossing is shown in Figure 10-1.

Since the culvert siphons are located at the terminus of the conveyance system, it is expected that most sediment would settle before reaching the siphons.

10.1.2 Dimensions and Levels

The main dimensions and capacities for each culvert siphon are shown in Table 10-1. The new approach channel invert levels correspond with the invert elevation of the existing approach channels to Banks PP and Jones PP and would be adjusted for the final hydraulic grade line (HGL). Invert levels at the low point of the siphon are controlled by the bathymetry, ground conditions, the need to establish full flow conditions, and depth of cover required to account for scour in the existing approach channel or prevent other damage to the structural integrity of the siphon.

For full flow conditions, the roof of the siphon is calculated to be approximately 15 feet below the lowest point of the crossing waterway or roadway/railroad. This will be verified with additional hydraulic calculations and coordination with owners of Byron Highway/SPRR. To resist uplift when dewatered, culvert siphons are supported on piles.

The new approach channel embankments are at the same elevation as those at NCCF, gradually sloping to the elevation of the embankments of the existing approach canals of the Banks and Jones PPs. The embankment will be higher than the existing ground level. At the Byron Highway and SPRR culvert siphon, a 100-foot buffer will be maintained between the toe of the canal embankment and the toe of the existing roadway.

The culvert siphon inlet and outlet structures are within the footprint of the forebay and canal embankments. These structures have flared wing walls to form a smooth transition from the much wider forebay or canal into the siphon, and vice versa.

10.1.3 Foundations

The geologic and geotechnical setting is described in Section 3.3. The native subsoils in the areas of Clifton Court Forebay and Byron Highway are clayey and silty soils underlain by interlayers of silty and clayey sand and clay and silt sediments of alluvial floodplain deposits. The geometry of the siphons is such that near surface organic soils and peat would be removed during construction. At this stage of engineering, and without detailed boring data at each siphon crossing, all siphon structures (inlet, barrel and outlet) are assumed to be supported on a pile foundation.

10.1.4 Control Structures

Control structures are at the inlet to the siphons to regulate upstream WSE and flow through the siphons. In order to isolate a siphon for repairs and inspections, stop logs will also be provided at the downstream end of the siphon barrel. Control structures are also located at the end of the new approach channels to control the amount of flow delivered to Jones and Banks pumping plants.

Radial gates will be used to provide control flow through the canal because they efficiently transfer hydrostatic loads through the trunnion and have for a lower hoist capacity requirement than other gates.

Each gate is actuated by electric motors. A typical radial gate is shown in the photo.

Typical Radial Gate on the Delta Cross Channel

10.1.4.1 Mechanical – Gates, Hoists, and Other

Each gate has an independent electric hoist for remote operation. Gate operation during a power outage is by portable generators.

The gates include bottom and side seals to control leakage while closed. Gate slots are not required.

10.1.4.2 Control Modes and Control Basis

All equipment at the control structures operates either locally or remotely. Table 10-2 describes the controls equipment for the siphon control structure.

Table 10-1: Summary – Proposed Culvert Siphons and Inline Control Facilities

Structure	**ID**	**Location**	**Flow (cfs)**	**Number of Box Culverts**	**Box Culvert Width (ft)**	**Box Culvert Height[3] (ft)**	**No. of Radial Gates**	**Estimated Siphon Length (ft)[1]**
Siphon	2	NCCF Outlet Channel	15,000	3	23	23	3	1,500
Siphon	3	Byron Highway/UPRR	10,300	2	23	23	2	1,000
Inline Control Structure (Banks PP)	1	NCCF Outlet Channel	10,300	NA[2]	NA[2]	NA[2]	3	650
Inline Control Structure (Jones PP)	2	NCCF Outlet Channel	4,600	NA[2]	NA[2]	NA[2]	2	476
Inline Control Structure	3	Cal Aqueduct	10,300	NA[2]	NA[2]	NA[2]	3	650
Inline Control Structure	4	Delta Mendota	4,600	NA[2]	NA[2]	NA[2]	2	476

[1] Siphon Length = Length of Transition Structures + Length of Control Structures + Length of Culvert Box.

[2] There are no box culverts associated with the inline control facilities.

[3] Box Culvert Height does not include culvert transition zone dimensions

Notes:

cfs = cubic feet per second
ft = feet
ID = identification
NA = not available

Table 10-2: Equipment Controls

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

PLC = Programmable logic controller
SCADA = Supervisory control and data acquisition
VFD = Variable frequency drive

10.1.4.3 Electrical – Cables, Ducts, and Conduits

Ducts or conduits in the walls of the siphon culverts carry power and/or control cables across the waterways.

10.2 Construction Methodology

The siphons are constructed as large multiple-box culvert structures using cofferdams, shoring, and open cut-and-cover construction methods with conventional CIP concrete structures. Cofferdams are used at the NCCF Outlet siphon. Shoring is used at the Byron Highway/SPRR siphon.

10.2.1 Duration

The construction duration and staging sequence depends on the length of the culvert siphon and the crossing. The siphon at NCCF Outlet may have to be constructed in two phases, each phase lasting approximately one year, unless flood control or regulatory restrictions shorten the work window. In the first phase, a temporary cofferdam is constructed approximately halfway along the length of the siphon and then the area is dewatered and excavated to the desired lines and grade. Half of the total length of the culvert siphon is constructed inside the cofferdam, temporarily plugged, and backfilled to the desired waterway bottom configuration. During the second phase, the cofferdam would be re-installed across the other half of the siphon, the area would be dewatered, and the remainder of the siphon would be constructed and backfilled. This way, the waterway can be used during construction with temporary construction zone restrictions. At the NCCF Outlet siphon, the connection from the existing CCF to the existing approach canal to Banks PP may need widening for planned use.

For the Byron Highway/SPRR siphon, construction is in one phase. Byron Highway may need to be re-routed around the construction area.

10.2.2 Construction Footprint

The following footprint is needed during construction:

- Construction and laydown for each siphon inlet – 15 acres.
- Siphon – 250 to 500 feet along the length of the crossing, plus area for re-routing the crossing. At NCCF Outlet siphon, this could include backup levees and/or bypass channels. The Byron Highway may include re-routing the highway.

10.3 O&M Considerations

10.3.1 Hydraulic Capacity and Sediment

The two principal criteria determining siphon size is flow velocity through the culvert siphons and the hydraulic head loss across the siphons. All siphons have a low point for sediment deposition. Higher water velocity flushes sediments through the siphon while increasing hydraulic head loss and pumping costs.

Sediment enters the siphons from the upstream waterway, from the canal floor or sides, and from windblown material. The California SWP Vol. II Conveyance Facilities Bulletin No. 200, November 1974, page 13, states:

> "Sediments can build up to sizable amounts in canal inverts and, where feasible, were excluded from the canal. Wind-blown soils in desert and agricultural areas, particularly during cropland preparations, can produce a considerable amount of sediment. Sediment traps were incorporated in the invert of the canal sections in certain reaches. These traps are rectangular hopper type structures…" (DWR, 1974a).

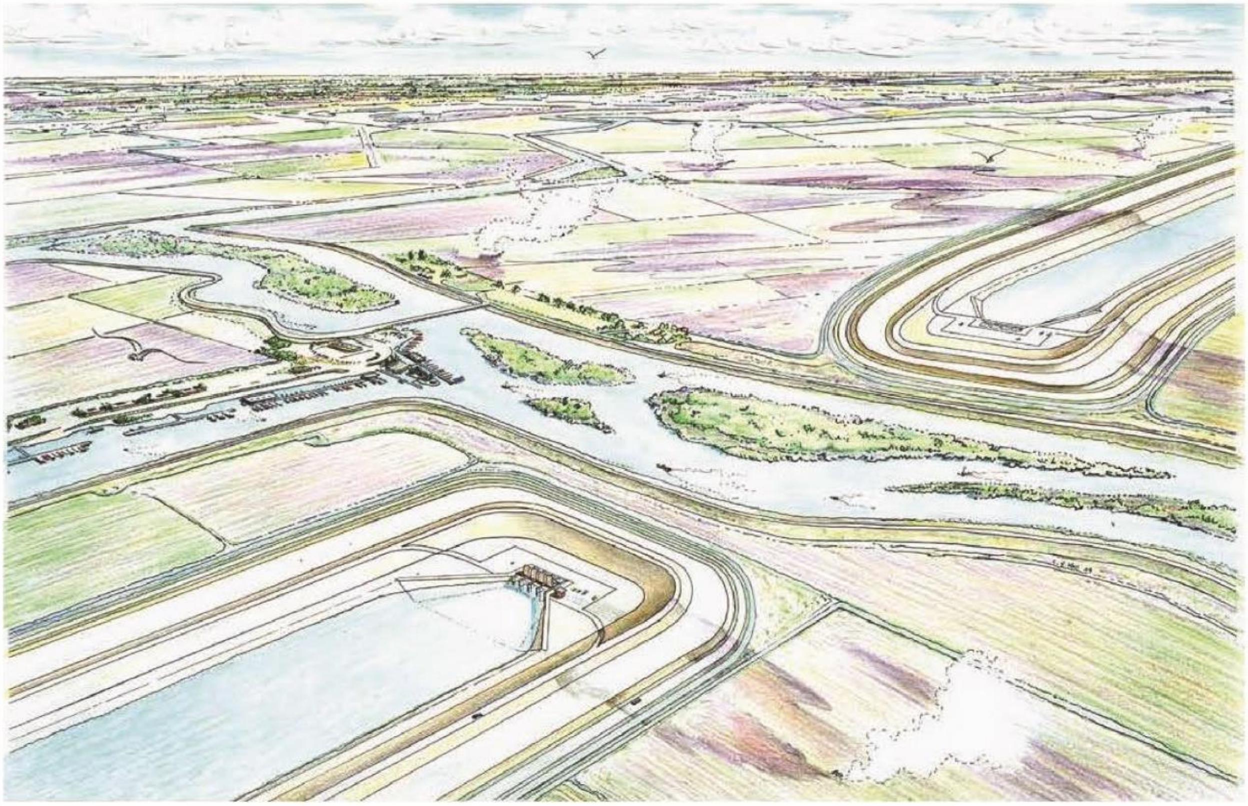
The water velocity used to keep siphons clean in previous projects was reviewed. By back calculation, the flow velocity of siphons for the Peripheral Canal (DWR, 1974b) was 8.8 fps. A velocity in the range of 3 fps is considered as a typical non-scouring velocity for the Delta soil conditions. A design water velocity of between 3 and 10 fps is a compromise between potential sedimentation and head loss and supports the proposed culvert sizing of the siphons.

10.3.2 Safety Concerns

Floating debris can enter the siphon intakes. To mitigate debris and to enhance public safety, floating barriers with hanging safety chains or trash racks are placed across the canals upstream of the siphons. Additionally, there are notice boards warning the public about the danger of the siphons.

10.3.3 Control Structures

Control structures consisting of inlet transition, gate structure, and outlet transition are at two locations in the new approach channel to allow for isolation of sections for maintenance. Stop logs and stop log channels are provided at the inlet and outlet structures to allow individual cells to be isolated, dewatered, de-silted, and maintained.



Final Draft
April 1, 2015

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

CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

Typical Siphon Crossing

FIGURE
10-1

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Figure 10-1: Culvert Siphon

SECTION 11.0

Tunnels

The MPTO/CCO 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 MPTO/CCO.

11.1 Description

11.1.1 Tunnel Reaches and Drives

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

Table 11-1: Tunnel Descriptions

Reach	Start of TBM drive	Completion of TBM drive	Internal Finished Diameter, ID (feet)	Number of Tunnel Bores	Flow Capacity (cfs)[a]	Tunnel Length (miles)
1	Intake No. 2	Intake No. 3 w/Junction Structure	28	1	3,000	1.99
2	Intermediate Forebay	Intake No. 3 w/Junction Structure	40	1	6,000	6.74
3	Intermediate Forebay	Intake No. 5	28	1	3,000	4.77
4	Intermediate Forebay	Staten Island	40	2	9,000	9.17
5	Bouldin Island	Staten Island	40	2	9,000	3.83
6	Bouldin Island	Bacon Island	40	2	9,000	8.86
7	North Clifton Court Forebay (NCCF)	Bacon Island	40	2	9,000	8.29

[a] Individual tunnel capacity listed. From the Intermediate Forebay to NCCF capacity is through both tunnel bores.

Notes:

cfs = cubic feet per second

ID = inside diameter

The size of each tunnel reach is dictated by the required hydraulic capacity 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), while Reaches 4 through 7 are designated as the Main Tunnels (see Figure 11-1).

All tunnels slope continuously from north to south, without siphons. The preliminary tunnel inverts 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 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.

11.1.1.1 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.

11.1.1.2 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.

11.1.1.3 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.

11.1.1.4 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 NCCF. The tunnels are constructed from 100-foot and 113-foot ID shafts at Staten Island, Bouldin Island, Bacon Island, and NCCF. Each tunnel is constructed from individual shafts, resulting in two shafts per work site, except possibly at Bouldin Island. The Bouldin Island shafts are used as the launching shafts for adjacent tunnel reaches, so this site may require four construction shafts. The Bacon Island and Staten Island shafts are reception shafts for adjacent tunnel reaches. The shafts adjacent to IF and NCCF will be used as launching shafts (see Figure 11-1).

11.1.2 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 6.0, “Intakes,” and Section 14.0, “Forebays.”

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

- Each North Tunnel bore starts and ends in control structures, except at the pumping plants.
- At the pumping plants, valves on the discharge piping will isolate the upstream end of the North Tunnels.
- At the discharge to the forebays and the inlets of the Main Tunnels at the IF, sets of primary and secondary fixed-wheel roller gates are used to isolate each tunnel and allow its dewatering for inspection.

For the Main Tunnels, control structures are adjacent to the forebays (intermediate control structures are not provided). The control structures also serve as transitions between the tunnels and forebays or between different diameter tunnels (the Junction Structure, for example).

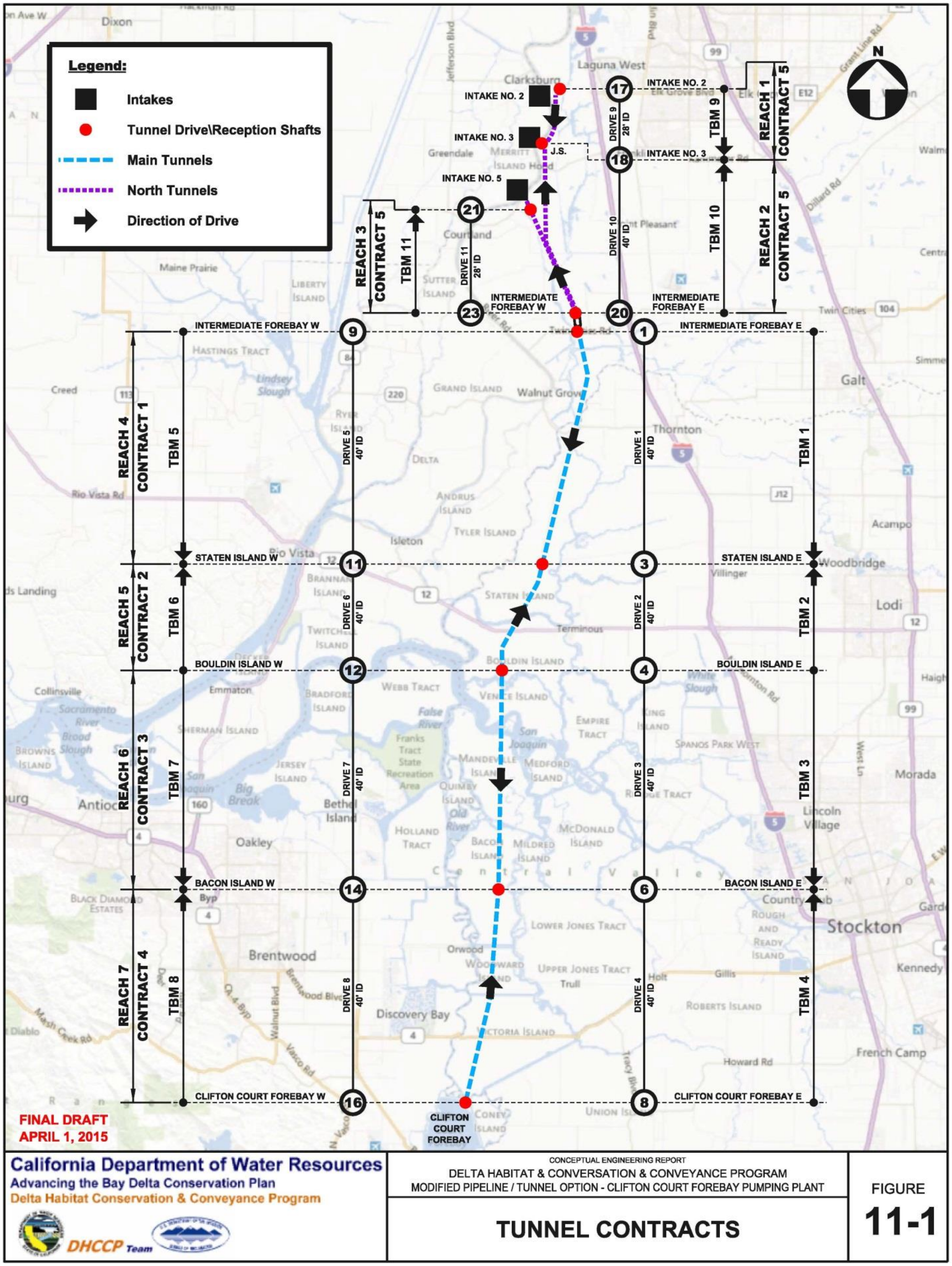

Figure 11-1 Tunnel Reaches

11.1.3 Dewatering and Filling

Piping and valve assemblies are provided to facilitate tunnel dewatering at the most downstream shaft on each tunnel (the junction structure at Intake No. 3, two North Tunnel shafts at the IF inlet structure, and the two Main Tunnel shafts at the NCCF inlet structure). The assemblies are installed at one or more other shafts on the Main Tunnels to speed dewatering. The pipe assemblies are installed within the annular fill area between the larger construction shaft and the smaller finished shaft to connect the piping from the surface to the tunnel invert. 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 pipe and valve assemblies can also be used to fill from the IF to the North Tunnels adjacent to the IF and from the NCCF 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 IF, NCCF, Intake No. 3 Junction Structure, 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.

11.1.4 Finished Facilities Security

Finished surface facilities for the tunnel system consist primarily of the top of the drive/reception shafts. Openings on the top of the shafts are securely locked, and each of these sites is fenced. 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

This section summarizes the assumed construction techniques and the sequence of activities. The final selection of tunneling methods and the number of construction sites will be determined according to ground conditions and construction schedule. Based on current tunneling technologies and general contract practice, the conceptual design designates tunnel construction reaches and sites for overall project planning.

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.

11.2.1 Advance Works

To prepare for the startup of the Main Tunnels, pre-tunneling construction work will be required. For North Tunnels construction, preparatory work consisting of site development similar to the Main Tunnels will be required.

11.2.1.1 Access Routes

The size, weight, and volume of construction-related traffic requires that the access routes to each site are analyzed to determine what changes 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 best management practices (BMP).

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 issues, the soft ground condition is 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 feet on center 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 tunnel construction, it is advantageous to commence these operations early so that the overall project schedule can be improved. The goal is to form the access road onto the site from the nearest highway, improve the load-carrying capacity of the site within the tunnel work area, install wick drains and necessary drainage blankets, and form the permanent elevated platform for the drive and reception shafts.

11.2.1.2 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 in time to avoid the use of onsite power generation. As discussed in Section 19.2, as soon as the 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 from 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 NCCF, 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 19.0, “Power Supply and Grid Connections,” for an overview of power supply options.

11.2.1.3 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. Re-usable tunnel material (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.

11.2.2 Mobilization

During mobilization, development of a workflow, establishing construction manpower, stockpiles materials, and stations equipment at the construction site will be established. The work area should be set up to best expedite construction activities; locate offices, warehouse, staging, or laydown areas; and consider the optimal configuration to manage labor, materials, and equipment in and out of the site.

11.2.3 Shaft Construction

11.2.3.1 Arrangements

Shafts are required along the proposed tunnel alignment to facilitate construction, operation, and maintenance of the conveyance system. During the construction phase, shafts are used to launch the TBMs to initiate tunnel mining, support their operation, and retrieve the TBMs on completion of the tunnel. After construction, the 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.

Some shafts are only used as launching or reception shafts. The Bouldin Island shafts are drive shafts to support the construction of the tunnels of Reaches No. 5 and 6. The Junction Structure shaft and the Staten and Bacon Island shafts are reception shafts for all adjacent tunnel reaches.

In the event that major TBM repair is 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 the purpose of TBM repair and to provide a safe haven during construction. These shafts will be located along the tunnel alignment. Once all necessary repairs are complete, the construction access shafts will be backfilled to pre-construction conditions.

All shafts are circular in plan to provide a more efficient structure with thinner walls and less obstructive bracing. The Main Tunnels and the North Tunnels (Tunnel Reaches 2 and 3) are assumed to be spaced at 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. The size of the temporary construction shafts for the tunnels will be set by the size of the tunnel constructed and constrained by the space required to undertake the drive of the TBMs into the currently anticipated ground conditions.

Anticipated shaft and tunnel configurations for the North Tunnels are as follows:

- For the North Tunnels, 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. These minimum sizes for construction-phase only are constrained by the space required to undertake the drive of the TBM into the currently anticipated ground conditions.
- 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 the river intakes 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 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. After mining, the north tunnel construction access shafts will be backfilled..

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 feet.

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 drive or 100 feet ID for reception of a 40-foot ID tunnel. It is anticipated that a shaft would be constructed along each tunnel drive to allow the construction contractor the ability to repair the TBM prior to completing each tunnel drive. These shafts are assumed to be 85 feet ID during construction to retain adequate structural integrity after the formation of the tunnel eyes, however these shafts will be utilized during construction and backfill to preconstruction condition, the size of these shafts will be determined by each construction contractor. The finished sizes of the shafts for the Main Tunnels are assumed to be 20 feet ID minimum to allow future operation and maintenance.

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 Table 3-4, the finished shaft area pad elevations are approximately 32 to 34 feet msl, except for NCCF, 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 3.0, "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 initial shaft construction 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 earthworks, 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 in Section 11.4.1.

Fill quantities required for the tunnel shaft area site pads and RTM disposal quantities generated at these sites are listed in Table 11-2.

Table 11-2: Tunnel Shaft Site Pad Fill Area and RTM Disposal Acreage for North and Main Tunnels

Site	Work Area Site Pad Fill Volume (cy)	In-Place RTM Volume (cy)	RTM Disposal and Top Soil Storage Area(1,2) Site (acres)
North Tunnel Shaft - Intake No. 2 (RTM and launch shaft)	-	517,880	54
North Tunnel Shaft –Intake No. 3 (Junction Structure\ Reception Shaft)	-	-	-
North Tunnel Shaft –Intake No. 5 (Reception Shaft)	-	-	-
IF : North Tunnel and Main Tunnel (RTM and Launch shafts)	-	3,918,464	405
Staten Island Reception Shafts	429,000	-	-
Bouldin Island Launch Shafts	640,000	11,701,184	1,209
Bacon Island Reception Shafts	141,000	-	-
Clifton Court Forebay (RTM and Launch Shaft, and CCF dredge material)	900,000(3)	14,568,400	903
Totals	2,110,000	30,705,928	-

1) Assumed 1 foot of topsoil stripped and stored.

2) Disposal areas filled 6 to 10 feet high for all sites except for CCF and Glanville Tract, which will be 10 to 15 feet high.

3) Fill pad volume is for CCF 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.

11.2.3.2 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 construction of this shaft will be designed to accommodate flows to meet hydraulic and structural criteria. The preliminary shaft size is 113 feet ID, with final size to be determined during preliminary design. Layout of the junction shaft and intake arrangement is shown on the Concept Drawings (Volume 2).

The finished permanent pad level of the junction shaft is set for protection against flooding similar to all other shafts. The high-water elevation inside the junction shaft will be determined based on detailed hydraulic analysis during preliminary design.

11.2.3.3 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 10 feet long and 4 to 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.

11.2.3.4 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 an unreinforced mass concrete plug to withstand ground water pressure, with optional relief wells to relieve uplift pressure during tunnel construction.

11.2.3.5 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. Additional equipment is also attached to this structure during the drive and reception of the TBMs to prevent inundation by water or soil.

11.2.3.6 Secondary Lining

Slurry wall construction techniques do not typically produce a permanent water-tight structure, but minimum water infiltration can be controlled with tight shaft construction tolerances and surrounding ground improvement/treatment. For the permanent shaft structure (such as the CCO pumping plant), a secondary final lining will be required inside the shaft. The finished diameter of the permanent access shaft will be determined by operation requirements, and it is not necessary to retain the construction-sized shaft openings for maintenance. For shafts that convey water, the finished clear internal diameter will be determined based upon hydraulic requirements. For ventilation shafts, the clear internal diameter of the finished shafts will be equal to the finished diameter of the adjacent connecting tunnel.

In terms of leakage, the permanent shaft lining needs to be designed to a standard suitable for a water-retaining structure. The annulus between the diaphragm wall and the final lining can be filled with select tunnel excavation, low-density fill, or backfill slurry. The reduced diameter of the permanent shafts also reduces the span of the roof slabs, some of which might be subject to internal pressures. The shaft lining will also be designed to withstand seismic loads.

11.2.4 Concrete Supply

The provision of a large volume of concrete from a reliable source is critical for the diaphragm wall, tremie plug, and secondary lining operations. Concrete batch plants are expected to be placed at each shaft site.

11.2.5 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) TBMs.

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. Grounds containing a high percentage of fines and clays put excessive demand on the slurry pumping system, and slurry TBM may not be feasible under such conditions. A slurry system will likely have a higher power demand than a similar EPB system.

The tunneling methodology for the MPTO/CCO 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.

11.2.6 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 not anticipated that a PVC T-lock liner is required, and the PVC liner could complicate the tunnel construction and long-term operation. Once detailed geotechnical data is available during preliminary design, the segment liner will be designed to limit water leakage by considering surrounding ground-liner interaction and ground 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 the majority 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 CCF, 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 preliminary 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.

11.2.7 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. Additional segment storage space needs to be added to the plant space requirements and could be several times the space required for the plant. The segments can be transported by barge, rail, or truck where these modes of transport are available.

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.

11.2.8 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 substantial. It is currently expected that the TBMs complete the final stage of their delivery to the site by road.

11.2.9 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 hauling or transporting to 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 has been assumed) is stripped and stockpiled and used for RTM pile cover following the placement of the tunnel spoils.

RTM volumes and proposed disposal areas for the North Tunnels and Main Tunnels are shown in Table 11-2.

The maximum anticipated height of tunnel spoil placement is currently planned at 6 to 10 feet for all sites except the NCCF 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.

11.3 Conceptual Tunnel Construction Schedule

11.3.1 Shaft Schedule

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.

It is expected that the adjacent shafts on each tunnel reach or reception site operated by one contractor are constructed concurrently to make efficient use of the bentonite processing plant and site concrete batching facilities.

Due to the number of shafts that are constructed concurrently, it is expected that there is a constraint 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.

Typical durations for shaft construction once the shaft working pad area has been completed, including substantial completion of anticipated ground settlements and mobilization of equipment for guide wall construction, are shown below:

Shaft Working Pad	96w days
Guide Walls	
• Piling for Guide Walls	10w days
• Excavate and Support Guidewalls	15w days
• Construct Guidewalls	10w days
Diaphragm Walls	
Mobilize and Setup for Diaphragm Walls	15w days

Construct Diaphragm Walls

A 113-foot ID diaphragm wall requires 37 panels. At 10 feet of perimeter length and 3w days per panel average, 110w days are required. A 100-foot ID shaft requires 33 panels and 98w days

Demobilize Diaphragm Wall Subcontractor	15w days

Shaft Excavation

Excavate in Wet	75w days

Excavate 226 feet of depth at an average of 3 feet per day.

Base Slab

• Construct Tremie Base Slab	30w days
• Tunnel Eye and Headwall	30w days
• Ground Treat Tunnel Eye(s)	30w days
• Preparation for TBM Break-out/Break-in	30w days

Tremie formation is based on 1 foot per day average production.

The information above suggests the following minimum construction periods:

- 370w days for a 113-foot ID Main Tunnel shaft
- 360w days for a 100-foot ID shaft

This schedule information will need to be reviewed when finishing details are developed in more detail.

Concurrent activities with shaft construction include:

Grouting

• Mobilize Grouting Equipment	10w days
• Vertical Grouting to Tunnel Eyes	10w days
• Demobilization	10w days

Dewatering

• Mobilize Dewatering Equipment	10w days
• Install Dewatering	20w days
• Control/Dewater as Required	Varies
• Demobilization	10w days

Completion activities after tunnel construction is finished include (assuming the deepest shafts at NCCF):

Secondary Lining

• Construct Benching	20w days
• Internal Structures	20w days
• Secondary Line to Full Height of Shaft	65w days

Shaft Finishings

• Construct Roof Slab	40w days
• Demobilization	20w days

Therefore, the completion activities for the deepest shafts at NCCF require 165w days. This schedule will need to be reviewed when finishing details are developed in more detail.

11.3.2 Tunneling Schedule

North Tunnel Reach Nos. 2 and 3 are assumed to be driven from the IF to the reception shafts near Intake Site No. 3 (Junction Structure) and adjacent to 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 adjacent to Intake No. 2 to the Junction Structure near Intake No. 3. Once the activities at the intakes are further developed, it might be necessary to reconsider the direction of some or all of these drives.

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. Once the overall schedule has been developed and other factors are evaluated, this might be reconsidered.

Based on the required construction schedule, and using industry average tunneling rates for the anticipated ground conditions, including contracting considerations, 11 TBMs are assumed to be required (Table 11-3). The lead time required from procurement (NTP) to design, build, and ship new machines on-site is approximately 1.5 years, with an additional 6 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 11-3: Tunnel-boring Machines

Reach	Number of Tunnel Bores	Outside Cut Diameter (feet)	Number of TBMs Required
1 and 3 (North Tunnels)	Single	31	2[a]
2 (North Tunnels)	Single	44	1
4 through 7 (Main Tunnels)	Dual	44	8

[a] If driven concurrently, two TBMs assumed for the conceptual tunnel construction schedule

Note:

TBM = tunnel-boring machine

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 rate of 35 feet per day was assumed.

When two TBMs are being driven from adjacent shafts on a single site by the same contractor, it might be required that a minimum distance be specified to stagger the drive so that potential ground support loss due to proximity to tunneling can be minimized. For the program cost estimate, a TBM advance rate of 35 feet per day was assumed.

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 it is assumed that a TBM might not enter a shaft to complete its drive if the TBM driving away from that shaft is still constructing its tunnel or if another TBM using the same reception shaft has not completed its drive or has not been completely removed. These situations create physical, contractual, and health and safety interfaces that must be addressed during future project development.

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. A starter tunnel can be built to provide proper TBM drive, but ground improvement/treatment must be provided in advance of this feature. The TBM must be launched with limited backup equipment; and 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.

Daily and weekly maintenance of the TBM are required. In addition to regular maintenance, it is possible to plan major maintenance stops, and an allowance of 4 calendar weeks has been included for each tunnel drive between shafts and at vent shafts. There is a residual risk that unplanned stoppages requiring major maintenance or repair might also be required, and an allowance of 4 calendar weeks has been included for unplanned stoppages for each tunnel drive. The entry to the shafts and exit from shafts will limit tunnel production as there is no tunnel footage gained on the schedule. An allowance of 2 calendar weeks has been included to receive a TBM into a shaft, and an allowance of 6 calendar weeks is included to exit a vent shaft and advance 600 feet from that shaft.

Environmental documentations and permit conditions will be issued as a part of the contract documents. All bidders are required to review and incorporate such requirements prior to submitting the bids. Using the information described above, the following is a summary of the conceptual tunnel construction schedule components.

Launch TBM to 600 Feet Fully Assembled	24 weeks
Drive 200 Feet per Week to Next Shaft	Varies
Planned Maintenance	4 weeks
One Unplanned Intervention	4 weeks
Launch TBM to 600 Feet from Shaft	6 weeks
Drive 200 Feet per Week to Next Shaft	Varies
Planned Maintenance	4 weeks
One Unplanned Intervention	4 weeks
Arrival at Reception Shaft	2 weeks
Disassemble and Remove	12 weeks

11.4 Safety

11.4.1 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.

11.4.2 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 beneath the anticipated alignment for the tunnels. As these gas fields are present near 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. The TBMs 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 ventilation requirements, as well as special access and egress requirements, are imposed by Cal/OSHA (at a minimum). Additionally, all equipment used in the tunnels needs to be intrinsically safe.

11.4.3 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.

11.4.4 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.

11.4.5 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 Public Draft 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.

11.4.6 Ground Improvement

Ground improvement is 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.

11.4.7 Behavior Under Seismic Events

The preliminary design ground motions for the tunnels and shafts have an average annual recurrence interval of 1,000 years, as discussed in Section 3.0, “Overview of Conveyance Option.” 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. Also, as noted in Section 3.0, 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.

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.

11.5 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.

11.6 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.

SECTION 12.0

Bridges – Road and Railroad

With the exception of Highway 160, Highway 12, and Byron Highway, the MPTO/CCO does not include surface intersections or require new bridges for public roads.

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 15.0, "Levees," for sequencing of the relocation work.

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.

12.3 Byron Highway

Byron Highway will be temporarily rerouted during construction of the culvert siphon connecting the new approach canal and the existing Jones PP approach canal.

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SECTION 13.0

Utility and Infrastructure Crossings

This section identifies utility crossings associated with the MPTO/CCO 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, “Bridges – Road and Railroad.”

MPTO/CCO 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 MPTO/CCO 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 MPTO/CCO forebays and stockpile , 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, minimal conflicts are anticipated.

13.1.4 Natural Gas Wells and Fields

The gas fields crossed by the ROW include the Snodgrass Slough, Merritt Island, and River Island fields. Figure 13-1 shows the approximate location of known gas wells and gas fields. These gas fields lay 1,200 to 9,000 feet bgs, and no conflicts with the fields are anticipated.

Available information indicates plugged and abandoned natural gas and oil wells in the conveyance footprint and within borrow, spoil, or muck areas.

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 MPTO/CCO conveyance tunnels will be constructed well below the aqueduct at the north end of Woodward Island, and no conflicts are anticipated.

13.1.6 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 evaluated. Minimal conflicts are anticipated for the tunnel alignments.

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

13.2.1 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.

13.2.2 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.

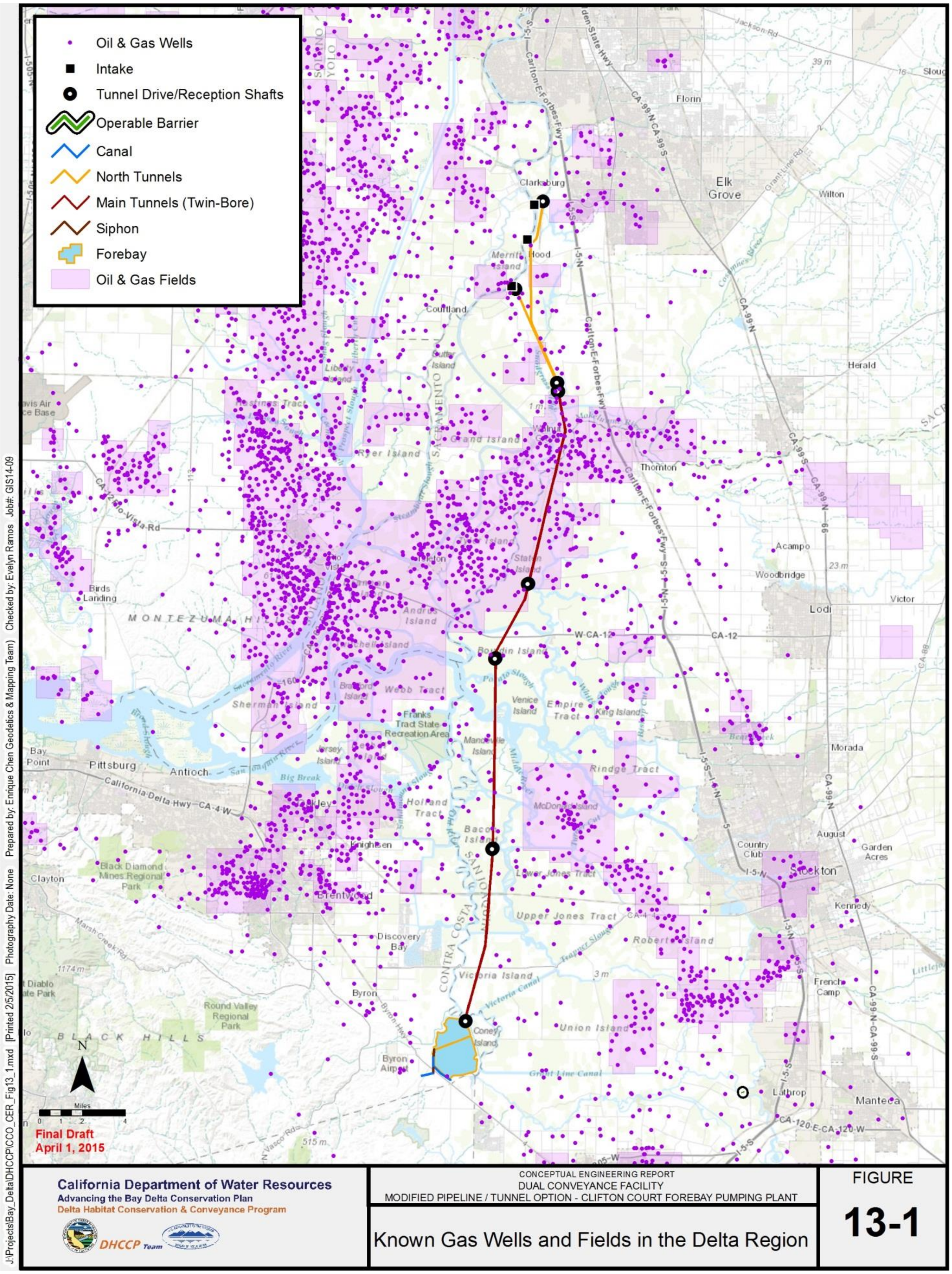

Figure 13-1: Known Gas Wells and Fields in the Delta Region

13.2.3 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.

13.2.4 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.

13.2.5 Mokelumne Aqueducts

The main tunnels from the IF to NCCF 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.

13.2.6 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/S

13.2.7 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.

SECTION 14.0

Forebays

14.1 Description and Site Plan

The MPTO/CCO includes two forebays for the following applications:

- The Intermediate Forebay (IF) is mainly a pass-through facility providing an atmospheric break at the inlet to the Main Tunnels from IF to North Clifton Court Forebay (NCCF). 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.
- North Clifton Court Forebay (NCCF) provides 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.

The existing CCF is expanded into the tract area adjacent and immediately south of it. This modified CCF will be divided into two forebays: NCCF, which will take water from the north Delta intakes, and South Clifton Court Forebay (SCCF), which will receive water through operation of the existing intake gates.

14.1.1 Intermediate Forebay

14.1.1.1 Requirements and Assumptions

Hydraulic Connections. The IF is designed to be hydraulically isolated from other Delta waterways. The only source of water 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 size of the IF in conjunction with the operation of the pumping plant at CCF.

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 is a pass-through facility with no daily operational storage and no regulation of flow to the Main Tunnels. The IF is the smallest practical size to provide sufficient velocity reduction to deposit sediment not removed at the intake sedimentation basins.

Existing Embankments. The existing embankments and levees of the Zacharias Ranch, Glanville Tract, and Snodgrass Slough near the IF do not meet DHCCP flood protection criteria of crest elevation +32.2 feet. It is assumed that the existing embankments and levees also do not meet DHCCP seismic criteria and cannot be raised to meet the flood protection elevation. The IF embankments are designed to meet the required flood protection.

Seepage. A slurry cutoff trench beneath the 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.

The 4160V to 480V transformers will feed the low voltage switchboard via an automatic transfer switch (ATS). The switchboard and the ATS will be located within the electrical building's low voltage electrical room. The 480V switchboard will then distribute power to all the minor loads.

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

Working clearances will be provided per the National Electrical Code within the electrical building to allow for front and rear access to the switchgear and front access to the switchboard. 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.

A control room will be located within the electrical building. This control room shall be responsible for monitoring and controlling the IF and communicating with the SCADA system.

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 engineering. 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 sites 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.

14.1.1.2 General Description

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 and Invert Elevation. 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 the28-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.

The ground beneath the shallow foundation of the reinforced concrete inlet structure might have to be improved to increase soil strength and prevent seismically-induced settlement. Ground improvement will be further evaluated when site-specific subsurface data becomes available.

Outlet Structure and Invert Elevation. The outlet structure is at the south end of the IF. 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 NCCF 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.

The ground beneath the shallow foundation of the reinforced concrete outlet structure might have to be improved to increase soil strength and prevent seismically-induced settlement. Ground improvement will be further evaluated when site-specific subsurface data becomes available.

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.

Trash 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 DHCCP 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. Dependent 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 slopes could be steepened. 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.

14.1.1.3 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 Glanville Tract indicate low potential for liquefaction.

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 3.0, “Overview of Conveyance Option”). 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.

14.1.2 North Clifton Court Forebay

14.1.2.1 Requirements and Assumptions

Hydraulic Connections. NCCF is conceptually designed to be hydraulically isolated from other Delta waterways. The only source of water will be from the north Delta intakes. The only outlet from NCCF will be the new approach channel connecting to the existing Banks PP approach channel and Jones PP approach canal.

Fish Protection. Fish are excluded from NCCF, as they are throughout the entire MPTO/CCO system. The NCCF connection to the approach channel to Banks PP is downstream of the existing Skinner Fish Facility (upstream of the Banks PP), and the NCCF connection to the approach canal to Jones PP is downstream of the existing Tracy Fish Collection Facility (upstream of the Jones PP).

Head Loss Allowance. Approximately 1 foot of head loss is anticipated in the new approach canal between NCCF and either the Banks PP or 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 the day, a forebay is proposed to balance inflow with outflow to support the Jones PP. Similarly, a forebay is necessary to enable the Banks PP to maximize its operation when electrical power rates are lowest (off-peak). Preliminary calculations indicate an operational storage requirement between 4,400 and 5,900 AF (see Section 4.0, "Conveyance System Operations").

Active Storage Availability. The area of the proposed NCCF is within the improved CCF perimeter embankment, with a new divider embankment separating CCF into two cells. The water surface area of NCCF is approximately 806 acres (Elevation +1.1 feet) at minimum pool elevation. As described in Section 4.0, “Conveyance System Operations,” the operating ranges for the isolated north Delta and dual operations scenarios would be +1.1 to +7.1 ft and +5.1 to 14.7 ft, respectively. These operating ranges result in approximately 4,970 AF and 8,100 AF of potential active storages in NCCF for the isolated north Delta and dual operations scenarios, respectively. Additional operating storage can be obtained if the operating range is increased, which appears feasible. Otherwise, the Banks PP may need to pump during a short portion of the on-peak pumping period.

Seepage. Some areas of the NCCF foundation might be subject to significant underseepage and piping, based on silty sand and clean fine sand layers encountered in the soil borings and CPTs. A slurry cutoff trench will be constructed beneath the new embankment to protect the foundation of the embankment. Based on the available subsurface data, the cutoff trench will extend down to an elevation of -50.0 feet. A drain at the toe of the outer embankment slope will limit saturated conditions at the ground surface.

Excavated Material. Limited soils information and subsurface data show that soils in the vicinity of NCCF consist predominantly of silty and sandy clays and are generally overlain by 2 to 10 feet of tidal peat and mud. This depth is simplified to an assumed 6 feet of peat for this stage of the forebay engineering. It is estimated that approximately 30 percent of the materials below the peat layer, excavated as part of the proposed NCCF cut, can be reused for embankment construction.

Dredging. The NCCF is dredged to the approximate original design elevation of CCF (Elevation -5.0). Limited soils information and subsurface data show that soils within the vicinity of NCCF consist predominantly of silty and fine sand. It is estimated that approximately 50 percent of the dredge materials will be reusable for embankment construction, levee fortifications, and other applications within the Delta.

Dam Regulatory Authority. It is assumed that the NCCF 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 NCCF 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.

14.1.2.2 General Description

Location. NCCF is located in the northern half of the existing CCF. Water is delivered to NCCF 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 7.0 “CCF Pumping Plant.”

Bottom Elevation. The design bottom elevation for NCCF is -5.0 feet. The bottom elevation of the existing CCF was set at -6.9 feet (DWR, 1974). While specific information is limited, significant sedimentation has occurred since original construction. On occasion, DWR has dredged to maintain a channel. The constrained flow area is reported to result in approximately 1 foot of head difference from the inlet to the outlet of CCF at high flows. More significantly, CCF experiences problems with pond weed, which builds up on the Skinner Fish Facility and hinders flows to the Banks PP. Its growth is related to detention time, velocity through the forebay, and water depth. Harvesting machinery operates on a continuous basis during the year to remove pond weed. NCCF will have to be maintained with regular harvesting, dredging, and cleaning. For additional forebay maintenance considerations, see Section 14.3.

Siphon to Approach Canals. A siphon structure is underneath the existing CCF outlet to a new approach channel. The inlet to the siphon is at the southwest corner of NCCF and goes to the transition structure of the new approach channel. For additional siphon information and considerations, see Section 10, “Culvert Siphons -- Shallow Crossings.”

Connections to Pumping Plants. A section of the new approach channel, approximately 7,000 feet long, connects NCCF to the existing approach canal leading to the Banks and Jones PP.

The bottom of the new approach channel drops from the forebay bottom elevation of -5.0 feet to match the depth at the existing approach canal to the Banks PP at the connection point. A control structure with two sets of radial gates will be installed at the downstream end of this new approach channel to hydraulically isolate the existing SWP facilities from NCCF. The shallow foundation beneath this structure must be improved to prevent strength loss and seismic settlement. The ground improvement should be to elevation -50.0 feet within the footprint of the structure and beyond the structures by a distance of approximately 25 feet.

The nominal capacity of this channel is 10,300 cfs. The connection to the existing approach canal is at an angle of approximately 45 degrees. While such an arrangement is not ideal from a hydraulic connection standpoint, the site is constrained by the existing Southern Pacific Railroad (SPRR) and Skinner Fish Facility. The low velocity (4 fps at maximum flow and minimum headwater) is expected to limit the negative impacts to hydraulic performance. This connection requires further investigation in the next stage of engineering.

The new approach channel also connects NCCF to the existing approach channel of the Jones PP. A control structure with two sets of radial gates installed at the downstream end of the branch hydraulically isolates the existing CVP facilities from NCCF. The shallow foundation beneath this structure must be improved to prevent strength loss and seismic settlement. The ground improvement should be to elevation -50.0 feet within the footprint of the structure and beyond the structure by a distance of approximately 25 feet. This branch of the new channel has a capacity of 4,600 cfs, matching the capacity of the Jones PP.

Emergency Spillway. An emergency spillway located on the east side of NCCF carries emergency overflow (9,000 cfs, the maximum inflow) to the Old River, so a containment area is not necessary. The shallow foundation beneath this structure must be improved to prevent strength loss and seismic settlement. The ground improvement should be to elevation -50.0 feet within the footprint of the structure and beyond the structure by a distance of approximately 25 feet. Considering the Old River flood elevation and the operating levels within NCCF, a labyrinth spillway set at +17.0 feet and a length of 240 feet is indicated.

Isolation. Two radial gate control structures are within the new approach channel to hydraulically isolate existing SWP and CVP facilities from NCCF, and additional control structures are installed within the existing approaches to isolate NCCF from the Banks approach canal upstream of the Skinner Fish Facility and to isolate NCCF from Old River upstream of the approach channel to the Jones PP. 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 gate structures at existing approaches enables the system to export water using one of the three proposed operation scenarios described in Section 4: Isolated North Delta Operation, Isolated South Delta Operation, and Dual Operation.

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. The new culvert siphons located on the new approach channels will also have trash racks. Thus, a higher debris load is not expected due to the NCCF connections downstream of the fish protection facilities.

Embankment Layout. NCCF is developed by constructing an embankment within the existing CCF embankment and a Divider Embankment through the middle of the existing CCF. New embankments are built of engineered fill.

Embankment Design. The embankment crest elevation for NCCF, Divider Embankment, and the new approach channel is +24.5 feet, which includes considerations for flood levels and SLR. This elevation is based on the recommended design embankment flood protection level (DHCCP Team, 2009). This protection level gradually lowers to an approximately 10 percent slope to where the new approach channel meets the embankment elevation of the existing approaches. The toe of the new embankment is set at 25 feet from the toe of the parallel existing embankment or levee. Excavation at the toe of the existing embankment and levees might require the use of tied-back sheet piles, dewatering, and other geotechnical precautions to prevent failures of existing embankments and levees. Additional stability analysis will be conducted for preliminary design.

The embankment cross-section consists of engineered fill placed on suitable foundation material at a 4H:1V slope on both the upstream and downstream sides of the embankment. The embankment crest is 32 feet wide, consisting of a 24-foot-wide, two-way maintenance access road with 4-foot shoulders on each side. In addition, maintenance roads at the new approach channel join the roads at the existing approach canal to the Banks PP.

The upstream side of the new embankment includes 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 and final design.

14.1.2.3 North Clifton Court Embankment Stability

Foundation Conditions. Subsurface conditions for NCCF were estimated based on recent borings and CPT soundings around the perimeter of the existing CCF. Soil borings and CPT soundings completed in the year 2001 for the Italian Slough Intake project were also reviewed for evaluating subsurface conditions.

The generalized soil profile within the upper 15 feet consists of soft to medium stiff clay with organics, clay with increasing thickness with depth down to approximately 80 feet, and dense sand and stiff clay to the maximum depths explored. Sand layers were also encountered in many of the borings and CPT soundings; however, these materials were conservatively ignored for the stability analyses, except where considering liquefaction. Medium dense sand or silt subject to liquefaction during the design earthquake event was considered present from 10 to 20 feet below the existing ground surface.

Method of Analysis. Loose to medium dense sand or silt subject to liquefaction during the design earthquake event was considered present from 5 to 15 feet and 40 to 55 feet below the existing ground surface. The preliminary stability of NCCF embankments was evaluated in similar manner as described for IF above. For pseudo-static analyses, a design PGA of 0.57 g (corresponding to a pseudo-static coefficient K_H of 0.23g) was used for NCCF (see Section 3.0, “Overview of Conveyance Option”).

Material Properties Used in the Stability Analyses. The total stress shear strength parameters for the foundation clay soil were based on laboratory test results and undrained shear strength interpretations from the CPT soundings for the Italian Slough Intake geology report. 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 15 feet and 40 to 55 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 used.

Conditions Evaluated. The NCCF 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 NCCF 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.

14.1.3 South Clifton Court Forebay

14.1.3.1 Requirements and Assumptions

Hydraulic Connections. SCCF is designed to be hydraulically dependent on Delta waterways and retain the same operation criteria as the existing CCF. Flow is diverted off of West Canal through the modified existing intake control structure off of Old River. The outlet from SCCF is the existing outlet connecting the existing CCF to the existing Banks PP approach channel.

Fish Protection. The Skinner Fish Facility will continue to operate according to existing operation criteria.

Byron-Bethany Irrigation District. The BBID continues to divert water from the channel immediately upstream of the Banks PP. BBID's water needs and diversion structure are not affected by the proposed alignment and forebay.

Head Loss Allowance. Approximately 1 foot of head loss is anticipated in the approach canal between SCCF and the Banks PP.

Forebay Storage Need. SCCF is necessary to enable the existing Banks PP to maximize its operation when electrical power rates are lowest and divert water from the south Delta when required to meet existing flow and water quality standards. To maintain current operations during and after construction of NCCF, the southern portion of CCF will be expanded to incorporate the adjacent property to the south. This will provide the additional storage necessary to maintain current operations. This combined area makes up the SCCF.

Active Storage Availability. The area of the proposed SCCF is constrained by the Divider Embankment to the north, the existing CCF embankment to the east, the existing Jones PP approach canal to the south, and SPRR to the southwest. Acreage available for water storage is approximately 1,691 acres (Elevation +1.1 ft). As described in Section 4.0, "Conveyance System Operations," constraints on the existing pumping plants limit the normal operating range to 7.0 feet (elevation +1.1 to +8.1 feet). This operating range results in approximately 12,050 AF of potential active storage in SCCF. Additional operating storage can be obtained if the operating range is increased, which appears feasible. Otherwise, the Banks PP will need to pump during a short portion of the on-peak pumping period.

Existing Embankments. The existing CCF embankments, and the existing levees in the vicinity of the CCF, do not meet DHCCP flood protection criteria elevation +24.5 feet[1]. Based on limited geologic exploration and geotechnical analysis in this area, the existing embankments likely cannot be raised to meet the flood protection elevations.

Seepage. Some areas of the SCCF foundation are subject to significant underseepage and piping based on soil borings and CPTs silty sand and clean fine sand layers. A slurry cutoff trench beneath the embankment protects the foundation of the embankment. Based on the available subsurface data, the cutoff trench is extended down to an elevation of -50.0 feet. A drain at the toe of the outer embankment slope limits saturated conditions at the ground surface. Based on the limited geological data in the surrounding areas, embankment foundation seepage analysis is considered preliminary.

Excavated Material. Limited soils information and subsurface data show that soils in the vicinity of SCCF consist predominantly of silty and sandy clays and are generally overlain by 2 to 10 feet of tidal peat and mud. This depth is simplified to an assumed 6 feet of peat. From the SCCF cut, it is estimated that approximately 30 percent of the materials below the peat layer might be reusable for embankment construction or as engineered fill.

Dredging. The portion of SCCF that lies within the extents of the existing CCF is dredged to an elevation of approximately -10.0 feet. Limited soils information and subsurface data show that soils within the vicinity of SCCF consist predominantly of silty and fine sand.

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[1] DWR flood protection criteria are the 200-year flood event plus projected SLR. Full flood protection might be obtained in the future by isolating the export facilities from the existing conveyance channels (from CCF and Old River) and by installing protection at key locations on Byron Highway and SPRR.

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

Security. Site security for the SCCF 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. Sites and buildings require credentialed entry through access control gates and secure doors, respectively.

14.1.3.2 General Description

Location. SCCF encompasses the southern portion of the existing CCF and the property adjacent to and south of CCF. The existing ground surface elevation at the adjacent property ranges from -5 to +5 feet except to the southwest, where the ground elevation is approximately +25 feet.

Bottom Elevation. The design bottom elevation for SCCF is -10.0 feet. The bottom elevation of the existing CCF was set at -6.9 feet (DWR, 1974). While specific information is limited, significant sedimentation has occurred since original construction. On occasion, DWR has dredged to maintain a channel. The constrained flow area is reported to result in approximately 1 foot of head difference from the inlet to the outlet of CCF at high flows. More significantly, CCF experiences problems with pond weed, which builds up on the Skinner Fish Facility and hinders flows to the Banks PP. Its growth is related to water detention time, velocity, and depth in the forebay. Harvesting machinery operates on a continuous basis during the year to remove pond weed. SCCF has to be maintained with regular harvesting, dredging, and cleaning. For additional forebay maintenance considerations, see Section 14.3.

Inlet Structure. The existing CCF inlet structure is modified to meet the new embankment elevation and consists of a reinforced concrete structure with several multi-gated bays. The shallow foundation beneath this structure must be improved to prevent strength loss and seismic settlement. Ground improvement to elevation -50.0 feet should be conducted within the footprint of the structure and beyond the structure by a distance of approximately 25 feet.

Connections to Pumping Plants. SCCF continues to use the existing approach canal to Banks PP.

Trash Racks. The fish protection facilities at the Skinner Fish Facility on the approach to the Banks PP intercept most of the floating debris brought in through the SCCF conveyance system. At Banks Pumping Plant, trash racks (approximately 4-inch bar spacing) protect the pumps.

Embankment Layout. SCCF is developed by constructing an embankment within the existing CCF embankment, a Divider Embankment through the middle of the existing CCF, and an embankment along the perimeter of the adjacent property. The new Divider Embankment inside the existing CCF separates NCCF and SCCF.

Embankment Design. The embankment crest elevation for SCCF and approach canals is +24.5 feet, which includes considerations for flood levels and SLR. This elevation is based on the DHCCP recommended design embankment flood protection level at Byron Tract 2 (DHCCP Team, 2009). This protection level gradually lowers at an approximately 10 percent slope to where the forebay approach canal meets the embankment elevation of the existing approaches to either the Banks PP or Jones PP. The toe of the new embankment will be set at 25 feet from the toe of the parallel existing embankment or levee. Excavation at the toe of the existing embankment and levees requires the use of tied-back sheet piles, dewatering, and other geotechnical precautions to prevent failures of existing embankments and levees. Additional stability analysis will be conducted for preliminary design.

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 crest is 32 feet wide, consisting of a 24-foot-wide, two-way maintenance access road with 4-foot shoulders on each side. In addition, maintenance roads are provided at the new approach canal, joining the roads at the existing approach canal to the Banks PP.

The upstream of the new embankment includes 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 and final design.

14.1.3.3 South Clifton Court Embankment Stability

Foundation Conditions. Subsurface conditions for SCCF were estimated based on recent borings and CPT soundings around the perimeter of CCF and Byron Tract 2.

The generalized soil profile consists of soft to medium stiff clay within the upper 15 feet, with organics, clay with increasing thickness with depth down to approximately 80 feet, and dense sand and stiff clay to the maximum depths explored. Medium dense sand or silt subject to liquefaction during the design earthquake event was considered to be present from 10 to 20 feet below the existing ground surface.

The preliminary stability of SCCF embankments was evaluated as described for IF and NCCF above. For pseudo-static analyses, a design PGA of 0.57 g was used for SCCF (see Section 3.0, "Overview of Conveyance Option").

Material Properties Used in the Stability Analyses. The total stress shear strength parameters for the foundation clay soil were based on undrained shear strength interpretations from the CPT soundings. The undrained shear strength of the clay generally increases with depth at a ratio of 0.45 or greater compared to the effective overburden pressure. The scattered sand soil layers were conservatively ignored in the preliminary stability analyses, except where the effect of liquefaction was considered between depths of 10 to 20 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 SCCF embankment slopes will be 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 SCCF embankment slopes of 4H:1V are acceptable in regards to slope stability under all the conditions analyzed. Long-term slope stability is determined to have a factor of safety of at least 1.5, per requirements of Reclamation's Design Standard 13.

14.2 Construction Methodology

The NCCF and SCCF are 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, during construction of the siphon underneath the existing CCF outlet to Banks pumping plant, during construction of an upgraded South Delta Intake Structure feeding into the new SCCF, and when the new approach channel is connected to the existing approaches.

All forebay 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 typical marine construction plus special considerations related to levee connections and flood protection (refer to Section 15.0, "Levees").

The list below provides some of the major construction elements:

- Driving sheet piles to depths required to achieve hydraulic cutoff. Considering subgrade of the embankments and associated conveyance structures is on the order of 5 to 17 feet below sea level, pile installation is challenging. Drilled piers, pipe piles or displacement piles are employed as the primary foundation system under structures as required to resist uplift.

- Underwater construction, such as tremie slab placement and sheet pile trimming.
- Cofferdam construction, shoring, and bracing.
- Site access and dewatering.

Particular construction elements common or unique to the forebay facilities include:

- Staging and storage area and construction zone prep (5 to 10 acres per each structure; 20 to 40 acres total, including tunneling shafts or inlet/outlet structures).
- Ground improvement (assumed to consist of jet grouting in this CER) beneath the structures as required.
- Sheet pile cofferdams, shoring, bracing, and hydraulic cutoff.
- General earthwork (e.g., excavation, spoil, backfill, levee construction).
- Dewatering wells, construction water treatment, return to watercourse.
- Installation of drilled piers, including drilling into place and structural fill.
- Foundation preparation and structural slab construction inside cofferdam.
- Cast-in-place (CIP) reinforced concrete construction (formwork, reinforcing steel assembly, embed installation, concrete pumping and placement, floating and finishing, stripping, and curing).
- Metalwork fabrication, machining, assembly, and installation (roller gates, embeds, bulkheads, , guiderails, catwalks, guardrail/handrail, ladders, hatches, etc.).
- Erosion control (underwater placement of riprap/geotextile).
- Miscellaneous civil site work (e.g., fencing, gates, access roadways and ramps, hydroseeding, landscaping, 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.).

14.2.1 General Excavation and Embankment for IF Adjacent Shaft Construction Areas

The ground surface elevation at the IF site averages elevation 0 feet. The IF interior ground is fairly uniform and is not graded to a specific elevation. The outer perimeter of the IF embankment, with a top elevation of 32.2 feet, is constructed around the 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 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 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 (see Section 22.0, “Spoils Disposal Sites”).

14.2.2 General Excavation for the NCCF and SCCF

NCCF is the northern portion of the existing CCF, and the embankment foundation at this location is excavated to provide the forebay an invert of -5.0 feet. The expected ground elevation is between -5.0 to +2.5 feet, depending on the sedimentation in the area.

SCCF consists of the existing south half of CCF combined with the adjacent property to the south. The existing southern portion of the CCF is excavated to the design elevation of -10.0 feet. The ground surface elevation in Bryon Tract 2 typically ranges from -5 to +5 feet. The adjacent property is excavated to provide an invert of -10.0 feet over the entire basin (including embankment foundation).

The new embankments for NCCF and SCCF are constructed by excavating the embankment foundations down to suitable material, dewatering, and installing the slurry cutoff wall. Approximately 6.5 million cy of earth fill is required for embankment. The required embankment material is borrowed from within the limits of the respective forebays to the extent possible or from borrow sites (see Section 21.0, “Borrow Sites”). Dewatering and moisture conditioning of onsite soils is required.

The total excavated material in the CCF area is approximately 8 million cy, excluding the dredging material within the existing CCF. Dewatering is required for excavation operations. Most of this material is expected to be high in organics and unsuitable for use in embankment construction and requires disposal (see Section 22.0, “Spoils Disposal Sites”).

14.2.2.1 Clifton Court Forebay Phased Construction

Suggested Phasing:

- Phase 1 – SCCF expansion West
- Phase 2 – SCCF expansion East
- Phase 3 – CCF Southern Embankment Removal
- Phase 4 – Dredging
- Phase 5 – Partition CCF Forebay
- Phase 6 – NCCF East Side Embankment
- Phase 7 – NCCF West Side Embankment
- Phase 8 – NCCF North Side Embankment.

Typical Activities in Phases:

- Clear and grub necessary existing vegetation for construction activities.
- Temporary or permanent relocation or installation of power lines as needed.
- Drive sheet piles to enclose construction area using a barge or from land where possible.
- Dewater cofferdam area.
- Dewater and excavate down to foundation depth. Excavation equipment includes scrapers, excavators, bulldozers, off-road and on-road trucks as deemed appropriate.
- Offhaul unsuitable material to spoil areas from excavation areas – Suitable material excavated is placed in lifts as fill for foundation.
- Fill operations using similar equipment as excavation operations, but also include compaction equipment, rollers, motor graders, and water trucks or water pulls to place material in lifts until finish heights are reached.
- Import fill for embankment fill.
- Rip rap slopes using excavators, loaders and trucks as required.

Phase 1 - Drive sheet piles on southwest side of CCF by outflow channel to facilitate new channel and new embankment work. Clear, grub, and perform exploration of SCCF expansion property to find suitable soils for embankment fills and potential spoil areas. After dewatering, excavate with scrapers, excavators and trucks. Backfill using same type of equipment along with compaction equipment and water trucks or water pulls.

After embankment fills are completed in the channel areas, channel sheet piles are installed. Excavation between sheet pile walls begin to foundation depths. Dewater as needed for deeper excavations. Install tiebacks of sheet piles as excavation progresses towards the bottom of the channel. Place concrete bottom after walls and excavation are complete. A tremie slab might be required under the finished concrete bottom slab.

Work areas and sheet piles for siphon construction (see Section 10.0, “Culvert Siphons”) are concurrent with channel construction at both locations – west outlet of CCF and at Byron Highway/SPRR. Control structures (see Section 10.0, “Culvert Siphons”) are constructed concurrently with the channel construction in new locations. Control structures to be built in existing waterways are delayed until the end of the project or built in two phases to avoid impacting current water deliveries.

Phase 2 - Drive sheet piles on southeast side of forebay by inflow gates to facilitate new embankment work. Embankment construction similar to what was described in Phase 1. Construction of new inflow gates occurs concurrently with the embankment construction.

Relocation or raising of power transmission towers within this phase occurs concurrently with the embankment construction.

Phase 3 - Drive sheet piles between two sets of sheet piles on south side of CCF. Excavate down to invert elevation and use suitable fill material in embankment fill for Phases 1 and 2.

Phase 4 – Dredge existing CCF to finish invert elevation. Dredging work is with a cutter head dredge, a dragline type dredge, or other acceptable dredging technique. Silt curtains or other means of limiting turbidity in the existing forebay are used as required. Once material is dried out in spoil area, any suitable material can be stockpiled for use as embankment fill as needed in any of the phases still under construction.

Phase 5 – Drive sheet piles for partitioning forebay. Once Phase 1 & 2 are completed enough to allow water to be introduced into the new forebay section on the south of CCF, water is introduced into the new section until water height of the two locations is even, then the sheet piles from Phase 3 are removed and possibly utilized for Phase 5 sheet piles. After all sheet piles are placed, construction of the partition embankment commences as detailed above. Dewater NCCF area now blocked off by partition sheet piles. Now Phase 6, 7 and 8 can commence concurrently without requiring sheet piles.

Work areas and sheet piles for siphon construction (see Section 10.0, “Culvert Siphons”) are concurrent with partition construction for the north half of the west outlet siphon structure for CCF.

Phase 6 – Drive sheet piles on east side embankment past new spillway location. Once Phase 5 partition is placed completely across the existing CCF and water is allowed to fill SCCF, sheet piling north of partitioning embankment might not be required. Embankment construction is similar to what was described above.

Construct spillway concurrently 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.

Phase 7 – Drive sheet piles on west side embankment as needed. Once Phase 5 partition is placed completely across the existing CCF and water is allowed to fill new SCCF, sheet piling north of partitioning embankment might not be required. Embankment construction is similar to what is described above.

Additional pad material needs to be placed at northwest corner of NCCF for future siphon structure and outlet structures from Main Tunnels (see Section 10.0, “Culvert Siphons,” and Section 11.0, “Tunnels”). This material can be placed alongside the embankment fill in lifts in the same area and/or stockpiled close by for future use as needed for backfill around future structure.

Phase 8 – Drive sheet piles on north side embankment as needed. Once Phase 5 partition is completely across the existing CCF and water is allowed to fill new SCCF, sheet piling north of partitioning embankment might not be required. Embankment construction is similar to what is described above.

14.2.3 General Excavation for the Existing South Embankment of Clifton Court Forebay

The embankment elevation for the existing CCF typically ranges from +17 to +18 feet. The south embankment of CCF will be excavated to provide an invert of -10.0 feet over the embankment foundation footprint, requiring the removal of over 1.2 million cy of material. It is anticipated that embankment excavation requires a steel sheet pile cofferdam to enclose the planned area of the existing embankment. The steel sheet piles are designed to key into an underlying impervious layer where present to facilitate a positive seepage cutoff. Dewatering is required for excavation operations. Most of the excavated material is expected to be sandy, clayey sand, and might be suitable for reuse in embankment construction and requires minimum disposal (see Section 22.0, “Spoils Disposal Sites”).

14.2.4 Completion of New Intermediate Forebay Embankment

After the shafts 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. Approximately 1.9 million cy of excavation and 2 million cy of fill material are required for completing the IF embankments. The required embankment material is from within the limits of the respective forebays when possible or from borrow sites (see Section 21.0, “Borrow Sites”). Moisture conditioning of onsite soils is required.

14.2.5 New Clifton Court Forebay Embankment

The new embankments for the NCCF and SCCF are constructed by installing a sheet pile cofferdam, dewatering, excavating the embankment foundations down to suitable material, and possibly installing a slurry cutoff wall. After the cutoff wall is completed, the embankments are constructed of compacted fill to the desired height. Approximately 9.3 million cy of fill are required for the modified CCF embankments, which includes the divider embankment separating the NCCF from the SCCF, approach canal embankments, spillway pad, and siphon outlet pad.

14.2.6 New Spillway and Stilling Basin

The RCC spillway for IF and NCCF is constructed such that the foundations are excavated to suitable material, similar to the new forebay embankments. Due to the soil foundation at both IF and NCCF, the RCC spillway design incorporates water-stopped contraction joints at frequent intervals to accommodate some settlement and deformation. Ground improvement beneath structures, a possible sheet pile cofferdam, and/or dewatering, might be required.

14.2.7 New Canal to Banks and Jones Pumping Plants

The new canal system connecting NCCF to the Banks PP and Jones PP is constructed using a braced sheet pile system with a concrete tremie seal bottom. This construction method allows the new canal to fit in the limited space between the SCCF and the SPRR and the Skinner Fish Facility.

14.2.8 New Forebay Structures

The new forebay inlets, outlets, modified SCCF inlet, and control structures are constructed using conventional reinforced-concrete methods. The IF structures, NCCF inlet structures, and SCCF inlet structures are constructed within sites surrounded by new earthen embankments. Some water control is required at these locations. The NCCF outlet siphon is constructed in sheet pile cofferdams with tremie seals and dewatering typical of cofferdam construction. The approach sections for the NCCF control structures is constructed similar to the new approach channel, with braced sheet pile walls and a tremie concrete bottom slab. The shallow foundations beneath concrete structures adjacent to the forebays 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.

14.2.9 Banks and Jones Approach 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 similar to the new approach channel, 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.

14.3 Maintenance Considerations

14.3.1 Harvesting

It is expected that the new NCCF and SCCF 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.

14.3.2 Trash Rack Cleaning

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.

14.3.3 Sediment Handling

The majority of easily settled sediments are removed at the sedimentation basins at each intake facility (see Section 6.0, “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 NCCF. Additional opportunity exists at NCCF to settle finer sediments, but given the upstream sediment removal and the larger storage available at the forebay, sediment accumulation at NCCF is expected to be minimal over a 50-year period.

14.3.4 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.

14.3.5 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.

14.3.6 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.

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SECTION 15.0

Levees

This section describes the work associated with Sacramento River levees. Work is done on levees along the Sacramento River and at the South Delta facilities operated by DWR and Reclamation adjacent to the CCF. At the South Delta, many of the levees are not Federal Flood Control Project levees and are not under the jurisdiction of USACE and the CVFPB. All work on these levees will be done with conventional cofferdam construction and conducted in accordance with DWR and Reclamation requirements. The Sacramento River levees are Federal Flood Control Project levees under the jurisdiction of USACE and CVFPB, and specific requirements are applicable to penetrations of these levees.

Three Sacramento River intake systems convey water from the river, through the levee, to landside sedimentation basins via an on-bank intake structure and gravity collector box conduits. 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.

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 DHCCP Intake Facilities”. Additional details of the pad fill arrangement can be found in Sections 6.0, “Intakes and Sedimentation Facilities,” and Section 14, “Forebays.”

15.1 Configuration of Facilities in the Levee

The levee at the intake sites is a traditional trapezoidal embankment 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. 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.

The levee prism is the geometric shape defining the levee section. The levee prism is defined by a levee crest width of at least 30 feet, with 3:1 side slopes extending down the river bottom and land-side ground surface. CVFPB and USACE regulate all facilities and activities within and adjacent to the levee prism. 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 3.0, “Overview of Conveyance Option,” 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 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 6.0, “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 6.0, “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 landside 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.

15.2 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 DHCCP Intake Facilities."

Table 15-1
Sequence of Construction Activities

Activity	Predecessor	Description
1	—	Construct a slurry cutoff wall landside of the existing levee prism for the relocation of Highway 160.
2	1	Construct a well point dewatering system to allow for excavation and placement of new fill slopes for the relocation of Highway 160.
3	1,2	Excavate and remove peat soils to suitable subgrade beneath the area for the widened levee and relocated Highway 160.
4	3	Construct soil improvements beneath widened levee area and the fill slopes required for the permanent relocation of Highway 160.
5	4	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.
6	5	Construct the roadway for the relocated segment of Highway 160 and reroute traffic to new permanent alignment on widened levee section.
7	6	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.
8	6	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.
9	7, 8	Excavate within the intake structure cofferdam and installed diaphragm wall. Install dewatering system within area enclosed by cofferdam.
10	9	Construct drilled casings and reinforced concrete piers beneath the intake structure. Install tremie seal within cofferdam.
11	10	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.
12	11	Construct soil Improvements between the cofferdam diaphragm wall and any unimproved soils up to the slurry cutoff wall installed with Highway 160 relocation.
13	12	Construct remaining length of intake box conduits between the intake structure and the previously placed box conduits constructed with the relocation of Highway 160. Construct fills to finish grade between intake structure and Highway 160.
14	6	Construct well point dewatering system or implement other means to dewater areas for construction of remaining landside facilities, including the sedimentation basins, tunnel intake structure, and sediment drying lagoons.
15	14	Construct soil improvements beneath landside facilities susceptible to settlement.

Activity	Predecessor	Description
16	15	Construct all remaining landside facilities and appurtenant works. Close isolation gates and remove sheet pile cofferdam in front of intake structure.
17	16	Complete work and commission project.

15.3 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 land-side structures and elevated pads , require penetration of the existing levee prism.

15.3.1 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 colletor 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.

15.3.2 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.

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SECTION 16.0

Channel Enlargement Measurements

No channel enlargements are proposed for the MPTO/CCO.

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SECTION 17.0

Operable Barrier

17.1 Physical/Structural Component

The MPTO/CCO 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.

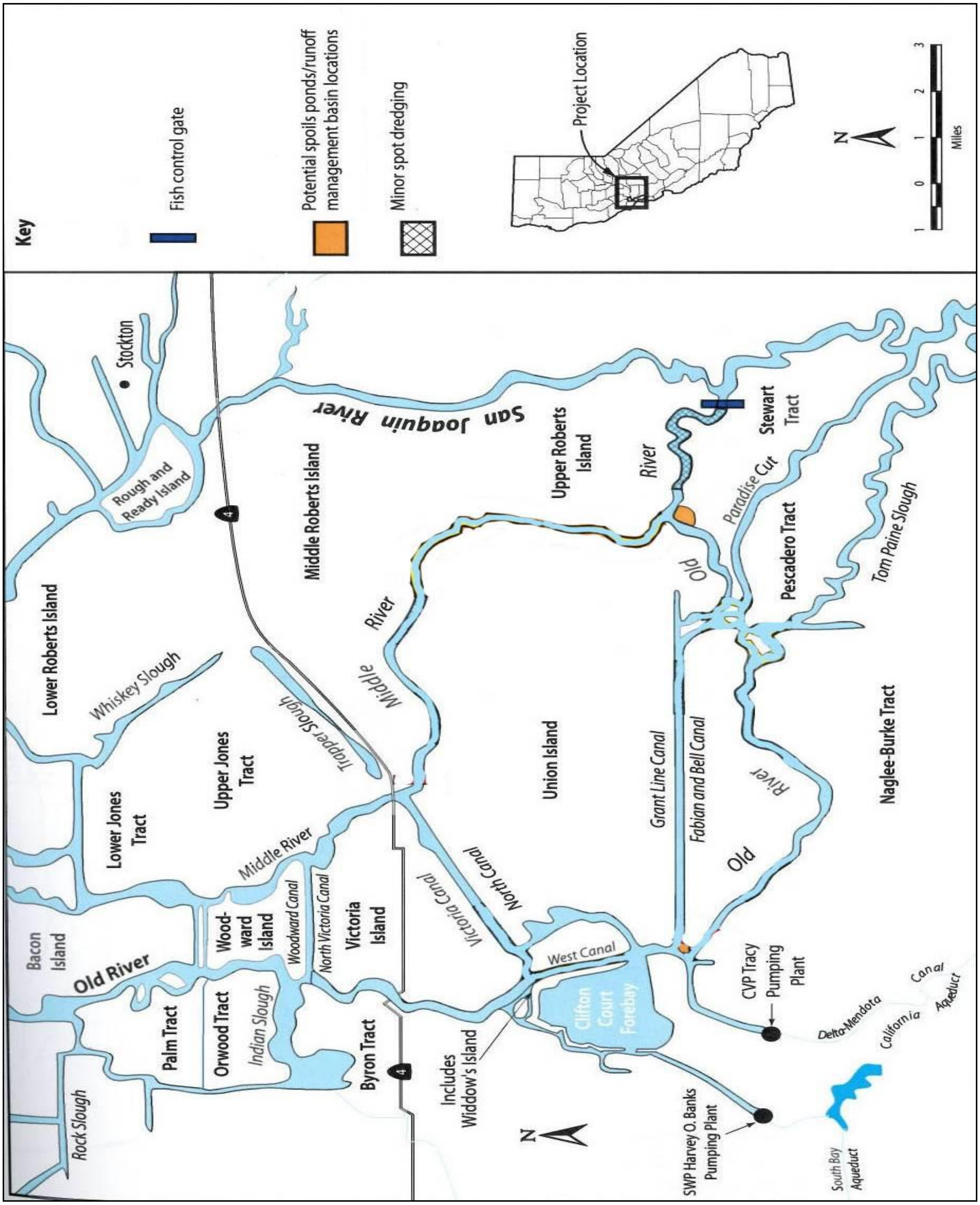

Figure 17-1 Fish Control Gate Location

Source: California Department of Water Resources.

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 DHCCP facilities. The new system will be used with the existing system to offer redundant paths for reliability.

Administrative and operational sites of the MPTO/CCO 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
- CCF Pumping Plant
- NCCF and SCCF inlet and outlet 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 runs from the south end of the new tunnel at the CCF pumping plant to the Delta Field Division O&M Center. The conduit runs adjacent to the expanded CCF, 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 CCF 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 CCF 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 Construction Methodology

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 are used.
- **Main Tunnels from IF to Clifton Court 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.
- **NCCF.** 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.

18.4 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.

18.5 System Control Description/Design Criteria

18.5.1 System Overview

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, two (2) Clifton Court Pumping Plants, 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 Clifton Court Pumping Plant wet well, where it is pumped to Clifton Court Forebay. Under certain hydraulic conditions, water can bypass the pumps and gravity flow to Clifton Court Forebay via a conveyance channel that discharges into NCCF.

18.5.1.1 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.

18.5.1.2 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.

18.5.1.3 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.

18.5.1.4 CLIFTON COURT PUMPING PLANTS AND PUMPS

The Clifton Court 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 Clifton Court 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).

18.5.2 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 Clifton Court Forebay elevation), the water can flow by gravity to Clifton Court Forebay. The ability to operate in gravity flow mode is dependent upon the final CCF 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 CCF 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.

18.5.3 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
- Clifton Court Forebay elevation.

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 Clifton Court Forebay.

18.5.4 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 19-1: Peak Construction Power Requirements

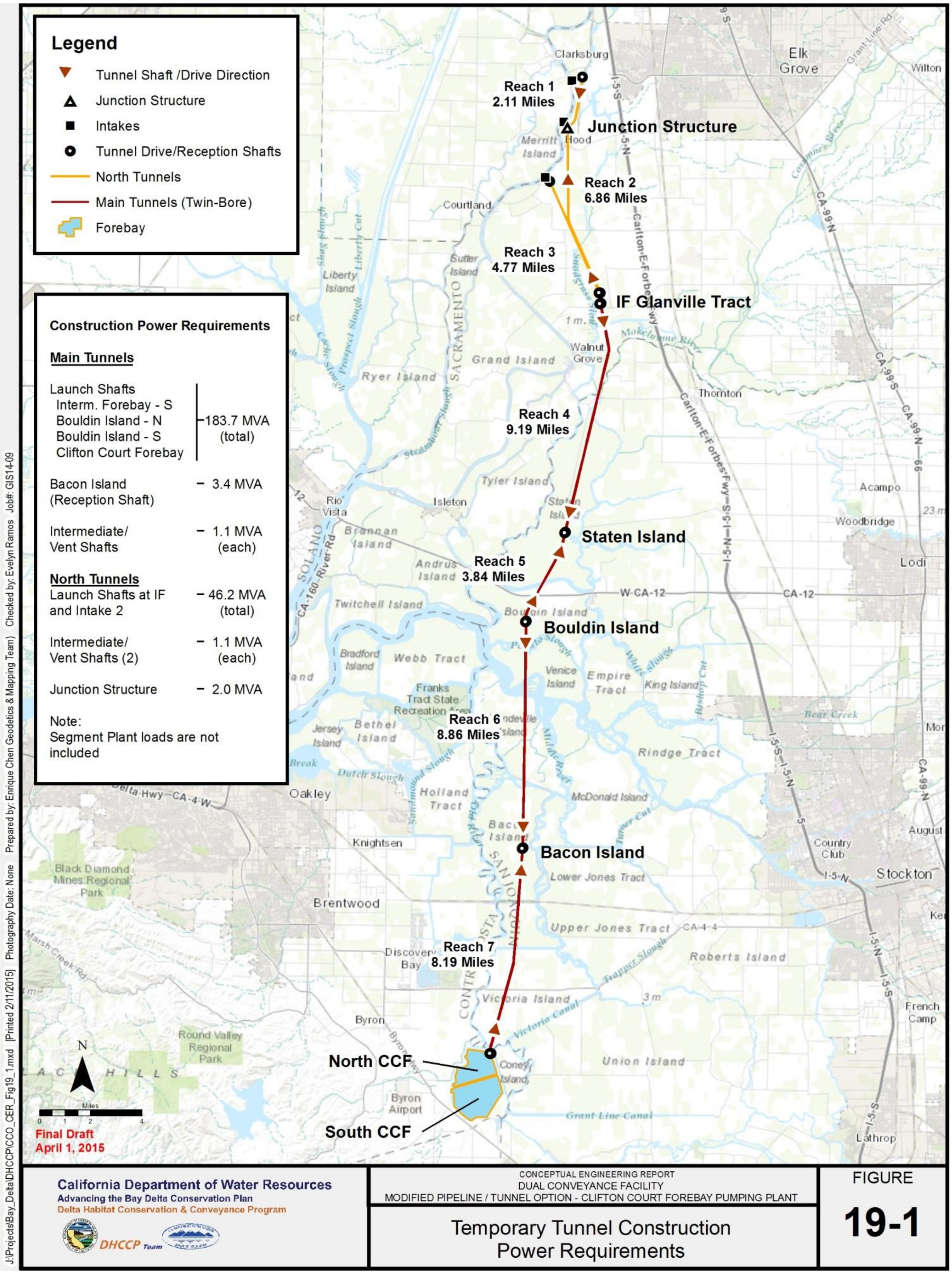
MPTO/CCO Component	MVA
Main Tunnel Drive Shaft Reach 4 (– IF - Staten)	47.7
Main Tunnel Drive Shaft Reach 5 (–Bouldin - Staten)	41.8
Main Tunnel Drive Shaft Reach 6 (Bouldin - Bacon)	47.1
Main Tunnel Drive Shaft Reach 7 (CCF - Bacon)	47.1
Main Intermediate/Vent Shaft E/W (Mandeville)	2.2
Main Intermediate/vent Shaft E/W (Victoria)	2.2
Main Reception Shaft (Bacon)	3.4
North Tunnel Drive Shaft Reach 1 (Intake No.2 – Junction Structure)	10.5
North Tunnel Drive Shaft Reach 2 (IF – Junction Structure)	23.7
North Tunnel Drive Shaft Reach 3 (IF - Intake No. 5)	12.0
North Tunnels (29’ and 20’)Intermediate/Vent Shafts	2.2
North Tunnel Junction Structure	2.0
Total	242.0

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

CCF/IF = Clifton Court Forebay/Intermediate Forebay

E/W = East Tunnel Bore/West Tunnel Bore

MVA = megavolt-amperes



Construction Power Requirements

Main Tunnels

Launch Shafts Interm. Forebay - S Bouldin Island - N Bouldin Island - S Clifton Court Forebay	183.7 MVA (total)
Bacon Island (Reception Shaft)	3.4 MVA
Intermediate/ Vent Shafts	1.1 MVA (each)

North Tunnels

Launch Shafts at IF and Intake 2	46.2 MVA (total)
Intermediate/ Vent Shafts (2)	1.1 MVA (each)
Junction Structure	2.0 MVA

Note:
Segment Plant loads are not included

Figure 19-1: Tunnel Construction Power Requirements

19.2 Power Supply and Transmission Corridor Options

Three electric utility transmission service providers could provide transmission interconnection and services to deliver electrical power to the project: PG&E, 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 TBMs) 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.

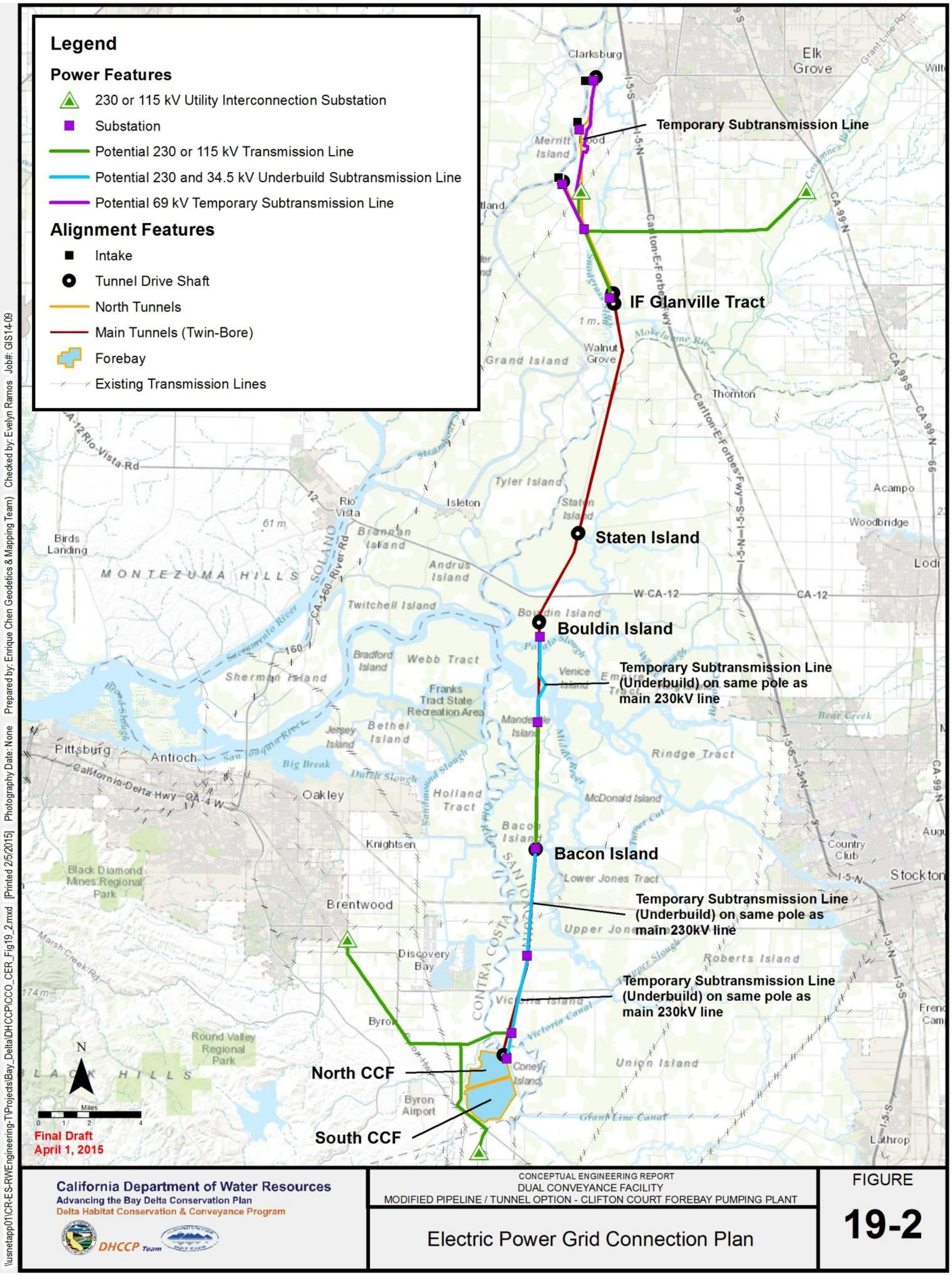


Figure 19-2 Electric Power Grid Connection Plan

19.2.1 Preliminary Interconnection Options

WAPA, PG&E 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 subtransmission 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 Clifton Court area, a new transmission line would be extended from an existing nearby substation to a new substation by the pumping plant area, where electrical power would be transformed from 230 kV to 115 kV for transmission to the tunnel shaft areas and to 13.8 kV or 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 WAPA transmission facilities near the north (IF) and south (CCF) ends of the project provide potential points of interconnection for the project. At the north end, the project could potentially connect to an existing WAPA 230 kV transmission line east of the IF. From this line, a new transmission line (at 230 kV, 115kV or 69kV, depending on the utility studies) would extend to a new substation at the IF to serve both the North Tunnel and Main Tunnel construction loads. At the south end, the project potentially connects 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 a new 230 kV substation 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 NCCF. Lower voltage lines would be used to power intermediate and reception shaft sites between the main drive shafts.

There are also PG&E transmission facilities near the north (IF) end and northwest of CCF that could provide potential points of interconnection for the project. At the north, there is an existing PG&E 115 kV line from which a new line (either 115 kV or 69 kV, depending on utility studies) could be extended to the IF, where a new substation would be constructed to serve temporary construction loads. Northwest of CCF, there is an existing PG&E 230 kV substation from which a new 230 kV line could be extended toward CCF, where a new 230 kV substation would be built to serve the pumping plant. From this new substation, a new line would extend north to support construction at sites north of NCCF.

SMUD transmission facilities are located to the northeast of IF. A new transmission line (at 230 kV, 115 kV or 69kV, depending on utility studies) could be extended from an SMUD-planned 230 kV substation to a new substation near IF. To serve construction loads, a new transmission line would be extended from this new substation north toward the intakes as needed and south to support construction sites along the northern tunnels and at the IF.

19.2.2 Water Codes Statutes

The following water code section and related statutes applies to the facilities described in this section.

- Water § 259
- Water § 11590
- Water § 11592

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 MPTO/CCO pump plants. Provisions of a second incoming electrical service should be studied in preliminary design.

SECTION 20.0

Skinner Fish Facility

CCF, the existing 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 MPTO/CCO does not affect 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 North and South Clifton Court Forebays, 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 MPTO/CCO 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 over land by truck or earth moving equipment and over 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 site.
- Borrow areas should be of sufficient size to accommodate large excavation and material handling equipment.
- Borrow areas not immediately adjacent to construction areas should be in close proximity to transportation facilities capable of handling the anticipated quantity of borrow 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 18 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, 6.5 million cy for the CCF 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 Clifton Court 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 landfarming 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 shown on the Concept Drawings (Volume 2) near IF on the east end of Glanville Tract (for fills at the intakes and IF) and within the expanded SCCF on the south end (for fills at the expanded CCF). 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 MPTO/CCO. The other identified borrow sites are expected to be sufficient.

Spoils disposal sites have been identified (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.

21.2 Construction Methodology

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.

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.

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.

Table 21-1: Summary of Potential Borrow Source Characteristics

Unit Name	Symbol	Age	Description	General Location	Potential Borrow Material	Suitability for Borrow	Rippability	Construction Considerations
Yuba River Gold Fields	YGF	Modern	Well-graded gravel.	East of Yuba City.	Gravel	High	High	Unit consists of washed river rock, which may not be suitable for many applications.
Floodplain Basin Deposits	Qb	Holocene	Fine-grained silt and clay derived from the same sources as modern alluvium. Distal facies of unit Qa. Thickness varies from 1 or 2 meters to 60 meters.	Found throughout the Sacramento and San Joaquin Valleys; prevalent in the Delta. A number of different Quaternary deposits have been grouped with this single unit based upon similar geotechnical characteristics as potential borrow material.	Silt and clay	Variable	High	Most areas underlain by Quaternary basin deposits have extensive surface development, either agricultural or urban. Localized units may have highly variable grain-size distribution. Although satisfactory borrow sites may exist throughout this formation, generally in pre-historic fluvial channels, the reserves at a specific location are typically limited. Depth to groundwater is highly variable. The highly variable nature of this unit over short distances indicates this unit would not be a suitable source for large quantities of borrow material.
Modesto Formation (alluvium)	Qm	Late Pleistocene	Gravely sand, silt, and clay.	Alluvial deposits in the center of the Sacramento and San Joaquin Valleys.	Sand, silt, gravel, and clay	Medium	High	Shallow groundwater is also associated with this unit in some areas. Dewatering of even small borrow areas would likely be required and there is a potential for cross-contamination of near-surface aquifers.
Montezuma Formation (poorly consolidated, clayey sand)	Qmz	Early Pleistocene	Poorly stratified clayey sand and pebbly sand.	Montezuma Hills, southwest of Rio Vista.	Clay and sand	High	High	The Montezuma Hills property is currently owned by an environmental land trust. 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.

TABLE 21-1 (continued)

Summary of Potential Borrow Source Characteristics

Unit Name	Symbol	Age	Description	General Location	Potential Borrow Material	Suitability for Borrow	Rippability	Construction Considerations
Turlock Lake Alluvium	Qtl	Early Pleistocene	Sandstone, siltstone, and conglomerate derived mainly from Sierran granitic and metamorphic rocks; non-marine. Also includes Corcoran Clay.	Eastern edge of the Sacramento and San Joaquin Valleys.	Sand, silt, gravel, and clay	Medium	Medium	Would require 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.
San Pablo Group (marine sediments)	Msp	Late Miocene	Sandstone, mudstone, siltstone, and shale with minor quantities of tuff.	Southwestern border of Sacramento and San Joaquin Delta area.	Sand and silt	Low	Low	A substantial amount of processing may be required to achieve the desired grain size distribution.
Upper Cretaceous Marine Sedimentary Rocks	Ku	Late Cretaceous	Sandstone and shale.	West of CCF.	Sand	Low	Low	A substantial amount of processing may be required to achieve the desired grain-size distribution.
Panoche Formation	Kp	Late Cretaceous	Sandstone, shale, siltstone, conglomerate lenses; marine.	West and Southwest of CCF.	Sand, silt, and gravel	Low	Low	Property underlain by this unit is currently developed into several large-scale wind power farms. A substantial amount of processing may be required to achieve the desired grain-size distribution.
Franciscan Complex (melange)	Kjf	Late Cretaceous to Jurassic	Melange, greenstone, sandstone, shale, conglomerate, metagraywacke, limestone, chert, serpentinized ultramafic rock.	Coastal Ranges west of Interstate 5 and south of Interstate 580.	Sand and gravel	Low	Low	Mineral composition, degree of lithification, and grain-size distribution make this unit unsatisfactory for use as engineered fill, rock slope protection, or crushed rock.

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SECTION 22.0

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 30.7 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 8 million cy) of dredge material are also expected to be removed from NCCF and SCCF. Smaller quantities of excess excavated materials are expected at other facility sites, including about 1.9 million cy at IF and approximately 1.6 million cy at each intake site.

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. 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).

Excess excavated material at NCCF and SCCF area will be disposed in a disposal/borrow area on Byron Tract, northwest of NCCF and between Italian Slough and Byron Highway. Off-road earth moving and hauling equipment moves the material from NCCF and SCCF area to the disposal area.

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 feet above preconstruction grade for all spoil sites and 10 feet above preconstruction grade for NCCF 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, Haul Routes, and Other Construction-Related Elements

This section describes a variety of temporary facilities 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 Haul Routes[1]

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, and 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

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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).

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.

23.3 Barge Traffic and Landing Facilities[2]

Barges may be required for delivery of equipment and materials, hauling fill material, and possibly for muck hauling. 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. Barge routes and landing sites will be selected by the construction contractor and will be expected to comply with the following criteria:

- 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. Improvements will be required and could include:

- 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.

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[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 2012 (CDM Smith, 2012).

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.

23.4 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.

23.5 Temporary and Permanent Footprint

Construction of the MPTO/CCO components will result in temporary construction and permanent facility footprints. Table 23-1 summarizes the projected footprint acreage for each MPTO/CCO 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 MPTO/CCO 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 MPTO/CCO 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 MPTO/CCO Facility Components*

Facility Component	(Temporary) During Construction Acreage[a]	(Permanent) As-constructed Acreage[b]
Intake Facilities		
Intake No. 2	37 acres	167 acres
Intake No. 3	60 acres	98 acres
Intake No. 5	55 acres	101 acres
Intake Subtotal	**152 acres**	**366 acres**
North Tunnels[e]	**323 acres**	**237 acres**
Intermediate Forebay		
IF (Including Overflow Containment Area)	8 acres	243 acres
Main Substation	31 acres	0 acres
IF Reusable Tunnel Material Disposal Area	0 acres	405 acres
Intermediate Forebay Subtotal	**39 acres**	**648 acres**
Main Tunnels[c,e]	**276 acres**	**2,749 acres**
Clifton Court Forebay[d]	**2,145 acres**	**2,121 acres**
Overall MPTO/CCO Project	**2,935 acres**	**6,121 acres**

[a] Includes borrow material, and control structures work area

[b] Includes re-usable tunnel material

[c] Refer to Chapter 11 for a detailed breakdown of Main Tunnel sites

[d] Includes Clifton Court Forebay Pumping Plant

[e] Permanent acreages include tunnel subsurface impacts

Notes:

CCF = Clifton Court Forebay

IF = Intermediate Forebay

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SECTION 24.0

Construction and Constructability Considerations

24.1 Overview

This section presents an overview of preliminary tasks to facilitate construction; discusses factors affecting constructability; discusses selection of various construction methodologies; describes the conceptual construction schedule; and identifies other concerns or issues that could affect construction. Anticipated construction methodologies are provided in the previous sections for each project facility or feature.

24.2 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, this 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.

Appendix G represents the 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. The recommendations set forth in Appendix G are to be considered preliminary and are based on several key criteria, such as the delivery schedule, basic tunnel configurations and alignment, and other assumptions.

24.3 Preliminary Construction Tasks

24.3.1 Permitting and Plan Preparation

DWR will ensure that all permitting requirements are fulfilled prior to commencement of any construction activity. A specific state requirement is the Storm Water Pollution Prevetion Plan (SWPPP). Another major permitting effort is for in-river construction requiring USACE and other permits or approvals. ROWs and easements must also be in place. Cal/OSHA requires the DWR 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). (See Section 26.0, Permits.) Permits are required for each fuel storage depot.

To minimize site impacts and comply with permit requirements, DWR is required to develop safety plans, site utilization plans, traffic control plans, hazardous material management and containment plans, and various work plans. In these plans, DWR indicates 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, DWR sets up staging areas that include offices, shops, safety zones, utilities, material handling, etc. Considering the magnitude of need and the schedule demands, DWR have their own concrete batch plants at several work sites. Only after required plans are developed and approved, DWR will begin mobilizing.

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 and grubbing, constructing site work pads, and defining and building construction access roads. Before site work commences, the contractor implements erosion and sediment controls in accordance with SWPPP. Although DWR plans 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 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 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 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 are properly contained as required for environmental and regulatory compliance.

24.4 Constructability

24.4.1 Definition

Constructability is defined by various industry construction associations as the following:

- Constructability is the extent to which the design of the work facilitates ease of construction, subject to the overall requirements for the completed project. (Construction Industry Research and Information Association)
- Constructability is a system for optimal integration of construction knowledge and experience in planning, engineering, procurement, and field operations while balancing the various project and environmental constraints to achieve overall building objectives.
- Constructability is a system for achieving optimum integration of construction knowledge in the building process and balancing the various project and environmental constraints to achieve maximization of project goals and building performance. (Construction Industry Institute in Australia)

These definitions emphasize that for a structure or project to be “constructable,” it has to equally consider the effects of construction across the existing spatial, social, and environmental conditions. The next section addresses the conditions to be considered when selecting a preferred construction methodology.

24.4.2 Factors Affecting Constructability

The alignment passes through residential and agricultural areas and near the vicinity of utilities, railroads, aqueducts, culturally sensitive lands, and navigable and recreational waterways in the Delta. Accordingly, selection of construction methods considers:

- 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.

Although factors can be generalized across the project, each feature is assessed according to governing conditions or constraints local to the area and specific to the type of structure. In the following section, the discussion will focus on the constructability of each conveyance project feature.

24.4.3 Modified Pipeline/Tunnel Clifton Court Option Constructability

This section follows the order of appearance of the project facilities from the previous sections in this report, and describes in general the constructability concepts for these facilities.

Typical Construction Methodology: Methodology applies to most conventional construction activities and facilities.

- 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.
- 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 on the Concept Drawings (Volume 2).
- The fill pad 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.
- Foundation piles are driven to required depths beneath structures.
- The ground beneath most of the structures is improved (refer to the Concept Drawings in Volume 2 for limits of ground improvement).
- Structures are installed by conventional construction techniques.
- Temporary power for construction is required at all work sites.

Typical Constructability Considerations:
Consideration applies to most conventional construction activities and facilities.

- Presence of existing utilities can conflict with sheet piling, bracing, and excavations.
- Presence of existing buildings and other structures must be considered.
- Intersection of existing maintenance road must be maintained.
- Construction below the water table requires dewatering and treating water before disposal.
- Construction can have environmental impacts that need mitigation.
- Work adjacent to the railroad must be considered.
- Presence of soft/weak sub-grade soils requires improvement.
- Suitable ground improvement methodologies are to be determined during preliminary engineering and final design.

24.4.3.1 Intake Facilities

Construction Methodology:

- On-bank construction on the river side of the levee with a cofferdam around the work area.
- Ground beneath the structure is improved, foundation piers are drilled to required depths or displacement piles are used, and a tremie concrete slab is placed before dewatering.
- Box conduits are constructed from the intake structure to the sedimentation basins by a cut-and-cover approach to connect the intakes and sedimentation basins, requiring the existing levee to be excavated.
- The intake structure is constructed of cast-in-place concrete after portions of the box conduits are constructed at the intake base.

Constructability Considerations:

- Levee stability must be maintained during construction.
- Levee instability due to water seepage and soil erosion along the box conduits after construction must be monitored and prevented.
- Adequate environmental protection for fish and other species during the construction phase must be maintained.
- An endangered species exclusionary period (to be verified during environmental review process) is expected restrict any work in the river outside of the cofferdam between about March 1 and August 1 each year. Specific restrictions will be stipulated in the final environmental documentation for the project.
- Flood control 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.
- 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.
- Construction in the levees and on the river side of the levees requires approval and permitting from USACE and others.
- Potentially small construction windows due to permit requirements could cause scheduling concerns.

Refer to Section 15.0, Levees for additional information regarding constructability for the intakes and the work in the levee.

24.4.3.2 Sedimentation Basins

Construction Methodology:

- 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.
- 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 on the Concept Drawings (Volume 2).
- 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 for the Concept Drawings (Volume 2).
- The fill pad 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.
- A slurry wall is around the site with a perimeter berm to prevent flooding and minimize seepage through the levee.

Constructability Considerations:

- The soil or geologic conditions at the location dictates the depth of the excavation as well as the degree of ground improvement required and the type of foundation to use.
- Levee stability during construction must be maintained.
- Levee stability, due to water seepage and soil erosion after construction, must be monitored.
- Large quantities of imported fill material are required.
- Diaphragm wall methods may be used for the sedimentation basin and perimeter walls. Other methods with suitable long life (100 or more years) can also be employed to construct the walls of these structures.

24.4.3.3 Pipelines and Box Conduits

Construction Methodology:

- Excavation is conducted in the dry using excavators, scrapers, trucks, and conveyors; or in the wet using excavators or draglines.

Constructability Considerations:

- See typical constructability considerations for conventional construction activities.

24.4.3.4 Canals (Approach Canals to Jones and Banks Pumping Plants)

Construction Methodology:

- Vertical externally braced sheet pile wall canal and a tremie concrete base is assumed due to severe space limitations.
- Excavation can be in the dry using excavators, scrapers, trucks, and conveyors, or in the wet using excavators or draglines.

Constructability Considerations:

- See typical constructability considerations above.

24.4.3.5 Culvert Siphons

Construction Methodology:

- The approach is an open cut-and-cover construction methods with conventional CIP concrete structures.
- Constructed as large multiple-box culvert structures using cofferdams.
- Other methods include construction of a bypass channel and redirect the slough away from the work area. In these cases, construction can be continuous for each slough. Water siphons are constructed half at a time, highway/railroad siphon can be constructed all at once.
- For larger sloughs or where other restrictions exist, the culvert siphons would have to be constructed in two phases, each phase lasting one year unless flood control or fisheries issues force a shortened work window.
- For longer culvert siphons, it could be necessary to construct the culverts in three phases over three years.

Constructability Considerations:

- Because the construction is conceived to be over water, measures are required to maintain adequate fish protection during the construction phase, including turbidity controls.
- Over water construction depends on the water level in the river. Construction occurs only during the low level season. Four-month window in the low water season (August 1 to November 30) for driving steel sheeting to construct a cofferdam, or performing any work activities in the water (e.g., excavation using a dragline). The slough needs to remain navigable during construction.
- Over water construction necessitates approval and permitting from USACE.

24.4.3.6 Tunnels

Construction Methodology:

- The approach will be tunneling using a pressurized-face TBM (see Section 11.0, Tunnels).
- Shafts for tunnel access will most probably be constructed using diaphragm wall methods and excavated in the wet with placement of a tremie seal to prevent bottom instability.

Constructability Considerations:

- 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 will probably be classified as “potentially gassy,” requiring specific safety measures and equipment during construction.
- Unforeseen geological conditions could 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 should be 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.

24.4.3.7 Utilities and Infrastructure Crossings

Construction Methodology:

- The various construction approaches include trenching, pipe rerouting, power line relocation, and others.

Constructability Considerations:

- 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.

24.4.3.8 Forebay

Construction Methodology:

- The approach for the IF, the southern portion of SCCF, and all of NCCF might be done in the dry using a large, open-cut excavation using excavators and haul trucks or scrapers. Due to high water table, dewatering may be needed during the entire construction period.
- The approach to the NCCF and SCCF embankment work includes sheet pile cofferdam, dewatering, and open cut excavation using excavators, haul trucks, graders, scrapers and barge-mounted sheet pile driver equipment.
- 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.
- Removal of the existing CCF embankment is done using sheet pile cofferdam, dewatering, and open cut excavation using excavators, haul trucks, graders, scrapers, loaders and barge-mounted sheet pile driver equipment.
- 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 NCCF or structures are built half at a time to allow water to continue to flow down existing channels.
- Dredging is done using a hydraulic dredging system that includes a cutterhead, pump, barge-mounted and HDPE pipelines to move slurry material into the settling basin or spoil site. Dragline is also possible.

Constructability Considerations:

- The Jones PP and Banks PP must be maintained fully operational during construction, except for a few short duration (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.
- Availability of suitable engineered fill materials needed for embankment construction must be addressed.
- Tying into the existing aqueduct system requires completion of construction with minimum interruption.
- Large areas might be required for permanent storage of spoils.
- The North and South Clifton Court forebays include spillways due to DSOD requirements (see Section 14.0, Forebays).
- Access between the Skinner Facility and Banks PP must be maintained during construction of the rectangular channel between the Banks Approach Channel and NCCF. Phasing of forebay construction shown in detail (see Section 14.0 Forebays).

24.4.3.9 Controls and Communications

Construction Methodology:

- Duct banks can be constructed parallel to surface facilities, and radio towers can be used for communication.

Constructability Considerations:

- Installed utilities and communication lines must be protected during construction.
- Integrating the system is dependent on finalizing components at the appropriate time.

24.4.3.10 Power Supply and Grid Connections

Construction Methodology:

- The general approach is to install high-voltage lines using wooden or steel poles embedded into the ground.
- Generators and substation are placed on concrete slabs (permanent or temporary) or steel frame structures (temporary only) with suitable foundations and containment areas.

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 DHCCP design standards.

24.4.3.11 Spoils Disposal Sites and Borrow Areas

Construction Methodology:

- The approach is conventional earth moving, using bulldozers, graders, conveyors, and haul trucks. Spoils are handled in 12-inch lifts with nominal compaction.
- Dewatering of spoils with control and treating of water are required. Topsoil is stripped and re-spread over these areas, as applicable.

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 dictates the depth of the fill, as well as offsets to mitigate the effects of settlement.
- Soil and geologic conditions dictates 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.
- Control and treatment of water from spoil piles will be maintained.

24.5 Other Aspects Related to Construction

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.

Addressing constructability issues and proposing construction methods must also allow for contractor flexibility and ingenuity. As design progresses, construction methods and constructability issues will be further refined and clarified for the selected alternative.

24.6 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 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.

24.6.1 Assumed Design and Construction Packages

For the purpose of developing a construction schedule, 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.

24.6.1.1 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.

24.6.1.2 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:

- **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.
- **Expanded CCF site.** CCF construction has been divided into eight phases. Phasing: Phase 1 – SCCF West embank improvement; Phase 2 – SCCF East embankment improvement; Phase 3 – CCF Southern Embankment Removal; Phase 4 – Dredging; Phase 5 – Partition CCF Forebay; Phase 6 – CCF East Side Embankment improvement; Phase 7 – CCF West Side Embankment improvement; Phase 8 – CCF North Side Embankment improvement. Once Phase 8 embankments are complete up to flood heights, the siphon structure from the main tunnel to the NCCF inlet location can be started from the CCF side.
- **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 drive shafts of another tunnel contract as their reception shafts. In one case, 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.

All of the shared work site, coordination, and associated construction sequencing required for the work must 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.

24.6.1.3 Primary Construction Packages

- Intakes, and Sedimentation Facilities, at Site No. 2.
- Intakes, Sedimentation Facilities, and Junction Structure at Site No. 3.

- Intakes, and Sedimentation Facilities, at Site No. 5.
- North Tunnels (Reaches 1,) enabling contract for preliminary site and access development.
- North Tunnel (Reaches1, 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 northeast Clifton Court.
- North Clifton Court Pump Plant.
- Intermediate Forebay Embankment.
- Intermediate Forebay Structures.
- North and South Clifton Court Forebays Embankment and Structures.
- Communications.
- Power Supply (from power provider up to and including Pumping Plant Substations).
- Power Supply (from power provider up to and including Pumping Plant Substations).

24.6.2 Modified Pipeline/Tunnel Clifton Court Option Implementation Schedule Summary

A summary of key schedule information follows.

- The conceptual schedule duration is just under 15 years from the beginning of preliminary design to the completion of start and commissioning activities.
- 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 evaluatation 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.
  - 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 MPTO/CCO 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 MPTO/CCO facilities described in this CER.

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SECTION 26.0

Permits

Implementation of the MPTO/CCO as described in this CER has to 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 MPTO/CCO 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/DHCCP 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 MPTO/CCO as described in this CER will be determined during the final design phase, based on the requirements listed in the BDCP Permitting Handbook.

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SECTION 27.0

Architectural Considerations

The architectural considerations are divided into three sections: existing conditions, programming, and design guidelines. First, the existing conditions describe how the Clifton Court 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 tunnel.

27.1 Existing Conditions

27.1.1 Proposed Tunnel Route

The tunnel will begin at the intake facilities along the Sacramento River. The tunnel will flow south toward the Clifton Court Forebay, with the terminus located south of the King's Island residential development.

27.1.2 Site

27.1.2.1 Location

The Byron Highway allows views across the Clifton Court Forebay demonstrating the spaciousness of the elongated island, which is owned by the Department of Water Resources (DWR). The proposed pump plant location is at the northeast corner of the island. The front edge of the island consists of the levee, which encompasses the entire Clifton Court forebay to protect from flooding and currently hides the interior of the island. Groves of trees are the only other visible reminder of the island over the levee crest. The island currently blends with the surrounding landscapes, but unlike the rest of that region, the site is currently void of continuous human activity like agriculture.

The West Canal runs along the eastern edge, while the Italian Slough flows to the north. The levee crest road follows the Italian Slough from Clifton Court Road east to the residential community. Only the northern portion of the island is made available to public traffic, while the rest of the island is blocked to public vehicles.

Figure 27-1: Photo of island lowlands taken facing east from the gravel levee crest road.

Figure 27-2: Photo taken on a gravel road overlooking the Old River.

27.1.2.2 Plants and Screening

The island site still has native vegetation covering the land. The northern half of the island is a dense cover, which provides a natural screen from the development on the neighboring island. Clusters of tall trees in the north punctuate the flat terrain. The southern tip of the island is more sporadic and less dense in covering and canopy. While low-lying flat lands fill the upper middle of the site, native grasses dominate the lower middle.

Figure 27-3: Natural screening of island from agriculture on opposite site of Old River toward agriculture screened by shrubs.

27.1.2.3 Site Entry

Entry points already exist to the north and south of the site. With a single entry point on either side, the road splits to encircle the land. One gravel road follows atop the levee adjacent the forebay, while the other winds beside the West Canal. Occasionally, a road drops down into the lowland areas for access. The trees are located nearer the roadways.

27.1.2.4 Surrounding Topography and Landscape

The flat island topography fits with the surrounding land, but also varies from uses with the adjacent regions. Agriculture is the dominant land use along the eastern edge of the site. The east also currently has the river entry into the forebay. Agriculture also is dominant along the flat, southern edge. The Byron Highway passes the forebay only 500 feet from the southern edge. Other infrastructure utilizing the forebay is on the southern side. Two miles to the south is Mountain House, a small residential development. The west edge beyond the forebay climbs in elevation to Altamont Pass, which has a visible wind farm. The northern boundary is defined by an estuary that has a small boat-based development (King’s Island) a half mile from the site. Besides King’s Island community, the residential developments around the site are too distant to currently have a view into the island site.

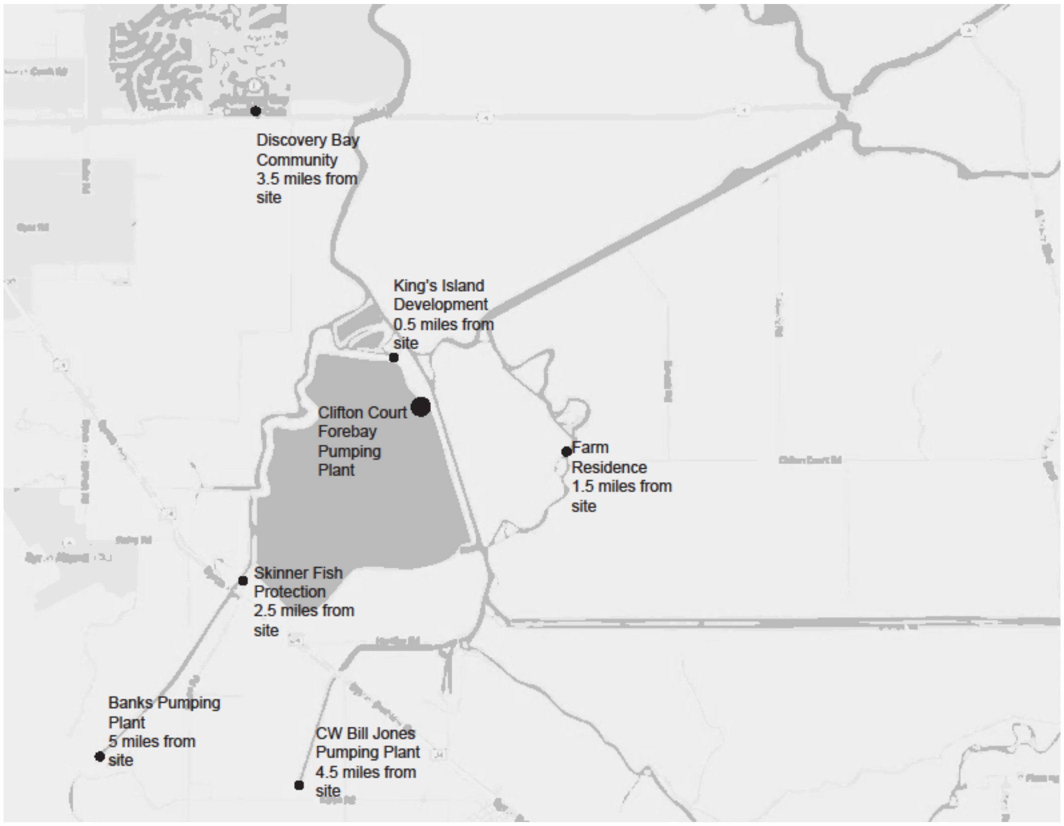


Figure 27-4: Vicinity Map including distances from prominent landmarks

27.1.3 Other Infrastructure Off the Forebay

Other monumental infrastructure projects currently occupy the Clifton Court Forebay. The Skinner Fish Protective Facility contains the fish in the forebay. The facility is largely made of concrete structures, but there are a few metal outbuildings. Harvey O. Banks Pumping Plant (see Figure 27-5), which lifts the water 244 feet to the Bethany Reservoir, is the head of the California Aqueduct system. The complex is made of simple concrete structures and can move water up to 10,000 cubic feet per second. The office building has more detail emphasis, with the concrete walls and roof framing the glazed entry. The C.W. “Bill” Jones Pumping Plant provides water into the Central Valley region. Its pump plant is a concrete structure with rhythmic openings in its façade.

Figure 27-5: Harvey Banks Pumping Plant. Channel from the Clifton Court Forebay heads to the pumping plant and up to the Bethany Reservoir.

27.1.4 Other Monumental Complexes

The California Aqueduct has monumental structures over the entire system to facilitate the transport of the water to southern California. These structures include the aqueduct pumping plants, like Dos Amigos, Teernink, and Chrisman. The Edmonston Pumping Plant (see Figure 27-6) raises the 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.

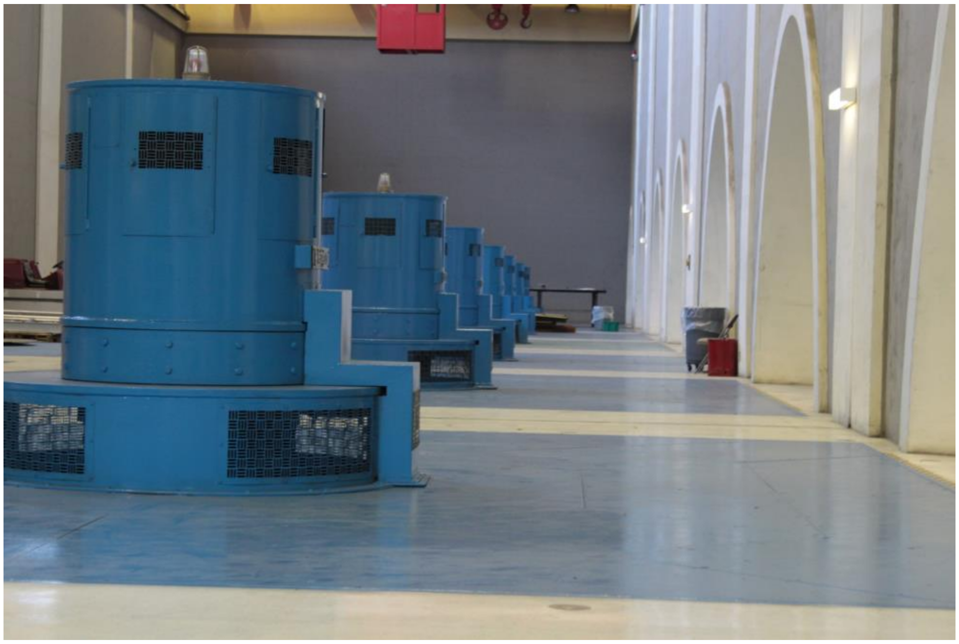


Figure 27-6: Edmonston Pumping Plant. Architectural features, like the arches, align with repetition of the pump heads.

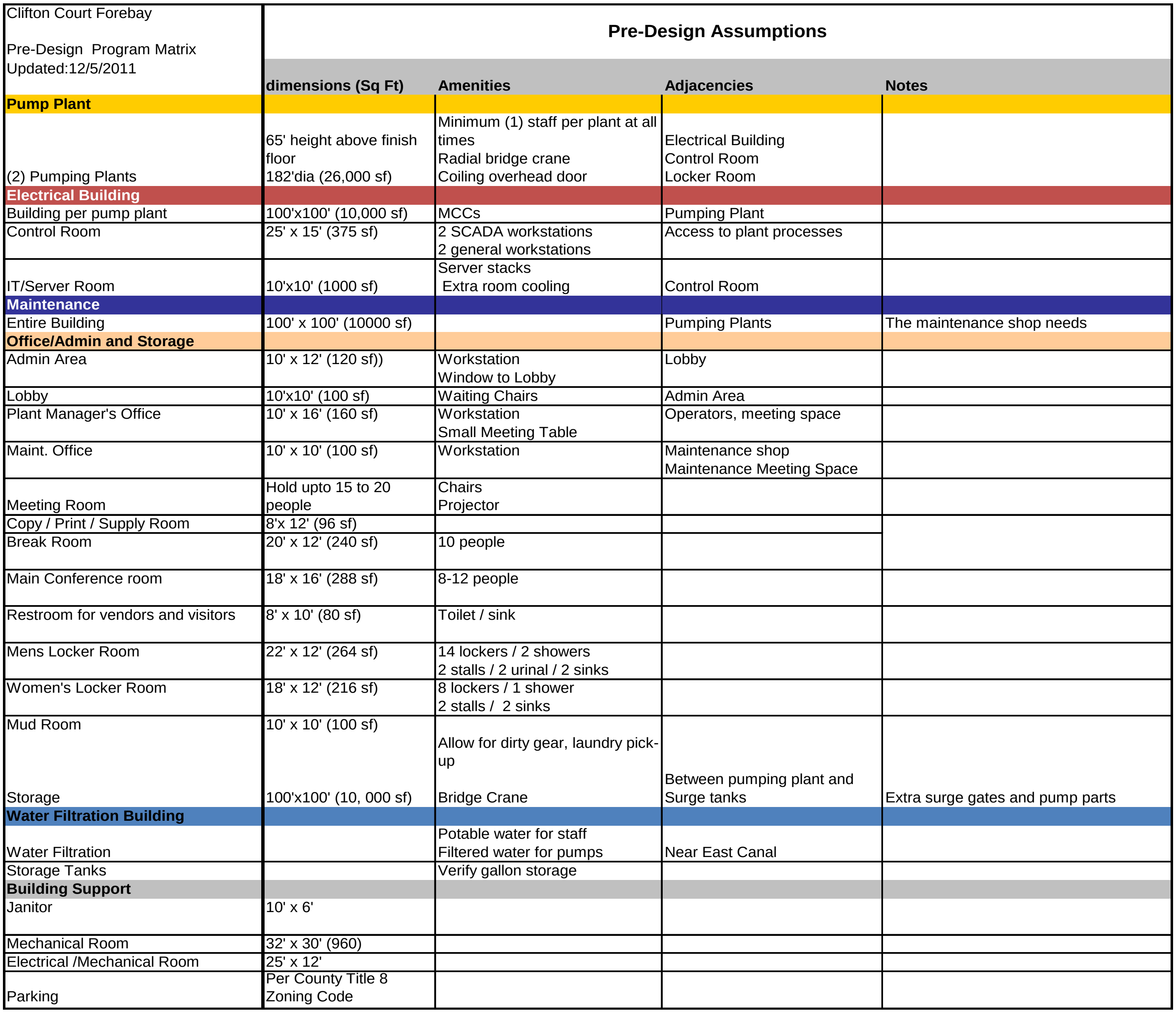
Figure 27-7: Edmonston Pumping Plant. Full height glazing and repetition of order.

27.2 Programming

27.2.1 Site

The elongated island for the pump plant has a few site constraints to reconcile for the plant to become the new intake tunnel terminus. To contain the increased amount of water during a 200 year flood, the levee needs a ten to fifteen foot height increase around the entire forebay perimeter. Raising the island elevation level with the new levee provides flat space requirements for the facility buildings. To allow for additional spacing requirements of the pump plant facility on the site, the island is extended west into the forebay. The northern edge of the island remains near the existing height as the levee road descends to the residential community, King's Island. As the northern portion of the island rises up to meet the site elevation, the height increase adds a buffer for the houses. Groves of trees planted along the incline and at the crest accentuate the visual barrier (see Figure 27-8). A security fence around the facility perimeter protects 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.

Figure 27-8: Site Section. Increase in berm height and plantings buffer view of King's Island residents from tall pump plants.



Clifton Court Forebay Pre-Design Program Matrix Updated:12/5/2011	Pre-Design Assumptions			
	dimensions (Sq Ft)	**Amenities**	**Adjacencies**	**Notes**
Pump Plant				
(2) Pumping Plants	65' height above finish floor 182'dia (26,000 sf)	Minimum (1) staff per plant at all times Radial bridge crane Coiling overhead door	Electrical Building Control Room Locker Room	
Electrical Building				
Building per pump plant	100'x100' (10,000 sf)	MCCs	Pumping Plant	
Control Room	25' x 15' (375 sf)	2 SCADA workstations 2 general workstations	Access to plant processes	
IT/Server Room	10'x10' (1000 sf)	Server stacks Extra room cooling	Control Room	
Maintenance				
Entire Building	100' x 100' (10000 sf)		Pumping Plants	The maintenance shop needs
Office/Admin and Storage				
Admin Area	10' x 12' (120 sf))	Workstation Window to Lobby	Lobby	
Lobby	10'x10' (100 sf)	Waiting Chairs	Admin Area	
Plant Manager's Office	10' x 16' (160 sf)	Workstation Small Meeting Table	Operators, meeting space	
Maint. Office	10' x 10' (100 sf)	Workstation	Maintenance shop Maintenance Meeting Space	
Meeting Room	Hold upto 15 to 20 people	Chairs Projector		
Copy / Print / Supply Room	8'x 12' (96 sf)			
Break Room	20' x 12' (240 sf)	10 people		
Main Conference room	18' x 16' (288 sf)	8-12 people		
Restroom for vendors and visitors	8' x 10' (80 sf)	Toilet / sink		
Mens Locker Room	22' x 12' (264 sf)	14 lockers / 2 showers 2 stalls / 2 urinal / 2 sinks		
Women's Locker Room	18' x 12' (216 sf)	8 lockers / 1 shower 2 stalls / 2 sinks		
Mud Room	10' x 10' (100 sf)	Allow for dirty gear, laundry pick-up		
Storage	100'x100' (10, 000 sf)	Bridge Crane	Between pumping plant and Surge tanks	Extra surge gates and pump parts
Water Filtration Building				
Water Filtration		Potable water for staff Filtered water for pumps	Near East Canal	
Storage Tanks		Verify gallon storage		
Building Support				
Janitor	10' x 6'			
Mechanical Room	32' x 30' (960)			
Electrical /Mechanical Room	25' x 12'			
Parking	Per County Title 8 Zoning Code			

Figure 27-9: 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 Clifton Court 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 contribute to the qualitative dialogue describing aesthetics; however, these recommendations are preliminary and actual materials selections will be finalized after DWR reviews the palette. These architectural design criteria will be refined and further developed 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 Clifton Court Forebay Site

The Clifton Court Forebay Pumping Plant centers on an axis that splits the site so that the pumping plants and the electrical buildings mirror each other. The axis also divides the substation into equal parts. The roadway follows the axis as two roads cross the island from the pump plants to the channels (see Figure 27-10). 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. Groves of a variety of trees act as a buffer along the north and east edge of the site. On the north, the groves provide an added screen of the operations from the residential community; the groves also provide another screen from the farmer’s to the east. 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 nearby residences’ privacy.

27.3.1.3 Alternate Architectural Site Layouts

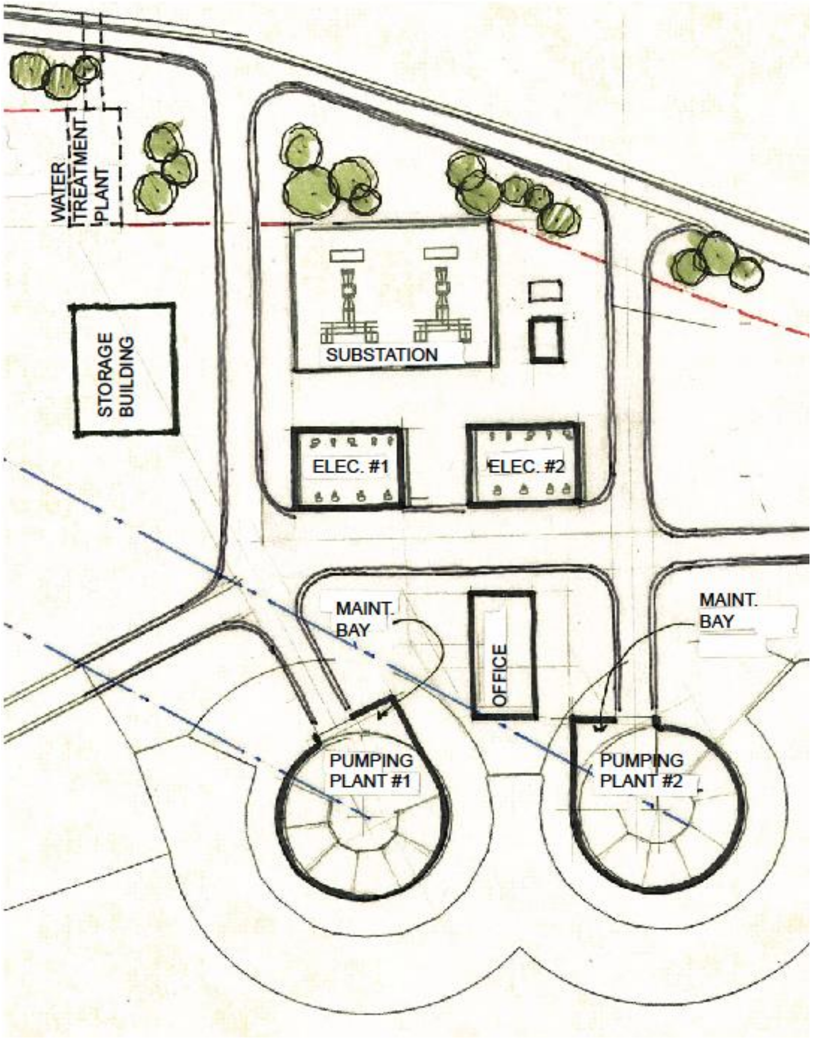


Figure 27-10: Architectural Site Study incorporating the component descriptions from previous sections.

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 (see Figure 27-3).

27.3.2.2 Clifton Court Forebay Landscaping

The Clifton Court Forebay landscaping follows the native, low-maintenance vegetation. When the island is raised up ten to fifteen feet, the entire area needs to be landscaped where structures are not being built to avoid erosion and pollution. In the northern portion of the island, taller, denser groves grow with bushes to buffer the residential community. Along the eastern edge, the site references the opposite side of the channel and has sparser tree plantings, but still enough to block views. 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.

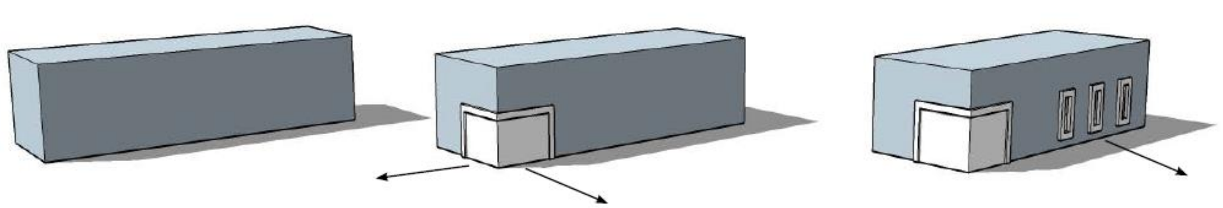


Figure 27-11: Concept Diagrams

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 27-5. The rhythmic repetition of the equipment within references the Edmonston Pumping Station in Figure 27-6.

The Office Building is an example of the Design Guidelines at work with the non-process buildings, see Figure 27-15. 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 27-12, 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 27-15.

Figure 27-12: Rendered perspective of pump plant concepts with rugged landmark and softer human scale.

27.3.3.4 Material Matrix

The material matrix is defined below in Figure 27-13.

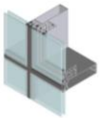
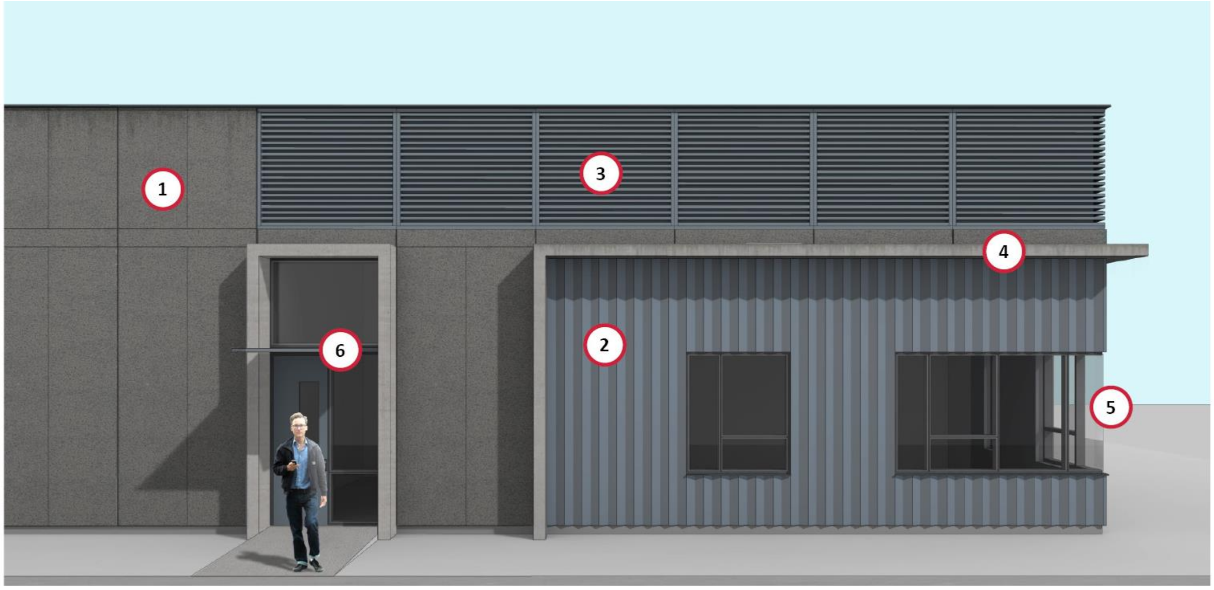
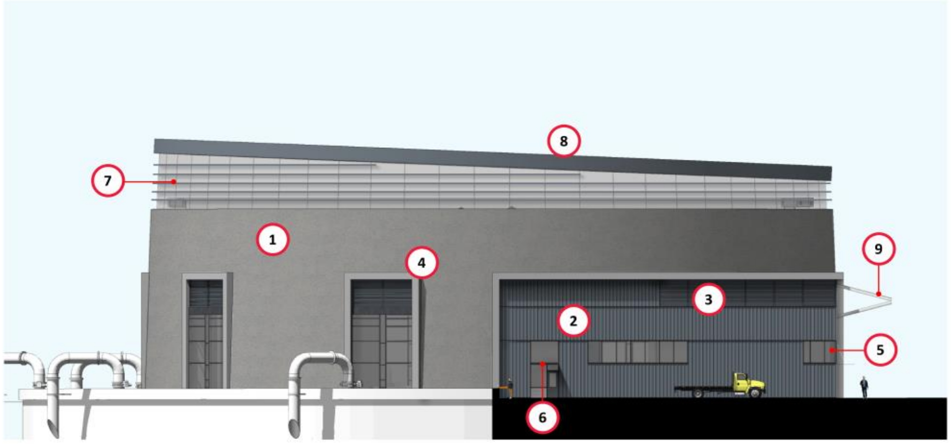
	Materials	Color	Notes
	Cast in Place w/ sand-blasted exposed aggregate	Aggregate Dark Grey	Primary exterior material of Pump Plant
	CarbonCast w/ sand-blasted exposed aggregate	Aggregate Dark Grey	Primary exterior material for rest of plant
	Precast concrete	Natural Grey	Secondary material form used to extrude from the building for human form
	Concealed Fastener Low-Maintenance Metal Panel	Natural Zinc	Infill from extruded precast fins
	SSG Curtainwall	Dark Anodized	Provides resiliency, corosion resistance, and maximum daylight
	Louver	Color to match metal panel	Provides air movement through building and punctuates the rhythmic elements inside
	Translucent Glazing	Frosted	Clerestory above cylinder for light into pumping plant

Figure 27-13: Material Matrix



1. PRECAST INSULATED CONCRETE PANELS
2. CONSEALED FASTENER METAL PANEL SYSTEM
3. MECHANICAL/ARCHITECTURAL LOUVERS
4. PRECAST CONCRETE SURROUND
5. SSG STOREFRONT SYSTEM
6. SIMPLE PLATE STEEL CANOPY AT ENTRIES

Figure 27-14: Graphic Depiction of Design Guidelines for Non-process Buildings



1. EXPRESSED AGGREGATE CAST IN PLACE CONCRETE
2. CONSEALED FASTENER METAL PANEL SYSTEM
3. MECHANICAL/ARCHITECTURAL LOUVERS
4. PRECAST CONCRETE SURROUND
5. SSG STOREFRONT WINDOW SYSTEM
6. SIMPLE PLATE STEEL CANOPY AT ENTRIES
7. TRANSLUCENT CURTAINWALL OR CHANNEL GLASS SYSTEM WITH INTEGRATED SUNSHADING DEVICE
8. STANDING SEAM ROOF SYSTEM
9. BIFOLD ACCESS DOOR

Figure 27-15: Graphic Depiction of Design Guidelines for Process Buildings

Note: Depicted materials are shown in typical colors and may vary

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SECTION 28.0

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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

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 Belknap, 1980). Here, the channel system was determined almost entirely by tidal flows that created an extensive sinuous dendritic channel network. Because of the differential amounts of inorganic sediment supply, the peat of the south-central Delta (San Joaquin River system) grades northwards into peaty mud and mud toward the natural levees and flood basins of the Sacramento River system (Atwater and Belknap, 1980). This is reflected in the thickness of peat across the Delta, which can be up to 30 feet thick in the central Delta, and thinning towards the north and south (URS/JBA, 2007c).

At the margins of the Delta, the freshwater tidal marshes merged with flood basin marshes at slightly higher elevations. Although the wetland species were the same, the underlying soils were different because the flood basins dried out every summer, preventing peat accumulation.

Over the last 150 years, the natural landscape elements of the Delta have been transformed by human activities. The large freshwater tidal marsh of the Delta has been converted by levee building into a highly dissected region of channels and levee-encircled islands used for agriculture (Simenstad et al., 2000). Today, the Delta contains over 55 “dry” islands or tracts that are protected from flooding by more than 1,100 miles of levees. Islands that were originally near sea level are now well below sea level and large areas of many islands are now more than 15 feet below sea level.

1.3 Regional Subsidence

For the last 5,000 years up to the 1850s, relative sea-level rise in the Delta was balanced by vertical marsh growth through biomass accumulation and sediment deposition (Atwater et al. 1979), resulting in the accumulation of great thicknesses of organic rich soils within the delta. Starting in the mid-1800s, many hundreds of miles of levees were constructed, allowing the isolation and draining of vast areas of the delta for agricultural use. The construction of these levees and drainage systems was largely completed by 1930 and the Delta had taken on its current appearance, with most of its 1,100-square-mile area reclaimed for agricultural use (Thompson, 1957). The original levees were usually less than 5 feet high, but had to be raised to keep up with the settlement of levees and the subsidence of the interior island soils. Prior to the agricultural development of the Delta, island surface elevations were at or near sea level. As the Delta islands have subsided, levee heights have become progressively greater. Some levees are now up to 25 feet above the interior island surfaces.

The dominant cause of this land subsidence in the Delta is decomposition of organic carbon in the peat soils (Ingebritsen and Ikehara, 1999). Prior to agricultural development, the soil was waterlogged and anaerobic (devoid of oxygen), so organic carbon accumulated faster than it could decompose. Drainage for agriculture led to aerobic (oxygen-rich) conditions that favor rapid microbial oxidation of the carbon in the peat soil. In some areas, groundwater extraction and gas field pumping can also contribute to local and regional subsidence.

The principal control on the magnitude of subsidence is the composition of the marsh soils. At a landscape scale, the soils of the central Delta, which are generally more organic-rich, exhibited the highest average historical rates of subsidence, between 0.10 and 0.16 feet per year (ft/yr) (Mount and Twiss, 2005). The more inorganic soils of the northern Delta exhibited lower rates of subsidence. On a local scale, the surface profile of individual islands is generally “saucer-shaped”, due to oxidation of the exposed peats in the interiors of the islands. Inorganic soils may be more prevalent at the island perimeter because of depositional processes.

Rates of subsidence on the Delta islands have declined since the 1950s because of improved land-use practices (Deverel and Rojstaczer, 1996; Deverel et al., 1998). Further subsidence is also constrained by the thickness of organic-rich sediments deposited during the mid- to late-Holocene. In the south and east Delta, historical subsidence has reduced or eliminated the organic-rich soils, whereas the thicker organic soils of the central and west Delta continue to subside. Mount and Twiss (2005) found that post-1950 subsidence rates were 20 to 40 percent less than the average rate between 1925 and 1981 (URS/JBA, 2007b).

2.0 SEISMICITY

2.1 Seismotectonic Setting

Active faulting and earthquakes in central California result from transpressional deformation related to movement of the Pacific plate is to the northwest relative to the North American plate. Most of this movement is accommodated along the major strike-slip fault systems of the San Andreas and Hayward-Calaveras fault systems, which lie to the west of the Delta (Figure A-4). Other strike-slip faults nearer the Delta also accommodate the motion between the tectonic plates, and some plate motion is taken up on reverse and thrust faults like those in the Coast Ranges-Sierran Block boundary zone (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 ≤ 3:1 ratio of strike-slip to dip-slip displacement, the implied rate of net oblique slip is less than 0.6 to 1.8 mm/yr. For the Midway fault, DRMS estimated a long-term average rate of dextral offset of about 0.2 mm/yr based on 800 m of late Cenozoic right-separation and an assumed duration of deformation (active during the past ~3.5 Ma). For maximum magnitude, DRMS adopted a floating earthquake model with a weighted range of magnitudes that favors rupture of all or most of the combined length of the Black Butte and Midway faults (URS/JBA, 2007a).

2.3 Historical Seismicity

The Delta has exhibited a low level of historical seismicity, and the seismicity that has occurred is difficult to correlate to any of the Delta seismic sources, which is not an unusual observation for buried faults. No **M** ≥ 4.0 events have occurred in the past 40 years in the Delta and no **M** ≥ 5.0 earthquakes have occurred in the Delta in historical times (Figure A-4). The absence of significant seismicity in the Delta does not necessarily indicate the absence of seismogenic structures. The neighboring CRSB boundary zone (Figure A-5) has been, for the most part, not seismically active and yet the occurrence of large earthquakes (**M** > 6) such as the 1892 Vacaville-Winters and 1983 Coalinga earthquakes are testimony to the seismogenic potential of buried faults (Wong et al., 1988). There have been about 15 earthquakes of approximately moment magnitude (**M**) 6.0 or greater in the San Francisco Bay region in historical times (Figure A-4).

The most significant earthquakes to the Delta are discussed in more detail below.

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 (~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

Survey web page http://earthquake.usgs.gov/eqcenter/eqinthenews/2007/nc40204628/nc40204628.php). Geologists did not find any surface rupture along the fault; surface rupture is unusual for an earthquake of this size and depth.

2.4 Background Seismicity

To account for the hazard from background (floating or random) earthquakes that are not associated with known or mapped faults, regional seismic source zones were used in the DRMS seismology study (URS/JBA, 2007a). In most of the western United States, the maximum magnitude of earthquakes not associated with known faults usually ranges from **M** 6 to 6½. Repeated events larger than these magnitudes generally produce recognizable fault-or-fold related features at the earth's surface (e.g., dePolo, 1994). An example of a background earthquake is the 1986 **M** 5.7 Mt. Lewis earthquake that occurred east of San Jose.

For a probabilistic seismic hazard analysis (PSHA), like that performed for the DRMS study, earthquake recurrence estimates of the background seismicity in each seismic source zone are required. The DRMS site region was divided into two regional seismic source zones: the Coast Ranges and Central Valley (URS/JBA, 2007a). The recurrence parameters for the Coast Ranges source zone were adopted from Youngs, et al. (1992). They calculated values for background earthquakes based on the historical seismicity record after removing earthquakes within 10-km-wide corridors along each of the major faults. The recurrence values for the Central Valley zone were estimated by URS as part of the DRMS study (URS/JBA, 2007a and Figures A-6 and A-7). The maximum earthquake for the source zones is **M** 6.5 ± 0.3.

2.5 Seismic Hazards

The DRMS study (URS/JBA, 2008) evaluated the vulnerability of levees to seismic hazards. Historically, there have been 166 Delta and Suisun Marsh flood-induced levee failures leading to island inundations since 1900. No reports have been found to indicate that seismic shaking has ever induced significant damage. However, the lack of historical damage is not a reliable indicator that Delta levees are not vulnerable to earthquake shaking. Furthermore, the present-day Delta levees, in their current configurations, have not been significantly tested by the moderate to high seismic shaking that can be expected. Unlike flood-induced failures, earthquake-induced levee failures tend to extend for thousands of meters if not kilometers.

The largest earthquakes experienced in recent history in the region include the 1906 Great San Francisco earthquake and the 1989 Loma Prieta earthquake. The 1906 earthquake occurred while the levees were in their early stages of construction, were much smaller than they are today, and were not representative of the current configuration. The epicenter of the 1989 Loma Prieta earthquake was too distant and registered levels of shaking in the Delta too small to cause perceptible damage to the levees. Nonetheless, the DRMS seismic analysis team performed a special simulation analysis of the 1906 Great San Francisco earthquake to evaluate the potential effects of this event on the current levees (URS/JBA, 2008).

In addition to the simulation of these largest regional earthquakes, the DRMS study also evaluated recent smaller and closer earthquakes (URS/JBA, 2008). The earthquakes, and their impacts, that were evaluated include the 1980 Livermore earthquake (**M** 5.8) and the 1984

Morgan Hill earthquake (**M** 6.2). Except for the 1906 earthquake, which would have caused deformations of some of the weakest levees, the other earthquakes were either too small or too distant to cause any significant damage to the Delta levees. These results are consistent with the seismic vulnerability prediction model developed for the DRMS study (URS/JBA, 2008).

For further information on the DRMS levee vulnerability study, which evaluated existing levees, please see the DRMS study (URS/JBA, 2008).

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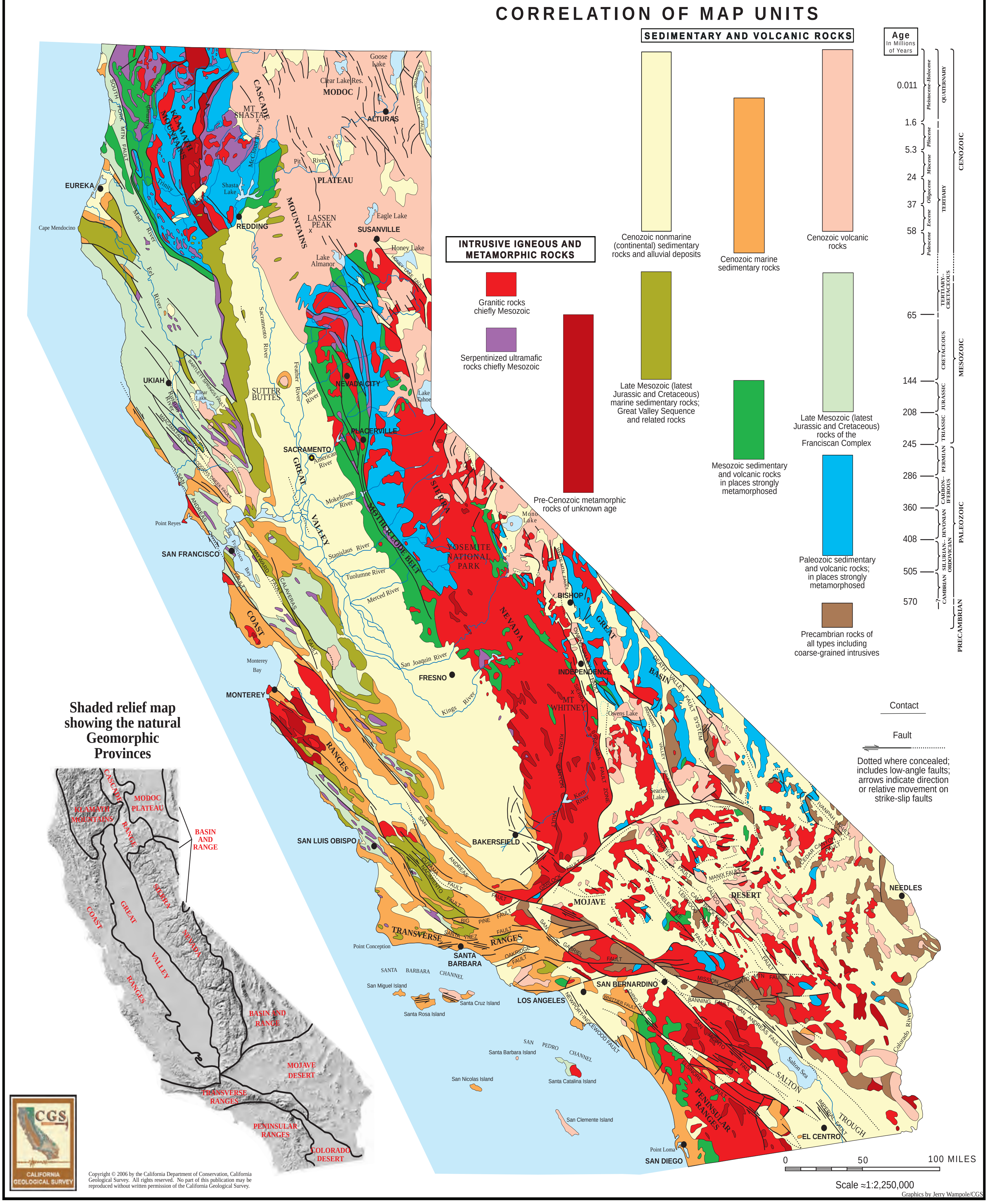
CALIFORNIA GEOLOGICAL SURVEY
JOHN G. PARRISH, Ph.D., STATE GEOLOGIST

STATE OF CALIFORNIA - ARNOLD SCHWARZENEGGER, GOVERNOR
THE RESOURCES AGENCY - MIKE CHRISMAN, SECRETARY
CALIFORNIA DEPARTMENT OF CONSERVATION - BRIDGETT LUTHER, DIRECTOR

MAP SHEET 57
SIMPLIFIED GEOLOGIC MAP OF CALIFORNIA

SIMPLIFIED GEOLOGIC MAP OF CALIFORNIA



Shaded relief map showing the natural Geomorphic Provinces

CALIFORNIA GEOLOGICAL SURVEY

N

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

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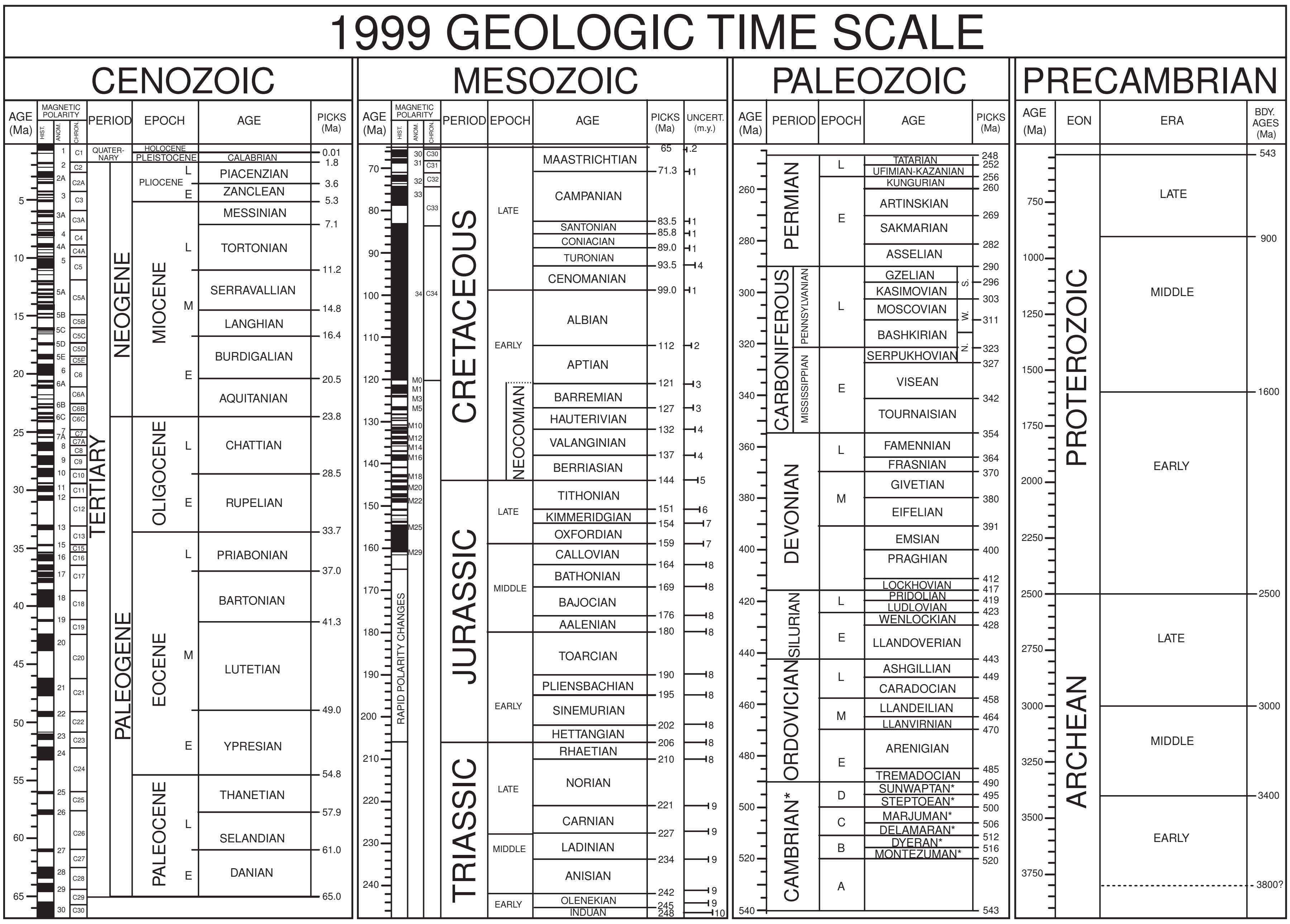
Regional Geologic Setting

Figure
A-1



1999 GEOLOGIC TIME SCALE

CENOZOIC

PERIOD		EPOCH		AGE	PICKS (Ma)
QUATERNARY		HOLOCENE			0.01
		PLEISTOCENE		CALABRIAN	1.8
TERTIARY	NEOGENE	PLIOCENE	L	PIACENZIAN	3.6
			E	ZANCLEAN	5.3
		MIOCENE	L	MESSINIAN	7.1
				TORTONIAN	11.2
			M	SERRAVALLIAN	14.8
				LANGHIAN	16.4
			E	BURDIGALIAN	20.5
				AQUITANIAN	23.8
	PALEOGENE	OLIGOCENE	L	CHATTIAN	28.5
			E	RUPELIAN	33.7
		EOCENE	L	PRIABONIAN	37.0
			M	BARTONIAN	41.3
				LUTETIAN	49.0
			E	YPRESIAN	54.8
		PALEOCENE	L	THANETIAN	57.9
				SELANDIAN	61.0
			E	DANIAN	65.0

MESOZOIC

PERIOD	EPOCH		AGE	PICKS (Ma)	UNCERT. (m.y.)
				65	.2
CRETACEOUS	LATE		MAASTRICHTIAN	71.3	1
			CAMPANIAN	83.5	1
			SANTONIAN	85.8	1
			CONIACIAN	89.0	1
			TURONIAN	93.5	4
			CENOMANIAN	99.0	1
	EARLY		ALBIAN	112	2
			APTIAN	121	3
		NEOCOMIAN	BARREMIAN	127	3
			HAUTERIVIAN	132	4
			VALANGINIAN	137	4
			BERRIASIAN	144	5
JURASSIC	LATE		TITHONIAN	151	6
			KIMMERIDGIAN	154	7
			OXFORDIAN	159	7
	MIDDLE		CALLOVIAN	164	8
			BATHONIAN	169	8
			BAJOCIAN	176	8
			AALENIAN	180	8
	EARLY		TOARCIAN	190	8
			PLIENSBACHIAN	195	8
			SINEMURIAN	202	8
			HETTANGIAN	206	8
TRIASSIC	LATE		RHAETIAN	210	8
			NORIAN	221	9
			CARNIAN	227	9
	MIDDLE		LADINIAN	234	9
			ANISIAN	242	9
	EARLY		OLENEKIAN	245	9
			INDUAN	248	10

PALEOZOIC

PERIOD		EPOCH	AGE		PICKS (Ma)
					248
PERMIAN		L	TATARIAN		252
			UFIMIAN-KAZANIAN		256
		E	KUNGURIAN		260
			ARTINSKIAN		269
			SAKMARIAN		282
			ASSELIAN		290
CARBONIFEROUS	PENNSYLVANIAN	L	GZELIAN	S.	296
			KASIMOVIAN		303
			MOSCOVIAN	W.	311
			BASHKIRIAN		323
	MISSISSIPPIAN	E	SERPUKHOVIAN	N.	327
			VISEAN		342
			TOURNAISIAN		354
DEVONIAN		L	FAMENNIAN		364
			FRASNIAN		370
		M	GIVETIAN		380
			EIFELIAN		391
			EMSIAN		400
			PRAGHIAN		412
			LOCKHOVIAN		417
SILURIAN		L	PRIDOLIAN		419
			LUDLOVIAN		423
		E	WENLOCKIAN		428
			LLANDOVERIAN		443
ORDOVICIAN		L	ASHGILLIAN		449
			CARADOCIAN		458
		M	LLANDEILIAN		464
			LLANVIRNIAN		470
		E	ARENIGIAN		485
			TREMADOCIAN		490
CAMBRIAN*		D	SUNWAPTAN*		495
			STEPTOEAN*		500
		C	MARJUMAN*		506
			DELAMARAN*		512
		B	DYERAN*		516
			MONTEZUMAN*		520
		A			543

PRECAMBRIAN

EON	ERA	BDY. AGES (Ma)
		543
PROTEROZOIC	LATE	900
	MIDDLE	1600
	EARLY	2500
ARCHEAN	LATE	3000
	MIDDLE	3400
	EARLY	3800?

GEOLOGICAL SOCIETY
OF AMERICA

Product code CTS004. Compilers: A. R. Palmer, John Geissman

*International ages have not been established. These are regional (Laurentian) only. Boundary Picks were based on dating techniques and fossil records as of 1999. Paleomagnetic attributions have errors, Please ignore the paleomagnetic scale.

Sources for nomenclature and ages: Primarily from Gradstein, F., and Ogg, J., 1996, *Episodes*, v. 19, nos. 1 & 2; Gradstein, F., et al., 1995, SEPM Special Pub. 54, p. 95–128; Berggren, W. A., et al., 1995, SEPM Special Pub. 54, p. 129–212; Cambrian and basal Ordovician ages adapted from Landing, E., 1998, *Canadian Journal of Earth Sciences*, v. 35, p. 329–338; and Davidek, K., et al., 1998, *Geological Magazine*, v. 135, p. 305–309. Cambrian age names from Palmer, A. R., 1998, *Canadian Journal of Earth Sciences*, v. 35, p. 323–328.

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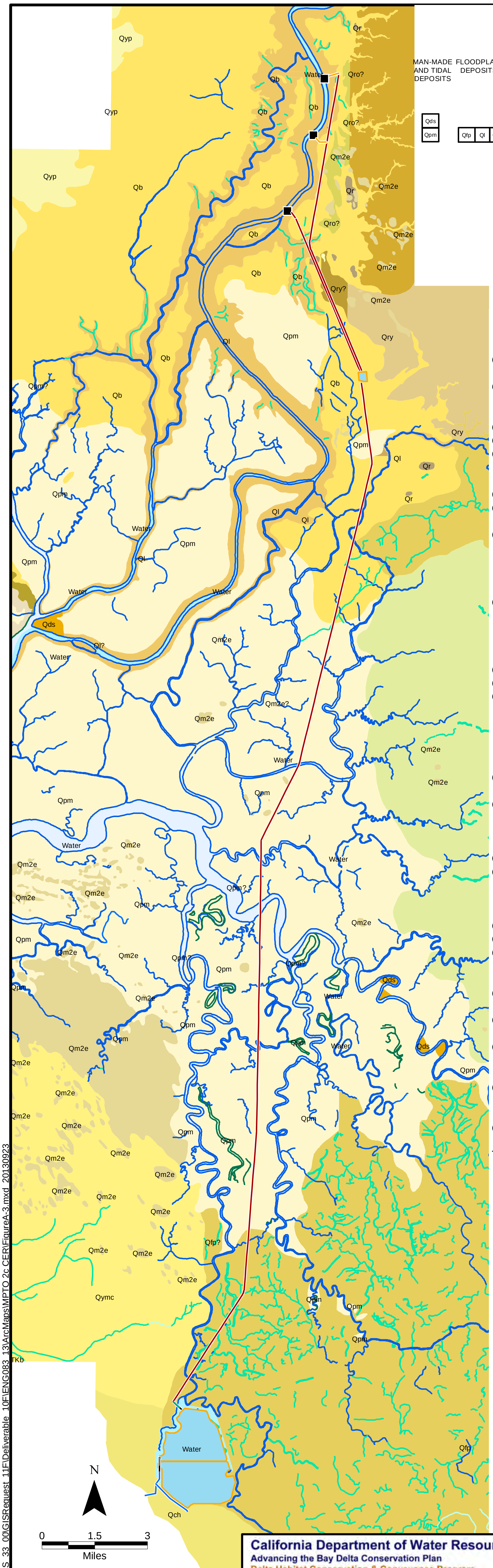
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY MODIFIED PIPELINE/TUNNEL OPTION

Geologic Time Scale

Figure
A-2



CORRELATION OF MAP UNITS

Note: relative ages generally well established
within individual columns but generally speculative between columns

MAN-MADE AND TIDAL DEPOSITS	FLOODPLAIN DEPOSITS	EOLIAN DEPOSITS	ALLUVIAL-FAN DEPOSITS DERIVED FROM GLACIATED DRAINAGE BASINS	ALLUVIAL-FAN DEPOSITS DERIVED FROM UNGLACIATED DRAINAGE BASINS: Putah Creek	Montezuma Hills and vicinity	Antioch and vicinity	Marsh Creek and vicinity	Corral Hollow drainage to Brushy Creek	Calaveras River and vicinity		
Qds Qpm	Qfp Ql Qb			Qyp	Qym	Qya	Qymc	Qch	Qcr	Holocene	QUATERNARY
		Qe: Qm2e Qoe	Qm Qr: Qry Qro	Qop	Qom Qmz	Qia Qoa	Qomc			Pleistocene	
							TKb			TERTIARY AND CRETACEOUS	

Qds HYDRAULIC-DREDGE SOILS (Holocene; post-1900)

Qpm PEAT AND MUD OF TIDAL WETLANDS AND WATERWAYS (Holocene)

ALLUVIUM OF SUPRATIDAL FLOD PLAINS (Holocene) -- Consists of:
Qfp Alluvial-floodplain deposits, undivided
Ql Natural-levee deposits (Holocene)
Qb Flood-basin deposits (Holocene)

EOLIAN DEPOSITS (Pleistocene) -- consists of:
Qe Eolian deposits, undivided (upper Pleistocene)
Qm2e Eolian deposits of upper member of the Modesto Formation (upper Pleistocene)
Qoe Older eolian deposits (upper Pleistocene

ALLUVIAL-FAN DEPOSITS DERIVED FROM GLACIATED DRAINAGE BASINS (Pleistocene) -- consists of:

Qm MODESTO FORMATION (Pleistocene) -- Dense stipple - loose sand, probably eolian; light stipple - loose sand and silt, chiefly fluvial; no stipple - compact silt and very fine sand

RIVERBANK FORMATION (Pleistocene) - As mapped, divided into:
Qr Riverbank Formation, individed (upper Pleistocene)
Qry Younger unit of the Riverbank Formation (upper Pleistocene)
Qro Older unit of the Riverbank Formation (upper Pleistocene)

ALLUVIAL-FAN AND ALLUVIA-TERRACE DEPOSITS DERIVED FROM UNGLACIATED DRAINAGE BASINS (Holocene and Pleistocene) -- consists of:

ALLUVIUM OF PUTAH CREEK (Holocene and Pleistocene) - Divided into:
Qyp Younger alluvium of Putah Creek (Holocene and(or) upper Pleistocene)
Qop Older alluvium of Putah Creek (upper Pleistocene)

ALLUVIUM OF MONTEZUMA HILLS AND VICINITY (Holocene and Pleistocene) -- Divided into:
Qym Younger alluvium of Montezuma Hills and vicinity (Holocene)
Qom Older alluvium of Montezuma Hills and vicinity (Pleistocene)

ALLUVIUM OF ANTIOCH AND VICINITY (Holocene and Pleistocene) -- Divided into:
Qya Youngest alluvium of Antioch and vicinity (Holocene)
Qia Intermediate alluvium of Antioch and vicinity (Upper Pleistocene)
Qoa Oldest alluvium of Antioch and vicinity (upper Pleistocene)

ALLUVIUM OF MARSH CREEK AND VICINITY (Holocene and Pleistocene)
Qymc Younger alluvium of Marsh Creek and vicinity (Holocene and upper Pleistocene)
Qomc Older alluvium of Marsh Creek and vicinity (Pleistocene)

Qch ALLUVIUM OF CREEKS FROM THE CORRAL HOLLOW DRAINAGE TO BRUSHY CREEK (Holocene and(or) upper Pleistocene)

Qcr ALLUVIUM OF CALAVERAS RIVER AND VICINITY (Holocene and(or) upper Pleistocene)

Qmz MONTEZUMA FORMATION (Pleistocene)

TKb BEDROCK (Tertiary and upper Cretaceous)

LEGEND

Holocene channels
- Channel subject to non tidal flow
- Channel subject to non tidal flow - queried
- Tidal channel
- Tidal channel - queried
- Tidal channel - probably abandoned before 1850

Modified Pipeline/Tunnel Alignment
- ■ Intake
- Canal
- Conveyance Pipeline
- Siphon
- Tunnel
- Forebay

DATA SOURCE:
Atwater, Brian F., 1982. Geologic Maps of the Sacramento-San Joaquin Delta California. U.S. Geological Survey Miscellaneous Field Study MF-1401, Denver Colorado, 1982.

Atwater's report was converted to the digital geospatial format by David T. Hansen, G. James West, Patrick Welch, and Barbara Simpson, of the U.S. Bureau of Reclamation, Mid-Pacific Region, MPGIS Service, 2001.

This digital geologic data is based Atwater's hard-copy report. The digital data captures surficial geologic units, 1850 tidal wetland areas, late Holocene tidal and non tidal channels, and shorelines of perennial lakes, circa 1910. The field studies covered period of about 1978 to 1981. Data was compiled on 7.5-minute USGS topographic quadrangle maps for the area by Brian Atwater in 1982.

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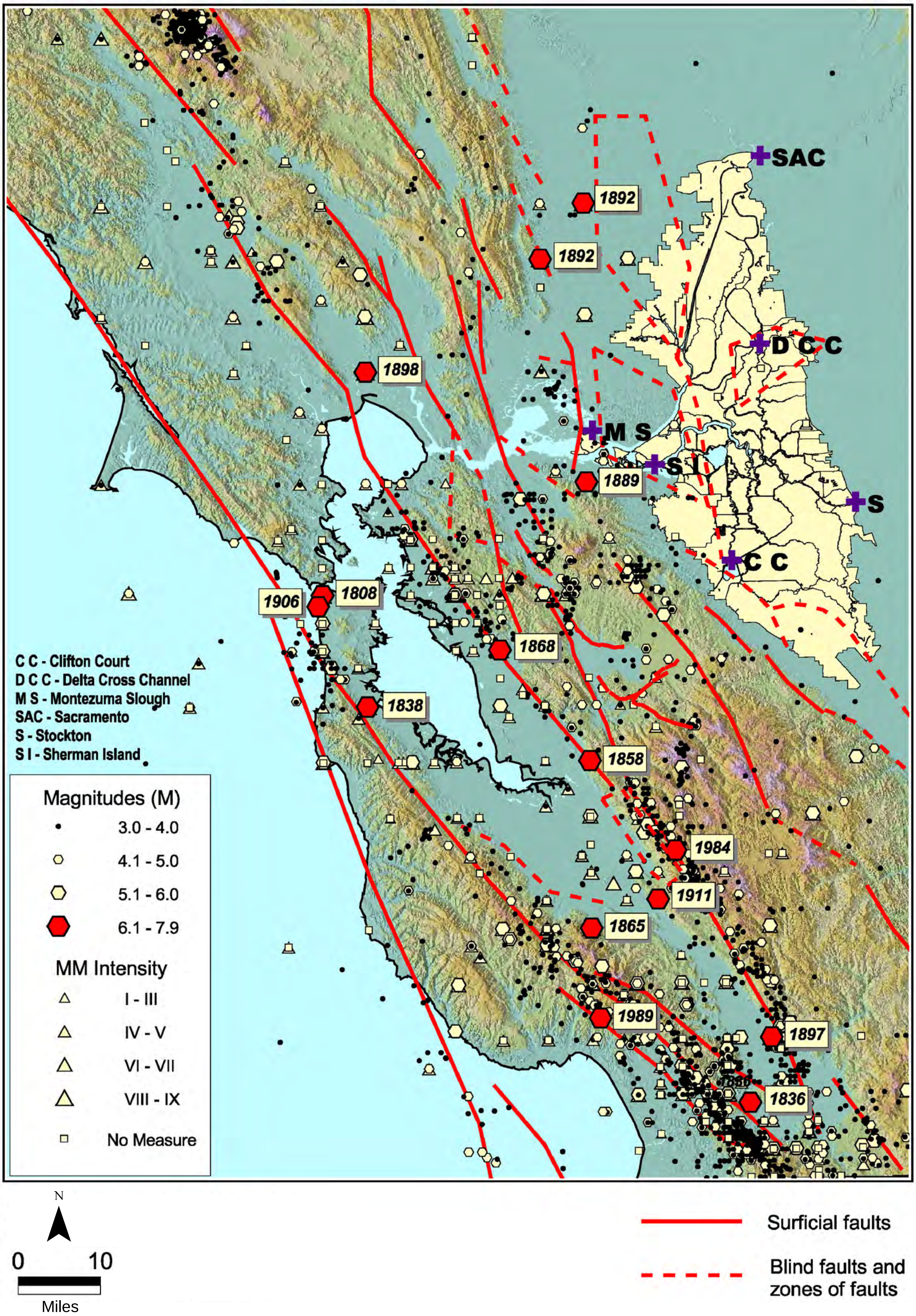
DUAL CONVEYANCE FACILITY MODIFIED PIPELINE/TUNNEL OPTION

Geology of the Sacramento-San Joaquin Delta

Figure
A-3



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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY MODIFIED PIPELINE/TUNNEL OPTION

Active Faults and Historical Seismicity of the San Francisco Bay and Delta Region (M 3.0), 1800-2006

Figure
A-4

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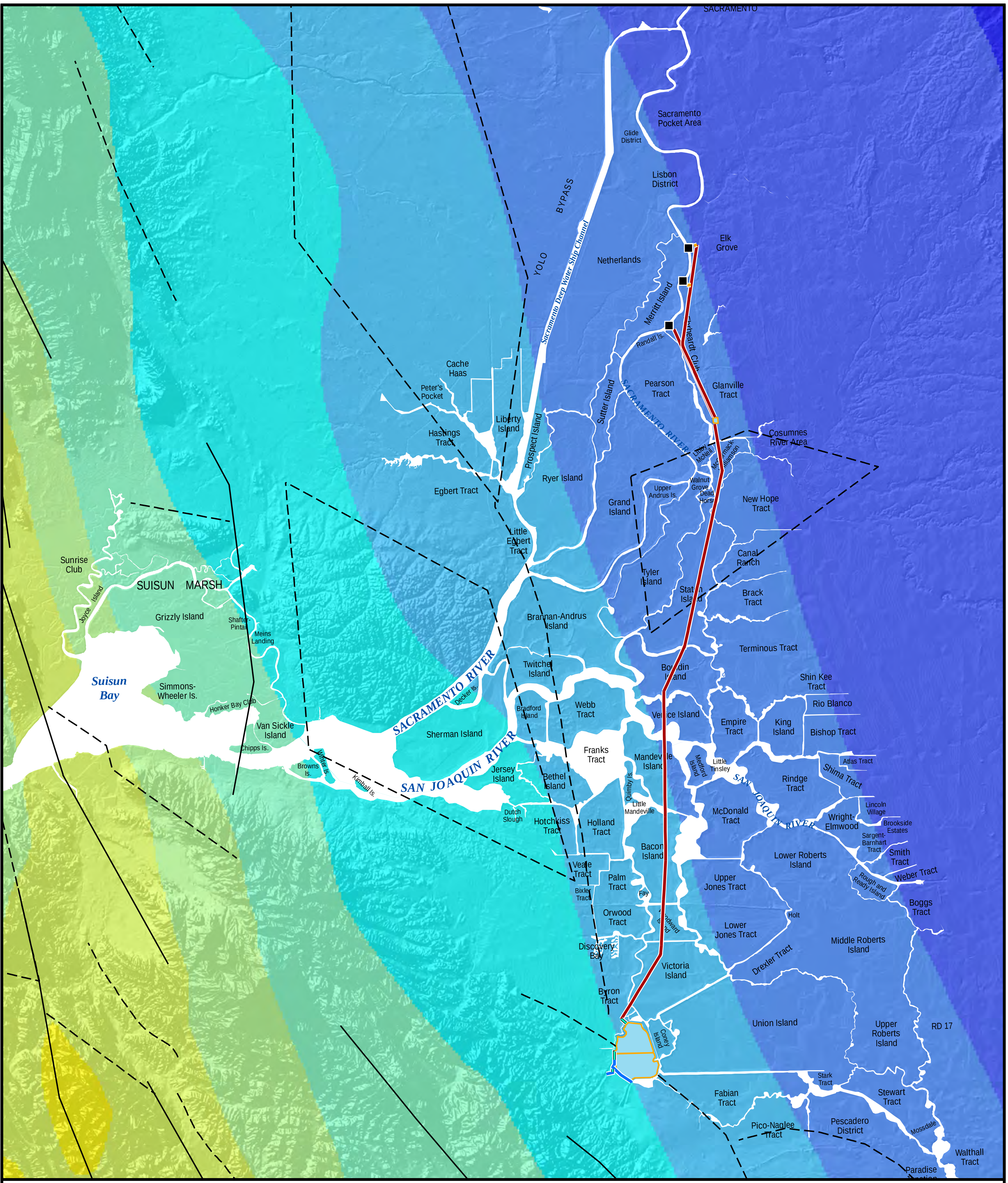
DUAL CONVEYANCE FACILITY MODIFIED PIPELINE/TUNNEL OPTION

Active Faults in the Delta Region

Figure
A-5



Faults

Surficial fault used in the hazard analysis

Blind fault used in the hazard analysis

Modified Pipeline/Tunnel Alignment

■ Intake

Canal

Conveyance Pipeline

Siphon

Tunnel

Forebay

PGA (g), 200-year return period

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

Source: DWR, 2007. Delta Risk Management Strategy (DRMS)
Phase 1 Risk Analysis: Topical Area: Seismology (Draft 2),
prepared by URS Corporation. June.

N

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Miles

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PGA Hazard for a 200-year Return Period

Figure
A-6

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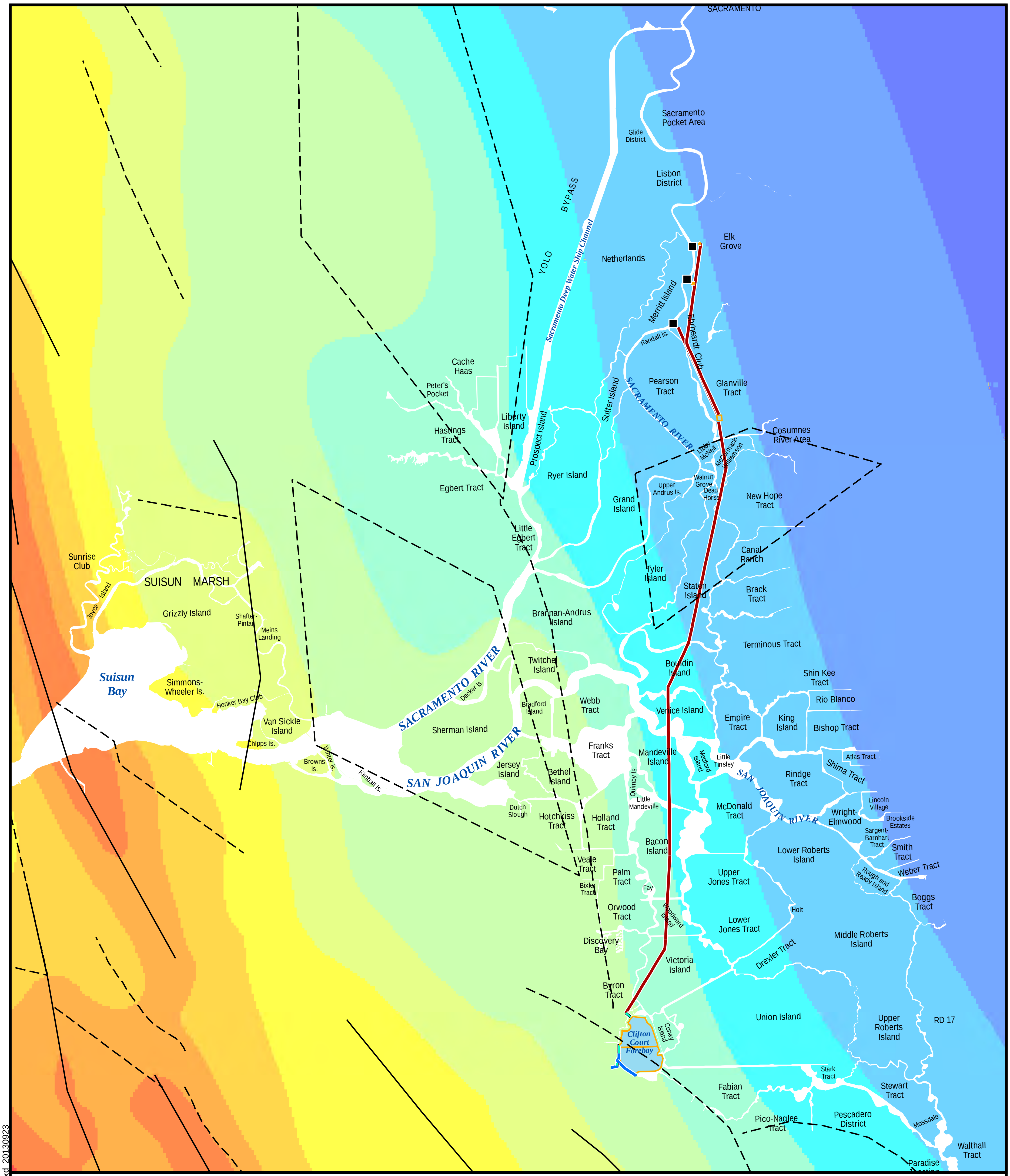

Faults

─── Surficial fault used in the hazard analysis

- - - Blind fault used in the hazard analysis

Modified Pipeline/Tunnel Alignment

■ Intake

Canal

Conveyance Pipeline

Siphon

Tunnel

Forebay

PGA (g), 500-year return period

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

Source: DWR, 2007. Delta Risk Management Strategy (DRMS)
Phase 1 Risk Analysis: Topical Area: Seismology (Draft 2),
prepared by URS Corporation. June.

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DUAL CONVEYANCE FACILITY MODIFIED PIPELINE/TUNNEL OPTION

PGA Hazard for a 500-year Return Period

Figure
A-7

Appendix B

Conceptual Level Construction Sequencing of DHCCP Intakes

BLACK & VEATCH
Building a world of difference.

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 CH2MHill (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

Phase	Activity	Predecessor	Description
Initial Groundwater Management	1	—	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 potion of Highway 160 to minimize potential seepage through and under the existing levee.
	2	1	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.
Highway 160 Relocation	3	1,2	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.
	4	3	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.
	5	4	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).
	6	5	Construct the roadway for the relocated segment of Highway 160 and reroute traffic to new permanent alignment on widened levee section.
Intake Structure Construction	7	6	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.
	8	6	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.
	9	7, 8	Excavate within the intake structure cofferdam and installed diaphragm wall. Install dewatering system within area enclosed by cofferdam and unwater cofferdam.
	10	9	Construct drilled casings and reinforced concrete piers beneath the intake structure. Install tremie seal within cofferdam.
	11	10	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.

Phase	Activity	Predecessor	Description
	12	11	Construct soil Improvements between the cofferdam diaphragm wall and any unimproved soils up to the slurry cutoff wall installed with Highway 160 relocation.
	13	12	Construct remaining length of intake box conduits between the intake structure and the previously placed box conduits constructed with the relocation of Highway 160. As required, demolish upper portions of intake structure diaphragm wall and construct fills to finish grade between intake structure and Highway 160.
Landside Facilities Construction	14	6	Construct well point dewatering system, perimeter slurry wall, or implement other means to dewater areas for construction of remaining landside facilities, including the sedimentation basins, tunnel intake structure, and sediment drying lagoons.
	15	14	Construct soil improvements beneath landside facilities susceptible to settlement.
	16	15	Construct all remaining landside facilities and appurtenant works. Close isolation gates and remove sheet pile cofferdam in front of intake structure when intake structure and landside facilities are ready to accept water.
	17	16	Complete work and commission project.

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

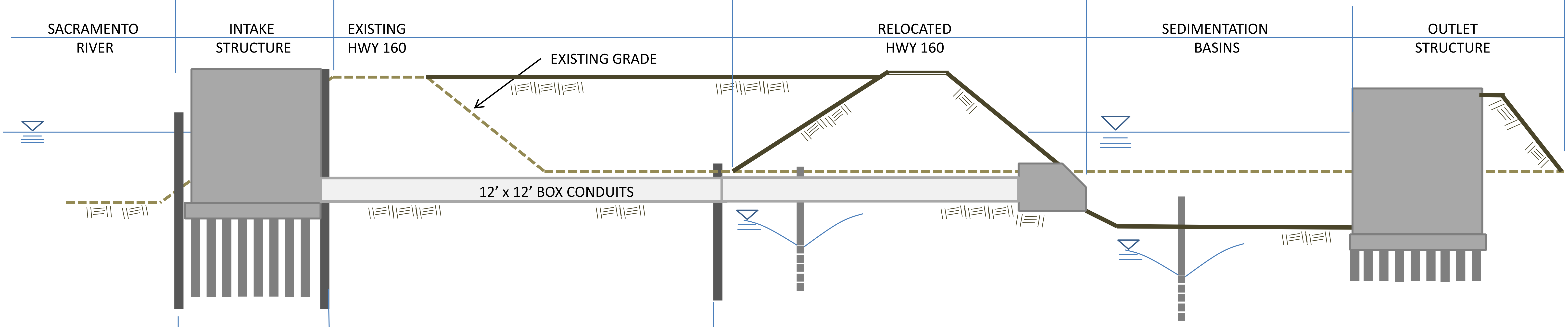
Activity 14 thru 17

(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

GeoPentech

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; 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 a, 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 1

Exploration Name	Data Source	Type	Year
Intake 2			
DCR2-DH-004	DHCCP GDR Rev0	Borehole	2010
DCR2-DH-005	DHCCP GDR Rev0	Borehole	2012
DCR2-DH-006	DHCCP GDR Rev0	Borehole	2010
DCR2-DH-007	DHCCP GDR Rev0	Borehole	2010

Exploration Name	Data Source	Type	Year
DCR2-DH-008	DHCCP GDR Rev0	Borehole	2010
DCR2-DH-009	DHCCP GDR Rev0	Borehole	2012
Intake 3			
DCR3-DH-013	DHCCP GDR Rev0	Borehole	2012
DCR3-DH-014	DHCCP GDR Rev0	Borehole	2012
DCR3-DH-015	DHCCP GDR Rev0	Borehole	2012
DCR3-DH-003	DHCCP GDR Rev0	Borehole	2010
DCR3-DH-005	DHCCP GDR Rev0	Borehole	2010
DCR3-DH-007	DHCCP GDR Rev0	Borehole	2010
DCN3-CPT-004	DHCCP GDR Rev0	CPT	2011
Intake 5			
DCR5-DH-013	DHCCP GDR Rev0	Borehole	2012
DCR5-DH-014	DHCCP GDR Rev0	Borehole	2012
DCR-DH-020	DHCCP GDR Rev0	Borehole	2009
DCR4-DH-004	DHCCP GDR Rev0	Borehole	2010
DCR4-DH-006	DHCCP GDR Rev0	Borehole	2010
DCR4-DH-008	DHCCP GDR Rev0	Borehole	2010
DCR-DH-004	DHCCP GDR Rev0	Borehole	2009
DCN4-DH-009	DHCCP GDR Rev0	Borehole	2011
DCN4-CPT-002	DHCCP GDR Rev0	CPT	2011
DCN4-CPT-009	DHCCP GDR Rev0	CPT	2011

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)[1] 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[1], 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 MCSL 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 a20-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. -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 land-ward 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 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

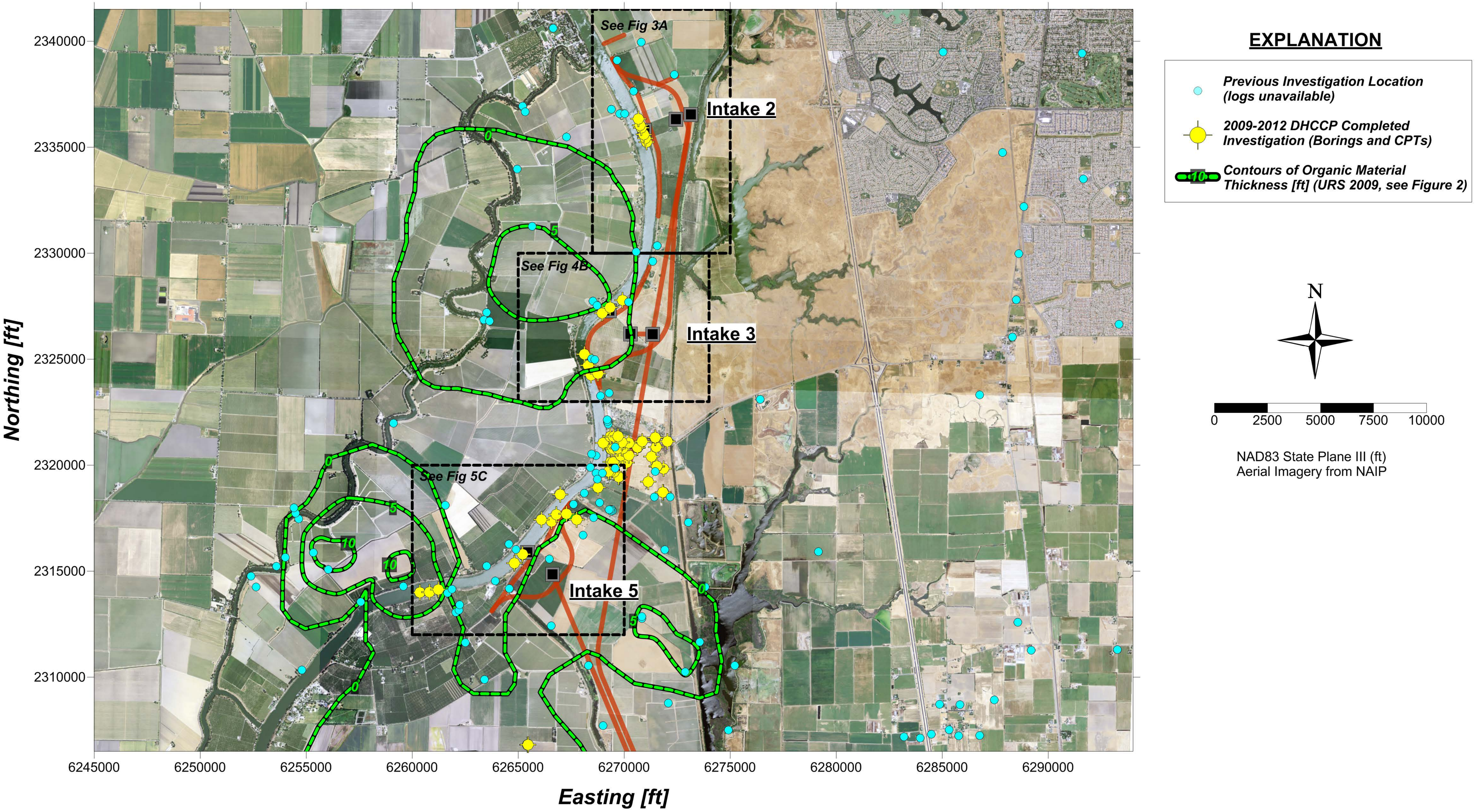
DWR, 2013. Draft 2009 through 2012 Geotechnical Data Report, Pipeline/Tunnel Option, Delta Habitat Conservation and Conveyance Program, Standard Agreement 4600008104, Task Order WGI 14, April 2013, Revision 0, Prepared by California Department of Water Resources. Dated April 2013.

URS, 2009. Delta Risk Management Strategy (DRMS) Phase 1, Risk Analysis Report, Final. Prepared by URS Corporation/Jack R. Benjamin & Associates, Inc. December 2008. Posted online 2009.

Table 2 – Comparison of Potential Ground Improvement Methods

	Advantages	Disadvantages	Costs*
Jet Grouting	• Typically lower replacement ratio than mixing (5%-20%) • No spoils created	• Less effective and consistent in clayey soils • QA/QC difficult and improvement difficult to verify • Typically used for specific local areas, not wide area treatment	Relatively Expensive $80-450/CY
Displacement/Compaction Grouting	• Relatively small equipment required • Lower replacement ratio than mixing (3-20%) • Minimal disturbance to surface • No spoils created	• Could require secondary and tertiary treatment for very low blowcount sand • May not be effective for clayey soils • Silty or clayey sands may require additional treatment • Some problems for shallow areas with little confinement	Wide range of costs depending on site-specific conditions $35-230/CY
DDC	• Very low cost per cubic yard of treated volume compared to other methods under favorable conditions • No spoils created	• Shallow groundwater may require bridging surface layer • 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	Very inexpensive for favorable conditions $1-4/SF
Stone Columns	• Effective for mitigating liquefaction susceptibility • Can provides additional bearing support • No spoils created	• Introduces new seepage paths. Infeasible where groundwater flow must be controlled • Requires large amount of import rock • Less effective for improving clayey material	Relatively inexpensive where feasible $15-50/LF
CDSM	• Can be used to improve both sandy and clayey soils • Depth limitation only based on equipment size • Relatively easy to QA/QC and verify improvement • Can provide additional bearing support • Spoils often make excellent backfill material	• Higher replacement ratio (20-50%) required than compaction grouting • Creates spoils that have to be handled/disposed	Moderate level costs $50-150/CY

*Note that the cost ranges included here are intended for comparison purposes only.



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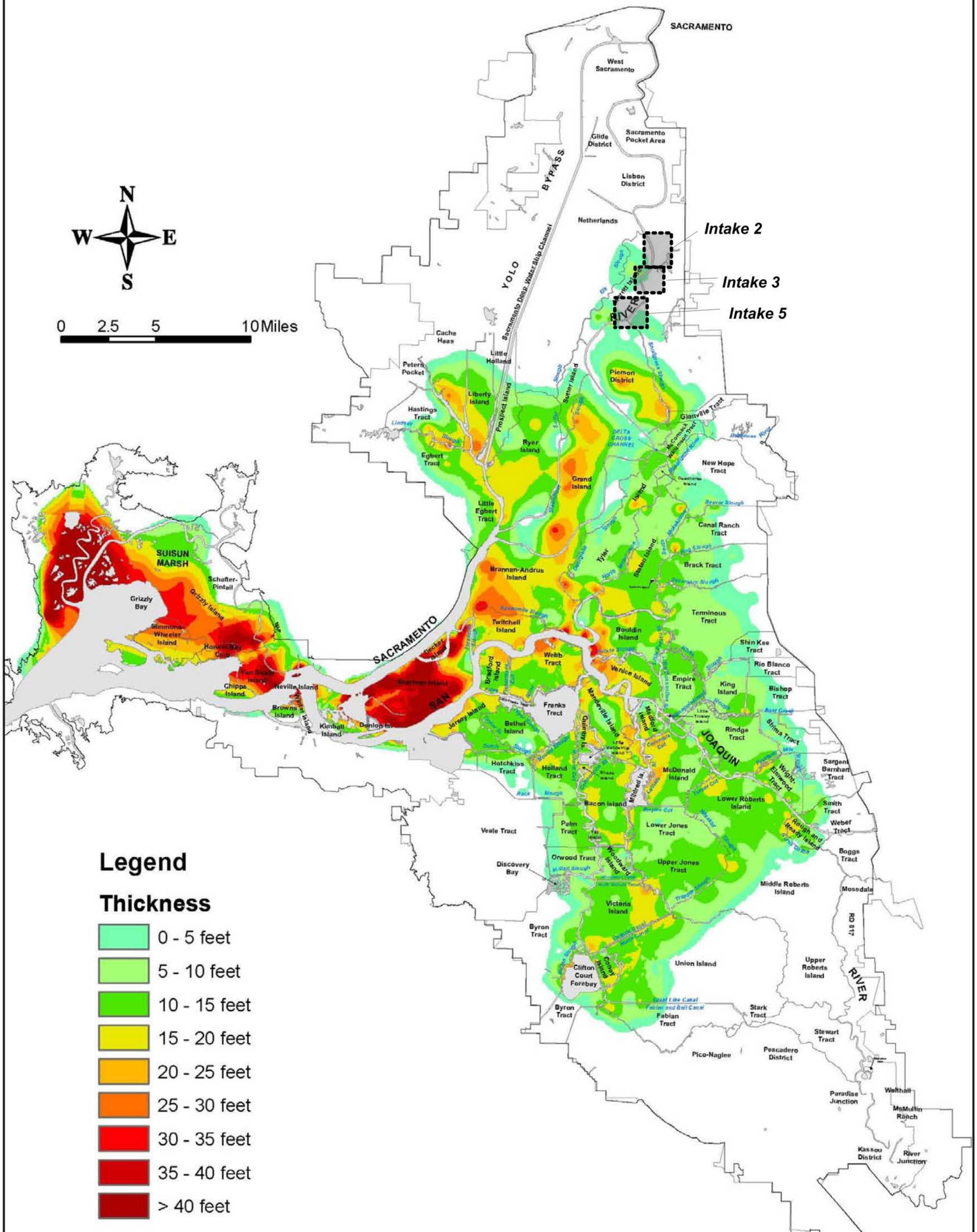
EXPLORATION COVERAGE MAP

Project No.: 15016A	Project: DHCCP Intake Facility Construction Sequencing	Date: Mar 2015	Figure 1

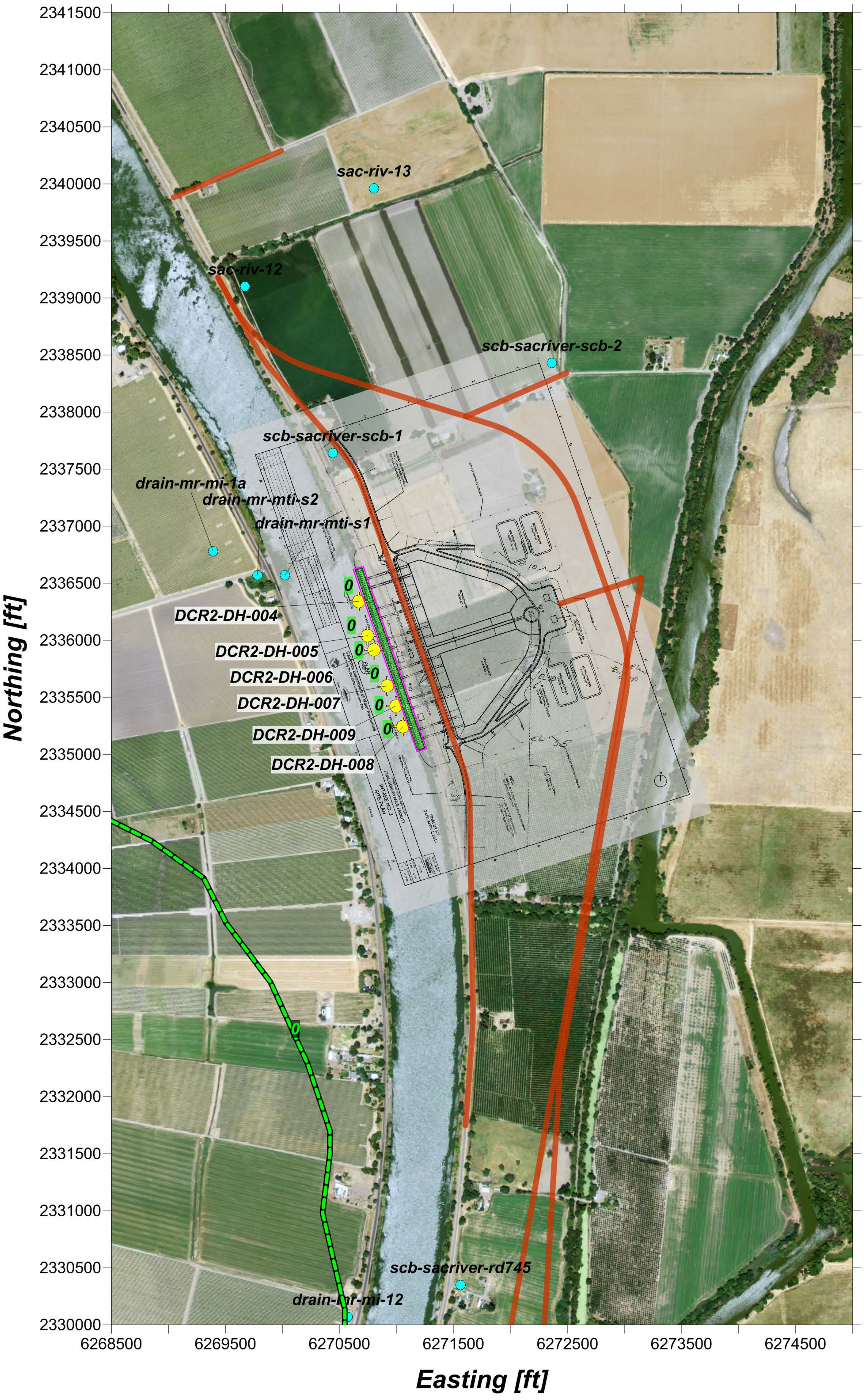


Note: Represents thickness of organic materials near levee toe.

Source: Final DRMS Study, URS, 2009. Figure 6-34.

Thickness of Organic Materials (URS, 2009)

Project No. 15016A	Date: Mar 2015	Project: DHCCP Intake Facility Construction Sequencing	Fig 2



EXPLANATION

●	***Previous Investigation Location (logs unavailable)***
⊕	***2009-2012 DHCCP Completed Investigation (Borings and CPTs)***
0	***Contours of Organic Material Thickness [ft] (URS 2009, see Figure 2)***
0	***Logged/Interpreted Thickness of Organic Material [ft] (boring/cpt)***

Notes:
- NAD83 State Plane III (ft)
- Imagery from NAIP Orthoimagery
- Facility plan from Conceptual Engineering Report Dual Conveyance Facility Intake No. 2 Site Plan, Sheet CCO-C-1011IT, transmitted by Black and Veatch March 9, 2015.
- Boring locations and names from DHCCP GDR Rev0 and DWR Geodatabase v101.

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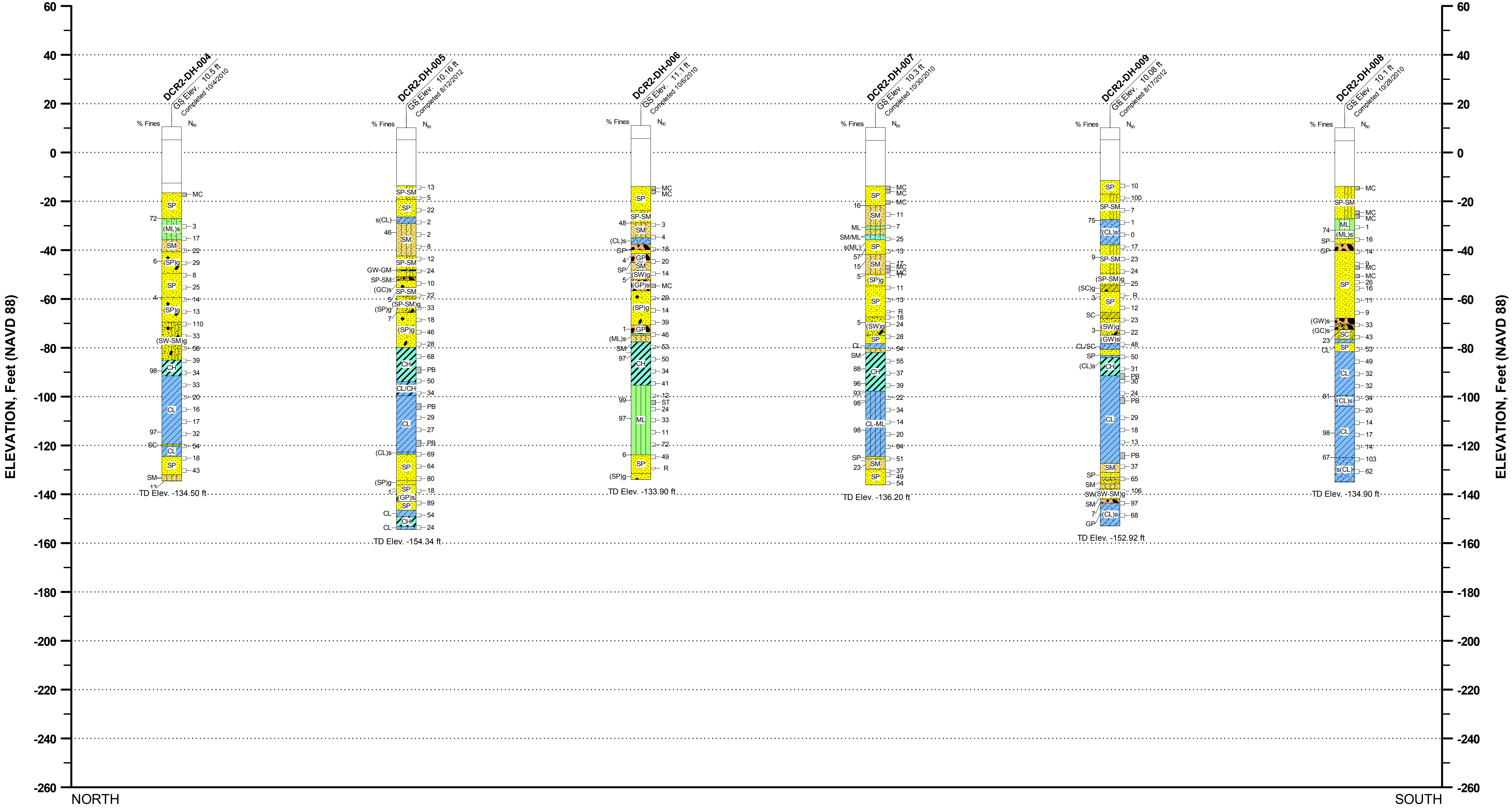
EXPLORATION COVERAGE MAP - INTAKE 2

Project No.: 15016A	Project: DHCCP Intake Facility Construction Sequencing	Date: Mar 2015	Figure 3A



HORIZONTAL DISTANCE NOT TO SCALE

NOTES:
1. CORRELATION OF LITHOLOGIES BETWEEN WIDELY SPACED BORINGS IS NOT RECOMMENDED
2. GS Elev FOR OVERWATER BORINGS IS THE ELEVATION OF THE DRILL DECK
3. SEE FIGURE 13, SHEETS 1 AND 2, KEY TO BORING AND GRAPHIC LOGS
4. SEE PLATE 2 FOR LOCATIONS OF BORINGS

DHCCP-Geotechnical
DWR-Internal Only
WORKING DRAFT - Subject to Change

0 20 40 60 80 100 Vertical Scale 40.00 ft/in.

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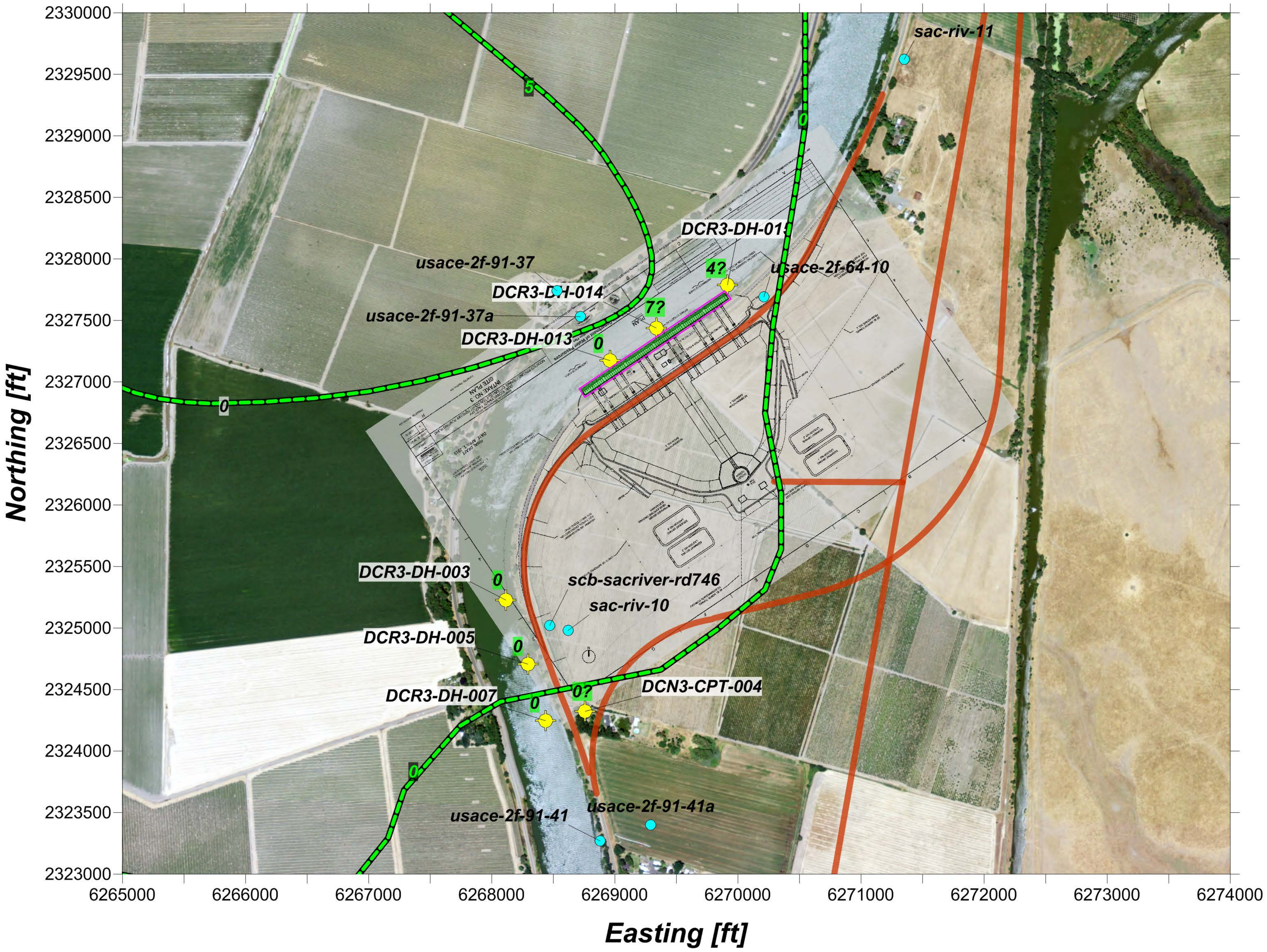
2009 THROUGH 2012
GEOTECHNICAL DATA REPORT

INTAKE 2
GRAPHIC BORING LOGS

Figure
3B

EXPLANATION

●	***Previous Investigation Location (logs unavailable)***
●	***2009-2012 DHCCP Completed Investigation (Borings and CPTs)***
0	***Contours of Organic Material Thickness [ft] (URS 2009, see Figure 2)***
0	***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-1011IT, transmitted by Black and Veatch March 9, 2015.
- Boring locations and names from DHCCP GDR Rev0 and DWR Geodatabase v101.

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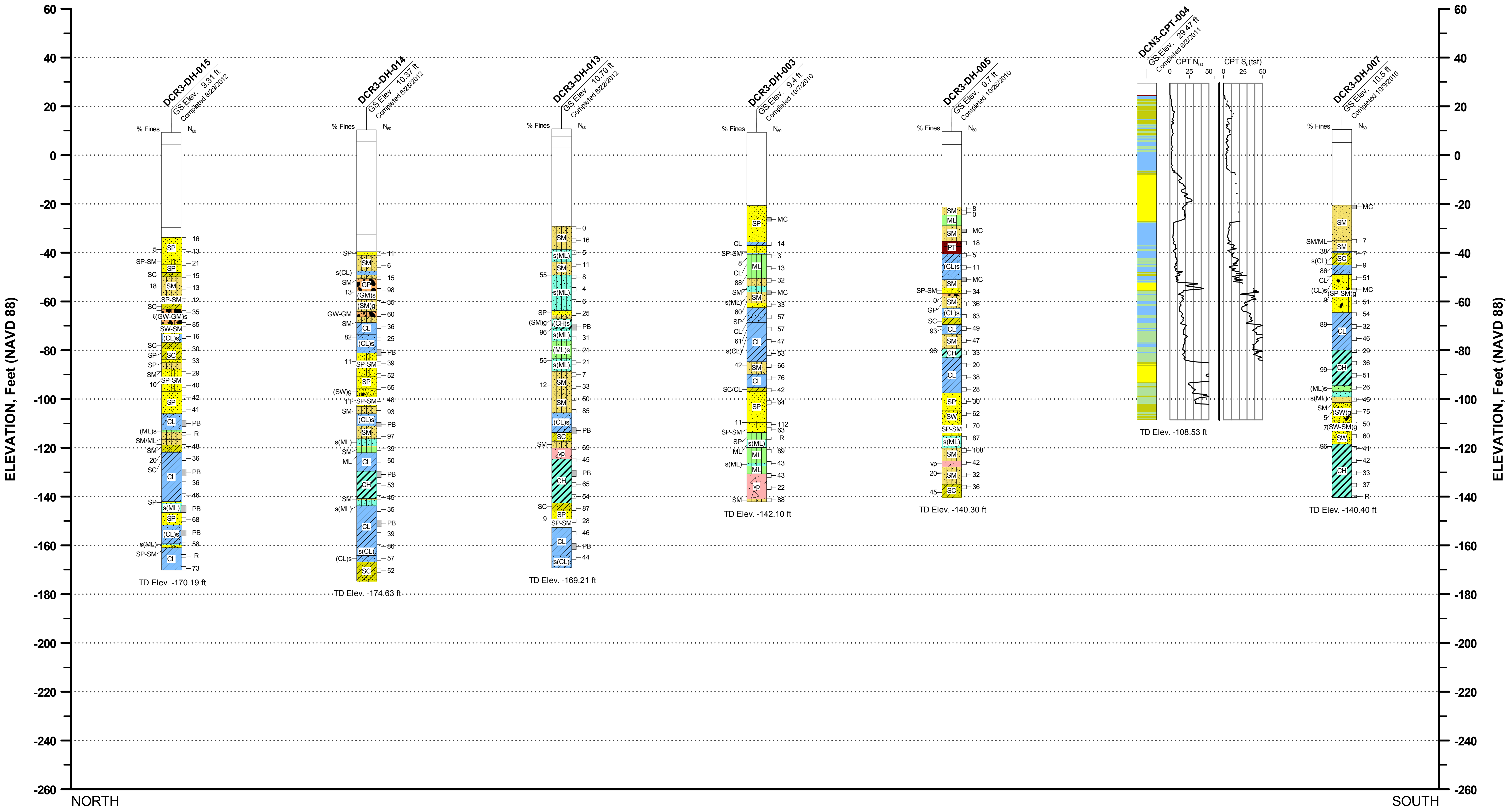
EXPLORATION COVERAGE MAP - INTAKE 3

Project No.: 12040A - TO3	Project: DHCCP Intake Facility Construction Sequencing	Date: Mar 2015	Figure 4A



HORIZONTAL DISTANCE NOT TO SCALE

NOTES:
1. CORRELATION OF LITHOLOGIES BETWEEN WIDELY SPACED BORINGS IS NOT RECOMMENDED
2. GS Elev FOR OVERWATER BORINGS IS THE ELEVATION OF THE DRILL DECK
3. SEE FIGURE 13, SHEETS 1 AND 2, KEY TO BORING AND GRAPHIC LOGS
4. SEE PLATE 2 FOR LOCATIONS OF BORINGS

DHCCP-Geotechnical
DWR-Internal Only
WORKING DRAFT - Subject to Change

0 20 40 60 80 100 Vertical Scale 40.00 ft/in.

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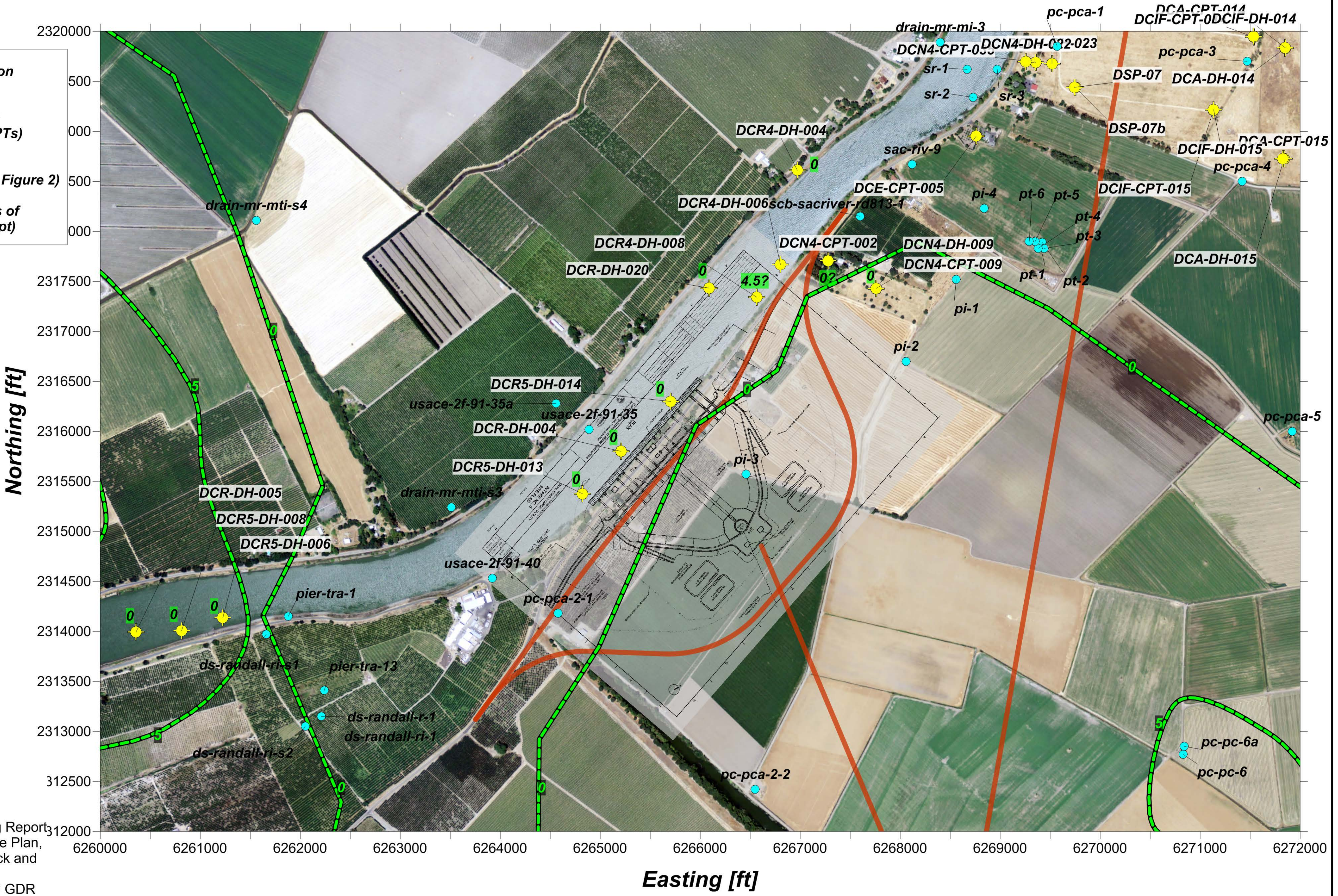
2009 THROUGH 2012
GEOTECHNICAL DATA REPORT

INTAKE 3
GRAPHIC BORING LOGS

Figure
4B

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)*
- **0** *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-1011IT, transmitted by Black and Veatch March 9, 2015.
- Boring locations and names from DHCCP GDR Rev0 and DWR Geodatabase v101.

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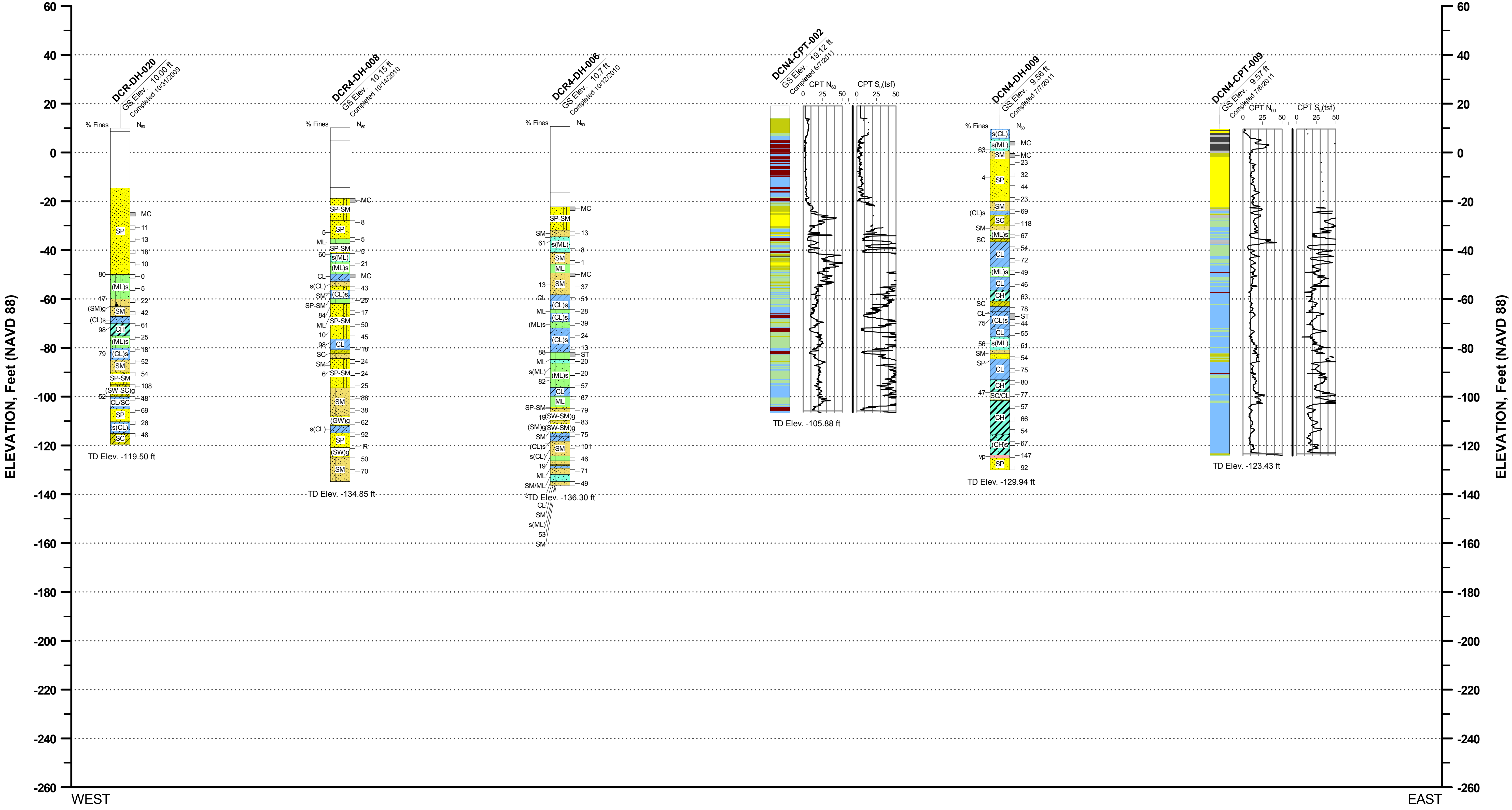
EXPLORATION COVERAGE MAP - INTAKE 5

Project No.: 12040A - TO3	Project: DHCCP Intake Facility Construction Sequencing	Date: Mar 2015	Figure 5A



HORIZONTAL DISTANCE NOT TO SCALE

NOTES:
1. CORRELATION OF LITHOLOGIES BETWEEN WIDELY SPACED BORINGS IS NOT RECOMMENDED
2. GS Elev FOR OVERWATER BORINGS IS THE ELEVATION OF THE DRILL DECK
3. SEE FIGURE 13, SHEETS 1 AND 2, KEY TO BORING AND GRAPHIC LOGS
4. SEE PLATE 2 FOR LOCATIONS OF BORINGS

DHCCP-Geotechnical
DWR-Internal Only
WORKING DRAFT - Subject to Change

0 20 40 60 80 100 Vertical Scale 40.00 ft/in.

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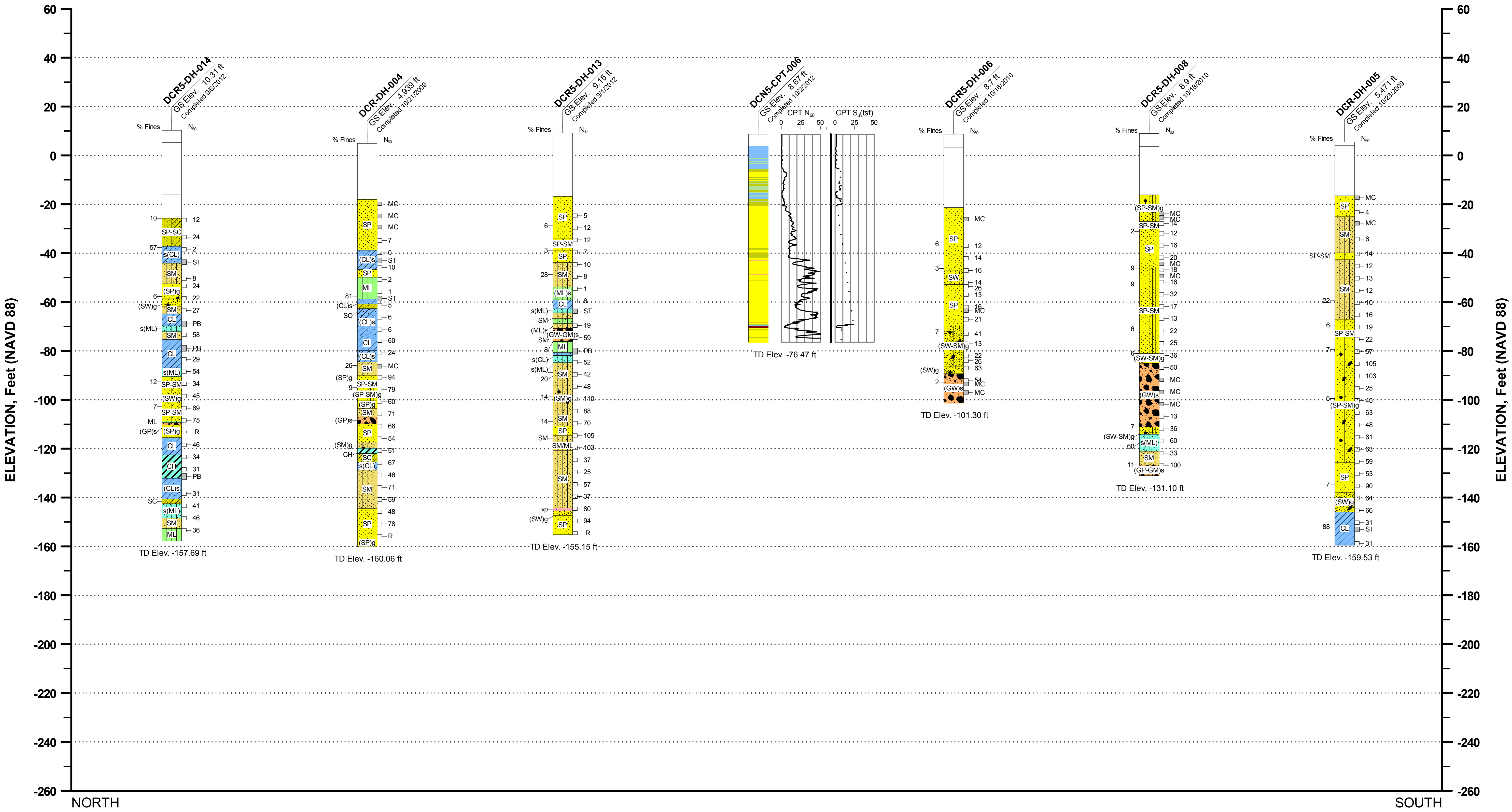
GEOTECHNICAL DATA REPORT

INTAKE 5 (1 of 2)
GRAPHIC BORING LOGS

Figure
5B



HORIZONTAL DISTANCE NOT TO SCALE

NOTES:

1. CORRELATION OF LITHOLOGIES BETWEEN WIDELY SPACED BORINGS IS NOT RECOMMENDED
2. GS Elev FOR OVERWATER BORINGS IS THE ELEVATION OF THE DRILL DECK
3. SEE FIGURE 13, SHEETS 1 AND 2, KEY TO BORING AND GRAPHIC LOGS
4. SEE PLATE 2 FOR LOCATIONS OF BORINGS

DHCCP-Geotechnical
DWR-Internal Only
WORKING DRAFT - Subject to Change

0 20 40 60 80 100
Vertical Scale
40.00 ft/in.

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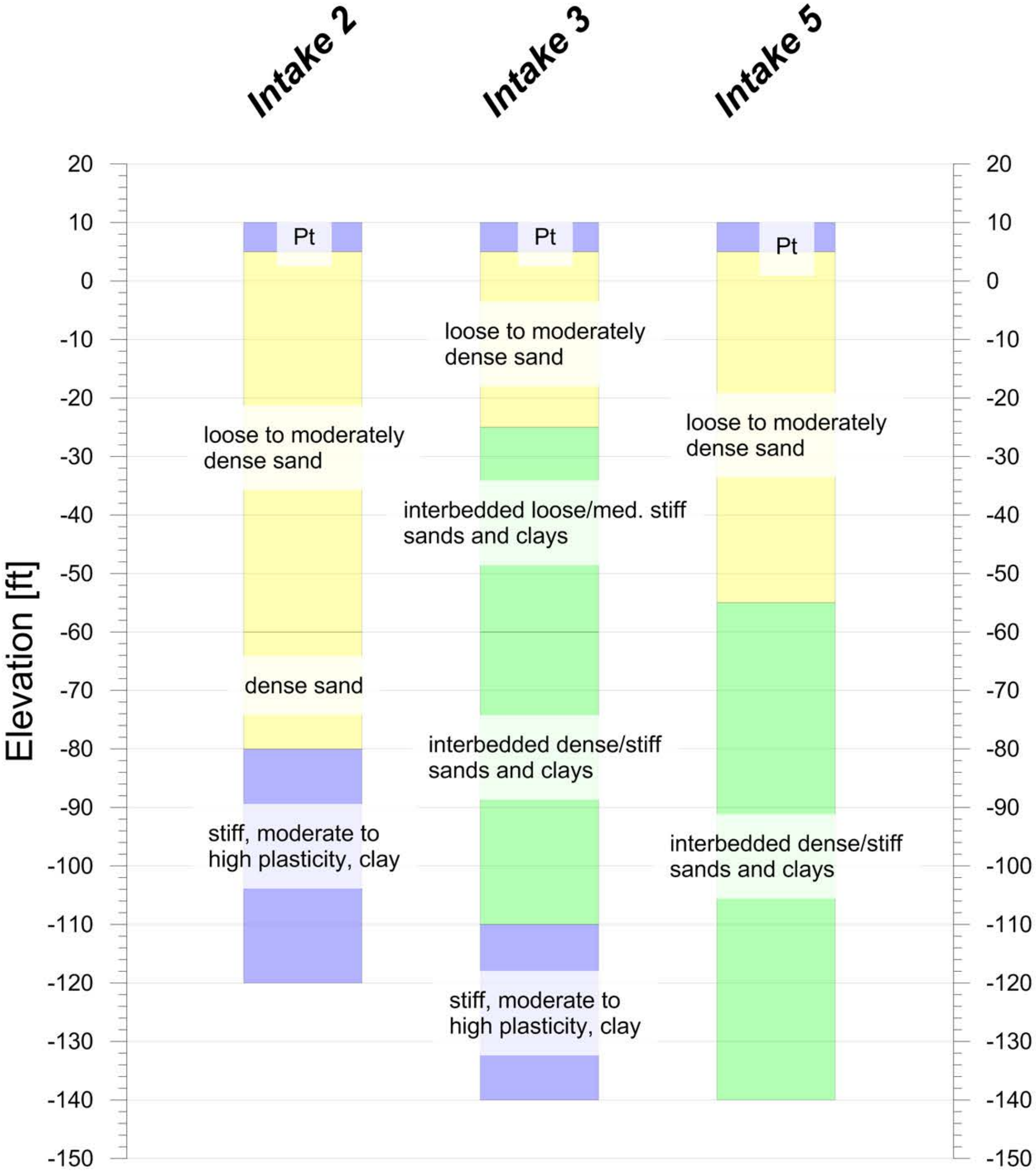
2009 THROUGH 2012
GEOTECHNICAL DATA REPORT

INTAKE 5 (2 of 2)
GRAPHIC BORING LOGS

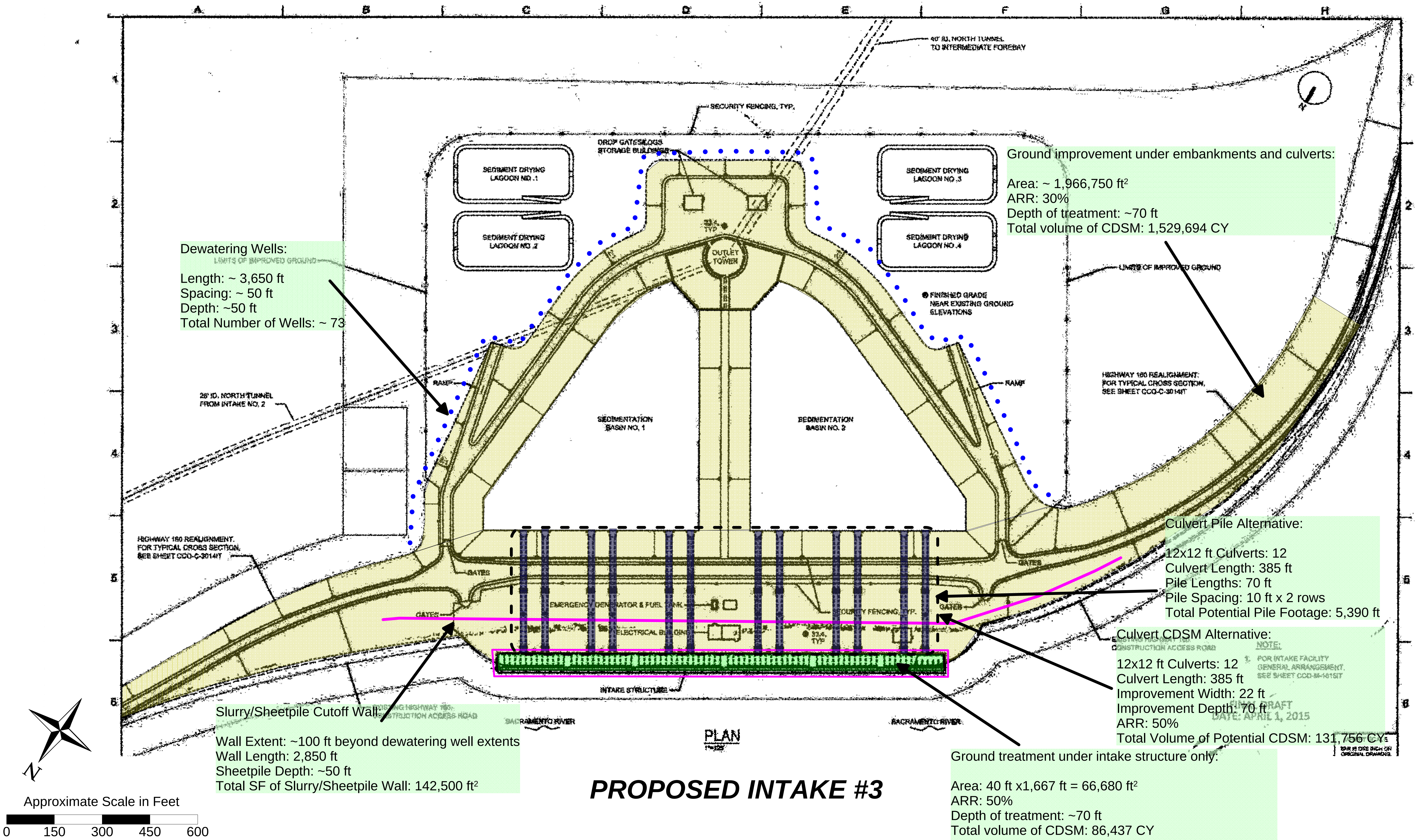
Figure
5C



Generalized Subsurface Profiles - Proposed Intake Locations

Project No. 15016A	Date: Mar 2015	Project: DHCCP Intake Facility Construction Sequencing	Fig 6

GeoPentech



Dewatering Wells:

Length: ~ 3,650 ft
Spacing: ~ 50 ft
Depth: ~50 ft
Total Number of Wells: ~ 73

Ground improvement under embankments and culverts:

Area: ~ 1,966,750 ft^2
ARR: 30%
Depth of treatment: ~70 ft
Total volume of CDSM: 1,529,694 CY

Culvert Pile Alternative:

12x12 ft Culverts: 12
Culvert Length: 385 ft
Pile Lengths: 70 ft
Pile Spacing: 10 ft x 2 rows
Total Potential Pile Footage: 5,390 ft

Culvert CDSM Alternative:

12x12 ft Culverts: 12
Culvert Length: 385 ft
Improvement Width: 22 ft
Improvement Depth: 70 ft
ARR: 50%
Total Volume of Potential CDSM: 131,756 CY

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 ft^2

Ground treatment under intake structure only:

Area: 40 ft x1,667 ft = 66,680 ft^2
ARR: 50%
Depth of treatment: ~70 ft
Total volume of CDSM: 86,437 CY
Number of Uplift Anchors (1/3 of CDSM Shafts): ~1,150

PROPOSED INTAKE #3

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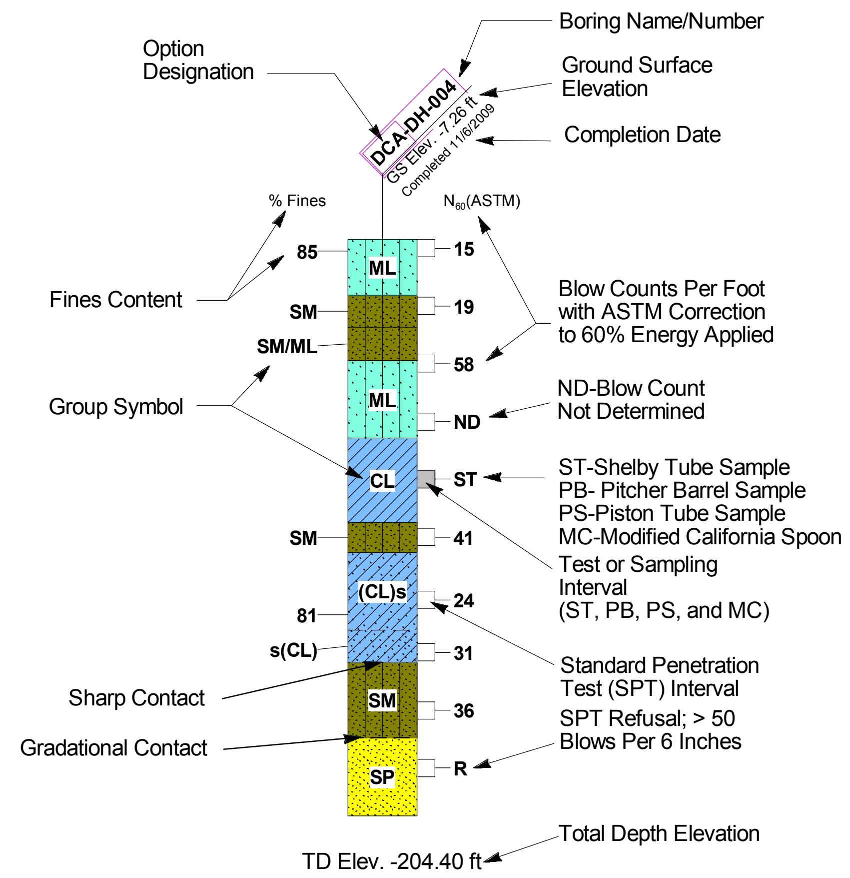
TYPICAL QUANTITY TAKEOFFS

Project No.: 15016A	Project: DHCCP Intake Facility Construction Sequencing	Date: Mar 2014	Figure 7

GROUP SYMBOLS AND NAMES

Graphic	Group Names	Graphic	Group Names	Graphic	Group Names	Graphic	Group Names	Graphic	Group Names	Graphic	Group Names
▨	Lean Clay (CL)	▨	Lean Clay with Sand (CL)s	▨	Lean Clay with Organics (CL)o	▨	Lean Clay with Sand and Organics (CL)s	▨	Sandy Lean Clay s(CL)	▨	Sandy Lean Clay with Gravel s(CL)g
▨	Sandy Lean Clay with Organics s(CH)o	▨	Lean to Fat Clay (CL/CH)	▨	Fat Clay (CH)	▨	Fat Clay with Sand (CH)s	▨	Fat Clay with Organics (CH)o	▨	Fat Clay with Sand and Organics (CH)s
▨	Sandy Fat Clay s(CH)	▨	Sandy Fat Clay with Organics s(CL)o	▨	Silty Clay with Sand (CL-ML)s	▨	Clayey Silt (ML/CL)	▨	Silt (ML)	▨	Silt with Sand (ML)s
▨	Sandy Silt s(ML)	▨	Silt with Organics (ML)o	▨	Elastic Silt (MH)	▨	Elastic Silt with Sand (MH)s	▨	Sandy Elastic Silt s(MH)	▨	Elastic Silt with Organics (MH)o
▨	Organic Lean Clay (OL)	▨	Organic Lean Clay with Sand (OL)s	▨	Sandy Organic Lean Clay s(OL)	▨	Organic Fat Clay (OH)	▨	Organic Fat Clay with Sand (OH)	▨	Organic Silt (OL)
▨	Organic Silt with Sand (OL)s	▨	Sandy Organic Silt s(OL)	▨	Organic Elastic Silt (OH)	▨	Peat (PT)	▨	Poorly Graded Sand (SP)	▨	Poorly Graded Sand with Gravel (SP)g
▨	Poorly Graded Sand with Clay (SP-SC)	▨	Poorly Graded Sand with Organics (SP)o	▨	Poorly Graded Sand with Silt (SP-SM)	▨	Poorly Graded Sand with Silt and Gravel (SP-SM)g	▨	Poorly Graded Sand with Silt and Organics (SP-SM)o	▨	Well-Graded Sand (SW)
▨	Well-Graded Sand with Gravel (SW)g	▨	Well-Graded Sand with Clay (SW-SC)	▨	Well-Graded Sand with Silt (SW-SM)	▨	Well-Graded Sand with Silt and Gravel (SW-SM)g	▨	Clayey Sand (SC)	▨	Clayey Sand with Gravel (SC)g
▨	Silty Clayey Sand (SC-SM)	▨	Silty Sand (SM)	▨	Silty Sand with Gravel (SM)g	▨	Silty Sand with Organics (SM)o	▨	Poorly Graded Gravel (GP)	▨	Poorly Graded Gravel with Sand (GP)s
▨	Poorly Graded Gravel with Silt and Sand (GP-GM)	▨	Well-Graded Gravel (GW)	▨	Well-Graded Gravel with Sand (GW)s	▨	Well-Graded Gravel with Silt and Sand (GW-GM)	▨	Clayey Gravel (GC)	▨	Clayey Gravel with Sand (GC)s
▨	Silty Gravel with Sand (GM)s	▨	Silty Gravel (GM)	▨	Tephra (vp)						

BORING EXPLANATION



SAMPLER TYPE AND RETAINED SAMPLE GRAPHIC SYMBOLS

▯ Standard Penetration Test (SPT)	▯ Hydropunch	▯ Piston	▯ Bag Sample	▯ Boxed Core
▯ Modified California (MCS)	▯ Punch Core (PCore)	▯ Pitcher Barrel	▯ Bulk Sample	
▯ No Sample Drilling (NSD)	▯ Dry Core (DCore)	▯ Shelby Tube	▯ Jar Sample	

NORMALIZED SOIL BEHAVIOR TYPE (CPT)

🟦 1: Sensitive Fine-grained	🟩 4: Clayey Silt to Silty Clay	🟧 7: Gravelly Sand to Sand
🟥 2: Organic Material	🟩 5: Silty Sand to Sandy Silt	⬛ 8: Very stiff Sand to Clayey sand
🟦 3: Clay to Silty Clay	🟨 6: Clean Sand to Silty Sand	⬜ 9: Very Stiff Fine-grained

OPTION DESIGNATION

DCA - Pipeline/Tunnel Option	DCN1 - Intake 1 Land	DCR1 - Intake 1 Overwater
DCE - ICF East Option	DCN2 - Intake 2 Land	DCR2 - Intake 2 Overwater
DCW - ICF West Option	DCN3 - Intake 3 Land	DCR3 - Intake 3 Overwater
DCT - Through Delta Facility Option	DCN4 - Intake 4 Land	DCR4 - Intake 4 Overwater
DCR - Over Water	DCN5 - Intake 5 Land	DCR5 - Intake 5 Overwater
DCRA - Overwater Pipeline/Tunnel		

DRILLING METHOD GRAPHIC SYMBOLS

▯ Bucket Auger (BA)	▯ Hollow Stem Auger (HSA)	▯ Solid Stem Auger	▯ Rotary Wash
▯ Hand Auger	▯ Rock Coring	▯ Sonic	

WELL GRAPHIC SYMBOLS

▯ Blank casing in cement grout	▯ Blank casing in filter sand	▯ Native backfill or slough
▯ Blank casing in bentonite seal	▯ Slotted casing in filter sand	▯ Vibrating wire piezometer

Notes:

1) Soils were classified in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils (Visual Manual Procedure). Where laboratory testing was performed, classifications are presented in the remarks column of the log in general accordance with ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

2) Borderline symbols, two group symbols separated by a slash, may be used in field visual classification when (A) percentage of fines is estimated to be between 45% and 55%, (B) percentages of sand and gravel are estimated to be about the same, (C) soil could be either well graded or poorly graded, (D) soil could be either a silt or a clay, or (E) fine-grained soil has properties indicating that it is at the boundary between low and high plasticity.

3) Dashed line between soil types is used when (A) "inferred" boundary between soil layers or lithology or, (B) a gradational boundary, or (C) if loss in recovery is significant.

4) Solid line between soil types is used where a definitive boundary is observed between two materials.

FIELD AND LABORATORY TESTS

AL	Atterberg Limits - Liquid Limit, Plastic Limit, Plasticity Index (ASTM D4318)	**OC**	Organic Content (ASTM D2974)	**TV**	Pocket Torvane
CN	Consolidation (ASTM D2435)	**PA**	Particle Size Analysis (ASTM D422)	**UC**	Unconfined Compression (ASTM D2166)
CR	Corrosion, Sulfates, Chlorides (CTM 643; CTM 417; CTM 422)	**PD**	Pinhole Dispersion (ASTM D4647)	**UU**	Unconsolidated Undrained Triaxial (ASTM D2850)
CU	Consolidated Undrained Triaxial (ASTM D4767)	**PM**	Pressure Meter (ASTM D4719)	**UW**	Unit Weight (ASTM D2937)
DS	Direct Shear (ASTM D3080)	**PP**	Pocket Penetrometer	**XD**	XRay Diffraction
EI	Expansion Index (ASTM D4829)	**SA**	Soil Abrasivity (NTNU 2006)	**XR**	XRay Radiography (ASTM D4452)
HD	Hydrometer (ASTM D422)	**SG**	Specific Gravity (ASTM D854)		
		SL	Slury Abrasion (ASTM G75)		
		SW	Swell Potential (ASTM D4546)		

WATER LEVEL SYMBOLS

- ▽ First water encountered during drilling
- ▼ Water level measured at end of drilling
- ▽ Static water level reading (short-term)
- ▽ Static water leavel reading (long-term)

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2009 THROUGH 2012
GEOTECHNICAL DATA REPORT

KEY TO BORING AND GRAPHIC LOGS

Figure 13

Sheet 1 of 2

DHCCP-Geotechnical
DWR-Internal Only
WORKING DRAFT - Subject to Change

CONSISTENCY OF COHESIVE SOILS (AASHTO)

Descriptor	Pocket Penetrometer (tsf)	Torvane (tsf)
Very Soft	< 0.25	< 0.12
Soft	0.25 - 0.50	0.12 - 0.25
Medium Stiff	0.50 - 1.0	0.25 - 0.50
Stiff	1.0 - 2.0	0.50 - 1.0
Very Stiff	2.0 - 4.0	1.0 - 2.0
Hard	> 4.0	> 2.0

APPARENT DENSITY OF COHESIONLESS SOILS (ASTM 6066)

Descriptor	SPT N(60) - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE (ASTM D 2488)

Descriptor	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

SOIL PARTICLE SIZE (ASTM D 2488)

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

DESCRIPTION OF ORGANICS

Organic Content	Description	
50 to 100%	"Peat"	Fibric (Humification factor of H1 to H3)
		Hemic (Humification factor of H4 to H6)
		Sapric (Humification factor of H7 to H10)
15 to 50%	"Organic (Soil Name)"	
5 to 15%	"(Soil Name) with organics"	

For organic content > 50%, the degree of humification (humification factor from H1 to H10) is determined by taking a representative sample and squeezing it with your hand firmly. The color of the water or the amount of amorphous material expelled is used in addition to the intactness of the original plant material to place the organics into one of the humification classes (ASTM D 5715). The humification factor is used to determine the fiber content of the soil based on ASTM D4427.

PLASTICITY OF FINE-GRAINED SOILS (ASTM D 2488)

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

DRY STRENGTH OF FINE-GRAINED SOILS (ASTM D 2488)

Descriptor	Criteria
None	Dry specimen crumbles into powder with mere pressure of handling.
Low	Dry specimen crumbles into powder with some finger pressure.
Medium	Dry specimen breaks into pieces or crumbles with considerable finger pressure.
High	Dry specimen cannot be broken with finger pressure; will break into pieces between thumb and a hard surface.
Very High	Dry specimen cannot be broken between thumb and a hard surface.

DILATANCY OF FINE-GRAINED SOILS (ASTM D 2488)

Descriptor	Criteria
None	No visible change in the specimen.
Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear, or disappears slowly, upon squeezing.
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing.

TOUGHNESS OF FINE-GRAINED SOILS (ASTM D 2488)

Descriptor	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.
Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness.
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high stiffness.

CEMENTATION (ASTM D 2488)

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure.

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KEY TO BORING AND GRAPHIC LOGS

Figure 13

Sheet 2 of 2

Appendix C

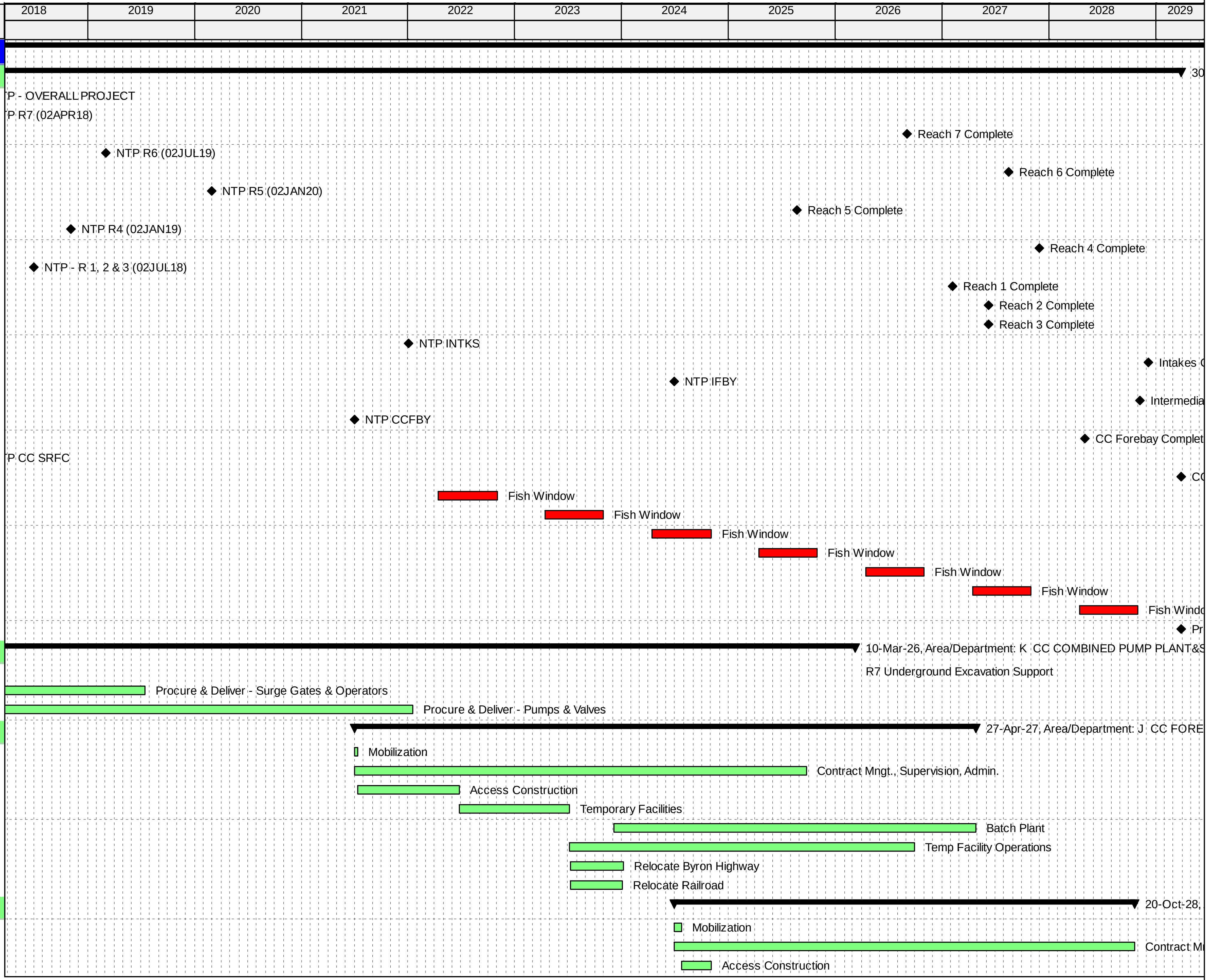
MPTO/CCO Conceptual Construction Schedule

CALIFORNIA WATER FIX

Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
Location: 01 Common Activities	
Area/Department: A MILESTONES	
A1000	NTP - OVERALL PROJECT
B1000	NTP R7 (02APR18)
B9000	Reach 7 Complete
C1000	NTP R6 (02JUL19)
C9000	Reach 6 Complete
D1000	NTP R5 (02JAN20)
D9000	Reach 5 Complete
E1000	NTP R4 (02JAN19)
E9000	Reach 4 Complete
F1000	NTP - R 1, 2 & 3 (02JUL18)
F9000	Reach 1 Complete
F9002	Reach 2 Complete
F9004	Reach 3 Complete
G1000	NTP INTKS
G9000	Intakes Complete
H1000	NTP IFBY
H9000	Intermediate Forebay Complete
J1000	NTP CCFBY
J9000	CC Forebay Complete
K1000	NTP CC SRFC
K9000	CC Surface Work Complete
W0020	Fish Window
W0030	Fish Window
W0040	Fish Window
W0050	Fish Window
W0060	Fish Window
W0070	Fish Window
W0080	Fish Window
Z9000	Project Complete
Area/Department: K CC COMBINED PUMP PLANT&SURGE SHAFT	
L50.0190	R7 Underground Excavation Support
L50.0300	Procure & Deliver - Surge Gates & Operators
L50.0310	Procure & Deliver - Pumps & Valves
Area/Department: J CC FOREBAY	
08002	Mobilization
08003	Contract Mngt., Supervision, Admin.
08004	Access Construction
08005	Temporary Facilities
08006	Batch Plant
08007	Temp Facility Operations
08050	Relocate Byron Highway
08060	Relocate Railroad
Area/Department: H INTERMEDIATE FOREBAY	
07002	Mobilization
07003	Contract Mngt., Supervision, Admin.
07004	Access Construction



🟦 Actual Work
🟥 Critical Remaining Work
▼▼ Summary
🟩 Remaining Work
◆ Milestone

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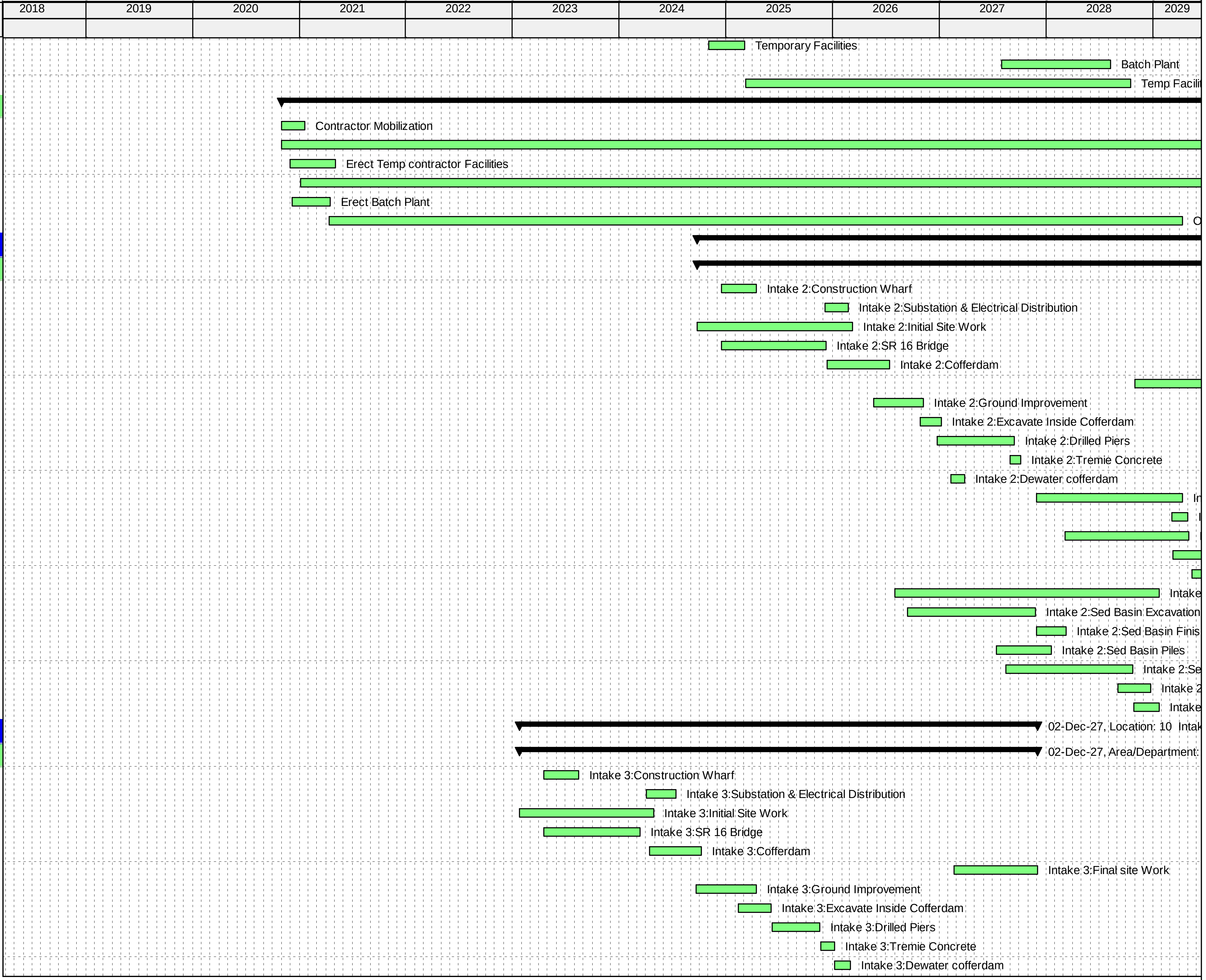
Rev 00

CALIFORNIA WATER FIX

Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
07005	Temporary Facilities
07006	Batch Plant
07007	Temp Facilities Operations
Area/Department: G INTAKES	
2	Contractor Mobilization
3	Contractor Staff
4	Erect Temp contractor Facilities
5	Operate Temp Facilities
6	Erect Batch Plant
7	Operate Batch Plant
Location: 09 Intake 2	
Area/Department: G INTAKES	
201	Intake 2:Construction Wharf
203	Intake 2:Substation & Electrical Distribution
205	Intake 2:Initial Site Work
206	Intake 2:SR 16 Bridge
207	Intake 2:Cofferdam
209	Intake 2:Final site Work
211	Intake 2:Ground Improvement
213	Intake 2:Excavate Inside Cofferdam
215	Intake 2:Drilled Piers
217	Intake 2:Tremie Concrete
219	Intake 2:Dewater cofferdam
221	Intake 2:Intake Concrete
223	Intake 2:Gates
225	Intake 2:Fish Screens
227	Intake 2:MEP
229	Intake 2:Finish Out
251	Intake 2:Sed Basin Deep Wells
253	Intake 2:Sed Basin Excavation
254	Intake 2:Sed Basin Finish Grade & Pave
255	Intake 2:Sed Basin Piles
257	Intake 2:Sediment Basin Concrete
259	Intake 2:Sed Basin Gates
261	Intake 2:Sed Basin MEP & Finish
Location: 10 Intake 3	
Area/Department: G INTAKES	
301	Intake 3:Construction Wharf
303	Intake 3:Substation & Electrical Distribution
305	Intake 3:Initial Site Work
306	Intake 3:SR 16 Bridge
307	Intake 3:Cofferdam
309	Intake 3:Final site Work
311	Intake 3:Ground Improvement
313	Intake 3:Excavate Inside Cofferdam
315	Intake 3:Drilled Piers
317	Intake 3:Tremie Concrete
319	Intake 3:Dewater cofferdam



🟦 Actual Work 🟥 Critical Remaining Work ▼ Summary
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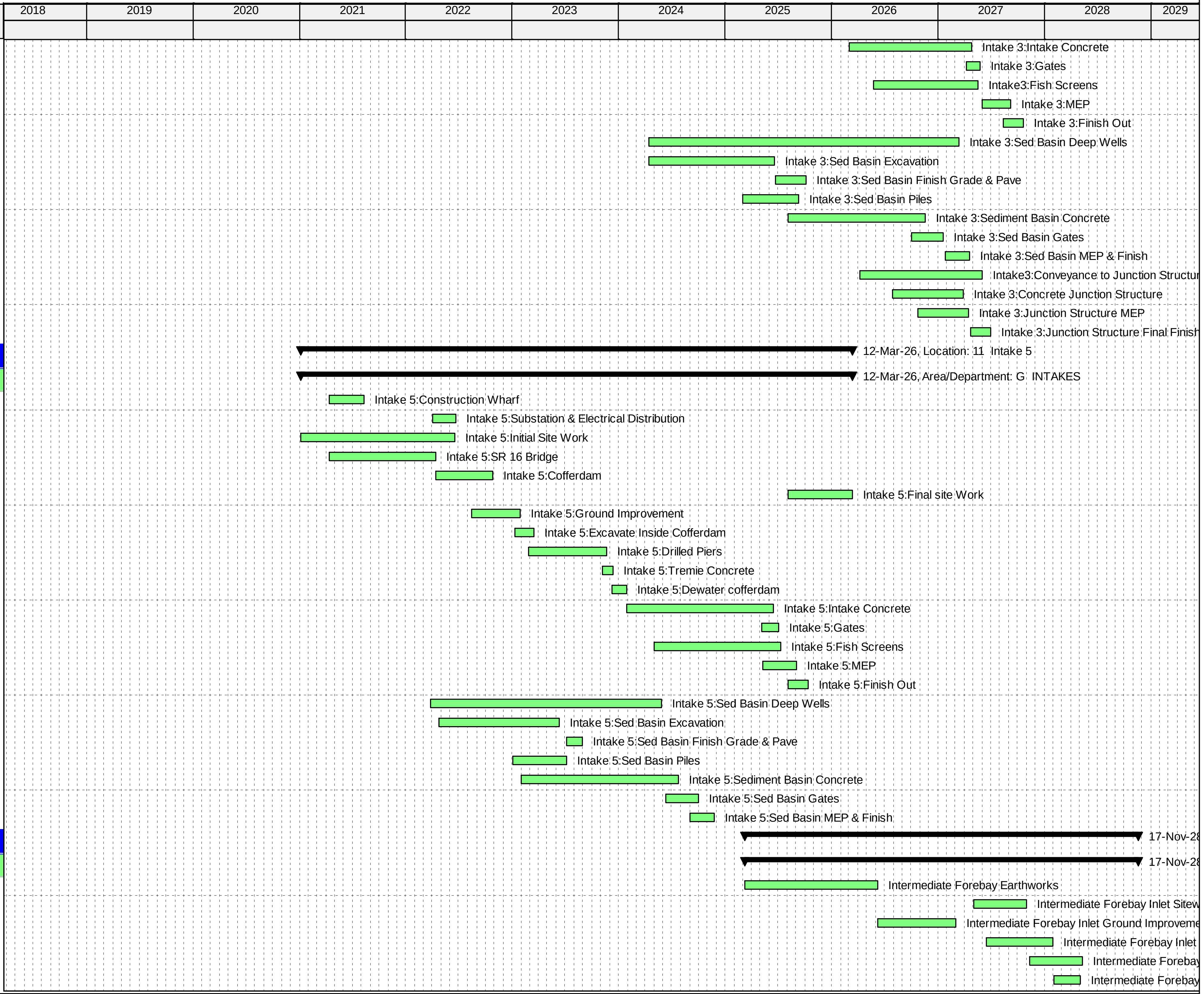
CALIFORNIA WATER FIX

Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
321	Intake 3:Intake Concrete
323	Intake 3:Gates
325	Intake3:Fish Screens
327	Intake 3:MEP
329	Intake 3:Finish Out
351	Intake 3:Sed Basin Deep Wells
353	Intake 3:Sed Basin Excavation
354	Intake 3:Sed Basin Finish Grade & Pave
355	Intake 3:Sed Basin Piles
357	Intake 3:Sediment Basin Concrete
359	Intake 3:Sed Basin Gates
361	Intake 3:Sed Basin MEP & Finish
365	Intake3:Conveyance to Junction Structure
367	Intake 3:Concrete Junction Structure
369	Intake 3:Junction Structure MEP
371	Intake 3:Junction Structure Final Finish/Cleanup
Location: 11 Intake 5	
Area/Department: G INTAKES	
501	Intake 5:Construction Wharf
503	Intake 5:Substation & Electrical Distribution
505	Intake 5:Initial Site Work
506	Intake 5:SR 16 Bridge
507	Intake 5:Cofferdam
509	Intake 5:Final site Work
511	Intake 5:Ground Improvement
513	Intake 5:Excavate Inside Cofferdam
515	Intake 5:Drilled Piers
517	Intake 5:Tremie Concrete
519	Intake 5:Dewater cofferdam
521	Intake 5:Intake Concrete
523	Intake 5:Gates
525	Intake 5:Fish Screens
527	Intake 5:MEP
529	Intake 5:Finish Out
551	Intake 5:Sed Basin Deep Wells
553	Intake 5:Sed Basin Excavation
554	Intake 5:Sed Basin Finish Grade & Pave
555	Intake 5:Sed Basin Piles
557	Intake 5:Sediment Basin Concrete
559	Intake 5:Sed Basin Gates
561	Intake 5:Sed Basin MEP & Finish
Location: 12 Intermediate Forebay Work	
Area/Department: H INTERMEDIATE FOREBAY	
07010	Intermediate Forebay Earthworks
07020	Intermediate Forebay Inlet Sitework
07022	Intermediate Forebay Inlet Ground Improvements
07035	Intermediate Forebay Inlet Concrete
07040	Intermediate Forebay Inlet Gates
07045	Intermediate Forebay Inlet Mech & Elect



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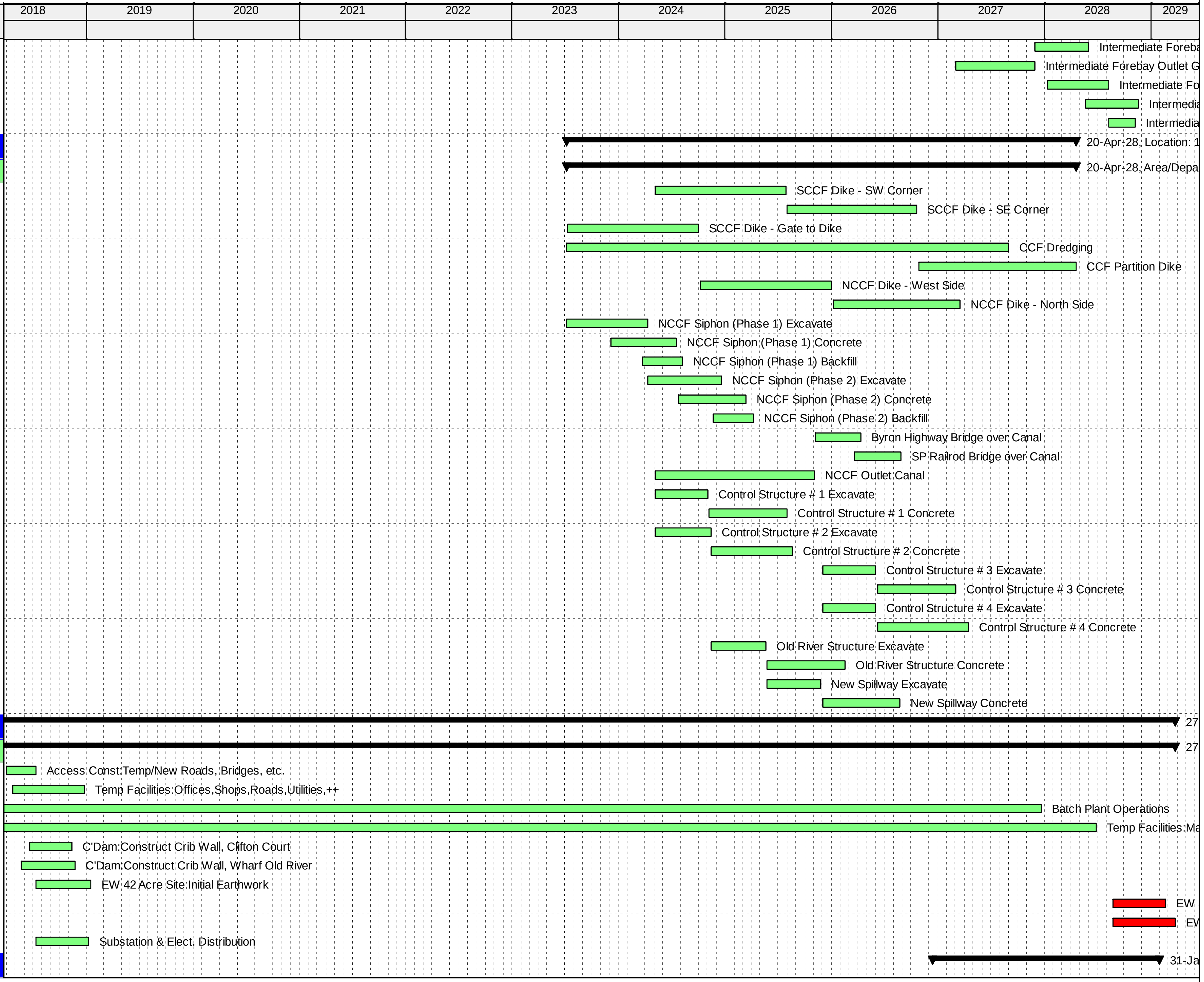
Activity ID	Activity Name
07050	Intermediate Forebay Outlet Sitework
07052	Intermediate Forebay Outlet Ground Improvements
07065	Intermediate Forebay Outlet Concrete
07070	Intermediate Forebay Outlet Gates
07075	Intermediate Forebay Outlet Mech & Elect
Location: 13 CC Forebay Work	
Area/Department: J CC FOREBAY	
08010	SCCF Dike - SW Corner
08012	SCCF Dike - SE Corner
08015	SCCF Dike - Gate to Dike
08025	CCF Dredging
08030	CCF Partition Dike
08040	NCCF Dike - West Side
08045	NCCF Dike - North Side
08071	NCCF Siphon (Phase 1) Excavate
08073	NCCF Siphon (Phase 1) Concrete
08075	NCCF Siphon (Phase 1) Backfill
08077	NCCF Siphon (Phase 2) Excavate
08079	NCCF Siphon (Phase 2) Concrete
08081	NCCF Siphon (Phase 2) Backfill
08090	Byron Highway Bridge over Canal
08092	SP Railrod Bridge over Canal
08096	NCCF Outlet Canal
08101	Control Structure # 1 Excavate
08103	Control Structure # 1 Concrete
08111	Control Structure # 2 Excavate
08113	Control Structure # 2 Concrete
08121	Control Structure # 3 Excavate
08123	Control Structure # 3 Concrete
08131	Control Structure # 4 Excavate
08133	Control Structure # 4 Concrete
08140	Old River Structure Excavate
08141	Old River Structure Concrete
08145	New Spillway Excavate
08146	New Spillway Concrete
Location: 14 Sitework	
Area/Department: K CC COMBINED PUMP PLANT&SURGE SHAFT	
L51.0010	Access Const:Temp/New Roads, Bridges, etc.
L51.0020	Temp Facilities:Offices,Shops,Roads,Utilities,++
L51.0040	Batch Plant Operations
L51.0051	Temp Facilities:Maintenance & Operations
L51.0060	C'Dam:Construct Crib Wall, Clifton Court
L51.0070	C'Dam:Construct Crib Wall, Wharf Old River
L51.0090	EW 42 Acre Site:Initial Earthwork
L51.0100	EW 42 Acre Site:Fill el -2 to +31.5,RipRap
L51.0110	EW 42 Acre Site:Final Site Work
L51.0550	Substation & Elect. Distribution
Location: 15 Pump Shafts	



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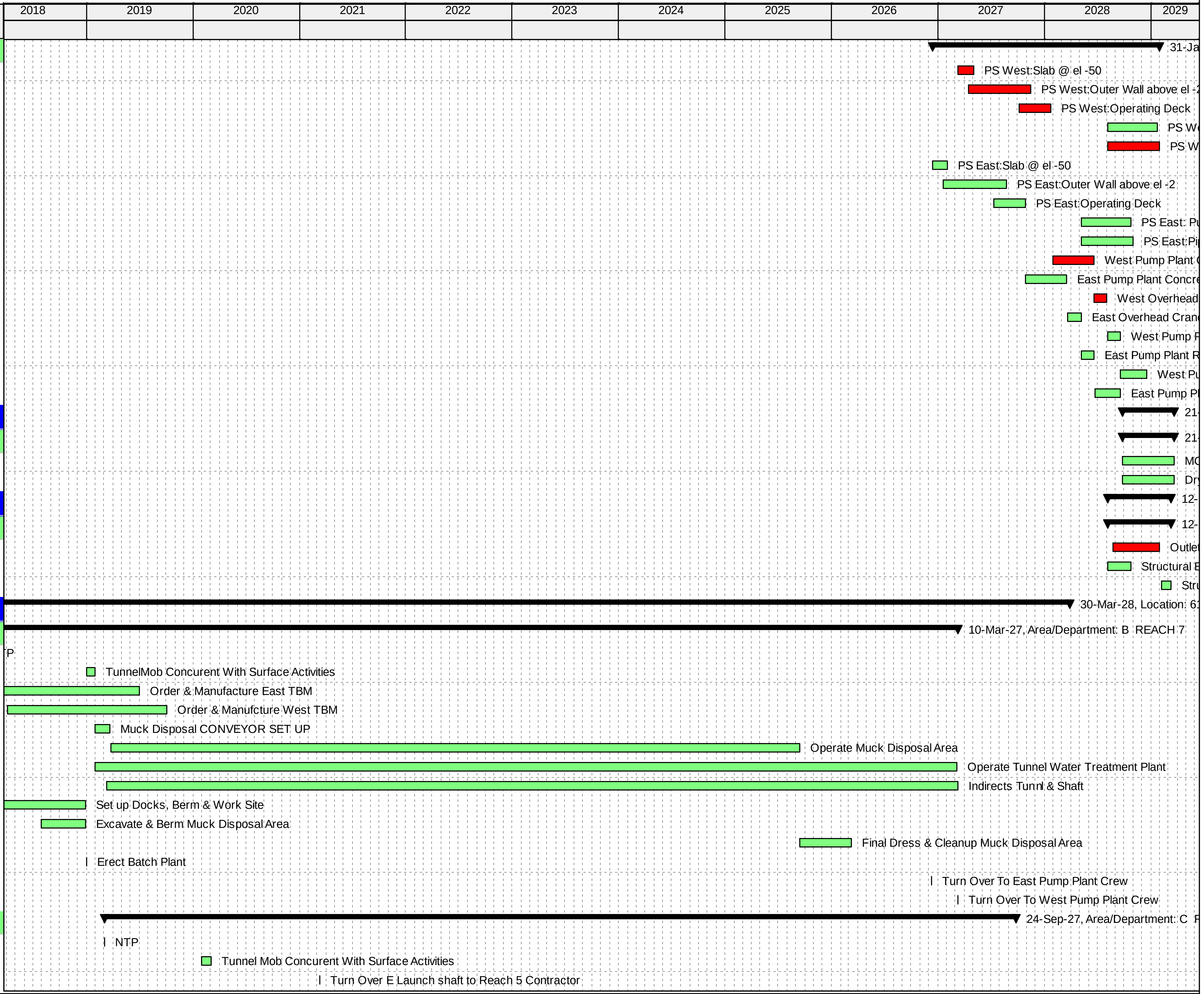
Activity ID	Activity Name
Area/Department: K CC COMBINED PUMP PLANT&SURGE SHAFT	
L51.0300	PS West:Slab @ el -50
L51.0310	PS West:Outer Wall above el -2
L51.0320	PS West:Operating Deck
L51.0340	PS West: Pumps & Operators
L51.0350	PS West:Piping, Pumps to Spillway
L51.0360	PS East:Slab @ el -50
L51.0370	PS East:Outer Wall above el -2
L51.0380	PS East:Operating Deck
L51.0400	PS East: Pumps & Operators
L51.0410	PS East:Piping, Pumps to Spillway
L51.0500	West Pump Plant Concrete Walls
L51.0505	East Pump Plant Concrete Walls
L51.0510	West Overhead Cranes
L51.0515	East Overhead Cranes
L51.0520	West Pump Plant Roof Structure
L51.0525	East Pump Plant Roof Structure
L51.0530	West Pump Plant Finish Out
L51.0535	East Pump Plant Finish Out
Location: 17 Ancilliary Structures	
Area/Department: K CC COMBINED PUMP PLANT&SURGE SHAFT	
L51.0540	MCC/Electrical Building (2 Bldgs)
L51.0560	Drywell Pump Plant Acess Building
Location: 18 Pump Plant Outlet Structure	
Area/Department: K CC COMBINED PUMP PLANT&SURGE SHAFT	
L51.0450	Outlet Structures (for both pump plants)
L51.0460	Structural Excavate
L51.0470	Structural Backfill
Location: 61 General	
Area/Department: B REACH 7	
B1000	NTP
B1010	TunnelMob Concurent With Surface Activities
B1170	Order & Manufacture East TBM
B1180	Order & Manufcture West TBM
B1390	Muck Disposal CONVEYOR SET UP
B2010	Operate Muck Disposal Area
B2030	Operate Tunnel Water Treatment Plant
B2040	Indirects Tunnl & Shaft
B2050	Set up Docks, Berm & Work Site
B2060	Excavate & Berm Muck Disposal Area
B2070	Final Dress & Cleanup Muck Disposal Area
B2080	Erect Batch Plant
B2100	Turn Over To East Pump Plant Crew
B2120	Turn Over To West Pump Plant Crew
Area/Department: C REACH 6	
C1000	NTP
C1010	Tunnel Mob Concurent With Surface Activities
C1370	Turn Over E Launch shaft to Reach 5 Contractor



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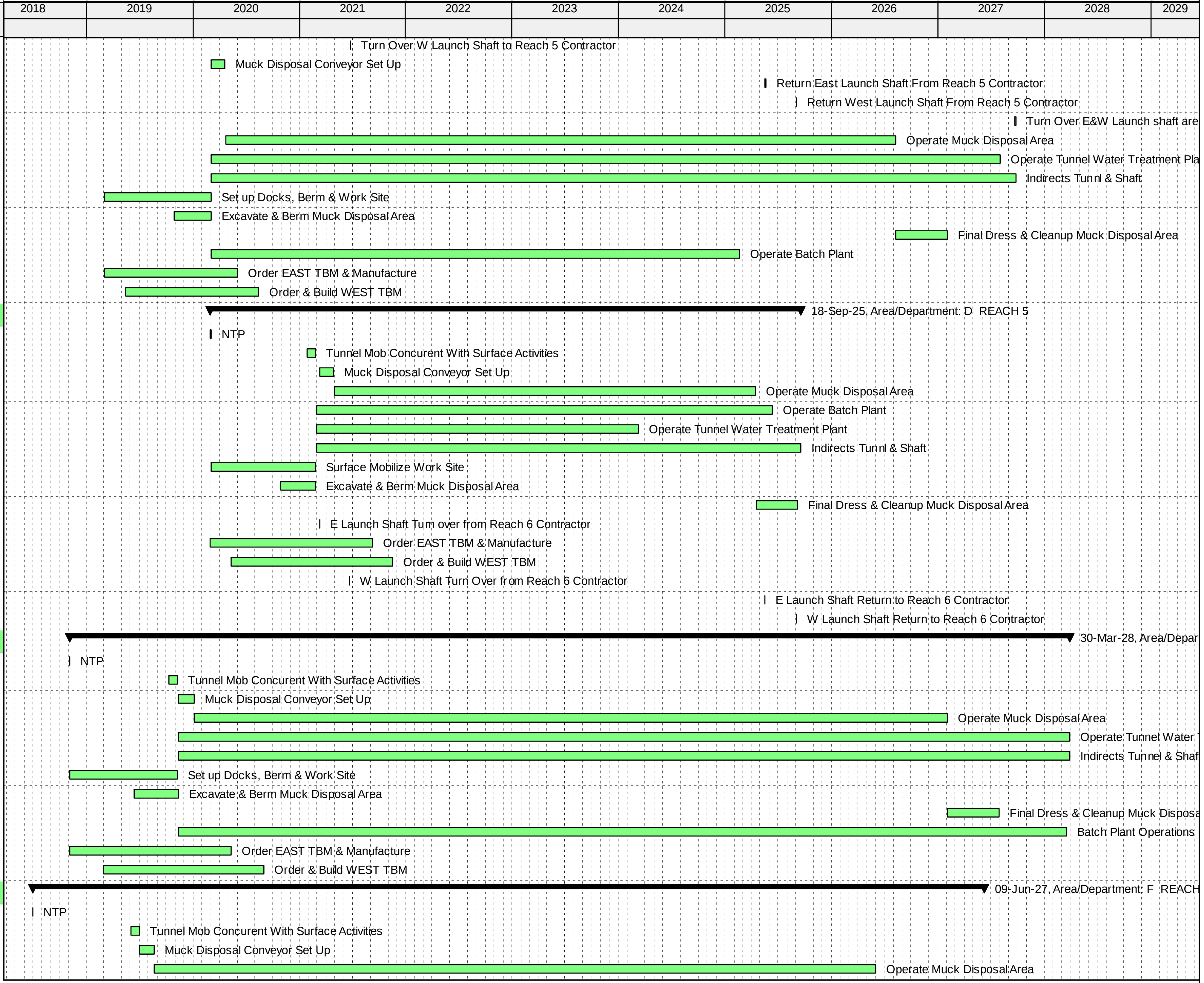
Activity ID	Activity Name
C1380	Turn Over W Launch Shaft to Reach 5 Contractor
C1390	Muck Disposal Conveyor Set Up
C1420	Return East Launch Shaft From Reach 5 Contractor
C1430	Return West Launch Shaft From Reach 5 Contractor
C1440	Turn Over E&W Launch shaft area to Surface Crews
C2010	Operate Muck Disposal Area
C2030	Operate Tunnel Water Treatment Plant
C2040	Indirects Tunnl & Shaft
C2050	Set up Docks, Berm & Work Site
C2060	Excavate & Berm Muck Disposal Area
C2070	Final Dress & Cleanup Muck Disposal Area
C2080	Operate Batch Plant
C2090	Order EAST TBM & Manufacture
C2100	Order & Build WEST TBM
Area/Department: D REACH 5	
D1000	NTP
D1010	Tunnel Mob Concurent With Surface Activities
D1390	Muck Disposal Conveyor Set Up
D2010	Operate Muck Disposal Area
D2020	Operate Batch Plant
D2030	Operate Tunnel Water Treatment Plant
D2040	Indirects Tunnl & Shaft
D2050	Surface Mobilize Work Site
D2060	Excavate & Berm Muck Disposal Area
D2070	Final Dress & Cleanup Muck Disposal Area
D2080	E Launch Shaft Turn over from Reach 6 Contractor
D2090	Order EAST TBM & Manufacture
D2100	Order & Build WEST TBM
D2120	W Launch Shaft Turn Over from Reach 6 Contractor
D2130	E Launch Shaft Return to Reach 6 Contractor
D2140	W Launch Shaft Return to Reach 6 Contractor
Area/Department: E REACH 4	
E1000	NTP
E1010	Tunnel Mob Concurent With Surface Activities
E1390	Muck Disposal Conveyor Set Up
E2010	Operate Muck Disposal Area
E2030	Operate Tunnel Water Treatment Plant
E2040	Indirects Tunnel & Shaft
E2050	Set up Docks, Berm & Work Site
E2060	Excavate & Berm Muck Disposal Area
E2070	Final Dress & Cleanup Muck Disposal Area
E2080	Batch Plant Operations
E2090	Order EAST TBM & Manufacture
E2100	Order & Build WEST TBM
Area/Department: F REACHES 1 2 3	
F1000	NTP
F1010	Tunnel Mob Concurent With Surface Activities
F1390	Muck Disposal Conveyor Set Up
F2010	Operate Muck Disposal Area



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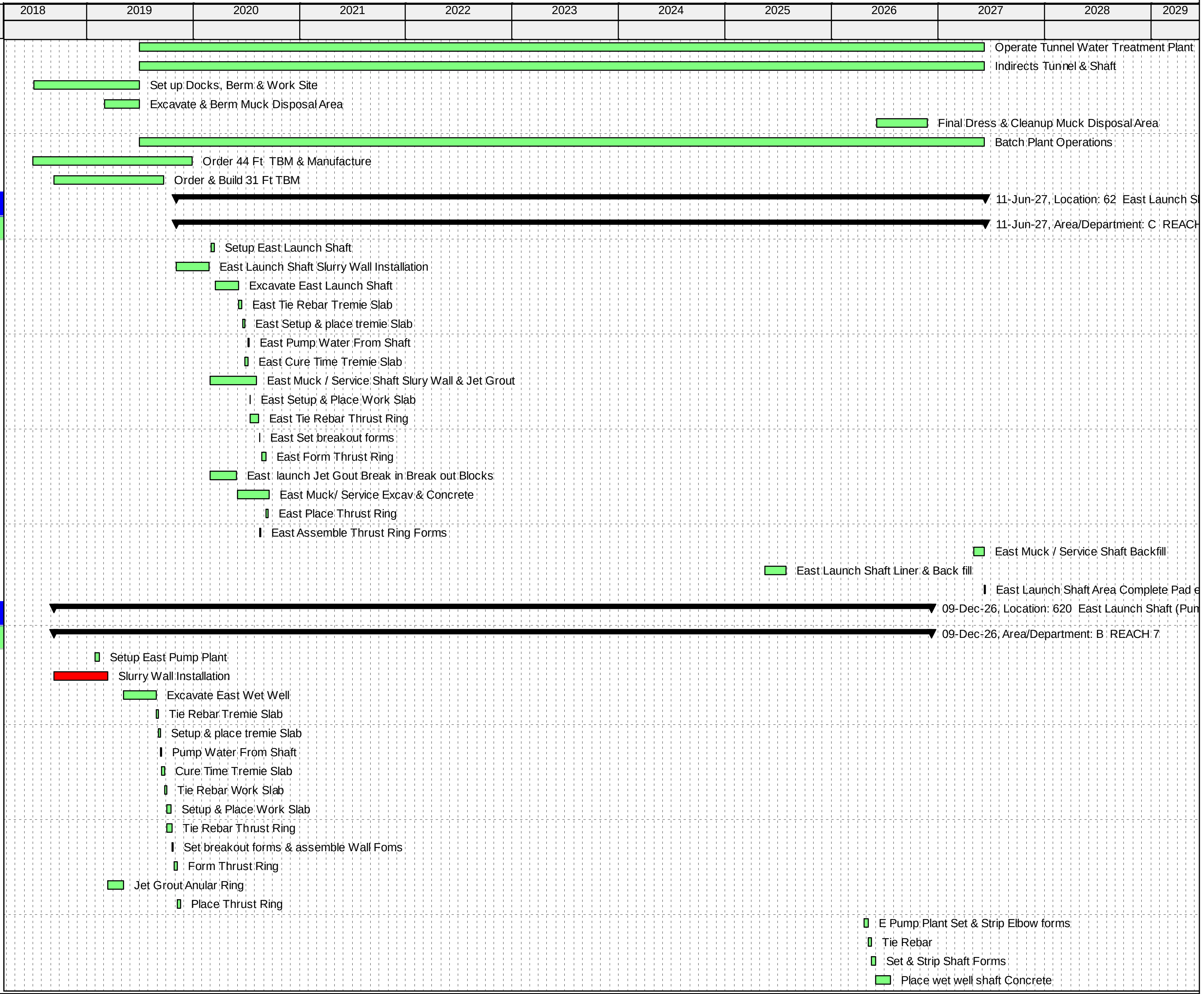
Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
F2030	Operate Tunnel Water Treatment Plant
F2040	Indirects Tunnel & Shaft
F2050	Set up Docks, Berm & Work Site
F2060	Excavate & Berm Muck Disposal Area
F2070	Final Dress & Cleanup Muck Disposal Area
F2080	Batch Plant Operations
F2090	Order 44 Ft TBM & Manufacture
F2100	Order & Build 31 Ft TBM
Location: 62 East Launch Shaft	
Area/Department: C REACH 6	
C1020	Setup East Launch Shaft
C1030	East Launch Shaft Slurry Wall Installation
C1040	Excavate East Launch Shaft
C1050	East Tie Rebar Tremie Slab
C1060	East Setup & place tremie Slab
C1070	East Pump Water From Shaft
C1080	East Cure Time Tremie Slab
C1090	East Muck / Service Shaft Slury Wall & Jet Grout
C1100	East Setup & Place Work Slab
C1110	East Tie Rebar Thrust Ring
C1120	East Set breakout forms
C1130	East Form Thrust Ring
C1140	East launch Jet Gout Break in Break out Blocks
C1150	East Muck/ Service Excav & Concrete
C1160	East Place Thrust Ring
C1310	East Assemble Thrust Ring Forms
C1340	East Muck / Service Shaft Backfill
C1400	East Launch Shaft Liner & Back fill
C1450	East Launch Shaft Area Complete Pad etc
Location: 620 East Launch Shaft (Pump/Surge Shaft)	
Area/Department: B REACH 7	
B1020	Setup East Pump Plant
B1030	Slurry Wall Installation
B1040	Excavate East Wet Well
B1050	Tie Rebar Tremie Slab
B1060	Setup & place tremie Slab
B1070	Pump Water From Shaft
B1080	Cure Time Tremie Slab
B1090	Tie Rebar Work Slab
B1100	Setup & Place Work Slab
B1110	Tie Rebar Thrust Ring
B1120	Set breakout forms & assemble Wall Foms
B1130	Form Thrust Ring
B1140	Jet Grout Anular Ring
B1160	Place Thrust Ring
B1290	E Pump Plant Set & Strip Elbow forms
B1300	Tie Rebar
B1320	Set & Strip Shaft Forms
B1330	Place wet well shaft Concrete



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Clifton Court Option

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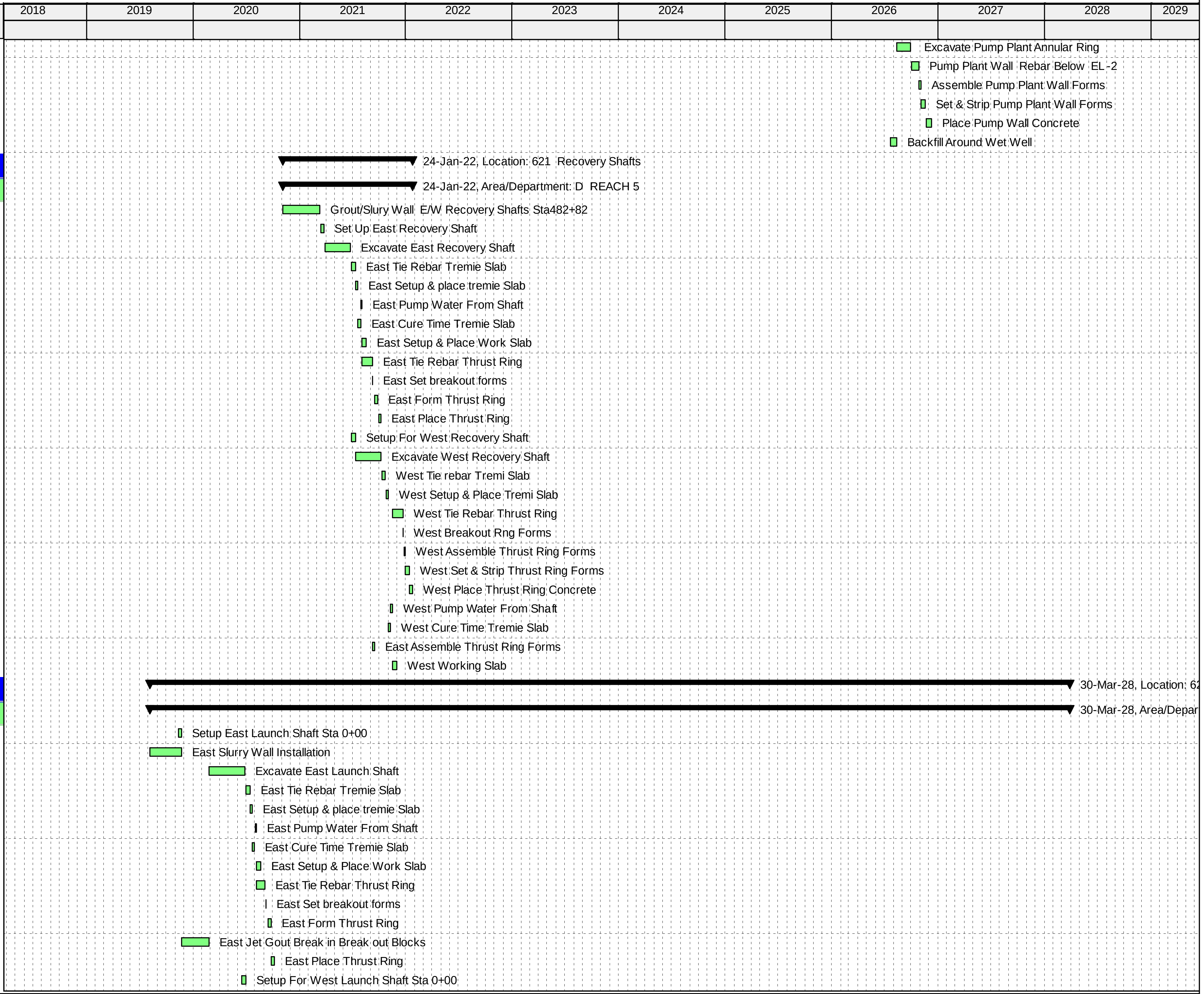
Activity ID	Activity Name
B1340	Excavate Pump Plant Annular Ring
B1350	Pump Plant Wall Rebar Below EL -2
B1360	Assemble Pump Plant Wall Forms
B1370	Set & Strip Pump Plant Wall Forms
B1380	Place Pump Wall Concrete
B1480	Backfill Around Wet Well
Location: 621 Recovery Shafts	
Area/Department: D REACH 5	
D1015	Grout/Slury Wall E/W Recovery Shafts Sta482+82
D1030	Set Up East Recovery Shaft
D1040	Excavate East Recovery Shaft
D1050	East Tie Rebar Tremie Slab
D1060	East Setup & place tremie Slab
D1070	East Pump Water From Shaft
D1080	East Cure Time Tremie Slab
D1100	East Setup & Place Work Slab
D1110	East Tie Rebar Thrust Ring
D1120	East Set breakout forms
D1130	East Form Thrust Ring
D1160	East Place Thrust Ring
D1170	Setup For West Recovery Shaft
D1190	Excavate West Recovery Shaft
D1210	West Tie rebar Tremi Slab
D1220	West Setup & Place Tremi Slab
D1230	West Tie Rebar Thrust Ring
D1240	West Breakout Rng Forms
D1250	West Assemble Thrust Ring Forms
D1260	West Set & Strip Thrust Ring Forms
D1270	West Place Thrust Ring Concrete
D1290	West Pump Water From Shaft
D1300	West Cure Time Tremie Slab
D1310	East Assemble Thrust Ring Forms
D1320	West Working Slab
Location: 622 East & West Launch Shafts	
Area/Department: E REACH 4	
E1020	Setup East Launch Shaft Sta 0+00
E1030	East Slurry Wall Installation
E1040	Excavate East Launch Shaft
E1050	East Tie Rebar Tremie Slab
E1060	East Setup & place tremie Slab
E1070	East Pump Water From Shaft
E1080	East Cure Time Tremie Slab
E1100	East Setup & Place Work Slab
E1110	East Tie Rebar Thrust Ring
E1120	East Set breakout forms
E1130	East Form Thrust Ring
E1140	East Jet Gout Break in Break out Blocks
E1160	East Place Thrust Ring
E1170	Setup For West Launch Shaft Sta 0+00



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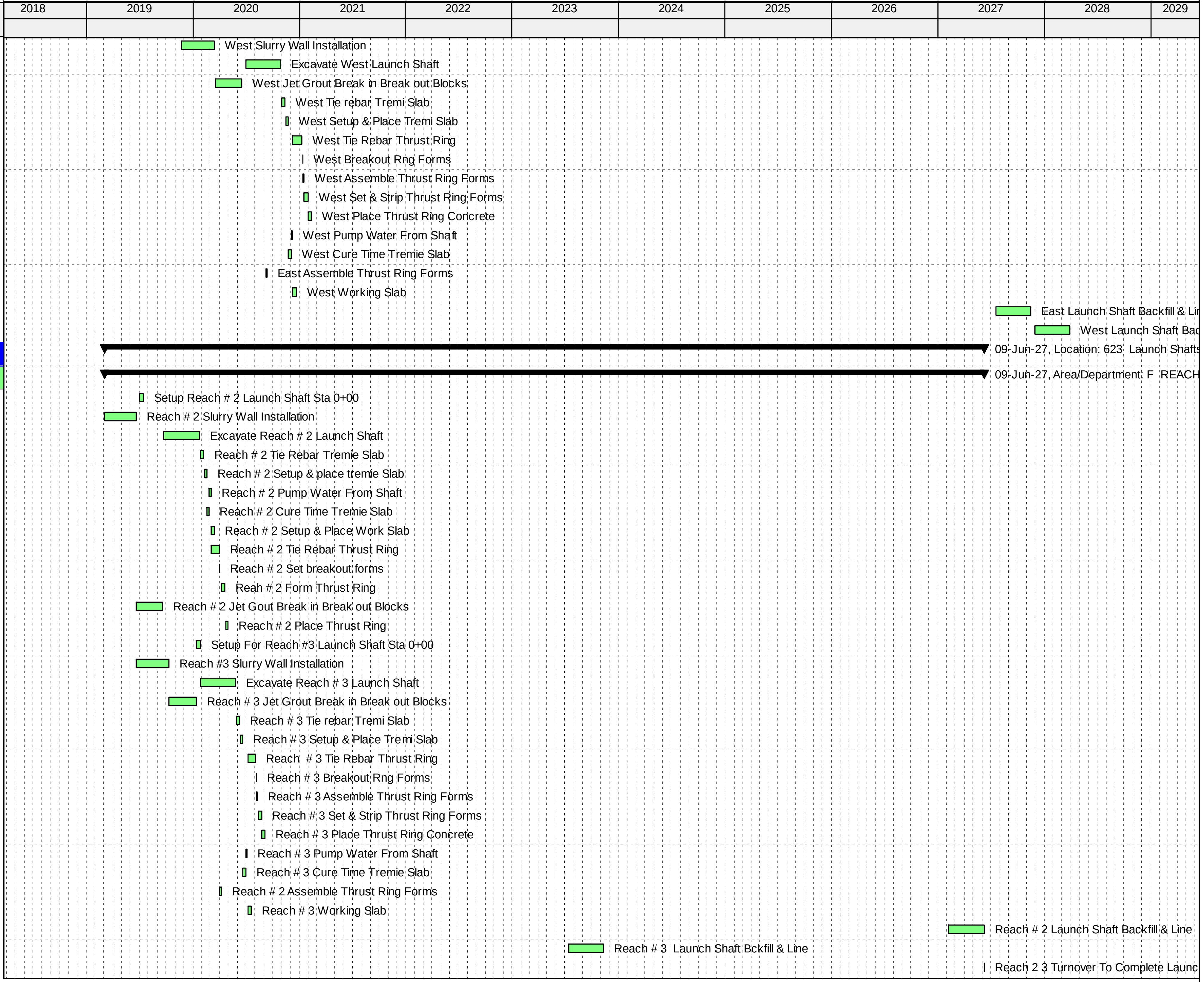
Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
E1180	West Slurry Wall Installation
E1190	Excavate West Launch Shaft
E1200	West Jet Grout Break in Break out Blocks
E1210	West Tie rebar Tremi Slab
E1220	West Setup & Place Tremi Slab
E1230	West Tie Rebar Thrust Ring
E1240	West Breakout Rng Forms
E1250	West Assemble Thrust Ring Forms
E1260	West Set & Strip Thrust Ring Forms
E1270	West Place Thrust Ring Concrete
E1290	West Pump Water From Shaft
E1300	West Cure Time Tremie Slab
E1310	East Assemble Thrust Ring Forms
E1320	West Working Slab
E1330	East Launch Shaft Backfill & Line
E1340	West Launch Shaft Backfill & Line
Location: 623 Launch Shafts	
Area/Department: F REACHES 1 2 3	
F1020	Setup Reach # 2 Launch Shaft Sta 0+00
F1030	Reach # 2 Slurry Wall Installation
F1040	Excavate Reach # 2 Launch Shaft
F1050	Reach # 2 Tie Rebar Tremie Slab
F1060	Reach # 2 Setup & place tremie Slab
F1070	Reach # 2 Pump Water From Shaft
F1080	Reach # 2 Cure Time Tremie Slab
F1100	Reach # 2 Setup & Place Work Slab
F1110	Reach # 2 Tie Rebar Thrust Ring
F1120	Reach # 2 Set breakout forms
F1130	Reah # 2 Form Thrust Ring
F1140	Reach # 2 Jet Gout Break in Break out Blocks
F1160	Reach # 2 Place Thrust Ring
F1170	Setup For Reach #3 Launch Shaft Sta 0+00
F1180	Reach #3 Slurry Wall Installation
F1190	Excavate Reach # 3 Launch Shaft
F1200	Reach # 3 Jet Grout Break in Break out Blocks
F1210	Reach # 3 Tie rebar Tremi Slab
F1220	Reach # 3 Setup & Place Tremi Slab
F1230	Reach # 3 Tie Rebar Thrust Ring
F1240	Reach # 3 Breakout Rng Forms
F1250	Reach # 3 Assemble Thrust Ring Forms
F1260	Reach # 3 Set & Strip Thrust Ring Forms
F1270	Reach # 3 Place Thrust Ring Concrete
F1290	Reach # 3 Pump Water From Shaft
F1300	Reach # 3 Cure Time Tremie Slab
F1310	Reach # 2 Assemble Thrust Ring Forms
F1320	Reach # 3 Working Slab
F1330	Reach # 2 Launch Shaft Backfill & Line
F1340	Reach # 3 Launch Shaft Bckfill & Line
F1350	Reach 2 3 Turnover To Complete Launch Shaft Area



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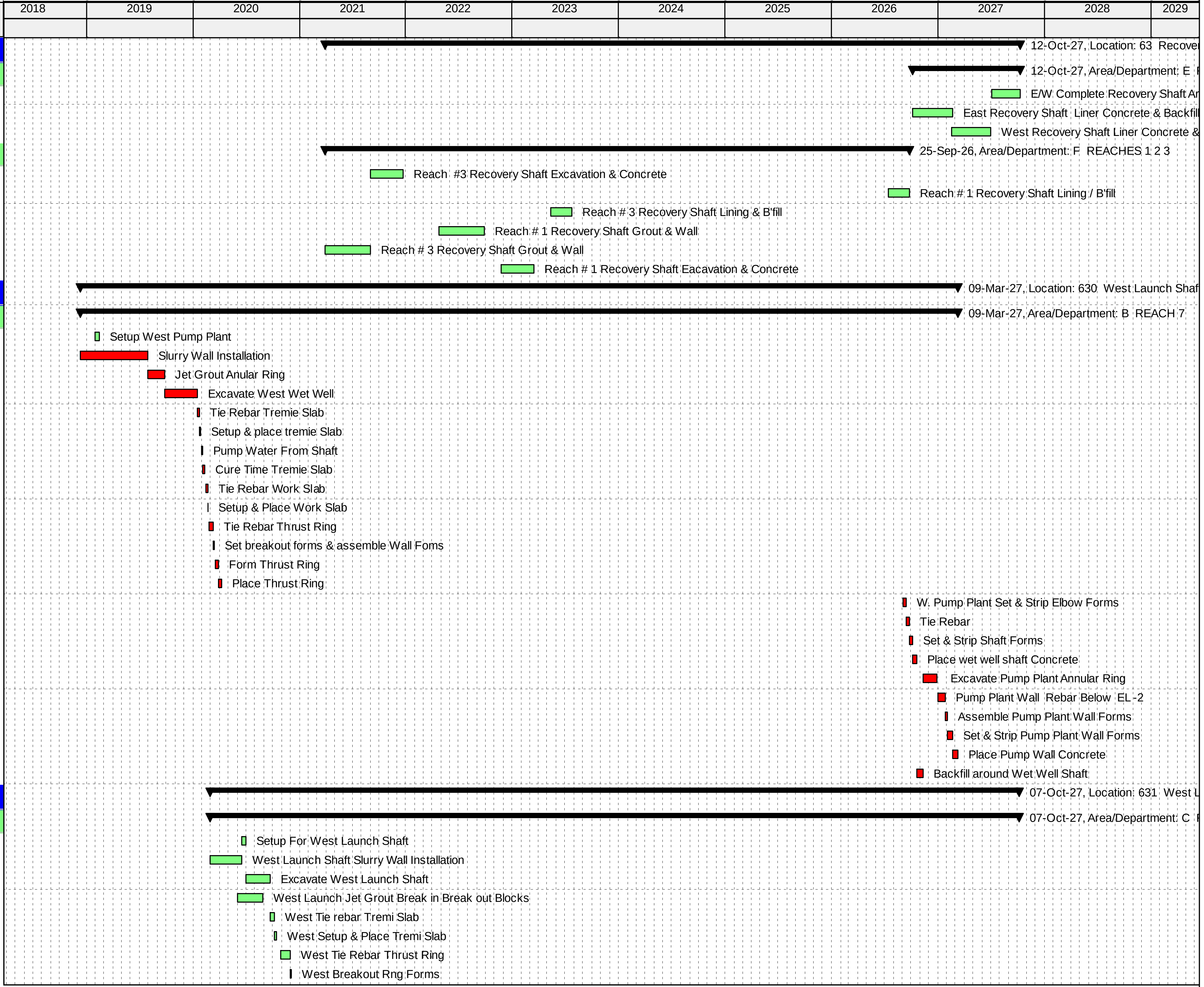
Activity ID	Activity Name
Location: 63 Recovery Shafts	
Area/Department: E REACH 4	
E2313	E/W Complete Recovery Shaft Area
E2333	East Recovery Shaft Liner Concrete & Backfill
E2353	West Recovery Shaft Liner Concrete & Backfill
Area/Department: F REACHES 1 2 3	
F2363	Reach #3 Recovery Shaft Excavation & Concrete
F2373	Reach # 1 Recovery Shaft Lining / B'fill
F2393	Reach # 3 Recovery Shaft Lining & B'fill
F2453	Reach # 1 Recovery Shaft Grout & Wall
F2463	Reach # 3 Recovery Shaft Grout & Wall
F2473	Reach # 1 Recovery Shaft Eacavation & Concrete
Location: 630 West Launch Shaft (Pump/Surge Shafts)	
Area/Department: B REACH 7	
B1510	Setup West Pump Plant
B1520	Slurry Wall Installation
B1530	Jet Grout Anular Ring
B1540	Excavate West Wet Well
B1550	Tie Rebar Tremie Slab
B1560	Setup & place tremie Slab
B1570	Pump Water From Shaft
B1580	Cure Time Tremie Slab
B1590	Tie Rebar Work Slab
B1600	Setup & Place Work Slab
B1610	Tie Rebar Thrust Ring
B1620	Set breakout forms & assemble Wall Foms
B1630	Form Thrust Ring
B1640	Place Thrust Ring
B1660	W. Pump Plant Set & Strip Elbow Forms
B1670	Tie Rebar
B1690	Set & Strip Shaft Forms
B1700	Place wet well shaft Concrete
B1710	Excavate Pump Plant Annular Ring
B1720	Pump Plant Wall Rebar Below EL -2
B1730	Assemble Pump Plant Wall Forms
B1740	Set & Strip Pump Plant Wall Forms
B1750	Place Pump Wall Concrete
B2090	Backfill around Wet Well Shaft
Location: 631 West Launch Shaft	
Area/Department: C REACH 6	
C1170	Setup For West Launch Shaft
C1180	West Launch Shaft Slurry Wall Installation
C1190	Excavate West Launch Shaft
C1200	West Launch Jet Grout Break in Break out Blocks
C1210	West Tie rebar Tremi Slab
C1220	West Setup & Place Tremi Slab
C1230	West Tie Rebar Thrust Ring
C1240	West Breakout Rng Forms



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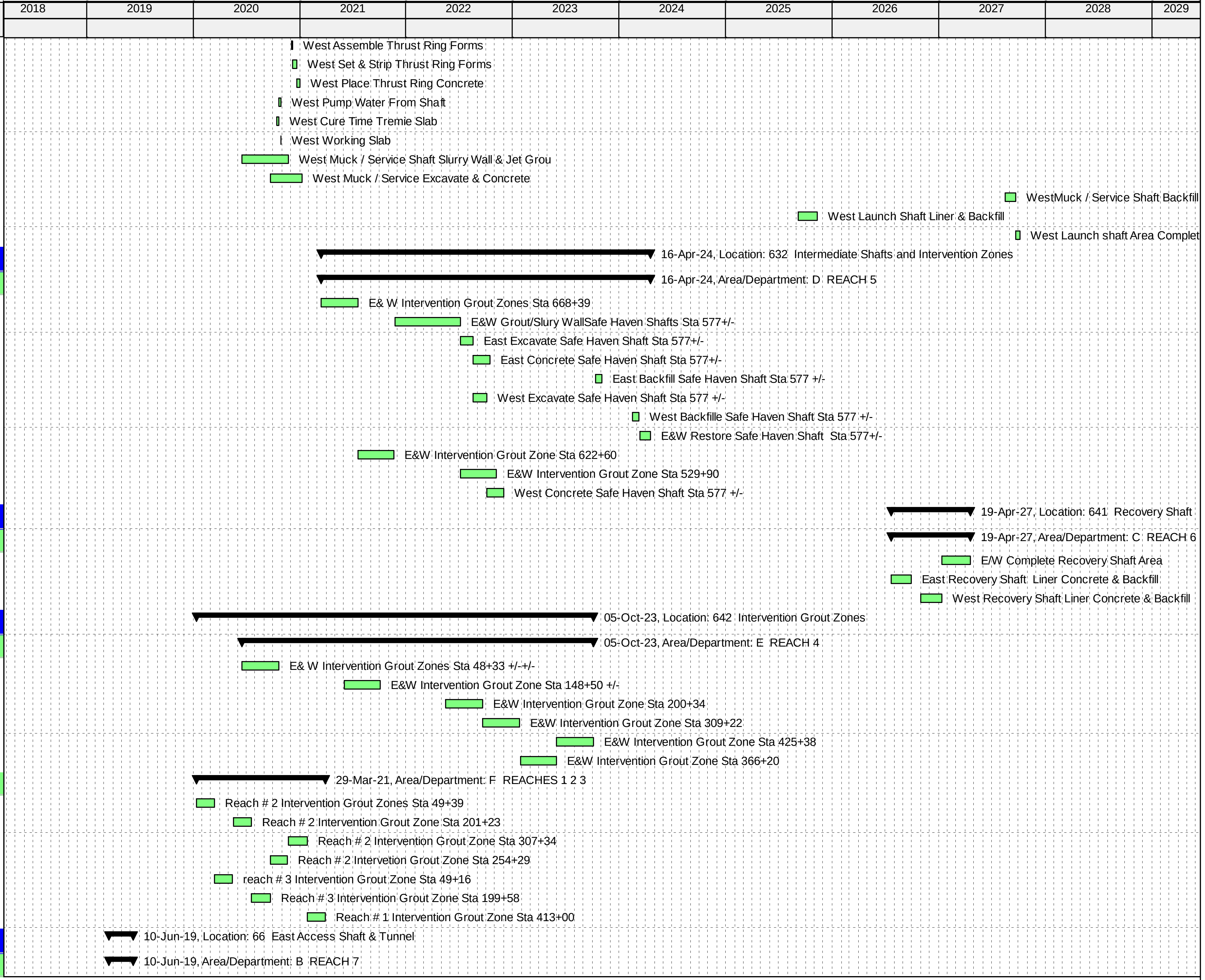
Activity ID	Activity Name
C1250	West Assemble Thrust Ring Forms
C1260	West Set & Strip Thrust Ring Forms
C1270	West Place Thrust Ring Concrete
C1290	West Pump Water From Shaft
C1300	West Cure Time Tremie Slab
C1320	West Working Slab
C1330	West Muck / Service Shaft Slurry Wall & Jet Grou
C1350	West Muck / Service Excavate & Concrete
C1360	WestMuck / Service Shaft Backfill
C1410	West Launch Shaft Liner & Backfill
C1460	West Launch shaft Area Complete Pad Etc.
Location: 632 Intermediate Shafts and Intervention Zones	
Area/Department: D REACH 5	
D2033	E& W Intervention Grout Zones Sta 668+39
D2043	E&W Grout/Slury WallSafe Haven Shafts Sta 577+/-
D2053	East Excavate Safe Haven Shaft Sta 577+/-
D2063	East Concrete Safe Haven Shaft Sta 577+/-
D2073	East Backfill Safe Haven Shaft Sta 577 +/-
D2083	West Excavate Safe Haven Shaft Sta 577 +/-
D2103	West Backfille Safe Haven Shaft Sta 577 +/-
D2113	E&W Restore Safe Haven Shaft Sta 577+/-
D2213	E&W Intervention Grout Zone Sta 622+60
D2223	E&W Intervention Grout Zone Sta 529+90
D2243	West Concrete Safe Haven Shaft Sta 577 +/-
Location: 641 Recovery Shaft	
Area/Department: C REACH 6	
C2313	E/W Complete Recovery Shaft Area
C2333	East Recovery Shaft Liner Concrete & Backfill
C2353	West Recovery Shaft Liner Concrete & Backfill
Location: 642 Intervention Grout Zones	
Area/Department: E REACH 4	
E2033	E& W Intervention Grout Zones Sta 48+33 +/-+/-
E2123	E&W Intervention Grout Zone Sta 148+50 +/-
E2213	E&W Intervention Grout Zone Sta 200+34
E2303	E&W Intervention Grout Zone Sta 309+22
E2323	E&W Intervention Grout Zone Sta 425+38
E2343	E&W Intervention Grout Zone Sta 366+20
Area/Department: F REACHES 1 2 3	
F2033	Reach # 2 Intervention Grout Zones Sta 49+39
F2123	Reach # 2 Intervention Grout Zone Sta 201+23
F2213	Reach # 2 Intervention Grout Zone Sta 307+34
F2233	Reach # 2 Intervetion Grout Zone Sta 254+29
F2303	reach # 3 Intervention Grout Zone Sta 49+16
F2323	Reach # 3 Intervention Grout Zone Sta 199+58
F2343	Reach # 1 Intervention Grout Zone Sta 413+00
Location: 66 East Access Shaft & Tunnel	
Area/Department: B REACH 7	



🟦 Actual Work
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▬ Summary
🟩 Remaining Work
◆ Milestone

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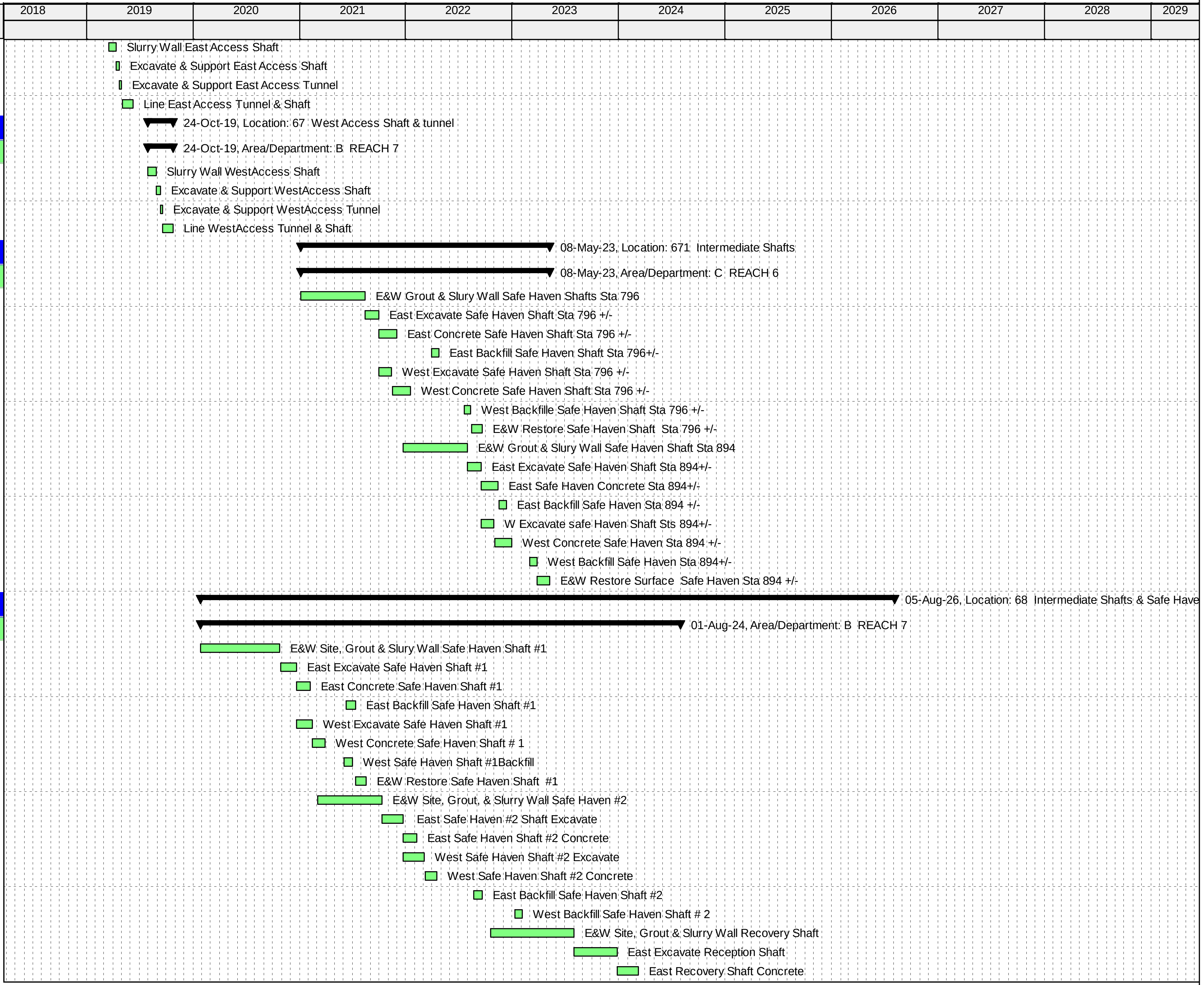
Activity ID	Activity Name
B1970	Slurry Wall East Access Shaft
B1980	Excavate & Support East Access Shaft
B1990	Excavate & Support East Access Tunnel
B2000	Line East Access Tunnel & Shaft
Location: 67 West Access Shaft & tunnel	
Area/Department: B REACH 7	
B2020	Slurry Wall WestAccess Shaft
B2021	Excavate & Support WestAccess Shaft
B2022	Excavate & Support WestAccess Tunnel
B2023	Line WestAccess Tunnel & Shaft
Location: 671 Intermediate Shafts	
Area/Department: C REACH 6	
C2043	E&W Grout & Slury Wall Safe Haven Shafts Sta 796
C2053	East Excavate Safe Haven Shaft Sta 796 +/-
C2063	East Concrete Safe Haven Shaft Sta 796 +/-
C2073	East Backfill Safe Haven Shaft Sta 796+/-
C2083	West Excavate Safe Haven Shaft Sta 796 +/-
C2093	West Concrete Safe Haven Shaft Sta 796 +/-
C2103	West Backfille Safe Haven Shaft Sta 796 +/-
C2113	E&W Restore Safe Haven Shaft Sta 796 +/-
C2133	E&W Grout & Slury Wall Safe Haven Shaft Sta 894
C2143	East Excavate Safe Haven Shaft Sta 894+/-
C2153	East Safe Haven Concrete Sta 894+/-
C2163	East Backfill Safe Haven Sta 894 +/-
C2173	W Excavate safe Haven Shaft Sts 894+/-
C2183	West Concrete Safe Haven Sta 894 +/-
C2193	West Backfill Safe Haven Sta 894+/-
C2203	E&W Restore Surface Safe Haven Sta 894 +/-
Location: 68 Intermediate Shafts & Safe Havens	
Area/Department: B REACH 7	
B2043	E&W Site, Grout & Slury Wall Safe Haven Shaft #1
B2053	East Excavate Safe Haven Shaft #1
B2063	East Concrete Safe Haven Shaft #1
B2073	East Backfill Safe Haven Shaft #1
B2083	West Excavate Safe Haven Shaft #1
B2093	West Concrete Safe Haven Shaft # 1
B2103	West Safe Haven Shaft #1Backfill
B2113	E&W Restore Safe Haven Shaft #1
B2133	E&W Site, Grout, & Slurry Wall Safe Haven #2
B2143	East Safe Haven #2 Shaft Excavate
B2153	East Safe Haven Shaft #2 Concrete
B2173	West Safe Haven Shaft #2 Excavate
B2183	West Safe Haven Shaft #2 Concrete
B2253	East Backfill Safe Haven Shaft #2
B2283	West Backfill Safe Haven Shaft # 2
B2313	E&W Site, Grout & Slurry Wall Recovery Shaft
B2323	East Excavate Reception Shaft
B2333	East Recovery Shaft Concrete



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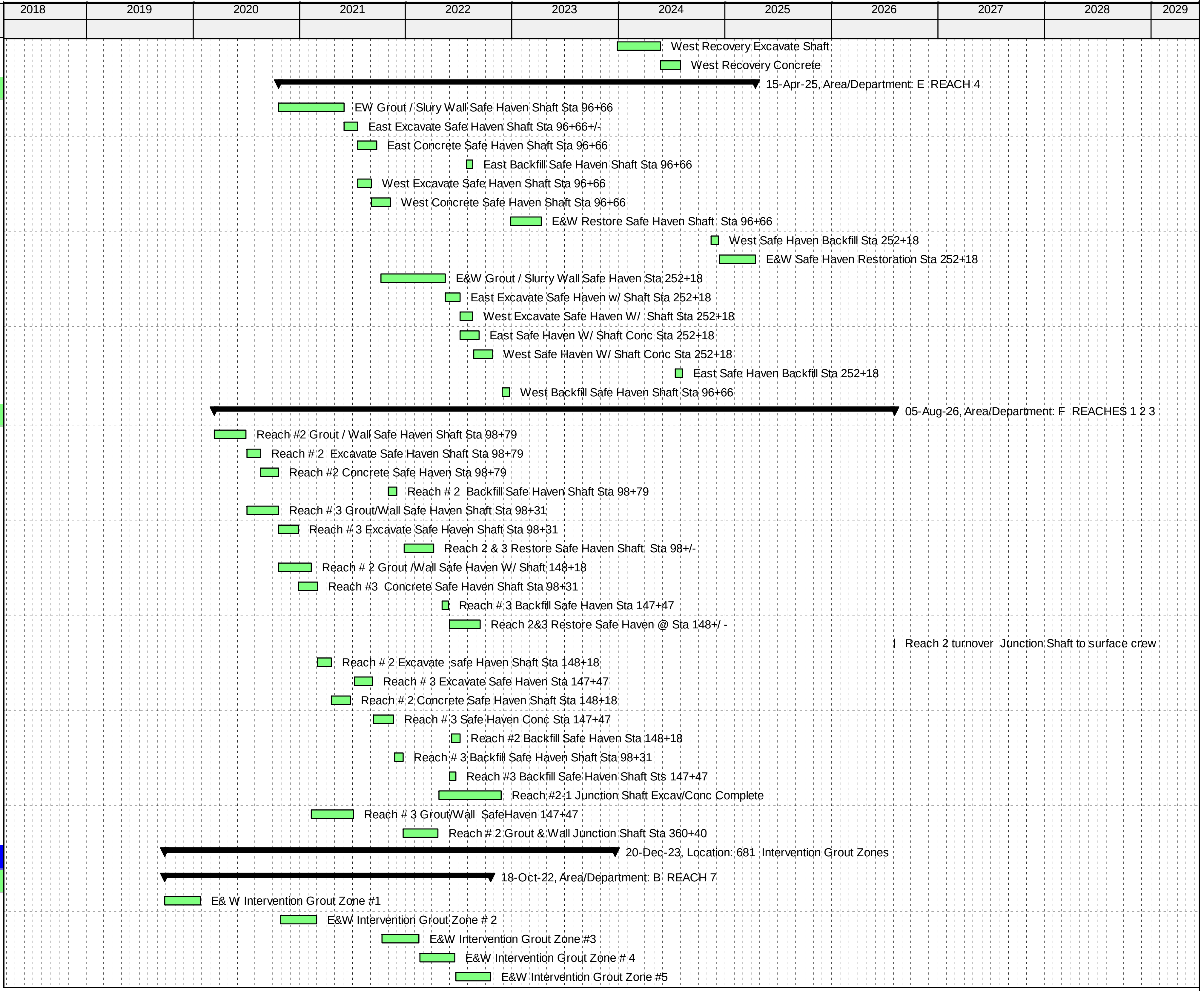
🟥 Critical Remaining Work
Summary
🟩 Remaining Work ◆
◆ Milestone

CALIFORNIA WATER FIX Clifton Court Option Draft Construction Schedule

Activity ID	Activity Name
B2343	West Recovery Excavate Shaft
B2353	West Recovery Concrete
Area/Department: E REACH 4	
E2043	EW Grout / Slury Wall Safe Haven Shaft Sta 96+66
E2053	East Excavate Safe Haven Shaft Sta 96+66+/-
E2063	East Concrete Safe Haven Shaft Sta 96+66
E2073	East Backfill Safe Haven Shaft Sta 96+66
E2083	West Excavate Safe Haven Shaft Sta 96+66
E2093	West Concrete Safe Haven Shaft Sta 96+66
E2113	E&W Restore Safe Haven Shaft Sta 96+66
E2193	West Safe Haven Backfill Sta 252+18
E2203	E&W Safe Haven Restoration Sta 252+18
E2233	E&W Grout / Slurry Wall Safe Haven Sta 252+18
E2243	East Excavate Safe Haven w/ Shaft Sta 252+18
E2253	West Excavate Safe Haven W/ Shaft Sta 252+18
E2263	East Safe Haven W/ Shaft Conc Sta 252+18
E2273	West Safe Haven W/ Shaft Conc Sta 252+18
E2283	East Safe Haven Backfill Sta 252+18
E2293	West Backfill Safe Haven Shaft Sta 96+66
Area/Department: F REACHES 1 2 3	
F2043	Reach #2 Grout / Wall Safe Haven Shaft Sta 98+79
F2053	Reach # 2 Excavate Safe Haven Shaft Sta 98+79
F2063	Reach #2 Concrete Safe Haven Sta 98+79
F2073	Reach # 2 Backfill Safe Haven Shaft Sta 98+79
F2083	Reach # 3 Grout/Wall Safe Haven Shaft Sta 98+31
F2093	Reach # 3 Excavate Safe Haven Shaft Sta 98+31
F2113	Reach 2 & 3 Restore Safe Haven Shaft Sta 98+/-
F2133	Reach # 2 Grout /Wall Safe Haven W/ Shaft 148+18
F2153	Reach #3 Concrete Safe Haven Shaft Sta 98+31
F2193	Reach # 3 Backfill Safe Haven Sta 147+47
F2203	Reach 2&3 Restore Safe Haven @ Sta 148+/ -
F2223	Reach 2 turnover Junction Shaft to surface crew
F2243	Reach # 2 Excavate safe Haven Shaft Sta 148+18
F2253	Reach # 3 Excavate Safe Haven Sta 147+47
F2263	Reach # 2 Concrete Safe Haven Shaft Sta 148+18
F2273	Reach # 3 Safe Haven Conc Sta 147+47
F2283	Reach #2 Backfill Safe Haven Sta 148+18
F2293	Reach # 3 Backfill Safe Haven Shaft Sta 98+31
F2313	Reach #3 Backfill Safe Haven Shaft Sts 147+47
F2353	Reach #2-1 Junction Shaft Excav/Conc Complete
F2403	Reach # 3 Grout/Wall SafeHaven 147+47
F2413	Reach # 2 Grout & Wall Junction Shaft Sta 360+40
Location: 681 Intervention Grout Zones	
Area/Department: B REACH 7	
B2033	E& W Intervention Grout Zone #1
B2123	E&W Intervention Grout Zone # 2
B2213	E&W Intervention Grout Zone #3
B2303	E&W Intervention Grout Zone # 4
B2363	E&W Intervention Grout Zone #5



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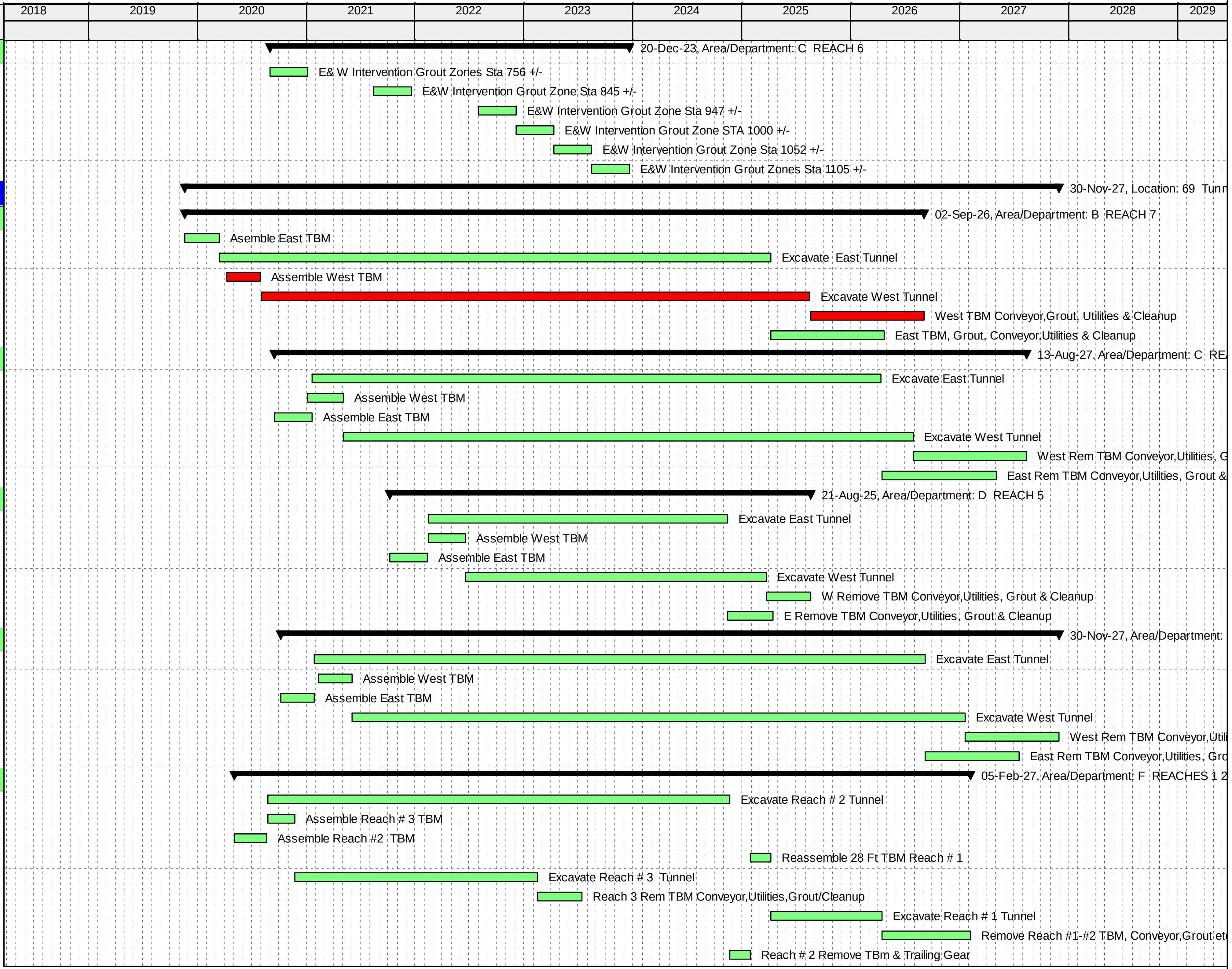
CALIFORNIA WATER FIX

Clifton Court Option

Draft Construction Schedule

Activity ID	Activity Name
Area/Department: C REACH 6	
C2033	E& W Intervention Grout Zones Sta 756 +/-
C2123	E&W Intervention Grout Zone Sta 845 +/-
C2213	E&W Intervention Grout Zone Sta 947 +/-
C2233	E&W Intervention Grout Zone STA 1000 +/-
C2303	E&W Intervention Grout Zone Sta 1052 +/-
C2323	E&W Intervention Grout Zones Sta 1105 +/-
Location: 69 Tunnel Excavate & Support	
Area/Department: B REACH 7	
B1150	Asemble East TBM
B1280	Excavate East Tunnel
B1650	Assemble West TBM
B1880	Excavate West Tunnel
B1890	West TBM Conveyor,Grout, Utilities & Cleanup
B2110	East TBM, Grout, Conveyor,Utilities & Cleanup
Area/Department: C REACH 6	
C1280	Excavate East Tunnel
C1650	Assemble West TBM
C1660	Assemble East TBM
C1880	Excavate West Tunnel
C1890	West Rem TBM Conveyor,Utilities, Grout & Cleanup
C2110	East Rem TBM Conveyor,Utilities, Grout & Cleanup
Area/Department: D REACH 5	
D1280	Excavate East Tunnel
D1650	Assemble West TBM
D1660	Assemble East TBM
D1880	Excavate West Tunnel
D1890	W Remove TBM Conveyor,Utilities, Grout & Cleanup
D2110	E Remove TBM Conveyor,Utilities, Grout & Cleanup
Area/Department: E REACH 4	
E1280	Excavate East Tunnel
E1650	Assemble West TBM
E1660	Assemble East TBM
E1880	Excavate West Tunnel
E1890	West Rem TBM Conveyor,Utilities, Grout & Cleanup
E2110	East Rem TBM Conveyor,Utilities, Grout & Cleanup
Area/Department: F REACHES 1 2 3	
F1280	Excavate Reach # 2 Tunnel
F1650	Assemble Reach # 3 TBM
F1660	Assemble Reach #2 TBM
F1670	Reassemble 28 Ft TBM Reach # 1
F1880	Excavate Reach # 3 Tunnel
F1890	Reach 3 Rem TBM Conveyor,Utilities,Grout/Cleanup
F1900	Excavate Reach # 1 Tunnel
F1910	Remove Reach #1-#2 TBM, Conveyor,Grout etc
F2110	Reach # 2 Remove TBm & Trailing Gear



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Appendix D
AECOM Surge Analysis Technical Memorandum

WORKING DRAFT (rev 0)

Technical Memorandum No. 3

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	December 3, 2014
Project Number	60322033

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

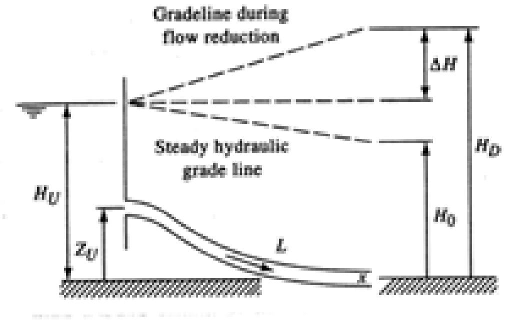
Pressure transients, also often referred to as surge, or water hammer, can occur whenever flow conditions change within a water transmission system. Examples of such conditions include a sudden flow change resulting from rapid closure of a valve or from loss of power to pumps. For the vast majority of 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.

$$b = 0.25\, g\, \pi\, \left(\frac{d}{a}\right)^2 \qquad \textbf{Eqn. 1}$$

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.

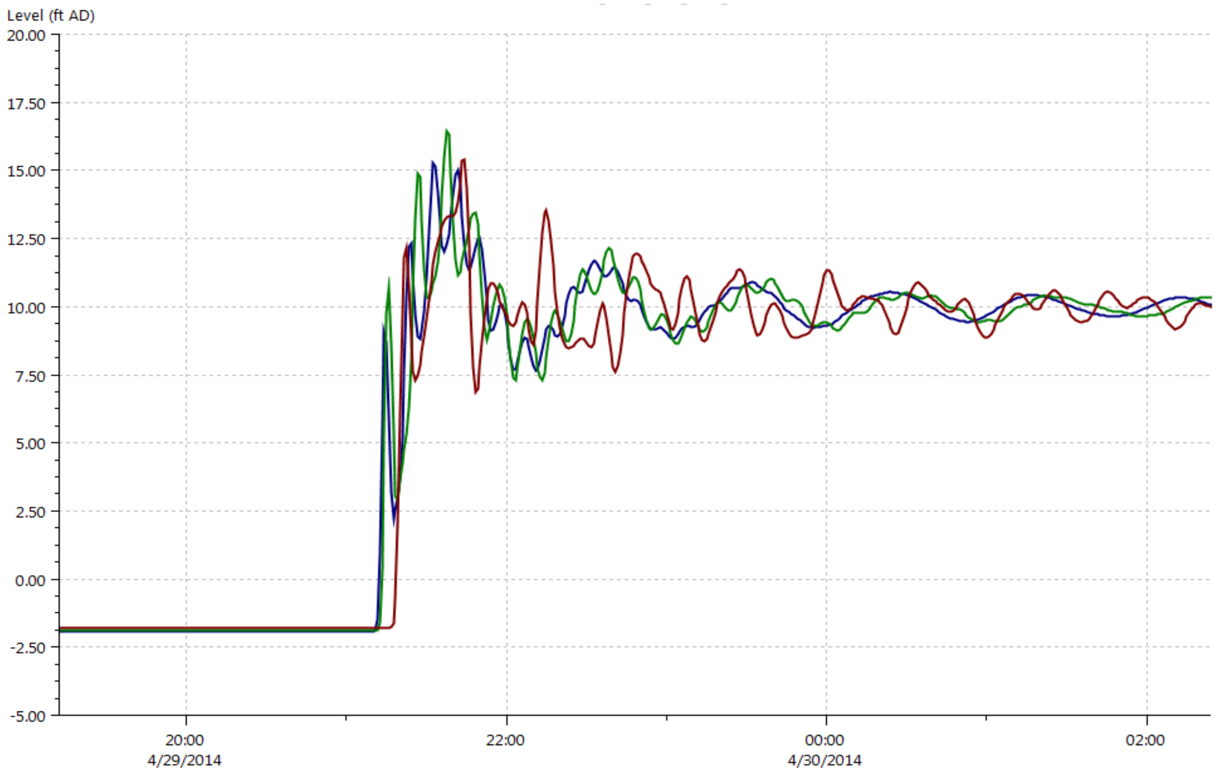


Figure 1 – HGL (ft) at the Staten Island Vent Shaft (located downstream of the IF)

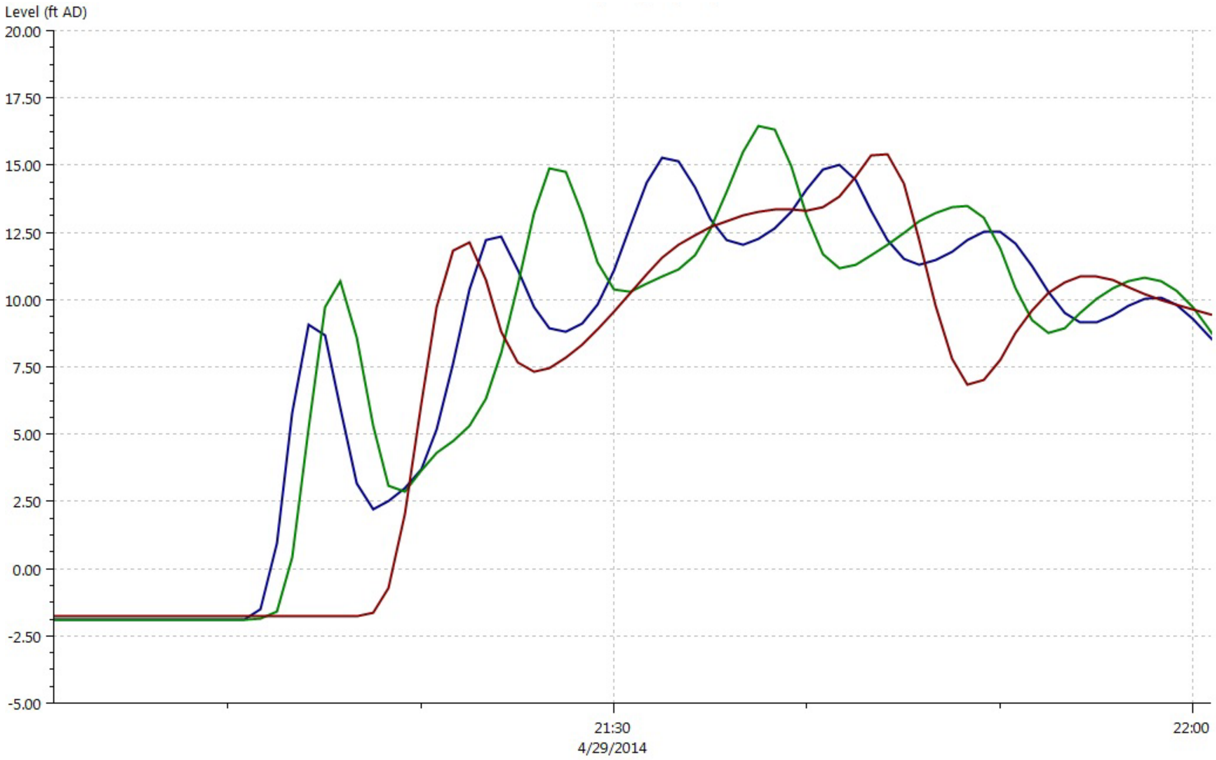


Figure 2 – Same as Figure 1, enlarged to show the first 40 minutes

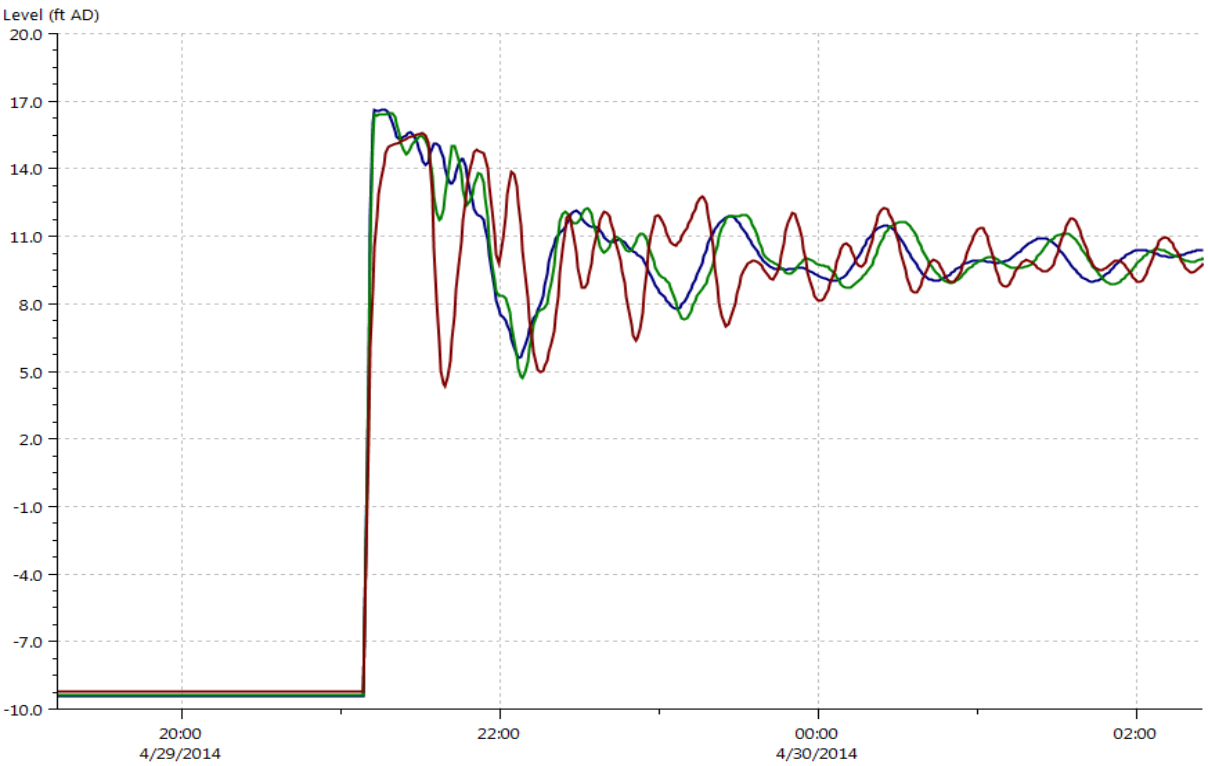


Figure 3 – HGL (ft) at the CC Surge Shaft

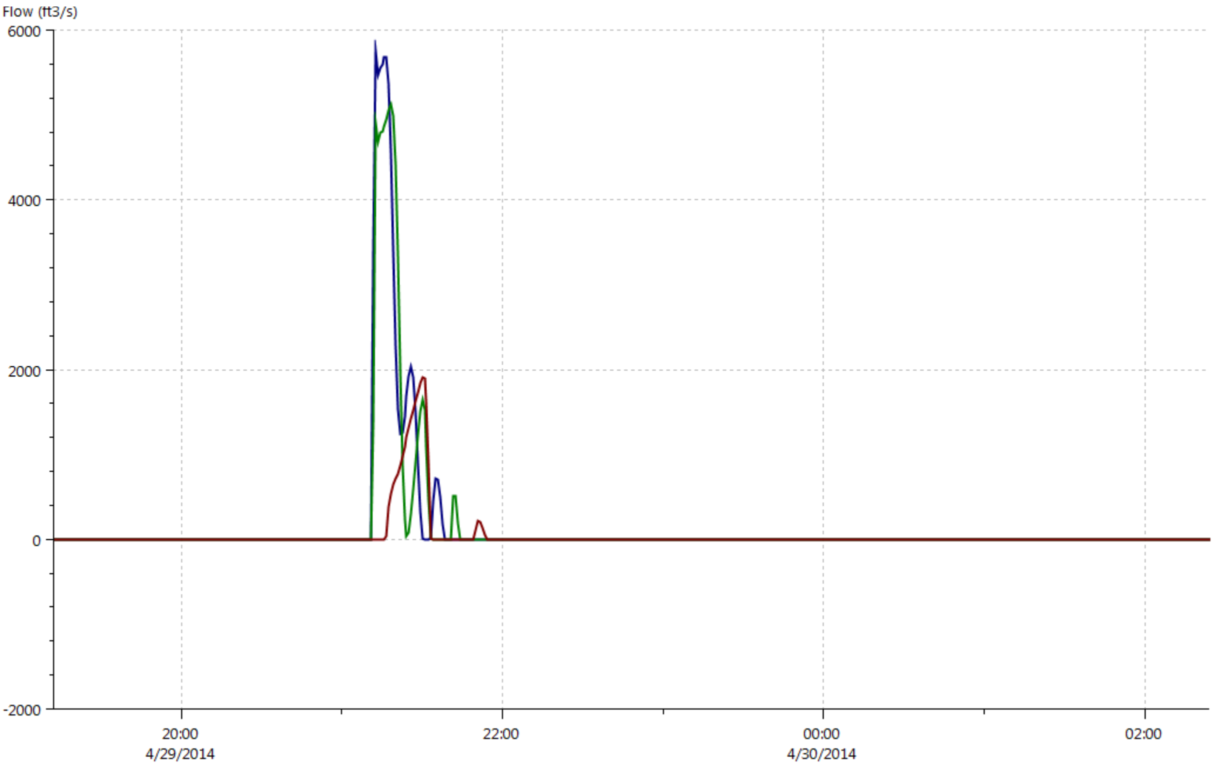


Figure 4 – Flow (cfs) in the Surge Bypass Channel

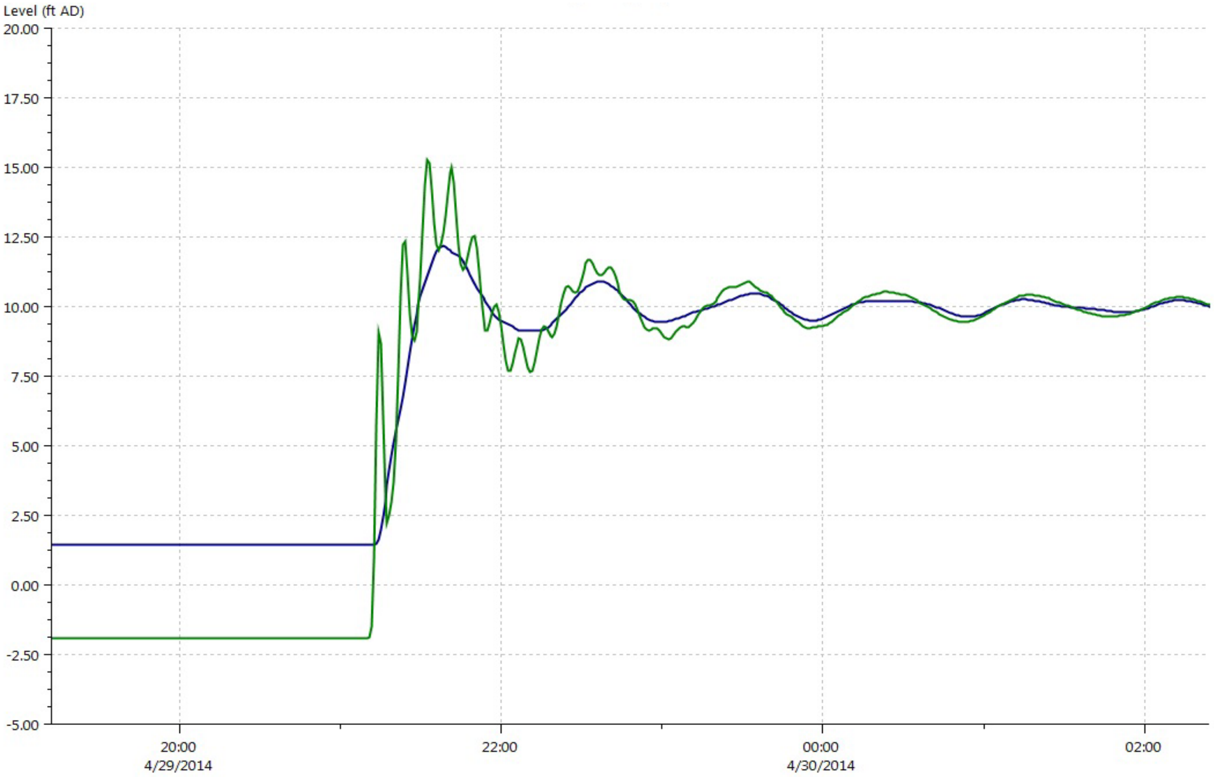


Figure 5 – HGL (ft) Upstream and Downstream of the IF; CELRAT = 76 Case Only

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 as 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

Protection Agency, Water Hammer and Mass Oscillation (WHAMO) by the United States Army Corps of Engineers, the Transient Analysis Program (TAP) by Applied Science, Inc., and InfoWorks CS by Innovyze.

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.

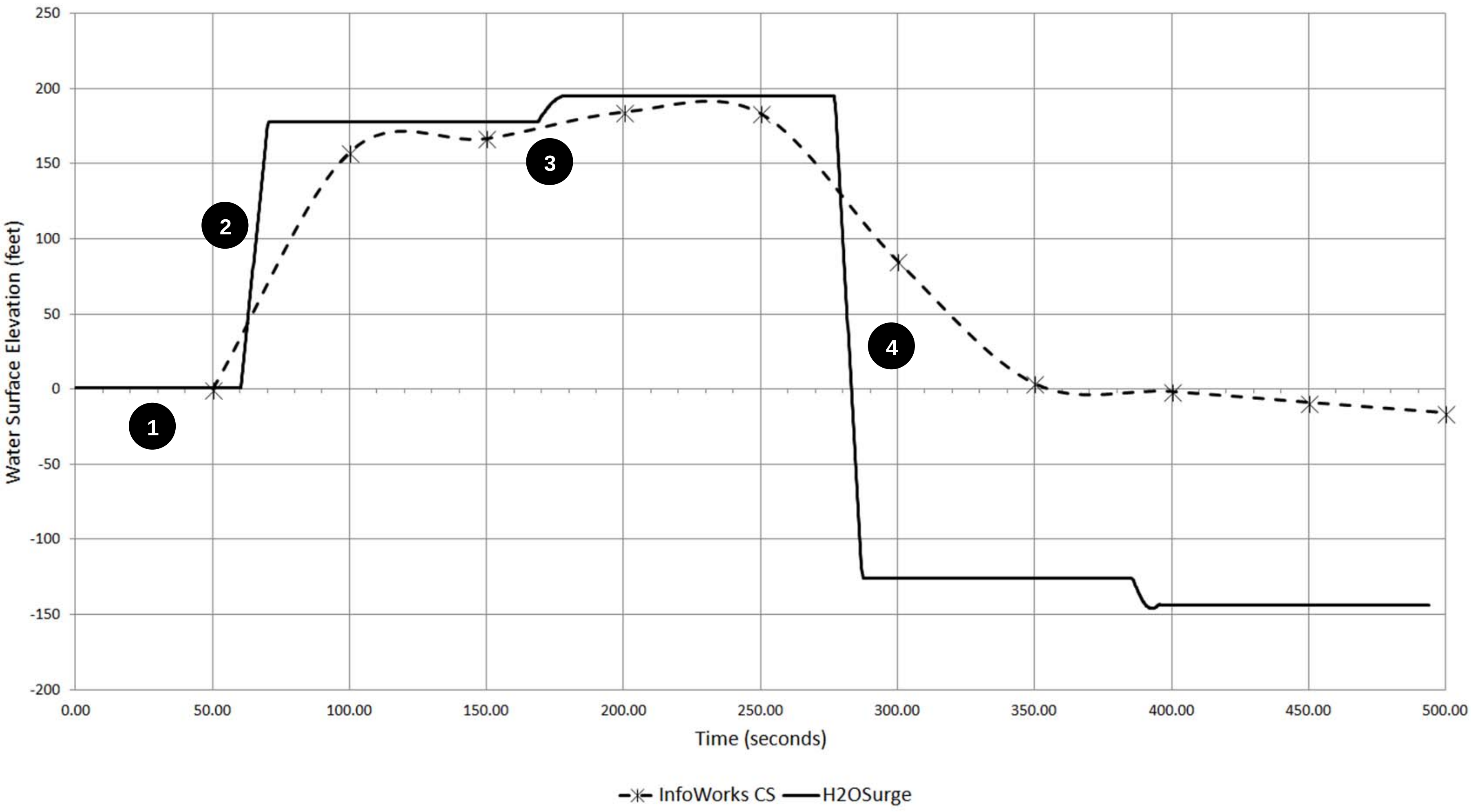


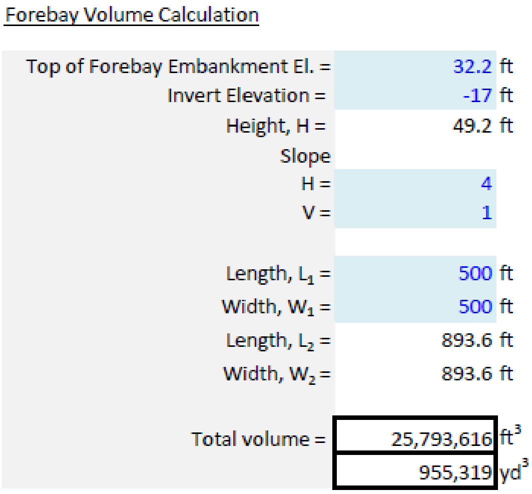
Figure 6 – InfoWorks CS vs. H2OSurge Comparison

2.3 Assumptions

2.3.1 Layout and Profile

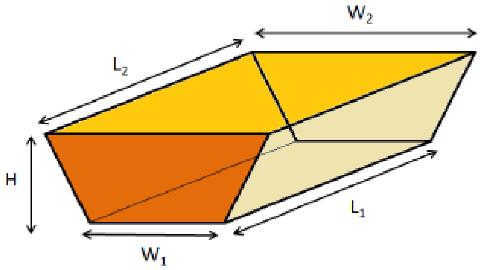
TM 3 builds upon draft of TM 2 (rev0) submitted to MWD in September 2014. In regards to the physical layout and profile, Run ID TM2_Ex. (TM 2, Table 1) was used. No other changes were made, except as follows:

- 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.



Forebay Volume Calculation

Top of Forebay Embankment El. =	32.2	ft
Invert Elevation =	-17	ft
Height, H =	49.2	ft
Slope		
H =	4	
V =	1	
Length, L_1 =	500	ft
Width, W_1 =	500	ft
Length, L_2 =	893.6	ft
Width, W_2 =	893.6	ft
Total volume =	25,793,616	ft^3
	955,319	yd^3



Known: Elevation, height, slope, Length L1, Width W1
Unknown: Length L2, Width W2

Figure 7 - New IF Layout[1]

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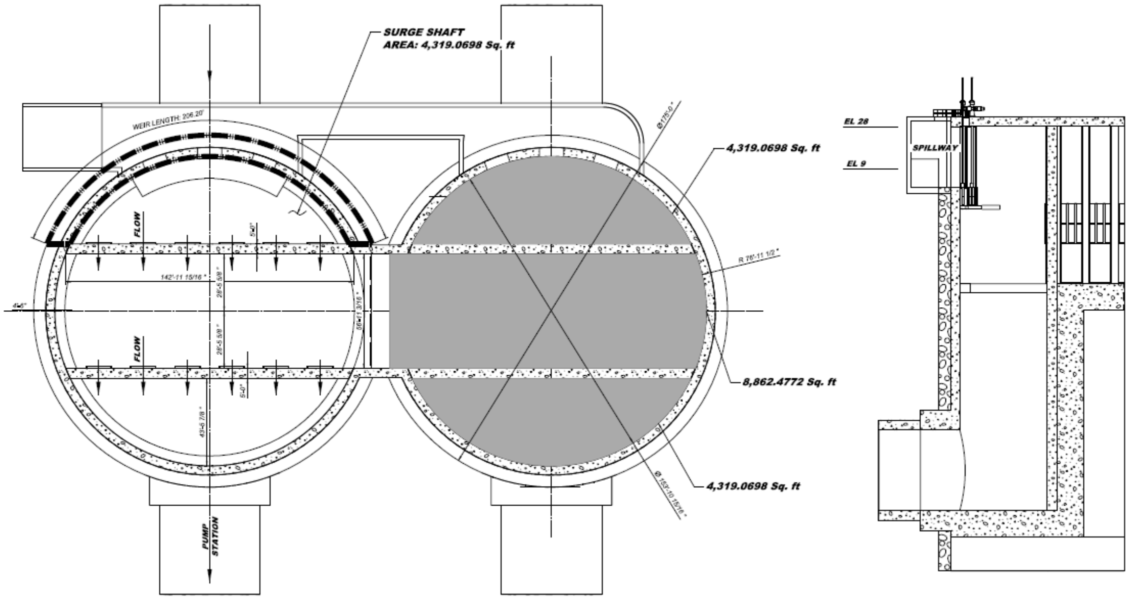
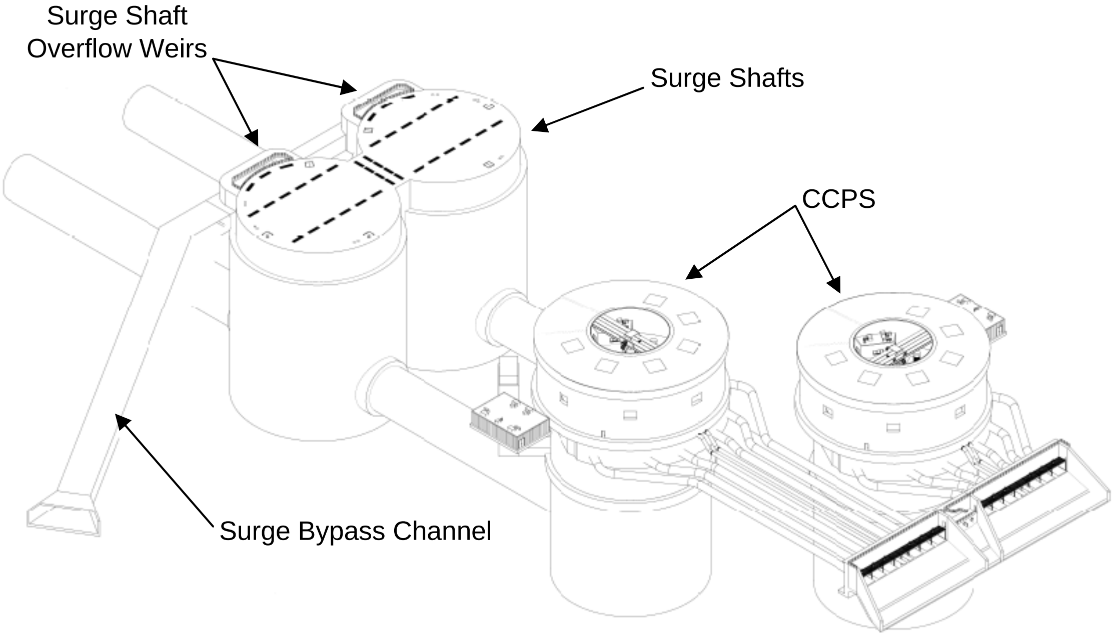
[1] *Provided to AECOM by MWD.*



(a) Plan View



(b) Iso-metric View

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.

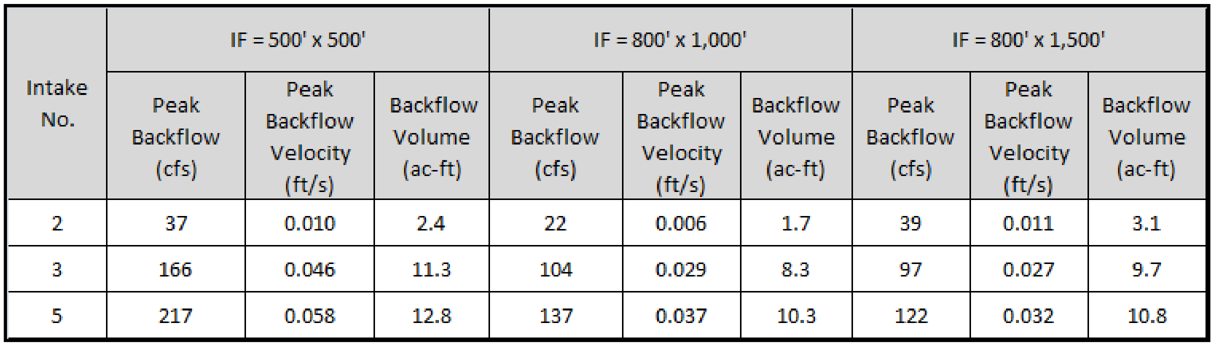
3.2 Testing the Influence of Various IF Sizes on the Transient Response

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 foot-print.

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



Intake No.	IF = 500' x 500'			IF = 800' x 1,000'			IF = 800' x 1,500'		
	Peak Backflow (cfs)	Peak Backflow Velocity (ft/s)	Backflow Volume (ac-ft)	Peak Backflow (cfs)	Peak Backflow Velocity (ft/s)	Backflow Volume (ac-ft)	Peak Backflow (cfs)	Peak Backflow Velocity (ft/s)	Backflow Volume (ac-ft)
2	37	0.010	2.4	22	0.006	1.7	39	0.011	3.1
3	166	0.046	11.3	104	0.029	8.3	97	0.027	9.7
5	217	0.058	12.8	137	0.037	10.3	122	0.032	10.8

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.

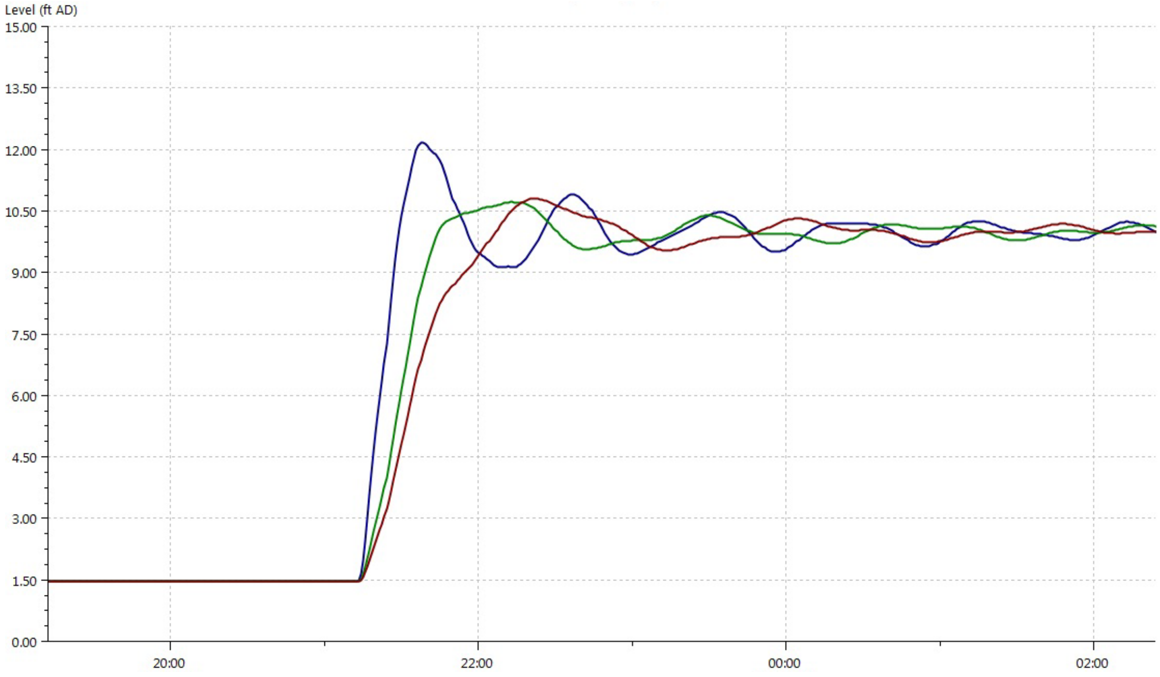


Figure 9 – HGL (ft) at STA 3150+70 Vent Shaft (located Upstream of the IF)

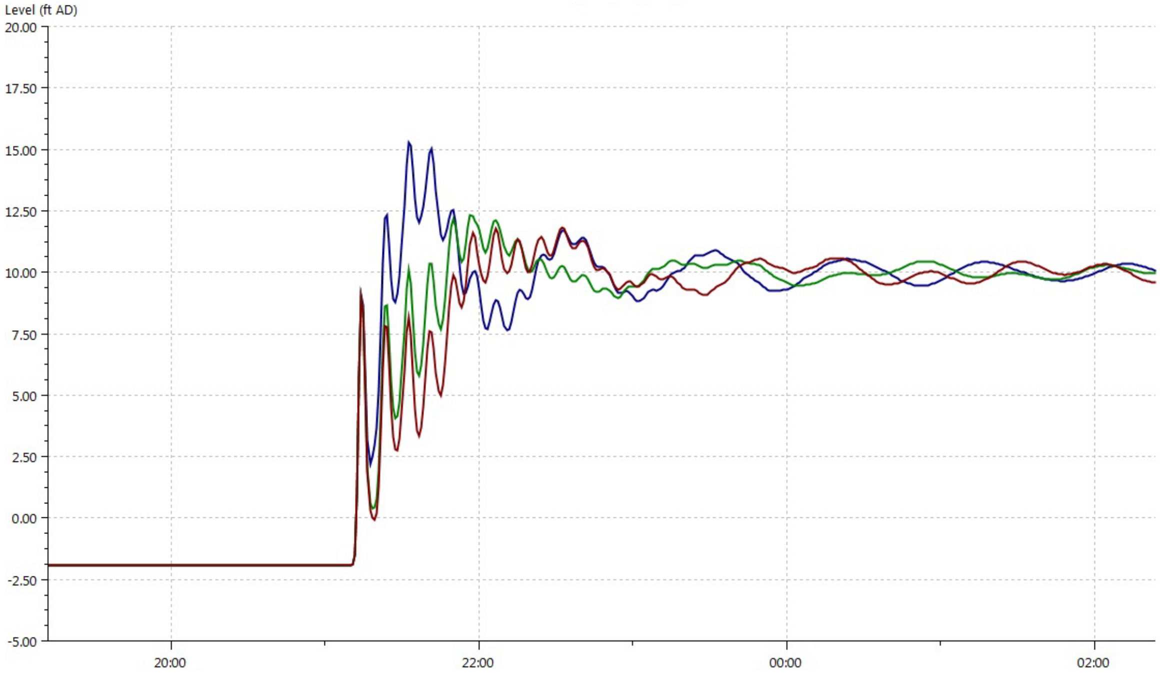


Figure 10 – HGL (ft) at Staten Island Vent Shaft (located Downstream of the IF)

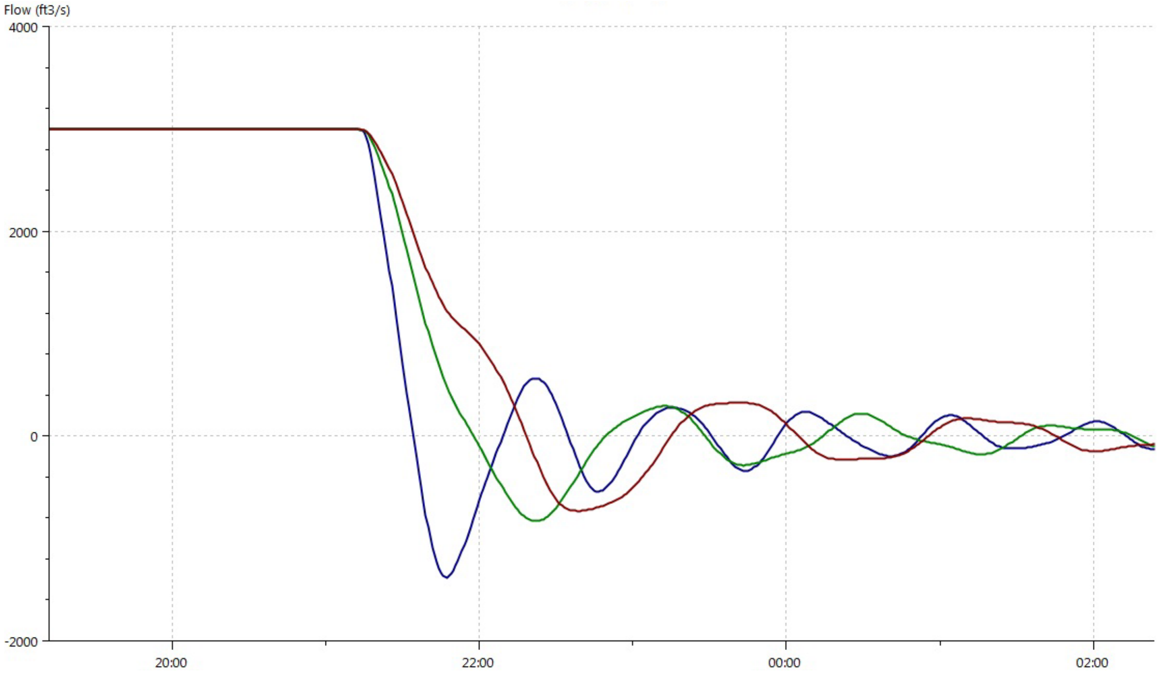


Figure 11 – Flow (cfs) History at Intake 5 Reception Shaft

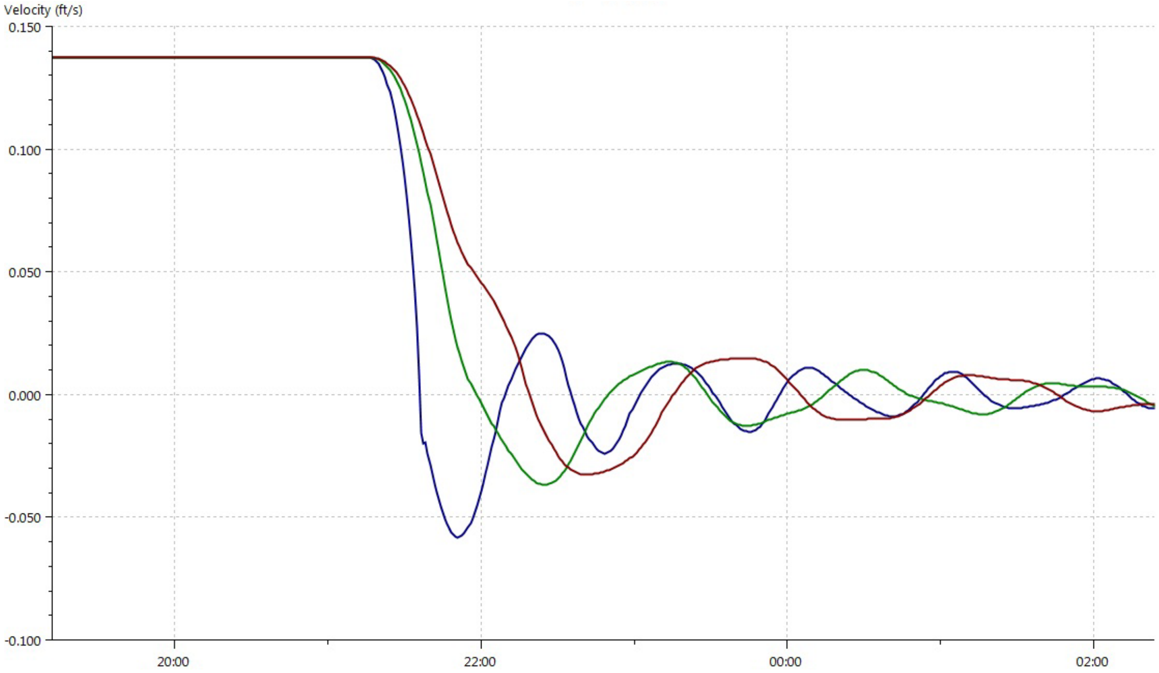


Figure 12 – Velocity (fps) History in an Intake 5 Sediment Basin

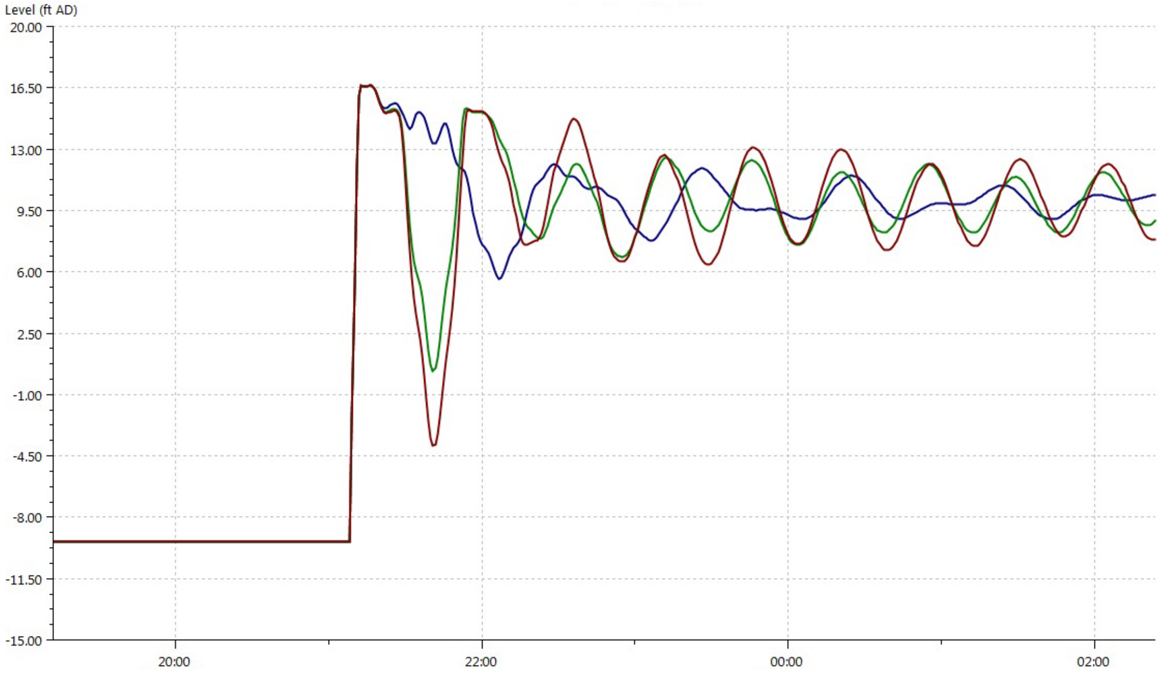


Figure 13 – HGL (ft) at CCPS Surge Shaft

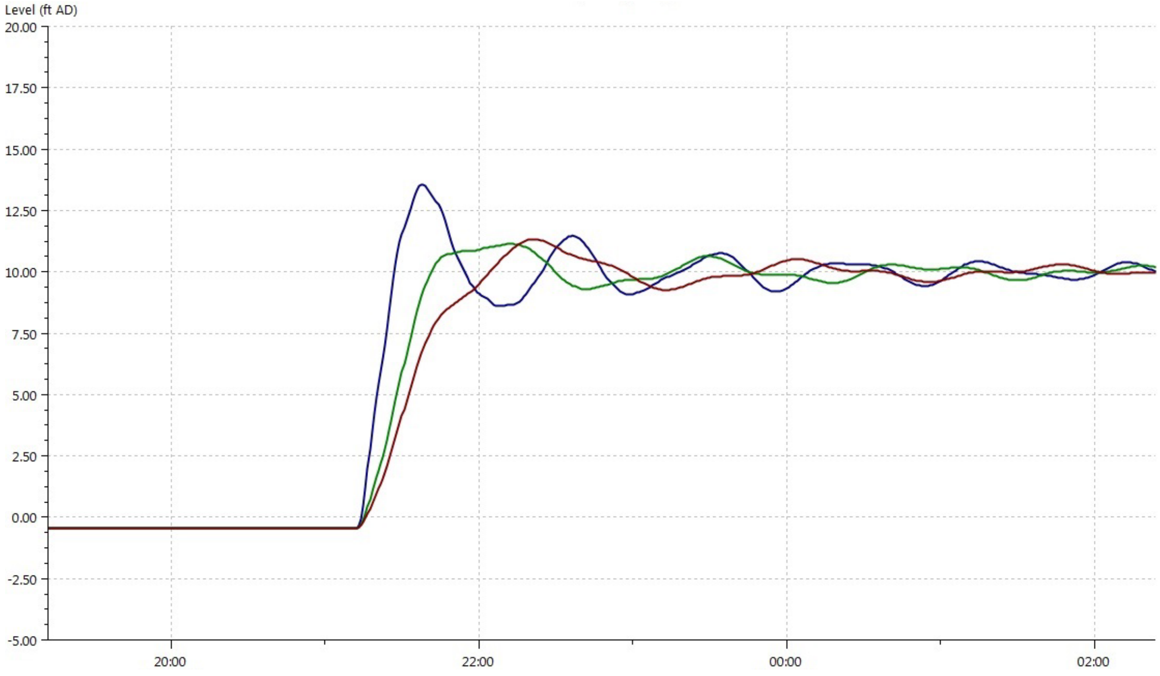


Figure 14 – HGL (ft) in IF

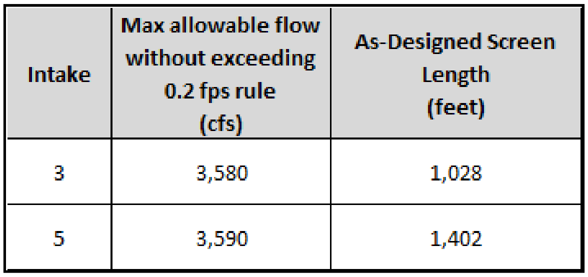
3.3 Evaluation of System Response with Intake 2 Off-line

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.

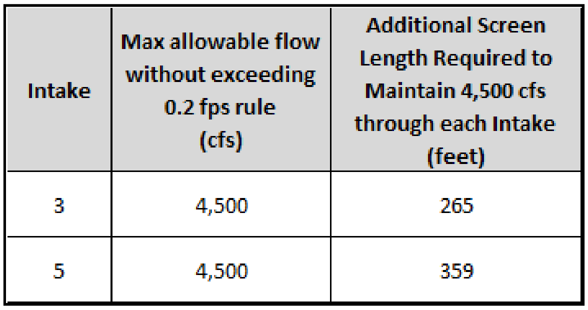
- Next, recognizing that 9,000 cfs could not be achieved, how many more screens would be required (in lineal feet) at both Intakes 3 and 5 to achieve a full 9,000 cfs? Results are summarized in Table 3 and present the additional screen length required to achieve the full 9,000 cfs without exceeding the 0.2 fps rule at the intake screens.

Table 2 – Maximum Flow Withdrawal (Intake 2 Off-line)



Intake	Max allowable flow without exceeding 0.2 fps rule (cfs)	As-Designed Screen Length (feet)
3	3,580	1,028
5	3,590	1,402

Table 3 – Required Additional Screen Length (Intake 2 Off-line)



Intake	Max allowable flow without exceeding 0.2 fps rule (cfs)	Additional Screen Length Required to Maintain 4,500 cfs through each Intake (feet)
3	4,500	265
5	4,500	359

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 → as-designed screen lengths and Green → increased screen lengths at Intakes 3 and 5.

Figure 15 and Figure 16 show the HGL on either side of the IF.

Figure 17and 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.

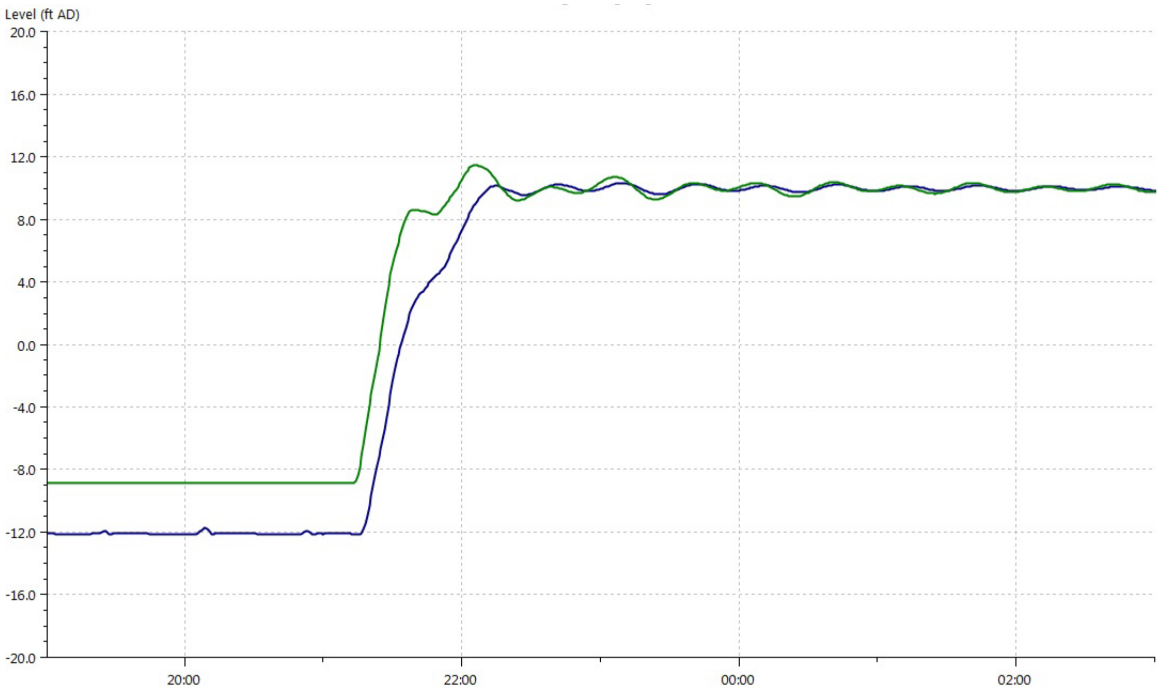


Figure 15 – HGL (ft) at STA 3150+70 Vent Shaft (located Upstream of the IF)

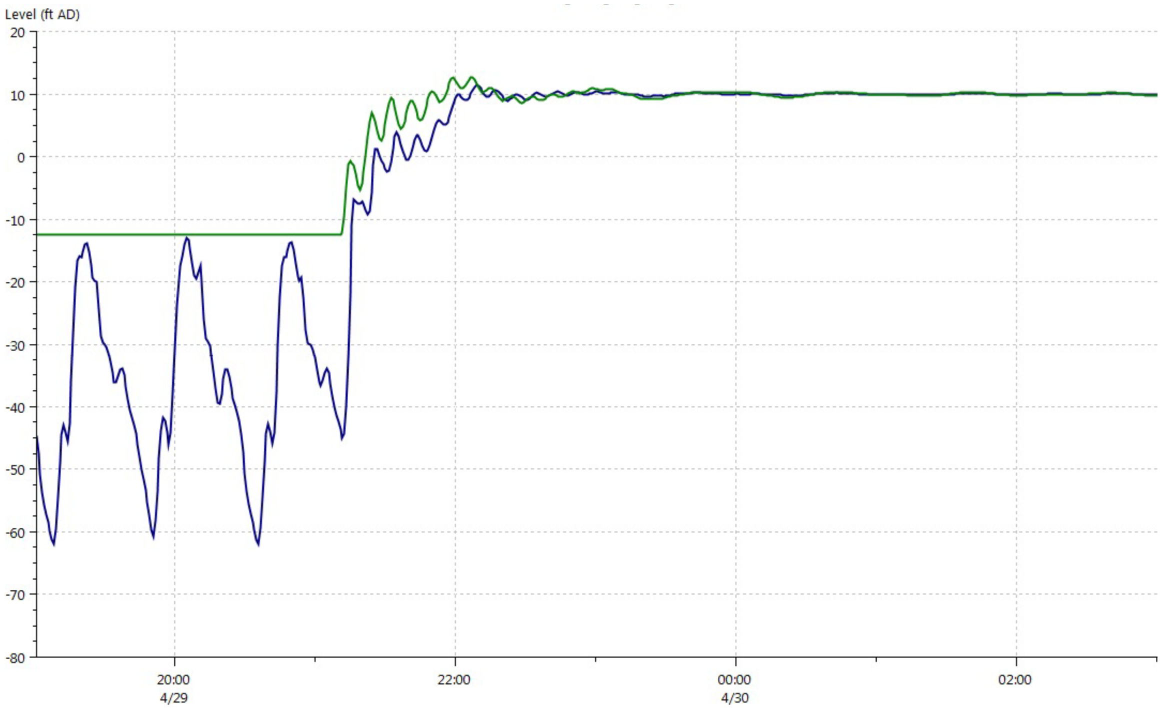


Figure 16 – HGL (ft) at Staten Island Vent Shaft (located Downstream of the IF)

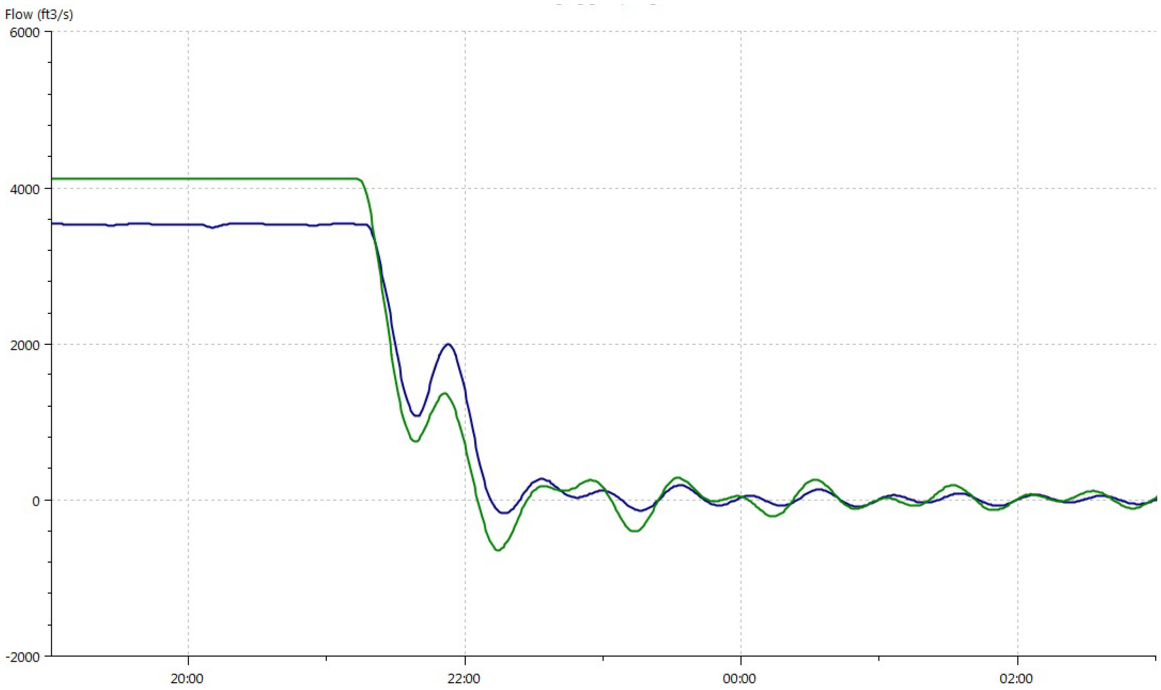


Figure 17 – Flow (cfs) History at Intake 5 Reception Shaft

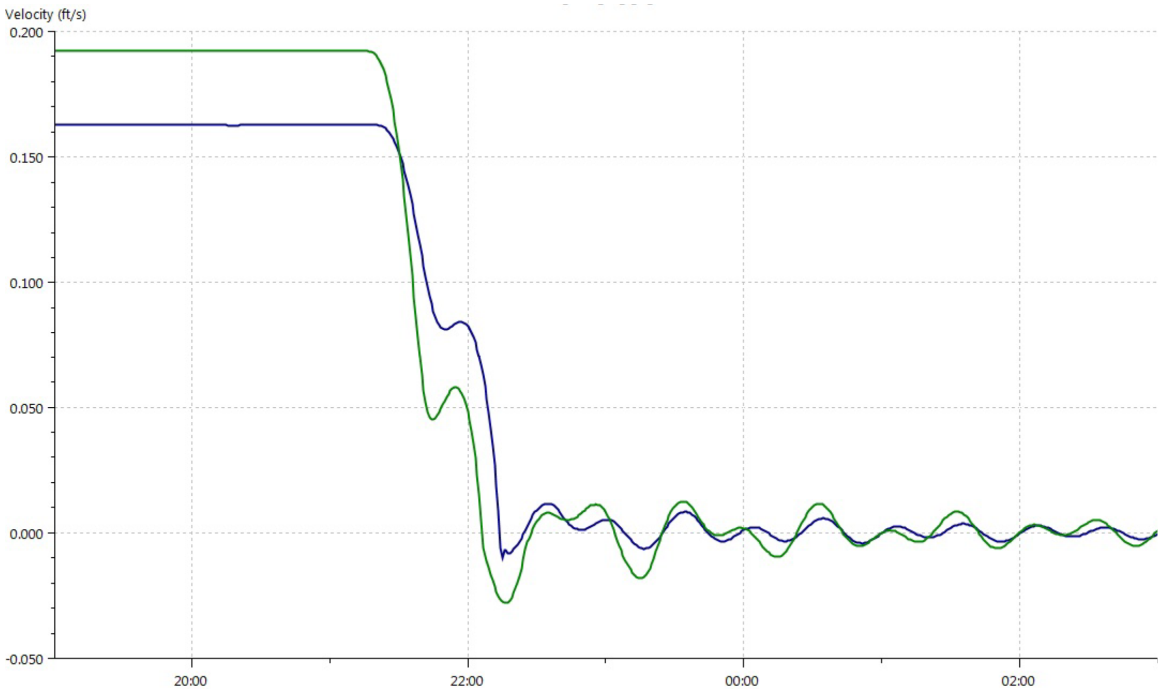


Figure 18 – Velocity (fps) History in an Intake 5 Sediment Basin

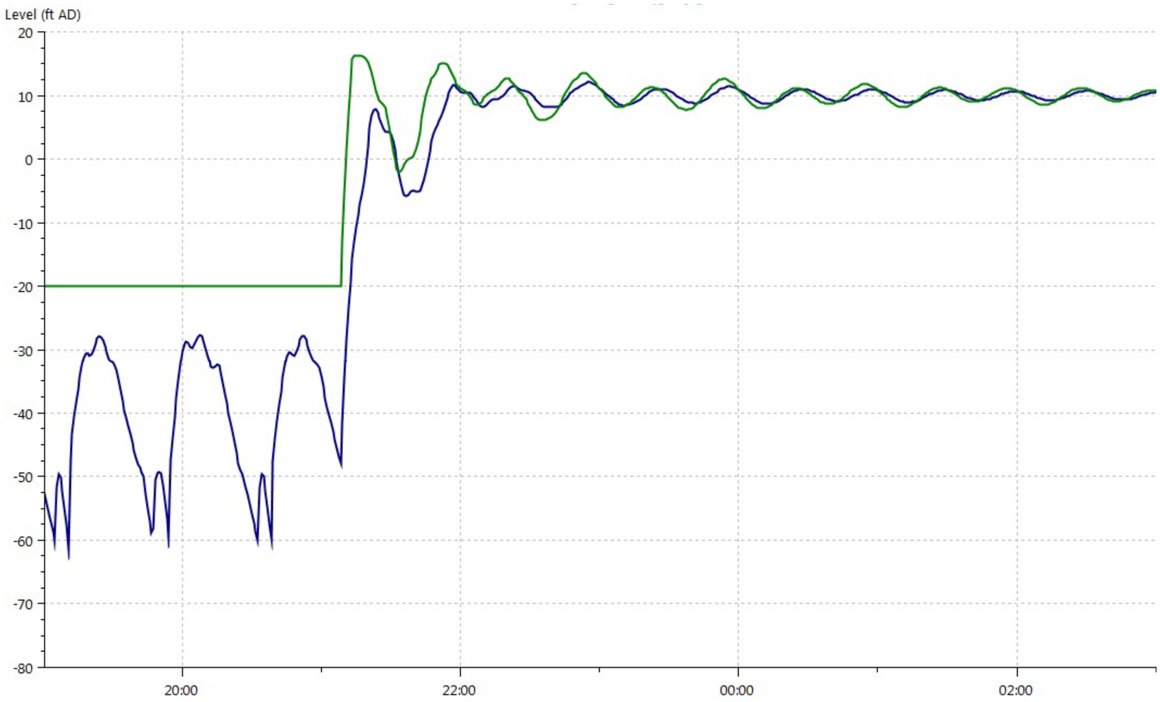


Figure 19 – HGL (ft) at CCPS Surge Shaft

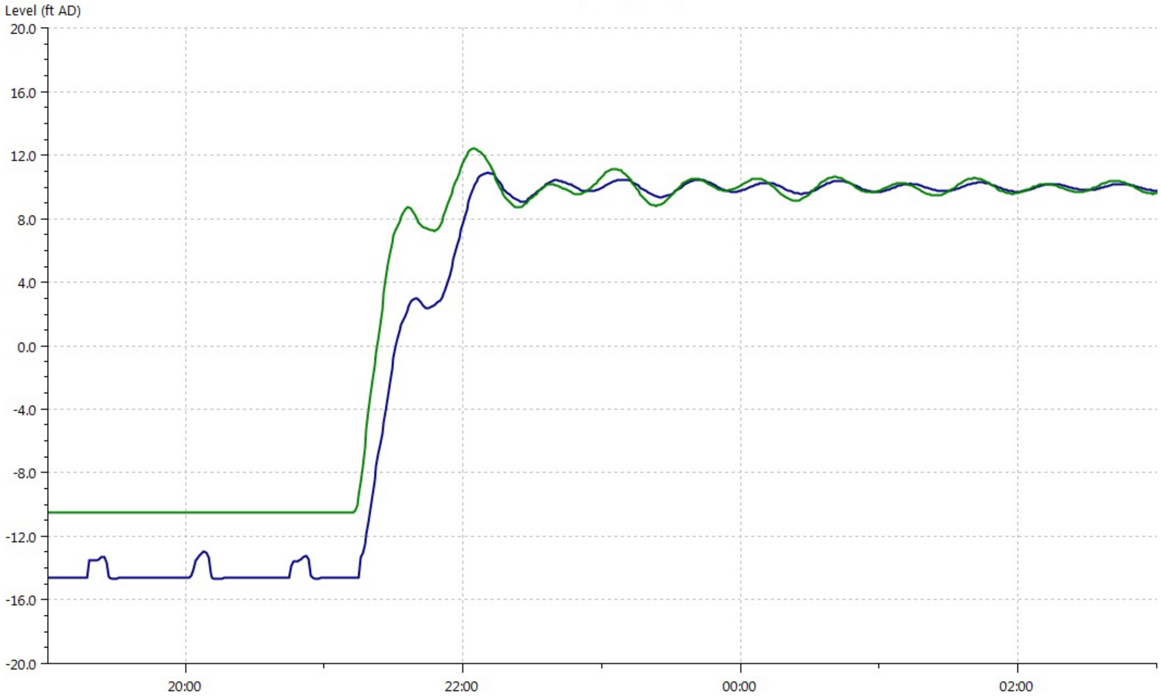


Figure 20 – HGL (ft) in IF

Appendix E
Pipe Materials

1.0 OVERVIEW

Materials and construction for the conveyance conduits between the Intake Pumping Plants and the new Canal are evaluated.

2.0 GENERAL DESCRIPTION OF CONVEYANCE SYSTEMS

Options to convey up to 15,000 cubic feet per second (cfs) of Sacramento River water from upstream of the Delta to the Banks and Jones Pumping Plants are under review. The Isolated Conveyance Facility (ICF)-West, ICF-East, and Dual Conveyance facilities each include five 3,000 cfs intakes and pumping plants on the Sacramento River that will pump through conveyance conduits to new canals for conveyance to the Banks and Jones Pumping Plants. The fourth option, Through Delta Facility (TDF), has two 2,000 cfs intake pumping plants that will pump through conveyance conduits to a new canal that feeds into the existing Delta canals.

3.0 PURPOSE AND SCOPE

To identify, evaluate, and recommend materials of construction for the conduits between the Intake Pumping Plants and the Canals.

A conduit size optimization evaluation indicated two conduits with a 200-square-foot open area per conduit (16 feet diameter if circular) is the optimum conduit size for 3,000 cfs conveyance capacity. For the 2,000 cfs two conduits with a 155-square-foot open area per conduit (14 feet diameter if circular) is the optimum conduit size.

4.0 LARGE CONVEYANCE INSTALLATION HISTORY

Since the optimum conveyance line size is very large compared with typical water transmission projects, a history of large diameter conveyance projects was first gathered to summarize industry experience in comparable sizes. Several large-scale projects have been implemented overseas, and the design and construction information may be applicable to this project. Pipe suppliers and manufacturers were also contacted to solicit their input with regard to large diameter conduit manufacturing and installation. Generally, a project of this scale will require special fabrication, transportation, and installation methods.

Table 1 summarizes the findings. The table shows that materials of construction successfully used for this size of pipe in the past includes CIP concrete, welded steel, and pre-stressed concrete cylinder pipe (AWWA C301).

Table 1: Large Conveyance Installation History

Pipe Type	Date	Size (ft)	Material Type	Length	Location
Steel Pipe					
Conveyance Pipe					
Central Arizona Project	1990s	21	Steel	1.7 miles	Arizona
Pacific Corp Swit 2 Project	--	11.5-16	Steel plates	--	--
Penstock					
Ghazi Barotha	--	5 @ 34.75	Steel-lined penstocks	100 meters	--
Gauley River Penstock	1990s	10-17	Steel	350 ft	West Virginia
Concrete Cylinder Pipe					
Pre-stressed concrete cylinder					
Central Arizona Project	1976	21	PCCP fabricated near site	4.5 miles	Arizona
San Onofre Project	1979	10-18	PCP non-cylinder	3.3 miles	California
MWD Castaic Project	1971	16.75	PCCP fabricated near site	5.9 miles	California
USBR Navajo Project	1975	15.75-17.5	PCCP fabricated near site	4.4 miles	New Mexico
Great man-made River	Phase 1 mid 1980s Phase 2 mid 1990s	2 @13	PCCP fabricated near site	490 & 510 miles	Libya
China South north	--	10	PCCP		China
Cast-in-Place Concrete Pipe					
Arch					
LACSD Joint Outfall	1973	12	Concrete	3.8 miles	California
Circular					
Tracy pump Plant d/c Lines	1947	15	Concrete	0.4 miles	California
Cast-in-Place Box Culvert					
Roundhill Reservoir	--	2 @ 19.6 x 19.6	Cast-in-place concrete	--	--
Batang Padang Hydro	--	40 ft x 40 ft	Cast-in-place concrete	200-300	--
KUMPP	Proposed	4 & 13' x 13'	Cast-in-place concrete	--	--
Other					
FRP					
Jubail Cooling water bypass	--	13.1	FRP	2 km	-
Tunnels					
Mangla	--	5 @ 26'	Steel grouted annular space	1800 ft each	--
Shanxi Wanjizsha Yellow River Diversion	--	18.3	--	--	China
Dokan dam	--	2 @ 36' and 39'	concrete lined tunnels 3'thck	--	--
Bombay outfalls under "sea"	--	11.4	--	7 km	India

FRP = fiberglass reinforced plastic
ft = feet
km = kilometer
KUMPP = Krishnapatnum Ultra Mega Power Project
LACSD = Los Angeles County Sanitary District
MWD = Metropolitan Water District of Southern California
PCCP = prestressed concrete cylinder pipe
Reclamation = United States Bureau of Reclamation

5.0 OPERATIONAL CONDITIONS AND REQUIREMENTS

For 3,000 cfs conveyance two 16-foot equivalent diameter lines are proposed. Similar material constraint conditions and economies would apply to the conveyance lines used for 2,000 cfs (two 14-foot equivalent diameter lines.)

5.1 Hydraulic Capacity and Pressure

5.1.1 Velocity

For each 3,000 cfs intake pumping plant the optimum conduit size provides a 200-square-foot open area per conduit and two conduits per intake pumping plant. With the planned pump capacity and transition structure configuration, the anticipated flow velocity ranges from 1.5 feet per second (fps) to 8.0 fps depending on the number of pumps in operation.

5.1.2 Pressure

The maximum design pressure between pumping plants and the canal is less than 30 pounds per square inch (psi). The water surface in the canal at the down stream end of the conveyance pipeline is above the conduit centerline at the canal and at the conveyance pipeline connection to the pumping plant transition structure. This will prevent the pipeline from draining during periods of no flow. The canal water surface elevation also sets the static pressure for the conveyance system. Static pressure is less than 15 psi.

5.2 Other Design Requirements

5.2.1 Depth of Cover

The minimum cover depth over the conduit will be 10 feet. This provides allowances for smaller size utility crossings, allows for some limited type of agricultural use over the top of the easement, and has allowances for erosion of the topsoil over time. The maximum cover depth will be at the pumping plant transitions structure, where berming around the pumping plant to raise it above flood level results in higher grade at the transition structure. In that area, the depth of cover may be up to 20 feet. Additional cover depths may be required to prevent conveyance line floatation. The conveyance line will be designed to withstand the earth load resulting from this depth of cover.

5.2.2 Depth-to-Groundwater

The depth-to-groundwater varies by intake and new canal location. Near the intake locations next to the Sacramento River the depth-to-groundwater fluctuates with river stage and local shallow aquifers.

Information available indicates the following general information:

Table 2: Depth-to-Groundwater

Option	General Depth-to-Groundwater
ICF-West	Varies from a few feet to 10 feet bgs
ICF-East and Dual Conveyance	Varies from a few feet to 6 to 8 feet bgs
Through Delta	Varies from a few feet to 6 to 8 feet bgs

bgs = below ground surface
ft = feet
ICF = Isolated Conveyance Facility

Because of the shallow groundwater elevations shown in Table 2 the pipeline material selected should be designed for continuous operation below the groundwater level. The presence of groundwater can impact pipeline corrosion rates and corrosion control design requirements depending on the conveyance material type. Also, with the conveyance lines installed below groundwater level, floatation must be prevented.

5.2.3 Length of Pipeline

Manufacturers that typically supply to water conveyance providers do not have fabrication facilities tooled to manufacture and handle pipeline with a diameter above 144 inches. Options to shop fabricated pipelines include field fabrication shops or fabricating in situ such as CIP circular or rectangular conduits. Field fabrication shops can require up to 50 acres for the fabrication facility but can be cost-effective if there is sufficient length of pipeline in a project.

The pipeline lengths between the intake pumping plants and the new canals for this project are relatively short and are estimated to total less than 11 miles per conveyance option.

5.2.4 Design Life

The pipeline conveyance facilities between the pumping plants and the new canals have a design life of 50 years. Material selection shall consider this design life requirement relative to corrosion control and maintenance.

5.2.5 Design Requirements Summary

The following summarizes conveyance conduit design requirements applicable to the materials and construction evaluation.

Table 3: Conveyance Conduit Materials Design Requirements

Criteria	Requirement
Flow Velocity Range, fps	1.5 to 8
Pressure, psi	< 30
Depth of Cover, ft soil	10 to 20
Groundwater	below groundwater
Design Life, yrs	50
Equivalent diameter, ft	14 to 18

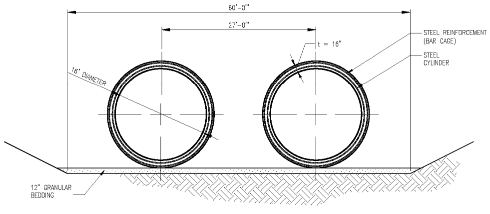
fps = feet per second
ft = feet
psi = pounds per square inch
yrs = years

6.0 CONVEYANCE CONDUIT MATERIALS GENERAL

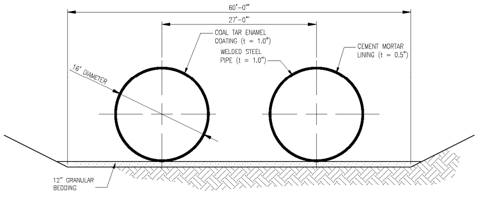
For pipelines 12 feet in diameter and smaller there are numerous pipeline material types commonly available. For line sizes above 12 feet in diameter, the range of material options is limited. Five conveyance conduit material/dimensional options were selected for evaluation based on the design criteria listed in Section 5.0, discussions with fabricators and industry experts, and the history of previous projects of this magnitude (discussed in Section 4). Conveyance conduit material option cross-sections are included on Figure 1. The cross-sections are conceptual and are suitable for general evaluations. Additional design refinements will be completed during the preliminary design phase of the project. The five material options selected for evaluation are:

- Steel pipe (AWWA C200 and plate welded)
- Concrete cylinder and concrete non-cylinder pipe (AWWA C300, C302, or C303)
- Circular CIP concrete pipe (ACI 301 and ACI 350-06)
- Rectangular CIP concrete box (ACI 301 and ACI 350-06)
- Arch CIP Concrete Conduit (ACI 301 and ACI 350-06)

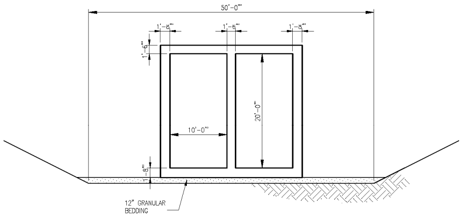
Pre-stressed concrete pressure pipe with a steel cylinder (AWWA C301) has been installed in diameters up to 21 feet, but was not considered for this project due to a history of failure of the high tension wire from corrosion and the fact that pre-stressed concrete pressure pipe with steel cylinder (AWWA C301) is not currently accepted by the California Department of Water Resources (DWR).



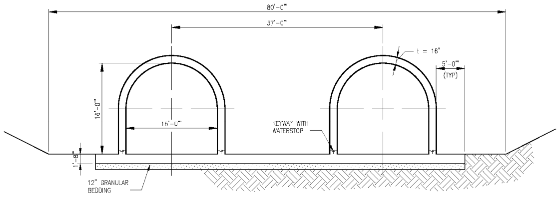
RCCP



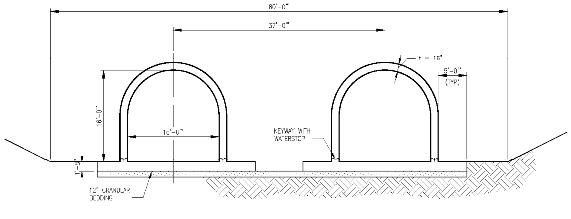
STEEL



CIP BOX CULVERT

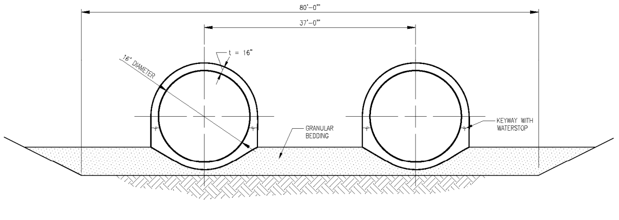


CIP ARCH WITH BASE SLAB



CIP ARCH WITH SEPARATE BASE SLAB

0 8'-0" 16'-0"



CIRCULAR CIP

Figure 1 – Conduit Material Options

7.0 CONSTRUCTABILITY

As a result of the large diameter and pipeline weights, the steel pipe and concrete cylinder pipeline material options will require unique fabrication and installation techniques. Heavy lifting equipment for both factory and field fabricated options will be required, both for fabrication and installation. Manufacturers of steel pipe and concrete cylinder pipe indicate their factory fabrication tooling is not designed for pipe sized over 12 feet in diameter. New fabrication equipment may need to be developed and built specifically for this project. In addition there are challenges associated with transport of the pipe from the factory to the field.

Field fabricated pipe has constructability issues including costs for installation and operation of the temporary field fabrication equipment. Space requirements for the fabrication site must also be considered.

7.1 Transport Considerations

In the case of pipeline shop-fabricated off site then shipped to the site there are three main options for shipping:

- Boat or barge (along the Sacramento River)
- Railway
- Truck

The first two options will require transport by truck from the river or railway station. Therefore, roadway transport requirements become a critical consideration for moving fabricated pipe to the site. California Department of Transportation (Caltrans) regulations for dimensions and weight govern requirements for use of public roads to transport pipe (steel or concrete). Regarding established maximums, Caltrans states:

“In the instance where a load exceeds 14’0” in width, 135’0” in overall length, over permit weight ranges or requires multiple width hauling equipment, the transporter, owner or manufacturer may request a variance to allow movement of the vehicle and/or load which exceeds these limits.”

The key submissions required to obtain a permit for a variance are:

- Proof that no other mode of transportation is reasonably available. This includes letters from railroad and/or barge companies for verification. A letter from the railroad is required only when the load is less than 16 feet 0 inches wide. A letter from the barge company will not be required when ports are not reasonably available for both points of origin and destination within the state.
- Scale drawings and/or photographs to establish that the load will be transported in the smallest size possible.

- Certification by the manufacturer/designer that the critical nature or technical or structural requirements prohibit field fabrication of small component pieces. Economics or fabrication ease is insufficient justification without a bona fide economic comparison furnished by the manufacturer/designer indicating the range of total costs for the various options of fabrication and transportation.

The maximums for weight and height are dependent on the delivery route to be taken. Weight limits are dependent on the bridge loading capacities along the route, but are generally limited to a load weight of 40,000 pounds (lbs). Height limits are dependent on the vertical clearance of any overhead structure required for the vehicle and/or load. A 3-inch minimum clearance must be maintained at all times. Written route review may be required from the applicant for heights greater than 17 feet 0 inches.

For extreme weights and dimensions, a California Highway Patrol (CHP) escort may be required. Table 4 summarizes the applicable conditions whereby a CHP escort is required.

7.1.1 Transport Comparisons

To compare transport limitations and the number of truck trips required for each material option, some general pipeline assumptions were made. As discussed previously, the load limits (weight, height, width) are assumed based on general legal dimensions established by Caltrans. The limits may decrease or increase depending on the route taken and the permit approval process. The assumptions for each material option are summarized in Attachment A.

Table 4: CHP Escort Table

Route Classification	Width (ft)	Length (ft)	Height (ft)	Speed (mph)
Multi-lane freeway or express way with 12' lanes, 4' shoulders	17'+ if 3 or more lanes. 16'+ if 2 lanes	Unlimited	No maximum[1]	Below 30 mph below posted maximum speed limit
Substandard freeway or 2 lane road with 12' lanes and 0' to 4' shoulders	15'+	Unlimited on controlled access roadways. 135' for others	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. 11' lanes. 0 – any shoulder	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. 10' lanes. 0 – any shoulder	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit
Two-lane road. Less than 10' lanes	15'+	135	No maximum[1]	Below 30 mph below posted maximum speed limit

The results of the pipeline transport comparison are summarized in Tables 4 and 5.

[1] Written route review may be required from the applicant for heights greater than 17 feet.

ft = feet
mph = miles per hour

Table 5: Shop Fabricated Steel and RCCP Transport

	Length of Section (ft)	Weight of Section (lbs)	Sections per Truck	Number of Truck Trips per Mile(1)
Steel (w/CML&C)	13	39,400	1	406
Steel (no CML&C)	20	41,500	1	264
RCCP	4	43,500	1	1,320

Note: (1) This is the number of trucks required to transport the pipe to the site. Additional truck trips may be required for other conveyance line requirements that are under investigation such as for cast-in-place concrete anchorage blocks required to prevent pipeline floatation.

CML&C = cement mortar-lined and coated
ft = feet
lbs = pounds
RCCP = reinforced concrete cylinder pipe
yd^3 = cubic yard

Table 6: CIP Circular Pipe and CIP Box Culvert Transport

	Total Volume of Concrete per Mile (ft^3)	Number of Trucks per Mile (10 yd^3 per truck max) (1)
2 CIP Circular Pipes	766,716	2,840
2 CIP Arches with Common Base Slab	1,571,011	5,819
2 CIP Arches with Separate Base Slab	1,119,096	4,145
CIP Box Culvert (2 w/common wall)	925,613	3,428

Note: (1) This is the number of trucks required to transport the concrete material for the conveyance line construction to the site. Additional truck trips may be required for other conveyance line requirements that are under investigation such as for CIP concrete anchorage blocks required to prevent pipeline floatation.

CIP = cast-in-place
ft^3 = cubic feet
yd^3 = cubic yard

Given a 14- to 16-foot-diameter pipeline, the limiting transport factor for shop-fabricated steel or concrete cylinder pipe is the weight. For the shop-fabricated pipeline options (Table 5) the number of individual sections and resulting number of joints impacts the number of welds required which in turn impacts cost and installation time. Shop-fabricated steel pipe can be shipped in longer lengths, thereby resulting in fewer truck trips and also less field joints. As shown in Tables 5 and 6, steel pipeline that is fabricated in the shop, and then assembled into pipe sections and lined and coated in the field, would require the least amount of transport trucks compared to other options.

8.0 MATERIALS

8.1 Steel

Spirally formed, fusion welded steel pipe has been manufactured and used in the United States since the late 1940s. Steel pipe has a demonstrated 50-years-plus service history. Steel pipe has many desirable qualities, which include durability, strength, economy, and reliability. Shop-fabricated steel pipe up to about 144 inches in diameter has been used extensively. There is limited experience for shop-fabricated steel pipe above 12 feet in diameter. However, there are a number of large (>14 feet) penstock applications where rolled steel plates have been shop manufactured and shipped to the construction site in sections and welded on site to form the pipe. In addition, steel tank erectors have confirmed that it is feasible to weld rolled steel plates (shipped or fabricated on site) vertically on site with a jig system and then to rotate the cylinder section for installation. Figure 2 shows a 21-foot-diameter spiral formed, fusion welded, steel pipe in shop fabrication conditions.

8.1.1 Availability

Due to the volume of steel that would be required for this project, advanced coordination and purchase agreements with fabricators should be considered. It may be necessary to assemble multiple bid packages should it be determined that there aren't any fabricators capable of constructing the entire conveyance system. Steel pipe would most likely be fabricated into pipe on or near the constructions site(s). Several steel pipe manufacturers have fabrication facilities that could be used for this project:

- Ameron (Tracy, CA)
- Northwest Pipe (Adelanto, CA and Portland, OR)
- Schuff Steel (Phoenix, AZ)
- Chicago Bridge and Crane (Claremont, CA)
- Advance Tank and Construction Company Inc. (Wellington, CO)

Figure 2: Fabrication of 21-Foot-Diameter Steel Pipe by Schuff Steel Company

Figure 3 shows field fabrication of a steel cylinder from bent plates.

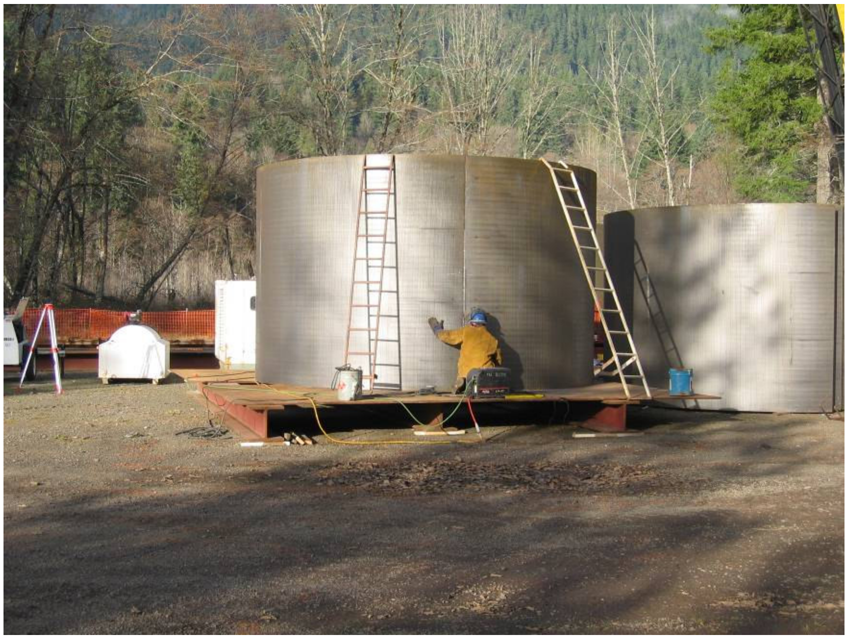


Figure 3: On-Site Fabrication of Bent Plate Steel Pipe

8.1.2 Transport/Installation

It is possible to ship steel pipe on trucks, but may not be economically feasible. The longest section of 16-foot-diameter bare steel pipe that can be shipped is 20 feet. The weight of the pipe and not the length is the controlling factor for shipment with truck transportation. Depending on the location of the fabrication shop, specialized pipe transport trucks may be used, such as the one shown on Figure 4.

Figure 4: Transportation of 21-Foot-Diameter Steel Pipe by Schuff Steel Company

Bare steel pipe can be supported with interior spider supports to prevent the pipeline from excessively deflecting during handling, shipment, and installation. If the steel pipe were cement mortar lined and coated in the shop, and then shipped, the increased weight would reduce the length of pipe per truck load from 20 feet to 13 feet. There would be concerns regarding pipeline deflection and potential damage to cement mortar lining and coating resulting from deflection during shipping. While shop-fabricated steel pipe may be a viable option, shipping it with cement mortar lining and coating does not appear to be practical.

The steel pipe transport equipment shown on Figure 4 was used both for transport from the fabrication shop as well as transporting the pipeline to the trench.

Rolled steel plates would be subject to the same trucking weight restrictions. For a 16-foot-diameter pipe, three sections (equivalent to a 20-foot-long pipe) can be trucked from the fabrication shop and assembled at the installation site. The rolled steel plates would be welded vertically on site with a jig system and then rotated for installation. A large crane could be used to move the steel cylinder into the trench.

Steel pipe is considered a flexible pipe system. To limit deflection of the pipeline it is important that the pipe embedment provide support around the pipe. Steel pipe will require more stringent and greater embedment requirements compared with the concrete and CIP conveyance options.

8.1.3 Maintenance

Much of the maintenance associated with steel pipe is related to preventing corrosion. Linings are used to protect the steel pipe from interior corrosion and erosion and coatings are used for corrosion protection on the exterior.

Linings are selected based on design flow velocities, handling and installation requirements and costs, service life requirements, and the physical and chemical characteristics of the water.

Cement mortar lining is relatively inexpensive, has been widely used, and has shown it can protect steel water pipelines under most operating conditions. Cement mortar lining can be used for continuous flow velocities up to about 15 to 20 fps. The expected velocities for the intake pumping plant conveyance are below this maximum allowable velocity. The water quality will need to be evaluated relative to sulfate and other constituents which could react with cement in the mortar. However, there are special cements and mortar mixes that can be used if needed.

Factory-installed cement mortar lining is a superior lining system; it will, however, add weight to the pipe and reduce the length of line that can be shipped per truck by about half of that for bare steel. It also reduces the allowable pipe deflection during handling, shipment and installation. If the steel pipeline is cement mortar lined prior to installation, the lining at the joints will have to be hand applied after the pipe is joined.

There are contractors who are able to apply a cement mortar lining to the pipe while it is in the trench. The field-applied cement mortar lining is more susceptible to variations in thickness and is not as dense as the lining applied centrifugally at the factory. A modified cement rich mortar lining mix should also be used for the field applied cement mortar lining. For a lining system applied in the field, a mobile applicator is constantly fed cement and centrifugally applies the cement mortar lining to the inside of the pipe. The in situ lining contractor estimated that with a 16-foot-diameter steel pipe and a 1/2-inch cement mortar lining, each machine could do 300 feet in a 10-hour day. The advantage of lining the pipe post shipping is that it significantly reduces the shipping weight of the pipe and reduces the likelihood for damage to the lining during shipping and installation. Once cement mortar lining is applied, the maximum pipeline deflection is 2 percent. This requires careful quality control of the pipe embedment compaction to prevent pipe deflection from soil loading. In situ cement mortar lining would be a viable option for the conveyance pipelines.

Cement mortar coating is usually applied in the shop by centrifugally spraying the pipe with cement mortar slurry to a thickness of at least 3/4 inch in conformance with American National Standards Institute (ANSI)/AWWA C205. If installed and handled correctly it has a superior service life history. Cement mortar coating will add weight to the pipe and reduce the length of line that can be shipped per truck by about half of that for bare steel. It also reduces the allowable pipe deflection during handling, shipment and installation. While it may be possible to cement mortar coat in the field by placing each bare steel section of pipe in the vertical position on a turn table and applying the mortar in a uniform fashion, handling and deflection considerations become a significant factor because of the low allowable deflection limits (less than 2%) with cement mortar coating.

Hot-applied coal tar enamel coating, ANSI/AWWA C203, has been used since the 1930s. To protect the coal tar enamel coating it is usually followed by a single layer of outerwrap consisting of glass-fiber felt, polyethylene-kraft paper, or polyethylene-elastomer laminate. Steel pipe with field-coated, hot-applied coal tar enamel coating (ANSI/AWWA C203) with the protective reinforced glass fiber inner and outer wrap would be a viable option for the proposed pipeline.

Regardless of the steel pipeline coating and lining, an impressed current cathodic protection system is recommended for longevity and reliability. With the high groundwater conditions, the cathodic protection system will require a higher current, for protection. In addition, it will require careful coordination with other utilities as stray currents can be an issue.

8.1.4 Joint Types

Steel pipe sections would be joined by welding. Several types of welded steel joints are available; however, lap welds are generally considered the most economical. At the expected operating pressures for this project, the lap welded joints can be fillet welded either internally or externally, or both. Butt welding of joints is more time-consuming because of the potential out-of-round pipe that makes welding tougher to control. Butt strap joints are more difficult to install, but are often used to install pipe assemblies or at changes of the pipe thickness. Butt strap joints should be considered for pipe closure assemblies.

If the steel pipeline is factory cement mortar lined, the lining at the joints will have to be hand applied after the pipe is joined.

8.1.5 Reliability

The advantage of steel pipe is that it is not typically prone to leakage. The concerns with steel pipe are exterior and interior corrosion, and flexure of the pipe. Correctly applying coating and lining, providing an impressed current cathodic protection system and providing steel pipe with adequate thickness would significantly reduce failures due to corrosion of the steel cylinder or joints.

8.1.6 Design Basis

Water industry standards for steel pipe are set forth in ANSI/AWWA C200. Design guidelines are set forth in AWWA M11. The assumptions in Table 8-1 were made for comparison purposes only.

Table 6: Steel Pipeline for Comparison Purposes

Inside Diameter after Lining (feet)	Wall Thickness (inches)	Lining Type	Coating type	Other
16	1.0	Field-installed cement mortar	Field-installed coal tar AWWA C203	Impressed current protection system

AWWA = American Water Works Association

8.1.7 Steel Pipe Summary

Hot-applied, coal tar, enamel-coated steel pipe that is cement mortar lined after installation is a viable option for the conveyance pipelines.

8.2 Concrete Cylinder and Concrete Pipe

The following three types of AWWA concrete pressure pipe are under consideration:

- Reinforced concrete cylinder pipe (AWWA C300)
- Reinforced concrete noncylinder pipe (AWWA C302)
- Bar-wrapped steel cylinder pipe (AWWA C303)

Design guidelines for these pipe types are included in AWWA Manual M9, Concrete Pressure Pipe.

Prior to acceptance of pretensioned concrete cylinder pipe (AWWA C303) in the 1960s, most of the concrete cylinder pipes used for pressure service above 55 psi was concrete cylinder pipe (AWWA C300). Both types have a thin steel cylinder embedded in concrete. The concrete cylinder pipe has mild steel reinforcing cages cast into the wall of the pipe and is suitable for pressures up to 250 psi. Concrete cylinder pipe is usually more expensive than bar-wrapped steel cylinder pipe.

Reinforced concrete noncylinder pipe (AWWA C302) has been used since the 1900s. It is used for pressure applications less than 55 psi and cover depths up to 20 feet are common. It is made with one or more reinforcing bar cages embedded in concrete. The concrete is placed by vertical or centrifugal casting method. Rubber gaskets joints have steel or concrete bell and spigot surfaces.

Bar-wrapped steel cylinder pipe (AWWA C303) has been manufactured and used extensively in the western United States since the 1960s. It consists of a concrete-lined and coated welded steel cylinder, helically wrapped with mild steel bar reinforcement under measured tension. It can be used working pressures up to 400 psi.

8.2.1 Availability

The manufacturers for concrete cylinder pipe (AWWA C300) and concrete noncylinder pipe (AWWA C302) indicate that their factory tooling is set up to fabricate 24- to 144-inch-diameter pipe. Larger diameter pipe would require special tooling and would result in more expensive pipe on a per unit basis. In addition, the weight of shop-fabricated concrete cylinder pipe would require manufacturing of pipe in shorter lengths to meet shipping weight limits. As a result, about three times as many truck trips would be required to transport concrete cylinder pipe to the project site as compared to bare steel pipe.

Manufacturers of AWWA C300 and C302 concrete cylinder pipe were asked about the feasibility of on-site fabrication of the pipe and indicated that on-site fabrication may be economically feasible depending on the pipeline lengths required for this project.

AWWA C303 is typically only provided in diameters 72 inches or less. Larger size bar-wrapped steel cylinder pipe is not considered a viable option by manufacturers who currently produce this material (Ameron or Northwest Pipe).

8.2.2 Transport/Installation

Figure 5 shows the on-site fabrication utilized by Ameron for their 21-foot-diameter AWWA C301 pipeline installed at the Central Arizona Project in the 1970s. Fabrication facilities required a 50-acre site. As discussed, AWWA C301 pre-stressed concrete cylinder pipe is not a recommended option for this project. However, similar area requirements would be needed for field fabricated C300 or C302 pipe.

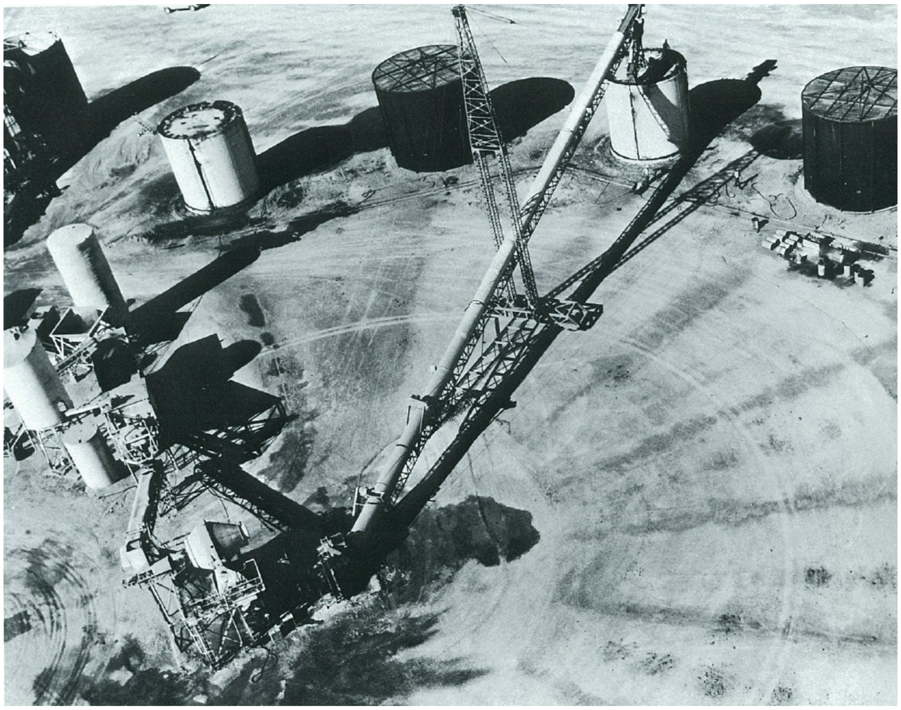


Figure 5: Ameron's On-Site Fabrication of Pre-Stressed Concrete Cylinder Pipe

Ameron has developed a process whereby they can pour, lift, and move sections of concrete cylinder pipe up to 21 feet in diameter. They use a radial stacker to pour concrete into several vertical pipe forms. The "Liftmobile" picks the pipe up and lays it horizontally on the ground. The "Pipemobile" then drives through the pipe section, hydraulically lifts it with the Pipemobile's mid-section, and drives into the trench and abuts to the previously laid pipe section (Figure 6).

In general, it is more economical to design rigid pipes (C300 and C302) to accommodate external loading with minimal bedding support than it is to require that the pipe be installed in highly compacted backfill. Bedding is required to avoid laying the pipe on hard, unyielding surfaces.

8.2.3 Maintenance

For most operating conditions, concrete pressure pipe is relatively maintenance free. The soil conditions and water quality will need to be evaluated relative to sulfate and other constituents which could react with the concrete. However, there are concrete mixes and cements that can be used if needed depending on any special water and soil chemistry conditions.

The field-applied cement mortar lining applied at the pipe joints after the pipe is joined is susceptible to thickness variations and can crack and spall, exposing the steel Carnegie type bell and spigot joints to the elements and increasing corrosion potential. The interior pipe joints should be periodically monitored for mortar cracking and damage and, if needed, should be repaired.

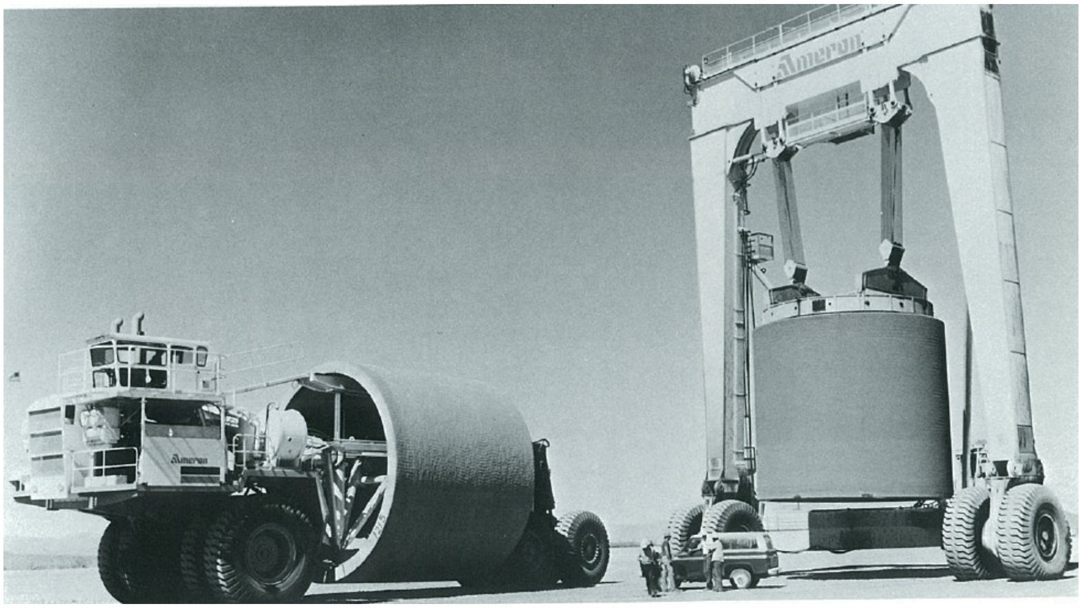


Figure 6: Ameron Used the "Pipemobile" and "Liftmobile" to Fabricate and Install 21-Foot-Diameter Pre-Stressed Concrete Cylinder Pipe On Site

8.2.4 Joint Types

Concrete cylinder pipe (AWWA C300), concrete noncylinder pipe (AWWA C302), and pre-stressed concrete cylinder pipe (AWWA C303) all can be provided with steel bell and spigot, Carnegie type joints. However, the joint connection for the concrete noncylinder pipe (AWWA C302) has less strength because it is welded to a steel collar piece and not welded to a steel cylinder.

Hydraulic Thrust can be resisted by the use of anchor blocks, by field welding adjacent pipe joints, or by a combination of both. Similar to steel pipe, field welding adjacent pipe joints instead of anchor blocks is a cost-effective option.

A mortar lining will have to be hand applied at the joints after the pipe is joined.

8.2.5 Reliability

Concrete pressure pipe is less susceptible to corrosion because the dense concrete layer at the interior and exterior of the line protects and passivates the steel reinforcing and cylinder. The hand applied cement mortar lining applied at the joints after the pipe is joined is more susceptible to thickness variations and can crack and spall exposing the steel Carnegie type bell and spigot joints to the elements and increasing corrosion potential.

8.2.6 Design Basis

Water industry standards for concrete pressure pipe are described in AWWA C300, AWWA C302 and AWWA C303. Design guidelines are set forth in AWWA M9. The assumptions in Table 8-3 were made for comparison purposes only.

Table 8-3: Concrete Pressure Pipeline for Comparison Purposes

ID After Lining (ft)	Pressure Limits AWWA C300 (psi)	Pressure Limits AWWA C302 (psi)	Pressure Limits AWWA C303(1) (psi)	C300 Maximum Earth Cover Depth (ft)	C302 Maximum Earth Cover Depth (ft)
16	250	55	400	>20	>20

Note: (1) Not available in diameters greater than 72 inches.

AWWA = American Water Works Association
ft = feet
ID = inside diameter
psi = pounds per square inch

8.2.7 Concrete Pressure Pipe Summary

As shown in Table 8-3, concrete cylinder pipe (AWWA C300) and reinforced concrete noncylinder pipe (AWWA C302) meet both the pressure and depth-of-cover requirements for this project. Pre-stressed concrete cylinder pipe (AWWA C303) cannot meet the optimum pipeline size requirements.

8.3 CIP Concrete Conduit

As a result of the potential fabrication and transportation costs and challenges with shop or field fabricated steel pipe and concrete pressure pipe, three configurations for CIP conduit are considered. They include:

- Circular CIP concrete pipe
- Rectangular CIP concrete box
- Arch CIP concrete conduit

There are two arch conduit configurations: arch conduits with a common base slab, and arch conduits without a common base slab. Arch conduits with a common base slab would require more concrete than the option of separate base slabs for each, but may provide constructability advantages with a common slab requiring less formwork and a larger surface for form placement and support.

8.3.1 Availability

The materials required for CIP concrete conveyance options are readily available and do not require specialized fabricators or equipment like other conduit options (steel pipe and concrete cylinder pipe).

8.3.2 Transport/Installation

For the CIP pipe options, the primary transportation concern is associated with the large number of concrete delivery trucks that may be required. Conceptual estimates for the number of 10-cubic-yard (yd^3) trucks required for a representative 1-mile length of conveyance facilities are included in Table 7-3. The volume of concrete utilized for CIP conveyance options will likely result in the contractor setting up a concrete batch plant at the project site rather than purchasing the concrete batches from outside suppliers. However, a suitable source for clean water will be required for the field batch plant.

Installation of CIP conveyance options would require significant formwork. In addition, the CIP options will require the trench to be open for about 2 months to provide time for formwork placement, concrete placement and curing, and stripping forms. This is two to three times more than what is required for the steel pipe or concrete pressure pipe options.

The CIP circular options would require specialized formwork. The rectangular shape culvert options would also require formwork, but it would be less complex due to common wall construction and flat shapes. The walls would be keyed into the base slab and a water stop will be provided at each joint.

For the arch options, the arches will be keyed into the base slab and a water stop would be provided at each joint. For the arch with separate base slab option, the base could be placed at the same time as the stem walls with water stops at the joints and the arch will be formed and placed after the walls are cured.

8.3.3 Maintenance

CIP concrete conveyance lines should be relatively maintenance free. The soil conditions and water quality will need to be evaluated relative to sulfate and other constituents which could react with the concrete. However, special concrete mixes and cements could be used, if needed, depending on any special water or soil chemistry conditions.

The interior pipe joints should be periodically monitored for cracking and damage and, if needed, should be repaired.

8.3.4 Joints

Wall to slab joints will be keyed where applicable and water stops will be provided at the joints to minimize leakage.

8.3.5 Reliability

Due to the number of construction joints and contraction/expansion joints, and the pressure requirements, CIP conveyance options may have a higher probability for leakage at the joints. A minimal amount of leakage may be acceptable due to the rural location of the project and the recharge potential to the surrounding groundwater. There will be less water loss through the CIP conveyance options than through the open and unlined canals.

8.3.6 Conceptual Design Basis

Table 8-5 has concrete wall thickness requirements for the CIP conveyance options based on a conceptual design analysis.

Table 8-5: Cast-In-Place Conveyance Options for Comparison Purposes

Option	Wall Thickness (inches)	Base thickness (inches)	Top thickness (inches)
Circular	16	--	--
Rectangular	Exterior Walls = 20 Interior Wall = 18	20	18
Arch	16	20	--

8.3.7 CIP Options Summary

CIP concrete is a viable option for the conveyance conduits between the pumping plants and the new canals and does not require construction of special potentially expensive on-site fabrication shops. As an added benefit, there are more companies who can construct these conveyance structures compared with the limited number of manufacturers who can fabricate large diameter pipelines.

9.0 SUMMARY

Based on the preliminary evaluations, the conveyance materials and configurations viable for the proposed conveyance pipelines between the pumping plants and new canals are summarized in Table 9-1.

Table 9-1: Conveyance Line Summaries

Material	Shape	Fabrication	Advantages	Disadvantages
Steel (AWWA C200) In situ cement mortar lined Coal tar enamel coated (Welded steel plates also included in this option evaluation)	Circular	Shop or Field Field	▪ Shop fabrication usually results in better weld quality. ▪ Lighter weight compared to other options ▪ Trench can be “closed up” in less time compared to cast-in-place options which has advantages relative to dewatering costs, dust control and safety. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Hauling pipe results in more opportunities for damage, such as excessive deflections. ▪ Flexible pipe requires more embedment compaction and greater quality control of the embedment. ▪ Will likely require cement mortar lining once installed and that lining is less dense and more susceptible to variations in thickness compared with factory-installed cement mortar lining. ▪ Will require impressed current cathodic protection.
Concrete Pressure Pipe (AWWA C300 & AWWA C302)	Circular	Shop or Field (field is more likely)	▪ Does not require impressed current cathodic protection. ▪ Requires less embedment compaction. ▪ Trench can be “closed up” in less time compared to cast-in-place options which has advantages relative to dewatering costs, dust control and safety. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Heavier than steel pipe ▪ Hand-applied cement mortar at joints can be susceptible to cracking. Requires periodic inspections and repairs. ▪ Few suppliers have indicated they can fabricate concrete pressure pipe greater than 12-foot-diameter. ▪ Will likely require field fabrication shop requiring up to 50 acres.
Cast-in-Place	Circular	In Trench	▪ Thinner wall thickness compared to other cast-in-place options. ▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Two separate pipes are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Bedding preparation under pipe haunches will be more difficult to place compared with other options. ▪ Curved formwork and reinforcement requires more labor, higher installation costs. ▪ More difficult to support formwork bracing compared with other cast-in-place options that have a flat slab at the base. ▪ Requires more time before the trench can be “closed up” compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures.

Material	Shape	Fabrication	Advantages	Disadvantages
				▪ Concrete joints are more susceptible to leakage.
Cast-In-Place Concrete	Rectangular	In Trench	▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Formwork is less costly and labor intensive compared with cast-in-place circular conduits. ▪ Embedment materials and embedment compaction not required. ▪ Bottom slab can be placed and cured first providing surface for formwork building up. ▪ Utilizes common wall construction, making conveyance system more compact and the narrowest trench width.	▪ Requires more time before the trench can be “closed up” compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures. ▪ Concrete joints more susceptible to leakage. ▪ Not as favorable from a failure isolation standpoint compared to two separate circular conduits. ▪
Cast-in-Place Concrete	Arch	In Trench	▪ More contractors/suppliers available who can build this compared to the steel pipe and concrete pressure pipe options. ▪ Formwork is less costly and labor intensive compared with cast-in-place circular conduits. ▪ Embedment materials and embedment compaction not required. ▪ Bottom slab can be placed and cured first providing surface for formwork building up conveyance sections. ▪ Two separate arches are better from a failure isolation standpoint compared to rectangular conduit with a common wall.	▪ Requires more time before the trench can be “closed up” compared to steel or fabricated concrete pressure pipe options which results in higher dewatering costs, more dust control and increased safety compliance measures. ▪ Concrete joints more susceptible to leakage. ▪ Arch formwork more labor intensive compared to rectangular formwork.

ATTACHMENT A

Truck Transportation Assumptions

Truck Transportation Assumptions

General

- Diameter (or equivalent diameter) = 16 feet
- Number of pipes between intake and canal = 2
- Unit Weight of Mortar = 150 lb/ft^3
- Unit Weight of Steel = 490 lb/ft^3
- Maximum total length of conveyance for East and Dual Conveyance = 10 miles
- Max Transport Load Weight = 40,000 lbs
- Max Transport Dimensions: Height = 17 feet, Width = 15 feet
- Concrete Truck Capacity = 10 yd^3

Steel Pipe

Option 1: Fabricated and Cement-Mortar-Lined and Coated Off Site

- Steel Cylinder Thickness = 1 inch
- CML and CMC thickness = 1.5 inches

Option 2: Fabricated Offsite, Cement-Mortar-Lined and Coated On Site

- Steel Cylinder Thickness = 1 inch
- CML and CMC thickness = 0 inch

Reinforced Circular Concrete Pipe (Fabricated Off Site)

- Thickness = 16 inches

Cast-In-Place Circular Concrete Pipe

- Wall thickness = 16 inches

Cast-In-Place Arch with Common Base Slab

- Arch thickness = 16 inches
- Base Slab Height = 20 inches
- Base Slab Width = 66 feet

Cast-In-Place Arch with Common Base Slab

- Arch thickness = 16 inches
- Base Slab Height = 20 inches
- Base Slab Width = 58 feet

Cast-In-Place Box Culvert

- Exterior Wall thickness = 20 inches
- Interior Wall thickness = 18 inches
- Top slab = 18 inches
- Bottom slab = 20 inches

Appendix F
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.

Floatation = Weight of Conduit + Weight of Soil – Buoyancy

The safety factor was also calculated to quantify the ratio of force of weight versus force of buoyancy. A ratio, or safety factor, equal to greater than 1 indicates that the force of weight is greater than that of the buoyancy force and the pipe would not float. A ratio of less than 1 signifies floatation would occur.

$$\text{Safety Factor} = \frac{\text{Weight of Conduit} + \text{Weight of Soil}}{\text{Buoyancy}}$$

Table 1 presents the assumptions used in the floatation calculations.

Table 1: Calculation Assumptions

Description	Units	Value
Unit weight concrete (W_{Conc})	lb/cf	150
Unit weight water (W_{H2O})	lb/cf	62.4
Unit weight non-saturated soil (W_{Soil})	lb/cf	110
Unit weight steel (W_{Steel})	lb/cf	490
Minimum Safety Factor	N/A	1.1

lb/cf = pound per cubic feet
N/A = not applicable

2.0 FLOATATION SENSITIVITY ANALYSIS

The following sections provide the results of the floatation analysis conducted for each conduit type. The effect of depth of cover and wall thickness are also analyzed for both pipe materials. However, because infinite thickness variations exist for concrete arches and boxes, individual investigations for these conveyance options will be conducted during subsequent design updates.

2.1 Cast-In-Place Concrete Pipe

Two floatation analyses were performed for cast-in-place concrete pipe: depth of cover, and wall thickness. The conveyance options for each 3,000 cfs intake include two 16-foot-diameter pipes. The pipes were evaluated independently, assuming lateral forces would be minimal and resulting in identical results for each pipe. Figure 1 provides an illustration of this installation and the forces considered for floatation.

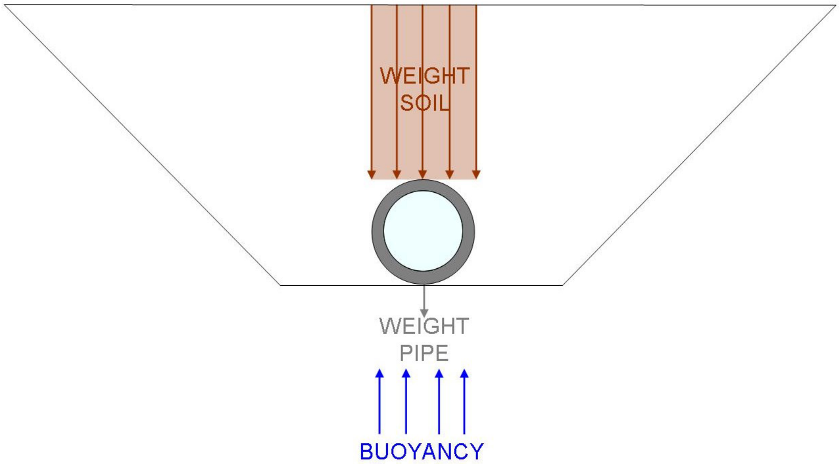


Figure 1: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the pipe, the width of the pipe outer diameter (OD), and height equal to the depth of cover (D=depth). The resulting area was then multiplied by the unit weight of non-saturated soil, equal to 110 pounds per cubic foot (lb/cf) specified in Table 1, resulting in pounds per foot (lb/ft) of soil.

$Weight\ of\ Soil\ (lb/ft) = OD \times D \times W_{Soil}$

The weight of pipe consisted of the weight of the cast-in-place concrete. This was determined by subtracting the area of the inner diameter (ID) of the pipeline from the area of the OD multiplied by the unit weight of concrete, assumed to be 150 lb/cf, resulting in lb/ft of concrete.

$Weight\ of\ Pipe\ (lb/ft) = (OD/2)^2\pi - (ID/2)^2\pi \times W_{Conc}$

Buoyancy is equal to the weight of water displaced by the pipe. This was calculated by determining the area occupied by the pipe multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the cover soil over the conduit must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

$$\text{Buoyancy (lb/ft)} = ((OD/2)^2\pi + OD \times \text{Cover Depth}) \times W_{H20}$$

2.1.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. A pipeline diameter of 16 feet was assumed, as determined in the Pipeline Optimization TM (URS Group, Inc. [URS], 2009a, pending) with a wall thickness of 1 inch per foot diameter, 16 inches. Table 2 and Figure 2 provide the results of this investigation.

Table 2: Concrete Pipe - Depth of Cover (16-foot-diameter)

Depth of Cover (ft)	Pipe Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	10.9	0.0	17.1	0.6
2	10.9	4.1	19.4	0.8
4	10.9	8.2	21.7	0.9
6	10.9	12.3	24.1	1.0
8	10.9	16.4	26.4	1.0
10	10.9	20.5	28.7	1.1
12	10.9	24.6	31.1	1.1
14	10.9	28.7	33.4	1.2

ft = foot
lb/ft = pound per foot

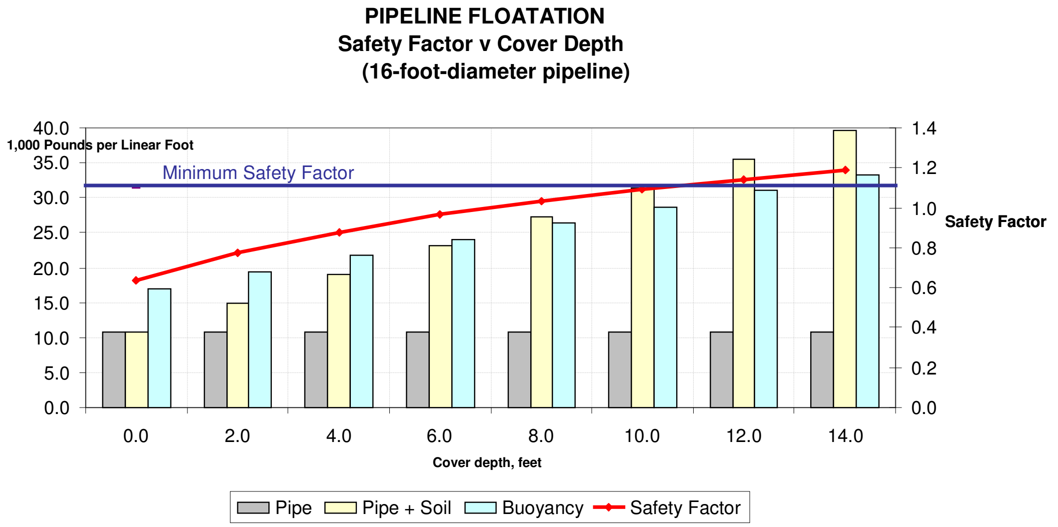


Figure 2: Concrete Pipe – Depth of Cover (16-foot-diameter)

As discussed in Section 1.3, a ratio of less than 1 indicates that the pipeline would float. Figure 2, shows that at a cover depth of less than 6 feet, the buoyant force would overcome the force of weight of the 16-foot-diameter pipe and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 10 feet is required.

Farming practices may cause disturbance of up to 6 feet of earth. An initial 16 feet of cover depth allows for 10 feet of pipe cover and 6 feet of soil for farming disturbance or erosion.

2.1.2 Wall Thickness

The second analysis provided for cast-in-place concrete pipes involved concrete thickness. This investigation assumed 16-foot-diameter pipes at various depths of cover, 0, 4, and 10 feet, and determined the floatation safety factor. While external loads and internal pressure design criteria would likely drive the design of concrete thickness, the results shown on Figure 3 present the potential effect on floatation.

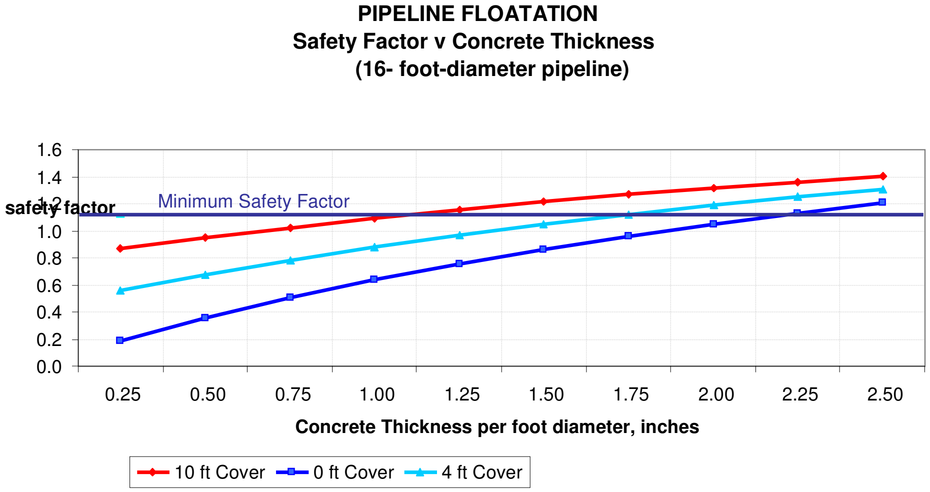


Figure 3: Concrete Pipe – Pipeline Wall Thickness

Figure 3 shows the sensitivity of pipe wall thickness to floatation prevention requirement that with 4 feet of cover, a thickness of 1.75 inch per foot diameter would prevent floatation, while a thickness of 2.25 inch per foot diameter would prevent conduit floatation without cover.

2.2 Steel Pipe

The same investigations conducted for cast-in-place concrete pipe were performed for steel pipe, using the same methodology as described in Section 2.1.

The weight of steel pipe consisted of the weight of steel, as well as the weight of the cement mortar lining. The lining thickness was assumed to be 0.5 inch, while steel thickness was assumed to be 1.0 inch thick. The pipe will likely be coated using coal tar epoxy, which is assumed to be negligible in this investigation. The approximate area per foot of each material was determined and multiplied by the unit weight, provided in Table 1. The cement mortar lining was assumed to have the same unit weight as concrete, 150 lb/cf.

$$Weight\ of\ Pipe\ (lb/ft) = (OD_{Coating}/2)^2\pi - (OD_{Steel}/2)^2\pi \times W_{Conc} + (OD_{Steel}/2)^2\pi - (ID_{Steel}/2)^2\pi \times W_{Steel} + (ID_{Steel}/2)^2\pi - (ID_{Lining}/2)^2\pi \times W_{Conc}$$

2.2.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Assuming a pipeline diameter of 16 feet with a steel thickness of 1.0 inch steel and 0.5 inch thick lining, Table 3 and Figure 4 provide the results of this investigation.

Table 3: Steel Pipe - Depth of Cover (16-foot-diameter)

Depth of Cover (ft)	Pipe Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	2.4	0.0	12.8	0.2
2	2.4	3.6	14.8	0.4
4	2.4	7.1	16.8	0.6
6	2.4	10.7	18.9	0.7
8	2.4	14.2	20.9	0.8
10	2.4	17.8	22.9	0.9
12	2.4	21.3	24.9	1.0
14	2.4	24.9	26.9	1.0
16	2.4	28.5	28.9	1.1
18	2.4	32.0	31.0	1.1
20	2.4	35.6	33.0	1.2

ft = foot
lb/ft = pound per foot

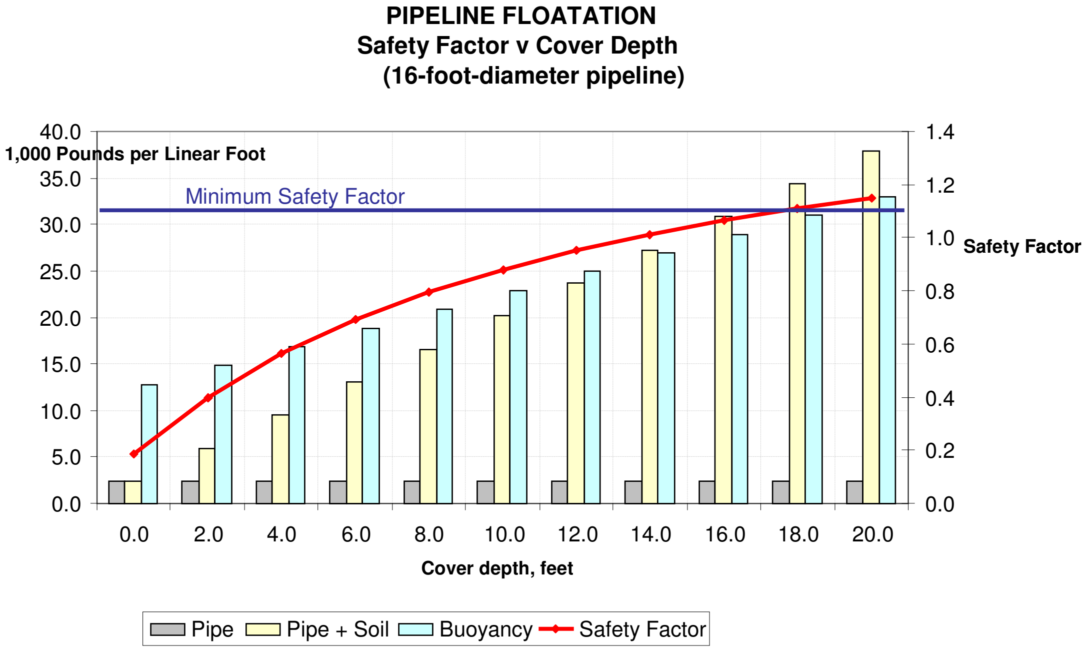


Figure 4: Steel Pipe – Depth of Cover

As discussed in Section 1.3, a ratio of less than 1 indicates that the pipeline would float. Figure 4, shows that at a cover depth of less than 12 feet the buoyant forces would overcome the force of weight of the 16-foot-diameter pipe and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 16 feet is required.

Farming practices may cause disturbance of up to 6 feet of earth. Accounting for 16 feet of pipe cover and 6 feet of soil for farming, the previous assumption of 10 feet of cover may not be sufficient to prevent floatation of steel pipe. A cover of approximately 22 feet would be necessary for steel pipe.

2.2.2 Wall Thickness

The second analysis provided for steel pipe involved steel thickness. This investigation assumed 16-foot-diameter pipes at various depths of cover, 0, 4, 7, and 10 feet, and determined the floatation safety factor at various diameter to thickness (D/t) ratios. While external load, deflection, and internal pressure design criteria would likely drive the steel thickness, the results provided on Figure 5 presents the potential effect on floatation. Table 4 provides the D/t ratios for the corresponding thicknesses.

Table 4: D/t Ratio to Steel Cylinder Thickness (16-foot-diameter)

Diameter to Thickness(D/t) ratio	Steel thickness, inch
160	1.200
200	0.960
240	0.800

The differences in thicknesses are minimal between D/t ratios, merely fractions of an inch, having a large impact on structural integrity and cost implications, although as shown on Figure 5 the effect on floatation is minimal. The figure shows that variance of the D/t ratio alone will not achieve the minimum safety factor of 1.1.

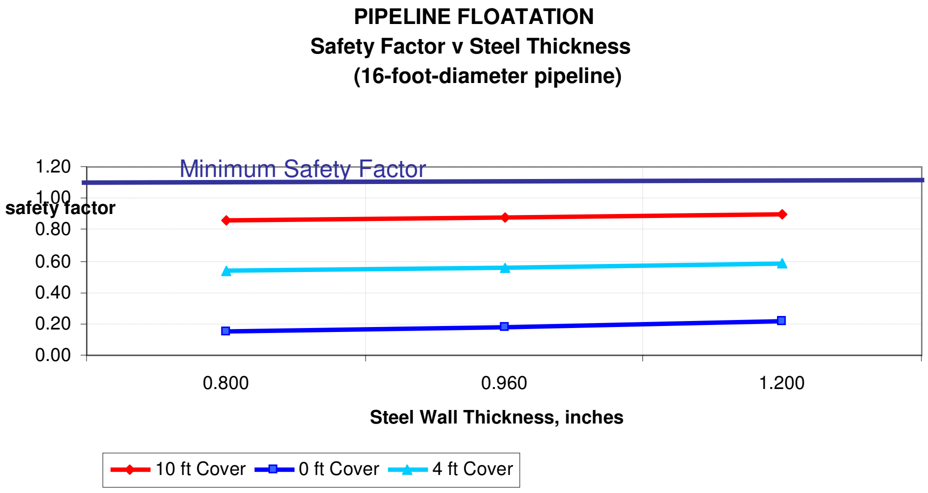


Figure 5: Steel Pipe – Pipeline Thickness

2.3 Concrete Arches

A sensitivity analyses was performed to observe the effect of various depths of cover for floatation with the concrete arch conduit option. Table 5 provides the assumed design criteria for the arches, while Figure 6 displays the layout of design.

Table 5: Arch Design Criteria

Design Criteria	Value
Number of arches	2
Arch height, feet	16
Arch width, feet	16
Arch thickness, inches	18
Distance between arches, feet	18
Distance from arch to edge of slab, feet	5
Slab width, feet	66
Slab thickness, inches	20

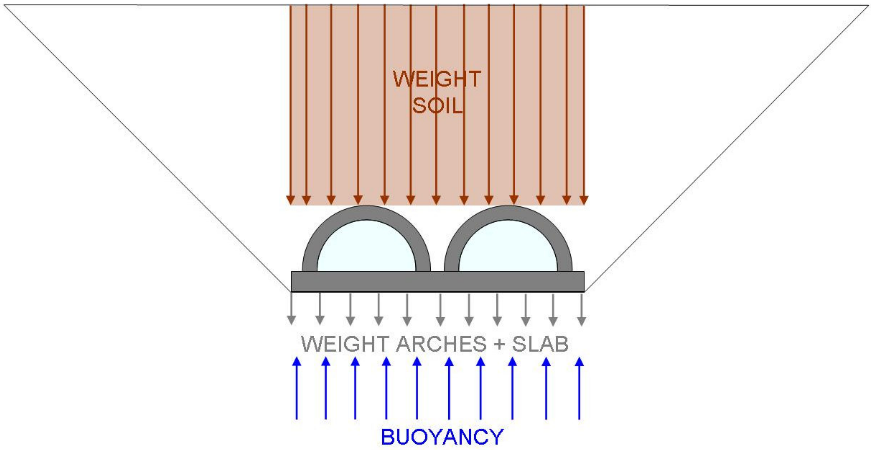


Figure 6: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the arches the width of the conduit slab (ws), and height equal to the depth of cover (D). The resulting area is then multiplied by the unit weight of soil, equal to 110 lb/cf specified in Table 1, resulting in lb/ft of soil.

$\text{Weight of Soil (lb/ft)} = w_S \times D \times W_{Soil}$

The weight of the conduit consisted of the weight of the cast-in-place concrete arches and slab. The weight of the arches was determined using the same method as described in Section 5.1 for circular pipes although divided by two and multiplied by the number of arches. The weight of slab was also determined by multiplying the width of slab (ws) by slab thickness (ts) and again by the unit weight of concrete.

$\text{Weight of Conduit (lb/ft)} = (2 \times \tfrac{1}{2} ((OD/2)2\pi - (ID/2)2\pi) + wS \times tS) \times WConc$

Buoyancy is equal to the weight of water displaced by the structure constructed, which equals the area occupied by the arches and slab multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the soil making up the conduit cover must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

$\text{Buoyancy (lb/ft)} = (2 \times \tfrac{1}{2} (OD/2)^2 \pi + w_S \times t_S + \text{Cover Depth} \times OD) \times W_{H20}$

2.3.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Table 6 and Figure 7 provide the results of this investigation.

Table 6: Arches - Depth of Cover

Depth of Cover (ft)	Conduit Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	28.9	0.0	24.6	1.2
2	28.9	14.5	32.8	1.3
4	28.9	29.0	41.0	1.4
6	28.9	43.6	49.3	1.5
8	28.9	58.1	57.5	1.5
10	28.9	72.6	65.7	1.5
12	28.9	87.1	74.0	1.6
14	28.9	101.6	82.2	1.6

ft = foot
lb/ft = pound per foot

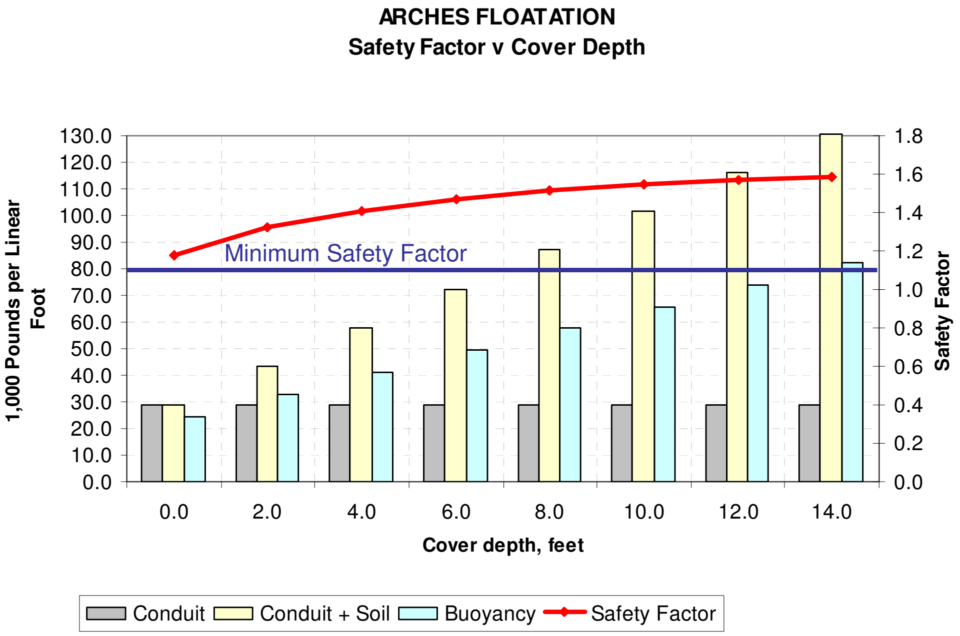


Figure 7: Arches – Depth of Cover

As discussed in Section 1.3, a weight to buoyancy ratio of less than 1 indicates that the pipeline would float. Figure 7 shows that without any cover the arches will not float, meeting the assumed safety factor.

Accounting for 6 feet of farming practices and no pipe cover, the total necessary depth of cover for the arches would be 6 feet.

2.4 Concrete Box Conduits

A floatation sensitivity analyses was also performed on the concrete box conduit alternative in terms of various cover depths. Table 7 provides the assumed design criteria for the boxes, while Figure 8 displays the layout of design.

Table 7: Box Conduit Design Criteria

Design Criteria	Value
Number of boxes	2
Box height, feet	20
Box width, feet	10
Wall thickness, inch	20
Top slab thickness, inch	18
Bottom slab thickness, inch	20
Total conduit width, feet	25
Total conduit height, feet	23

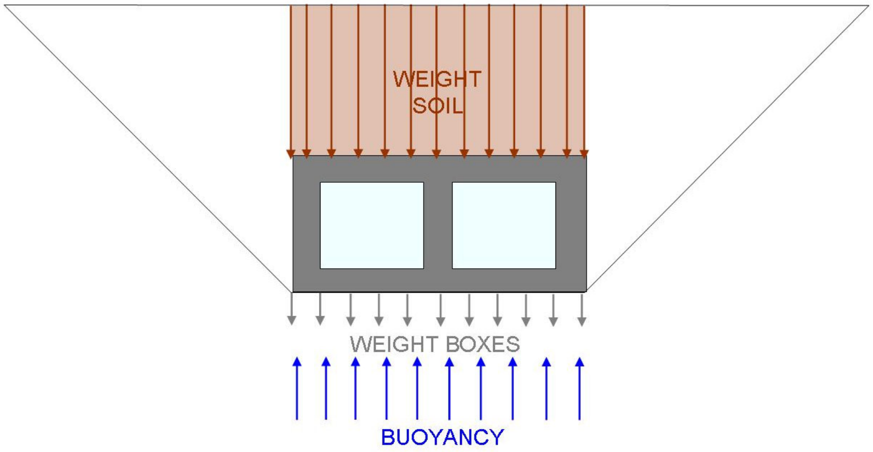


Figure 8: Conveyance Pipeline Trench

The weight of soil was calculated by assuming a rectangular mass of soil on top of the box conduit the width of the conduit (wc), and height equal to the depth of cover (D). The resulting area is then multiplied by the unit weight of soil, equal to 110 lb/cf specified in Table 1, resulting in lb/ft of soil.

Weight of Soil (lb/ft) = wc x D x WSoil

The weight of the conduit consisted of the weight of the cast-in-place concrete box subtracting the inner conduit boxes. The weight of conduit was determined by multiplying the wc by conduit height (hc), subtracting the area of the two boxes, box height (bh) multiplied by box width (bw), multiplying by the unit weight of concrete.

Weight of Conduit (lb/ft) =(wc x hc – 2 x (bh x bw)) x WConc

Buoyancy is equal to the weight of water displaced by the structure constructed, which equals the area occupied by the arches and slab multiplied by the unit weight of water, 62.4 lb/cf, resulting in lb/ft of buoyancy. The buoyancy of the soil making up the conduit cover must also be considered. This is determined by multiplying the area of cover by the unit weight of water.

Buoyancy (lb/ft) = (wc x hc + Cover Depth x wc) x WH20

2.4.1 Depth of Cover

Various depths of cover were investigated to determine the depth of soil necessary to counteract the buoyancy force and keep the pipelines in place. Table 8 and Figure 9 provide the results of this investigation.

Table 8: Box Conduit- Depth of Cover

Depth of Cover (ft)	Conduit Weight lb/ft (x1000)	Soil Weight lb/ft (x1000)	Buoyancy lb/ft (x1000)	Safety Factor
0	26.3	0.0	35.9	0.7
2	26.3	5.5	39.0	0.8
4	26.3	10.9	42.1	0.9
6	26.3	16.4	45.2	0.9
8	26.3	21.9	48.3	1.0
10	26.3	27.3	51.4	1.0
12	26.3	32.8	54.5	1.1
14	26.3	38.2	57.6	1.1
16	26.3	43.7	60.7	1.2
18	26.3	49.2	63.8	1.2

ft = foot
lb/ft = pound per foot

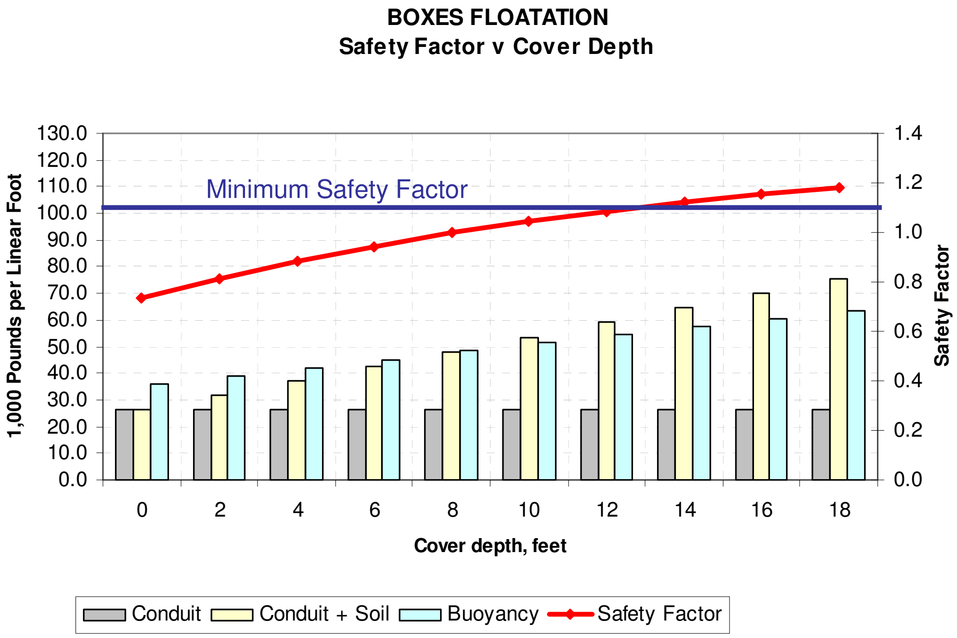


Figure 9: Concrete Box – Depth of Cover

A safety weight to buoyancy ratio of less than 1 indicates that the pipeline would float. As shown, a cover depth of less than 8 feet would allow the buoyant force to overcome the force of weight of the box conduits and floatation would occur. The figure also indicates that to achieve the minimum safety factor of 1.1, a cover depth of 12 feet is required, resulting in a necessary cover depth of 18 feet, including 6 feet for farming practices.

3.0 CONCLUSIONS

The floatation sensitivity analysis resulted in the following conclusions.

1. Concrete thickness variations affect floatation for cast-in-place circular pipes, whereas steel thickness has a minimal effect. Table 9 provides the thicknesses required to achieve the minimum factor of safety at various depths of cover for a 16-foot-diameter concrete pipe.
2. Each conduit type investigated would require varying cover depths to achieve the assumed safety factor. Table 10 provides these depths. It should also be noted that farming practices can cause disruption as deep as 6 feet. This is also accounted for in the following table.

Table 9: Required Thickness for Circular Concrete Pipe

Depth of Cover (feet)	Required Thickness to Prevent Floatation (inch)
0	32
4	28
10	16

Table 10: Required Depth of Cover

Conduit	Minimum Depth of Cover Required (feet)	Farming Depth (feet)	Total Depth of Cover (feet)
Concrete Pipe	10	6	16
Steel Pipe	16	6	22
Concrete Arches	0	6	6
Concrete Boxes	12	6	18

4.0 RECOMMENDATIONS

The preferred conduit minimum depth of cover is 10 feet. As shown in Table 10, meeting this design criterion may cause issues with floatation for several of the conduit types. The following floatation prevention alternatives will be further investigated and a preferred alternative will be identified:

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

Appendix G

Summary of Tunneling Contractor Workshops

DHCCP Tunnel Contractor Workshops

	Key Project Issues & Discussion							
Contractor	**Bouldin Island: Construction of four 40-foot ID tunnel headings**	**Tunnel drive length per reach and access to TBM along tunnel drive**	**Tunnel and Shaft Protection**	**Owner-furnished tunnel liner segment for the contractor/ Shaft construction**	**Contractor, operators, and equipment (TBM) availability**	**Contract delivery alternatives**	**Power requirements for TBM and site operations**	**CCIP vs OCIP / Bonding**	**PLAs, EPB, and Other Comments**
A	• Prefer to have two contractors with each one building one leg of the tunnel • Like the idea of one contractor sinking 4 shafts and turning it over • Prefer two contracts as one contract will get less competition • Prefer for owner to procure barging contract	• Consider intervention zones at one mile intervals as overkill; they are fine with two miles • 8 mile drives for EPB TBMs are acceptable • Use of the mid-drive access shaft location for major maintenance on the cutter head of shield brushes is a great feature	• Building a pad is the best option (as opposed to erecting barrier walls). • Suggested owner should pre-construct flood protection • Suggested doing a high/med/low risk analysis for equipment to be protected • Mound with ramp is a clever idea that cuts down on how many acres you need (about 3 acres for surface area)	• Owner should design as a way of quality control • Suggested owner look over "compressed negotiated design" Canadian approach • No need to have owner-furnished tunnel segments • Suggested segments be designed a bit thicker (add'l 2 to 3 in) than actually needed by strict calculations so that all of the rebar and other fab items and concrete consolidations could be done correctly • Agreed that segments would probably be barged	• Doesn't s see an issue in equipment (TBM) availability • The market is available and so are laborers • Considers a 6 month delay as adequate • Sureties will be the actual entity that "prequalifies" firms and JVs to participate in the work, since they won't bond firms or JVs that they don't have confidence in	• Prefer design bid build • This project isn't schematically complicated so other alternatives are not as beneficial	• Capable of building their own switchyard	• Favored CCIP • Owner needs a Safety Incentive Program • Get legislation to reduce performance bonding down to $500M, otherwise you will not be able to attract enough JVs to bid work	• Risk sharing: Get GBR as a baseline and take segment design liability from contractor • Don't like PLAs but not really opposed to the idea; they've seen more problems with PLAs • Prefer to have contractor input in PLA • Agreed this is most likely an EPB job • Political support: suggested getting AGC and SME/UCA involved in lobbying for project
B	• Prefer a single contract. • A $2.5B contract is not too big to handle • Did not like the idea of two contractors with each building one leg of the tunnel • If there are shared contracts contractor is concerned about managing flooding issues • If owner is considering two contracts then contractor strongly suggests setting up demarcation lines between the contracts	• Concerned about pressure issues between mile markers 2 and 4 • Recommend intervention zones should be located at one mile intervals • Contractor would like to inspect their cutter head every 1/2 mile. • 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 • Tunnel drive length of 8 miles is adequate.	• Suggested another option (aside from putting a 30-ft pad) would be to bring up a wall • Recommended owner elevate pad ahead of time with a different contract • Prefer owner to get a dirt guy and have protection ready by contractor's arrival (5	• Contractor would bid on a segment contract • Don't want to have design liability • No preference to having owner-furnished segments or to have contractor provide segments • 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 losw as $20 to $30/hr vs up to $60/hr for contractor wages • Providing the segments may be a good opportunity to use an alliancing contracting approach	• Suggested establishing a prequalification process for evaluating and selecting joint venture teams. Pay close attention to selection criteria such as: tunneling expertise, soft ground experience, number of years of experience • See no issue in operator and equipment availability • 6 month turn around on tunnel adverstisement/bid may be a bit tight if you have to do a new prequal process for each and every tunnel contract bid • Prequel JV teams, not individual contractors	• Did not see a benefit in using design-build project delivery on the tunnels	• Dewatering could be an issue as it would require a lot of power • See no problem in MWD bringing power to the site and contractor in control of distributing • To drive retrieval and vent shafts, contractors will need about 1MW of power at the job site. This needs to be provided in the form of a temp generator, or have pole-line power brought in. The current CEQA restriction on generator power at 300KW will not work.	• Prefer CCIP;they would have the smaller contractors sign up under the joint venture CCIP • Owner needs to make sure that the specification is clear.	• Risk sharing should be clearly spelled out. Include partnering, dispute resolution, provide incentives to resolve problems. • PLAs have more negatives than positives. Contractors want to be involved with union agreements • Identified this project as an EPB job
C	• Prefer separate contracts (smaller than 2.5 billion) • Prefer for one contractor to sink all 4 shafts • Concerned about issues in getting the muck up a vertical belt due to wetness (slump) might need to turn to slurry and pump it out OR squeeze out water • Recommended setting up shafts at north drive connecting tunnel to second shaft at south then allowing second contractor to go north from south shaft	• Would perform interventions every 1,000 ft to check their tools • Tri gas at 115' is required by CalOSHA for intervention • Would like to see safe haven every mile • Recommended having performance specs (ex: toughness of cutters) • Owner will not pay for shaft but will provide acces to the land • 40 foot per day is reasonable	• Prefer build up of pad prior to tunnel contractor coming in • Would need 7-10 acres per site for the pad if the contract required 2 contractors	• Owner's design eng has to set up a basic design for the shafts and ID about what can definitely NOT be done. Let contractors take it from there • Suggested oval shape for the shaft • 120 ft is max depth for slurry wall technology • Would not have a problem with owner providing segment • Contractor didn't seem in favor of taking the liability for segment design • Owner needs to be prescriptive about segment design	• A 6 month turn around on tunnel bids is acceptable only if everything is already designed • 6 month turn around not OK under DB • Prefer prequal for contracts • No problem with getting TBM manufacturers	• Prefer design bid build (DBB) • Owner should conform (standardize) all the contract specs from one bid to the next so that contractors can quickly prepare bids • Schedule shown during workshop is NOT doable if Design/Build is used. DB will take more time to execute than DBB	• 4.3MVA for the 20-footer shaft is low • Would prefer to have 600amp on site initially • Willing to provide information on power for ground freezing	• 33% bonding is acceptable • Contractor believes 50% of contract bonding makes sense • Contractor is flexible on either OCIP or CCIP • Equipement should be insured on builder insurance	• Clay and sand will mix at the cutter head making it perfect material for EPB • Suggested ground freezing, and talking to companies in the ground freezing industry • Don't see advantages in PLAs • They already cover a lot of PLA in union agreement, so there is no added benefit with PLAs • Would prefer for work sites to be "ready to go" when they arrive, have all the site prep out of the way so they can start work on shafts right away

DHCCP Tunnel Contractor Workshops

	Key Project Issues & Discussion								
Contractor	Bouldin Island: Construction of four 40-foot ID tunnel headings	Tunnel drive length per reach and access to TBM along tunnel drive	Tunnel and Shaft Protection	Owner-furnished tunnel liner segment for the contractor/ Shaft construction	Contractor, operators, and equipment (TBM) availability	Contract delivery alternatives	Power requirements for TBM and site operations	CCIP vs OCIP / Bonding	PLAs, EPB, and Other Comments
D	• Don't like the idea of one big contract. This will limit the number of bidders • Agreed with option where one contractor completes both shafts on the north or south sides and 'jumps' to the shaft on the other end, vacating the previous shafts for the other contractor to launch their TBM in the opposite direction	• 2 mile safe haven is a problem, consider safe haven at mile intervals • Recommended a minimum safe haven work area on the surface that is 4 tunnel diameters on each side of the TBM X 4 tunnel diameters in front of the TBM • Suggested we need to provide 200 feet of depth for jet grouting	• Recommended making the shaft pad contract one of the early contracts to help tunneling contractor focus on their tunneling efforts • Prefer pad to be built by owner and for contractor to take care of blue sky events	• Suggested we look at the shaft at Sangat on the Chanel tunnel for a good example of large shaft construction • Proposed building the shaft above ground and mucking out the center of the shaft, allowing it to sink into the ground. This concept is used in Chicago for 30' to 40' shafts • Agreed with the idea of Contractor-furnished liner while design is provided by owner	• 6 months between jobs for bidding seems OK, but no less • Prequal would be beneficial • Estimated that a contractor will have 70 to 100 people per reach (per billion dollars of construction) • There are sufficient number of TBM operators, therefore not as critical as the number of superintendents (150 for the project) and their assistants. • Leave purchase of TBM to contractor • After 8 miles of tunneling, the TBM has reach its useful life	• Suggested owner consider providing stipends if a DB contract is chosen. For the size of BDCP, a $2-3 million stipend would be considered low • Recommended contractor short list at 3 competitors • Suggested this project should be DBB • Consider CM/CG to get the contractor in early	• Suggested owner consider using natural gas powered generator for first shaft , consider package generator units that run with natural gas • Suggested owner look into natural gas system both fixed and packaged system • Tier 4 engines-emission of these machines and size limitation need more investigation	• Projects that exceed 5 years will have higher bonding costs • Suggested we have discussions with bonding companies • Don't have a preference for insurance type, but they need to price the risk • Suggested owner talk to insurance groups for more information • Owner needs to clearly define what is covered under the insurance	• Recommended EPB system • Slurry TBMs require less power than EPB but the extra equipment associated with a slurry system required more overall power than an EPB operation • Leave permits to owner as contractor's dislike getting permits • Suggested that owner set up early site work contracts and power contracts • Recommended a PLA; get AGC to help negotiate • PLA will prevent shut downs and promote labor harmony
E	• Large muck disposal sites are convenient, owner should consider separate muck handling contracts • Suggested that owner provide 1,000 ft between the north and south pad • Contractor can handle one single $2.3 B contract but owner would end up with less bidders • One contractor per leg is doable but there are concerns that one tunnel could laterally affect the other tunnel	• Tunnleing 200 ft/week is reasonable • Depth of the tunnel the at -150 ft is acceptable • Contractor has been successful at changing cutting tools at greater than 5 bar w/out hyperbaric intervention • Owner should consider having safe zones every mile • Owner should consider closer spacing on the south for the safe zone because of higher abrasive soils • Estimated that a contractor would require at least a couple of months for creating a grouted safe haven; ground freezing is an option but grouting is more cost effective • 8 mile drives are doable	• Owner should consider preloading the site (early site work contract) • Prefer to have all their equipment (segments, etc) on the pad so they would need about 20 acres	• Shaft sizes are doable • Contractor would take out TBM and refurbish them, it takes the same time as destroying the TBM • Suggested using secant piles or slurry walls to build the shafts • Would prefer to have the segments manufactured onsite near the tunnel launch shaft • Other representatives from this contractor would prefer to have the segments built off site and delivered b/c a new segment plant would need to pay prevailing wage • Because of the tunnel tension issues, contractor recommended owner should have control of liner design to ensure uniform design • Would prefer to have their own segments; they like to have control of their work • Estimated they would require 15-20 acres for tunneling and conveyor equipment work area	• 18 months for TBM procurement is tight, they feel we don't have any float • Owner should consider 2 years for construction of shafts and TBM • There are big projects being planned that may make it more difficult to get the required resources and the longer we delay the greater the competition for these resources • Equipment is available and should not be an issue • Suggested we consider refurbishing used machines • Will need to get operators from all over the region	• They feel comfortable with DBB • Recommend owner get industry review of plan before going out for bid • Contract delivery depends on when we need to know the final price of the job.	• For construction of shafts 1Megawatt is OK • Contractor would provide the necessary noise, mufflers, and particulate traps for the temporary site power generator	• Bonding a $1.5 B is achievable but a larger ($2.5B) project is more challenging • Owner needs to start talking about establishing a PLA, talk to AGC • Approach all labor unions • To avoid problems be proactive (anticipate problems before they occur)	• Owner should consider letting early contract s for site work, barge landing, and possibly shafts • They felt there are many advantages to have the site ready in advance of the tunnel contractor arriving on site • Prefer owner get permits • Concerend about ground settlement being a big risk issue • Owner needs to determine when is price needed. With DBB the price is determined later in the project. DB provides earlier price • Other risks include: flooding, GBR, gas • To minimize risk provide as accurate GBR as possible
F	• Prefer for one contractor to sink all 4 shafts and handing off to the second contractor • Having side by side contractors will affect each other's drives	• Owner should consider putting shafts under smaller contracts • Need safe haven every mile for ground surface improvement and approx 1 acre • No issues with 8 mile conveyor belt • 30 to 35 ft/day tunneling rate is more reasonable maybe at least for the first few contracts because there is a learning curve	• Pad for flood protection will settle so preloading should be done a few years ahead	• Contractor has the ability to build segment batch plant on site. They'll do a cost analysis to see if it's better to ship them in or produce onsite. Their preferncе is to cast segments on site. • Sharing a barge landing with other contractors shouldn't be a problem. • They are fine with owner providing lining and even providing segments but owner needs to consider what's most cost effective & schedule because sometimes the TBM outruns segment production	• The 6 month stagger is doable • Contractor packages/procurements should be similar in scope or else it could delay project • Most JVs will consist of 3-4 teams so reorganizing teams will take time • There are enough people for the job but if the highspeed rail is starting around the same time it might be a problem • This will be a global project so this will attract internation expertise	• On DB a 6 month stagger won't work • Owner should provide substantial stipend if they want contractors to design • They prefer DB	• Each contractor should have their own substation • Give contractor power at higher voltage so contractor decides how to step down	• Owner needs to talk to brokers. It might be worthwhile for owner to take risks for what-if events • Risk allocation is very important, it should include shaft protection • Prefer CCIP • Bonding might limit competition if kept at 100%	• They are fine with PLAs. • They want a project/program specific PLA b/c sometimes the language ties PLA to other jobs • They would like to negotiate their shifts so leave that as open as possible • Metro shared their prelim GBR/GDR with contractors and it proved to be helpful

DHCCP Tunnel Contractor Workshops

	Key Project Issues & Discussion								
Contractor	**Bouldin Island: Construction of four 40-foot ID tunnel headings**	**Tunnel drive length per reach and access to TBM along tunnel drive**	**Tunnel and Shaft Protection**	**Owner-furnished tunnel liner segment for the contractor/ Shaft construction**	**Contractor, operators, and equipment (TBM) availability**	**Contract delivery alternatives**	**Power requirements for TBM and site operations**	**CCIP vs OCIP / Bonding**	**PLAs, EPB, and Other Comments**
G	• No problem with having two contractors but there has to be independence • They suggested splitting East and West and having a contractor on each tunnel leg • They can do (and like) Mega projects but there are operational challenges (resources, phasing, schedule) and commercial challenges(procurement,bonding, capital) • If contractor sinks 4 shafts , they see trouble in setting up and disassembling TBM	• They think owner's biggest challenge will be the bottom plug • They think freezing makes sense but a lot of power is needed for such a long duration • They think owner should specify a minimum number of interventions • They would use safe havens to perform interventions	• Need early contracts for site prep	• 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 great quality • Flexibility is necessary if there is onsite production • In terms of segment design, the final design would be up to the owner • It makes sense for segments to come from owner because it's such a huge issues and there's a lot of contractor risk but owner should consider how this will affect the build time	• They think 18 months for TBM procurement is reasonable • There is TBM availability, we have enough resources but this requires a fair lead time. At least 12 months for production and 6 months for assembly and shipping • Owner is going to need a lot coordination with the local union	• There has to be a balance in risk-sharing; They've shied away from jobs that want contractor to take all the risk. • They prefer early contractor involvement in design; get contractors and designers together • They recommend looking at CM/GC approach for tunnel program to get contractors involved early		• Owner needs to get in touch with bonding companies ahead of RFP and RFQ • They recommend talking to insurance brokers • They are neutral on which insurance to get, but owner probably won't see savings from OCIP	• Owner should prevent contractors competing for people by covering a lot of those details in the PLA • They would like to be involved in the PLA negotiations • They suggested putting out a draft PLA to get comments on it • They like the idea of a draft GDR/GBR and knowing Geotech in advance • Contractors would support legislation but they need to do it though the AGC
H	• They prefer having one contractor sink all 4 shafts versus one big contract • A good separation between the N/S contractors would be 500-1,000 ft • They like the idea of having a separate contractor for muck handling (it would be an opportunity for more small business involvement)	• They think 40ft/day tunneling rate seems reasonable • There's no problem with 8 mile drive, machines can certainly last that distance • A contractor can decide if they need a safe haven • 2 miles for safe haven seems OK but they need more info for contractor to decide, it's too early to tell	• They think 3-5 acres for the pad is about right • They would focus on protecting their electrical gear	• They are OK with owner supplying the segments	• This project will kick off right when the Highspeed rail starts, so man power might be tight • Machines are available but getting professionals (especially in CA) might be an issue • Smaller contractors might be wary of jumping into a big project like this, we're probably looking at more national/int'l contractors • There's no one firm that's going to supply all staff - probably 2 to 3 JVs • They wouldn't want to do a prequal for all contracts, it'd be nice to do it once. Prequality ahead of time.	• Owner should streamline contracts, have similar language • DB and CM/GC might be difficult with this program • There are advantages to bringing contractor in early • Owner should consider breaking down program into pieces and have the first contract as CM/GC		• Bonding shouldn't be an issue; the market has grown and matured • The length of the project could create an issue • They suggested owner talk to bonding companies	• They are OK with PLAs, it's pretty standard for most big programs • They are not fans of OCIPs • Make sure PLA allows people to be brought in from out-of-state or out-of-country • Risk allocation depends on GBR • The Highspeed rail workshop with MTA was very helpful. It was an open forum, multi-day session with consultants and contractors; about 30 tunnel people, 5 from parsons, 2 from MTA • They would appreciate another opportunity to get together either individually or as a group
I	• It's very ambitious to think two contractors will be able to work together • They'd prefer to have one big contract, they're seeing plenty of 1.5B dollar contracts	• They think safe havens every 2 miles is pushing it. There will be 12-15 interventions before you get to two miles.	• They like the idea of early contracts for site prep work	• Owner should build segments. Contractor doesn't want that responsibility • The price to contract vs owner would not be that much different • There is no drawback with respect to limitation on TBM design • They prefer casting on-site	• Equipment shouldn't be an issue • Manpower might be an issue because of required skills and other concurrent projects • 18 months is enough to sink the shaft if you have power, water there already • 6 months is tight but doable as long as there is no delay	• They recommend having specifications workshop • They would also like to see a draft of RFPs • It's more important to select the contractor you're comfortable with • They are comfortable with DBB but it limits owner's ability to select by best value so it's not really their preference • DB allows you to have selection criteria . It gives contractor innovation but this project has a very specific liner design so that limits contractor innovation		• If it's DBB - They don't think the entire job needs to be bonded. you don't need 100% • They are fine with either CCIP or OCIP. CCIP is for general liability and worker's comp. OCIP is for general work. • Contractor's generally manage their claims better • In general, OCIP will be lower cost	• Risk allocation is very important • Their preference is to prequalify before bid • PLAs are not a big deal but the contractor has to be able to assign work. You need a good dispute resolution

DHCCP Tunnel Contractor Workshops

	Key Project Issues & Discussion								
Contractor	**Bouldin Island: Construction of four 40-foot ID tunnel headings**	**Tunnel drive length per reach and access to TBM along tunnel drive**	**Tunnel and Shaft Protection**	**Owner-furnished tunnel liner segment for the contractor/ Shaft construction**	**Contractor, operators, and equipment (TBM) availability**	**Contract delivery alternatives**	**Power requirements for TBM and site operations**	**CCIP vs OCIP / Bonding**	**PLAs, EPB, and Other Comments**
J	• Don't put contractors and shafts too close to each other • They like the idea of one contractor taking a leg. • They also like the idea of a big contract but they think bonding will be an issue. One contract would make muck disposal easier.	• 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 barge landing for each contractor • They are not very concerned about access. They feel they can work from inside the tunnel.	• They like the idea of having early contracts for site prep work • They feel the ultimate responsibility (in case of flood) should be with owner • Need 2-3 acres minimum for elevated pad. The more area available, the better it is for them.	• 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 load 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.	• 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. • Man power availability will be challenge but they can train people	• DB would take longer than DBB • CM/GC not appropriate for this type of project • DBB is the best approach	• 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	• They'd rather manage their own losses so they prefer CCIP • Owner should take care of Flood Policy • Owner should contact bonding brokers for more information	• The highspeed 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.

Appendix H

Hydraulic Analysis Technical Memorandum

AECOM

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FINAL DRAFT (Rev 4)

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.

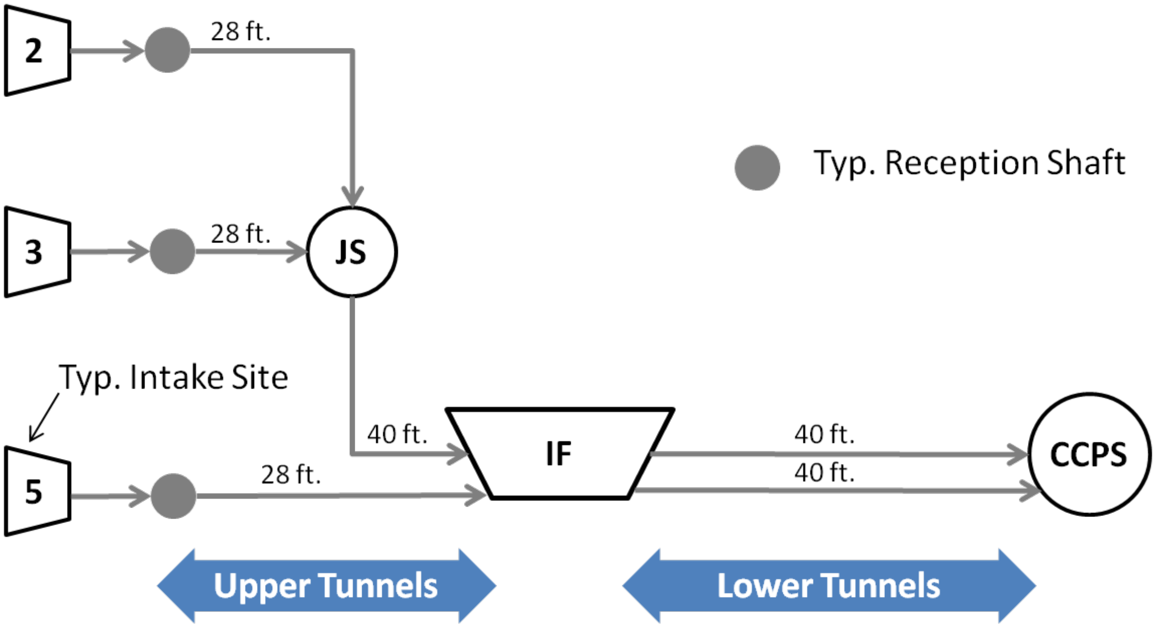


Figure 1 – Existing As-Proposed General Arrangement

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 (ex., 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

Parameter	Intake 2	Intake 3	Intake 5
Screen Size (W x H)	15'-7" x 12'-6"	15'-7" x 17'-0"	15'-7" x 12'-6"
Number of Screens	90	66	90
Screen Invert[1]	-10.0	-15.0	-11.0
Intake Pipe Invert	-9.0	-9.0	-10.0

[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:

$$h_s = K_s \left(\frac{t}{b}\right)^{\frac{4}{3}} \frac{v^2}{2g}$$

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 foot-print 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.

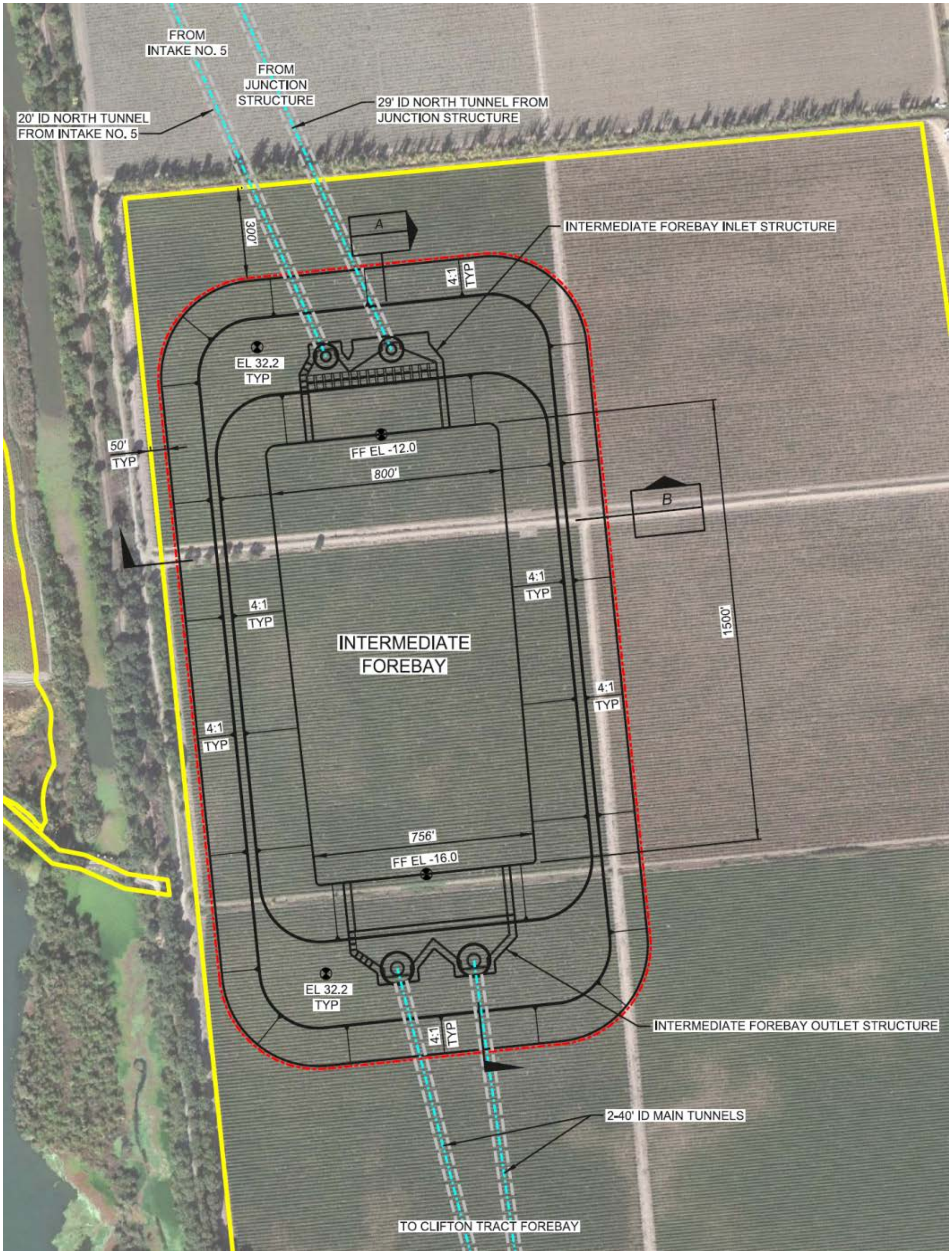


Figure 2 – Intermediate Forebay Refined Layout

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.

①	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.
②	System hydraulic capacity is limited by the 0.2 fps intake screen approach velocity limit discussed in Section 2.3.1 above.
③	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.

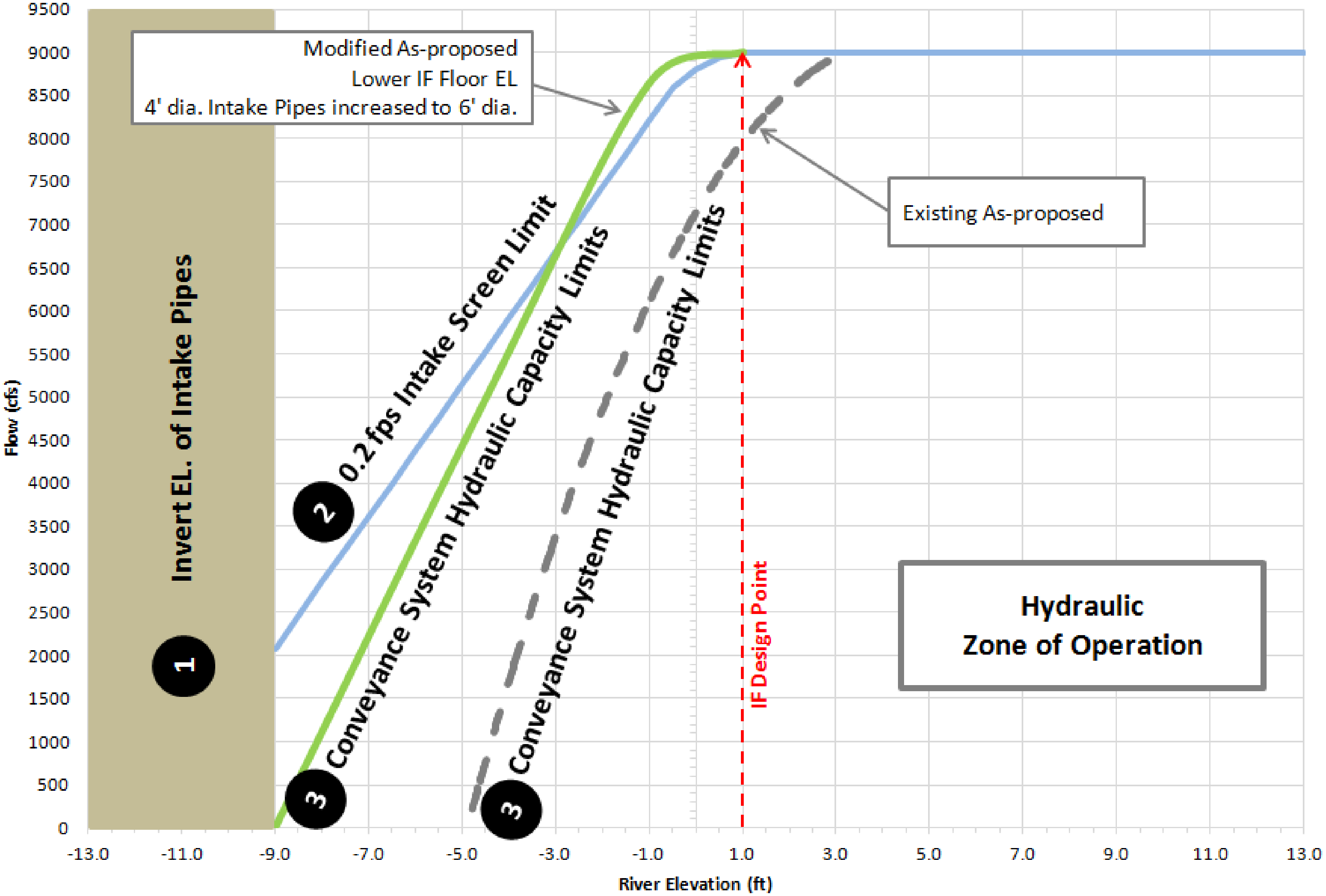


Figure 3 – As-Proposed Hydraulic Conveyance System Capacity Curve

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 signification 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=CdA*sqrt(2gh)) to calculate a representative percent gate opening. Cd=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 Alternatives1, 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

Run ID	Description	Intake River Elevation			Upper Tunnel Diameters				Intake Flow Rates (CFS) Unregulated by 0.2 fps Rule			Percent Gate Opening[1] of All 12 Gates to Balance Flows[2] Regulated by 0.2 fps Rule			Flow[2] 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)			Fish Screen Approach Velocity[5] at Peak Flow During Emergency Intake 2 Closing (FPS)		
															Upper Tunnels					
		Intake No. 2	Intake No. 3	Intake No. 5	Intake 2 - JS	Intake 3 - JS	JS - IF	Intake 5 - IF	Intake No. 2	Intake No. 3	Intake No. 5	Intake No. 2 % Gate Opening / Flow (CFS)	Intake No. 3 % Gate Opening / Flow (CFS)	Intake No. 5 % Gate Opening / Flow (CFS)	Intake 2 - JS	Intake 3 - JS	Intake 5 - IF	Intake No. 2	Intake No. 3	Intake No. 5
Ex.	As-proposed design with new IF layout and an increase of intake piping from 4' to 6'.	1.0			28	28	40	28	2,465	3,074	3,461	100% 3,000	13% 3,041	9% 2,982	3,000/0	3,041/3,603	2,982/3,016	0.00	0.22	0.19
Ex.1	Same as Ex. but at a higher river EL.	10.0			28	28	40	28	2,427	3,140	3,433	100% 3,000	12% 3,000	8% 3,000	3,000/0	3,000/3,961	3,000/3,286	0.00	0.23	0.19
Ex.2	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., 3,603 CFS) at Intake 3 observed during the Ex. run ID.	1.0			28	28	40	28	2,465	3,074	3,461	100% 3,000	13% 3,041	9% 2,982	3,000/0	3,041/3,603	2,982/3,016	0.00	0.19	0.19
Alt.1	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 40'.	1.0			30	22	40	25	2,964	2,999	3,037	100% 3,000	19% 3,000	17% 3,000	3,000/0	3,000/3,629	3,000/3,039	0.00	0.22	0.18
Alt.2	Same as Alt.1 but first reduced the upper tunnel diameter between the JS and IF by 20% and then adjusted the upper tunnel diameters to provide a better flow balance between the intakes at 9,000 cfs.	1.0			30	22	32	22	2,952	2,985	3,063	100% 2,999	0% 3,000	14% 3,001	2,999/0	3,000/4,319	3,001/3,038	0.00	0.26	0.18
Alt.3	Same as Alt.1 but IF floor elevation raised to EL -13 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.	1.0			40	30	40	28	2,831	2,849	3,320	100% 3,009	0% 3,026	11% 2,962	3,009/0	3,026/3,951	2,962/3,151	0.00	0.24	0.19

Notes

NC = Not Calculated

[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=CdA*sqrt(2gh)) was used to calculate a representative percent gate opening. Cd=0.8 (typical est. for radial gates) and h is the corresponding headloss 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' (H x W).

[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 EL 1.0 pulling 9,000 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% fouling, and using wetted gross screen area.

[6] HGL elevation immediately upstream of tunnel interface with junction structure.

Table 2 – Summary of Simulations (continued)

Run ID	Steady State Junction Structure Elevations (FT)[6]			Steady State Intermediate Forebay Elevations (FT)				Steady State CC Surge Shafts WSEL (FT)		Steady State Headloss (FT) Values Prior to Intake 2 Closure (All intakes open at 9,000 CFS; Gates used to balance flows between intakes)									
	Intake 2 Tunnel Interface	Intake 3 Tunnel Interface	Intake 2 Tunnel Interface	Floor EL[3]	All Intakes Open (9,000 CFS)	Only Intake 2 Closed (6,000 CFS)	[4] Pumping Drawdown (FT) / Min. IF Level (FT)	All Intakes Open (9,000 CFS)	Only Intake 2 Closed (6,000 CFS)	River --> Downstream Side of Roller Gates			Across JS		IF		Tunnel Losses		
	All Intakes Open (9,000 cfs)	All Intakes Open (9,000 cfs)	Only Intake 2 Closed (6,000 cfs)							Intake No. 2	Intake No. 3	Intake No. 5	Via Intake No. 2	Via Intake No. 3	Entrance	Exit	Via. Intake No. 2	Via. Intake No. 3	Via. Intake No. 5
Ex.	-3.35	-3.27	-6.71	-15 US / -19 DS	-10.6	-8.2	-1.8 / -12.4	-19.8	-12.2	0.99	2.71	5.75	1.85	1.96	1.41	0.30	7.25	5.44	4.40
Ex.1	6.14	6.12	3.19	-15 US / -19 DS	0.2	2.0	-1.5/-1.27	-8.7	-2.0	0.90	3.09	5.10	1.82	1.80	0.16	0.06	6.88	4.71	4.50
Ex.2	-3.35	-3.27	-9.49	-15 US / -19 DS	-10.6	-11.3	-2.0/-12.6	-19.8	-15.4	0.99	2.71	5.75	1.85	1.96	1.41	0.30	7.25	5.44	4.40
Alt.1	-2.58	-2.57	-5.98	-14 US / -18 DS	-10.1	-7.5	-1.9 / -12.0	-19.3	-11.5	1.09	1.09	1.37	1.98	1.99	1.60	0.34	6.40	6.40	8.10
Alt.2	-2.56	-2.52	-6.59	-22 US / -26 DS	-18.7	-11.1	-1.3 / -20.0	-27.9	-15.1	1.08	1.03	2.15	1.79	1.83	1.91	0.39	14.90	14.91	15.62
Alt.3	-1.04	-1.02	-4.11	-13 US / -14 DS	-7.9	-5.6	-1.6/ -9.5	-17.3	-9.7	1.10	1.04	3.48	1.83	1.85	1.05	0.58	4.92	4.96	4.39

<u>Notes</u>

NC = Not Calculated

[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=CdA*sqrt(2gh)) was used to calculate a representative percent gate opening. Cd=0.8 (typical est. for radial gates) and h is the corresponding headloss 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' (H x W).

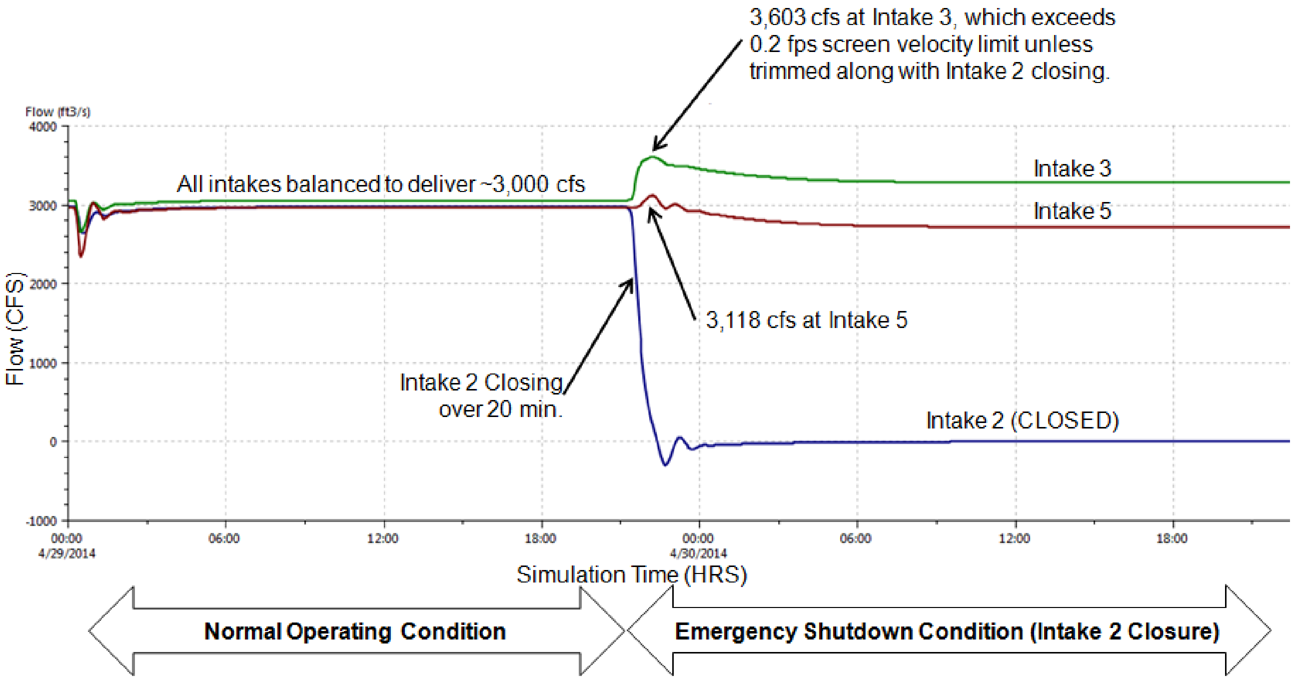
[2] Flow balance provided to within +/- 15 cfs of 3,000 cfs.

3 Pumping volume accounted for but allowance for a freeboard is not included, nor recommended for river EL 1.0 pulling 9,000 cfs.

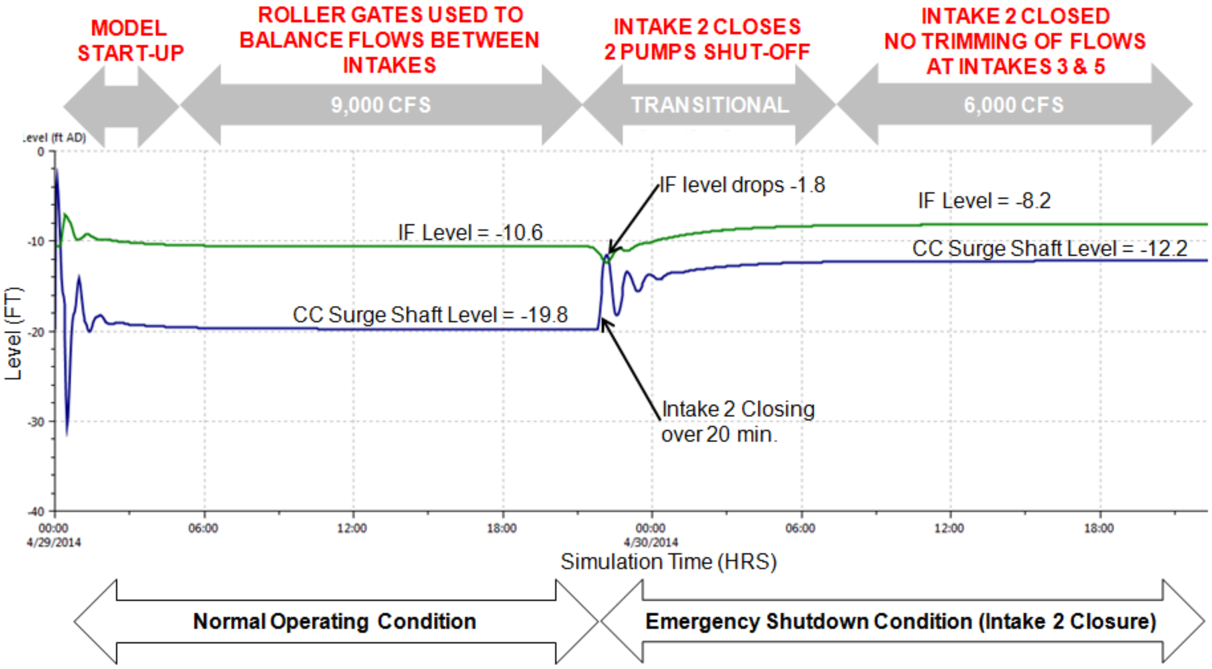
[4] Assumes pumps are controlled using a level set point at the IF for a given pump flow rate. This simulates a maximum drawdown in the IF caused by the closure of intakes gates (upstream control) as the pumps react to the dropping level and stage off. In practice, pumps may be controlled based on flow 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% fouling, and using wetted gross screen area.

[6] HGL elevation immediately upstream of tunnel interface with junction structure.

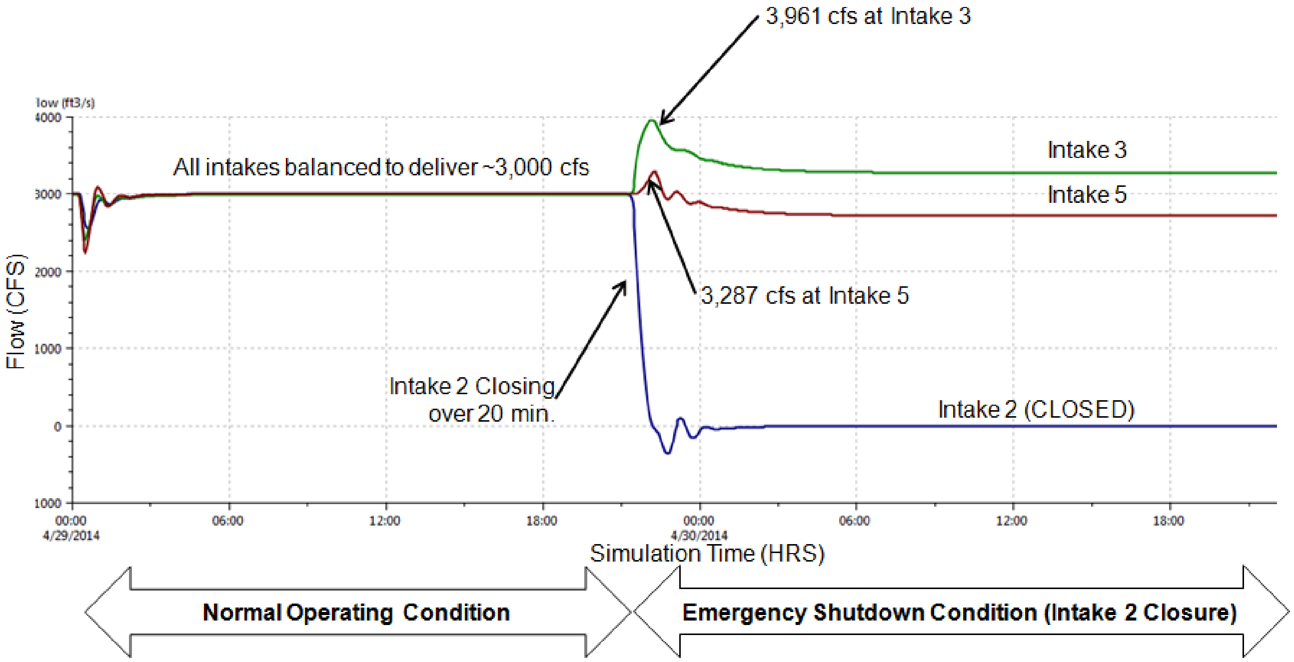


(a) Flow History at Intake Reception Shafts

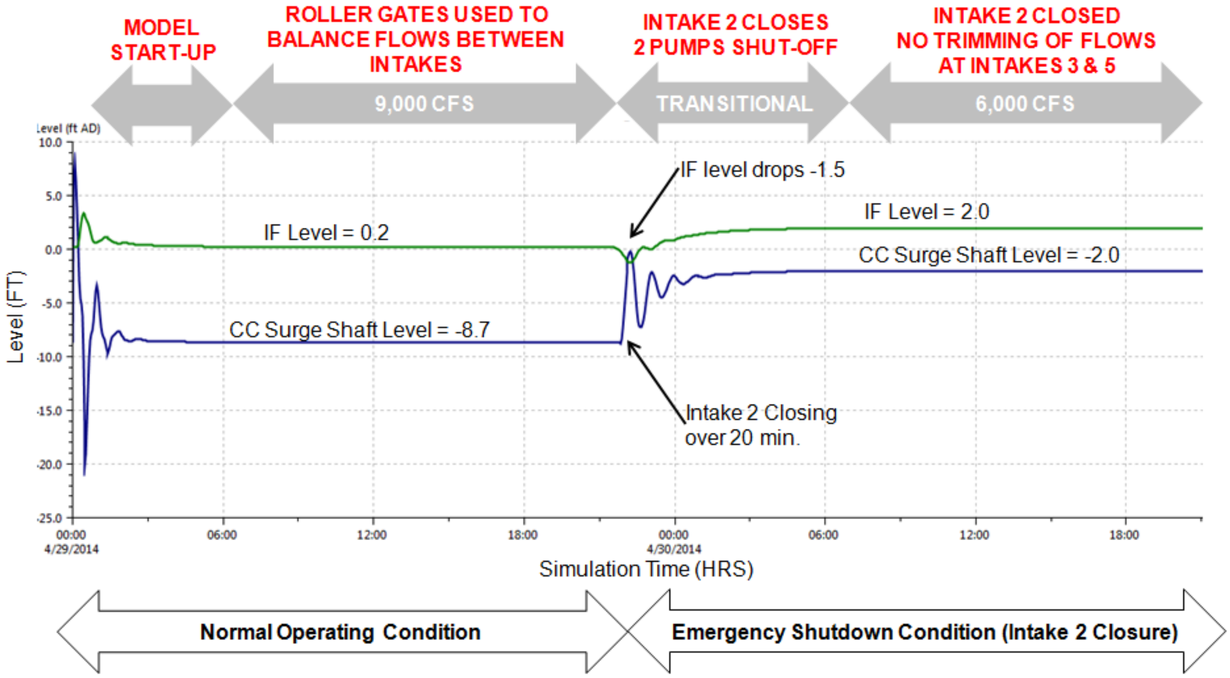


(b) Level History at IF and CC Surge Shafts

Figure 4 – Run ID Ex.

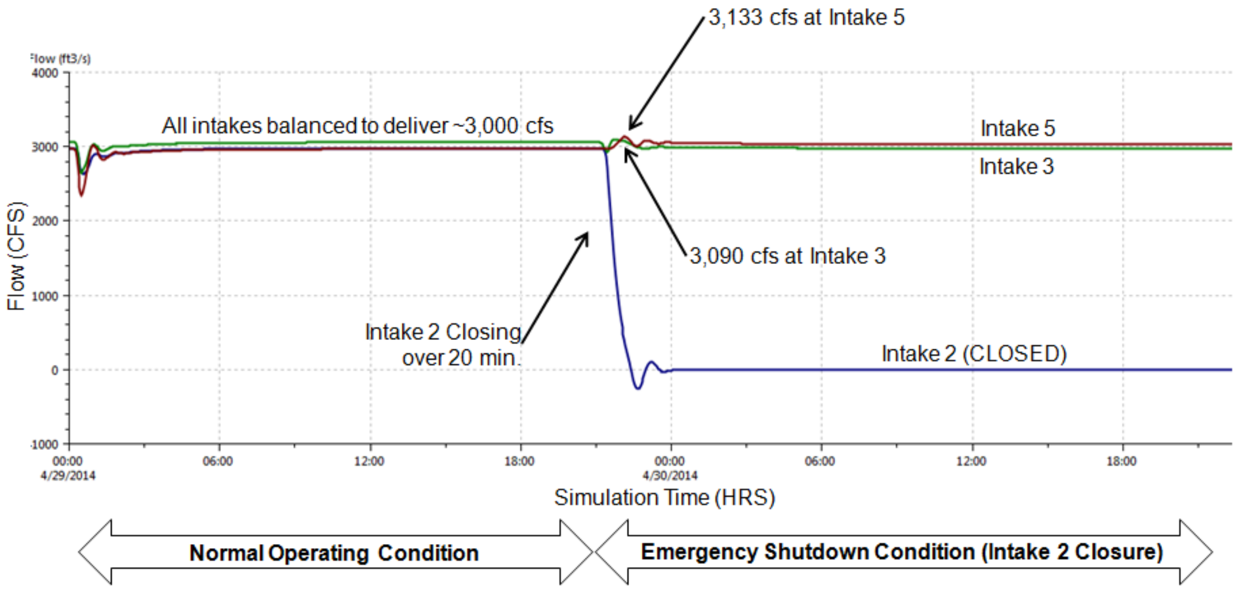


(a) Flow History at Intake Reception Shafts

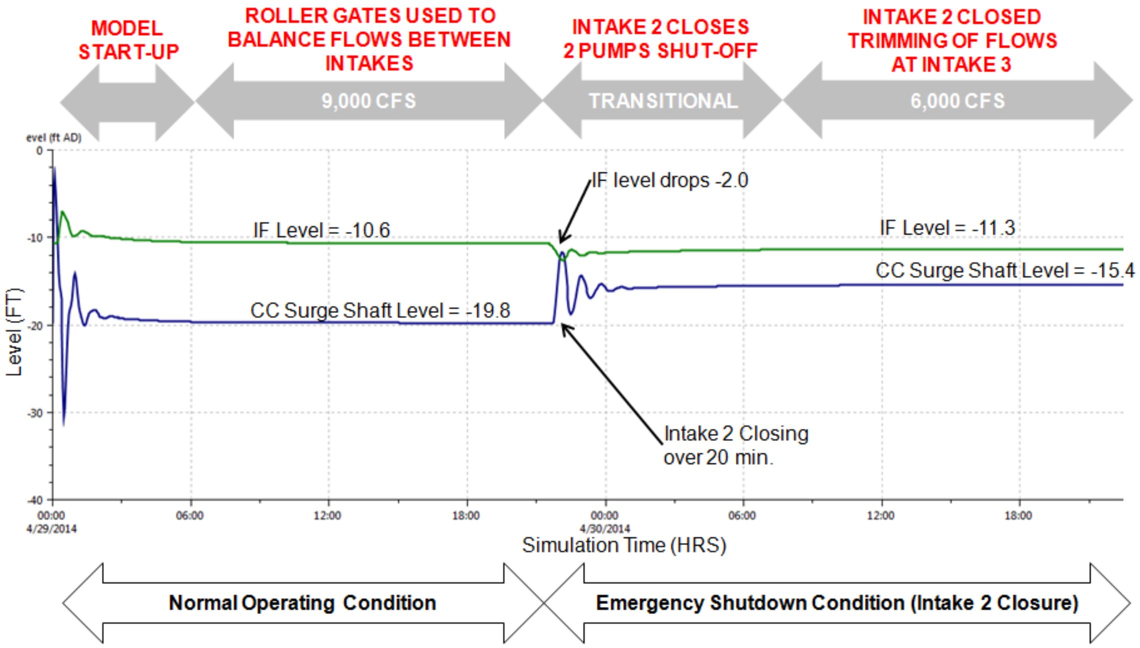


(b) Level History at IF and CC Surge Shafts

Figure 5 – Run ID Ex.1

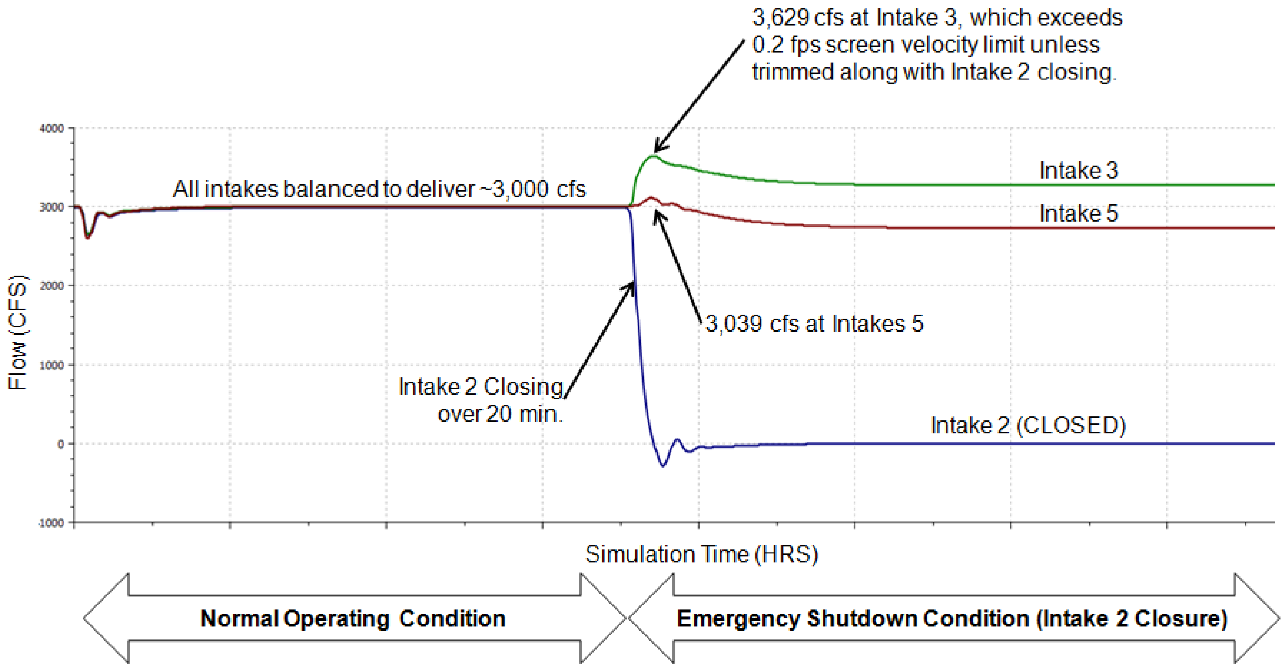


(a) Flow History at Intake Reception Shafts

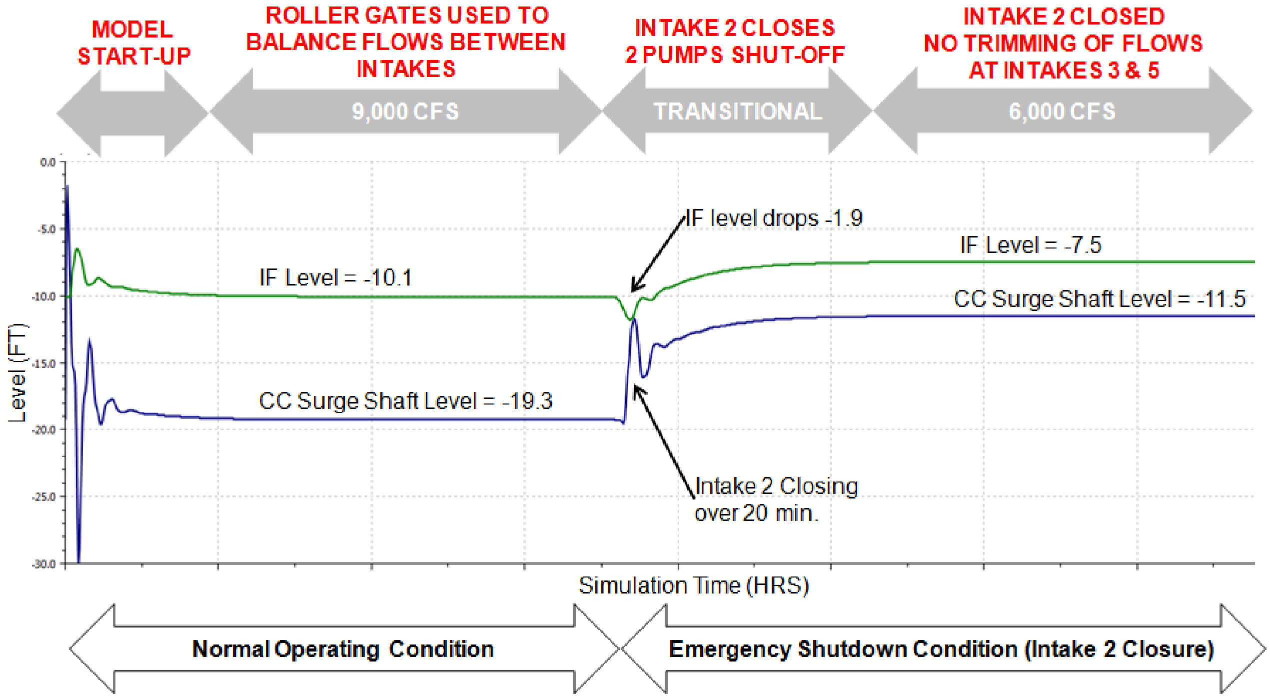


(b) Level History at IF and CC Surge Shafts

Figure 6 – Run ID Ex.2

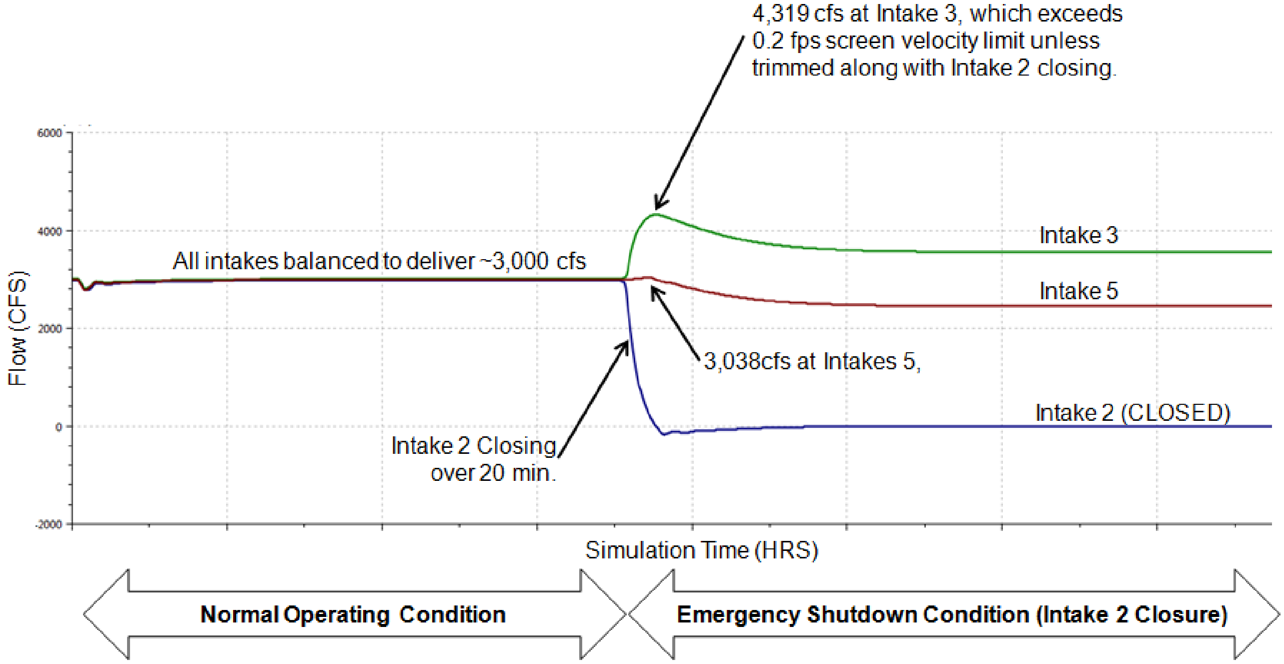


(a) Flow History at Intake Reception Shafts

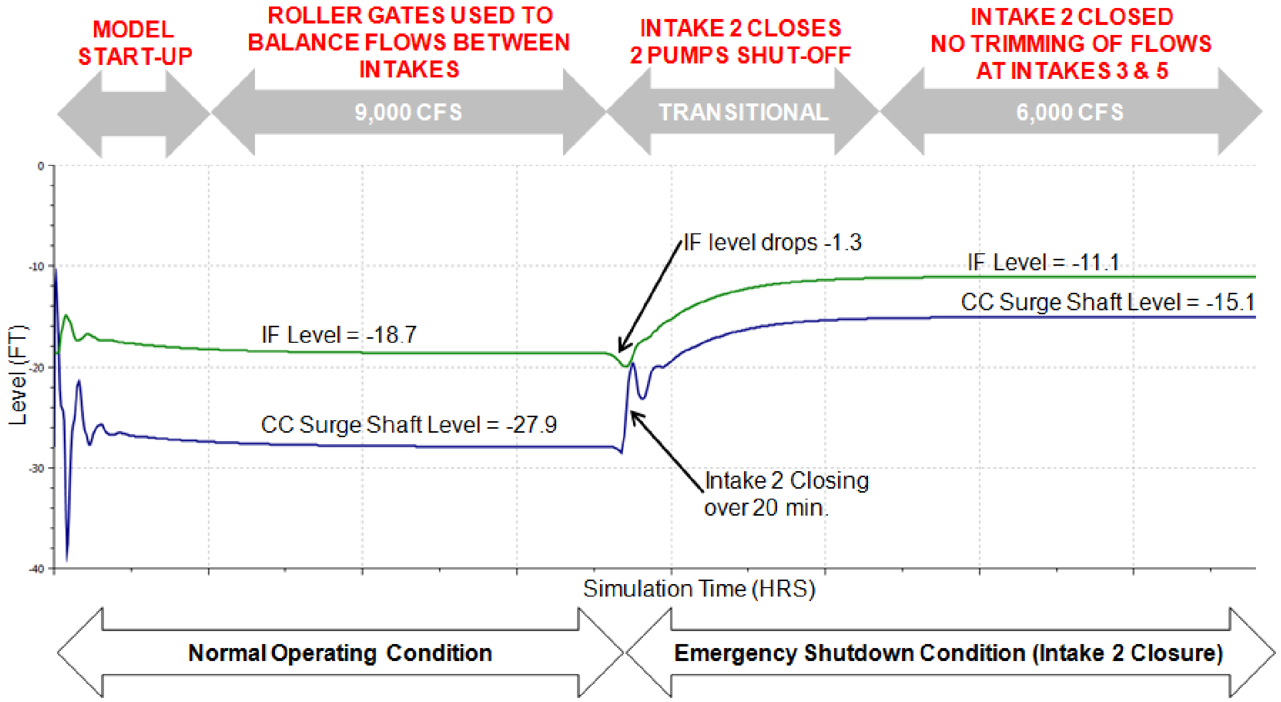


(b) Level History at IF and CC Surge Shafts

Figure 7 – Run ID Alt.1

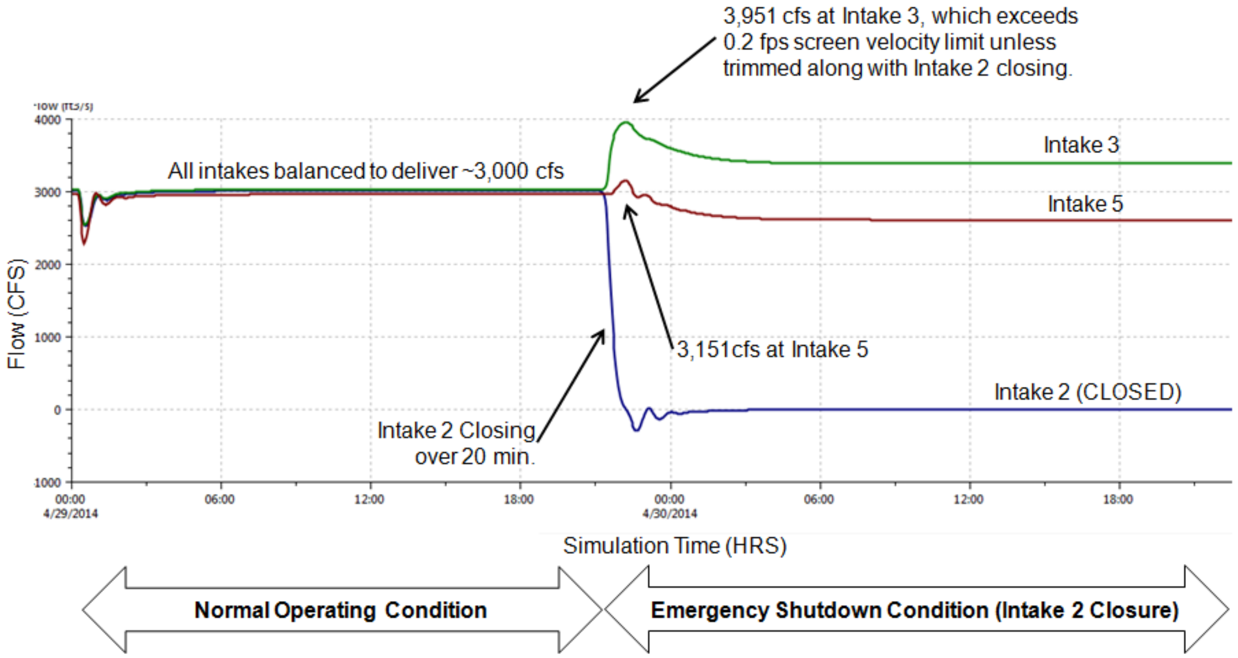


(a) Flow History at Intake Reception Shafts

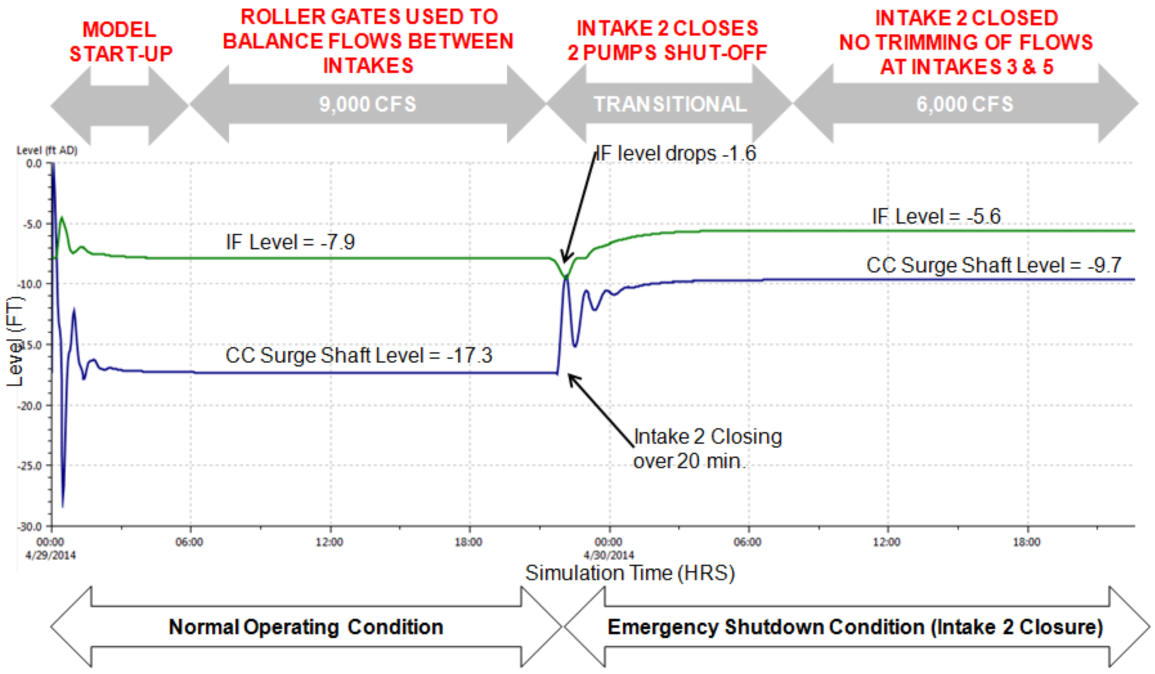


(b) Level History at IF and CC Surge Shafts

Figure 8 – Run ID Alt.2



(a) Flow History at Intake Reception Shafts



(b) Level History at IF and CC Surge Shafts

Figure 9 – Run ID Alt.3

Appendix I

Conceptual Design of Tunnel Segmental Lining System

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 1
Baseline for Comparison of Options to Resist Internal Pressure

	Internal Diameter	
Tunnel Feature	**40 feet**	**28 feet**
Thickness	22 inches	18 inches
Length	6 feet	6 feet
Segmentation	8 segments plus key 3 dowels/dowel positions per segment 1 dowel/dowel position on the key 25 dowel positions overall	7 segments plus key 3 dowels/dowel positions per segment 1 dowel/dowel position on the key 22 dowel positions overall
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

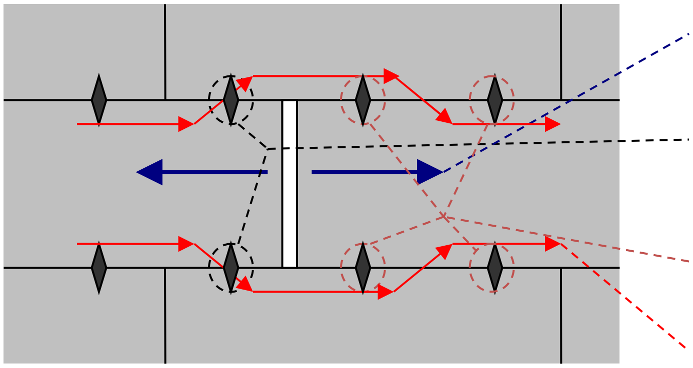
It is not straightforward to calculate a minimum design pressure because most analytical techniques used to establish the soil pressures on a lining are aimed at determining the upper-bound—or maximum—design pressures. Any reliance on soil pressures would require instrumentation to demonstrate that the 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



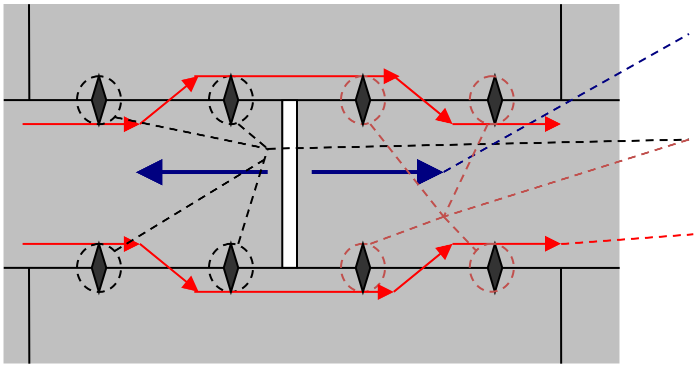
1. Opening created by segments moving apart circumferentially

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



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
Key Features of the Dowel System

Feature	40-foot-diameter Tunnel	28-foot-diameter Tunnel
Pressure	20.0 feet net internal	15.5 feet net internal
Tension per ring	718 kN (161 kips)	394 kN (88 kips)
Dowel pairs on side 1	2	1

TABLE 2

Key Features of the Dowel System

Feature	40-foot-diameter Tunnel	28-foot-diameter Tunnel
Shear on dowel on side 1	179 kN (40 kips)	197 kN (44 kips)
Displacement of dowel pair on side 1	3.1 mm	3.4 mm
Dowel pairs on side 2	2	2
Shear on dowel on side 2	179 kN (40 kips)	98 kN (22 kips)
Displacement of dowel pair on side 2	3.1 mm	1.5 mm
Combined tolerances on dowel location	2.0 mm	1.0 mm
Total worst case opening	8.2 mm	5.9 mm
Required test pressure	1.2 bar	0.9 bar
Radial (outward) deformation	9.7 mm	7.1 mm

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

Internal diameter	23 feet
Ring length	6 feet
Ring arrangement	6 segments plus key
Dowel arrangement	3 dowels per segment 1 dowel per key
Net internal water pressure	30.0 feet net internal
Net internal water pressure after accounting for minimum external effective ground pressure	8.4 feet net internal
Tension per ring	160 kN (161 kips)
Dowel capacity	160 kN

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.

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[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)

Project	Date	Location	Lining Type	Internal Pressure	Length	Internal Diameter
Piastra Andonno	1974	Italy	In situ	260 ft	37,400 ft	10.8 ft
Taloro	1976	Italy	In situ	300 ft	1,620 ft	18.0 ft
Grimsel Head-race Tunnel	1977	Switzerland	In situ	250 ft	650 ft	22.3 ft
Grimsel Tail-race Tunnel	1977	Switzerland	In situ	460 ft	200 ft	22.3 ft
Thun Tunnel	2007	Switzerland	Segmental	Not specified	3,940 ft	17.7 ft

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.

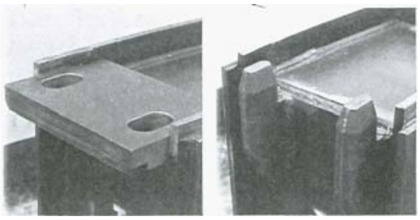
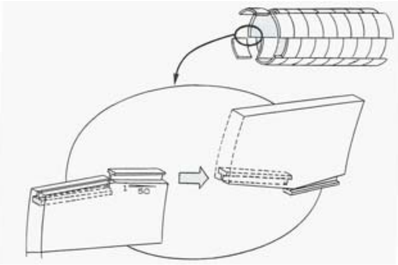
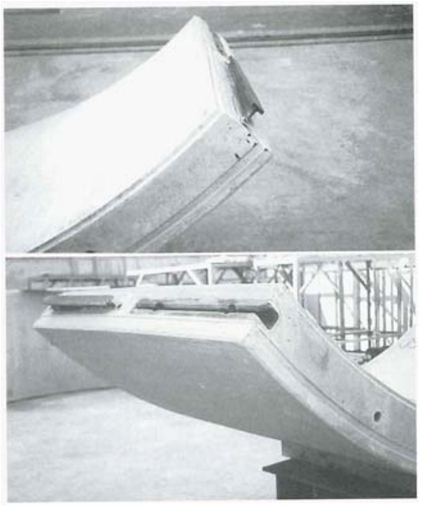
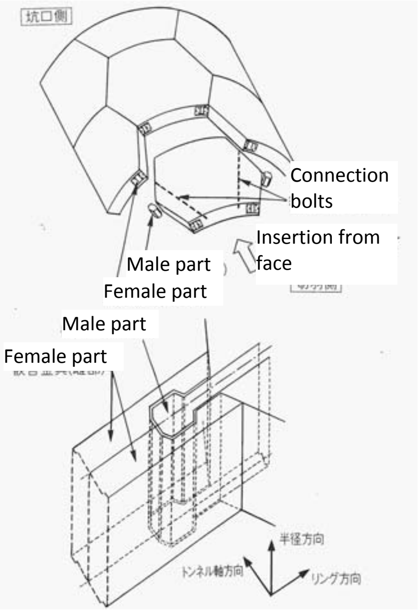
The original developers were contacted by CH2M HILL for information. Many of the manufacturers do not operate outside Japan and will not provide support for their use in the U.S. market. Therefore, if any of the solutions were to be adopted, they would require development from concept through prototyping 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

One Pass NM segments		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.
Guide lock segment	 	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.
Honeycomb segment connector		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.

Past uses of these systems are described in Table 6.

TABLE 6
Past Use of Solutions

System	Project Name	Year	Internal Diameter	Application
One Pass NM segments	Tokyo Metro Oedo Line	1999	22.3 ft	Rail
	Kamidagawa No.7 Ring Line	2004	43.3 ft	Sewer storage
Honeycomb segment connector	Tokyo Central Circular Route	2003	38.7 ft	Road
Guide lock segment	Fukuoka Sewer	1998	11.0 ft	Sewer

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

System	Advantages	Disadvantages
All systems	Tensile capacity can be achieved with very low levels of joint opening (probably less than 0.5 mm)	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.
One Pass NM segments	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.	Technology is not supported in the U.S., so development would be required.
Guide lock segment	Will assist with build accuracy. Provide schedule benefits from continuous mining.	Awaiting feedback from developer. Lack of support and patenting issues possible.
Honeycomb segment connector	Will assist with build accuracy. Provide schedule benefits from continuous mining.	Technology is not supported in the U.S., so development would be required.

TABLE 7

Advantages and Disadvantages of Ferrous Push fit Solutions

System	Advantages	Disadvantages

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

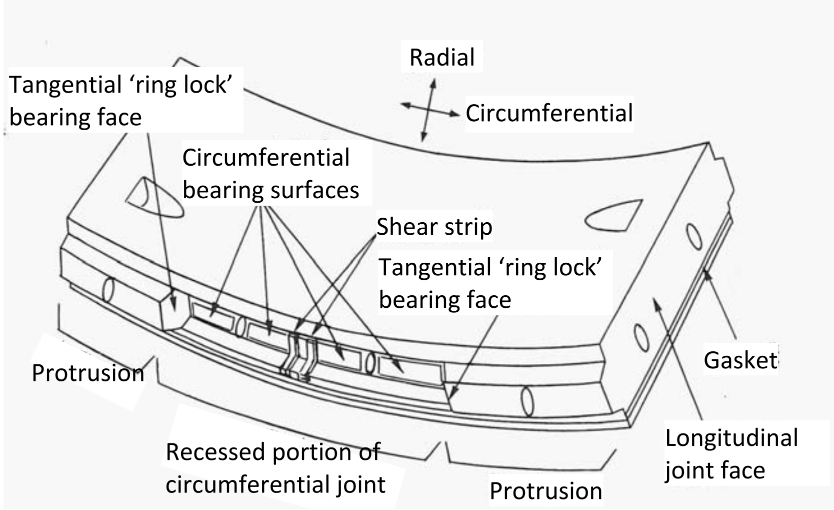
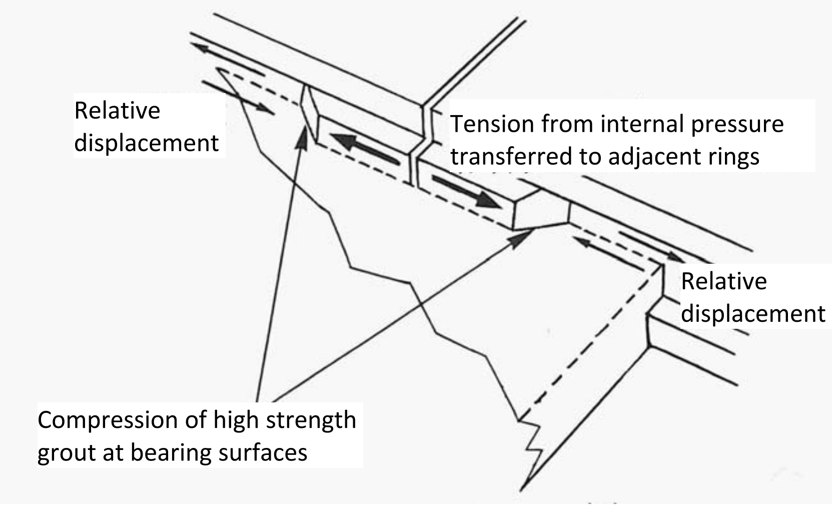


Figure 4 .Tensile Resisting Mechanism for 'Ring lock' Castellated System



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
Advantages and Disadvantages of Potential Solutions

System	Advantages	Disadvantages
Reliance on ground loads	Cost savings could be substantial if no other mitigations are required	Not straightforward to calculate. May not be sufficient to resist tension. Requires detailed ground investigation to evaluate

TABLE 8
Advantages and Disadvantages of Potential Solutions

System	Advantages	Disadvantages
Dowels to resist tension	Assists with build. Will allow some mobilization of ground pressure No durability issues No sockets on inside face Can be omitted if not required	Higher number of dowels – cost impact Plastic behavior at higher loads compromises system if encountered Higher reinforcement may be required in some areas
Systematic secondary grouting	Ensures no tension in the ring	Additional costs Minor increase in leakage Potential for heave needs to be managed No precedent for actively generating the pressure Locking in sufficient pressure may prove unachievable
Post-tensioning	Provides stiffer ring. May be possible to reduce reinforcement Can be omitted if not required	Increased costs from installation Lack of precedent at this size Durability could be difficult to secure
Alternative joint connector	Direct resistance across the joint No specialist equipment/activities Can be omitted if not required	Not used before
Ferrous push-fit solutions	Resists tension with low/no joint opening Assist with build Some types allow continuous mining.	Expensive Limited previous application and support Potential patent issues
Castellated joint	Resists tension with low/no joint opening Provides full resistance to pressure Design and execution guidance is available	Cost of grouting Trialing required Testing for TBM ram loads required Could require increased thickness. Additional reinforcement may be required

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.

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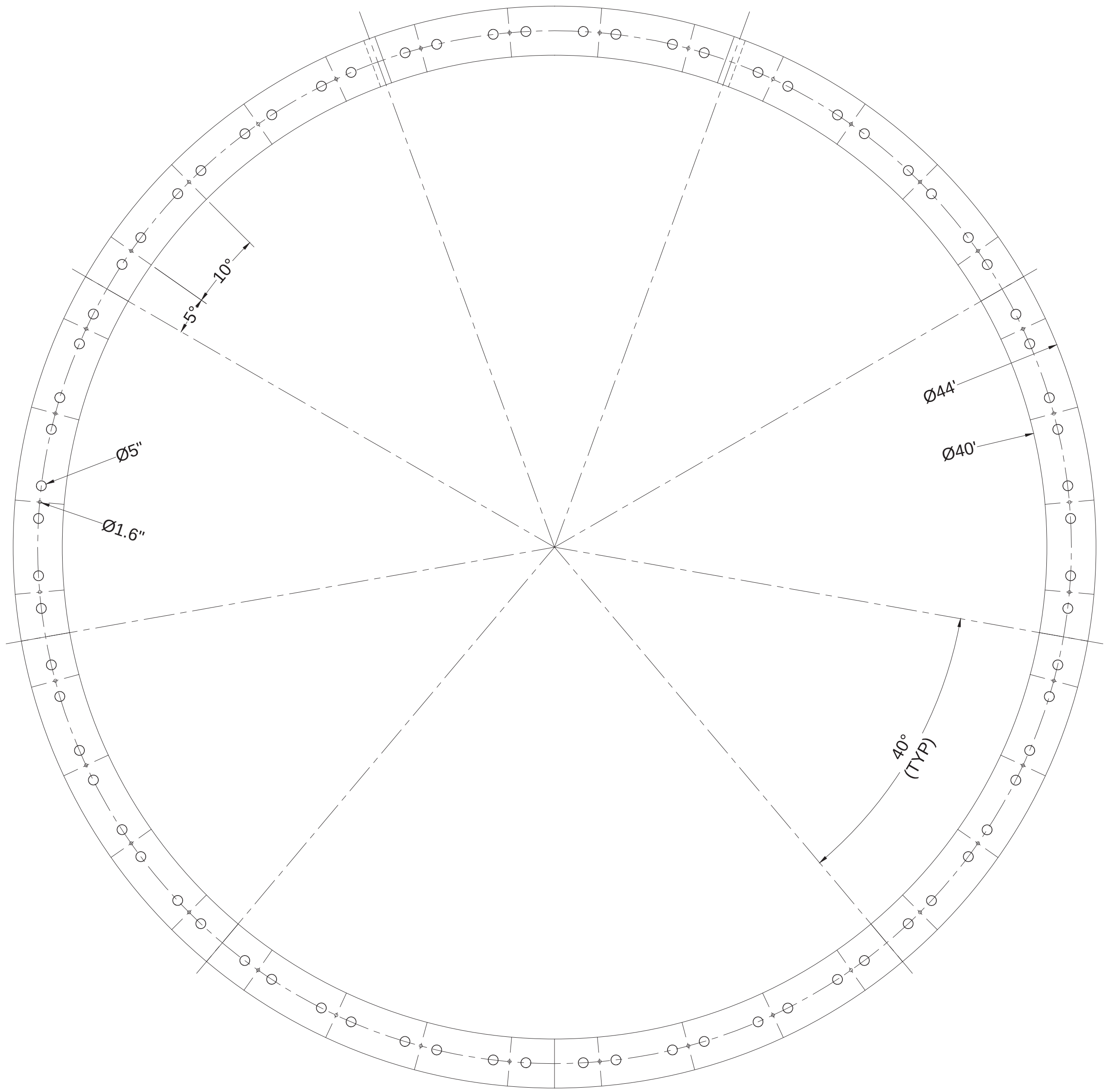
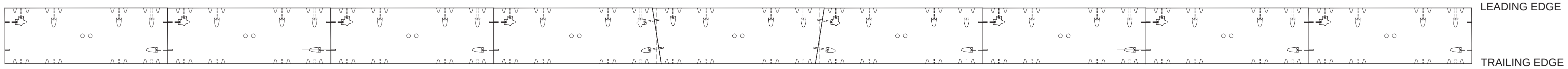
Appendix I

Conceptual Design of Tunnel Segmental Lining Drawings



ELEVATION ON CIRCUMFERENTIAL JOINT
(VIEWED IN DIRECTION OF DRIVE)



DEVELOPED PLAN

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

DESIGNED
A.HARDING

DRAWN
T.THOMPSON

CHECKED
A.FINNEY

APPROVAL RECOMMENDED

APPROVAL BY

CONCEPTIAL ENGINEERING REPORT
BDCP
MPTO/CCO

40' ID PRECAST TUNNEL LINING
DEVELOPED PLAN

PROJECT NO.
PROJECT NO

SHEET NO.
SHEET NO

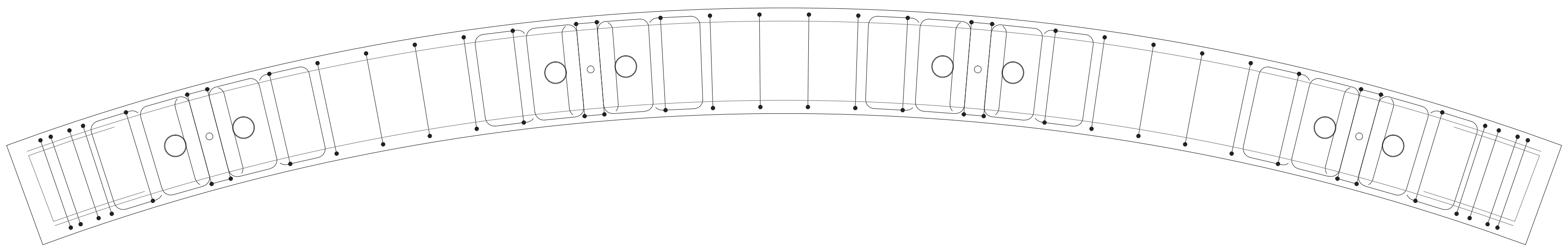
REV
REV

SEQUENCE NO.
SEQ. NO

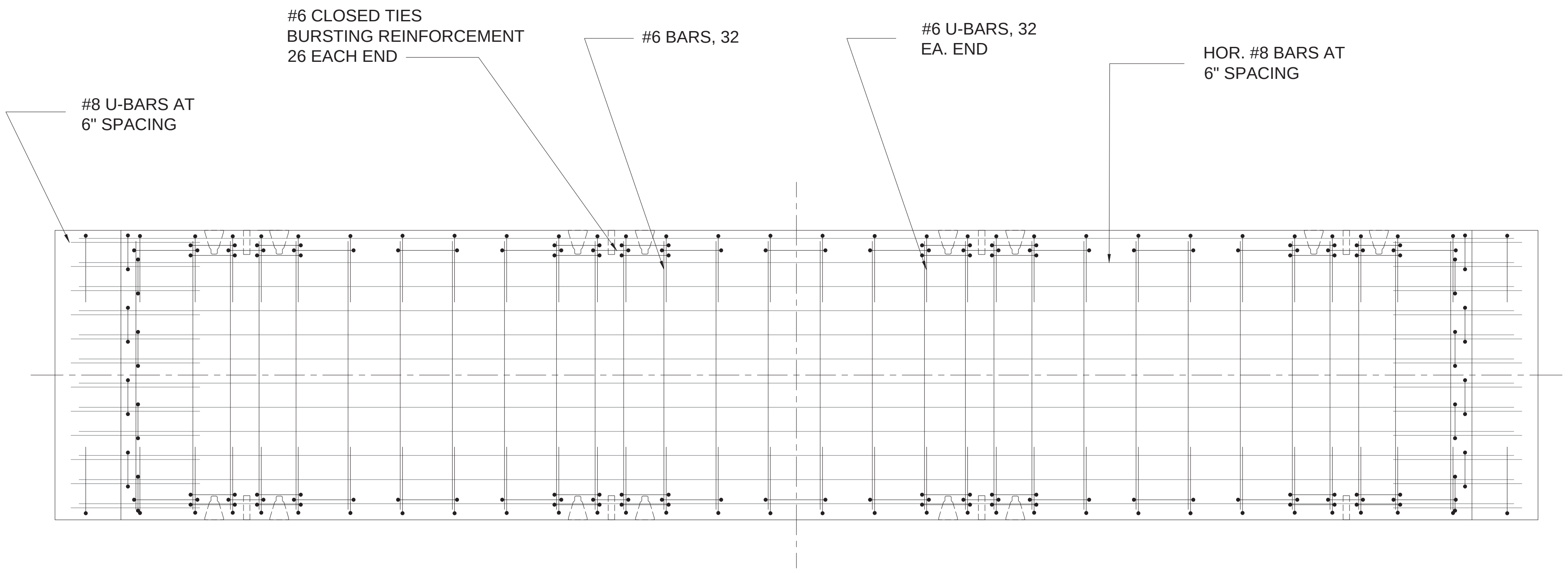
File Name dgnspec$$$

Edited By: Norberg, Neil

Printed By: Tthompson, Tim



ELEVATION ON CIRCUMFERENTIAL JOINT



PLAN ON INTRADOS

VERIFY SCALE
BAR IS ONE INCH ON
ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

DESIGNED
A.HARDING
DRAWN
T.THOMPSON
CHECKED
A.FINNEY

APPROVAL RECOMMENDED
APPROVAL BY

CONCEPTIAL ENGINEERING REPORT
BDCP
MPTO/CCO

40' ID PRECAST TUNNEL LINING
REINFORCEMENT

PROJECT NO.
PROJECT NO
SHEET NO.
SHEET NO
REV
REV
SEQUENCE NO.
SEQ. NO

File Name *dgnspec$$*

Edited By: Norberg, Neil

Printed By: Thompson, Tim

Appendix J

Conceptual Engineering Evaluation of Tunnel Lining Systems

JACOBS ASSOCIATES
Engineers/Consultants

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

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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

Tunnel Diameter 40 feet ID 20 inch Segment thickness Concrete cover 2 inches Ground Poisson's Ratio 0.3
Ground Elastic Modulus 600 ksf
4.166667 ksi

Alternative A1: One-pass Lining with Rebar and Bolts at Radial Joints

	Hoop Tensile Force (kip)	Steel Yield Stress (ksi)	Steel Design Stress (ksi)	Required Steel Area (sq inch)	Bar Size/ Bolt Dia	Bar Area (sq inch)	Bar Spacing s (inch)	Steel Area (sq inch)	Steel Area (%)	Calculated Steel Stress Fs (ksi)	Concrete Cover Dc (inch)	Crack Width w (inch)	Eurocode EC2 2004 Wk (inch)	(ACI 318 or ACI 224) Smax (inch)	(ACI 318 or ACI 224) Fsmax (ksi)	Bar Dia (inch)	Rizkalla Crack Spacing (inch)	ts (inch)	M	Circum-ferential Strain	Crack width (inch)
Reach 1 Head (ft)	**50.0**																				
5 ft segment	323.7	60	20.00	16.185	11	1.560	10	18.720	1.56%	17.29	2.705	0.00911	0.00968	26.2	20.36	1.409345	14.2	0.3	12.2919	0.00055	0.0078
3 Bolts (LF 1.4)	323.7	84.8	60.57	5.344	1.625	2.074	20	6.222	0.52%	52.03											
Reach 2 Head (ft)	**44.0**																				
5 ft segment	284.9	60	20.00	14.243	10	1.270	10	15.240	1.27%	18.69	2.636	0.00968	0.01116	23.9	20.46	1.271619	14.3	0.3	10.19277	0.000584	0.0083
3 Bolts (LF 1.4)	284.9	84.8	60.57	4.703	1.5	1.767	20	5.301	0.44%	53.73											
Reach 3 Head (ft)	**40.0**																				
5 ft segment	259.0	60	20.00	12.948	10	1.270	10	15.240	1.27%	16.99	2.636	0.00880	0.01014	26.8	20.46	1.271619	14.3	0.3	10.19277	0.000531	0.0076
3 Bolts (LF 1.4)	259.0	84.8	60.57	4.275	1.5	1.767	20	5.301	0.44%	48.85											
Reach 4 Head (ft)	**34.0**																				
5 ft segment	220.1	60	20.00	11.006	9	1.000	10	12.000	1.00%	18.34	2.564	0.00933	0.01189	24.4	20.57	1.128379	14.6	0.2	8.2384	0.000558	0.0081
3 Bolts (LF 1.4)	220.1	84.8	60.57	3.634	1.375	1.485	20	4.455	0.37%	49.41											
Reach 5 Head (ft)	**28.0**																				
5 ft segment	181.3	60	20.00	9.064	8	0.780	10	9.360	0.78%	19.37	2.498	0.00968	0.01363	22.9	20.67	0.996557	15.1	0.2	6.645952	0.00057	0.0086
3 Bolts (LF 1.4)	181.3	84.8	60.57	2.993	1.25	1.227	20	3.682	0.31%	49.24											

Table 2: One-pass Lining Alternative A1: Preliminary Segmental Lining Hoop Reinforcing Requirements

Reach No.	Net Differential Internal Water Pressure (feet)	Reinforcing Bar Size (hoop steel)	Bolt Size at Radial Joints (inch)
1	50 to 44	#11	1.625
2	44 to 40	#10	1.5
3	40 to 34	#10	1.5
4	34 to 28	#9	1.375
5	28 to 24	#8	1.25

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 3: Summary of Leakage Estimate for Lining Alternative A1

Reach No.	Net Internal Water Head	Crack Spacing	Crack Width	Lining Permeability	Head Loss Across Lining	Estimated Leakage Rate per Tunnel
	ft	in	in	ft/s (cm/s)	%	cfs/1000 lf
1	50	14.2	0.0078	0.000058 (1.76E-03)	5.0	11.3
2	44	14.3	0.0083	0.000070 (2.13E-03)	4.2	10.1
3	40	14.3	0.0076	0.000053 (1.60E-03)	5.5	9.0
4	34	14.6	0.0081	0.000064 (1.94E-03)	4.6	7.7
5	28	15.1	0.0086	0.000073 (2.22E-03)	4.0	6.3

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/m^2/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, normalized with respect to the total 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, Terzhaghi (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.

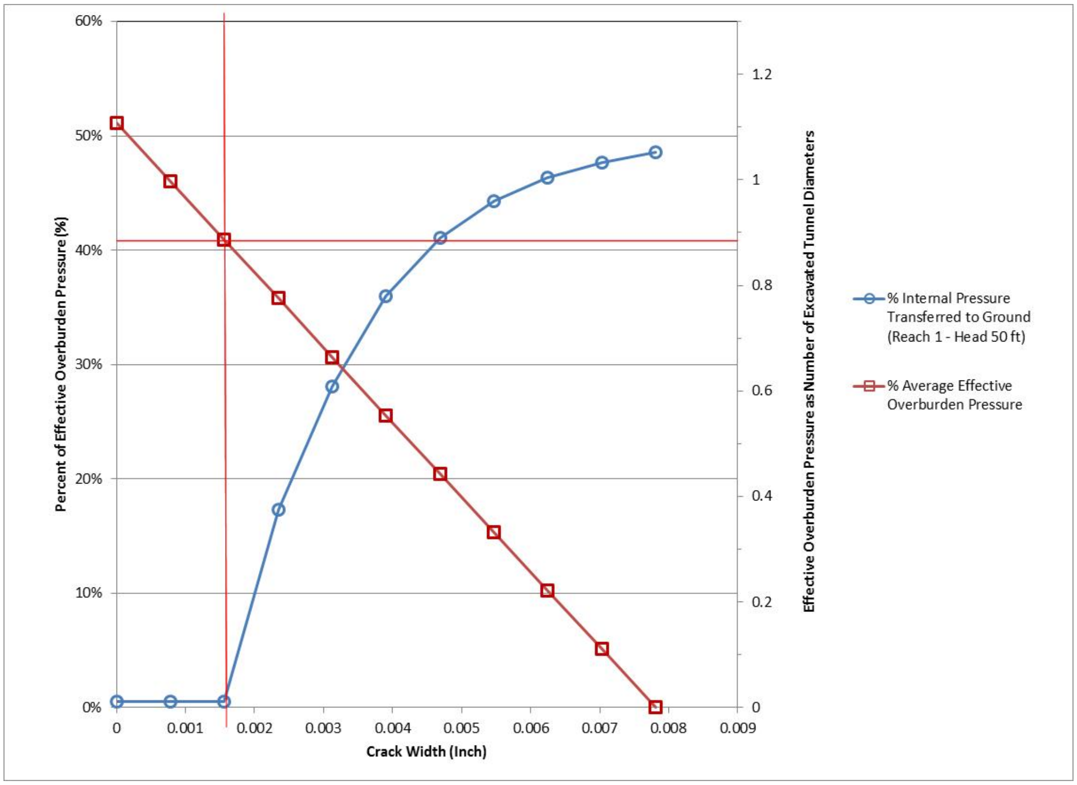

Figure 1: Reach 1 – Effective Overburden Pressure vs. Concrete Lining Crack Width

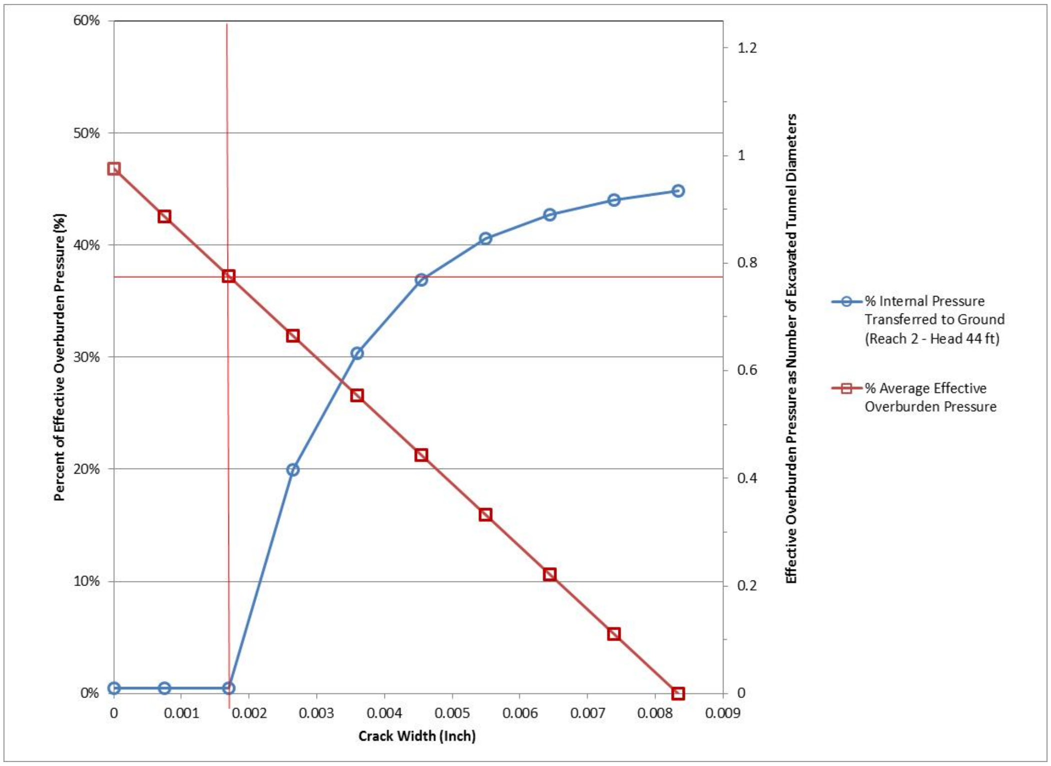

Figure 2: Reach 2 – Effective Overburden Pressure vs. Concrete Lining Crack Width

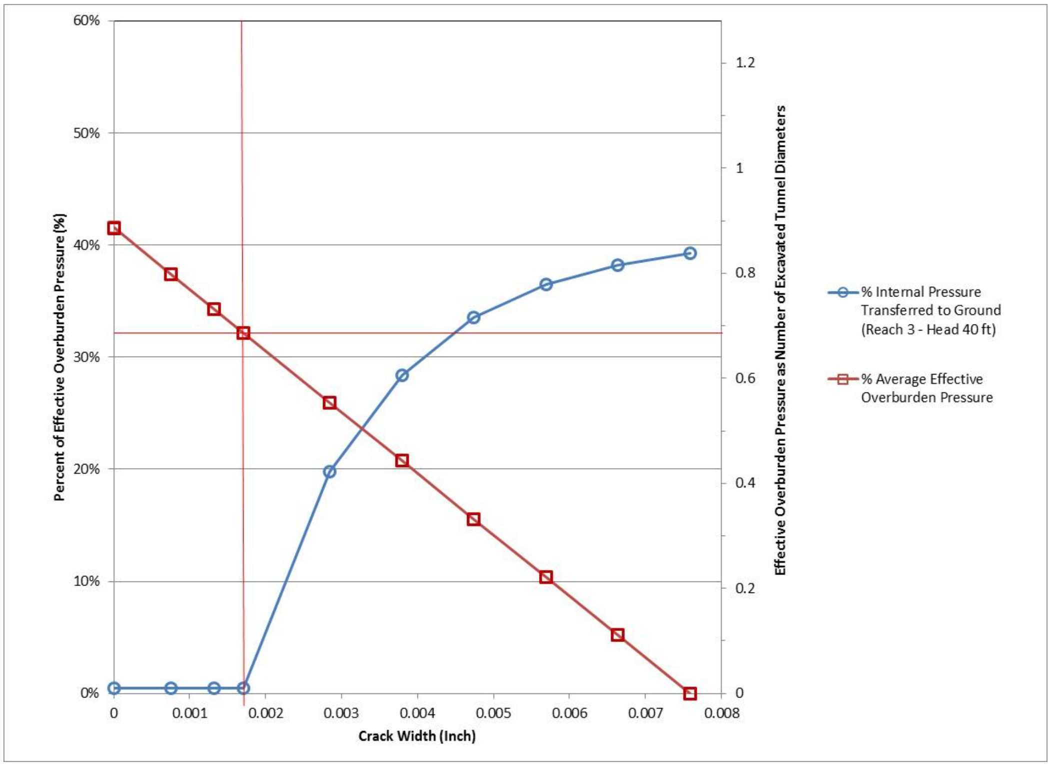

Figure 3: Reach 3 – Effective Overburden Pressure vs. Concrete Lining Crack Width

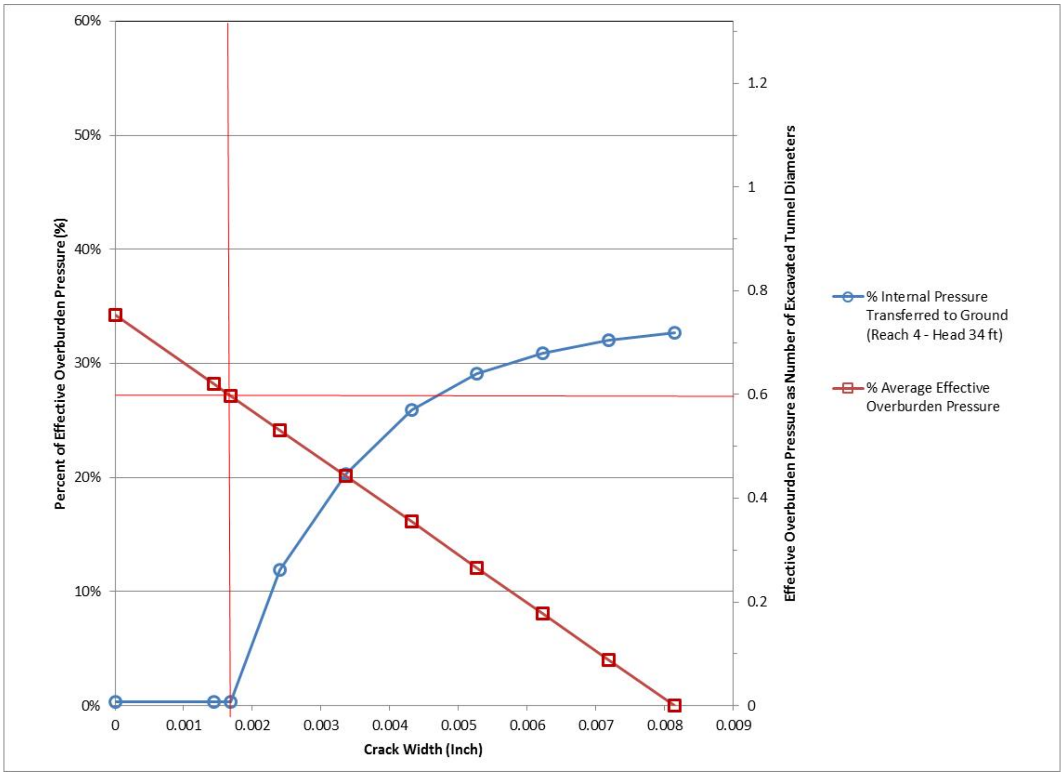

Figure 4: Reach 4 – Effective Overburden Pressure vs. Concrete Lining Crack Width

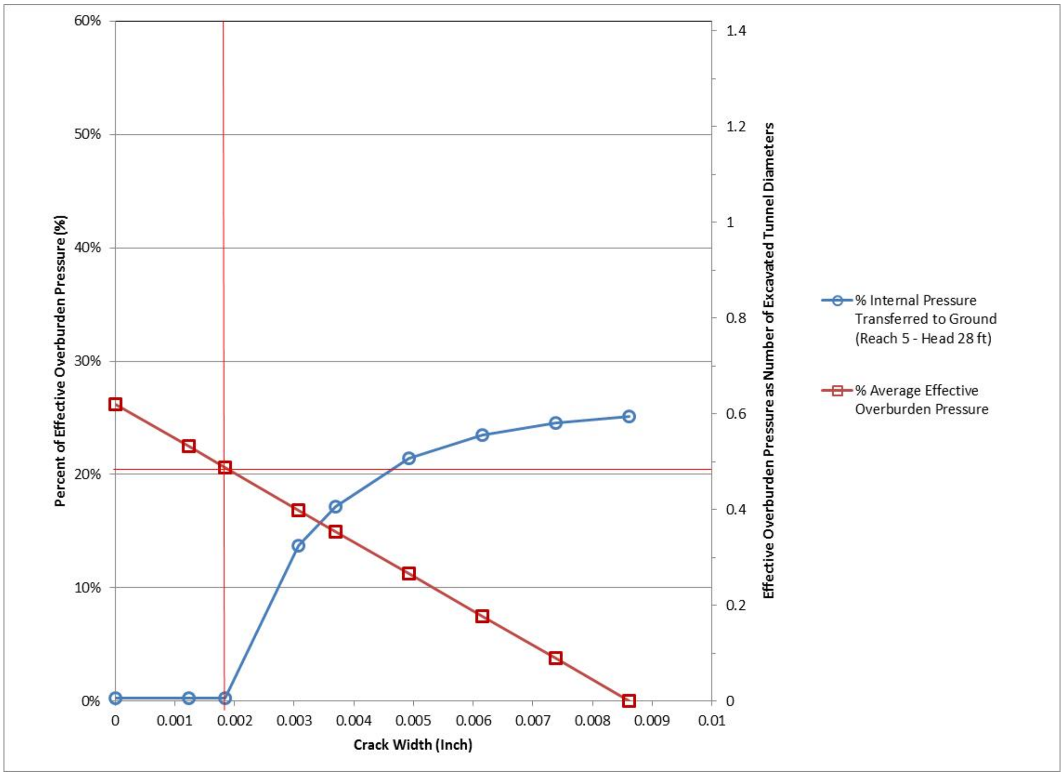

Figure 5: Reach 5 - Effective Overburden Pressure vs. Concrete Lining Crack Width

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

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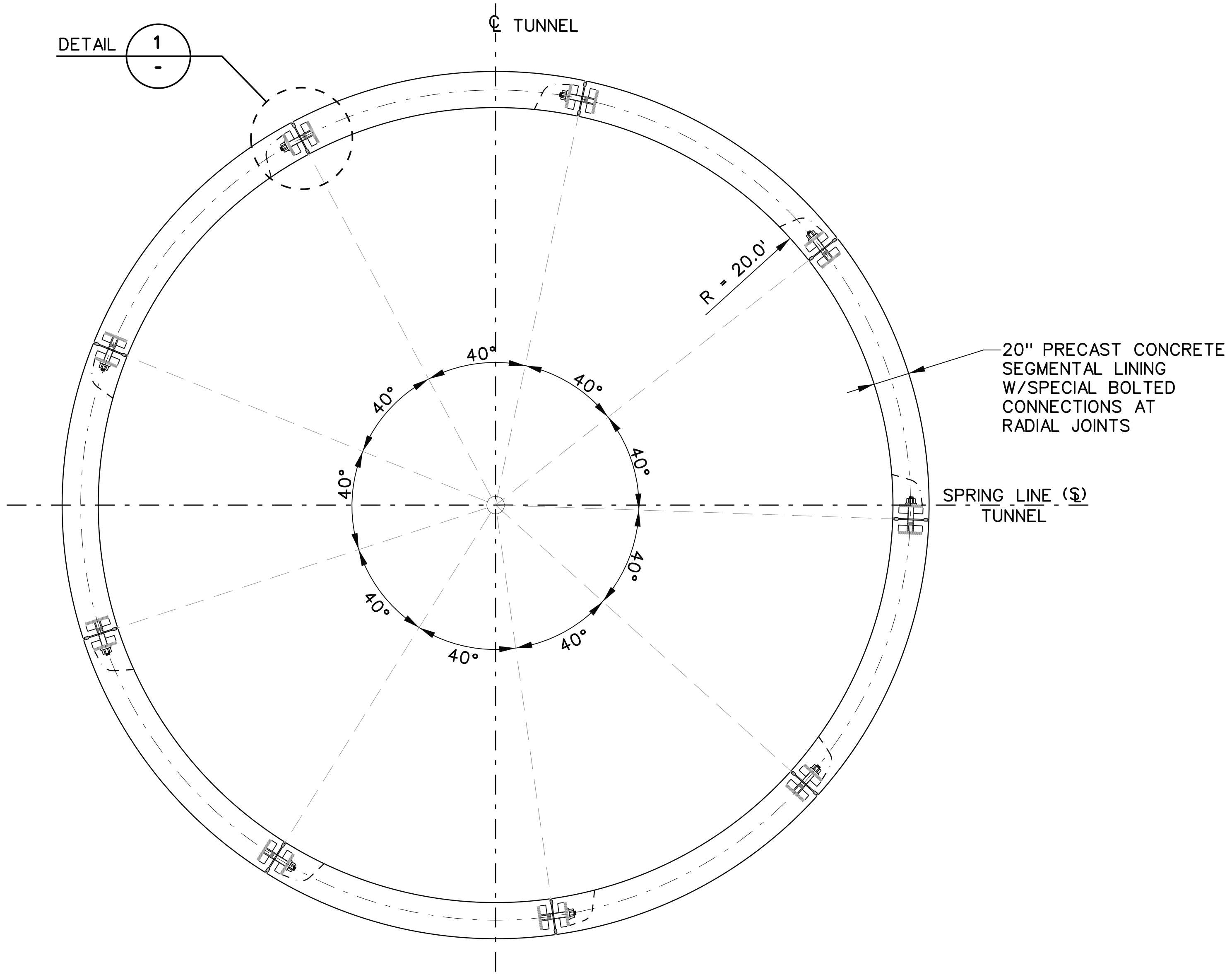
Appendix A: Drawings

Drawing SK-401: One-Pass Segmental Lining Gravity Tunnel: Alternative A1: Reinforcing with Bolted Connections at Radial Joints

Drawing SK-402: Typical Segmental Lining Reinforcing



TUNNEL CROSS SECTION
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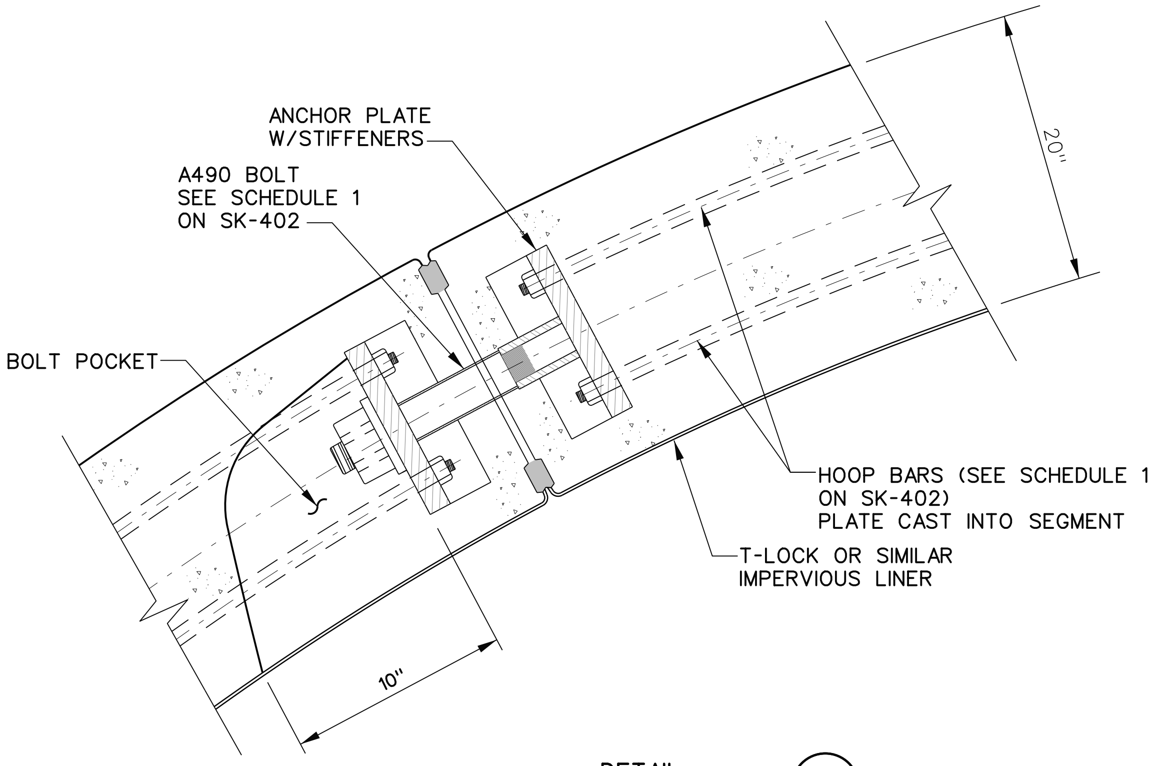


DETAIL 1 -
SCALE: NOT TO SCALE

NOTES:

1. CONCRETE 28-DAY STRENGTH: 8000 psi.
2. SEGMENT RING WIDTH: 5'-0".
3. BOLT/ANCHOR PLATE/REBAR: 4 PER SEGMENT RING

DRAFT - IN PROGRESS

NO	REVISION	BY	CHK	APP	DATE
△					
△					
△					
△					
△					

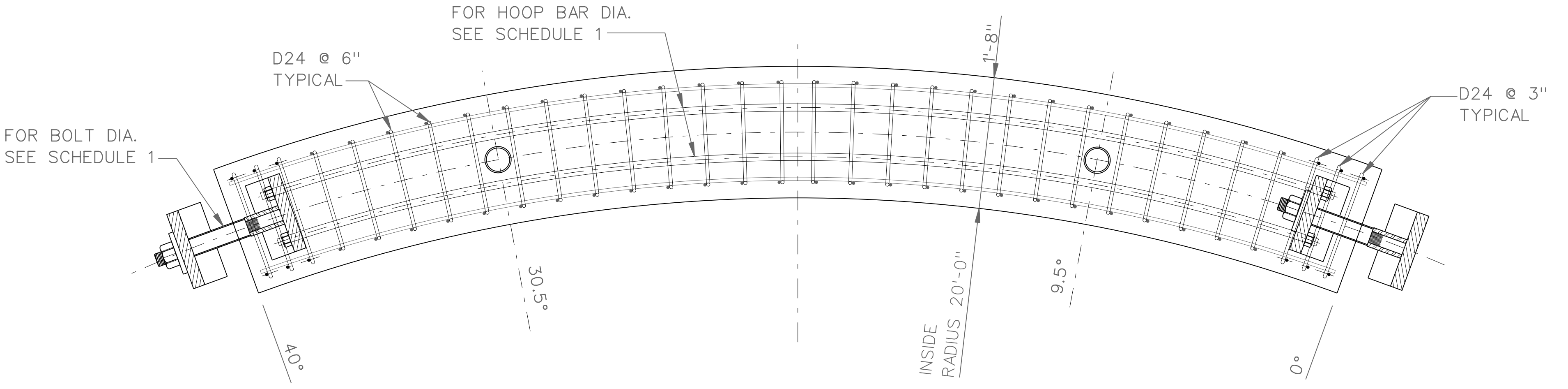
DESIGNED
DRAWN
CHECKED
APPROVED

JACOBS ASSOCIATES
Engineers/Consultants

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

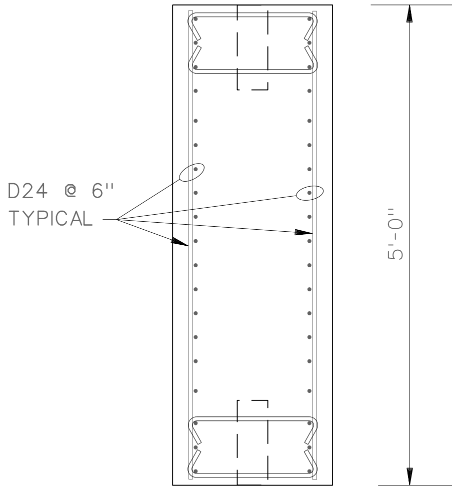
DATE: JULY 2012
SCALE: NOT TO SCALE
JOB NO: 4034.14
DWG NO: SK-401
REVISION 0
SHT NO: 1 OF 1



PLAN OF SEGMENTAL CONCRETE LINING

SCHEDULE 1

REACH No.	HOOP BAR DIA.	BOLT DIA. (INCH)
1	#11	1.625
2	#10	1.5
3	#10	1.5
4	#9	1.375
5	#8	1.25



SECTION A-A
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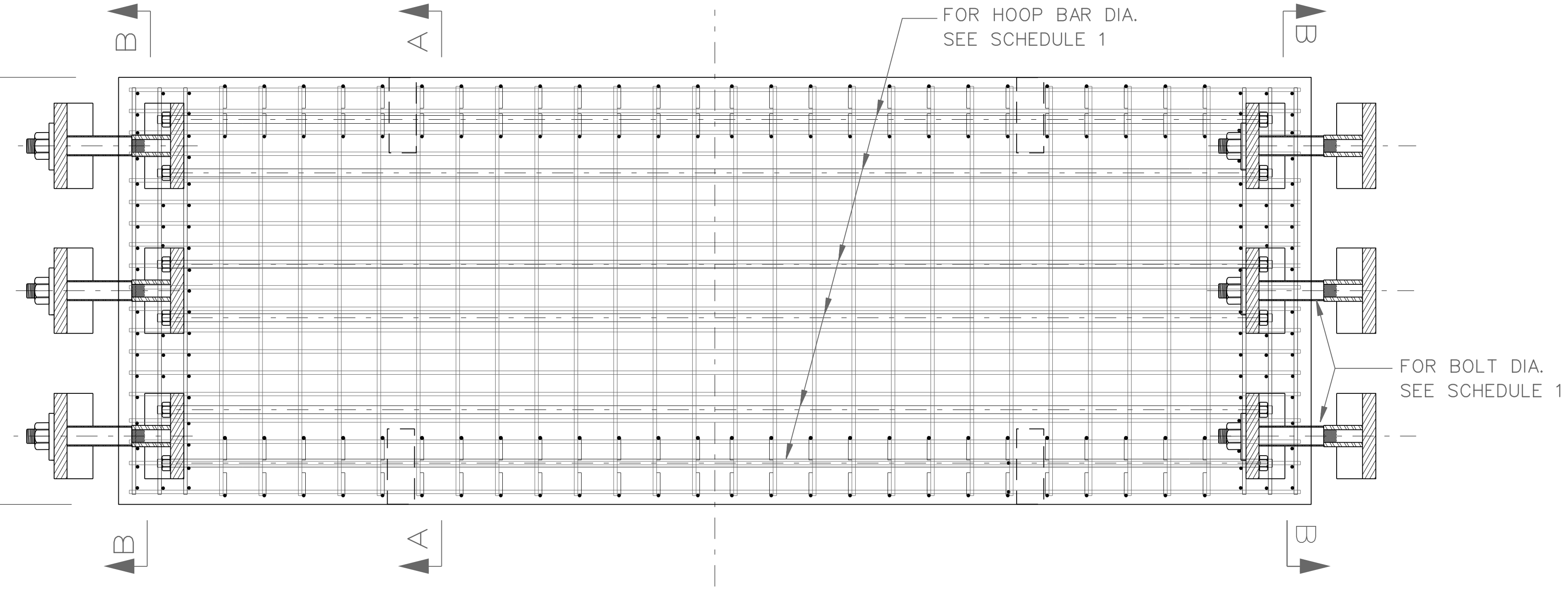
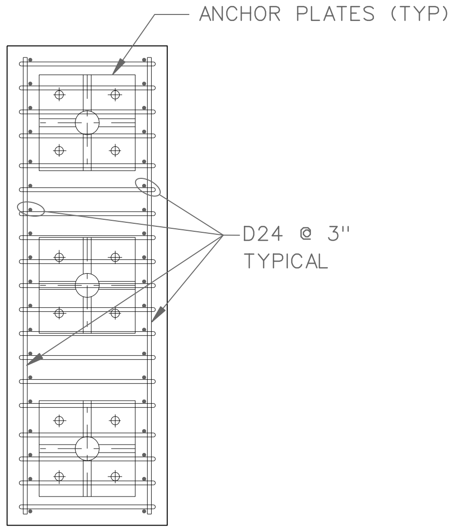


DEVELOPED INSIDE ELEVATION



SECTION B-B
SCALE: NOT TO SCALE

DRAFT - IN PROGRESS

NO	REVISION	BY	CHK	APP	DATE
△					
△					
△					
△					
△					

DESIGNED
DRAWN
CHECKED
APPROVED

JACOBS ASSOCIATES
Engineers/Consultants

METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA
FINAL TUNNEL LINING CONCEPTUAL ENGINEERING AND EVALUATION

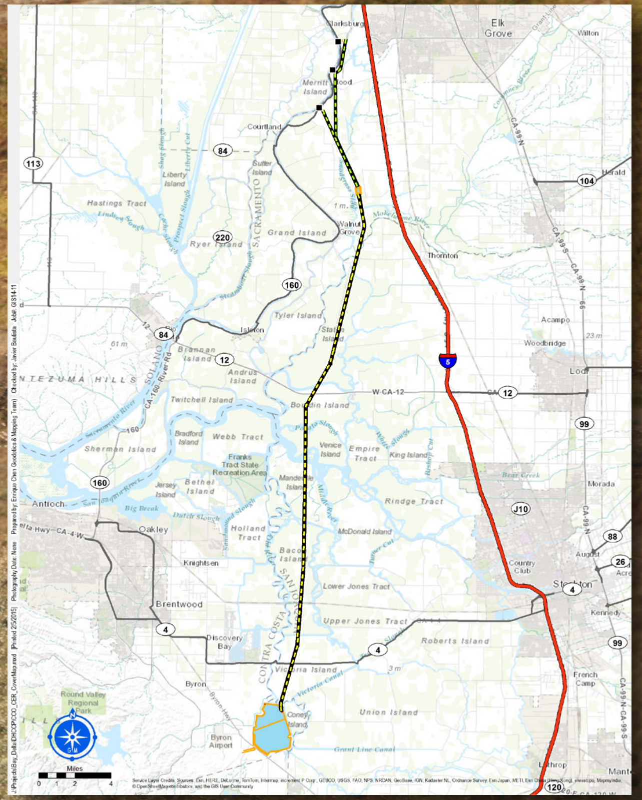
ONE - PASS SEGMENTAL LINING GRAVITY TUNNEL: ALTERNATIVE A1
TYPICAL SEGMENTAL LINING REINFORCING

DATE: JULY 2012
SCALE: NOT TO SCALE
JOB NO: 4034.14
DWG NO: SK-402
REVISION: 0
SHT NO: 1 OF 1

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

DHCCP Team

PRELIMINARY
NOT FOR CONSTRUCTION



Delta Habitat Conservation & Conveyance Program (DHCCP)

Final Draft: July 1, 2015

2

Conceptual Engineering Report

Modified Pipeline / Tunnel Option – Clifton Court Forebay Pumping Plant
Volume 2 – Conceptual Engineering Report Drawings

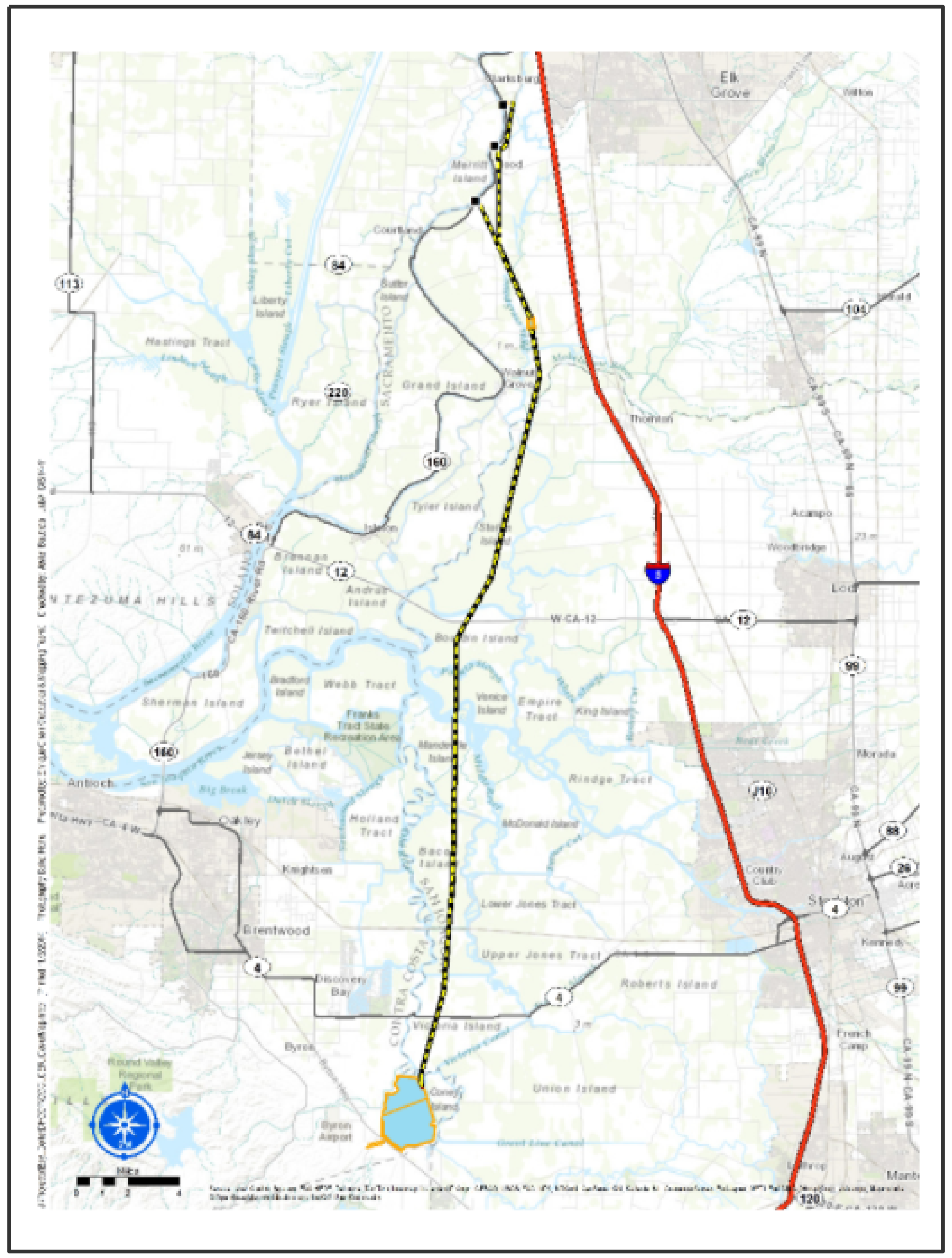
901 P Street Suite 411 B, Sacramento, CA 95814

STATE OF CALIFORNIA
CALIFORNIA NATURAL RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES

DELTA HABITAT CONSERVATION AND CONVEYANCE PROGRAM
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE/TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

PROJECT NO. 240010

CONCEPTUAL ENGINEERING REPORT
VOLUME 2
CONCEPT DRAWINGS - FINAL DRAFT

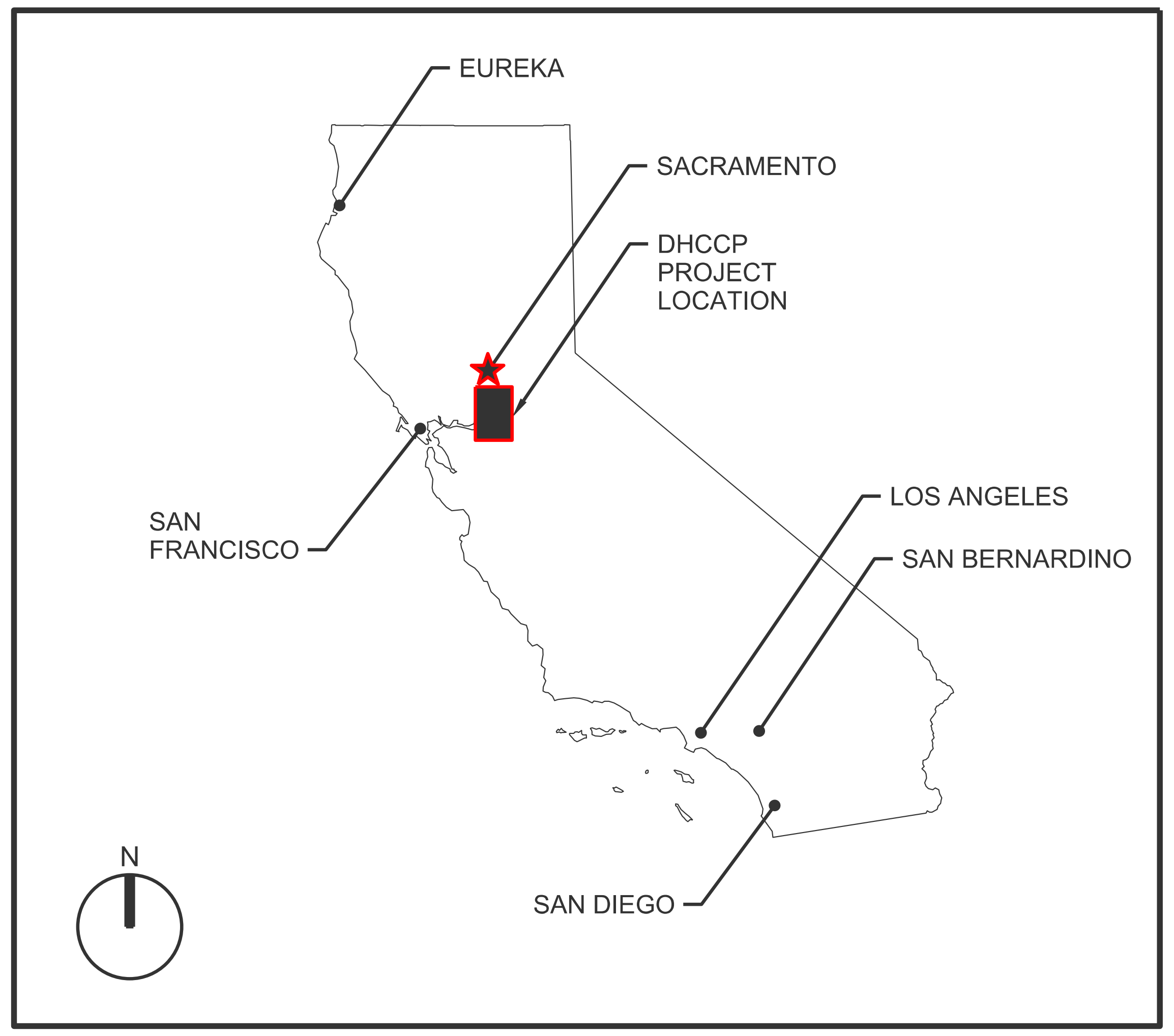


VICINITY MAP

N

NOTES:

1. CONCEPT DRAWINGS INCLUDED IN CER-VOLUME 2 DO NOT REPRESENT A DETAILED PRELIMINARY ENGINEERING DESIGN EFFORT. INFORMATION SHOWN HAS BEEN LAID OUT USING AVAILABLE RESOURCES TO CONVEY DESIGN INTENT. FACILITY LOCATIONS AND INDICATED DIMENSIONS, STATIONS, AND ELEVATIONS ARE APPROXIMATE AND SUBJECT TO CHANGE DURING SUBSEQUENT ENGINEERING EFFORTS.
2. TOPOGRAPHIC AND BATHYMETRIC INFORMATION USED IN THE DEVELOPMENT OF THIS CER HAS NOT BEEN GROUND PROOFED AND SHOULD BE CONSIDERED APPROXIMATE.
3. HORIZONTAL DATUM IS CALIFORNIA STATE PLAN E, ZONE 3, NORTH AMERICAN DATUM OF 1983 (NAD 83), SURVEY FEET.
4. ALL ELEVATIONS ARE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 88) UNLESS OTHERWISE SHOWN.
5. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL DATASET (NED), DATA IS FOR PRELIMINARY USE ONLY.



LOCATION MAP

FINAL DRAFT
DATE: APRIL 1, 2015

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

APPROVAL RECOMMENDED: APPROVED DATE

DEPUTY CHIEF
DIVISION OF ENGINEERING
REG CE NO. ______

CHIEF
DIVISION OF ENGINEERING
REG CE NO. ______

PROJECT NO. 240010
SHEET NO. CCO-G-0001GN
REV
SEQUENCE NO. 1 of 96

SEQUENCE NUMBER	SHEET NUMBER	TITLE
GENERAL		
1	CCO-G-0001GN	COVER SHEET
2	CCO-G-0002GN	DRAWING SHEET INDEX
3	CCO-G-1003GN	CONVEYANCE OVERVIEW 1 OF 2
4	CCO-G-1004GN	CONVEYANCE OVERVIEW 2 OF 2
HYDRAULICS		
5	CCO-C-4005HH	OVERALL HYDRAULIC PLAN AND PROFILE
6	CCO-C-4006HH	NORTH TUNNELS - HYDRAULIC PLAN AND PROFILES
7	CCO-C-4007HH	MAIN TUNNELS - HYDRAULIC PLAN AND PROFILE
CONTROL SYSTEM		
8	CCO-I-6008GN	OVERALL PROCESS FLOW DIAGRAM
9	CCO-I-6009GN	CONTROL SYSTEM ARCHITECTURE
INTAKES		
10	CCO-G-9010IT	TYPICAL INTAKE FACILITY RENDERING
11	CCO-C-1011IT	INTAKE NO. 2 SITE PLAN
12	CCO-C-1012IT	INTAKE NO. 3 SITE PLAN
13	CCO-C-1013IT	INTAKE NO. 5 SITE PLAN
14	CCO-C-3014IT	STATE ROUTE 160 REALIGNMENT AND SED. BASINS - TYPICAL CROSS SECTIONS
15	CCO-M-1015IT	TYPICAL INTAKE FACILITY GENERAL ARRANGEMENT & KEY PLAN
16	CCO-M-4016IT	INTAKE STRUCTURE TYPICAL PLANS
17	CCO-M-3017IT	INTAKE STRUCTURE TYPICAL SECTION
18	CCO-M-3018IT	SEDIMENTATION BASIN TYPICAL SECTION
19	CCO-M-4019IT	TYPICAL CONCRETE BOX CONDUIT ISOMETRIC
20	CCO-M-4020IT	TYPICAL OUTLET SHAFT ISOMETRIC
21	CCO-M-4021IT	INTAKE SEDIMENT JETTING PARTIAL PLAN AND DETAILS
22	CCO-M-2022IT	INTAKE STRUCTURE ELEVATION
23	CCO-M-2023IT	INTAKE FISH SCREEN PANEL ELEVATIONS
24	CCO-M-4024IT	OUTLET SHAFT @ INTAKE NO. 2 - PLAN AND SECTION
25	CCO-M-4025IT	OUTLET SHAFT @ INTAKE NO. 3 - PLAN AND SECTION
26	CCO-M-3026IT	OUTLET SHAFT @ INTAKE NO. 3- SECTIONS
27	CCO-M-4027IT	OUTLET SHAFT @ INTAKE NO. 5 - PLAN AND SECTION
28	CCO-C-4028IT	SEDIMENTATION DRYING LAGOON - TYPICAL PLAN
29	CCO-C-3029IT	SEDIMENTATION DRYING LAGOON - TYPICAL SECTIONS
30	CCO-C-4030IT	SEDIMENTATION DRYING LAGOON OUTLET PLAN AND SECTION
31	CCO-I-6031IT	INTAKE PROCESS FLOW DIAGRAM
32	CCO-E-6032IT	INTAKE NO. 3 SINGLE LINE DIAGRAM
TUNNELS		
33	CCO-C-1033TN	NORTH TUNNELS - PLAN AND PROFILE INTAKE NO. 2 TO INTERMEDIATE FOREBAY
34	CCO-C-1034TN	NORTH TUNNELS - PLAN AND PROFILE INTAKE NO. 5 TO INTERMEDIATE FOREBAY
35	CCO-C-1035TN	MAIN TUNNELS PLAN AND PROFILE - SHEET 1 OF 4
36	CCO-C-1036TN	MAIN TUNNELS PLAN AND PROFILE - SHEET 2 OF 4
37	CCO-C-1037TN	MAIN TUNNELS PLAN AND PROFILE - SHEET 3 OF 4
38	CCO-C-1038TN	MAIN TUNNELS PLAN AND PROFILE - SHEET 4 OF 4
39	CCO-S-4039TN	28' DIA. NORTH TUNNELS LINING DETAILS
40	CCO-S-4040TN	40' DIA. NORTH TUNNEL LINING DETAILS
41	CCO-S-4041TN	40' DIA. MAIN TUNNELS LINING DETAILS
42	CCO-S-4042TN	40' DIA. MAIN TUNNELS ALTERNATIVE LINING DETAILS - SHEET 1 OF 2
43	CCO-S-4043TN	40' DIA. MAIN TUNNELS ALTERNATIVE LINING DETAILS - SHEET 2 OF 2
44	CCO-C-1044TN	NORTH TUNNELS - INTAKES NO. 2 AND NO. 3 DRIVE AND RECEPTION SHAFTS - SITE PLAN
45	CCO-C-1045TN	NORTH TUNNELS - INTAKES NO. 5 RECEPTION SHAFT - SITE PLAN
46	CCO-C-1046TN	NORTH TUNNELS - CONSTRUCTION ACCESS - SITE PLAN
47	CCO-C-1047TN	NORTH TUNNELS/MAIN TUNNELS INTERMEDIATE FOREBAY - DRIVE SHAFTS - SITE PLAN
48	CCO-C-1048TN	MAIN TUNNELS - STATEN ISLAND CONSTRUCTION ACCESS - SITE PLAN
49	CCO-C-1049TN	MAIN TUNNELS - STATEN ISLAND RECEPTION SHAFTS - SITE PLAN
50	CCO-C-1050TN	MAIN TUNNELS - BOULDIN ISLAND DRIVE SHAFTS - SITE PLAN
51	CCO-C-1051TN	MAIN TUNNELS - MANDEVILLE ISLAND CONSTRUCTION ACCESS - SITE PLAN
52	CCO-C-1052TN	MAIN TUNNELS - BACON ISLAND RECEPTION SHAFTS - SITE PLAN
53	CCO-C-1053TN	MAIN TUNNELS - VICTORIA ISLAND CONSTRUCTION ACCESS - SITE PLAN
54	CCO-C-1054TN	MAIN TUNNELS - CLIFTON COURT FOREBAY DRIVE SHAFTS - SITE PLAN
INTERMEDIATE FOREBAY		
55	CCO-G-9055IF	INTERMEDIATE FOREBAY RENDERING
56	CCO-C-1056IF	INTERMEDIATE FOREBAY SITE PLAN
57	CCO-C-3057IF	INTERMEDIATE FOREBAY SECTIONS
58	CCO-M-4058IF	INTERMEDIATE FOREBAY INLET STRUCTURE PLAN
59	CCO-M-3059IF	INTERMEDIATE FOREBAY INLET STRUCTURE LONGITUDINAL SECTION

SEQUENCE NUMBER	SHEET NUMBER	TITLE
INTERM. FOREBAY-CONT.		
60	CCO-M-5060IF	INTERMEDIATE FOREBAY INLET STRUCTURE SECTION AND DETAILS
61	CCO-M-4061IF	INTERMEDIATE FOREBAY OUTLET STRUCTURE PLAN
62	CCO-M-3062IF	INTERMEDIATE FOREBAY OUTLET STRUCTURE LONGITUDINAL SECTION
63	CCO-M-5063IF	INTERMEDIATE FOREBAY OUTLET STRUCTURE SECTION AND DETAILS
64	CCO-M-4064IF	TYPICAL GENERAL ARRANGEMENT FILL AND DRAIN PIPING UPPER LEVEL - PLAN
65	CCO-M-4065IF	FILL AND DRAIN PIPING UPPER LEVEL - SECTION
66	CCO-M-4066IF	FILL AND DRAIN PIPING DETAIL PLAN AND SECTION
67	CCO-M-4067IF	TYPICAL FOREBAY DRAIN/FILL PLAN AND SECTION
68	CCO-E-6068IF	INTERMEDIATE FOREBAY SINGLE LINE DIAGRAM
TUNNEL SHAFTS		
69	CCO-C-1069TS	TYPICAL TUNNEL SHAFTS WORK AREA PLAN
70	CCO-C-1070TS	TYPICAL TUNNEL SHAFTS FINISHED PLAN
71	CCO-S-5071TS	NORTH AND MAIN TUNNELS TYPICAL DRIVE AND RECEPTION SHAFT - PLAN AND SECTION
72	CCO-S-5072TS	NORTH AND MAIN TUNNELS TYPICAL VENT SHAFT - PLAN AND SECTION
73	CCO-S-5073TS	NORTH AND MAIN TUNNELS TYPICAL FINAL SHAFT CONFIRGURATION - SECTIONS
CLIFTON COURT FOREBAY PUMPING PLANT		
74	CCO-G-9074PP	CLIFTON COURT FOREBAY PUMPING PLANT RENDERING
75	CCO-C-1075PP	CLIFTON COURT FOREBAY PUMPING PLANT SITE PLAN
76	CCO-M-1076PP	CLIFTON COURT FOREBAY PUMPING PLANT PLAN
77	CCO-M-1077PP	CLIFTON COURT FOREBAY PUMPING PLANT PLAN
78	CCO-M-3078PP	CLIFTON COURT FOREBAY PUMPING PLANT SECTIONS
79	CCO-E-6079PP	CLIFTON COURT FOREBAY PUMPING PLANT OVERALL SINGLE LINE DIAGRAM
80	CCO-E-6080PP	CLIFTON COURT FOREBAY PUMPING PLANT SINGLE LINE DIAGRAM
81	—	NOT USED
CLIFTON COURT FOREBAY		
82	CCO-C-1082FB	MODIFIED CLIFTON COURT FOREBAY OVERALL SITE PLAN
83	CCO-C-1083FB	MODIFIED CLIFTON COURT FOREBAY INLET ANFD OUTLETS - SITE PLANS
84	CCO-C-1084FB	CHANNEL APPROACHES TO PUMPING PLANTS SITE PLANS
85	CCO-C-3085FB	MODIFIED CLIFTON COURT FOREBAY EMBANKMENT SECTIONS I
86	CCO-C-3086FB	MODIFIED CLIFTON COURT FOREBAY EMBANKMENT SECTIONS II
87	CCO-C-3087FB	MODIFIED CLIFTON COURT FOREBAY EMBANKMENT SECTIONS III
88	CCO-C-4088FB	CHANNEL CONTROL STRUCTURES PLANS, SECTION, AND DETAIL
89	CCO-C-5089FB	CHANNEL CONTROL STRUCTURES PLAN AND SECTIONS
90	CCO-C-5090FB	NORTH CLIFTON FOREBAY SPILLWAY PLAN
91	CCO-C-3091FB	NORTH CLIFTON COURT FOREBAY SPILLWAY SECTIONS
92	CCO-C-1092FB	MODIFIED CLIFTON COURT FOREBAY SEDIMENT AREA PLAN AND SECTION
POWER AND COMMUNICATIONS		
93	CCO-E-1193GN	CONTROL AND COMMUNICATIONS OVERVIEW
94	CCO-E-1194GN	POWER SUPPLY AND GRID CONNECTIONS VICINITY MAP
OPERABLE BARRIERS		
95	CCO-C-1195OP	OLD RIVER FISH CONTROL STRUCTURE PLAN
96	CCO-C-4096OP	OLD RIVER FISH CONTROL STRUCTURE PROFILE

SHEET NO. LEGEND:

CCO-C-1001PP
FACILITY CODE
SHEET NUMBER
SHEET TYPE
DISCIPLINE
OPTION

FACILITY CODES:

GN	GENERAL
HH	HYDRAULICS
IT	INTAKES
TN	TUNNELS
IF	INTERMEDIATE FOREBAY
TS	TUNNEL SHAFTS
PP	CLIFTON COURT PUMPING PLANT
FB	CLIFTON COURT FOREBAY
OP	OPERABLE BARRIERS

SHEET TYPES:

0	GENERAL	5	DETAILS
1	PLANS	6	SCHEDULES AND DIAGRAMS
2	ELEVATIONS	7	USER DEFINED
3	SECTIONS	8	USER DEFINED
4	PLAN, SECTION AND DETAILS	9	3D REPRESENTATIONS

ABBREVIATIONS

@	AT
A	AMPERES
AF	AMPERE FRAME
APPD	APPROVED
APPROX	APPROXIMATE
AT	AMP TRIP
ATO	ALL TUNNEL OPTION
AUTO	AUTOMATIC
AUX	AUXILIARY
AVG	AVERAGE
BFV	BUTTERFLY VALVE
BOT	BOTTOM
BTF	BYRON TRACT FOREBAY
BUS	BINARY UNIT SYSTEM
C/C	CENTER TO CENTER
CCF	CLIFTON COURT FOREBAY
CDFG	CALIFORNIA DEPARTMENT OF FISH AND GAME
CE	CIVIL ENGINEER
CFS	CUBIC FEET PER SECOND
CL	CENTERLINE
CLR	CLEAR
CLSM	CONTROLLED LOW STRENGTH MATERIAL
CONC	CONCRETE
CS	CONSTANT SPEED
CVP	CENTRAL VALLEY PROJECT

ABBREVIATIONS -CONTINUED

DRF	DECANT RETURN FLOW
DIA	DIAMETER
DIMS	DIMENSIONS
DN	DOWN
DS	DOWNSTREAM, DREDGED SEDIMENT
DWG	DRAWING
E	EAST
(E), EXST	EXISTING
EG	EXISTING GRADE
EL	ELEVATION
FB	FOREBAY
FF	FINISH FLOOR
FG	FINISH GRADE
FLA	FULL LOAD AMPERE
FM	FLOW METER
FPS	FEET PER SECOND
FT	FEET
H	HEIGHT
HGL	HYDRAULIC GRADE LINE
HP	HORSE POWER
HPU	HYDRAULIC POWER UNIT
HS	HAND SWITCH
HWY	HIGHWAY
HZ	HERTZ
ICF	ISOLATED CONVEYANCE FACILITY
ID	INSIDE DIAMETER
IF	INTERMEDIATE FOREBAY
INV	INVERT
IPP	INTERMEDIATE PUMPING PLANT
KSI	KIPS PER SQUARE INCH
KV	KILOVOLTS
KVA	KILOVOLT-AMPERE
KW	KILOWATTS
LOR	LOCAL-OFF-REMOTE
LWL	LOW WATER LINE
M	MOTOR
MAX	MAXIMUM
MCC	MOTOR CONTROL CENTER
MECH	MECHANICAL
MIN	MINIMUM
MM	MILLIMETER
MVA	MEGAVOLT AMPERE
(N)	NEW
NAD	NORTH AMERICAN DATUM OF 1983
NAVD	NORTH AMERICAN VERTICAL DATUM OF 1988
NO	NUMBER
NOAA	NATIONAL OCEANIC ATMOSPHERIC ADMINISTRATION
NORM	NORMAL
NTS	NOT TO SCALE
OD	OUTSIDE DIAMETER
OG	ORIGINAL GROUND
OSC	OPEN/ STOP CLOSE
PF	POWER FACTOR
PH	PHASE
PNL	PANEL
PP	PUMPING PLANT
P&ID	PIPING AND INSTRUMENTATION DIAGRAM
PS	PUMP STATION
PSI	POUNDS PER SQUARE INCH
Q	FLOW RATE
R	RADIUS
RCC	REINFORCED CEMENT PIPE
RCP	REINFORCED CONCRETE PIPE
REG	REGISTERED
REV	REVISION
S	SLOPE
SED	SEDIMENTATION
SF	SQUARE FEET
SL	SPRING LINE
SLR	SEA LEVEL RISE
SQ	SQUARE
SS, SST	STAINLESS STEEL
STA	STATION
SUB	SUBMITTED
SWGR	SWITCHGEAR
SWP	STATE WATER PROJECT
TBD	TO BE DETERMINED
THK	THICK
TYP	TYPICAL
UD	UNDERDRAIN
UHMW	ULTRA HIGH MOLECULAR WEIGHT
UPRR	UNION PACIFIC RAILROAD
V	VELOCITY, VOLTS
VERT	VERTICAL
VFD	VARIABLE FREQUENCY DRIVE
W	WEST, WIDTH
w/	WITH
WC	WATER COIL
WL	WATERLINE
WSE	WATER SURFACE ELEVATION
WSP	WELDED STEEL PIPE
XFMR	TRANSFORMER
YR	YEAR
ZIT	POSITION INDICATING TRANSMITTER
ZSC	POSITION SWITCH CLOSED
ZSO	POSITION SWITCH OPEN
ø	DIAMETER

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DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 ▬ 1"

REV DATE DESCRIPTION SUB. APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

DRAWING INDEX

PROJECT NO. 240010
SHEET NO. CCO - G - 0002GN
REV 0
SEQUENCE NO. 2 OF 96

PLAN
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY

CONVEYANCE OVERVIEW
1 OF 2

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SHEET NO. CCO - G - 1003GN
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CONVEYANCE OVERVIEW
2 OF 2

PROJECT NO. 240010
SHEET NO. CCO - G - 1004GN
REV 0
SEQUENCE NO. 4 OF 96

A B C D E F H

PLAN

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PROFILE

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

OVERALL HYDRAULIC
PLAN AND PROFILE

VERIFY SCALE
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0 1"

PROJECT NO.
240010

SHEET NO.
CCO - C - 4005HH

REV	SEQUENCE NO.
0	5 OF 96

PLAN

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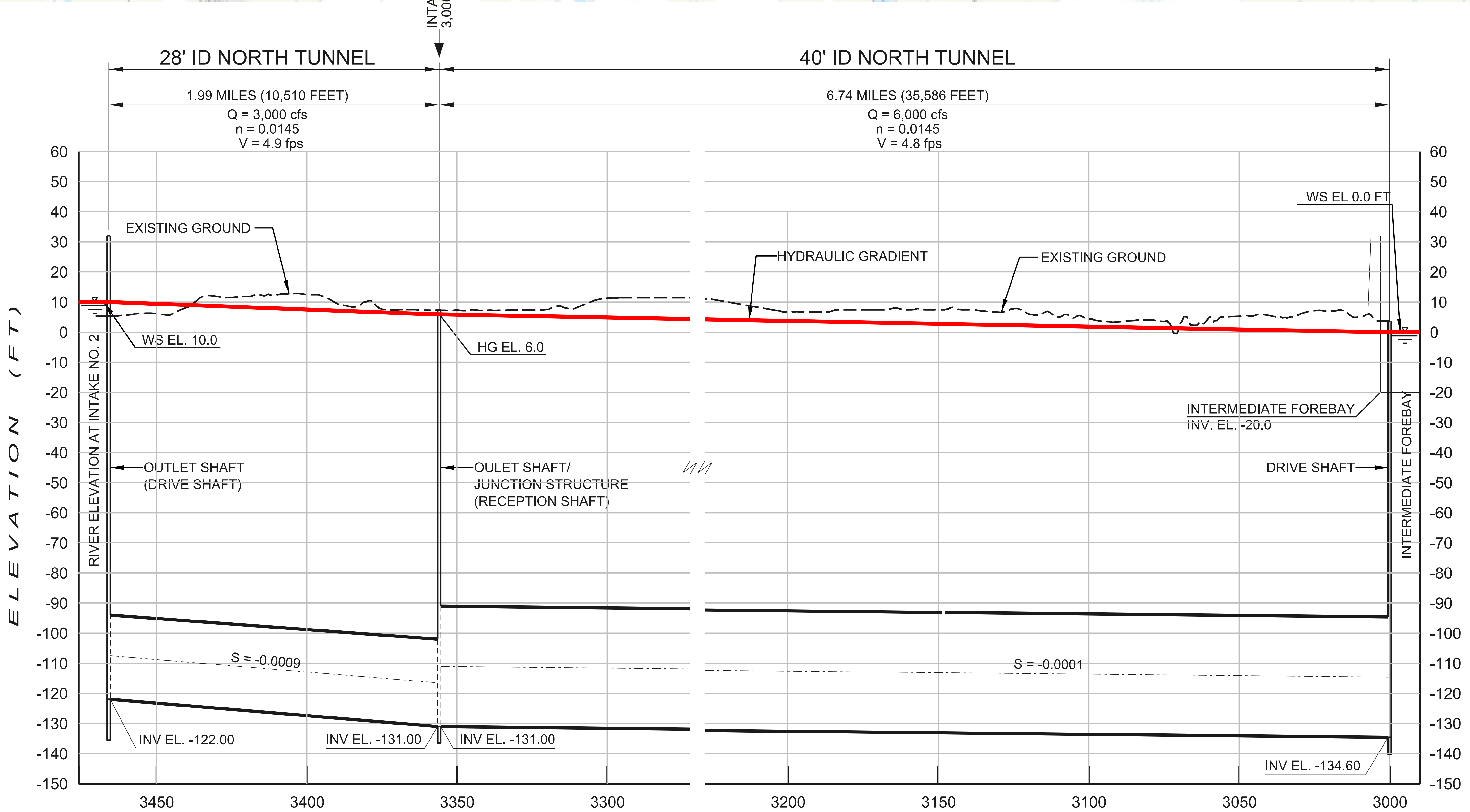
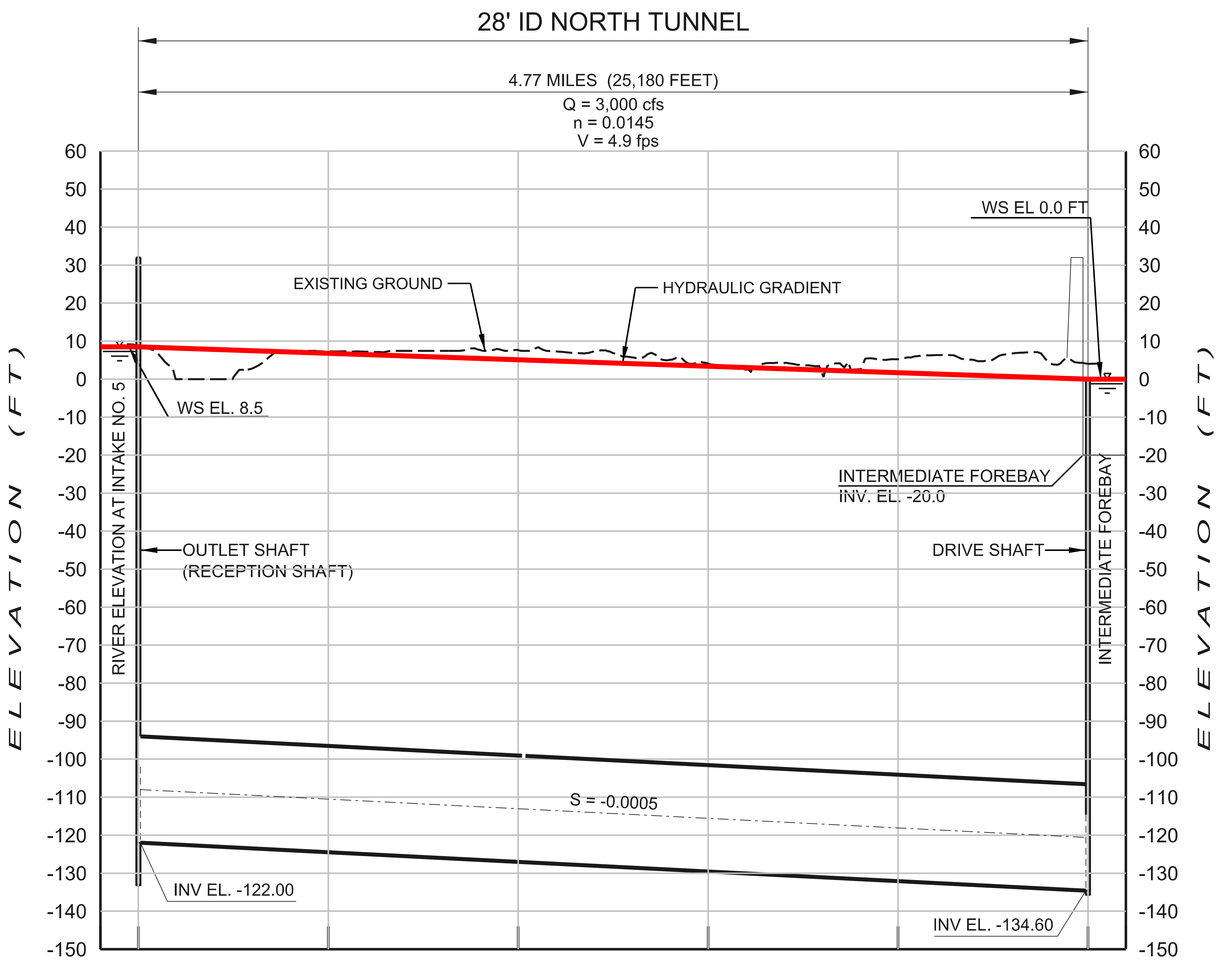


PROFILE 1 - INTAKE NO. 2 TO JUNCTION STRUCTURE TO INTERMEDIATE FOREBAY



PROFILE 2 - INTAKE NO. 5 TO INTERMEDIATE FOREBAY

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - HYDRAULIC
PLAN AND PROFILES

PROJECT NO. 240010
SHEET NO. CCO - C - 4006HH
REV 0
SEQUENCE NO. 6 OF 96

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Edited By: Bautista,Jav

Printed By: Bautista,Jav

PLAN

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DATE: APRIL 1, 2015

PROFILE

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - HYDRAULIC PLAN AND PROFILE

PROJECT NO.
240010
SHEET NO.
CCO - C - 4007HH
REV
0
SEQUENCE NO.
7 OF 96

NOTES:

1. ALL PUMPS ARE EQUIPPED WITH VARIABLE SPEED DRIVES.

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

OVERALL PROCESS FLOW DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - I - 6008GN
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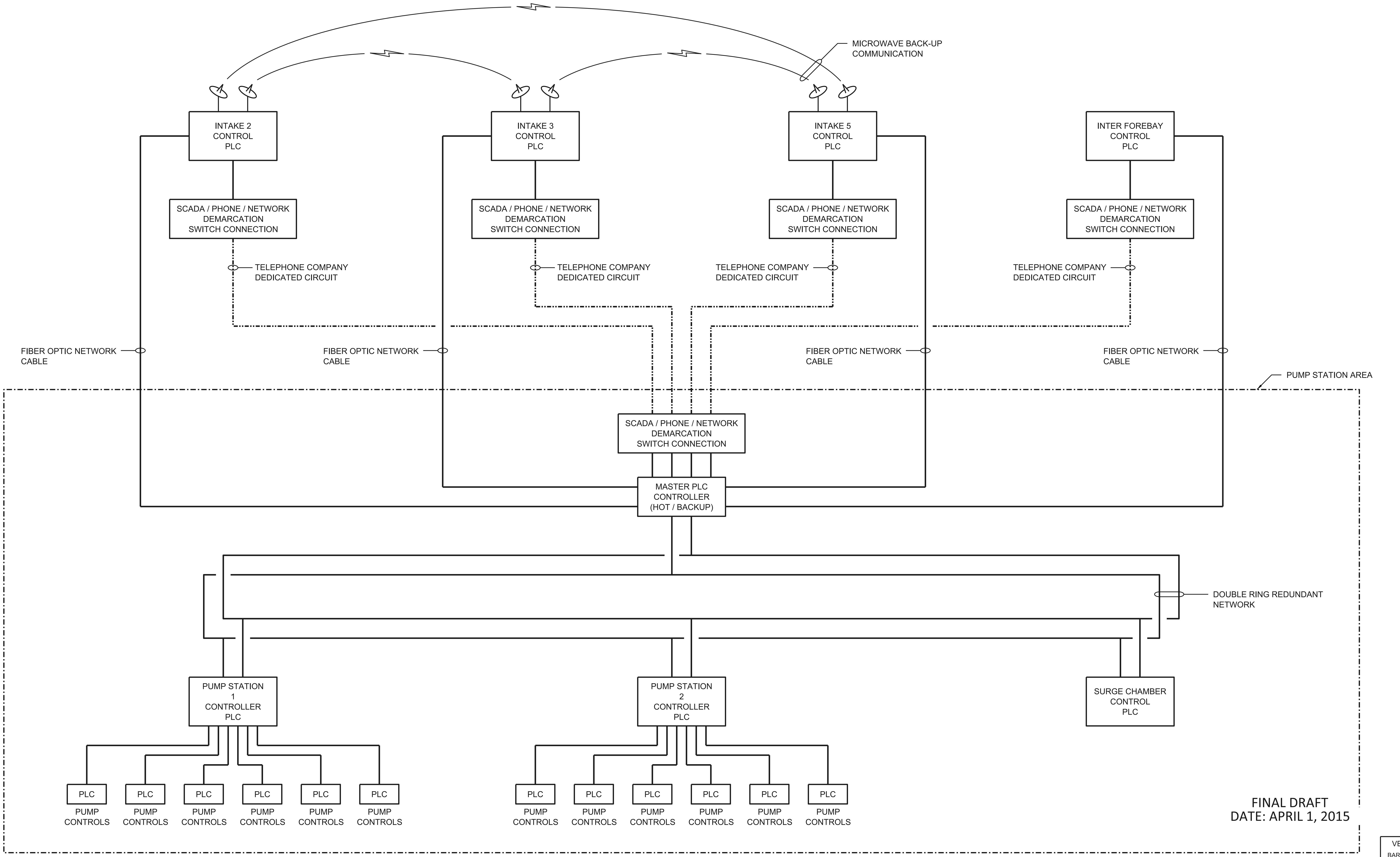

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CONTROL SYSTEM ARCHITECTURE

PROJECT NO. 240010
SHEET NO. CCO - I - 6009GN
REV 0
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL INTAKE FACILITY RENDERING

PROJECT NO. 240010
SHEET NO. CCO - G - 9010IT
REV 0
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Edited By: Mendez,Martin
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NOTE:

1. FOR INTAKE FACILITY GENERAL ARRANGEMENT, SEE SHEET CCO-M-1015IT

PLAN

1"=125'

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY

INTAKE NO. 2
SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1011IT
REV 0
SEQUENCE NO. 11 OF 96

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PLAN
1"=125'

NOTE:

1. FOR INTAKE FACILITY GENERAL ARRANGEMENT, SEE SHEET CCO-M-1015IT

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE NO. 3
SITE PLAN

PROJECT NO.
240010
SHEET NO.
CCO - C - 1012IT
REV
0
SEQUENCE NO.
12 OF 96

NOTE:

1. FOR INTAKE FACILITY GENERAL ARRANGEMENT, SEE SHEET CCO-M-1015IT

FINAL DRAFT
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PLAN
1"=125'

VERIFY SCALE
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY

INTAKE NO. 5
SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1013IT
REV 0
SEQUENCE NO. 13 OF 96

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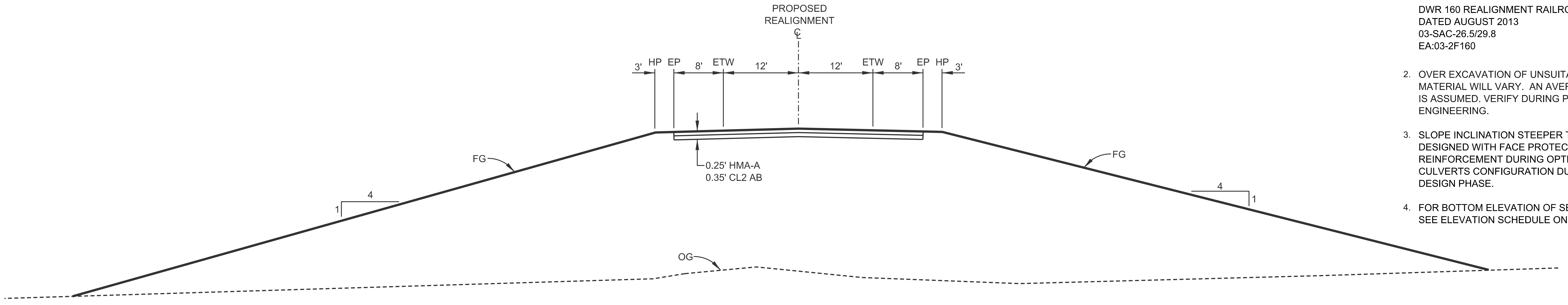
Printed By:

NOTES:

1. CROSS SECTION ARE BASED ON PRELIMINARY REPORT:

   DWR 160 REALIGNMENT RAILROAD DETOUR
   DATED AUGUST 2013
   03-SAC-26.5/29.8
   EA:03-2F160

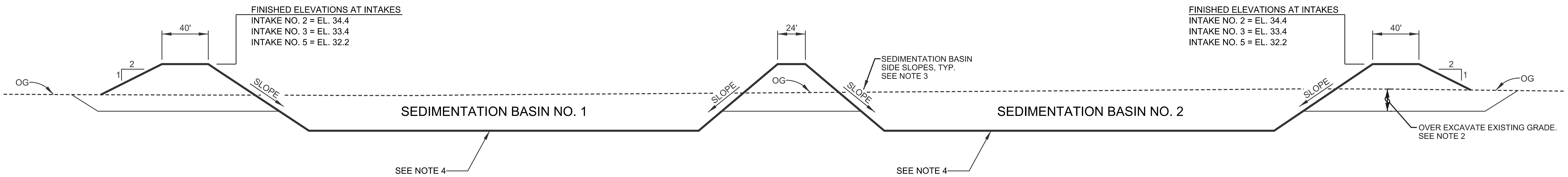
2. OVER EXCAVATION OF UNSUITABLE FOUNDATION MATERIAL WILL VARY. AN AVERAGE OF 6' IS ASSUMED. VERIFY DURING PRELIMINARY ENGINEERING.
3. SLOPE INCLINATION STEEPER THAN 2:1 WILL BE DESIGNED WITH FACE PROTECTION AND/OR SOIL REINFORCEMENT DURING OPTIMIZATION OF THE BOX CULVERTS CONFIGURATION DURING PRELIMINARY DESIGN PHASE.
4. FOR BOTTOM ELEVATION OF SEDIMENTATION BASINS, SEE ELEVATION SCHEDULE ON SHEET CCO-M-3018IT.



STATE ROUTE 160 REALIGNMENT - TYPICAL CROSS SECTION

NOTE: TYPICAL CROSS SECTION OF HIGHWAY 160 REALIGNMENT
IS LIMITED TO OUTSIDE OF INTAKE FACILITY BOUNDARY.

A SECTION
N.T.S.
CCO-C-1011IT
CCO-C-1012IT
CCO-C-1013IT



SEDIMENTATION BASINS - TYPICAL CROSS SECTION

B SECTION
N.T.S.
CCO-C-1011IT
CCO-C-1012IT
CCO-C-1013IT

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

STATE ROUTE 160 REALIGNMENT AND SED. BASINS
TYPICAL CROSS SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - C - 3014IT
REV 0
SEQUENCE NO. 14 OF 96

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1 PLAN
1" = 100'

VERIFY SCALE
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0 1"

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL INTAKE FACILITY
GENERAL ARRANGEMENT & KEY PLAN

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SHEET NO. CCO - M - 1015IT
REV 0
SEQUENCE NO. 15 OF 96

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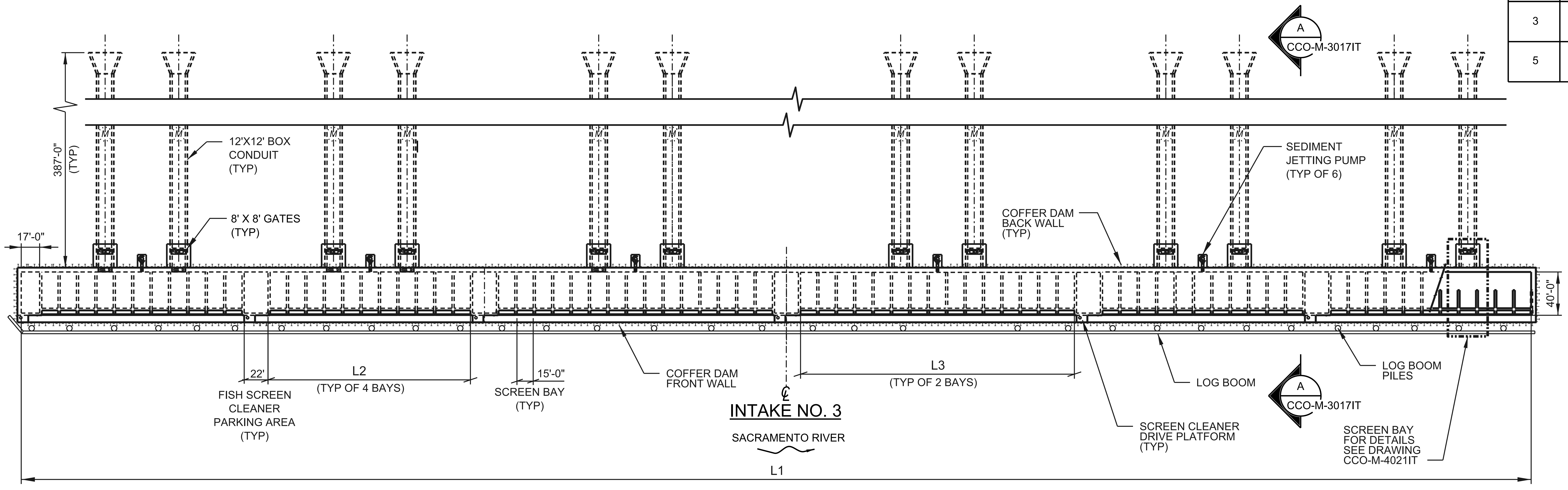
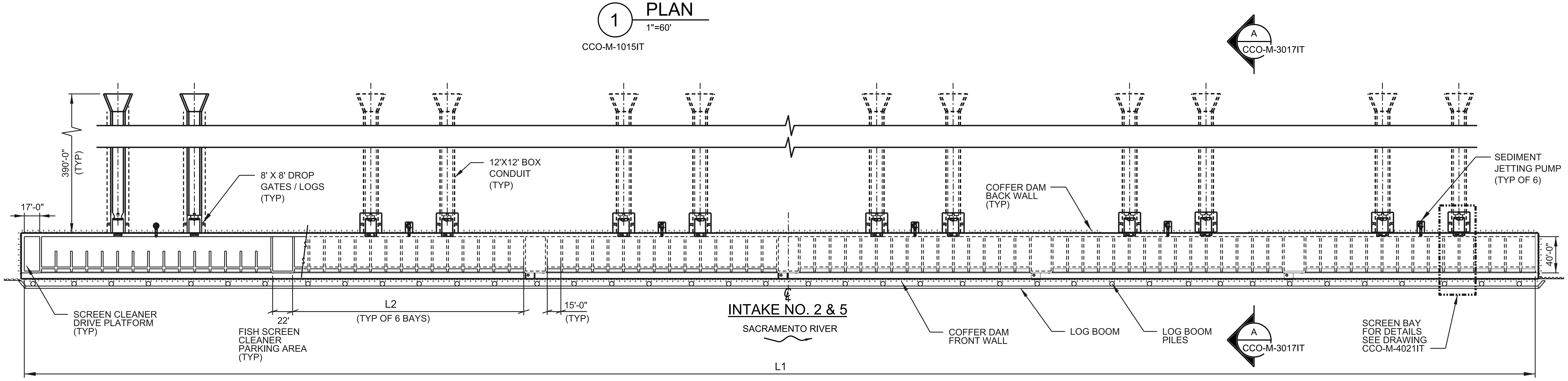
SCHEDULE

INTAKE NO.	APPROXIMATE RIVER MILE	NUMBER OF SCREEN BAYS	NUMBER OF STEEL CASED DRILLED PIERS	L1 (FT)	L2 (FT)	L3 (FT)
2	41.1	90	1,120	1,667	253	-
3	39.4	66	850	1,259	185	253
5	36.8	90	1,120	1,667	253	-



1 PLAN
1"=60'
CCO-M-1015IT



2 PLAN
1"=60'
CCO-M-1015IT

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE STRUCTURE
TYPICAL PLANS

PROJECT NO. 240010
SHEET NO. CCO - M - 4016IT
REV 0
SEQUENCE NO. 16 OF 96

ELEVATION SCHEDULE

INTAKE NO.	200-YR FLOOD WSE W/ SLR	DESIGN WSE	AVG RIVER BOT EL	A	B	C	D	E	F	G
2	31.4	1.9	-14	34.4'	-10.0	-12.0	-65.0	-85.0	-100.0	-100.0
3	30.4	1.6	-25	33.4'	-15.0	-17.0	-40.0	-60.0	-80.0	-100.0
5	28.4	0.7	-14	32.2'	-11.0	-13.0	-65.0	-55.0	-80.0	-100.0

ELEVATIONS SHOWN ARE BASED ON NAVD 88

NOTES:

1. GROUND IMPROVEMENT EXTENTS AND DEPTHS AND ALL TIP ELEVATIONS ARE PRELIMINARY AND BASED ON LIMITED GEOTECHNICAL DATA. FINAL EXTENTS AND ELEVATIONS ARE SUBJECT TO CHANGE DURING SUBSEQUENT ENGINEERING EFFORTS.

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A TYPICAL SECTION
1"=10'
CCO-M-4016IT

1" = 10' 0 10 20 30 FEET

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE STRUCTURE
TYPICAL SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 3017IT
REV 0
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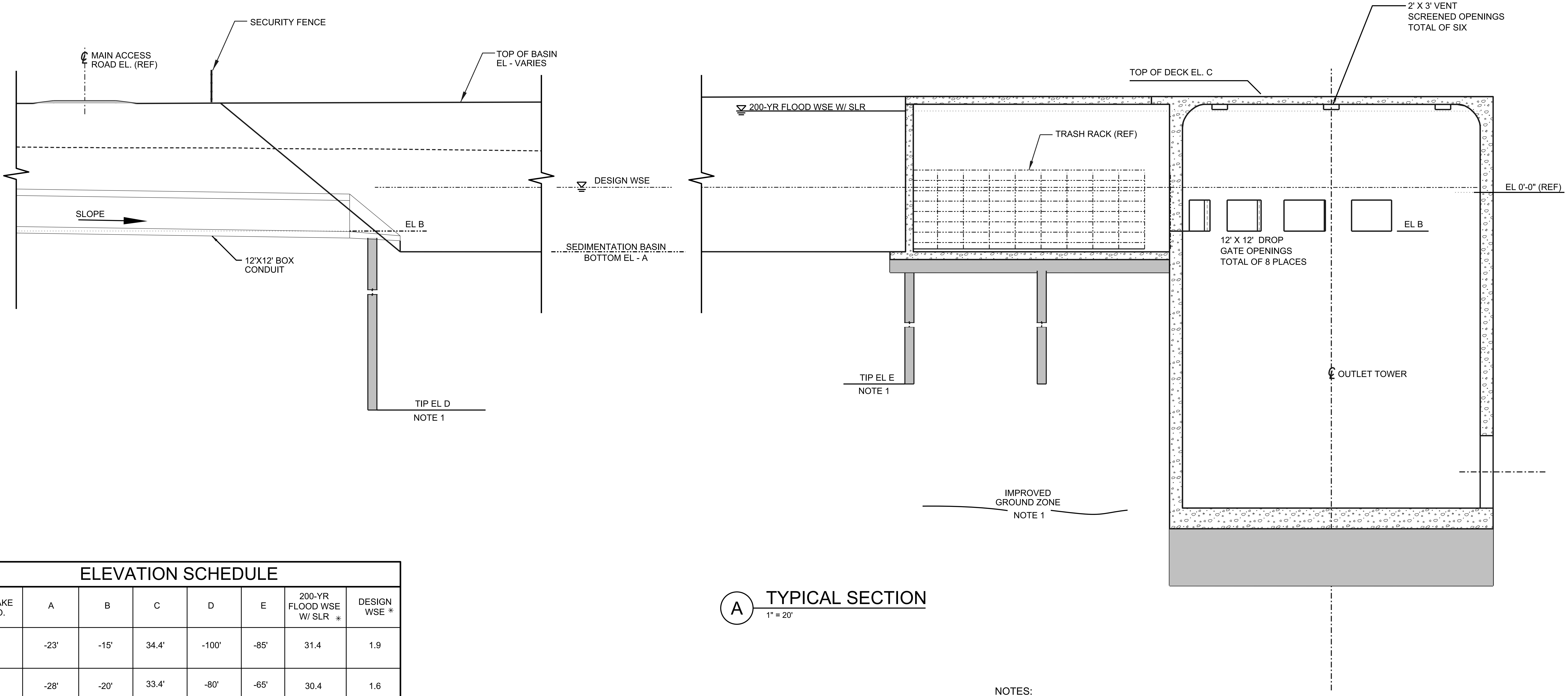
FOR CONTINUATION SEE CCO-M-3017IT



(A) TYPICAL SECTION

1" = 20'

ELEVATION SCHEDULE

INTAKE NO.	A	B	C	D	E	200-YR FLOOD WSE W/ SLR ✳	DESIGN WSE ✳
2	-23'	-15'	34.4'	-100'	-85'	31.4	1.9
3	-28'	-20'	33.4'	-80'	-65'	30.4	1.6
5	-24'	-16'	32.2'	-80'	-65'	28.4	0.7

✳ STATIC CONDITION IN SEDIMENTATION BASIN EQUAL TO ELEVATION IN RIVER.

NOTES:

1. GROUND IMPROVEMENT EXTENTS AND DEPTHS AND ALL TIP ELEVATIONS ARE PRELIMINARY AND BASED ON LIMITED GEOTECHNICAL DATA. FINAL EXTENTS AND ELEVATIONS ARE SUBJECT TO CHANGE DURING SUBSEQUENT ENGINEERING EFFORTS.

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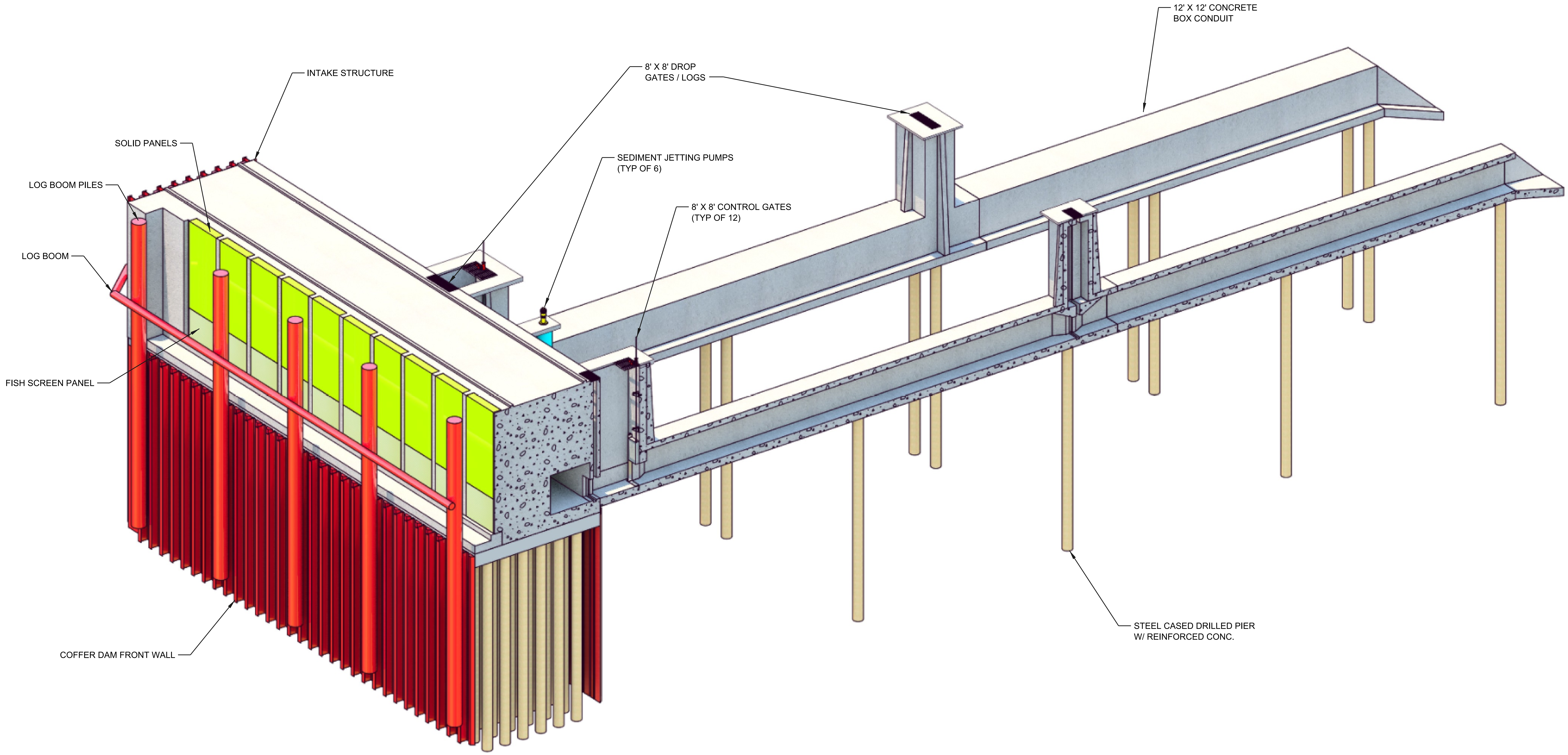
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

SEDIMENTATION BASIN
TYPICAL SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 3018IT
REV 0
SEQUENCE NO. 18 OF 96



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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL CONCRETE BOX CONDUIT ISOMETRIC

PROJECT NO. 240010
SHEET NO. CCO - M - 4019IT
REV 0
SEQUENCE NO. 19 OF 96

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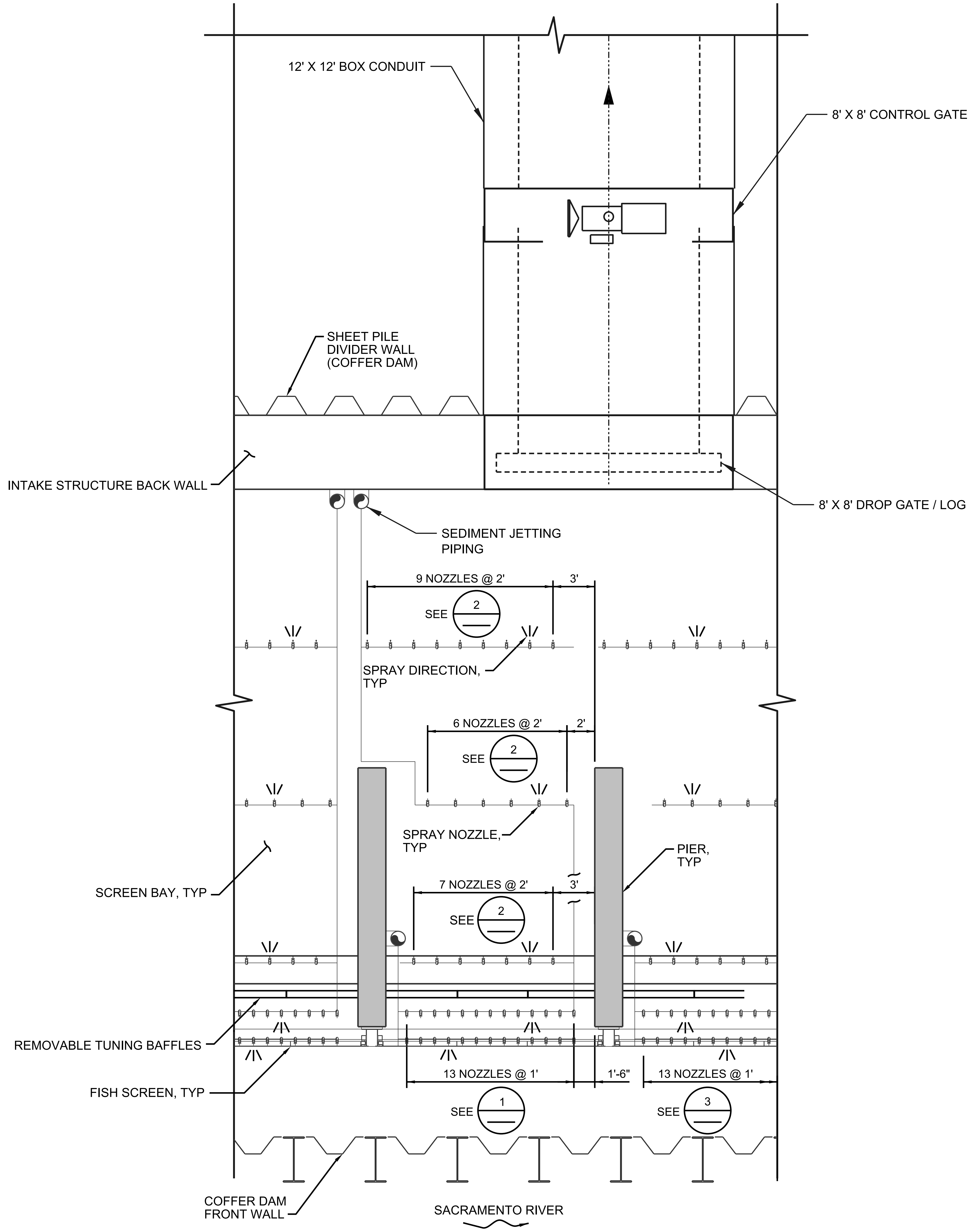
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

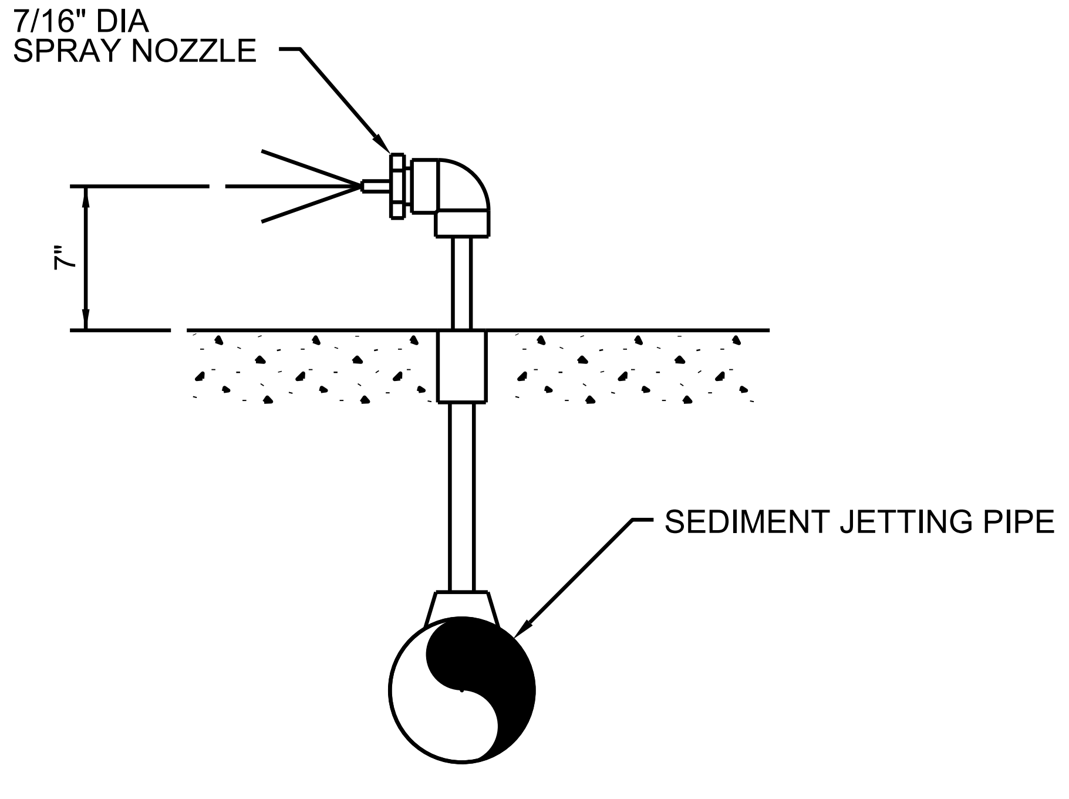
TYPICAL OUTLET SHAFT
ISOMETRIC

PROJECT NO. 240010
SHEET NO. CCO - M - 4020IT
REV 0
SEQUENCE NO. 20 OF 96



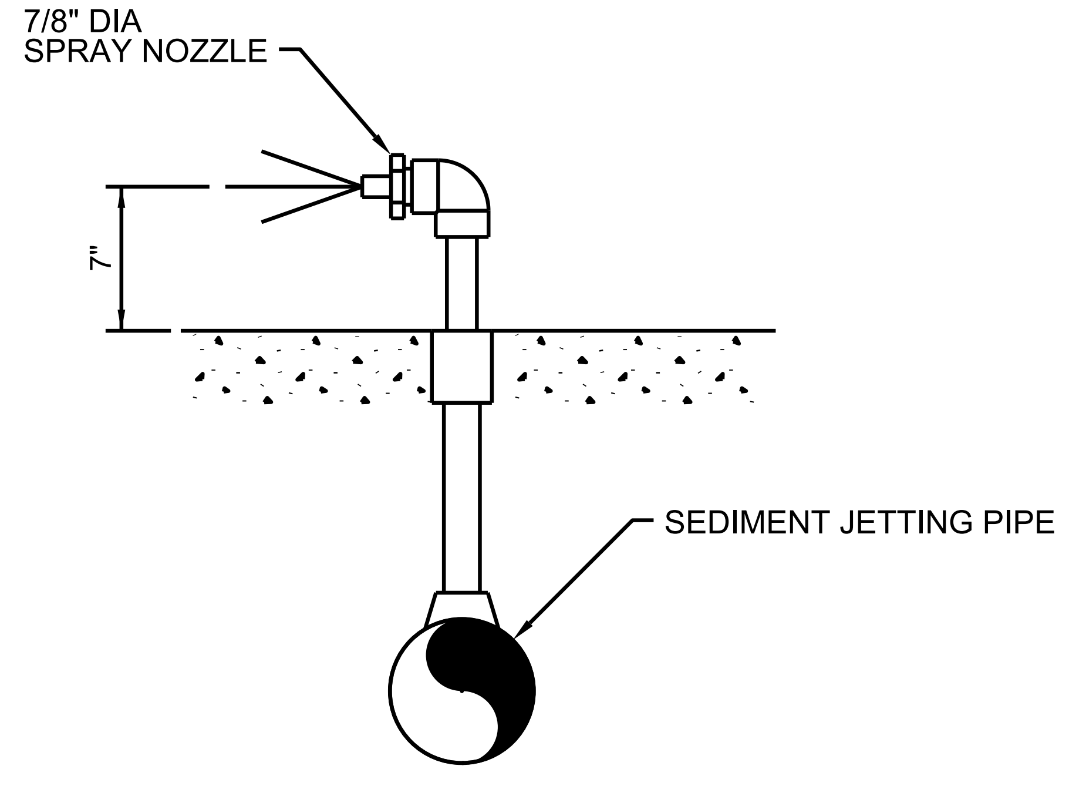
BELOW DECK

1 PARTIAL PLAN
3/16"=1'-0"
CCO-M-4016IT



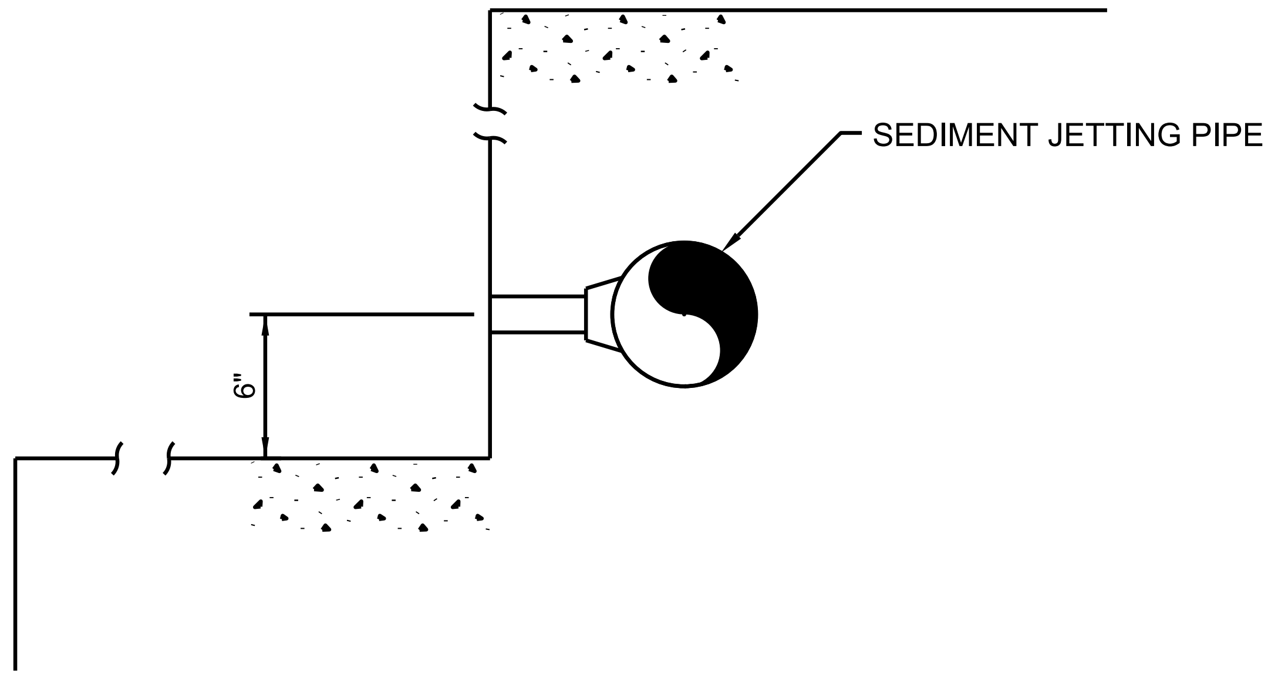
SPRAY NOZZLE

1 DETAIL
NTS



SPRAY NOZZLE

2 DETAIL
NTS



3 DETAIL
NTS

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE SEDIMENT JETTING
PARTIAL PLAN AND DETAILS

PROJECT NO. 240010
SHEET NO. CCO - M - 4021IT
REV 0
SEQUENCE NO. 21 OF 96

A ELEVATION

NTS

CCO-M-4021IT

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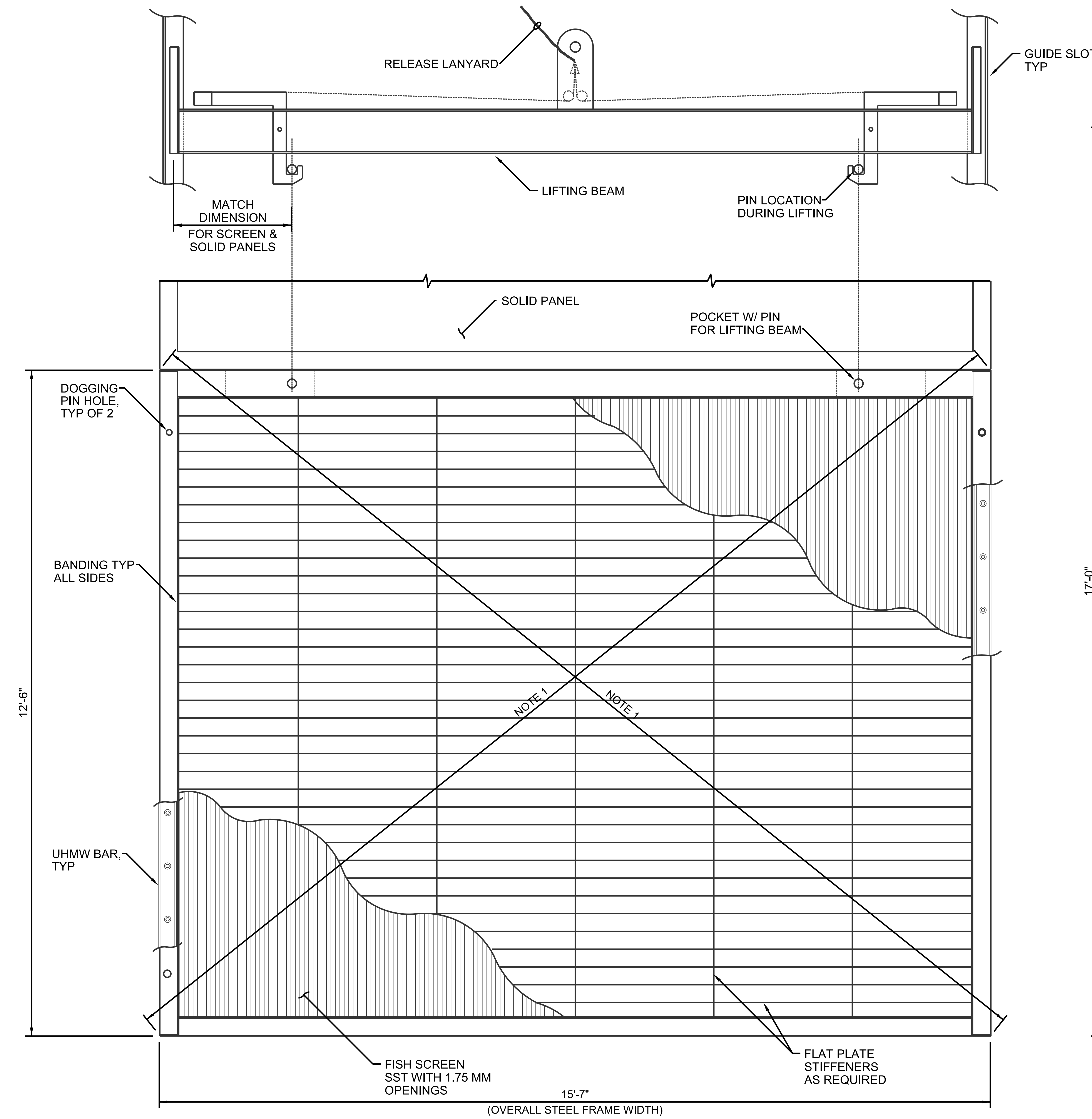
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

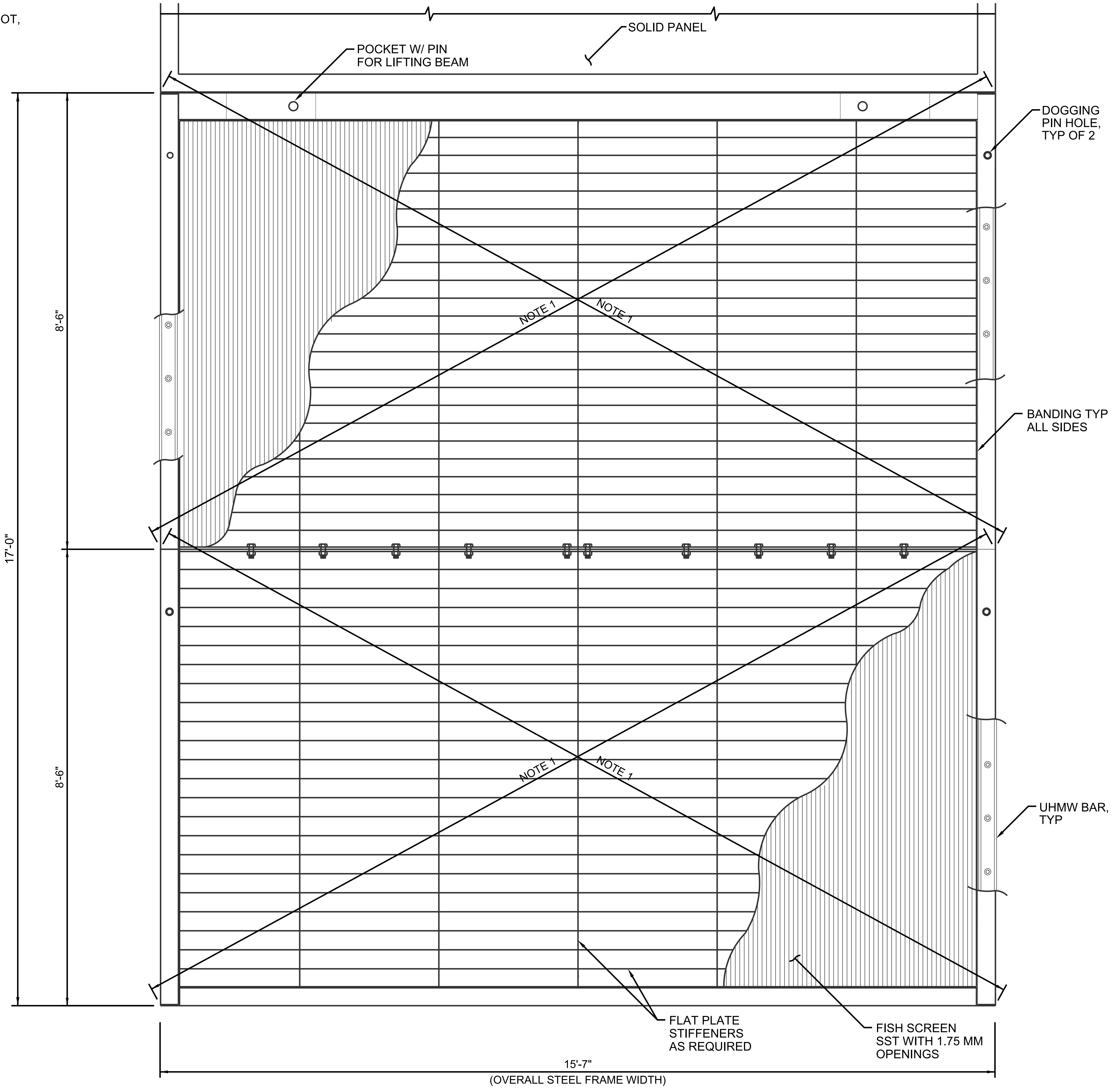
INTAKE STRUCTURE ELEVATION

PROJECT NO. 240010
SHEET NO. CCO - M - 2022IT
REV 0
SEQUENCE NO. 22 OF 96



INTAKE NO. 2 & NO. 5

A ELEVATION
3/4"=1'-0"
CCO-M-4021IT



INTAKE NO. 3

B ELEVATION
3/4"=1'-0"
CCO-M-4021IT

NOTES:

1. DIAGONAL DIMENSIONS OF ALL PANELS SHALL BE EQUAL. ALLOWABLE TOLERANCE IS +/- 1/8".
2. PROVIDE 3 SPARE FISH SCREEN PANELS PER INTAKE.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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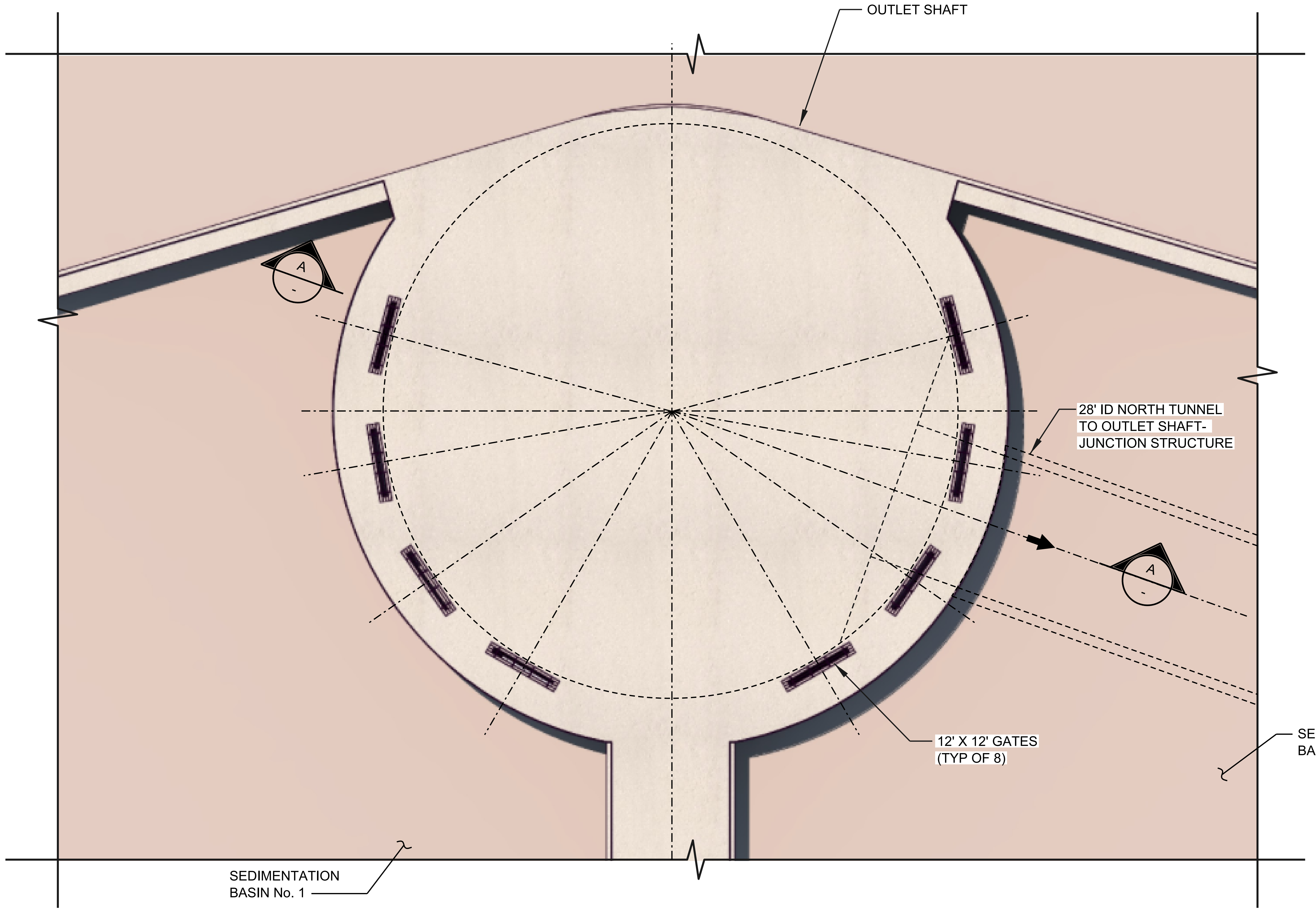
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE FISH SCREEN PANEL ELEVATIONS

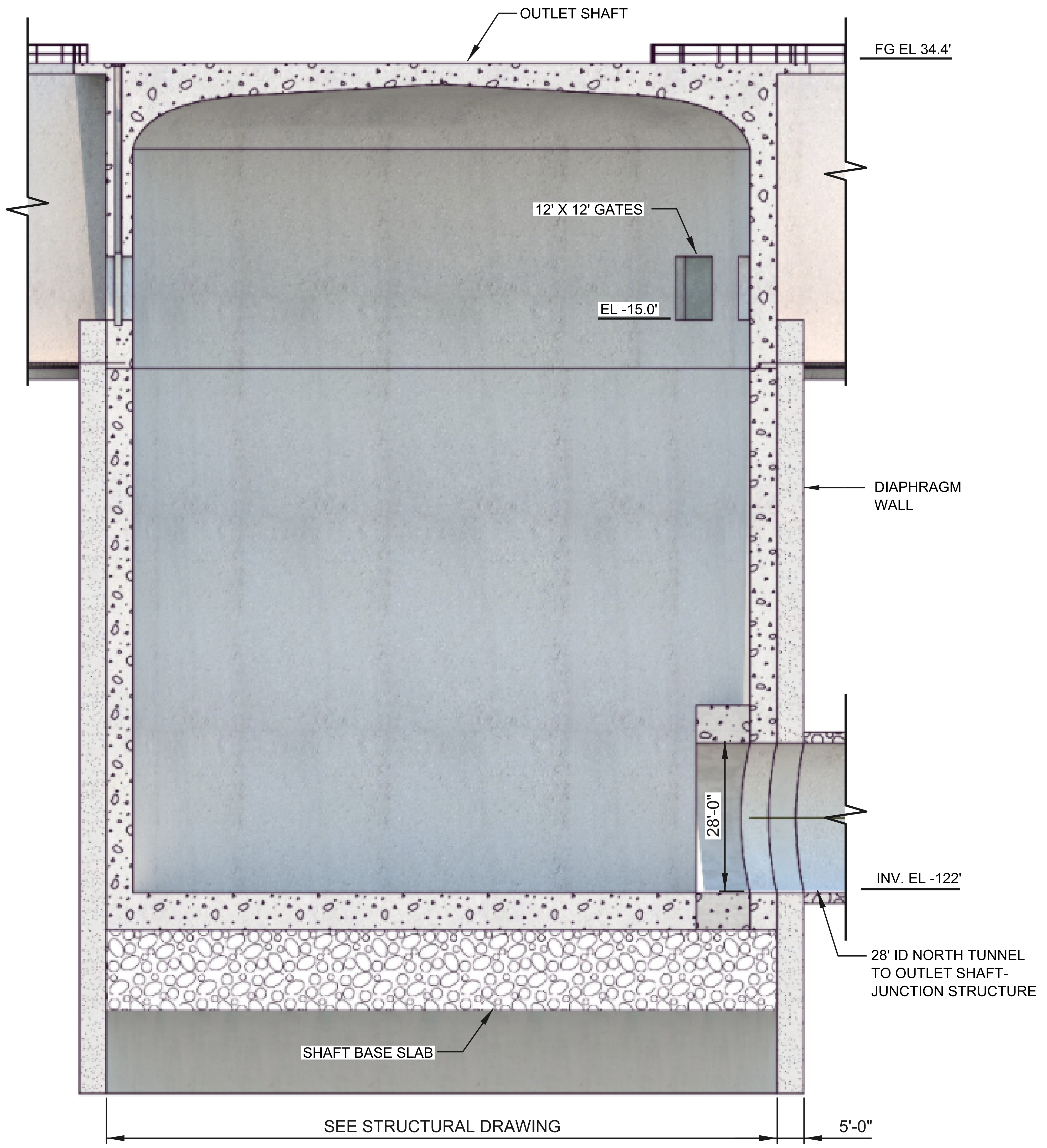
PROJECT NO. 240010
SHEET NO. CCO - M - 2023IT
REV 0
SEQUENCE NO. 23 OF 96



INTAKE No. 2

1 PLAN
1/16" = 1'-0"

CCO-M-1015IT



A SECTION
1/16" = 1'-0"

FINAL DRAFT
DATE: APRIL 1, 2015

1/16 " = 1'-0"   0   16   32   48 FEET

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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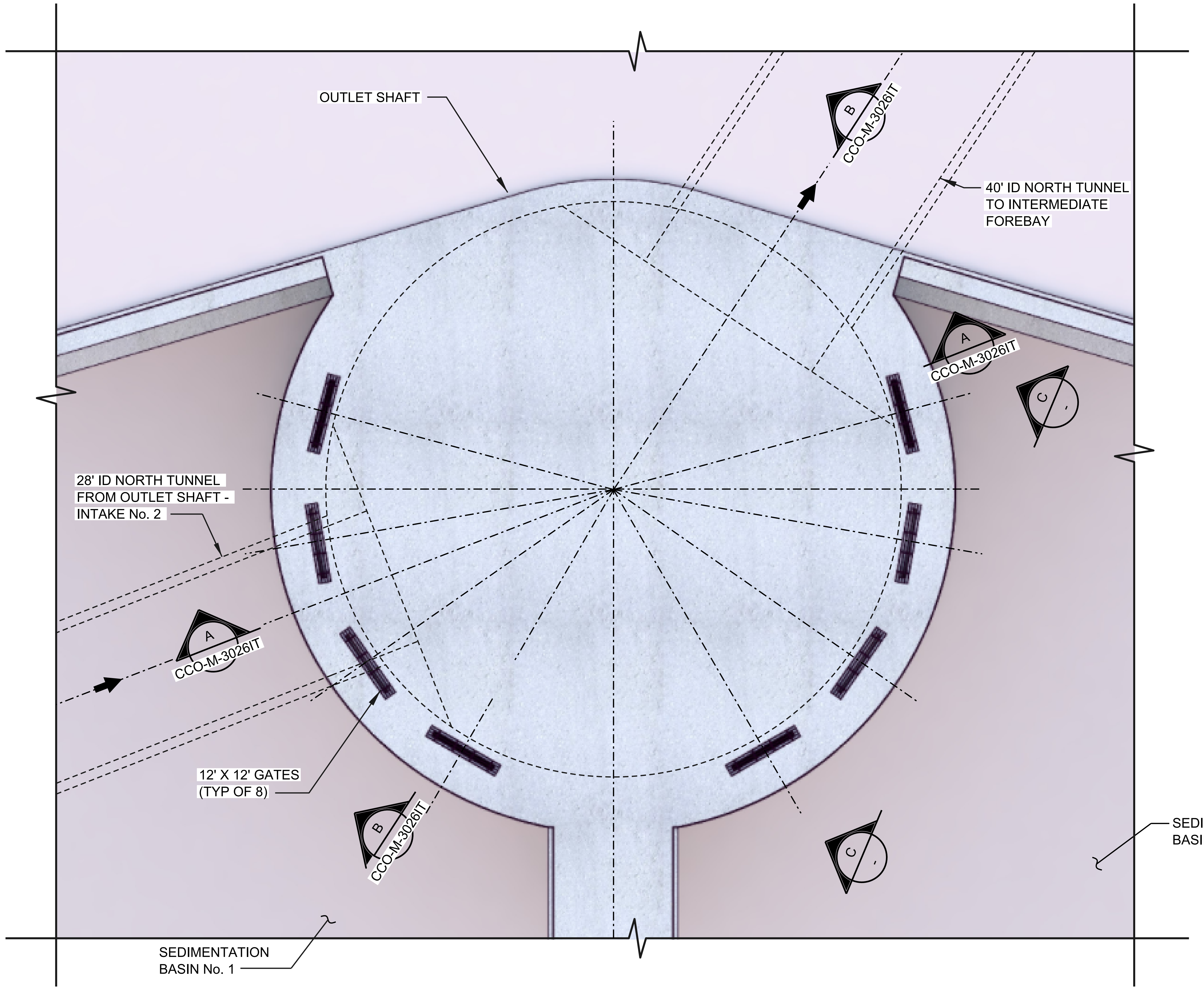
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

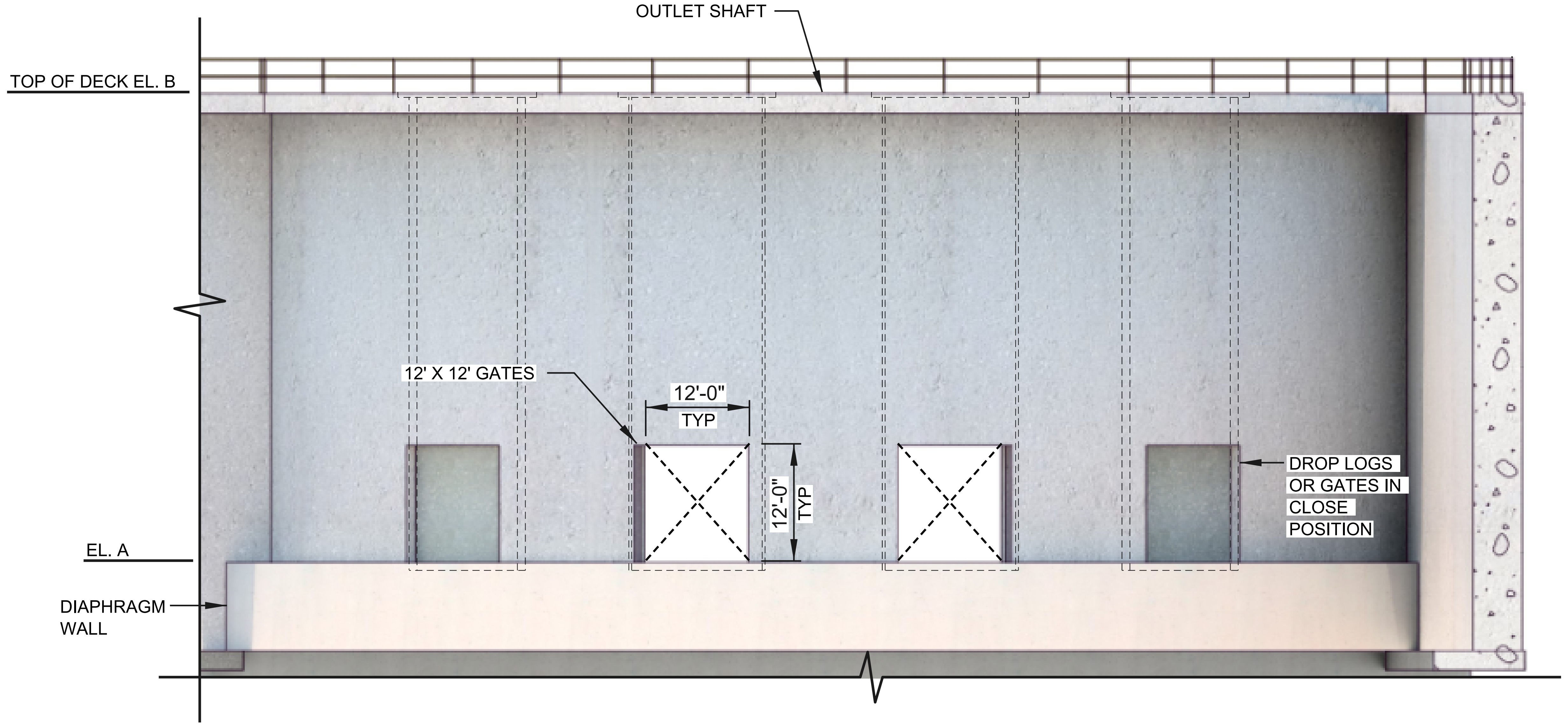
OUTLET SHAFT @ INTAKE NO. 2
PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 4024IT
REV 0
SEQUENCE NO. 24 OF 96



INTAKE No. 3

1 PLAN
1/16" = 1'-0"
CCO-M-1015IT



TYPICAL FOR INTAKE No. 2, 3 & 5

C SECTION
3/32" = 1'-0"

ELEVATION SCHEDULE

INTAKE NO.	A	B
2	-15'	34.4'
3	-20'	33.4'
5	-16'	32.2'

FINAL DRAFT
DATE: APRIL 1, 2015

3/32" = 1'-0" 0 16 32 FEET

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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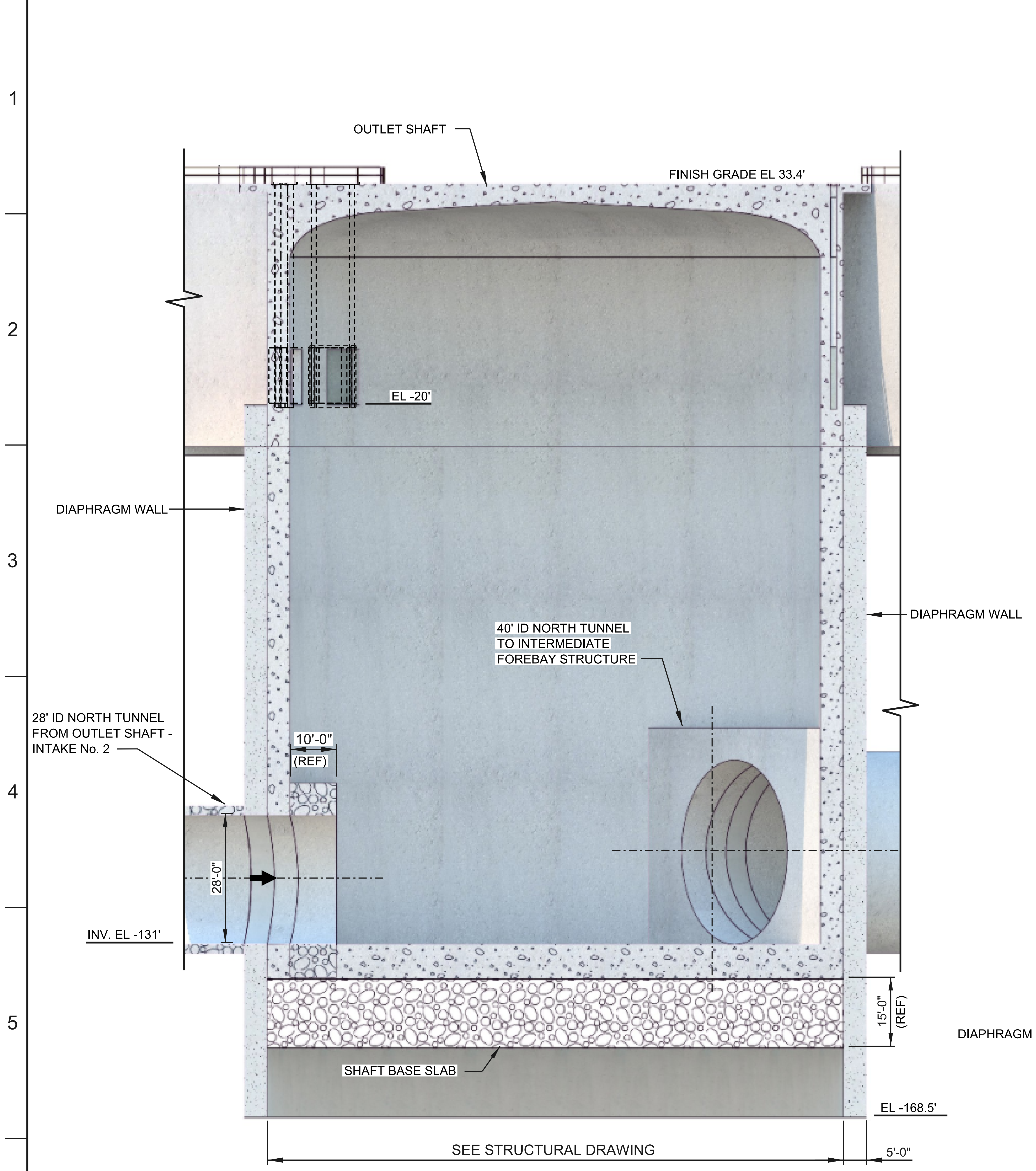
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

OUTLET SHAFT @ INTAKE NO. 3
PLAN AND SECTION

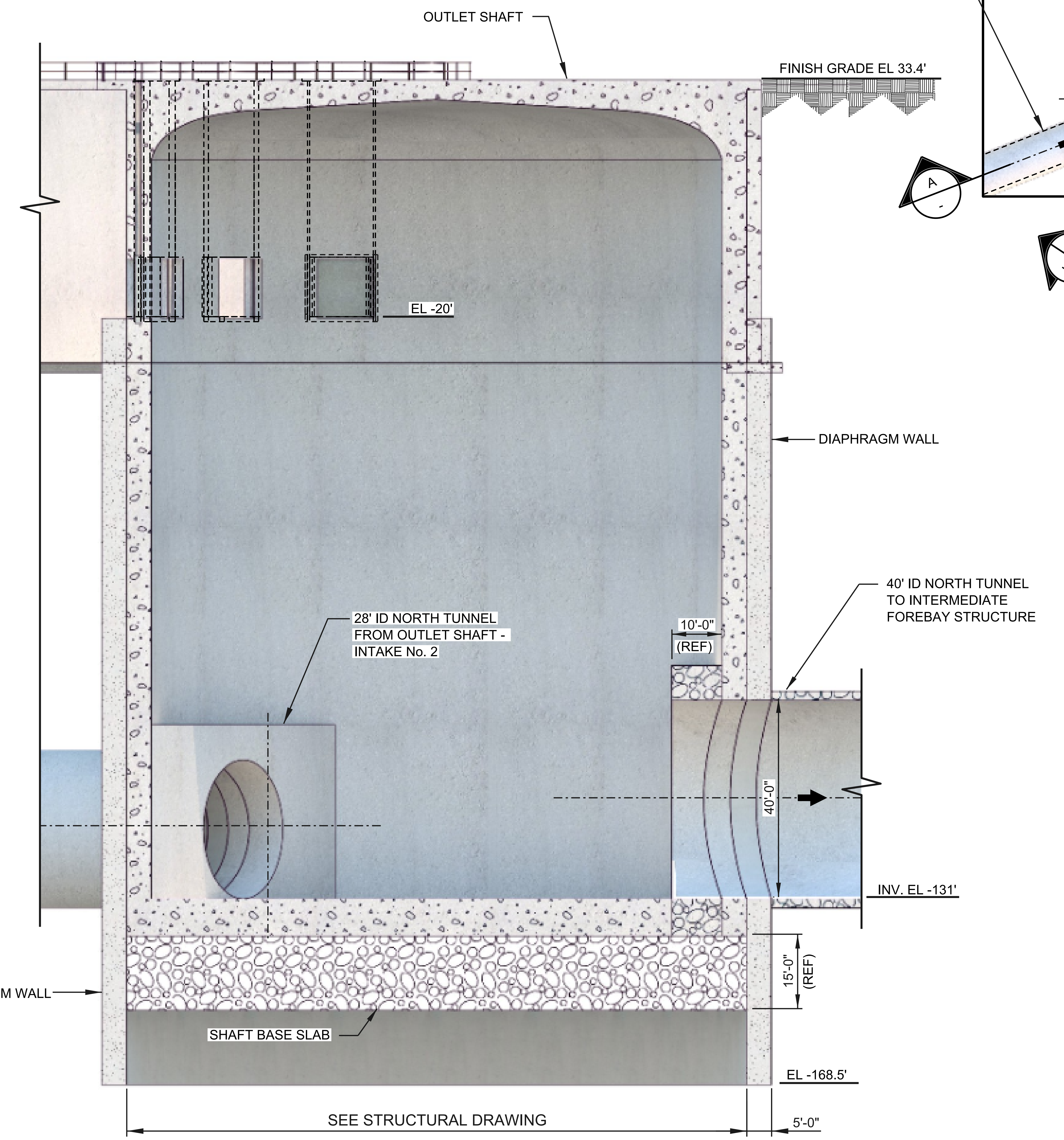
PROJECT NO. 240010
SHEET NO. CCO - M - 4025IT
REV 0
SEQUENCE NO. 25 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\06 Mechanical\CCO-M-025-240010.dgn
Edited By: Mendez,Martin
Printed By: Mendez,Martin



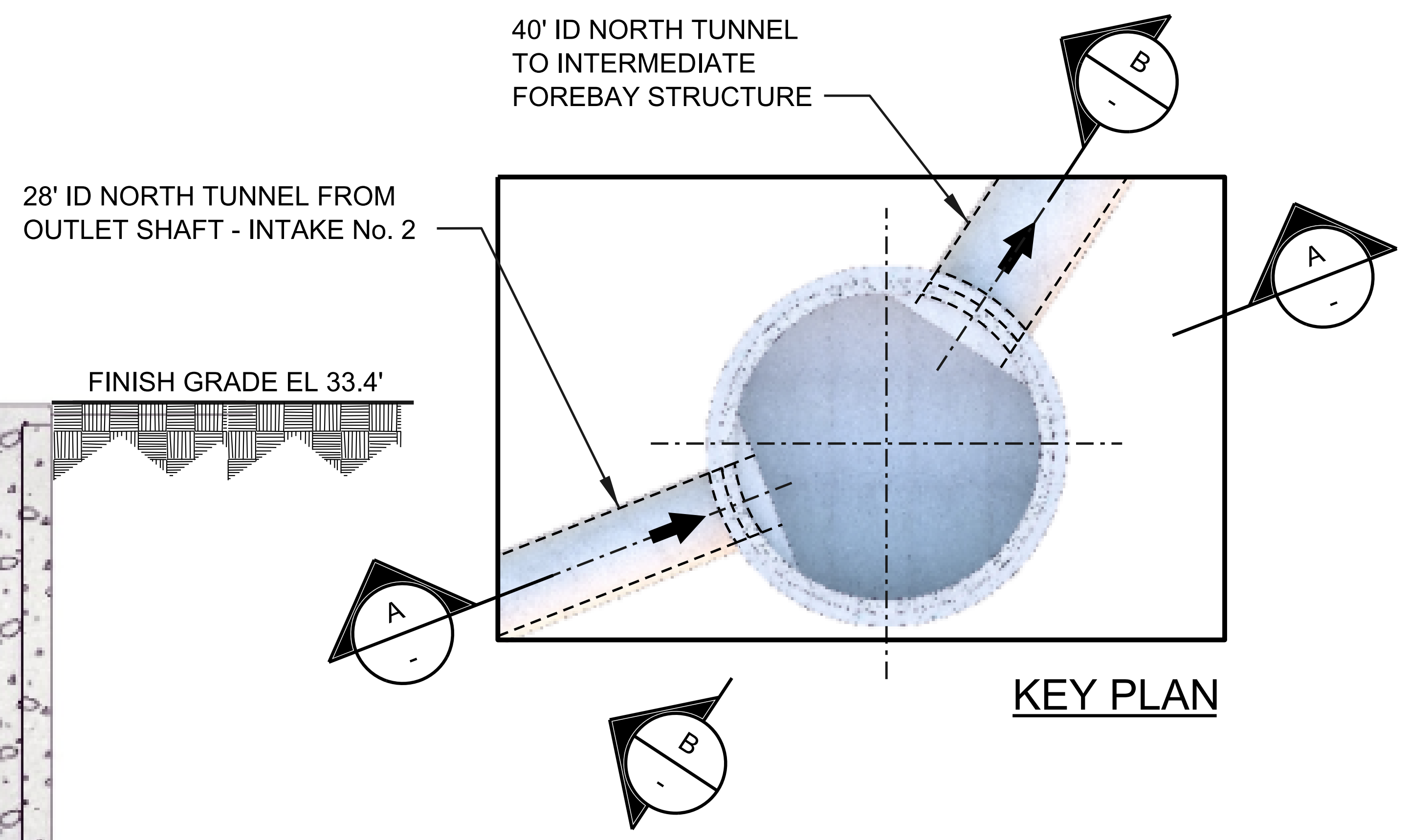
OUTLET SHAFT / JUNCTION STRUCTURE

A SECTION
1/16" = 1'-0"
CCO-M-4025IT



OUTLET SHAFT / JUNCTION STRUCTURE

B SECTION
1/16" = 1'-0"
CCO-M-4025IT



KEY PLAN

1/16 " = 1'-0" 0 16 32 48 FEET

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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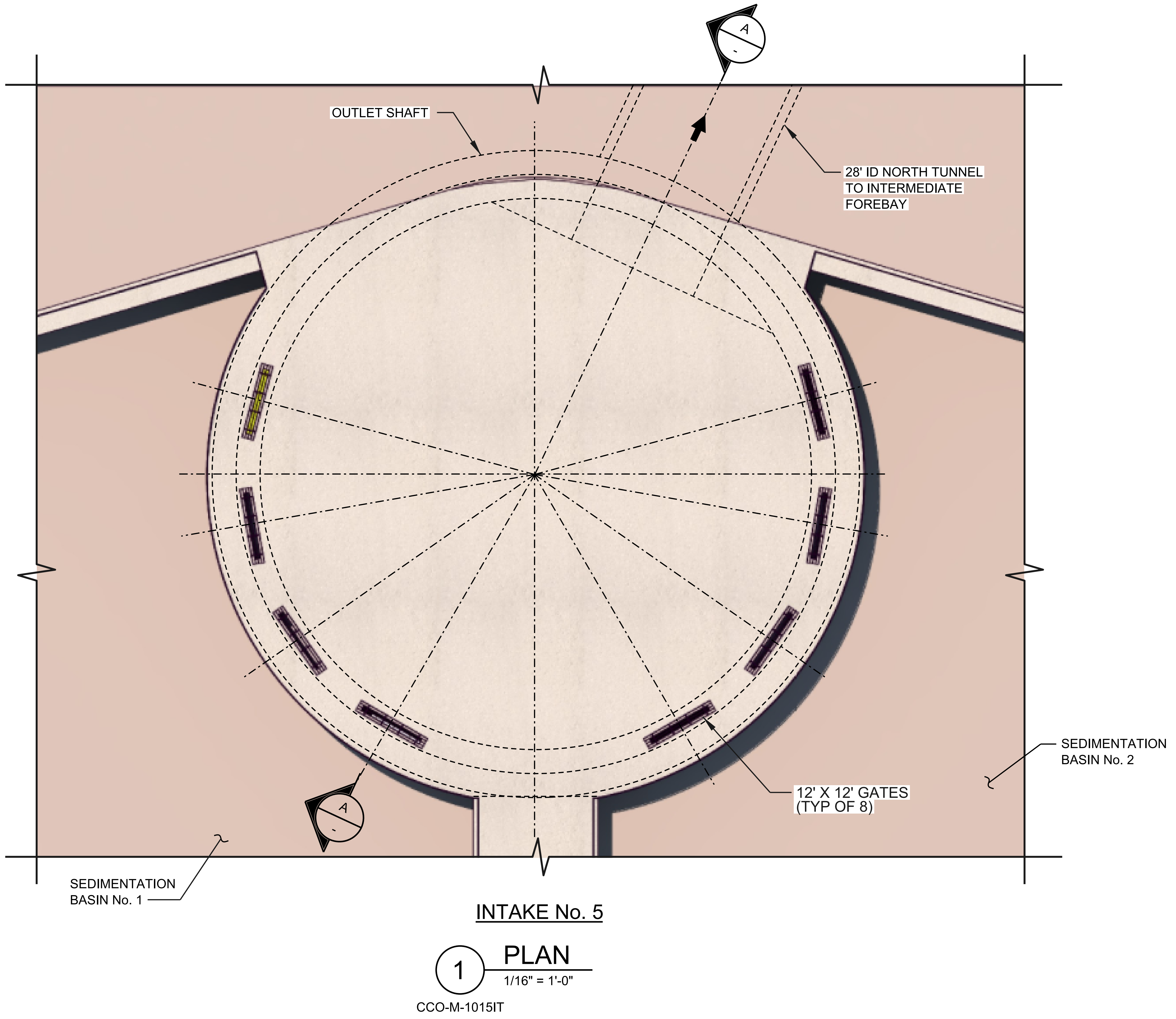
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

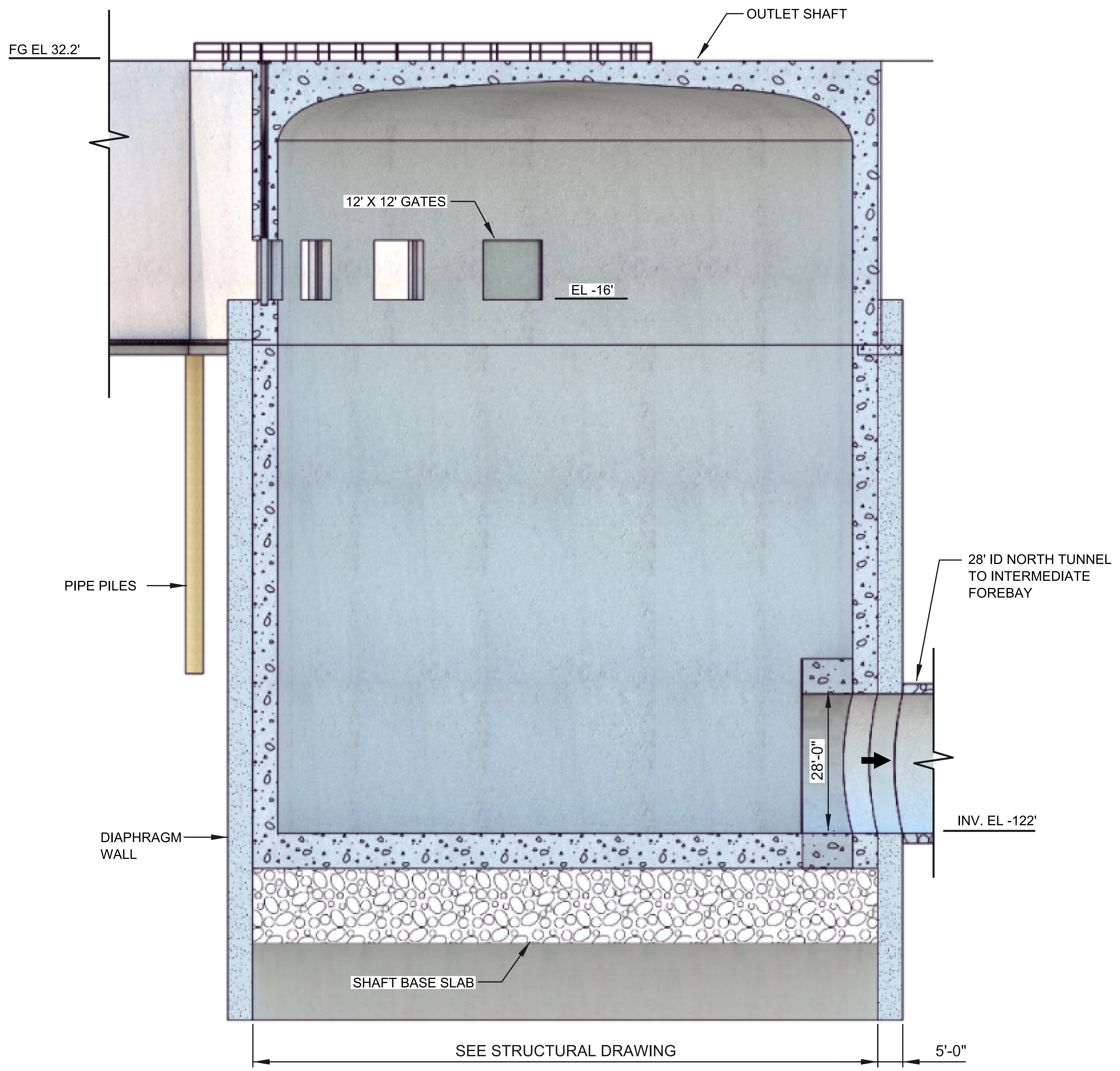
OUTLET SHAFT @ INTAKE NO. 3
SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - M - 3026IT
REV 0
SEQUENCE NO. 26 OF 96



INTAKE No. 5

1 PLAN
1/16" = 1'-0"
CCO-M-1015IT



A SECTION
1/16" = 1'-0"

FINAL DRAFT
DATE: APRIL 1, 2015

1/16 " = 1'-0" 0 16 32 48 FEET

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD	DESIGNED	APPROVAL RECOMMENDED
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

OUTLET SHAFT @ INTAKE NO. 5
PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 4027IT
REV 0
SEQUENCE NO. 27 OF 96

PLAN
1"=60'

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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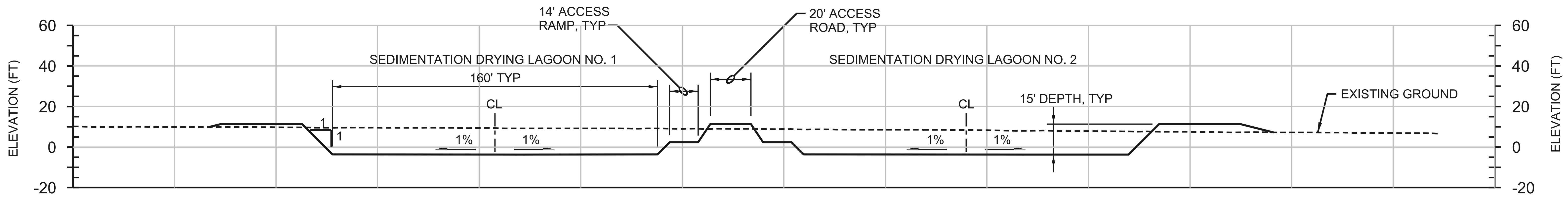
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

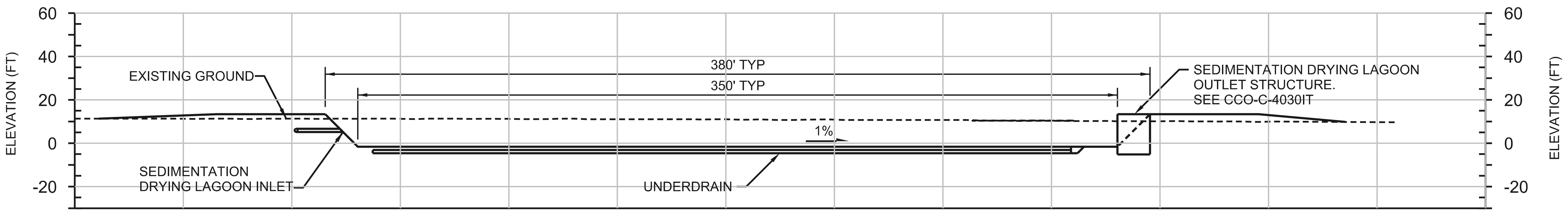
SEDIMENTATION DRYING LAGOON
TYPICAL PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 4028IT
REV 0
SEQUENCE NO. 28 OF 96



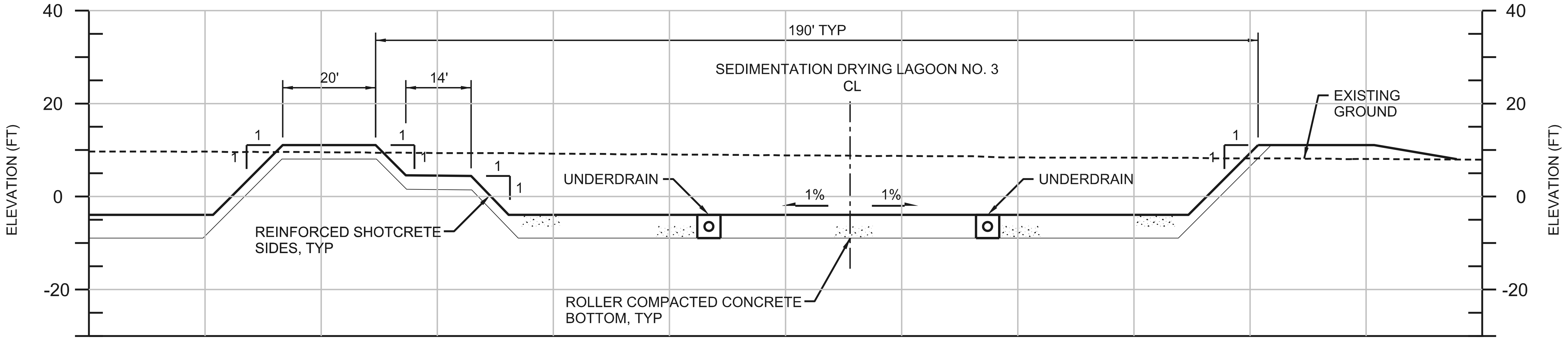
A SECTION
1"=40'

CCO-C-4028IT



B SECTION
1"=40'

CCO-C-4028IT



C SECTION
1"=20'

CCO-C-4028IT

NOTES:

1. SOLIDS LAGOON SECTIONS ARE REPRESENTATIVE OF INTAKE NO. 2. INTAKES NO. 3 AND 5 ARE SIMILAR IN SIZE AND DEPTH, EXCEPT SURROUNDING FINISH GRADE ELEVATIONS VARY.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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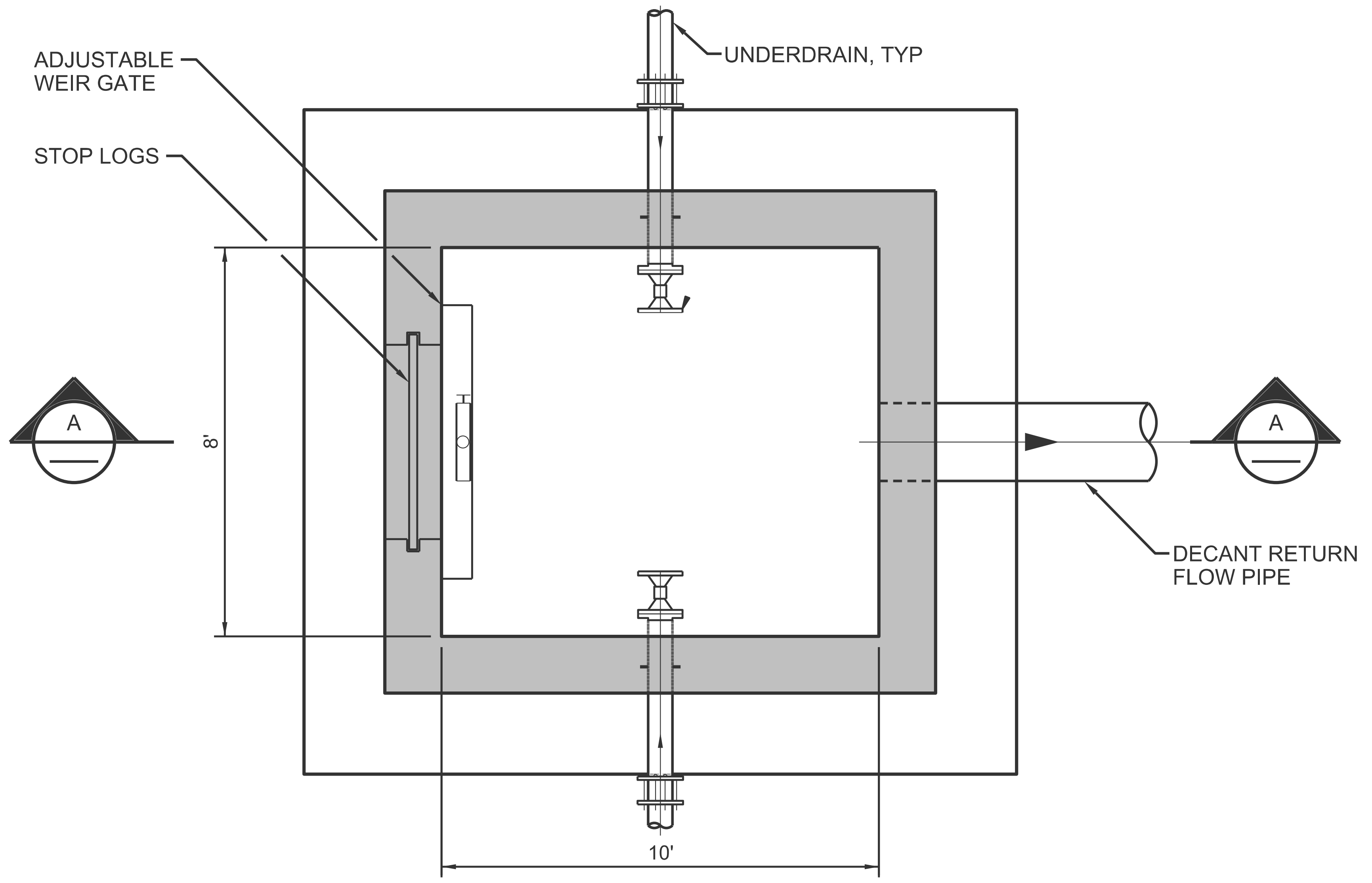
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

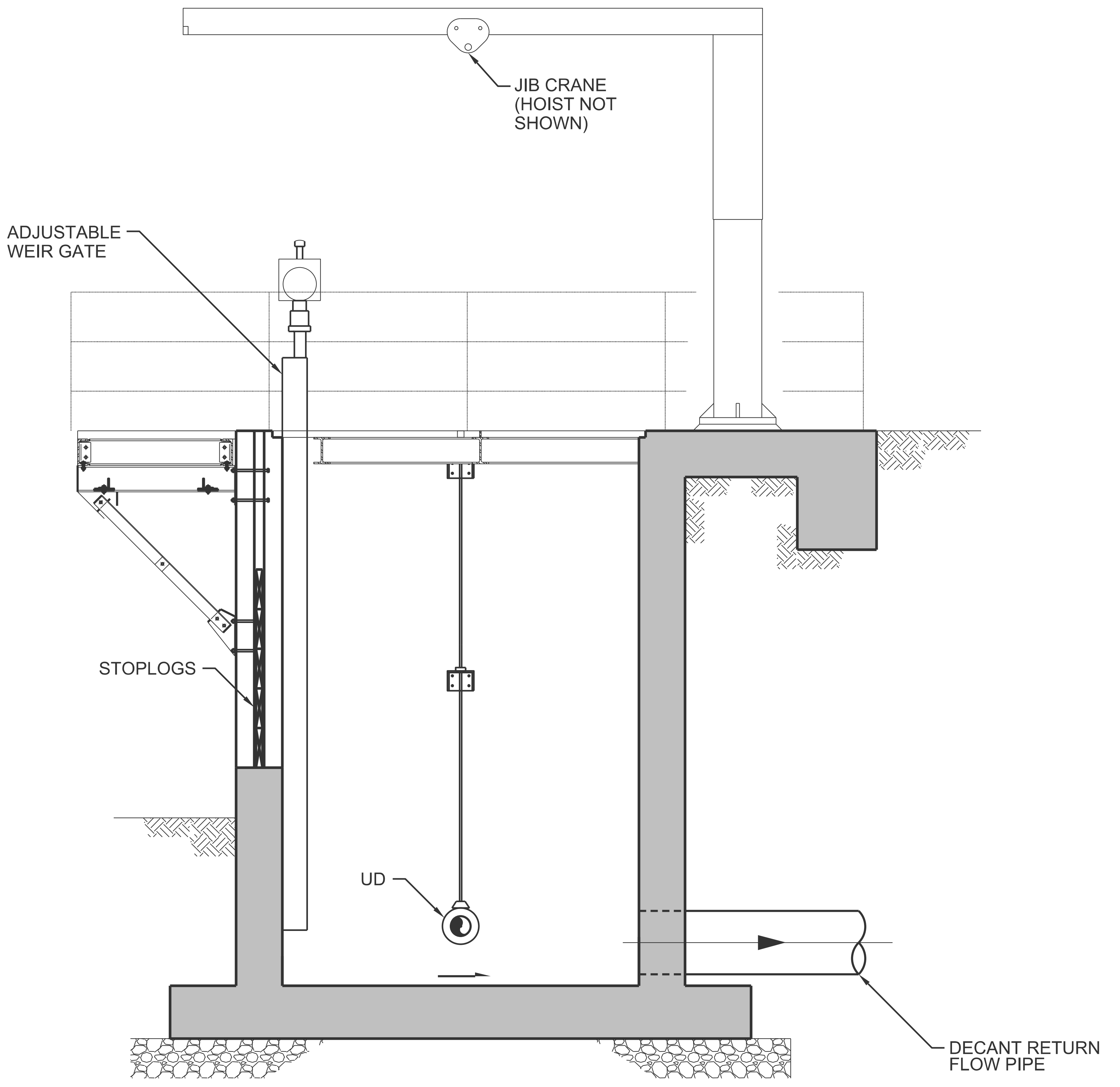
SEDIMENTATION DRYING LAGOON TYPICAL SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - C - 3029IT
REV 0
SEQUENCE NO. 29 OF 96



SOLIDS LAGOON OUTLET STRUCTURE

1 PLAN
1"=3/8"=1'-0"
CCO-C-4028IT



A SECTION
3/8"=1'-0"

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DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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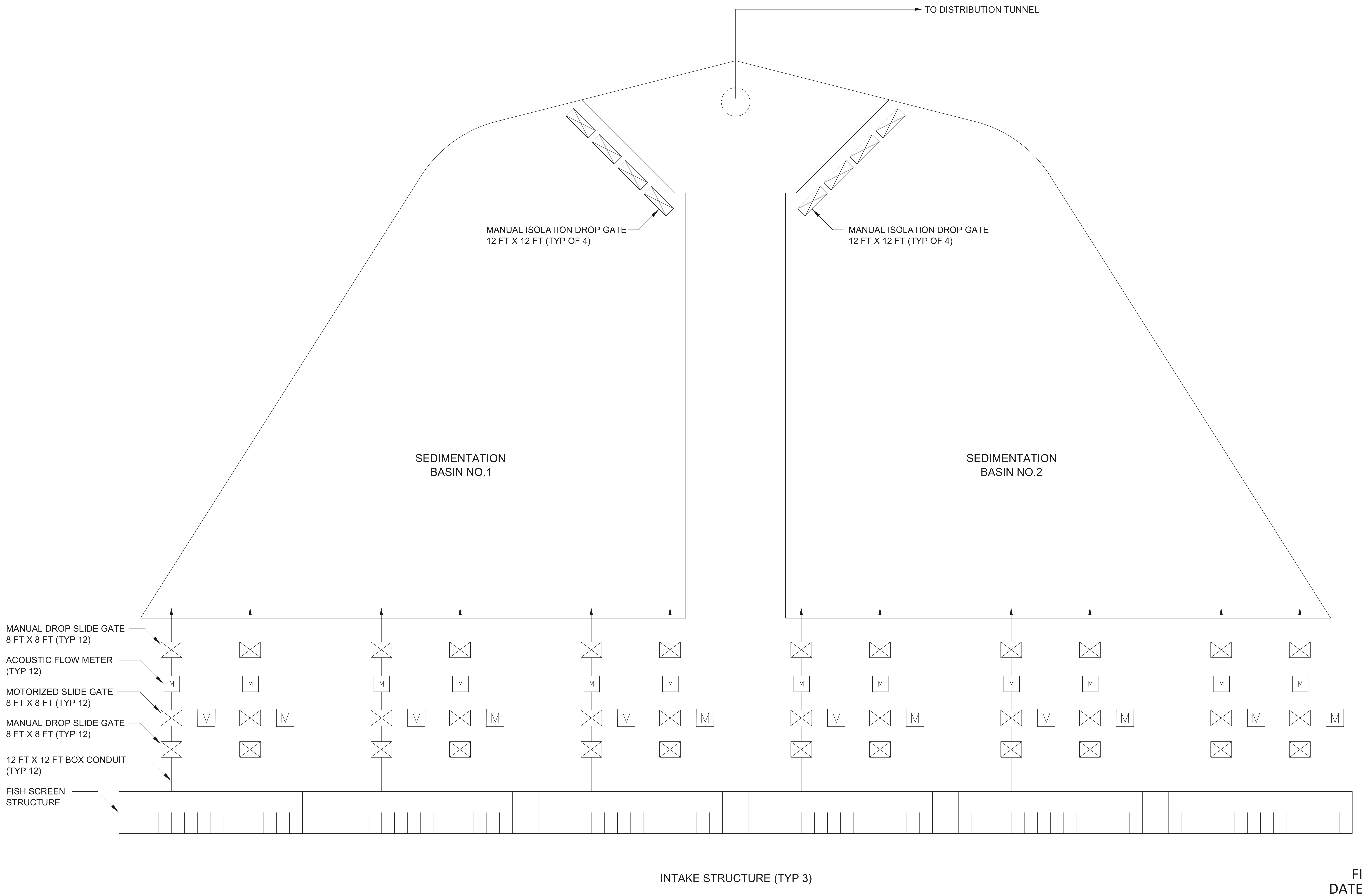
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

SEDIMENTATION DRYING LAGOON
OUTLET PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - C - 4030IT
REV 0
SEQUENCE NO. 30 OF 96



FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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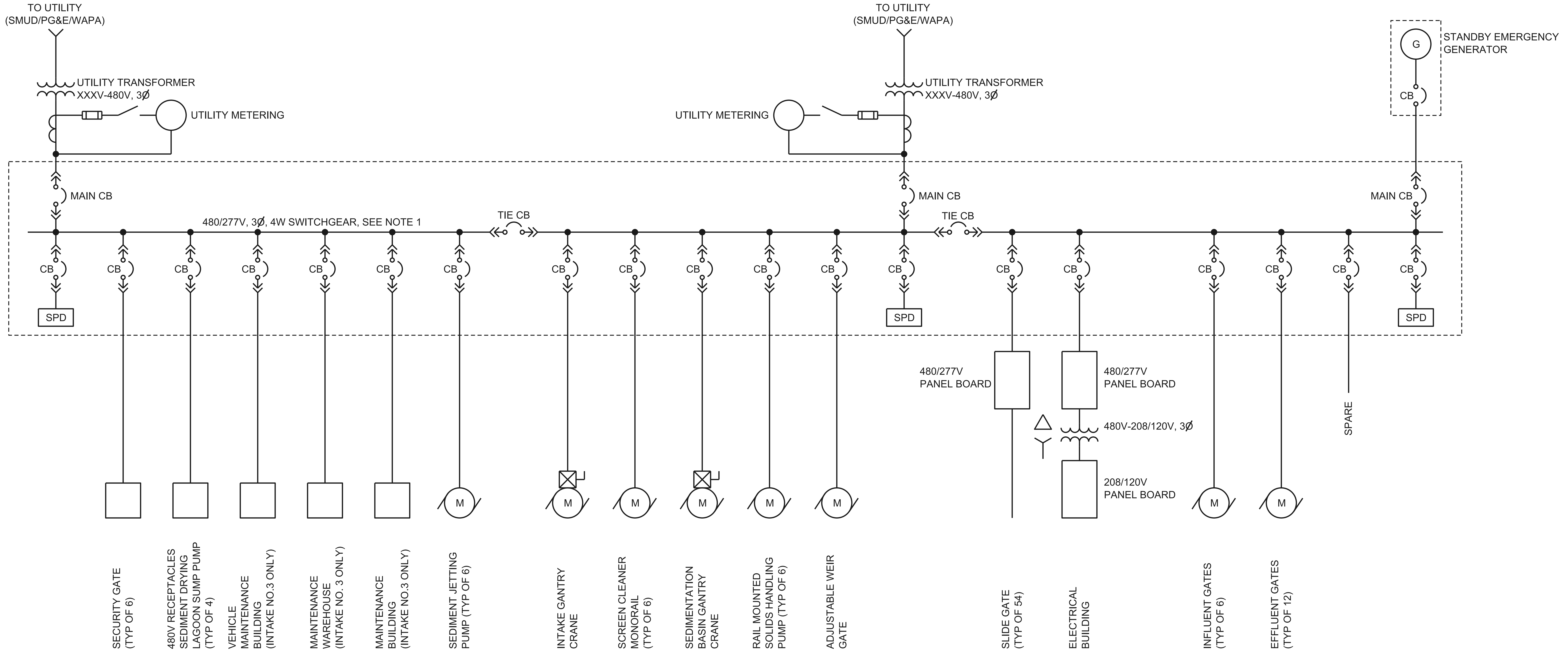
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE PROCESS FLOW DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - I - 6031IT
REV 0
SEQUENCE NO. 31 OF 96



INTAKE NO.3 SINGLE LINE DIAGRAM

NTS

(TYPICAL OF INTAKES NO. 2 AND NO. 5)

NOTE:

1. SWITCHGEAR SHALL BE LOCATED WITHIN ELECTRICAL ROOM.

GENERAL NOTE:

1. ALL ELECTRICAL EQUIPMENT SHALL BE INSTALLED ABOVE THE 200 YEAR FLOODPLAIN.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 ▬ 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTAKE
SINGLE LINE DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - E - 6032IT
REV 0
SEQUENCE NO. 32 OF 96

FINAL DRAFT
DATE: APRIL 1, 2015

NOTE:

1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.

PLAN

1"=1500'

PROFILE

VERTICAL EXAGGERATION = x50

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - PLAN AND PROFILE
INTAKE NO. 2 TO INTERMEDIATE FOREBAY

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

PROJECT NO. 240010
SHEET NO. CCO - C - 1033TN
REV 0
SEQUENCE NO. 33 OF 96

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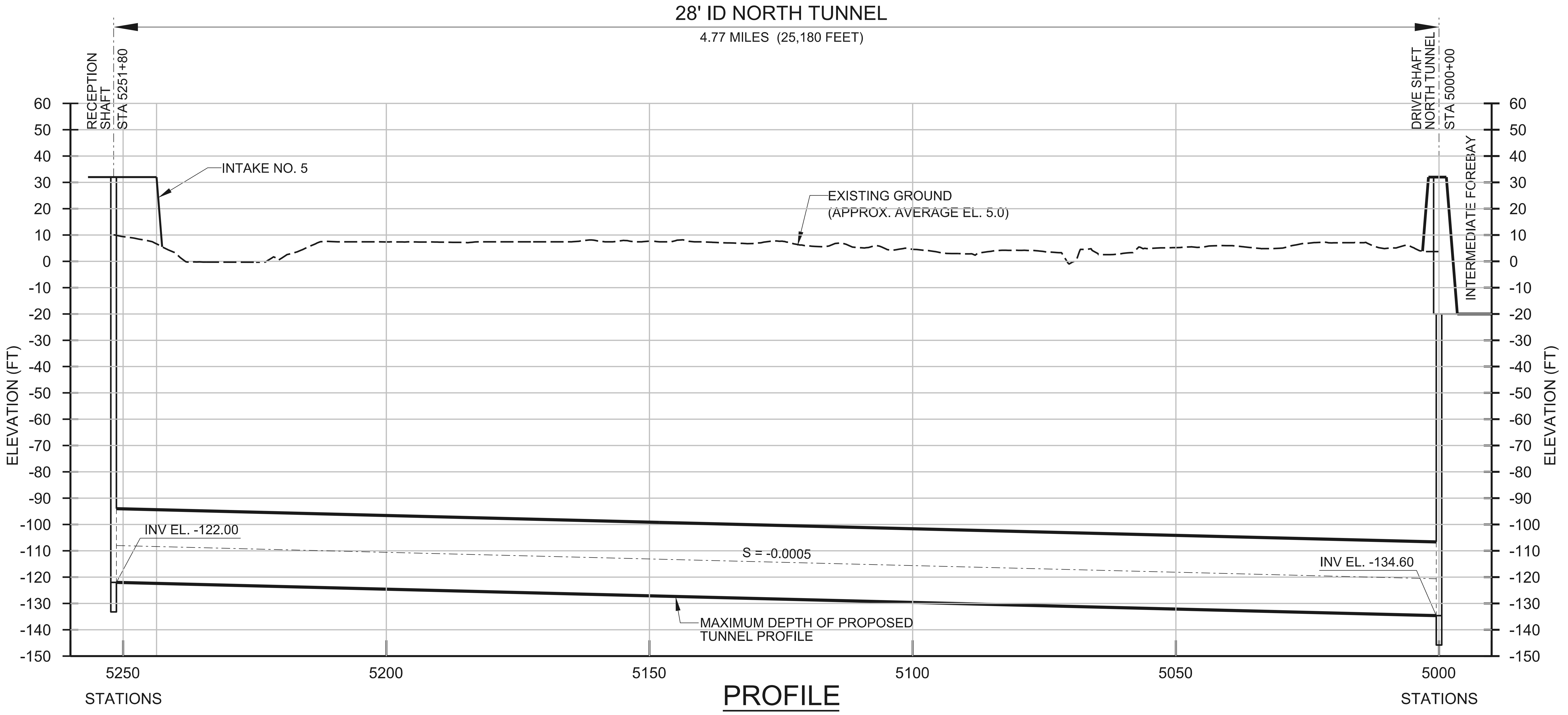
Edited By: Bautista,Jav
Printed By: Bautista,Jav

PLAN

1"=1500'

NOTE:

1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.



PROFILE

VERTICAL EXAGGERATION = x50

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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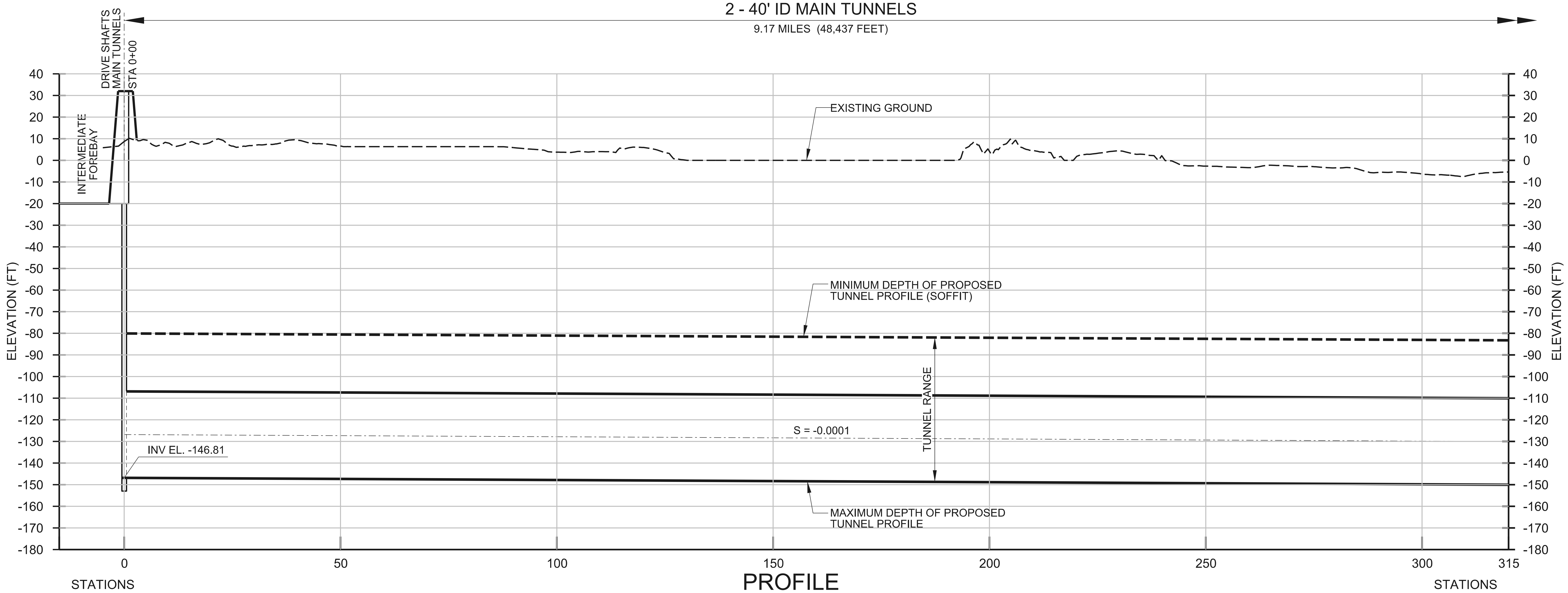
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - PLAN AND PROFILE
INTAKE NO. 5 TO INTERMEDIATE FOREBAY

PROJECT NO. 240010
SHEET NO. CCO - C - 1034TN
REV 0
SEQUENCE NO. 34 OF 96

NOTE:
1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.

PLAN
1"=1500'



PROFILE
VERTICAL EXAGGERATION = x50

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

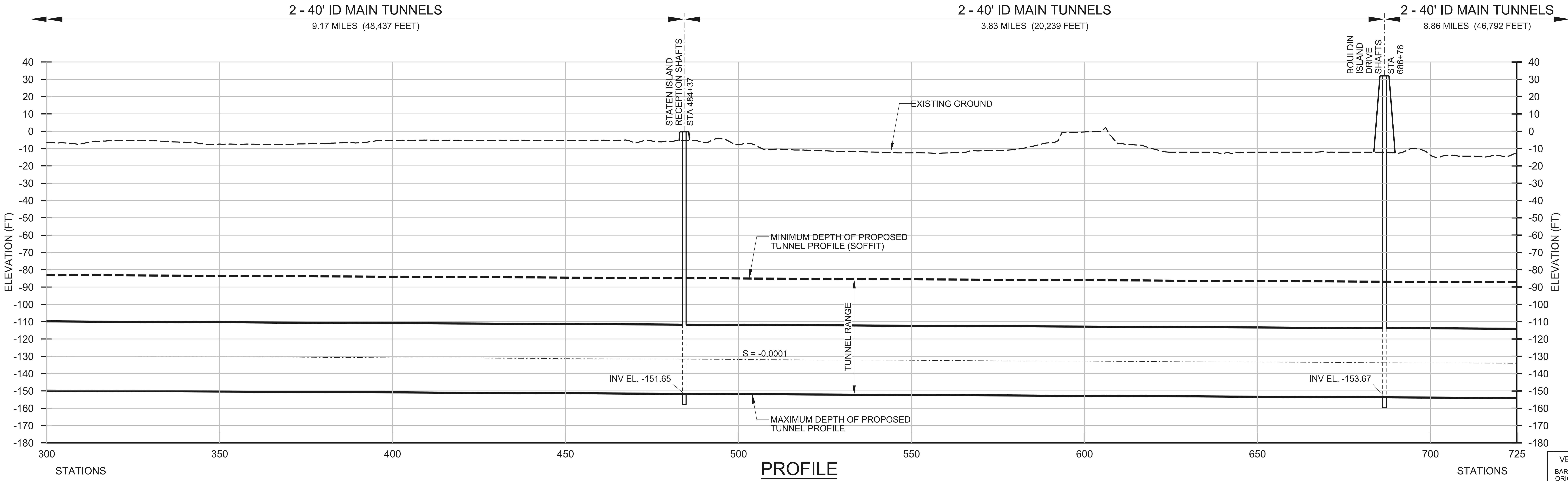
MAIN TUNNELS
PLAN AND PROFILE - SHEET 1 OF 4

PROJECT NO. 240010
SHEET NO. CCO - C - 1035TN
REV 0
SEQUENCE NO. 35 OF 96

FINAL DRAFT
DATE: APRIL 1, 2015

NOTE:
1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.

PLAN
1"=1500'



PROFILE
VERTICAL EXAGGERATION = x50

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS
PLAN AND PROFILE - SHEET 2 OF 4

PROJECT NO. 240010
SHEET NO. CCO - C - 1036TN
REV 0
SEQUENCE NO. 36 OF 96

FINAL DRAFT
DATE: APRIL 1, 2015

NOTE:
1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.

PLAN
1"=1500'

PROFILE
VERTICAL EXAGGERATION = x50

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS
PLAN AND PROFILE - SHEET 3 OF 4

PROJECT NO. 240010
SHEET NO. CCO - C - 1037TN
REV 0
SEQUENCE NO. 37 OF 96

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Edited By: Bautista,Jav

Printed By: Bautista,Jav

PLAN
1"=1500'

FINAL DRAFT
DATE: APRIL 1, 2015

NOTE:
1. TOPOGRAPHY DATA IS BASED ON USGS NATIONAL ELEVATION DATASET (NED). DATA IS FOR PRELIMINARY USE ONLY.

PROFILE
VERTICAL EXAGGERATION = x50

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV DATE DESCRIPTION SUB. APPD

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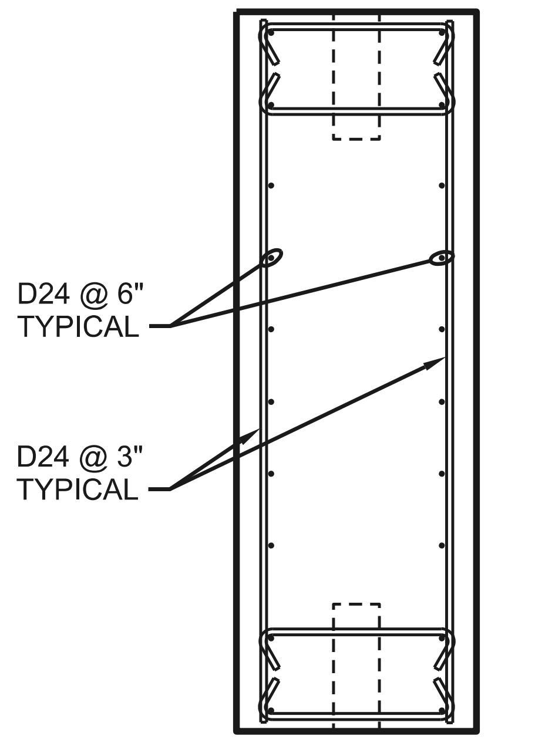
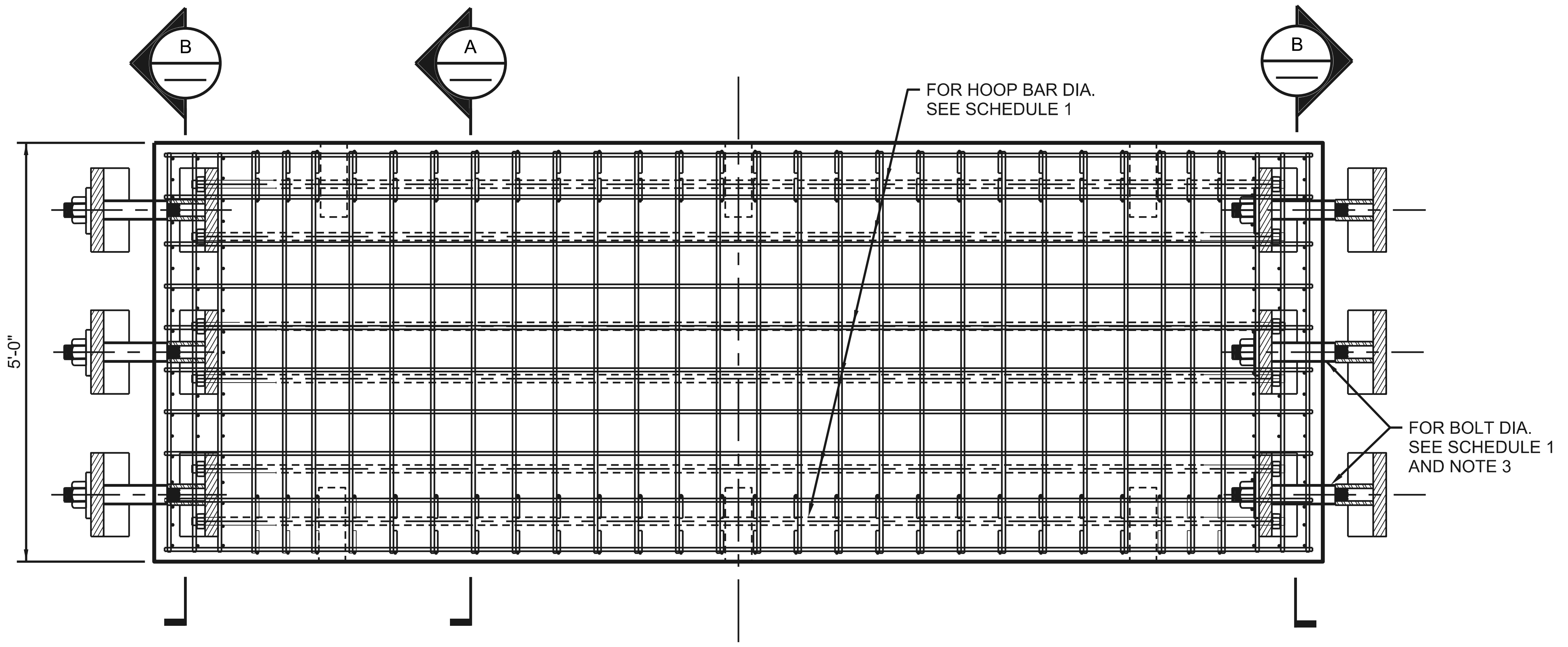
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS
PLAN AND PROFILE - SHEET 4 OF 4

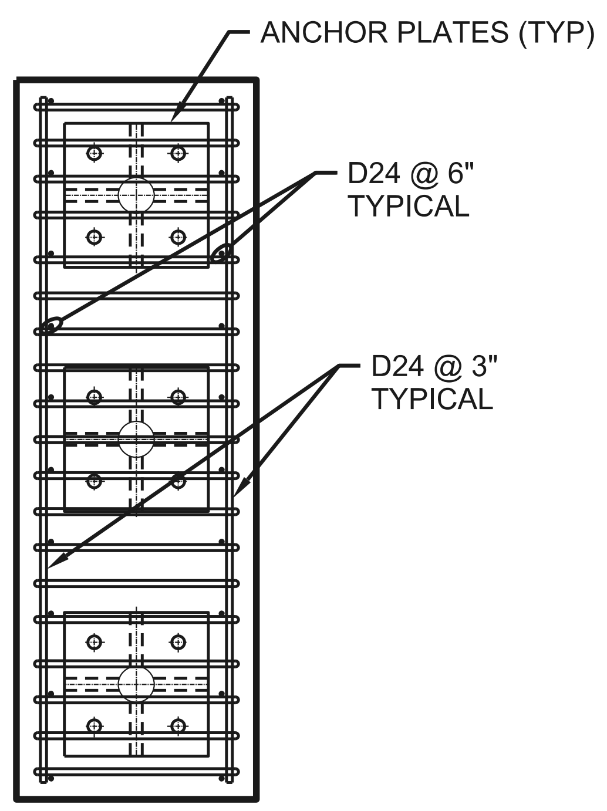
PROJECT NO.
240010
SHEET NO.
CCO - C - 1038TN
REV
0
SEQUENCE NO.
38 OF 96



A SECTION
3/4"=1"

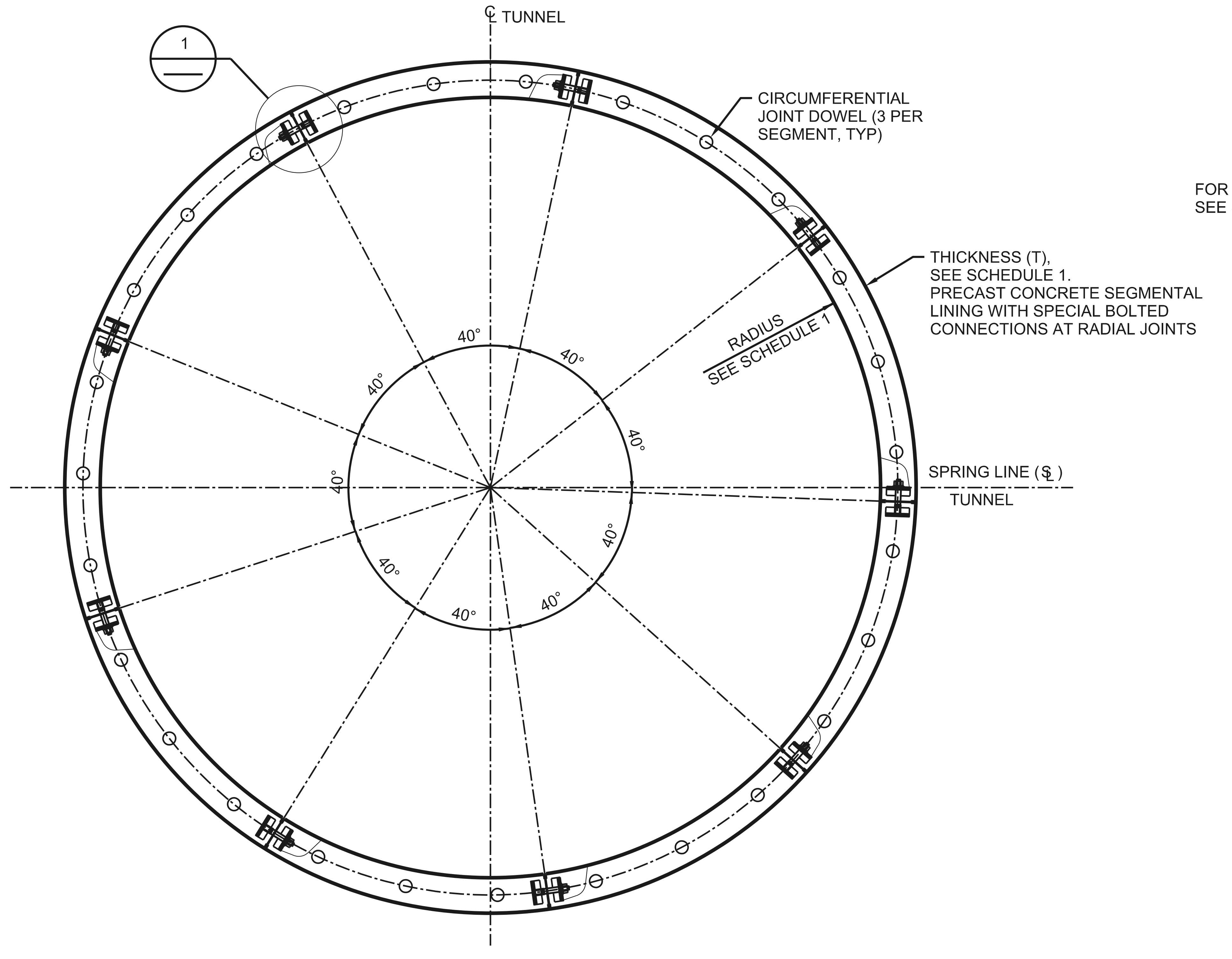


DEVELOPED INSIDE
ELEVATION
3/4"=1"

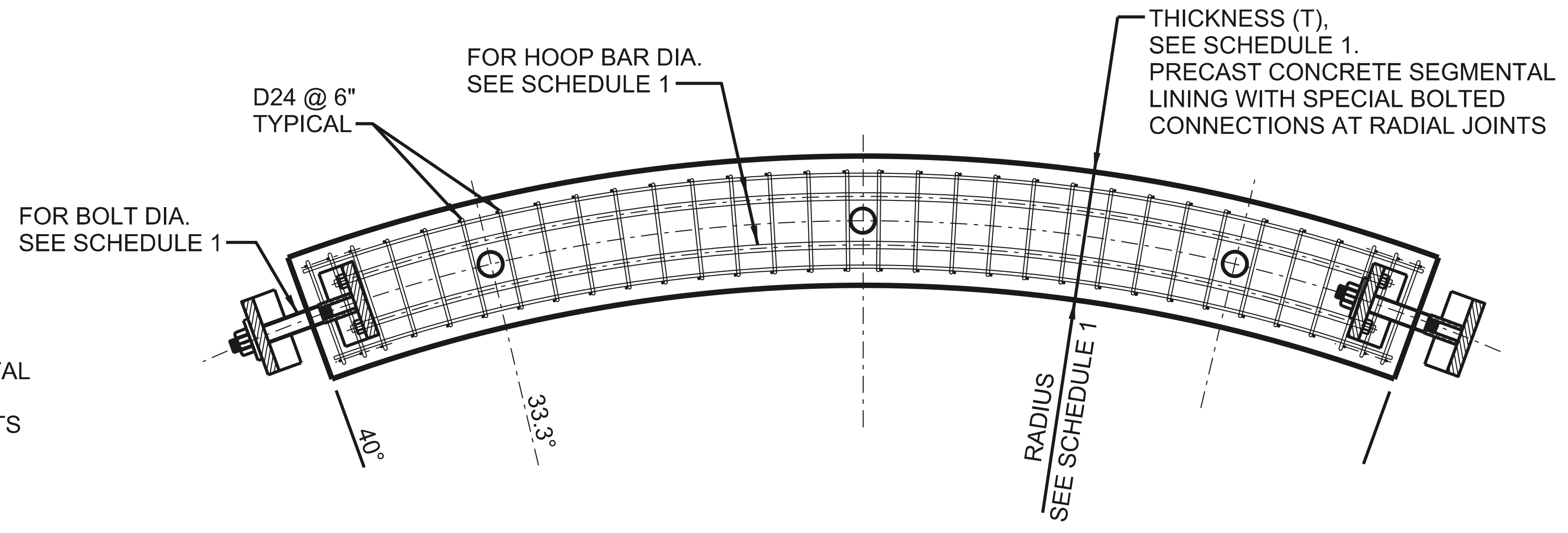


B SECTION
3/4"=1"

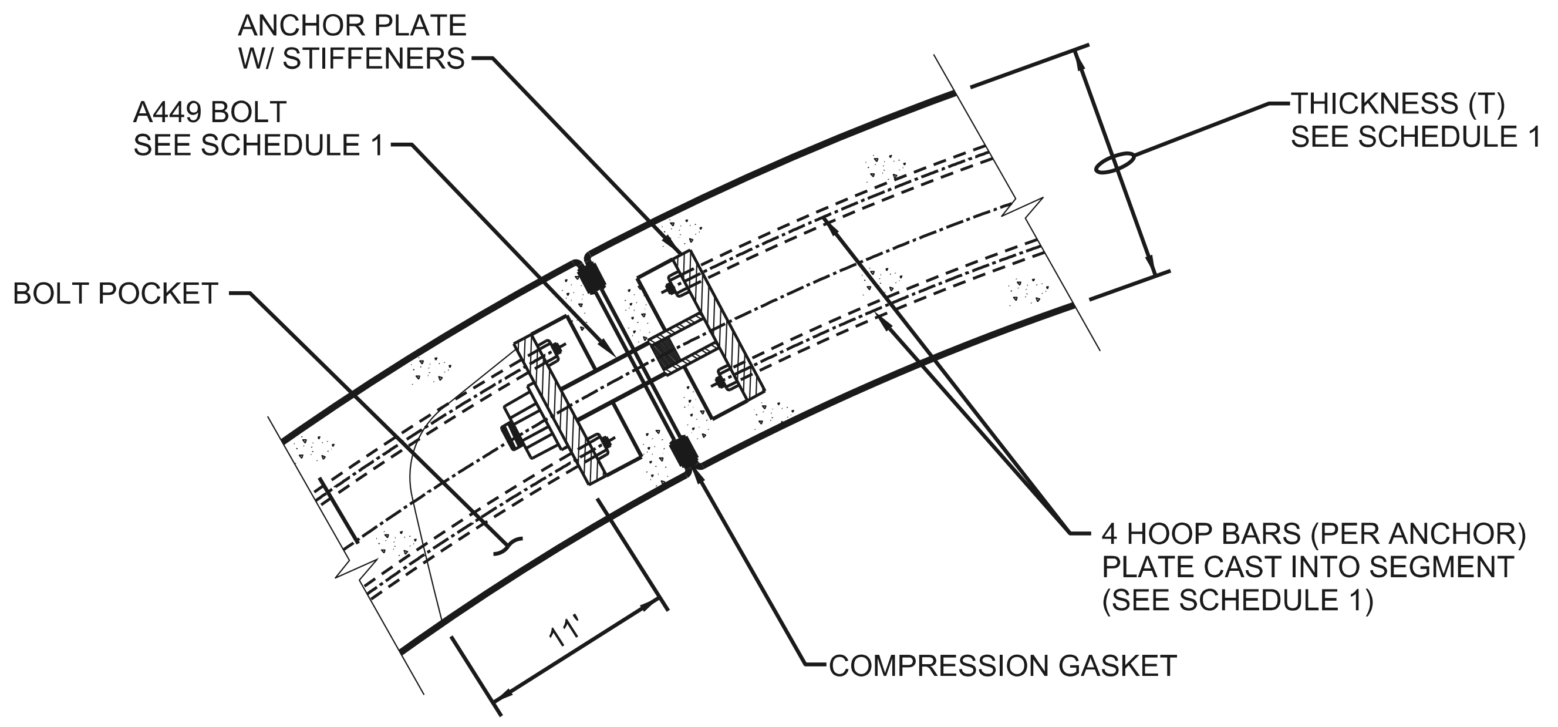
SCHEDULE 1					
REACH	MAX NET INTERNAL HEAD (FT)	INSIDE DIAMETER D (FT)	SEGMENT THICKNESS T (IN)	HOOP BAR SIZE	BOLT DIA (IN)
INTAKES 2, 5	15.5	28	16	#7	7/8



C SECTION
3/16"=1'-0"



PLAN OF SEGMENTAL CONCRETE LINING



1 DETAIL
1/8"=1'-0"

NOTES:

1. HOOP BAR STEEL fy = 60 ksi.
2. D24 REINFORCING STEEL IS FOR fy = 60 ksi. THE BAR SIZE MAY BE REDUCED TO D18 FOR fy = 80ksi.
3. BOLTS SHALL CONFORM TO ASTM A325 or ASTM A449.
4. CONCRETE 28-DAY STRENGTH: 7000 psi.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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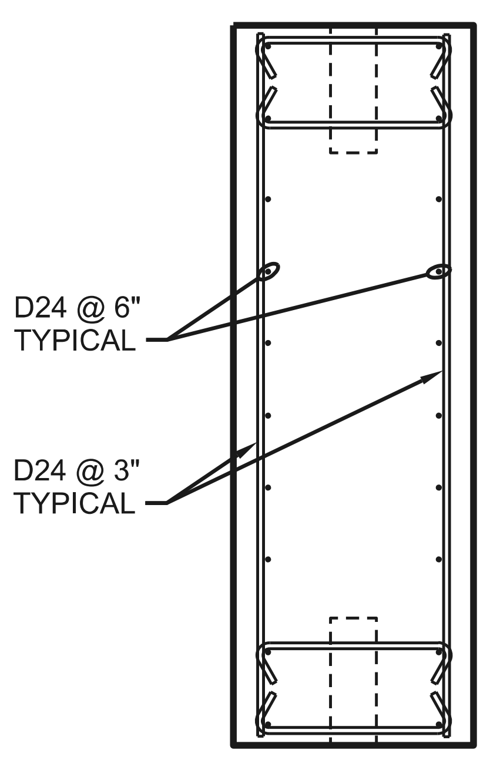
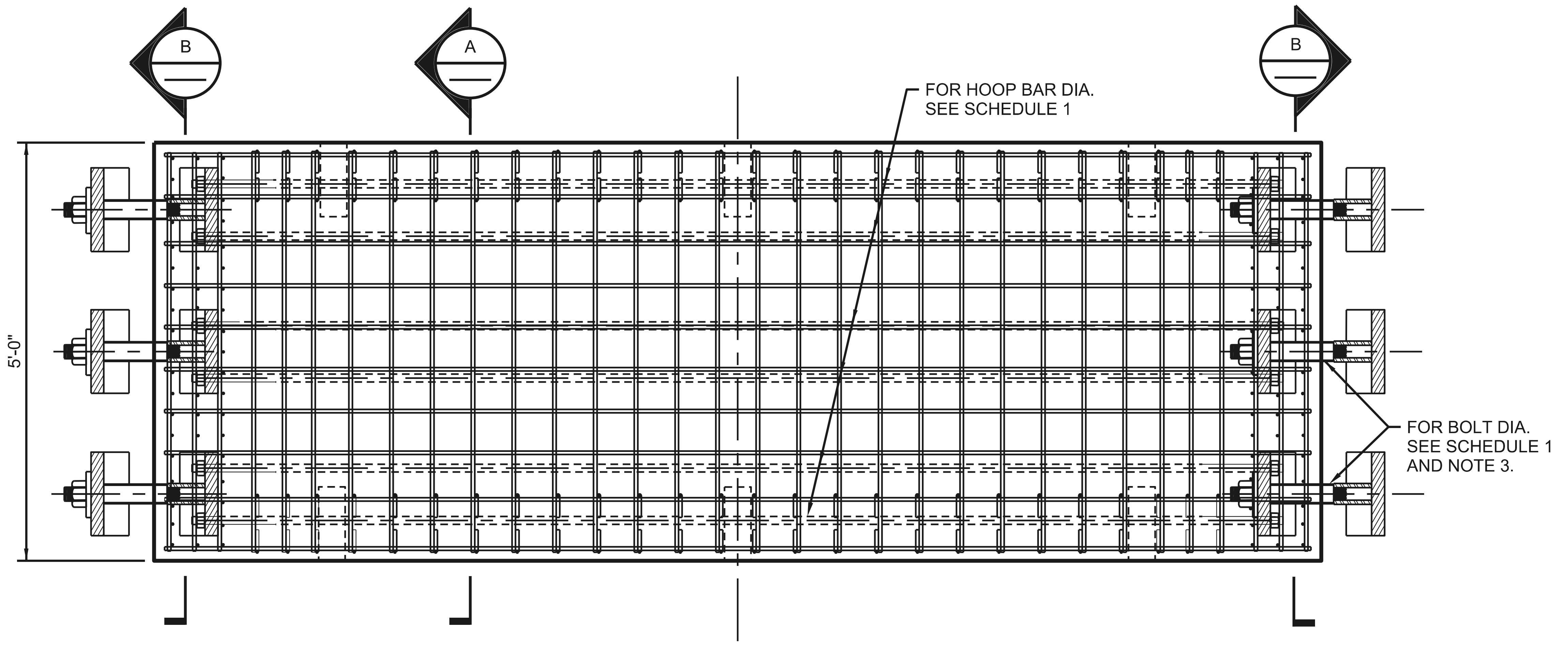
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

28' DIA. NORTH TUNNELS
LINING DETAILS

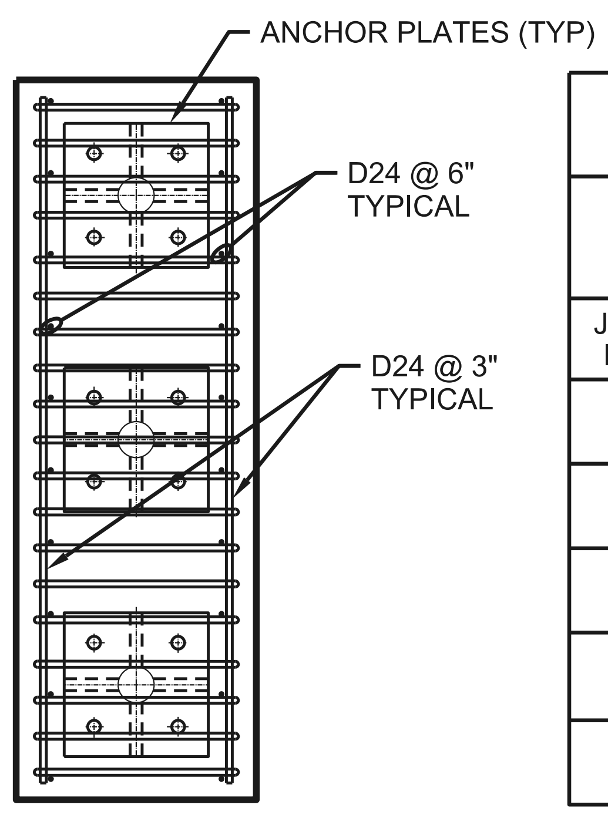
PROJECT NO. 240010
SHEET NO. CCO - S - 4039TN
REV 0
SEQUENCE NO. 39 OF 96



A SECTION
3/4"=1"

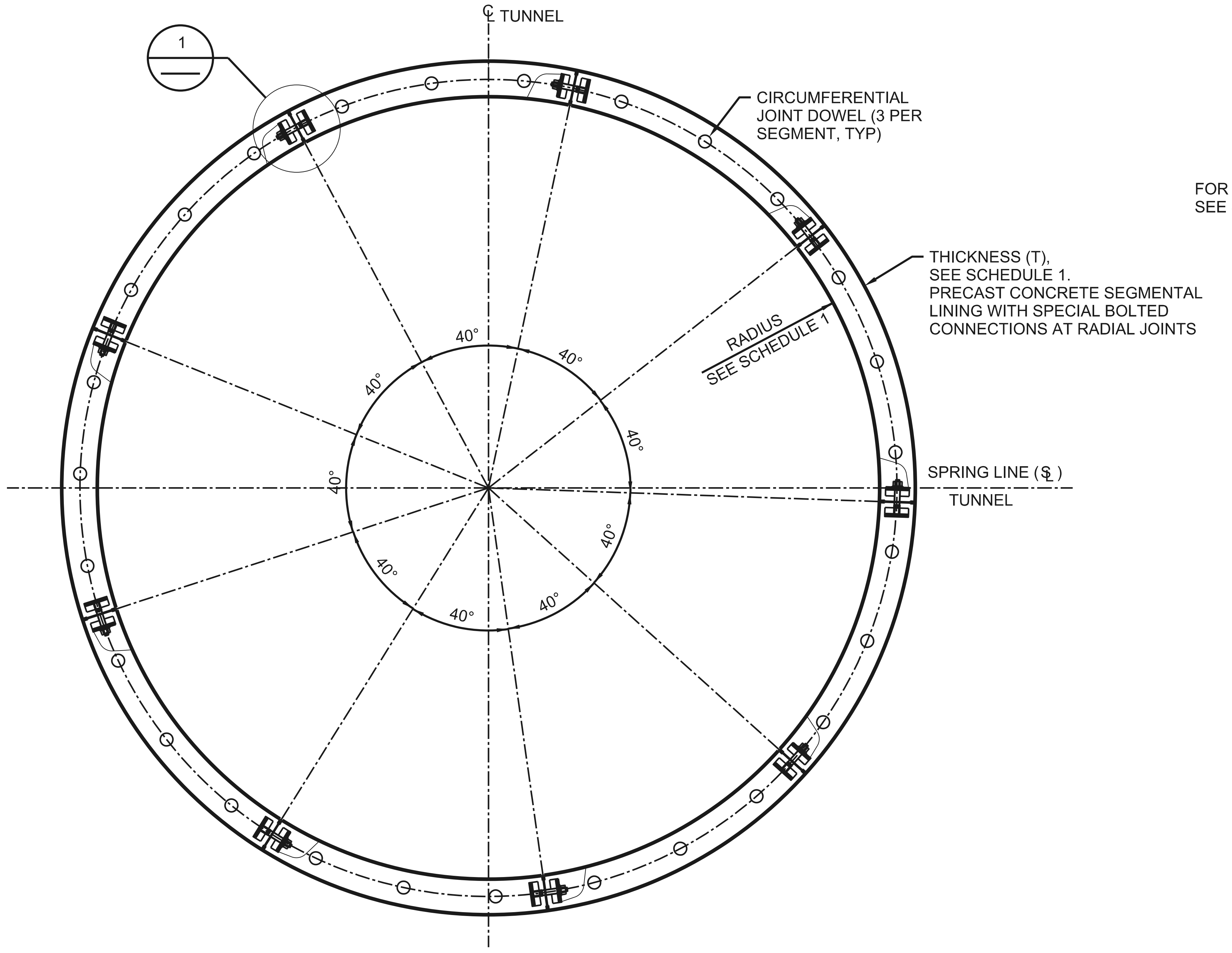


DEVELOPED INSIDE
ELEVATION
3/4"=1"

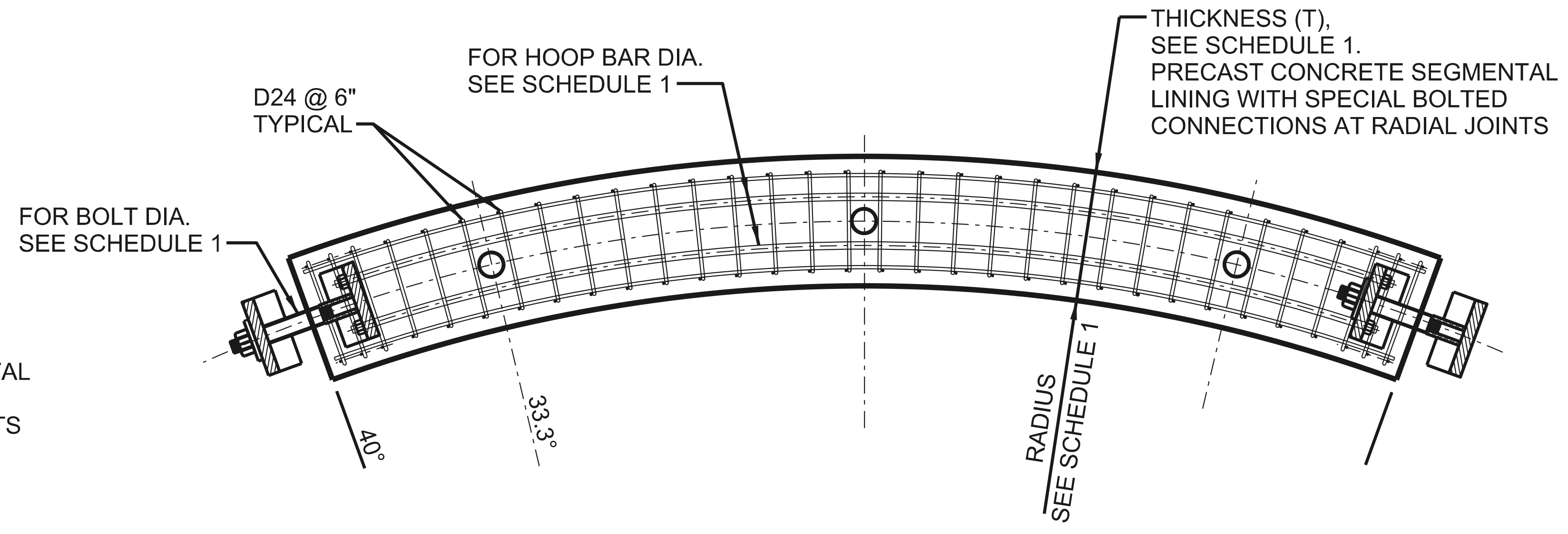


B SECTION
3/4"=1"

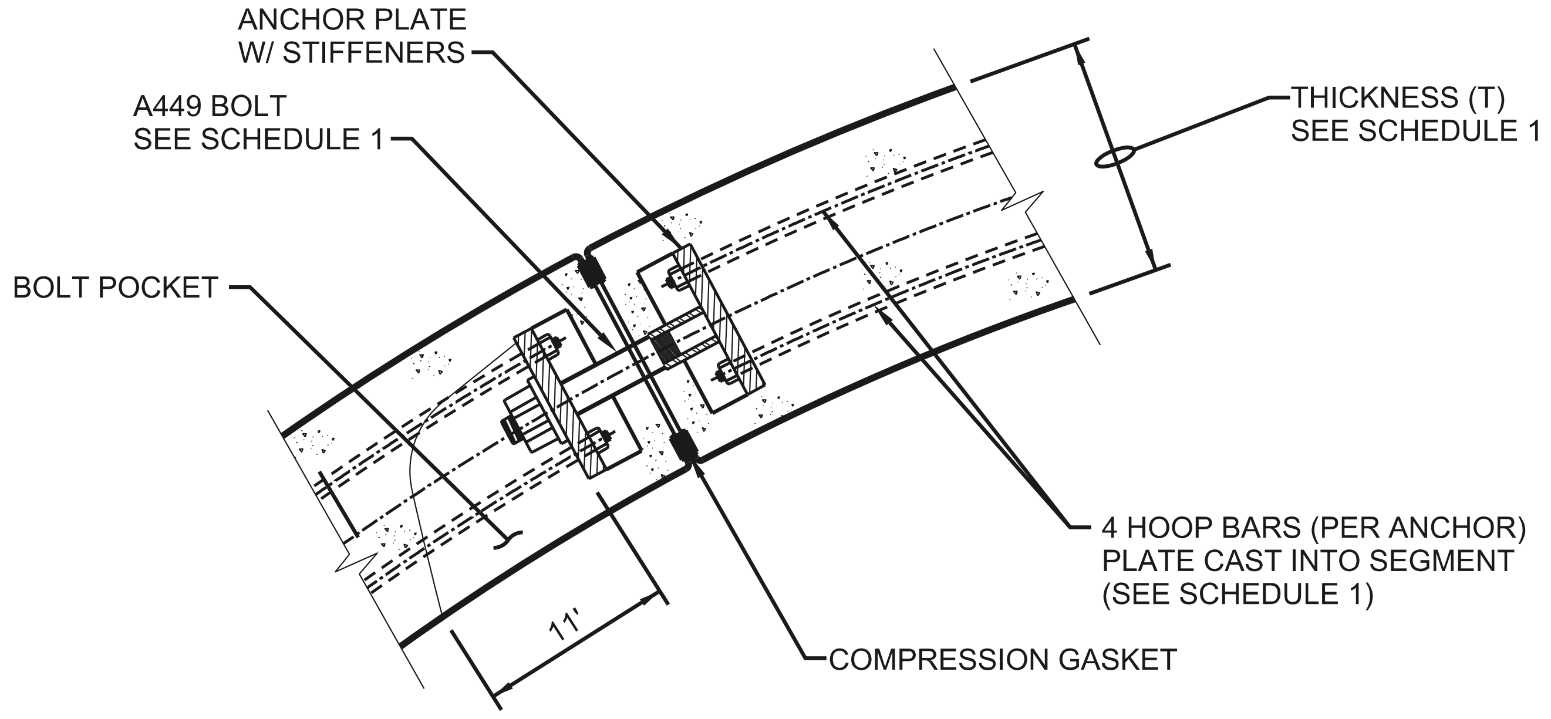
SCHEDULE 1					
REACH	MAX NET INTERNAL HEAD (FT)	INSIDE DIAMETER D (FT)	SEGMENT THICKNESS T (IN)	HOOP BAR SIZE	BOLT DIA (IN)
JUNCTION STRUCTURE TO INTERMEDIATE FOREBAY	20	40	20	#8	1



C SECTION
3/16"=1'-0"



PLAN OF SEGMENTAL CONCRETE LINING



1 DETAIL
1/8"=1'-0"

NOTES:

1. HOOP BAR STEEL fy = 60 ksi.
2. D24 REINFORCING STEEL IS FOR fy = 60 ksi.
   THE BAR SIZE MAY BE REDUCED TO D18
   FOR fy = 80ksi.
3. BOLTS SHALL CONFORM TO ASTM A325 or ASTM A449.
4. CONCRETE 28-DAY STRENGTH: 7000 psi.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

40' DIA. NORTH TUNNEL
LINING DETAILS

PROJECT NO. 240010
SHEET NO. CCO - S - 4040TN
REV 0
SEQUENCE NO. 40 OF 96

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Edited By: Bautista,Jav
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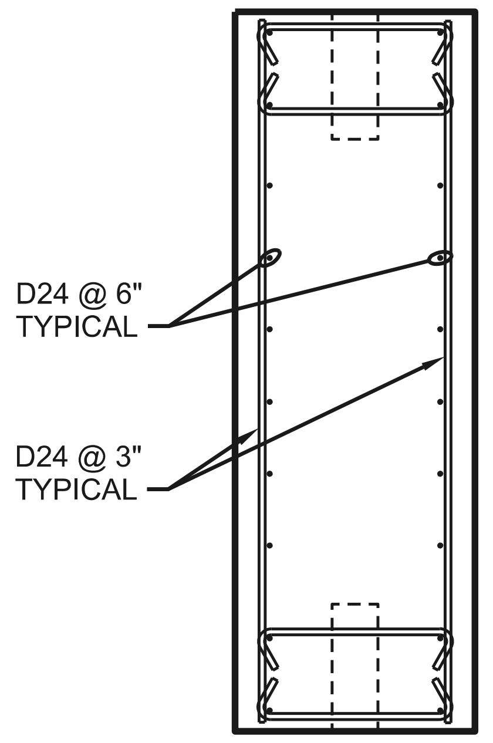
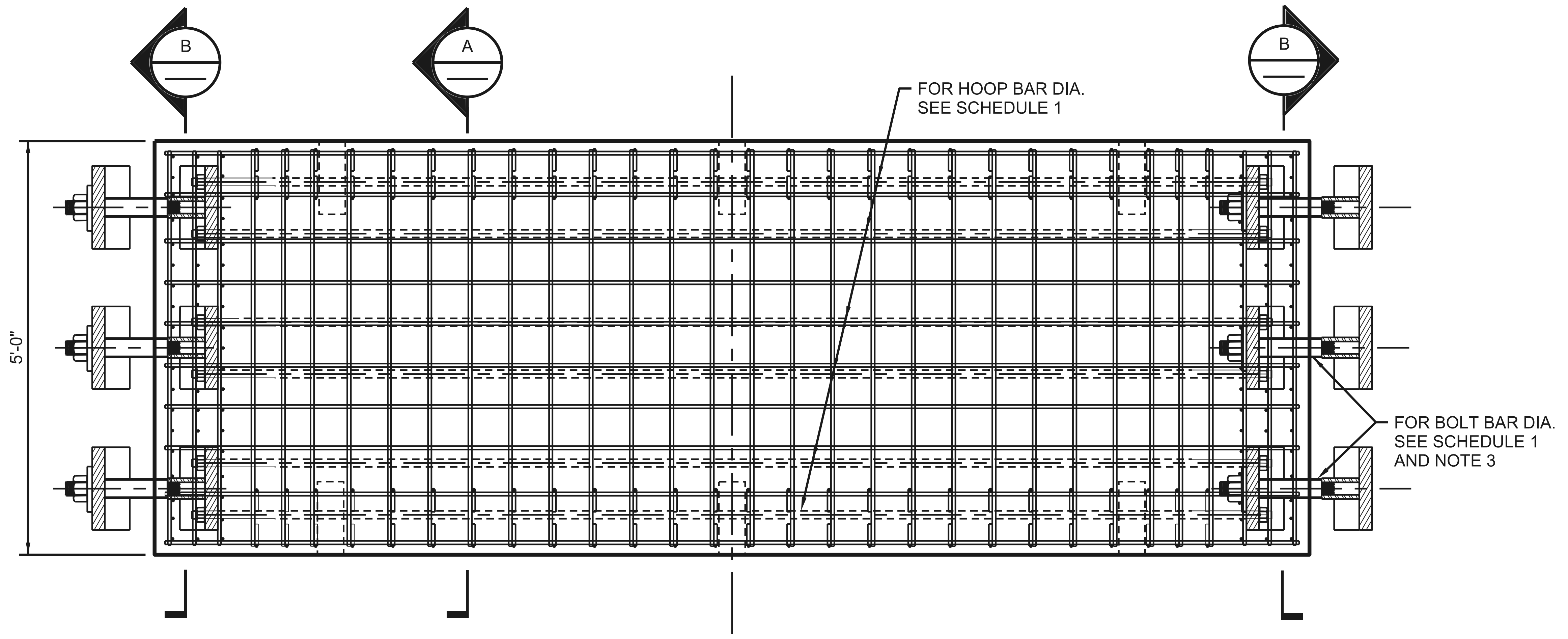
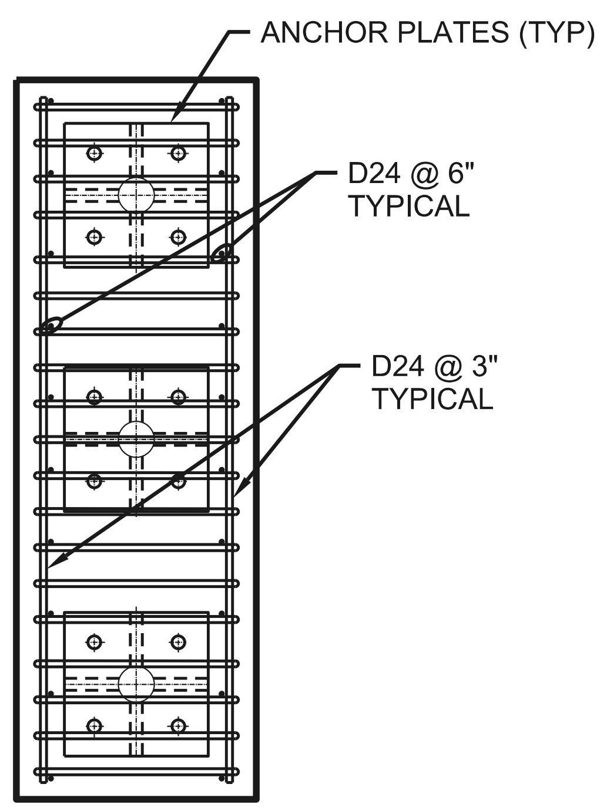


A SECTION
3/4"=1"

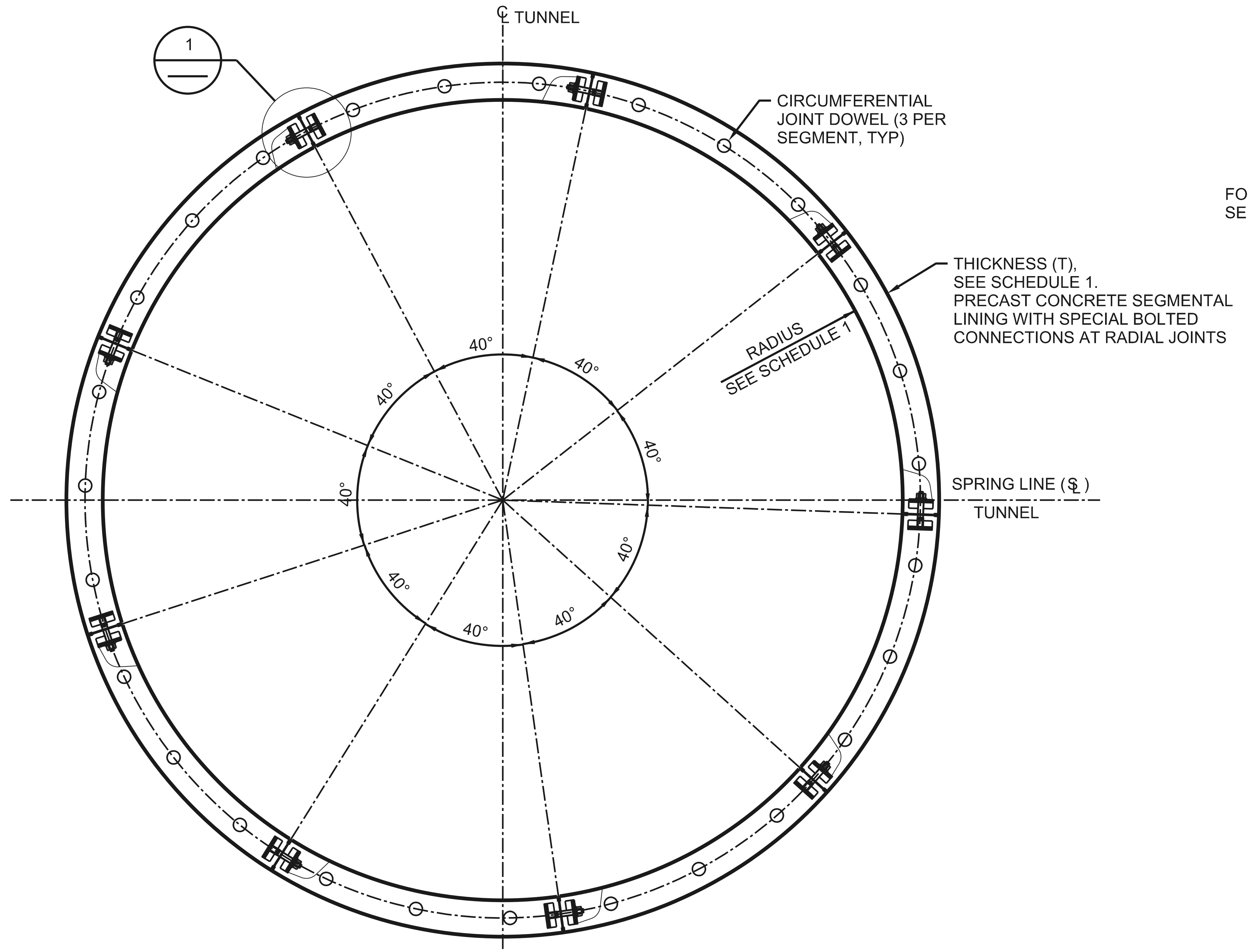


DEVELOPED INSIDE ELEVATION
3/4"=1"

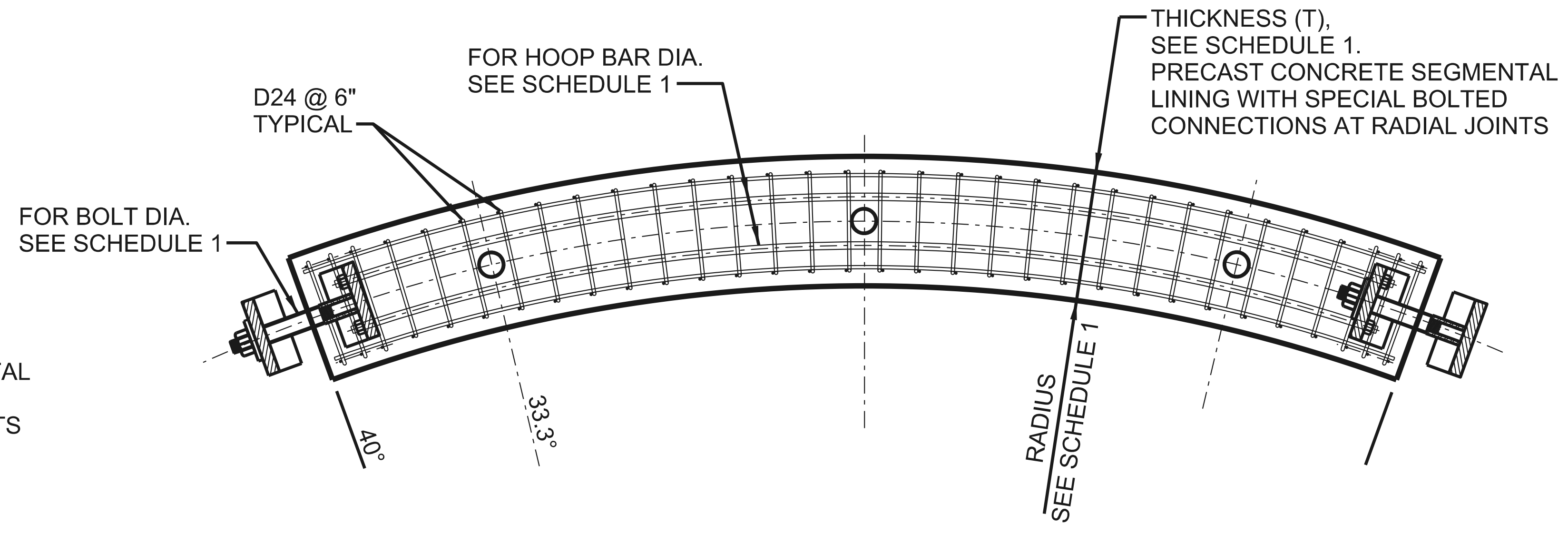


B SECTION
3/4"=1"

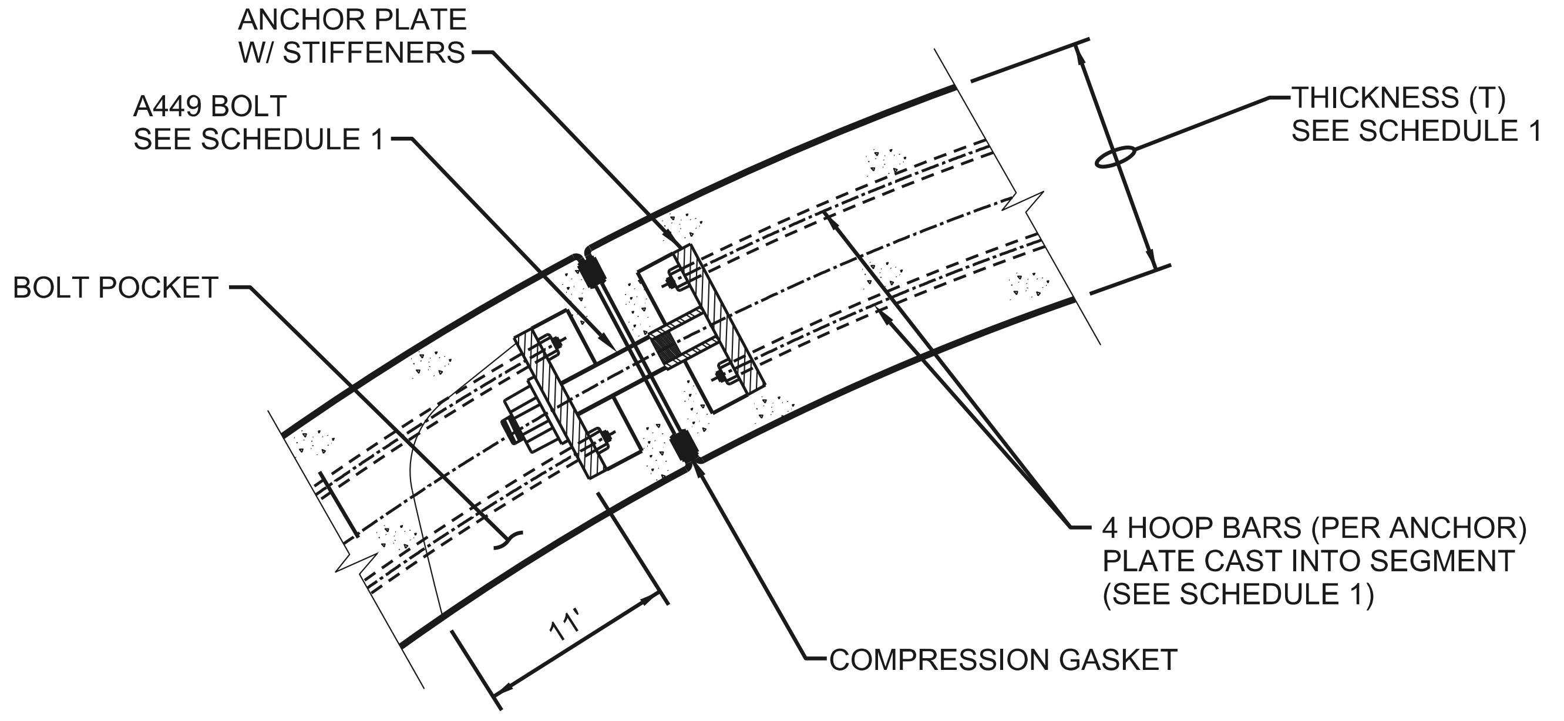
SCHEDULE 1					
REACH	MAX NET INTERNAL HEAD (FT)	INSIDE DIAMETER D (FT)	SEGMENT THICKNESS T (IN)	HOOP BAR SIZE	BOLT DIA (IN)
INTERMEDIATE FOREBAY TO STATEN ISLAND	9	40	20	#5	5/8
STATEN ISLAND TO BOULDIN ISLAND	9	40	20	#5	5/8
BOULDING ISLAND TO BACON ISLAND	9	40	20	#5	5/8
BACON ISLAND TO VICTORIA ISLAND	9	40	20	#5	5/8
VICTORIA ISLAND TO CLIFTON COURT FOREBAY	9	40	20	#5	5/8



C SECTION
3/16"=1'-0"



PLAN OF SEGMENTAL CONCRETE LINING



1 DETAIL
1/8"=1'-0"

NOTES:

1. HOOP BAR STEEL fy = 60 ksi.
2. D24 REINFORCING STEEL IS FOR fy = 60 ksi. THE BAR SIZE MAY BE REDUCED TO D18 FOR fy = 80ksi.
3. BOLTS SHALL CONFORM TO ASTM A325 or ASTM A449.
4. CONCRETE 28-DAY STRENGTH: 7000 psi.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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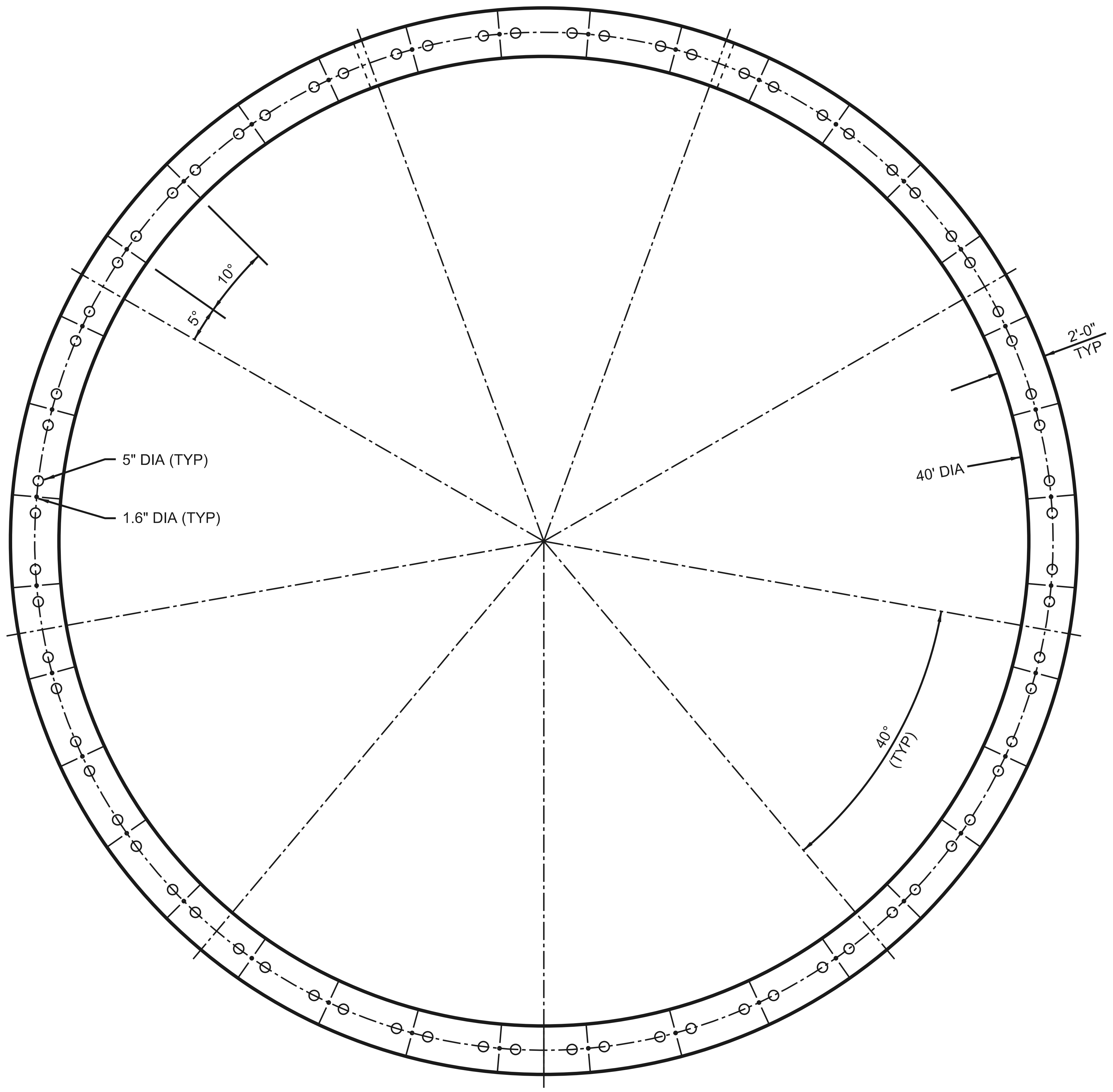
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

40' DIA. MAIN TUNNELS LINING DETAILS

PROJECT NO.
240010
SHEET NO.
CCO - S - 4041TN
REV
0
SEQUENCE NO.
41 OF 96



ELEVATION ON CIRCUMFERENTIAL JOINT
(VIEWED IN DIRECTION OF DRIVE)

DEVELOPED PLAN

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DATE: APRIL 1, 2015

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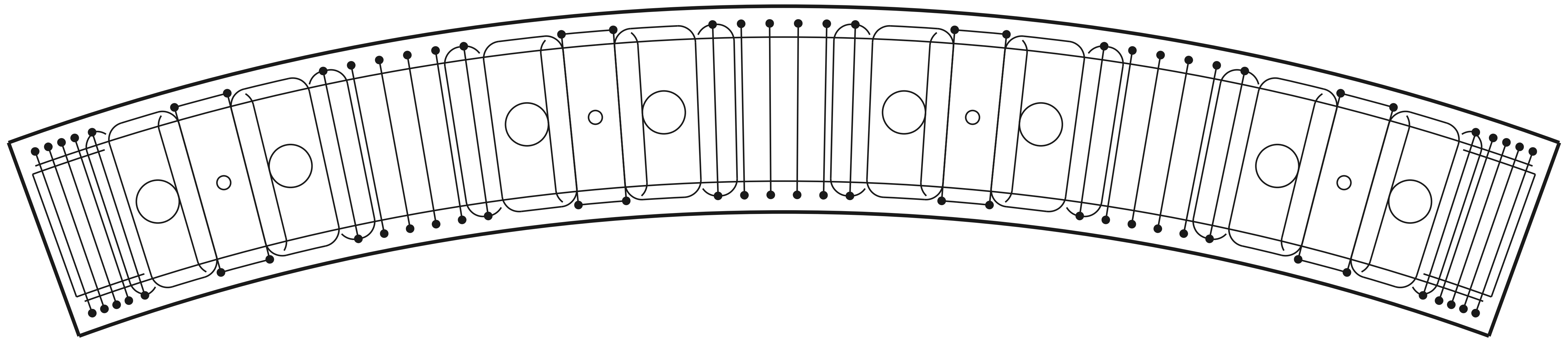
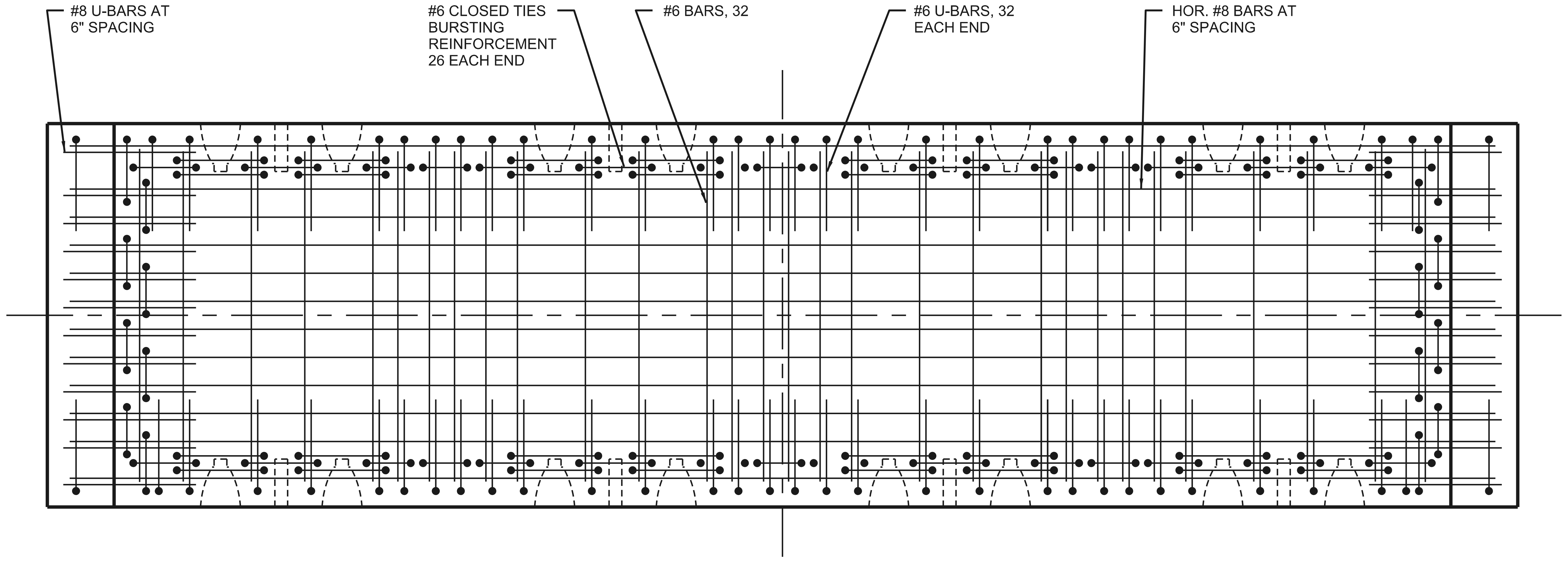
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

40' DIA. MAIN TUNNELS ALTERNATIVE
LINING DETAILS - SHEET 1 OF 2

PROJECT NO. 240010
SHEET NO. CCO - S - 4042TN
REV 0
SEQUENCE NO. 42 OF 96



ELEVATION ON CIRCUMFERENTIAL JOINT



PLAN ON INTRADOS

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

40' DIA. MAIN TUNNELS
ALTERNATIVE LINING DETAILS - SHEET 2 OF 2

PROJECT NO. 240010
SHEET NO. CCO - S - 4043TN
REV 0
SEQUENCE NO. 43 OF 96

PLAN
1"=600'

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - INTAKES NO. 2 AND NO. 3
DRIVE AND RECEPTION SHAFTS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1044TN
REV 0
SEQUENCE NO. 44 OF 96

PLAN
1"=600'

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 ▬ 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - INTAKES NO. 5
RECEPTION SHAFT - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1045TN
REV 0
SEQUENCE NO. 45 OF 96

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Edited By: Bautista,Jav
Printed By: Bautista,Jav

PLAN
1"=600'

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VERIFY SCALE
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS - CONSTRUCTION ACCESS SITE PLAN

PROJECT NO.
240010
SHEET NO.
CCO - C - 1046TN
REV
0
SEQUENCE NO.
46 OF 96

FINAL DRAFT
DATE: APRIL 1, 2015

PLAN
1"=700'

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH TUNNELS/MAIN TUNNELS
INTERMEDIATE FOREBAY - DRIVE SHAFTS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1047TN
REV 0
SEQUENCE NO. 47 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\02 Civil\CCO-C-047-240010

Edited By: Bautista,Jav

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PLAN
1"=500'

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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - STATEN ISLAND
CONSTRUCTION ACCESS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1048TN
REV 0
SEQUENCE NO. 48 OF 96

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PLAN
1"=600'

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - STATEN ISLAND
RECEPTION SHAFTS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1049TN
REV 0
SEQUENCE NO. 49 OF 96

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PLAN
1"=1000'

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - BOULDIN ISLAND DRIVE SHAFTS - SITE PLAN

PROJECT NO.
240010
SHEET NO.
CCO - C - 1050TN
REV
0
SEQUENCE NO.
50 OF 96

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DATE: APRIL 1, 2015

PLAN

1"=500'

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - MANDEVILLE ISLAND
CONSTRUCTION ACCESS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1051TN
REV 0
SEQUENCE NO. 51 OF 96

FINAL DRAFT
DATE: APRIL 1, 2015

PLAN

1"=600'

VERIFY SCALE

BAR IS ONE INCH ON ORIGINAL DRAWING.

0 ▬ 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - BACON ISLAND RECEPTION SHAFTS - SITE PLAN

PROJECT NO. 240010

SHEET NO. CCO - C - 1052TN

REV 0

SEQUENCE NO. 52 OF 96

A B C D E F H

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PLAN
1"=500'

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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Advancing the Bay Delta Conservation Plan
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - VICTORIA ISLAND
CONSTRUCTION ACCESS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1053TN
REV 0
SEQUENCE NO. 53 OF 96

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PLAN
1"=1200'

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MAIN TUNNELS - CLIFTON COURT FOREBAY DRIVE SHAFTS - SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1054TN
REV 0
SEQUENCE NO. 54 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\02 Civil\CCO-C-054-240010

Edited By: Bautista,Jav

Printed By: Bautista,Jav

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY RENDERING

PROJECT NO. 240010
SHEET NO. CCO - G - 9055IF
REV 0
SEQUENCE NO. 55 OF 96

PLAN
1"=250'

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY
SITE PLAN

PROJECT NO.
240010

SHEET NO.
CCO - C - 1056IF

REV
0

SEQUENCE NO.
56 OF 96

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Edited By: Bautista,Jav

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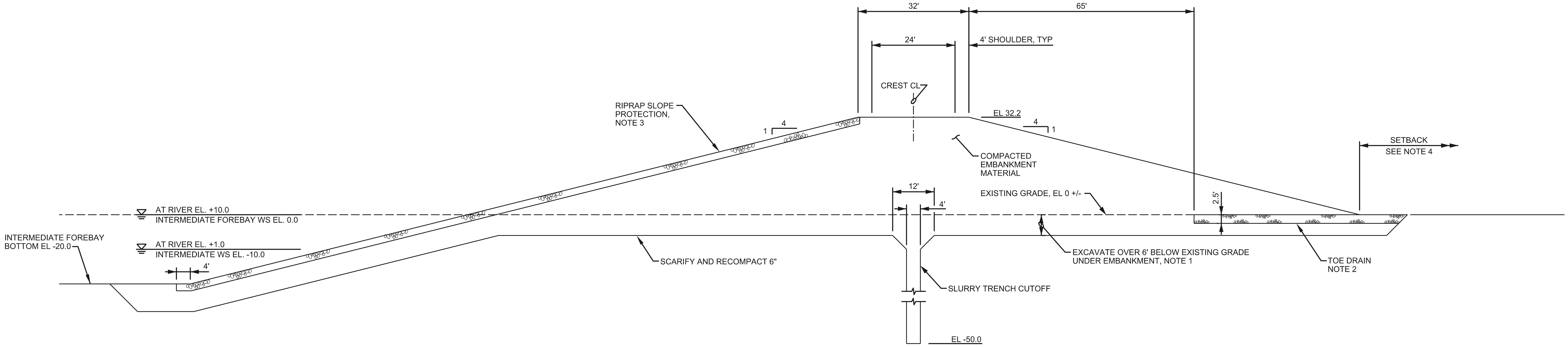
NOTES:

1. EXCAVATION OF UNSUITABLE FOUNDATION MATERIAL WILL VARY. AN AVERAGE OF 6' IS ASSUMED. VERIFY DURING PRELIMINARY ENGINEERING.
2. 2.5' OF PROCESSED DRAIN MATERIAL WITH FILTER LAYERS.
3. PLACE RIPRAP OVER FILTER LAYER.
4. TOE OF EMBANKMENT SHALL BE SET BACK 100 FEET FROM TOE OF EXISTING LEVEE/EMBANKMENT OR PROPERTY LINE, WHICHEVER IS GREATER.



A SECTION
1" = 15'
CCO-C-1056IF

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY
SECTION

PROJECT NO. 240010
SHEET NO. CCO - C - 3057IF
REV 0
SEQUENCE NO. 57 OF 96

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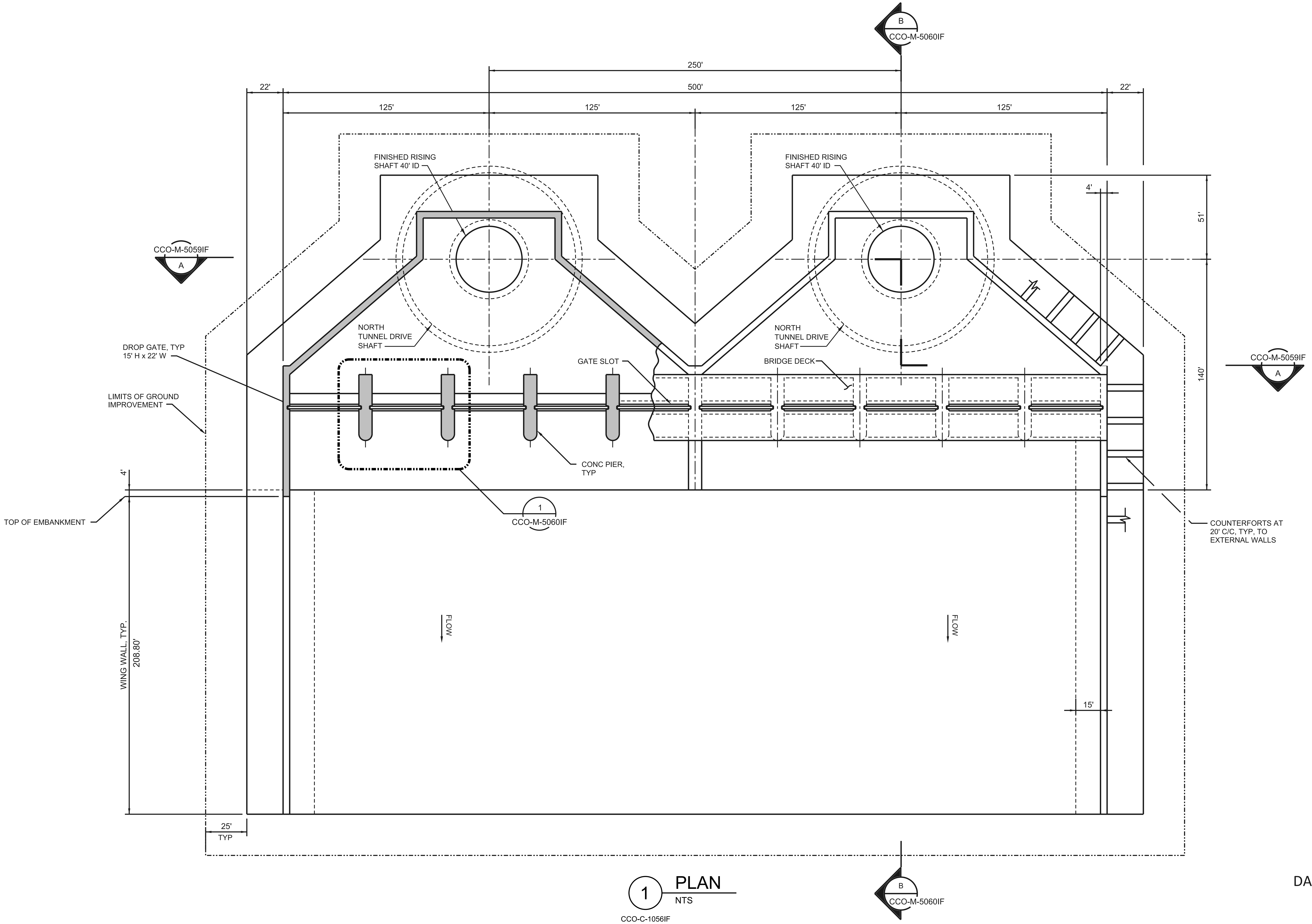

1 **PLAN**
NTS
CCO-C-1056IF

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY INLET STRUCTURE PLAN

PROJECT NO. 240010
SHEET NO. CCO - M - 4058IF
REV 0
SEQUENCE NO. 58 OF 96

NOTE:

1. GROUND IMPROVEMENT TO EL -50.0, SEE PLAN FOR HORIZONTAL LIMITS.

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A SECTION
NTS
CCO-M-4058IF

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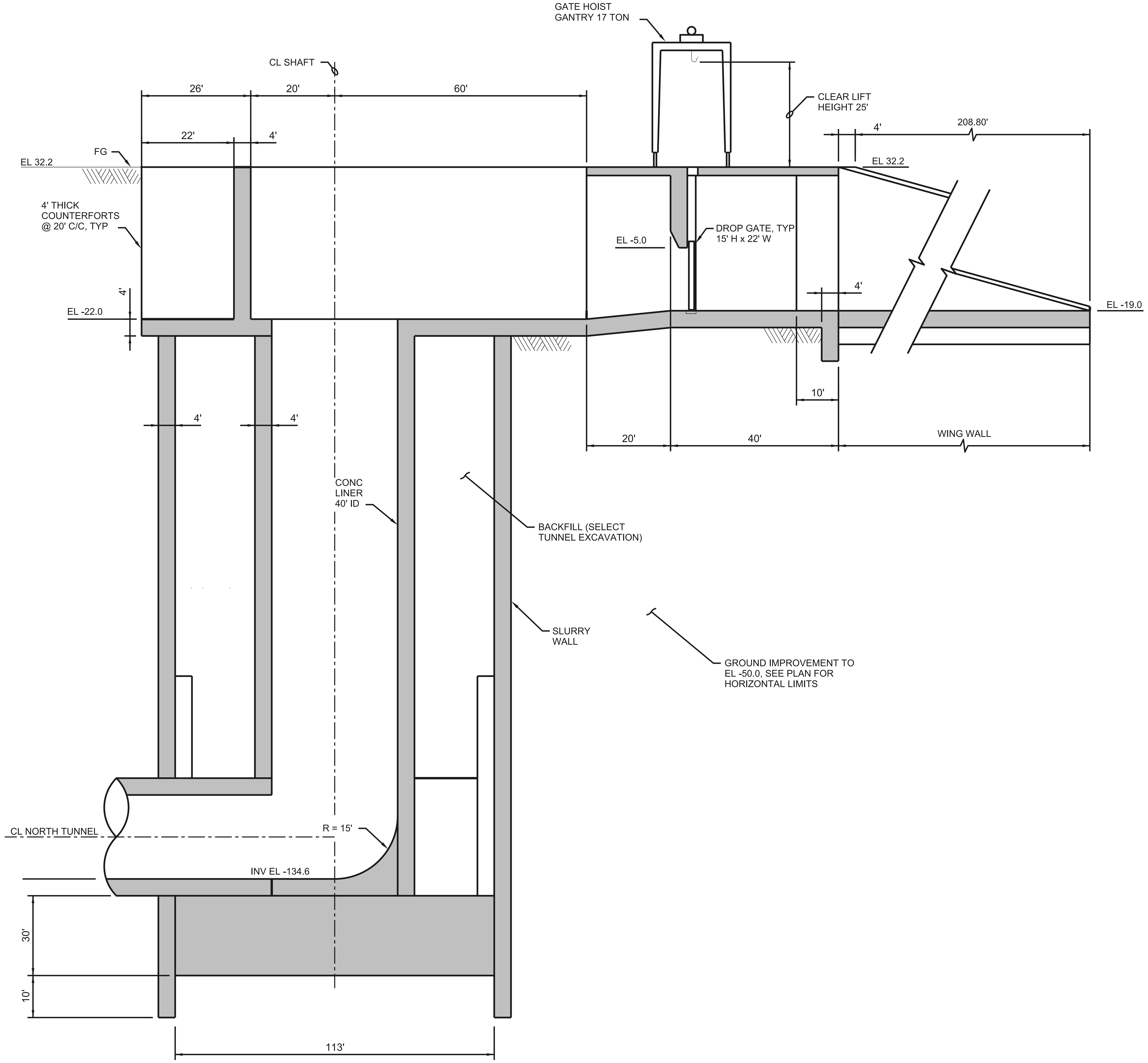
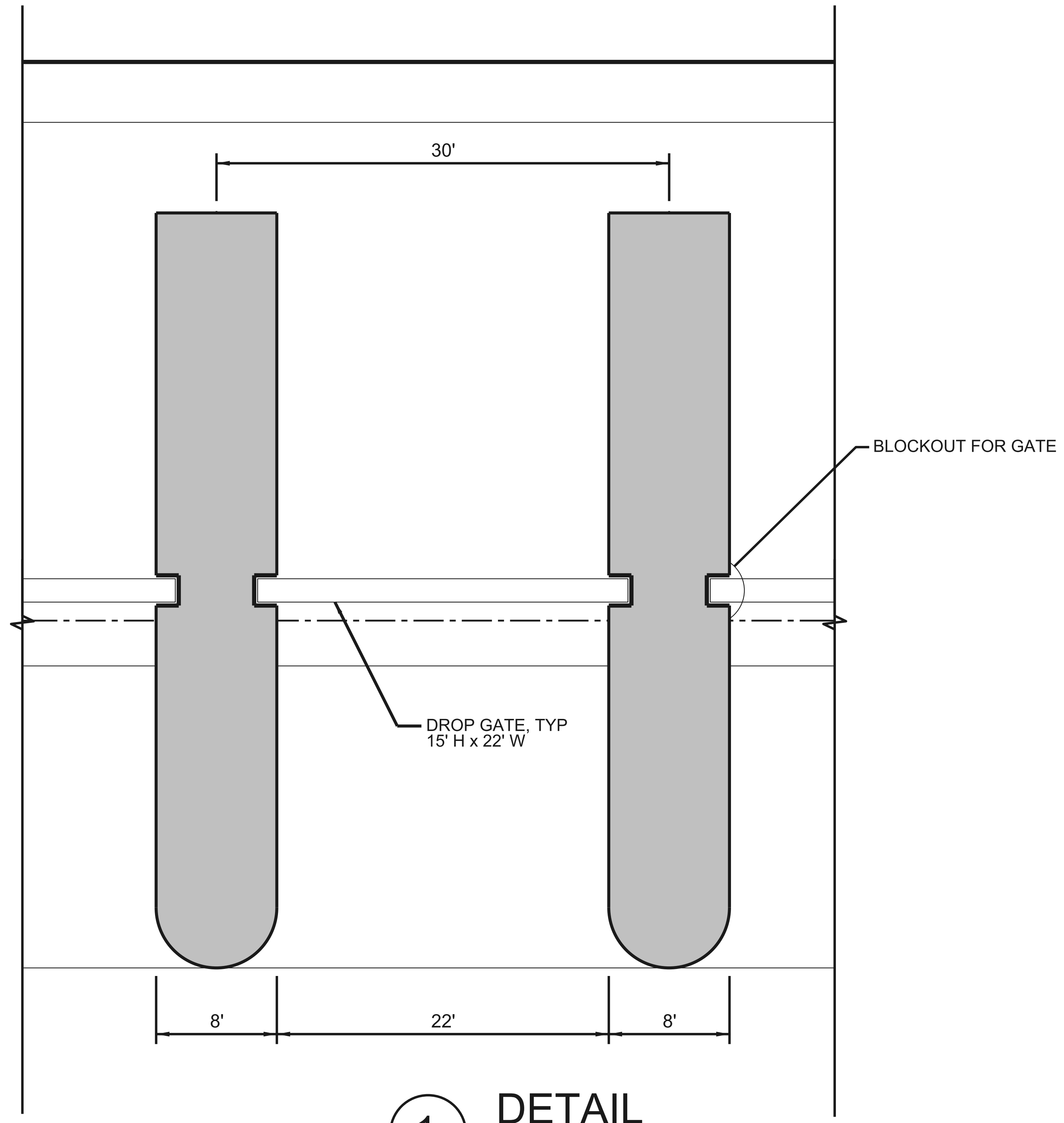
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY INLET STRUCTURE LONGITUDINAL SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 3059IF
REV 0
SEQUENCE NO. 59 OF 96



B **SECTION**
NTS
CCO-M-4058IF



1 **DETAIL**
NTS
CCO-M-4058IF

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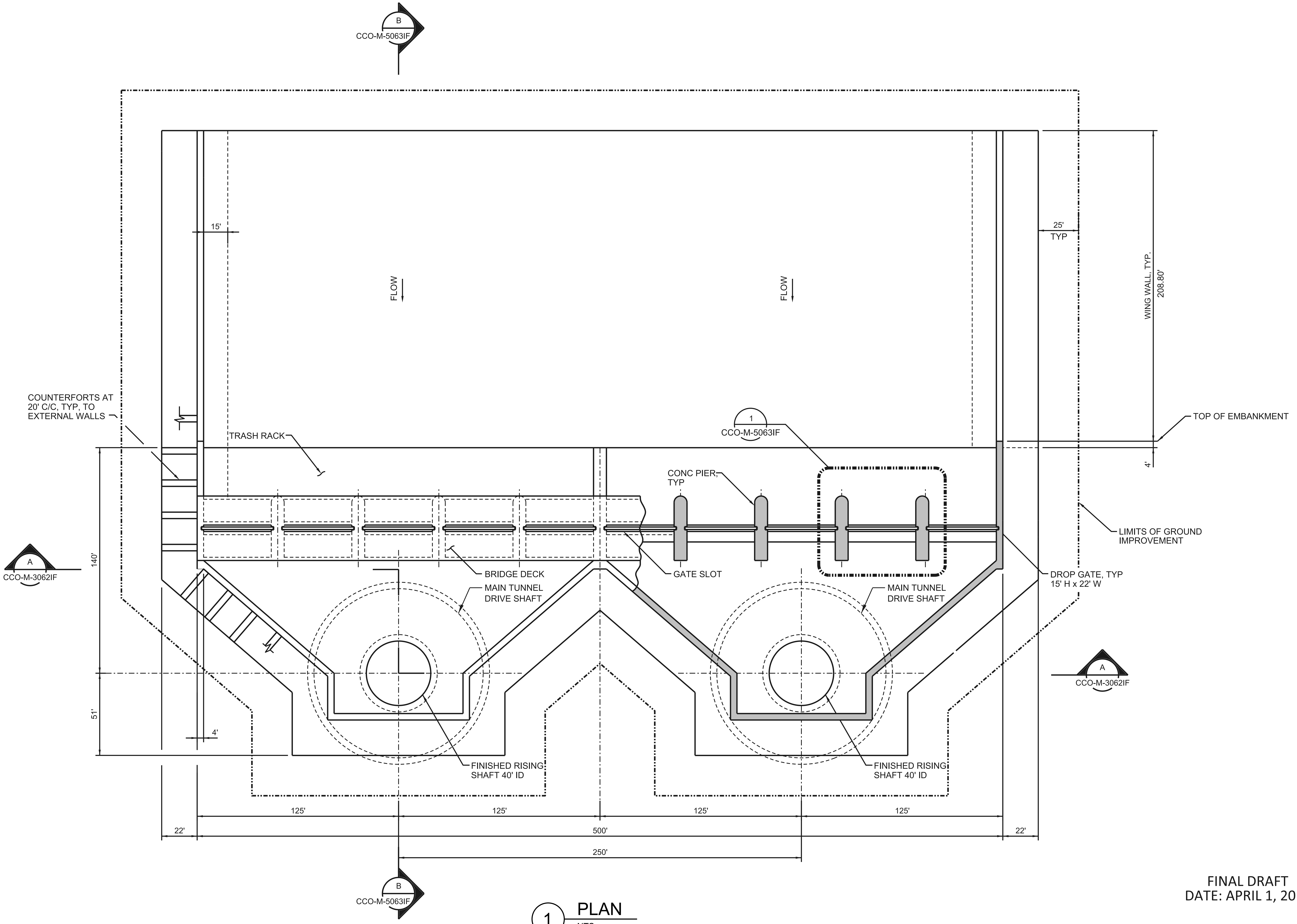
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY INLET STRUCTURE
SECTION AND DETAILS

PROJECT NO.
240010
SHEET NO.
CCO - M - 5060IF
REV
0
SEQUENCE NO.
60 OF 96



1 PLAN
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CCO-C-1056IF

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY
OUTLET STRUCTURE PLAN

PROJECT NO. 240010
SHEET NO. CCO - M - 4061IF
REV 0
SEQUENCE NO. 61 OF 96

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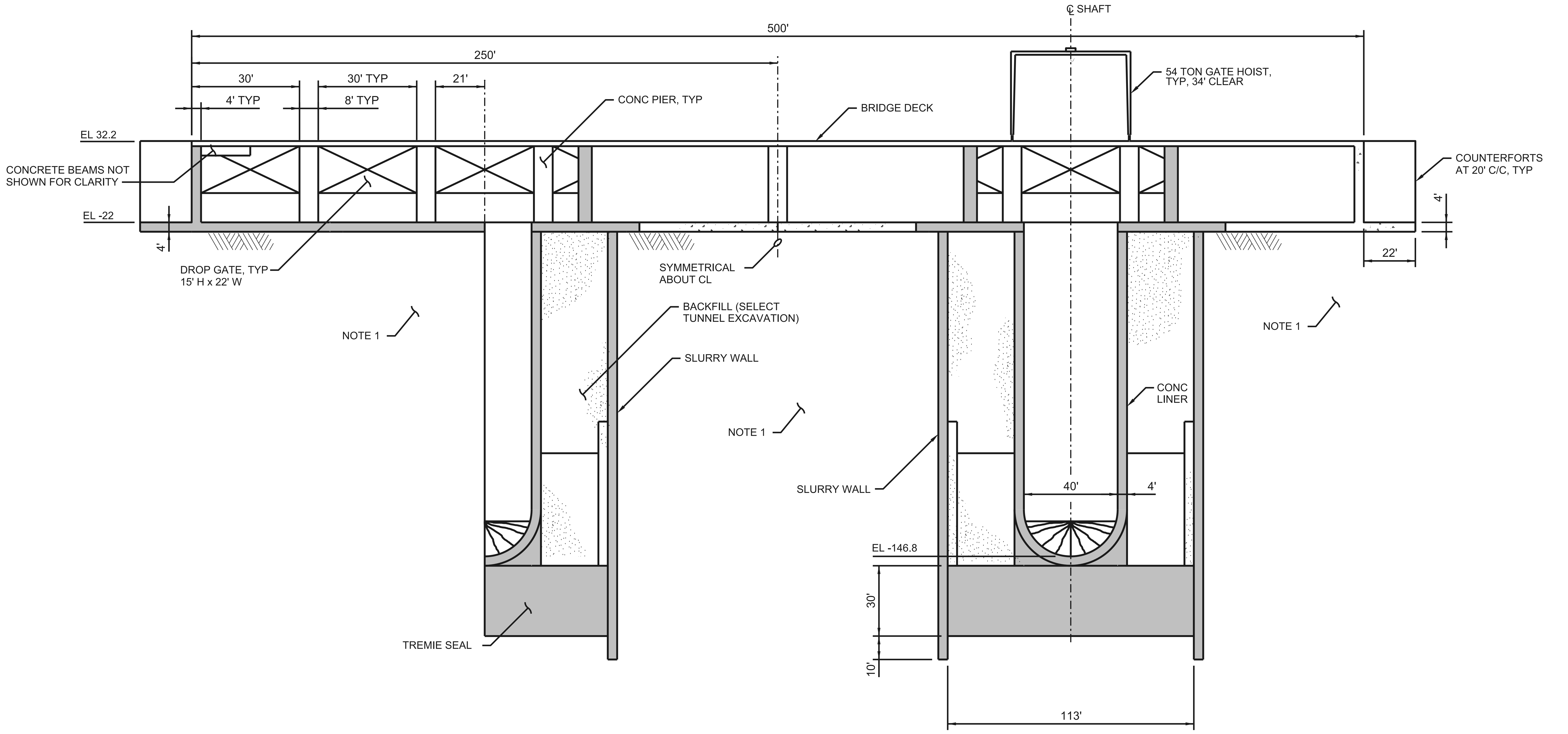

A SECTION
1/32"=1'-0"
CCO-M-4061IF

NOTE:

1. GROUND IMPROVEMENT TO EL -50.0. SEE PLAN FOR HORIZONTAL LIMITS.

FINAL DRAFT
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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY OUTLET STRUCTURE LONGITUDINAL SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 3062IF
REV 0
SEQUENCE NO. 62 OF 96

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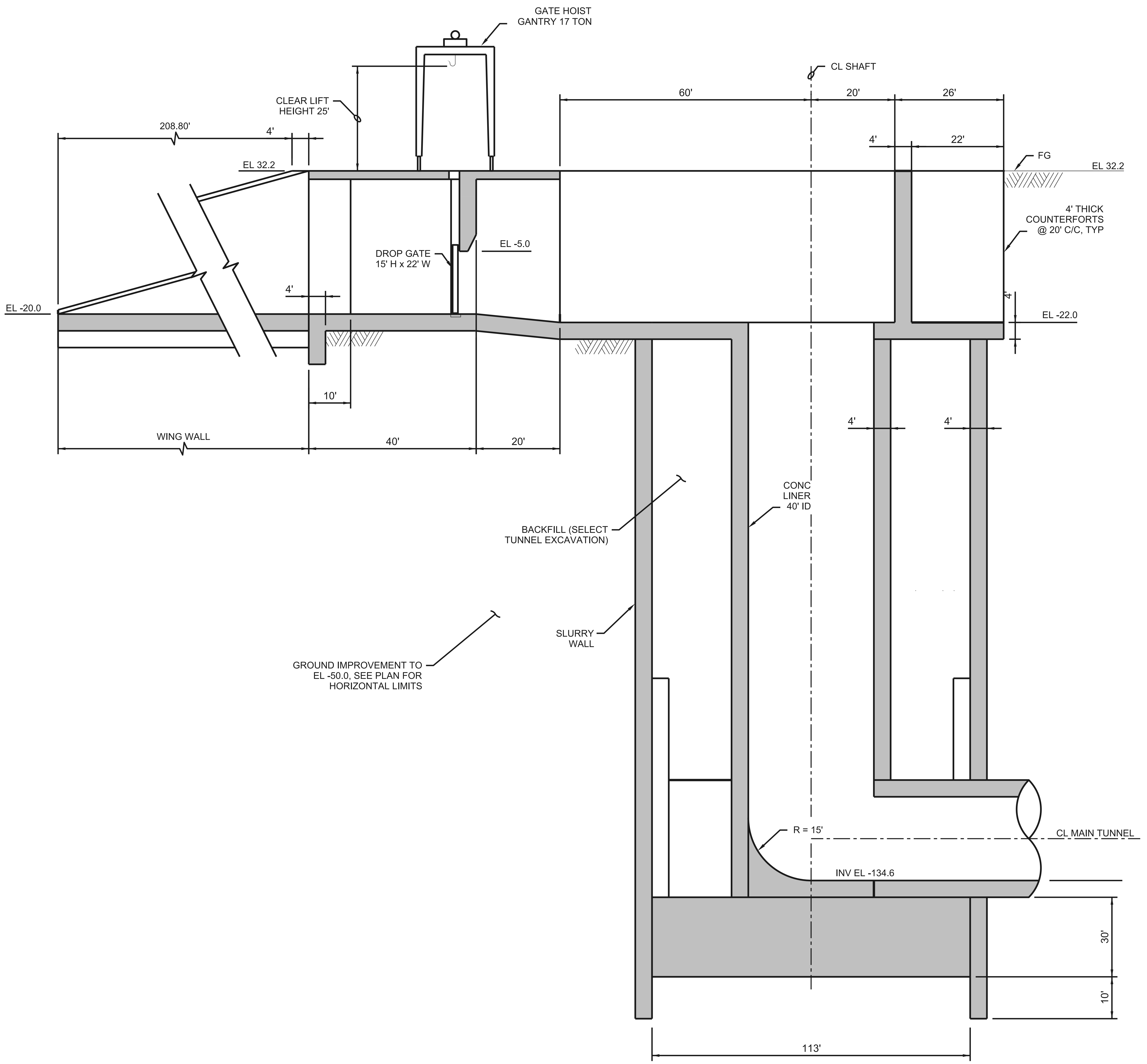
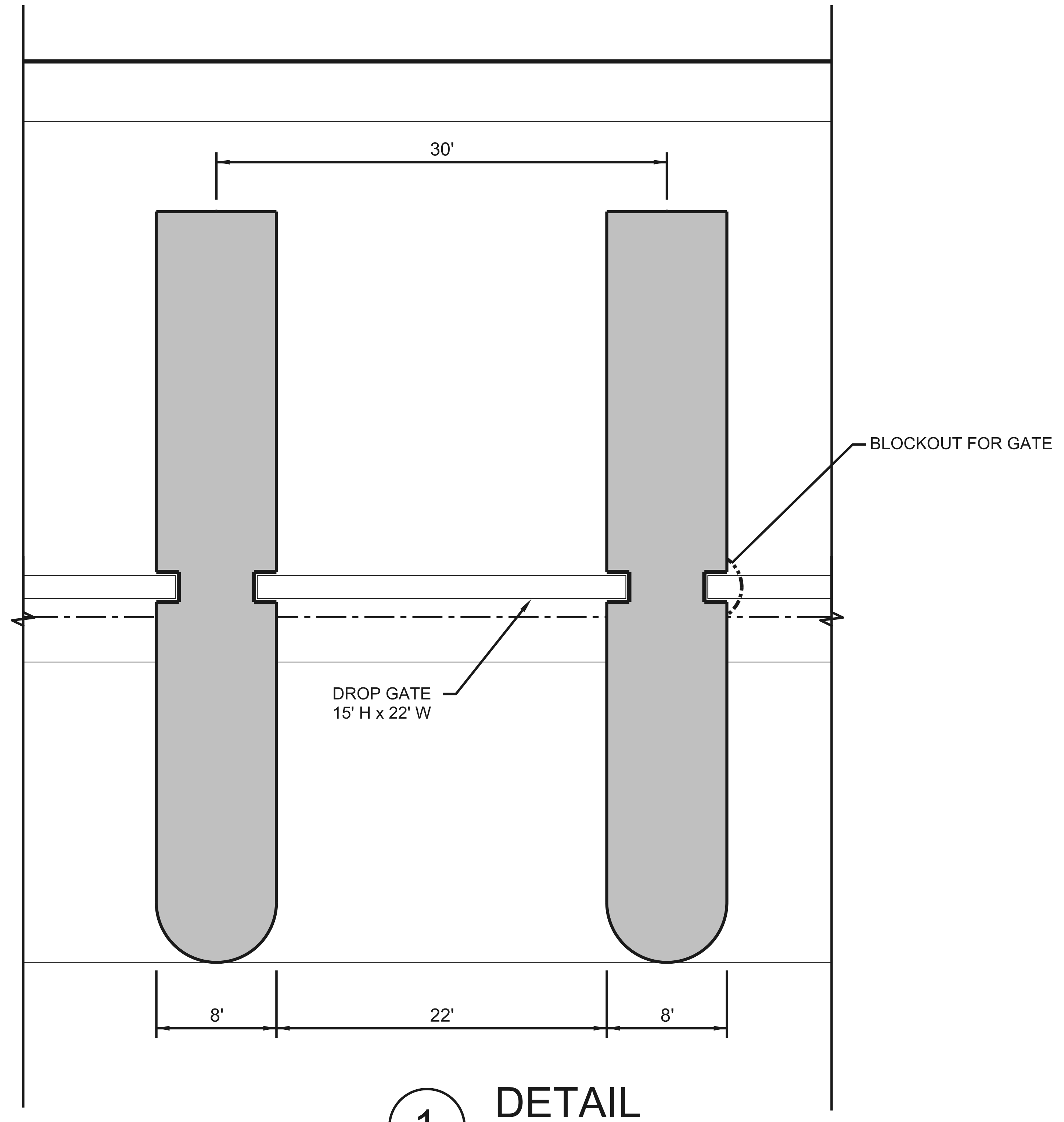


B SECTION
NTS
CCO-M-4061IF



1 DETAIL
NTS
CCO-M-4061IF

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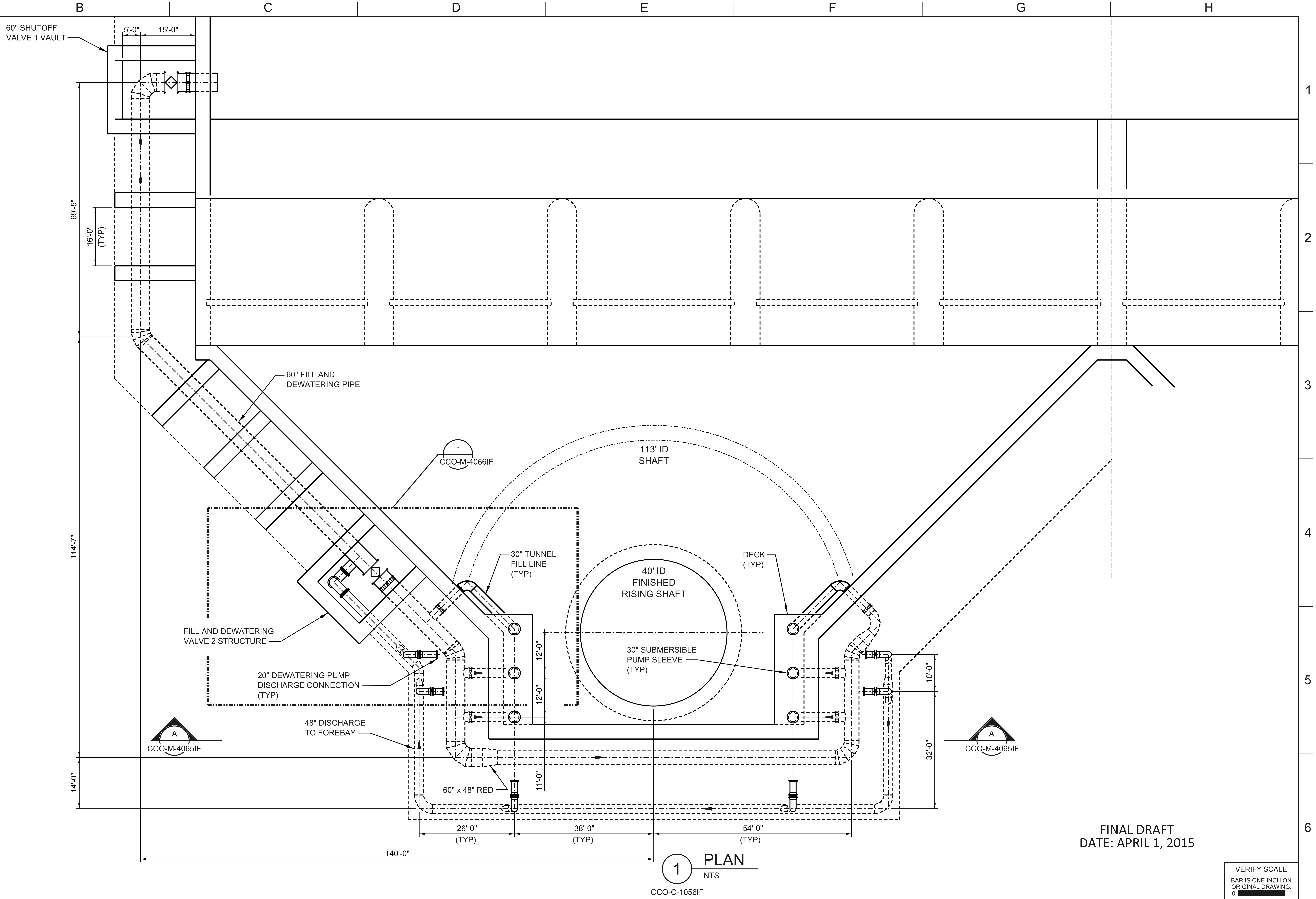
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY OUTLET STRUCTURE
SECTION AND DETAILS

PROJECT NO. 240010
SHEET NO. CCO - M - 5063IF
REV 0
SEQUENCE NO. 63 OF 96



1 PLAN

NTS

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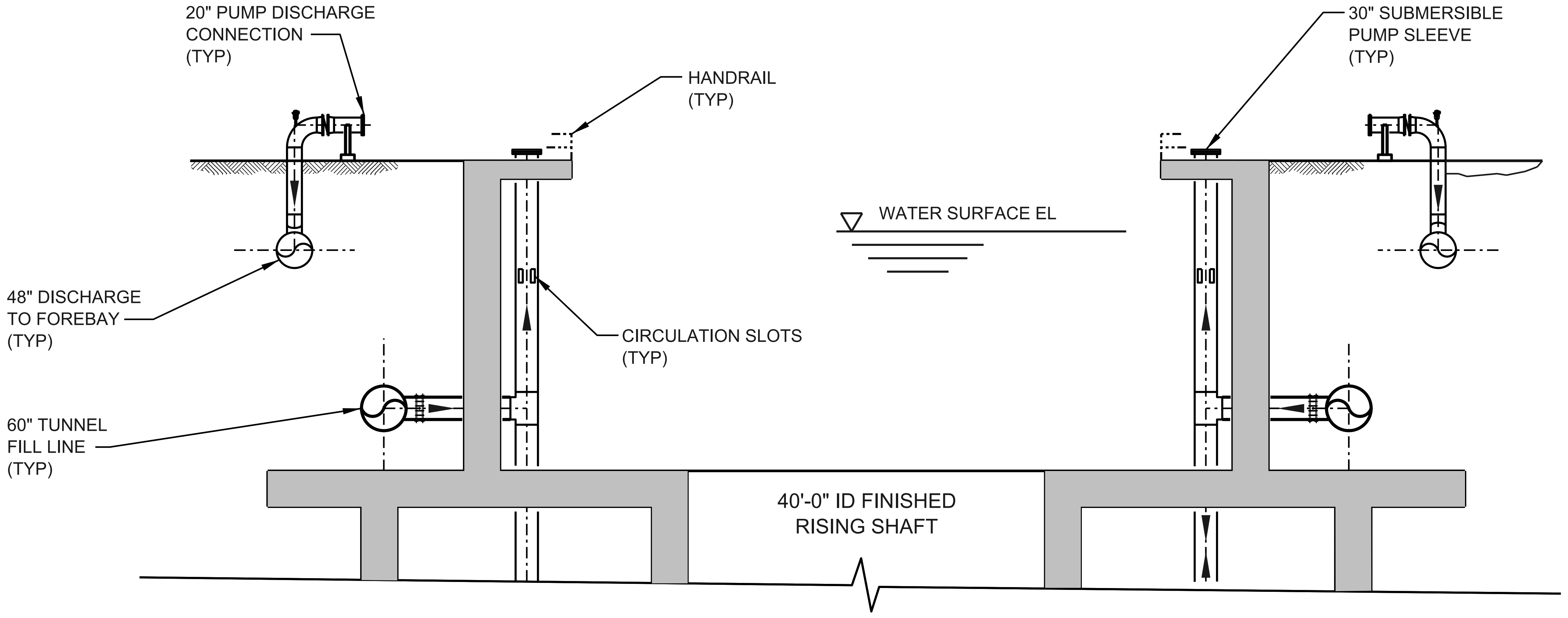
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL GENERAL ARRANGEMENT
FILL AND DRAIN PIPING UPPER LEVEL - PLAN

PROJECT NO. 240010
SHEET NO. CCO - M - 4064IF
REV 0
SEQUENCE NO. 64 OF 96



A SECTION
NTS
CCO-M-4064IF

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DATE: APRIL 1, 2015

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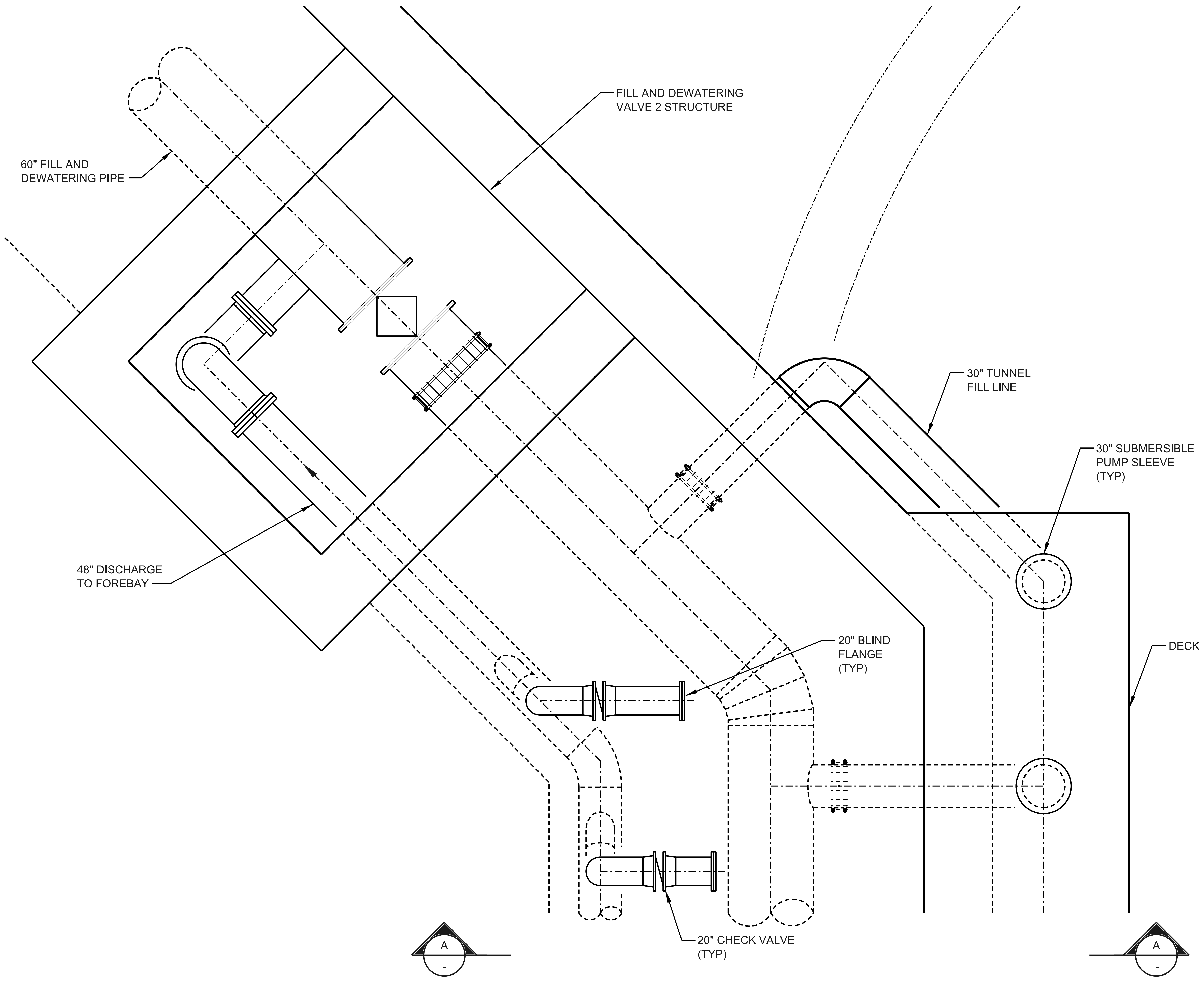
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

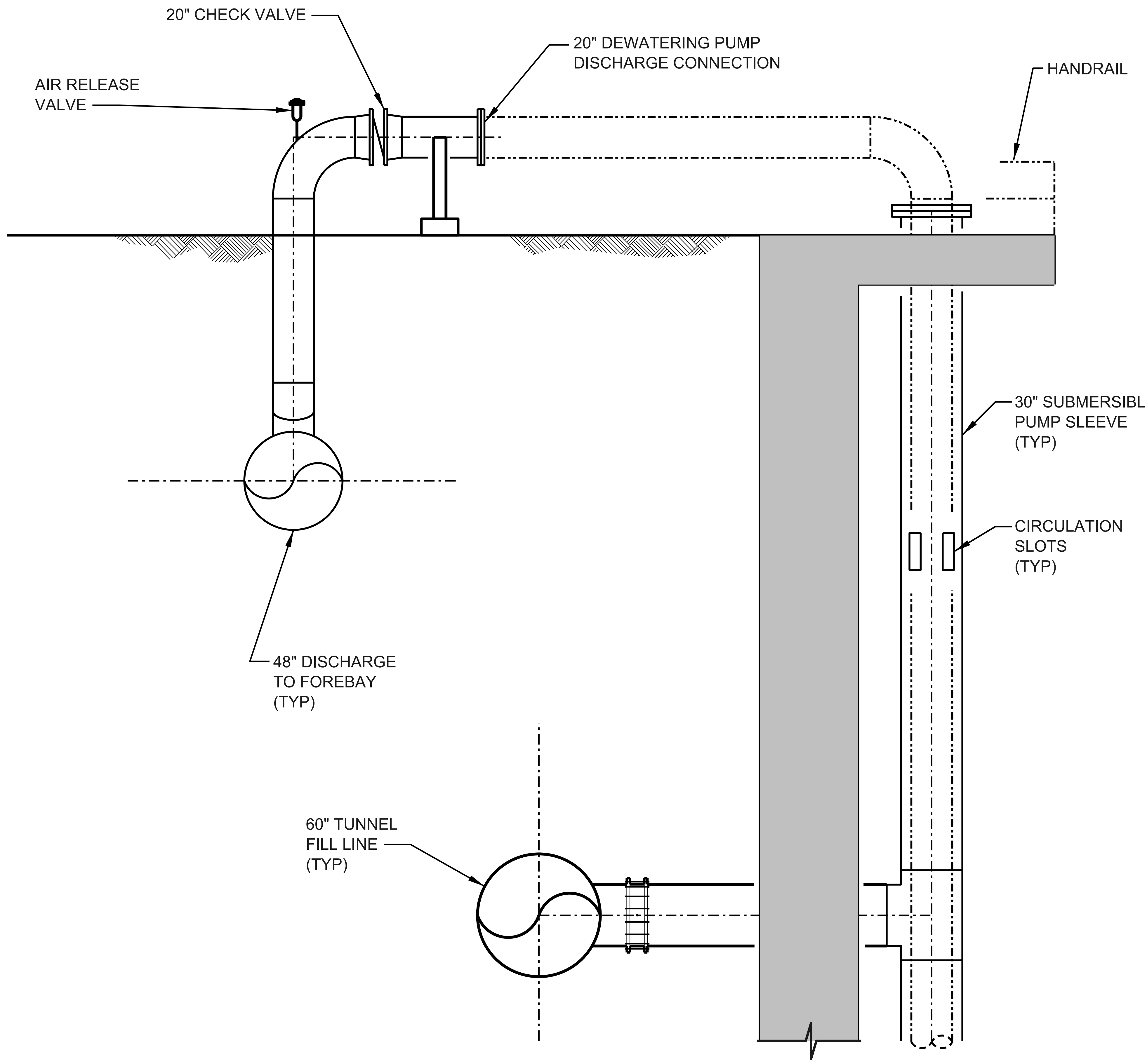
FILL AND DRAIN PIPING
UPPER LEVEL - SECTION

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

PROJECT NO. 240010
SHEET NO. CCO - M - 4065IF
REV 0
SEQUENCE NO. 65 OF 96



1 DETAIL
NTS
CCO-M-4064IF



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VERIFY SCALE
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0 1"

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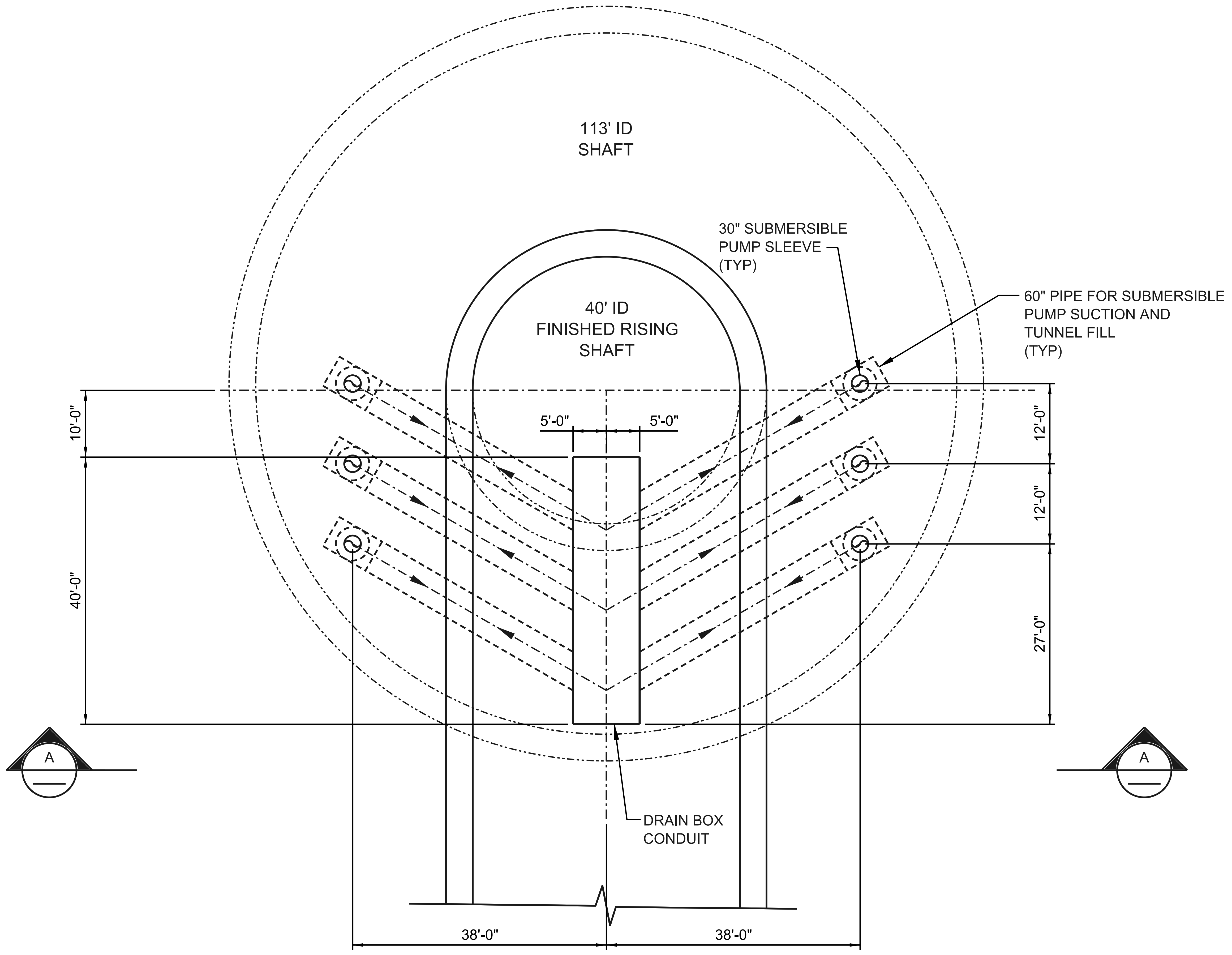
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

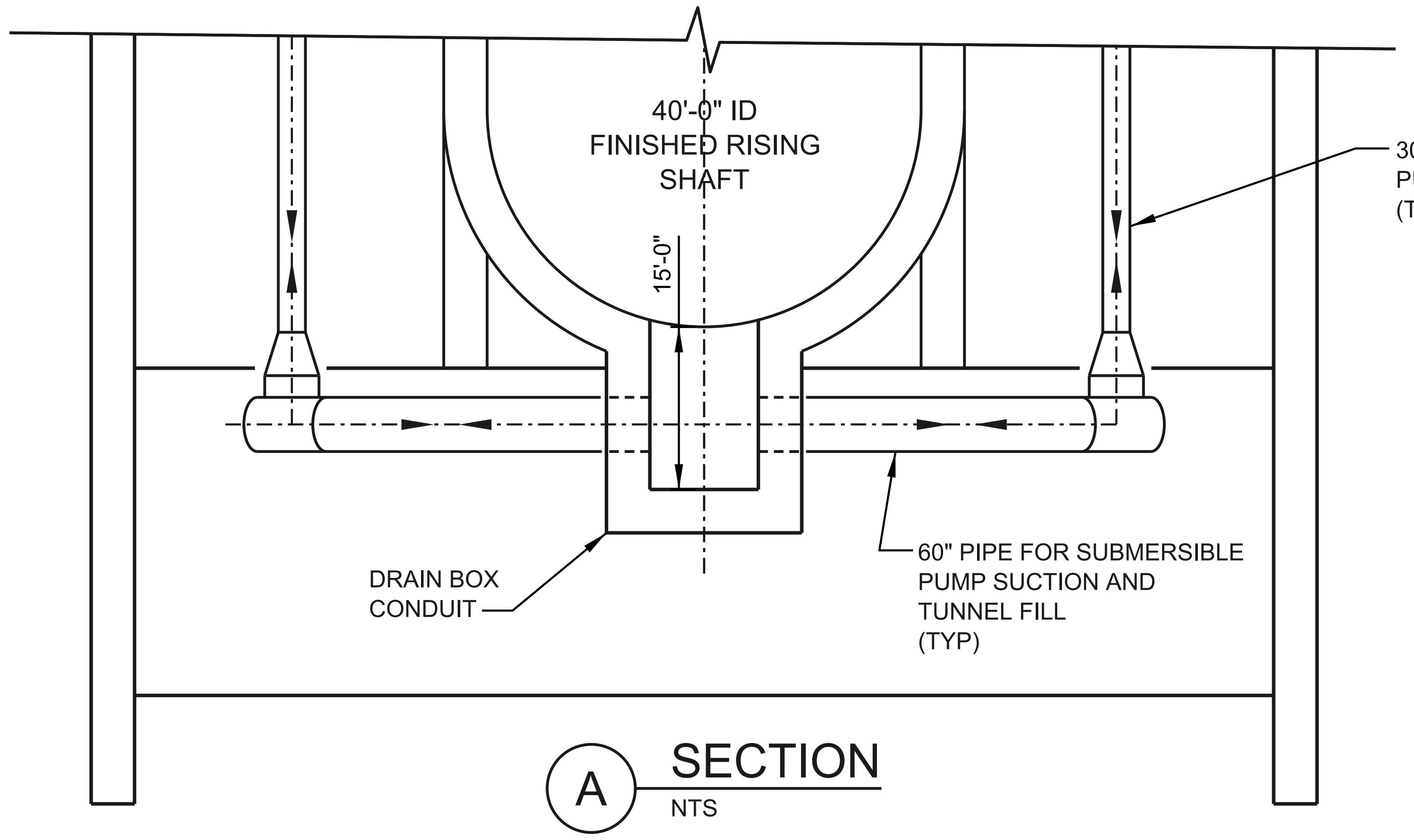
FILL AND DRAIN PIPING
DETAIL PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - M - 4066IF
REV 0
SEQUENCE NO. 66 OF 96



1 PLAN
NTS

CCO-M-4064IF



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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL FOREBAY DRAIN/FILL
PLAN AND SECTION

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 ▬ 1"

PROJECT NO. 240010
SHEET NO. CCO - M - 4067IF
REV 0
SEQUENCE NO. 67 OF 96

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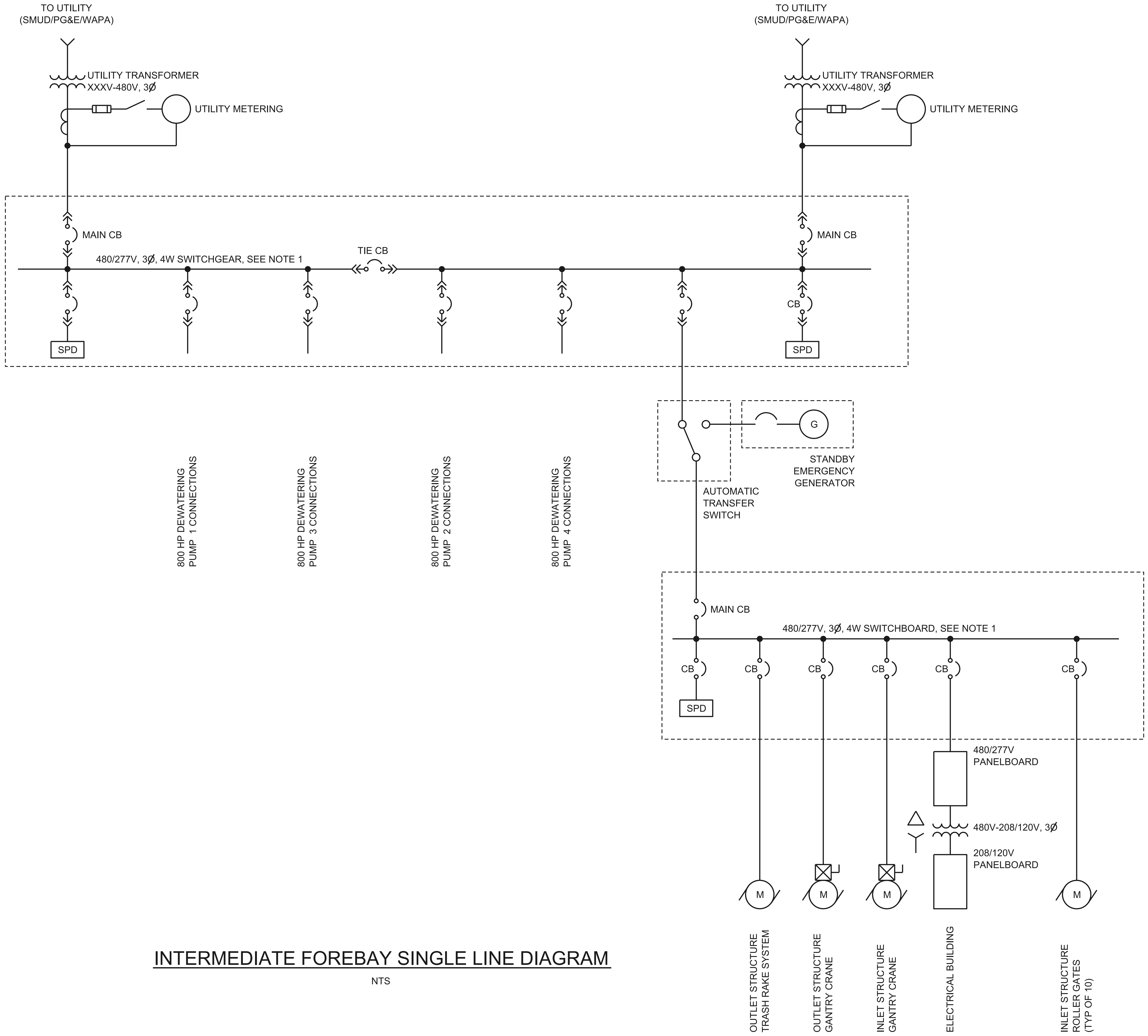

INTERMEDIATE FOREBAY SINGLE LINE DIAGRAM

NTS

NOTE:

1. SWITCHGEAR AND SWITCHBOARD SHALL BE LOCATED WITHIN ELECTRICAL ROOM.

GENERAL NOTE:

1. ALL ELECTRICAL EQUIPMENT SHALL BE INSTALLED ABOVE THE 200 YEAR FLOODPLAIN.

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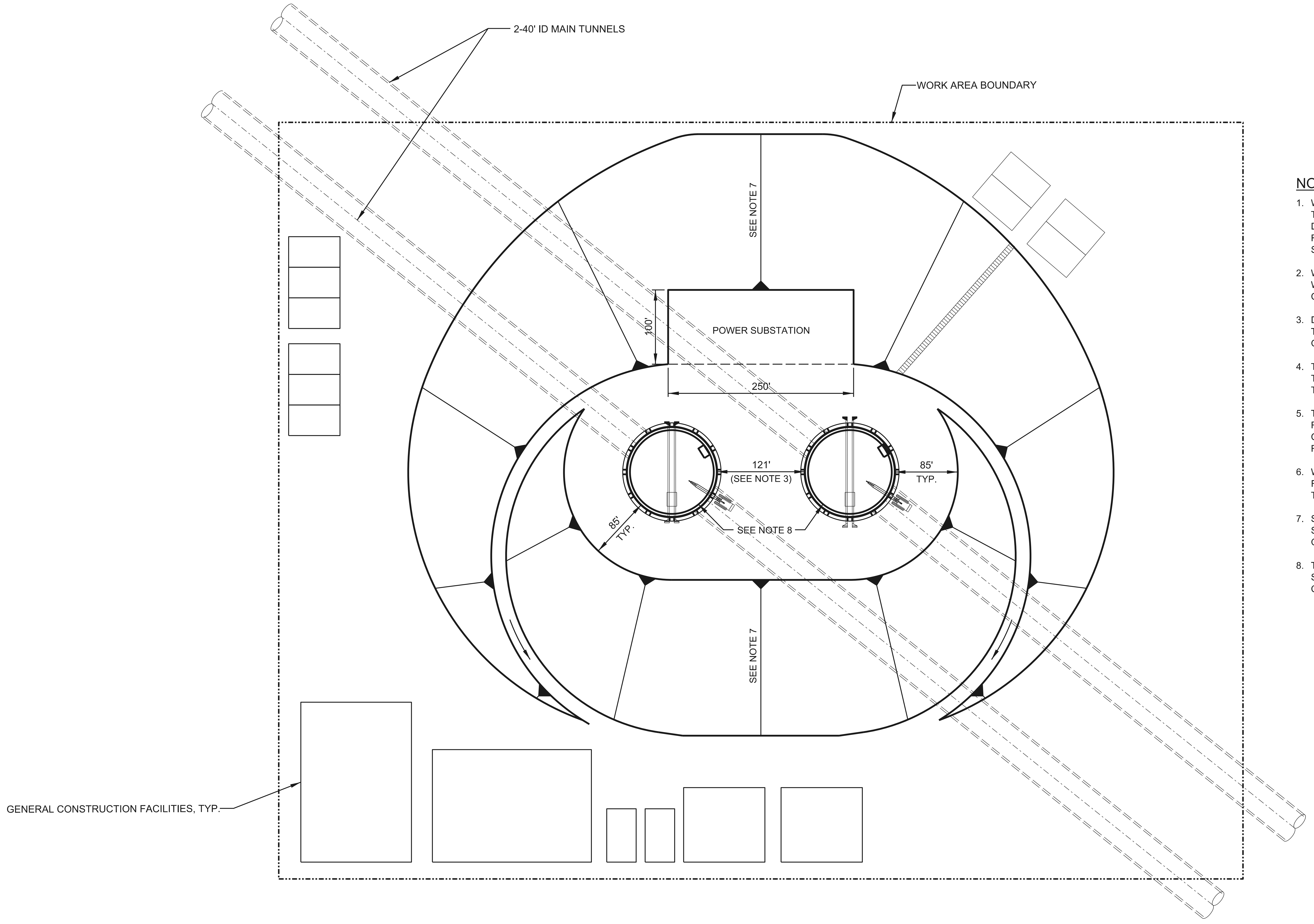
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

INTERMEDIATE FOREBAY
SINGLE LINE DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - E - 6068IF
REV 0
SEQUENCE NO. 68 OF 96



HWY 12

PLAN

1"=80'

NOTES

1. WORK AREA PLAN SHOWN HEREON IS TYPICAL FOR THE DRIVE SHAFTS FOR THE 40-FOOT ID MAIN TUNNELS. TWO SHAFTS WILL BE LOCATED AT EACH DRIVE SHAFT WORK AREA, ONE FOR EACH TUNNEL. WORK AREA PLANS FOR RECEPTION ONLY SHAFTS FOR THE 40-FOOT ID MAIN TUNNELS WILL HAVE SIMILAR, BUT SMALLER CONFIGURATIONS.
2. WORK AREAS FOR DRIVE OR RECEPTION SHAFTS FOR THE NORTH TUNNELS WILL ONLY REQUIRE ONE SHAFT AND WILL HAVE SIMILAR, BUT SMALLER CONFIGURATIONS.
3. DRIVE SHAFT SEPARATION FOR THE MAIN TUNNELS IS BASED ON A MINIMUM TWO SHAFT OUTSIDE DIAMETER CENTER-TO-CENTER SPACING (121 FEET OUTSIDE DIAMETER OF SLURRY WALL).
4. THE BOULDIN ISLAND WILL BE USED AS DRIVE SHAFTS FOR THE ADJACENT TUNNEL REACHES. THE CONTRACTORS SHALL COORDINATE THE USE OF THESE SHAFTS FOR LAUNCHING TUNNELING EQUIPMENT AND MATERIALS.
5. THE STATEN ISLAND AND BACON ISLAND SHAFTS WILL ONLY BE USED AS RECEPTION SHAFTS FOR THE ADJACENT TUNNEL REACHES. THE CONTRACTORS SHALL COORDINATE THE USE OF THESE SHAFTS FOR RETRIEVING TUNNELING EQUIPMENT AND MATERIALS.
6. WORK AREA LAYOUT SHOWN HEREON IS FOR CONCEPTUAL STUDY ONLY. FINAL TUNNEL SHAFT WORK AREA CONFIGURATIONS WILL BE DETERMINED BY THE CONTRACTOR'S MEANS AND METHODS.
7. SIDE SLOPES OF CONSTRUCTION SHAFT PAD ARE BETWEEN 3:1 - 5:1. SLOPE INCLINATIONS WILL BE DETERMINED DURING PRELIMINARY DESIGN ONCE GEOTECHNICAL AND ENGINEERING DATA IS AVAILABLE.
8. TUNNEL DRIVE, RECEPTION OR CONTRACTOR'S VENTILATION SHAFTS. SEE STRUCTURAL DRAWINGS CCO-S-5071TS AND CCO-S-5072TS .

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

TYPICAL TUNNEL SHAFTS
WORK AREA PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1069TS
REV 0
SEQUENCE NO. 69 OF 96

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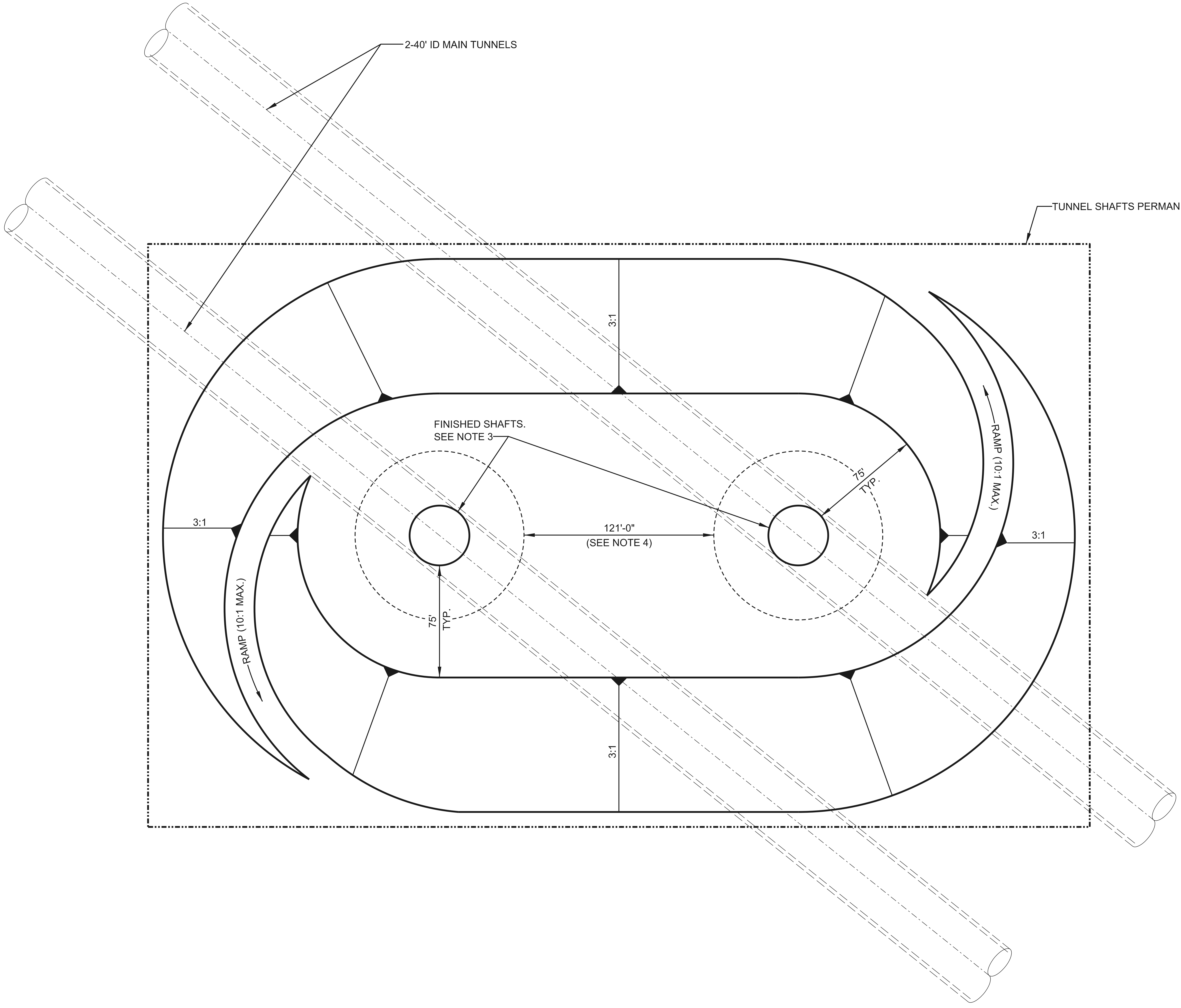

PLAN

1"=40'

NOTES

1. SHOWN HEREON IS THE TYPICAL CONFIGURATION OF A FINSIHED 40-FOOT ID MAIN TUNNEL DRIVE SHAFT SITE. TWO FINISHED SHAFTS WILL BE LOCATED AT EACH FINISHED PAD SITE, ONE FOR EACH TUNNEL.
2. NORTH TUNNELS FINISHED DRIVE AND RECEPTION SHAFT PAD SITES ONLY HAVE ONE SHAFT. FINISH DESIGN LAYOUTS VARY. SEE INTAKES SITE PLANS AND THE INTERMEDIATE FOREBAY PLAN FOR CONFIGURATION.
3. FINISHED SHAFT ID'S WILL VARY FROM 20' TO 40'. SEE STRUCTURAL DRAWING CCO-S-5073TS.
4. FINISHED SHAFT SEPARATIONS WILL DEPEND UPON THE ACTUAL CONSTRUCTION SHAFT SEPARATION SELECTED BY THE CONTRACTOR. FINISHED SHAFT SPACING SHOWN HEREON IS BASED ON THE MINIMUM MAIN TUNNELS SHAFT SEPARATION.
5. FINISHED TUNNEL SHAFT SITE PLAN LAYOUT SHOWN HEREON IS FOR CONCEPTUAL STUDY ONLY. FINAL FINISHED TUNNEL SHAFT SITE CONFIGUATIONS WILL BE DETERMINED BY THE CONTRACTOR'S WORK AREA CONFIGURATION AND THE SPACE REQUIREMENTS NEEDED FOR FACILITY OPERATON AND MAINTENANCE.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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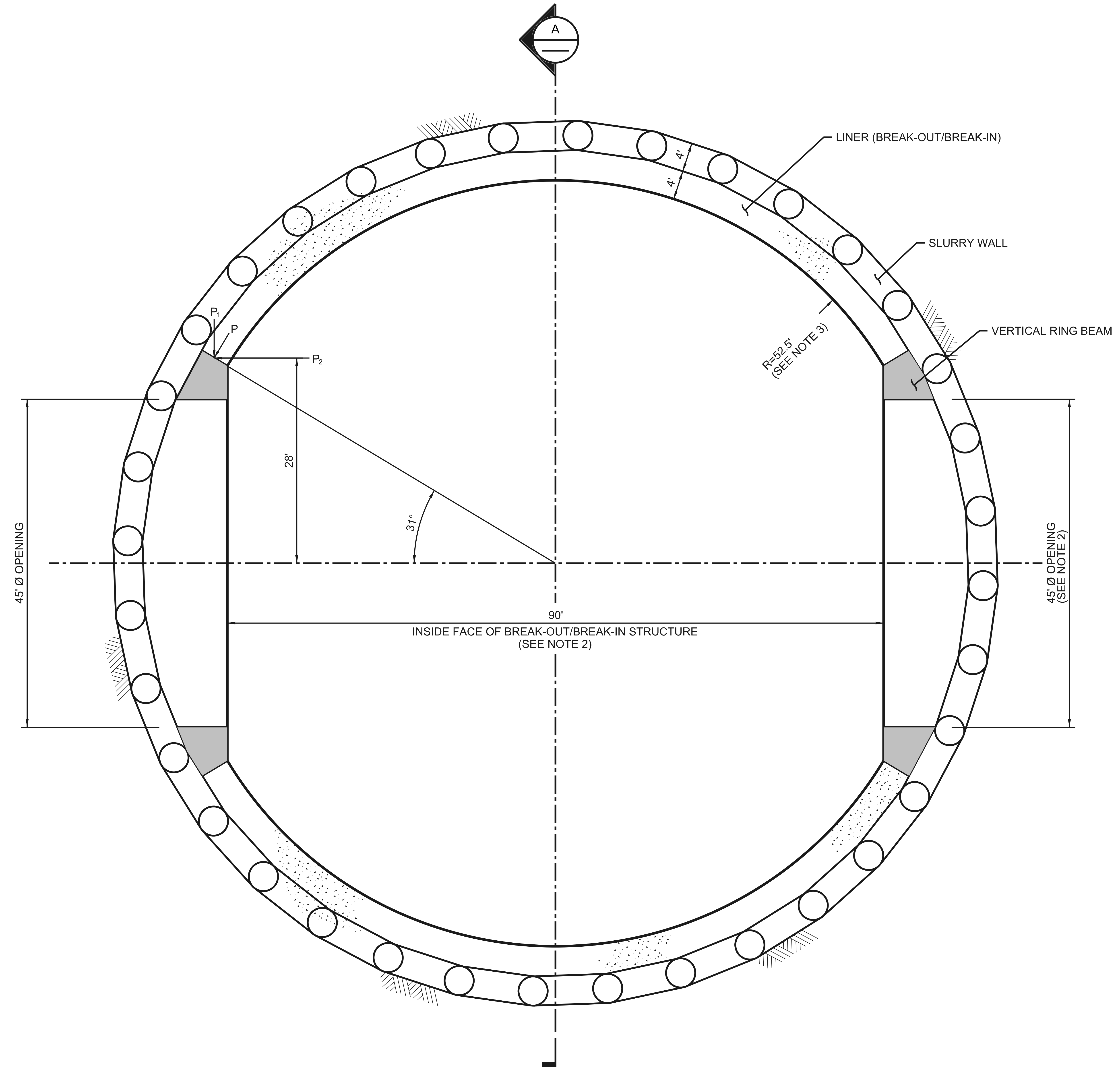
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

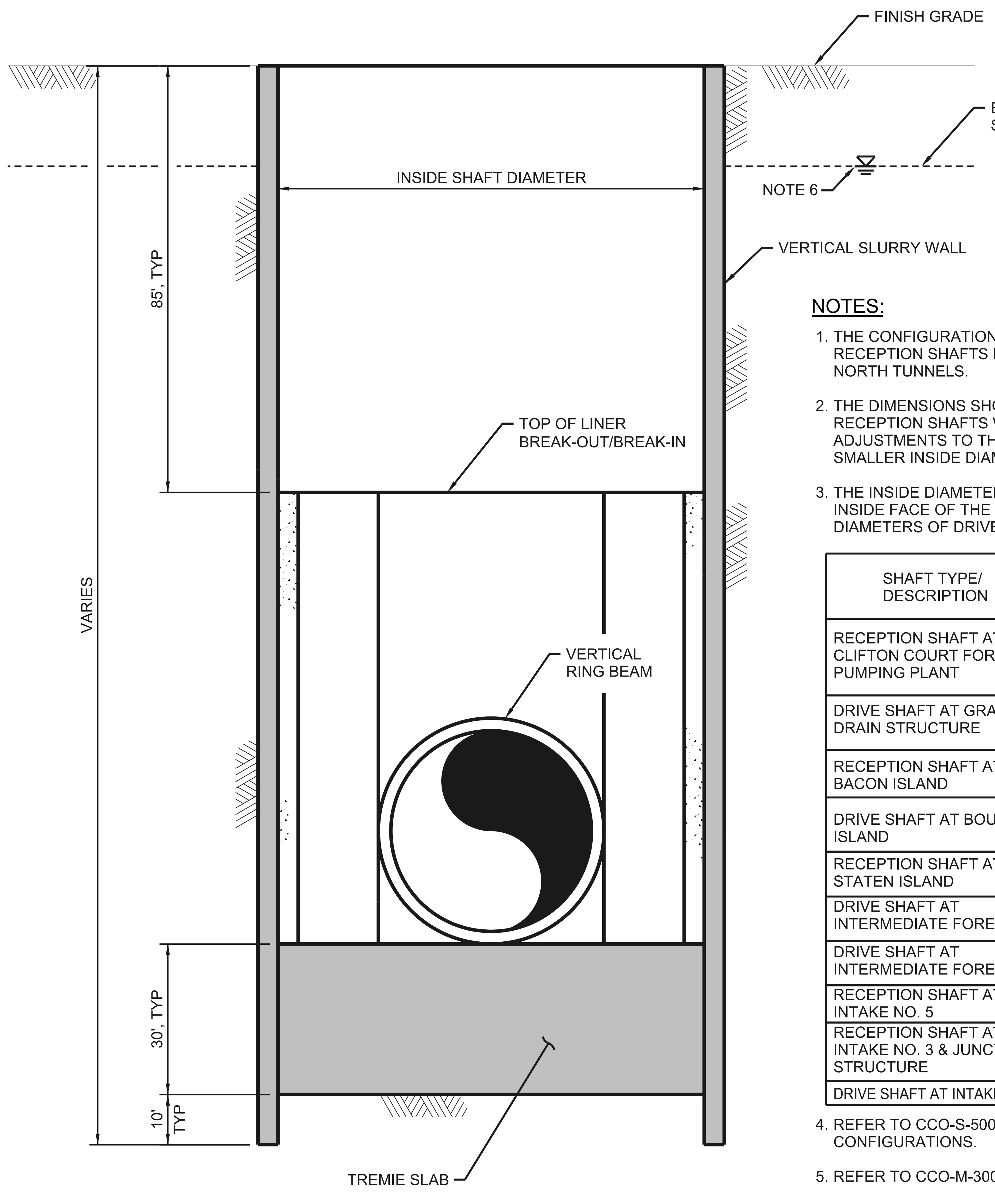
TYPICAL TUNNEL SHAFTS
FINISHED PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1070TS
REV 0
SEQUENCE NO. 70 OF 96



PLAN

1"=20'



A SECTION

1"=20"

NOTES:

1. THE CONFIGURATION SHOWN HEREON FOR THE DRIVE AND RECEPTION SHAFTS IS APPLICABLE FOR THE MAIN TUNNELS AND NORTH TUNNELS.
2. THE DIMENSIONS SHOWN HEREON ARE APPLICABLE FOR DRIVE AND RECEPTION SHAFTS WITH AN INSIDE DIAMETER OF 113 FEET. ADJUSTMENTS TO THESE DIMENSIONS SHOULD BE MADE FOR SMALLER INSIDE DIAMETERS.
3. THE INSIDE DIAMETER OF THE SHAFTS IS MEASURED FROM THE INSIDE FACE OF THE SLURRY WALL. THE ANTICIPATED INSIDE DIAMETERS OF DRIVE AND RECEPTION SHAFTS ARE AS FOLLOWS:

SHAFT TYPE/ DESCRIPTION	TUNNEL DESCRIPTION	TUNNEL DIAMETER (FEET)	SHAFT INSIDE DIAMETER (FEET) (* = SEE NOTE 7)
RECEPTION SHAFT AT CLIFTON COURT FOREBAY PUMPING PLANT	MAIN	40	* 150
DRIVE SHAFT AT GRAVITY DRAIN STRUCTURE	MAIN	40	113
RECEPTION SHAFT AT BACON ISLAND	MAIN	40	100
DRIVE SHAFT AT BOULDIN ISLAND	MAIN	40	113
RECEPTION SHAFT AT STATEN ISLAND	MAIN	40	100
DRIVE SHAFT AT INTERMEDIATE FOREBAY	MAIN	40	* 113
DRIVE SHAFT AT INTERMEDIATE FOREBAY	NORTH	40	* 113
RECEPTION SHAFT AT INTAKE NO. 5	NORTH	28	100
RECEPTION SHAFT AT INTAKE NO. 3 & JUNCTION STRUCTURE	NORTH	28 AND 40	* 113
DRIVE SHAFT AT INTAKE NO. 2	NORTH	28	100

4. REFER TO CCO-S-5003TS FOR FINAL DRIVE AND RECEPTION SHAFT CONFIGURATIONS.
5. REFER TO CCO-M-3005IT FOR FINAL JUNCTION STRUCTURE SHAFT CONFIGURATION.
6. HIGH GROUNDWATER LEVEL IS COINCIDENT WITH EXISTING GROUND SURFACE.
7. REFER TO MECHANICAL DRAWINGS FOR FINISHED SHAFT DIAMETER AT FACILITIES.

FINAL DRAFT
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY-MODIFIED PIPELINE / TUNNEL OPTION

NORTH AND MAIN TUNNELS TYPICAL DRIVE AND RECEPTION SHAFT - PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - S - 5071TS
REV 0
SEQUENCE NO. 71 OF 96

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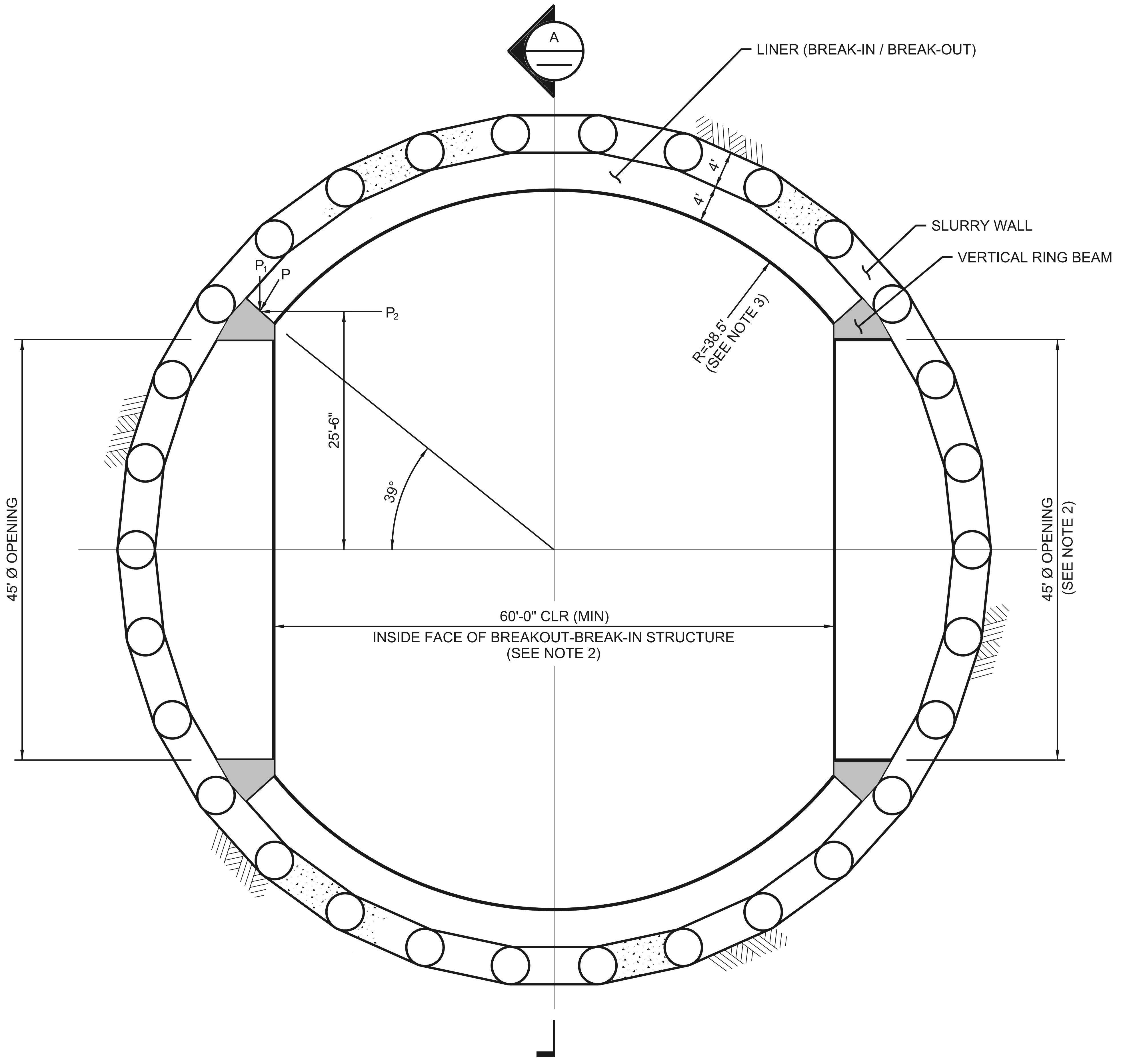
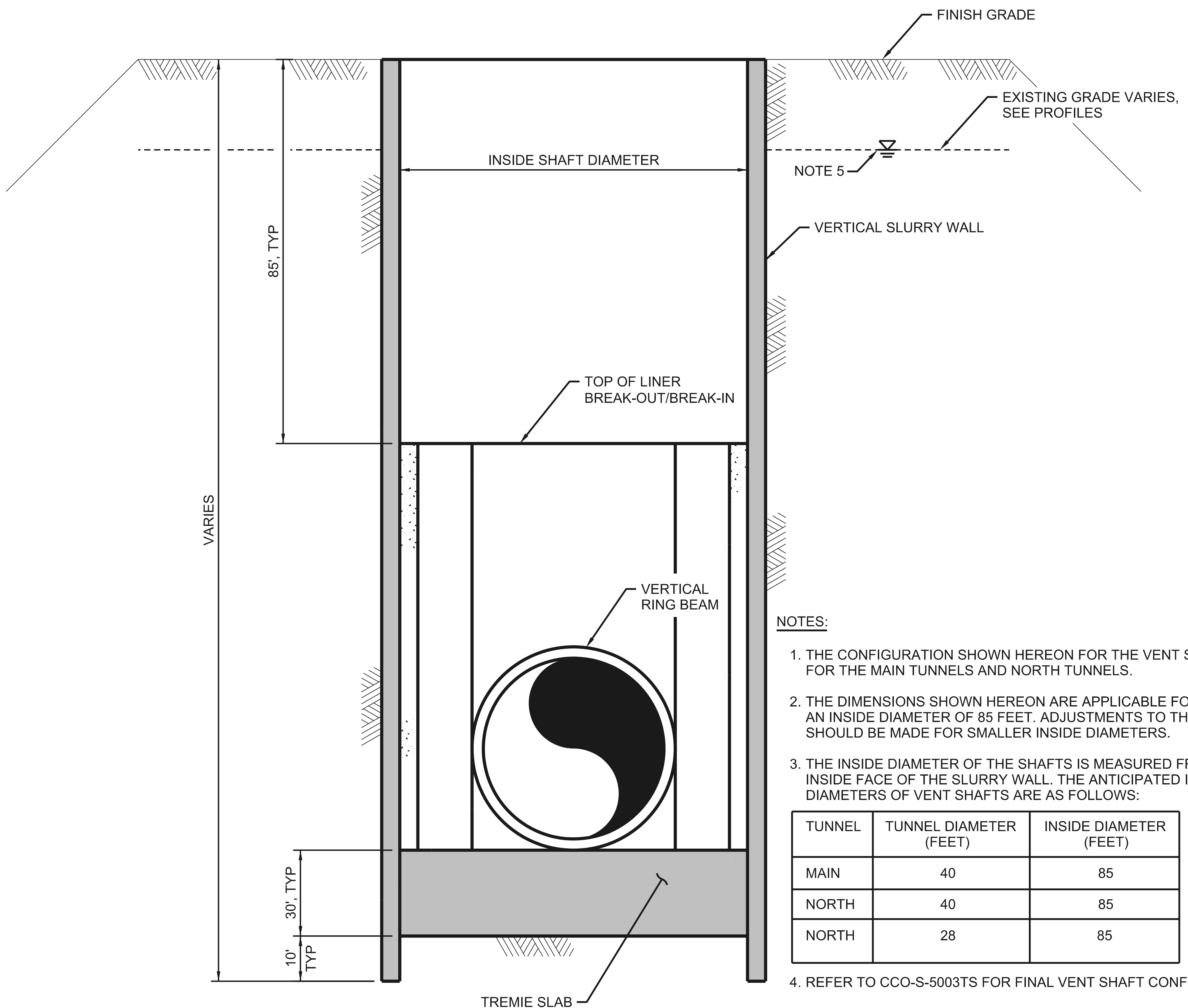


PLAN
1"=10"



(A) SECTION
1"=20"

NOTES:

1. THE CONFIGURATION SHOWN HEREON FOR THE VENT SHAFTS IS APPLICABLE FOR THE MAIN TUNNELS AND NORTH TUNNELS.
2. THE DIMENSIONS SHOWN HEREON ARE APPLICABLE FOR VENT SHAFTS WITH AN INSIDE DIAMETER OF 85 FEET. ADJUSTMENTS TO THESE DIMENSIONS SHOULD BE MADE FOR SMALLER INSIDE DIAMETERS.
3. THE INSIDE DIAMETER OF THE SHAFTS IS MEASURED FROM THE INSIDE FACE OF THE SLURRY WALL. THE ANTICIPATED INSIDE DIAMETERS OF VENT SHAFTS ARE AS FOLLOWS:

TUNNEL	TUNNEL DIAMETER (FEET)	INSIDE DIAMETER (FEET)
MAIN	40	85
NORTH	40	85
NORTH	28	85

4. REFER TO CCO-S-5003TS FOR FINAL VENT SHAFT CONFIGURATIONS.
5. HIGH GROUNDWATER LEVEL IS COINCIDENT WITH EXISTING GROUND SURFACE.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
BAR IS ONE INCH ON ORIGINAL DRAWING.
0 1"

REV	DATE	DESCRIPTION	SUB.	APPD

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY-MODIFIED PIPELINE / TUNNEL OPTION

NORTH AND MAIN TUNNELS TYPICAL VENT SHAFT - PLAN AND SECTION

PROJECT NO. 240010
SHEET NO. CCO - S - 5072TS
REV 0
SEQUENCE NO. 72 OF 96

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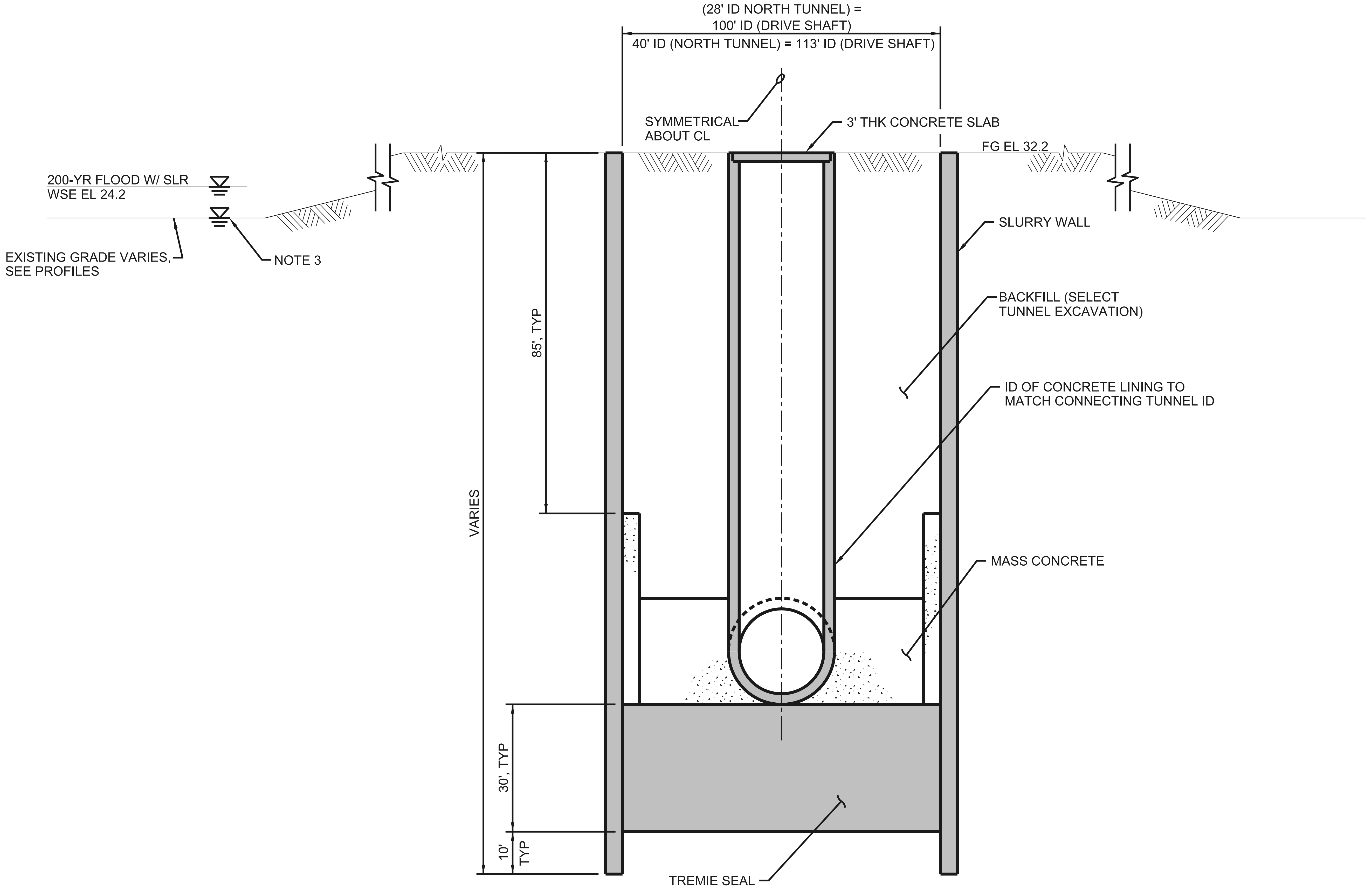
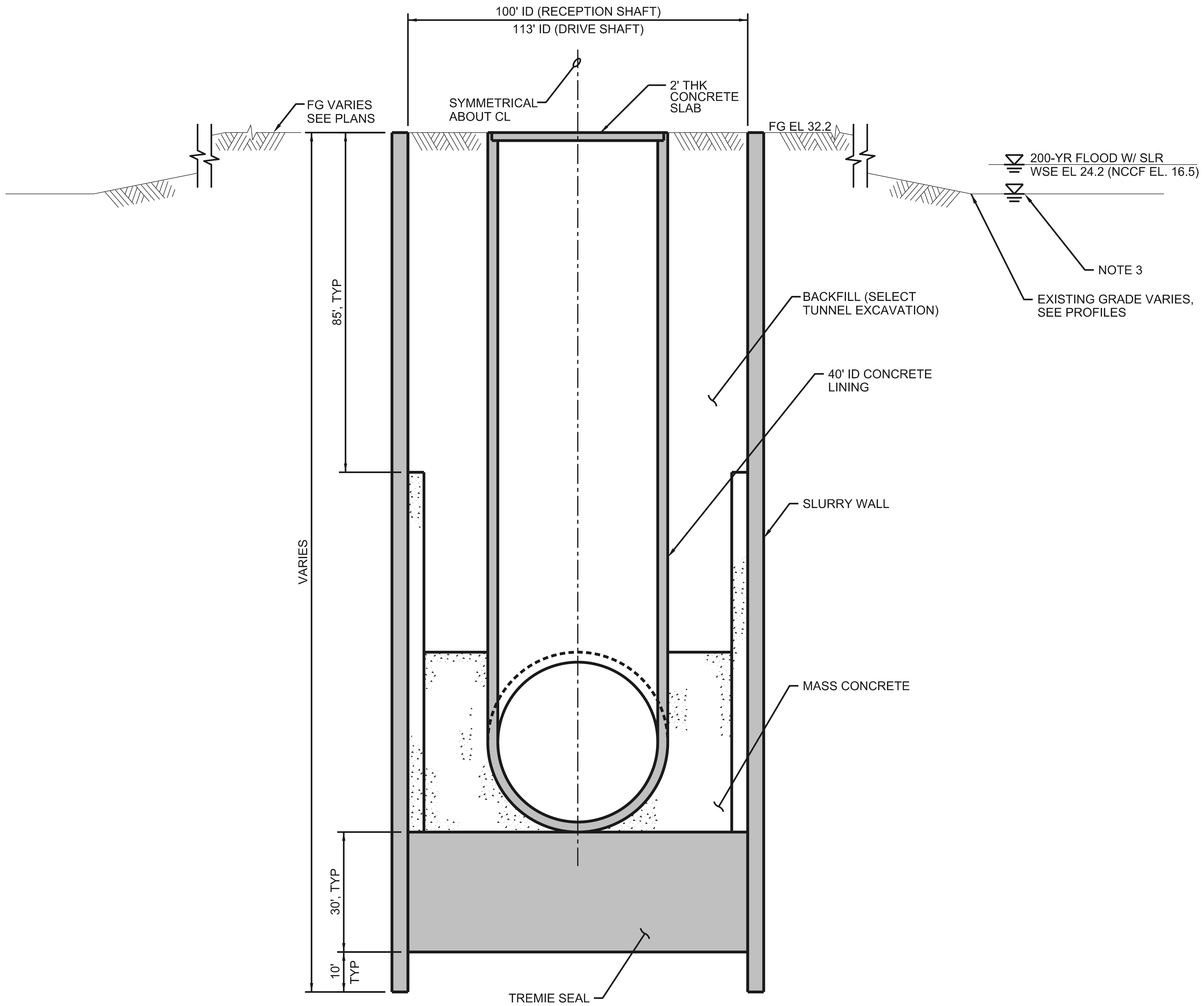


NOTES:

1. DIMENSIONS SHOWN HERE ARE TYPICAL UNLESS NOTE OTHERWISE ON SHEET CCO-S-5071TS.
2. HIGH GROUNDWATER LEVEL COINCIDENT WITH ORIGINAL GRADE.

NORTH TUNNELS

SECTION
1"=20'



NOTES:

1. DIMENSIONS SHOWN HERE ARE TYPICAL UNLESS NOTED OTHERWISE ON SHEET CCO-S-5071TS.
2. HIGH GROUNDWATER LEVEL COINCIDENT WITH EXISTING GRADE.

MAIN TUNNELS

SECTION
1"=20'

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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH AND MAIN TUNNELS TYPICAL
FINAL SHAFT CONFIRGURATION - SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - S - 5073TS
REV 0
SEQUENCE NO. 73 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\04 Structural\CCO-S-073-240010

Edited By: Bautista,Jav
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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY
PUMPING PLANT RENDERING

PROJECT NO. 240010

SHEET NO. CCO - G - 9074PP

REV 0

SEQUENCE NO. 74 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\06 Mechanical\CCO-M-074-240010.dgn

Edited By: Mendez,Martin

Printed By: Mendez,Martin

NOTE:

1. FOR PUMPING PLANT PLAN, SEE SHEET CCO-M-1076PP.

FINAL DRAFT
DATE: APRIL 1, 2015

PLAN
1"=200'

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY PUMPING PLANT
SITE PLAN

PROJECT NO. 240010

SHEET NO. CCO - C - 1075PP

REV 0

SEQUENCE NO. 75 OF 96

2 DETAIL
1"=80'
CCO-C-1075PP

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY PUMPING PLANT PLAN

PROJECT NO. 240010
SHEET NO. CCO - M - 1076PP
REV 0
SEQUENCE NO. 76 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\Documents\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\06 Mechanical\CCO-M-076-240010.dgn

Edited By: Mendez,Martin
Printed By: Mendez,Martin

NOTE:

1. STRUCTURAL CONFIGURATION OF PUMP SUCTION INLET COMPARTMENT, SHALL BE ADDRESSED DURING THE PREPARATION OF FINAL DESIGN REPORT.

1 PLAN

1"=25'

CCO-M-1076PP

FINAL DRAFT
DATE: APRIL 1, 2015

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY PUMPING PLANT PLAN

PROJECT NO. 240010
SHEET NO. CCO - M - 1077PP
REV 0
SEQUENCE NO. 77 OF 96

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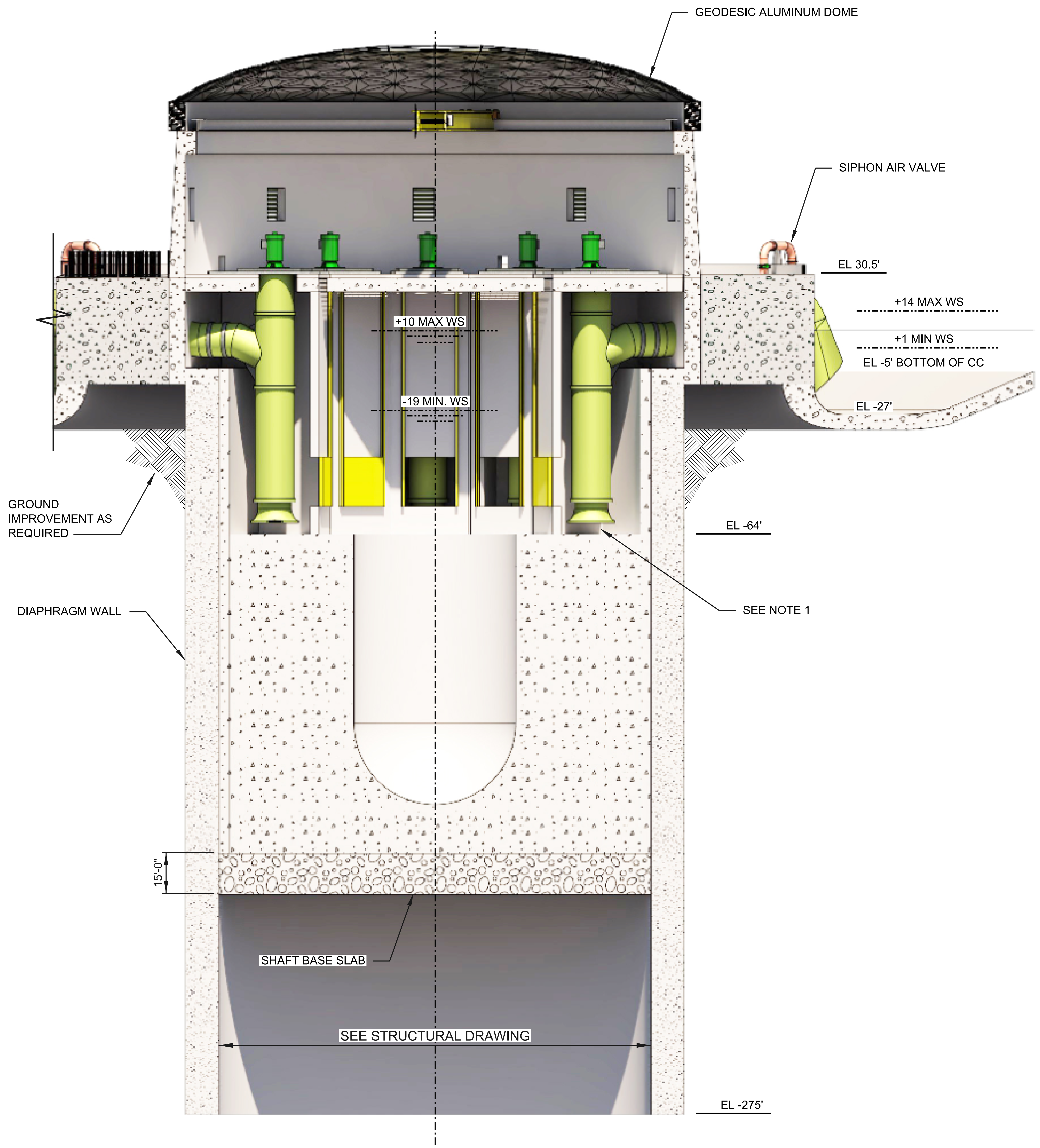
Edited By: Mendez,Martin
Printed By: Mendez,Martin

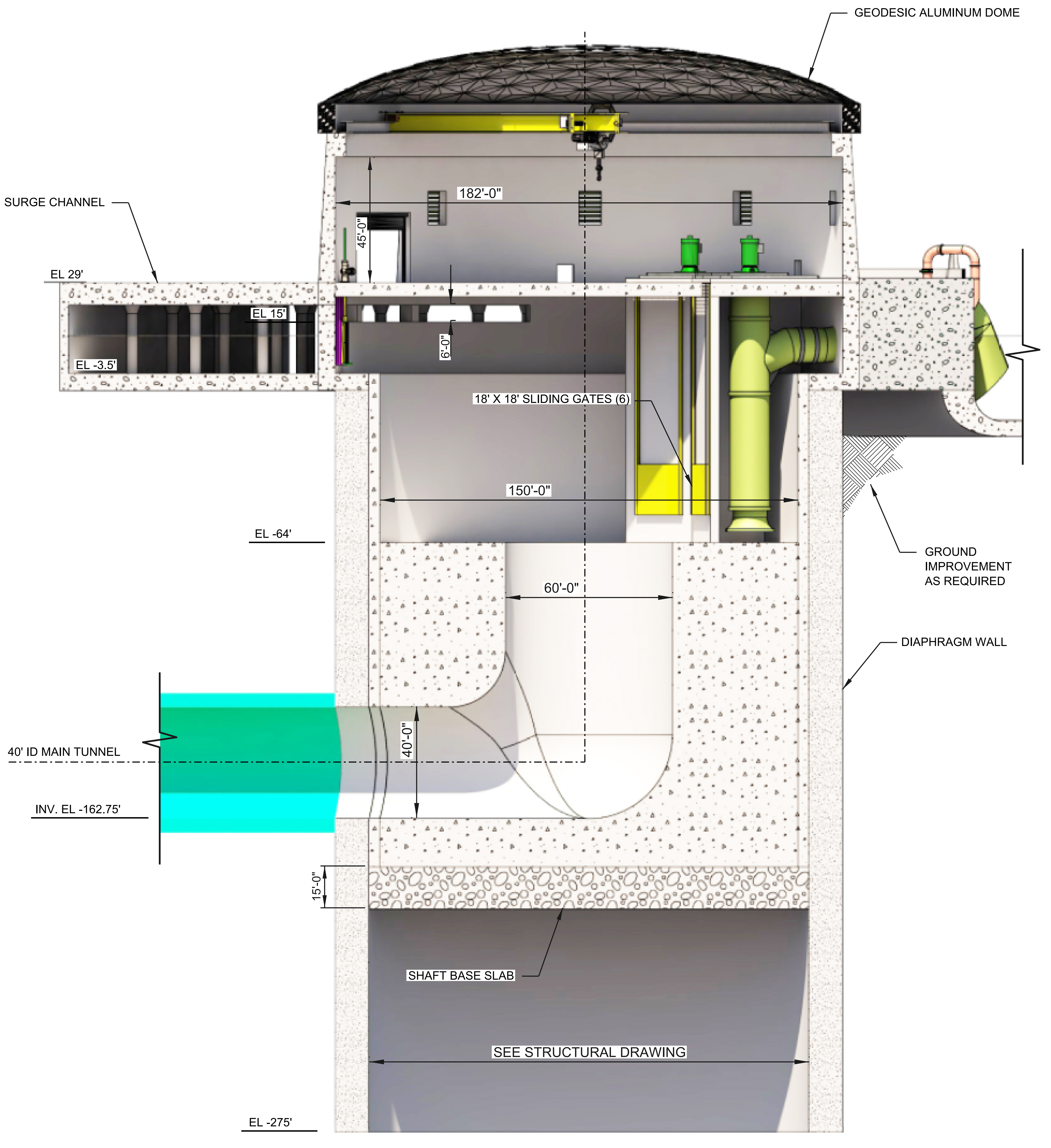


A TYPICAL SECTION

1"=25'

CCO-M-1077PP

1" = 25' 0 25 50 75 FEET



B TYPICAL SECTION

1"=25'

CCO-M-1077PP

NOTE:

1. STRUCTURAL CONFIGURATION OF PUMP SUCTION INLET COMPARTMENT, SHALL BE ADDRESSED DURING THE PREPARATION OF FINAL DESIGN REPORT.

FINAL DRAFT
DATE: APRIL 1, 2015

VERIFY SCALE
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0 1"

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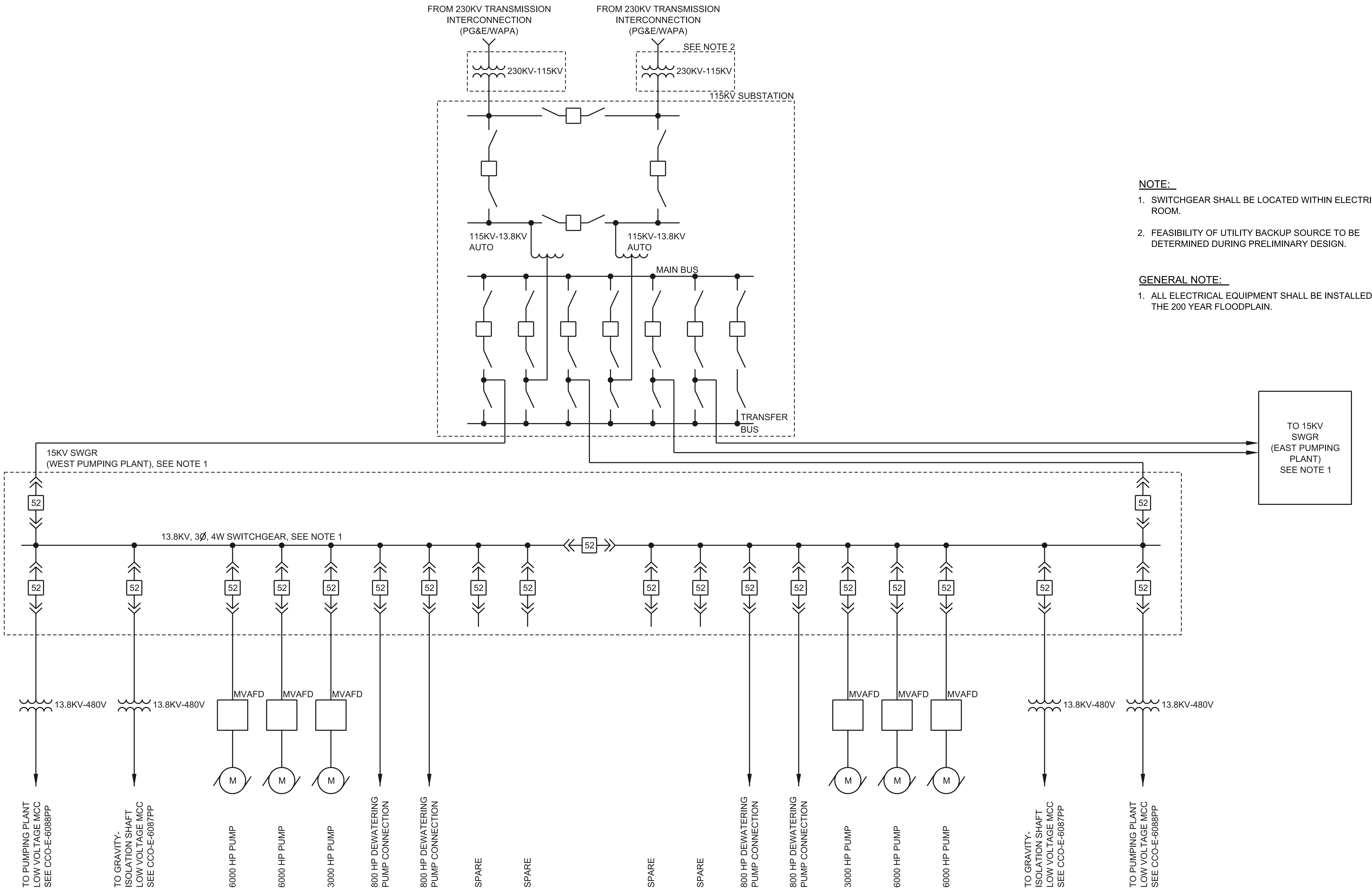
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY
PUMPING PLANT SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - M - 3078PP
REV 0
SEQUENCE NO. 78 OF 96



NOTE:

1. SWITCHGEAR SHALL BE LOCATED WITHIN ELECTRICAL ROOM.
2. FEASIBILITY OF UTILITY BACKUP SOURCE TO BE DETERMINED DURING PRELIMINARY DESIGN.

GENERAL NOTE:

1. ALL ELECTRICAL EQUIPMENT SHALL BE INSTALLED ABOVE THE 200 YEAR FLOODPLAIN.

FINAL DRAFT
DATE: APRIL 1, 2015

PUMPING PLANT MEDIUM VOLTAGE POWER DISTRIBUTION SINGLE LINE DIAGRAM (TYPICAL OF 2)

NTS

VERIFY SCALE
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0 1"

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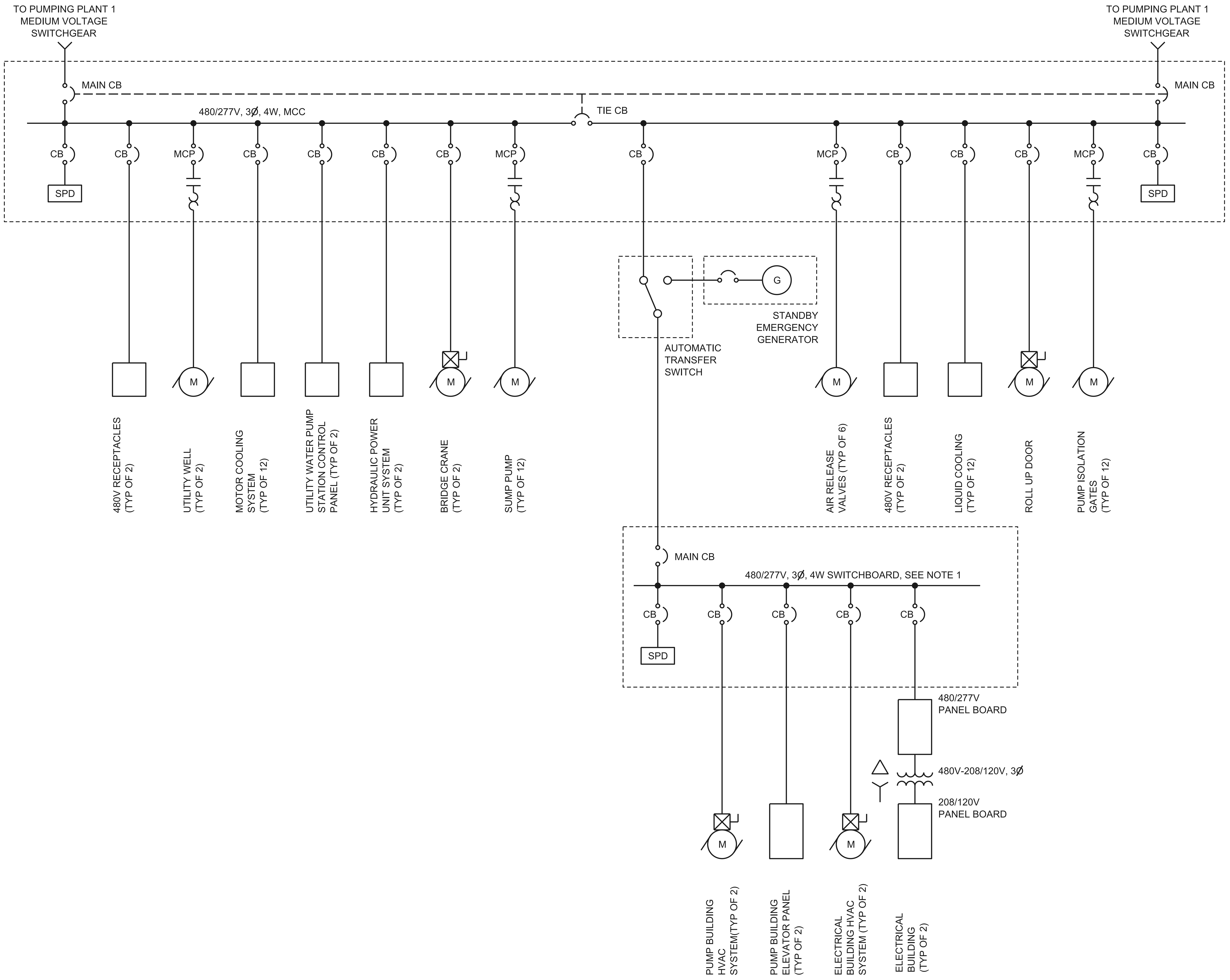
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY PUMPING PLANT
OVERALL SINGLE LINE DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - E - 6079PP
REV 0
SEQUENCE NO. 79 OF 96



PUMPING PLANT MCC SINGLE LINE DIAGRAM (TYPICAL OF 2)

NTS

NOTE:

1. MCC AND SWITCHBOARD SHALL BE LOCATED WITHIN ELECTRICAL ROOM.

GENERAL NOTE:

1. ALL ELECTRICAL EQUIPMENT SHALL BE INSTALLED ABOVE THE 200 YEAR FLOODPLAIN.

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CLIFTON COURT FOREBAY PUMPING PLANT
SINGLE LINE DIAGRAM

PROJECT NO. 240010
SHEET NO. CCO - E - 6080PP
REV 0
SEQUENCE NO. 80 OF 96

FINAL DRAFT
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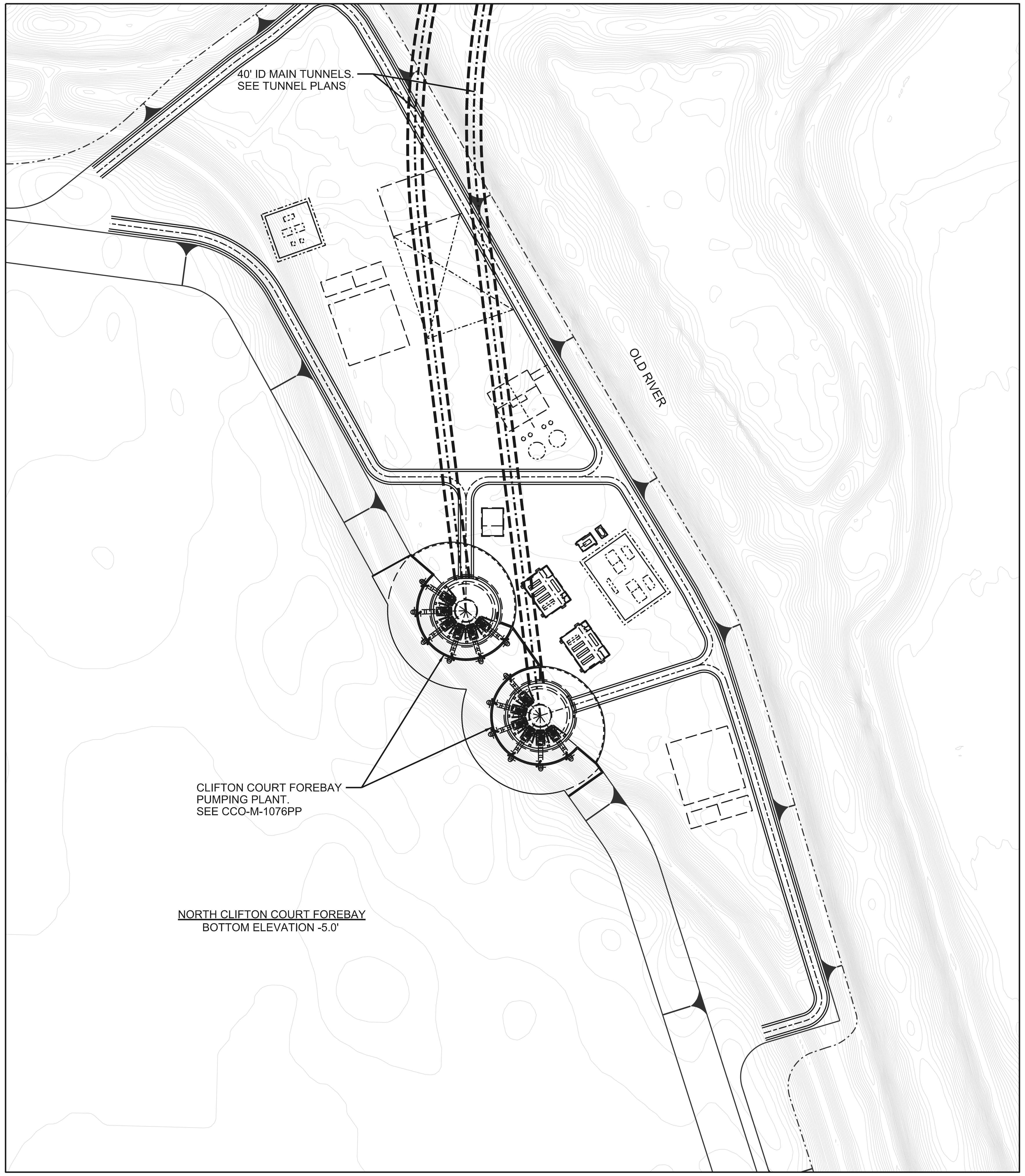
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MODIFIED CLIFTON COURT FOREBAY
OVERALL SITE PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1082FB
REV 0
SEQUENCE NO. 82 OF 96



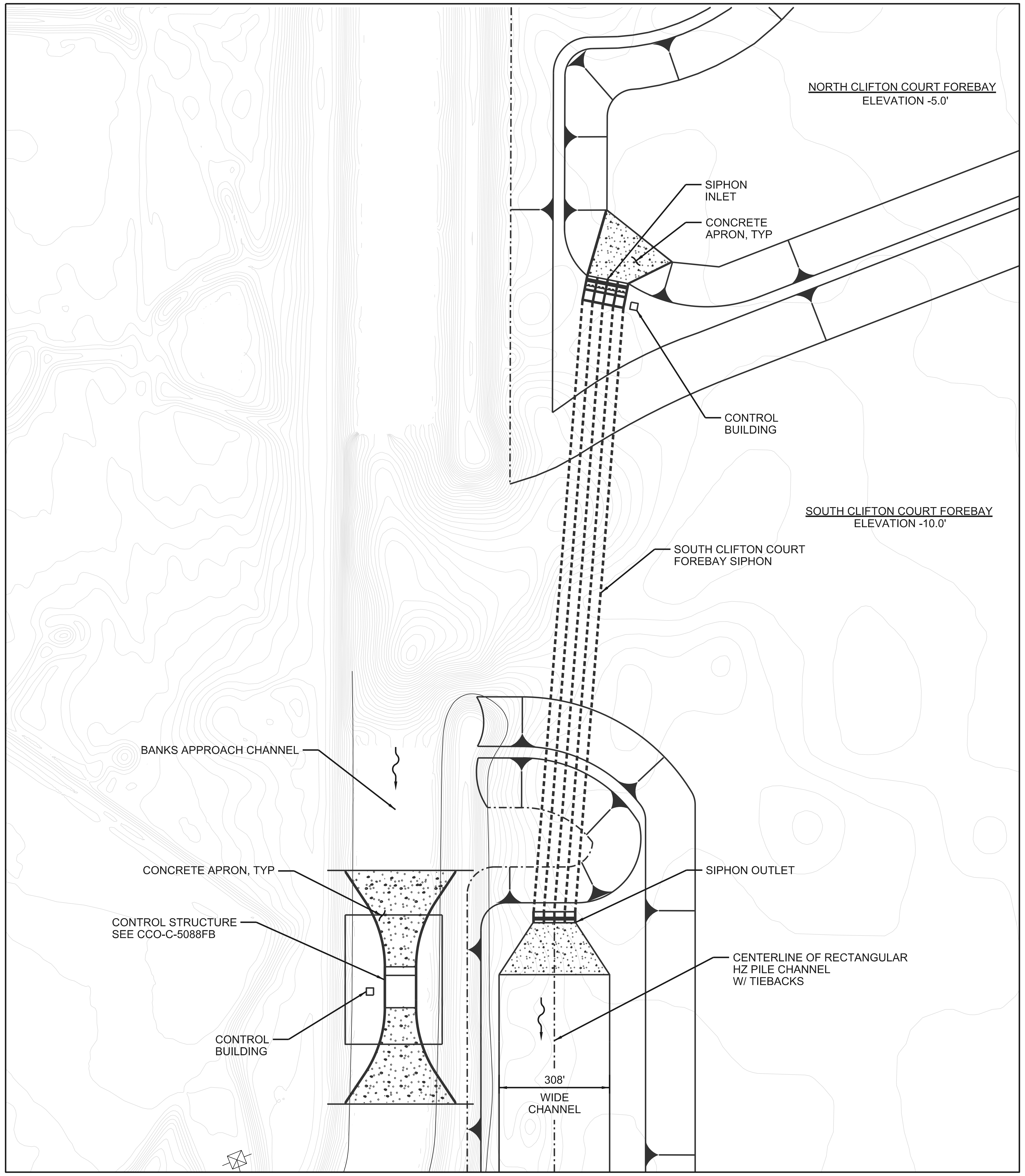
TUNNEL TRANSITION TO CLIFTON COURT FOREBAY PUMPING STATION

PARTIAL PLAN

1"=200'

N

0 200 400 600
Scale In Feet



OUTLETS FROM NORTH & SOUTH CLIFTON COURT FOREBAYS TO BANKS & JONES PUMPING PLANTS

PARTIAL PLAN

1"=200'

N

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0 1"

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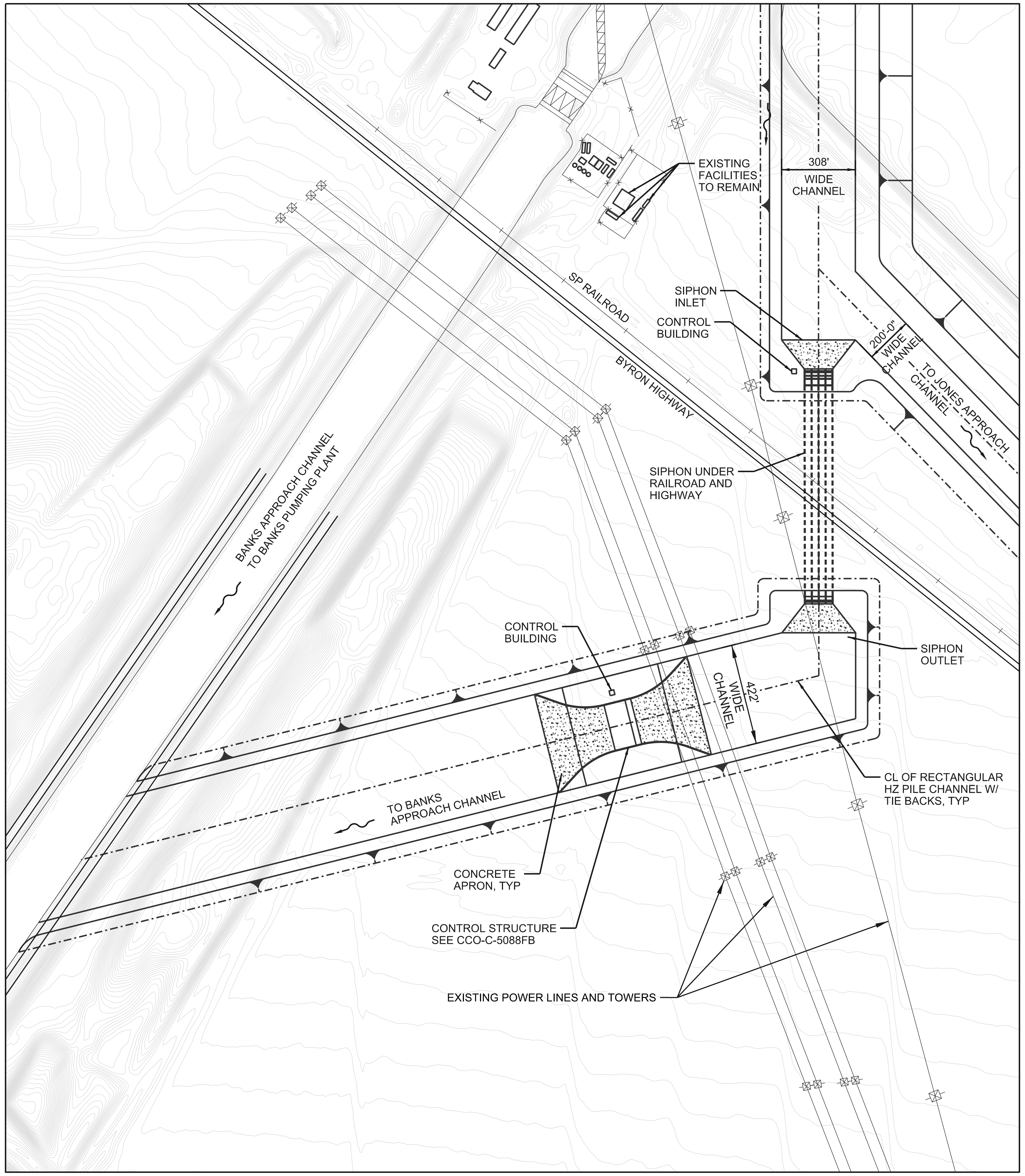
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MODIFIED CLIFTON COURT FOREBAY INLET AND OUTLETS - SITE PLANS

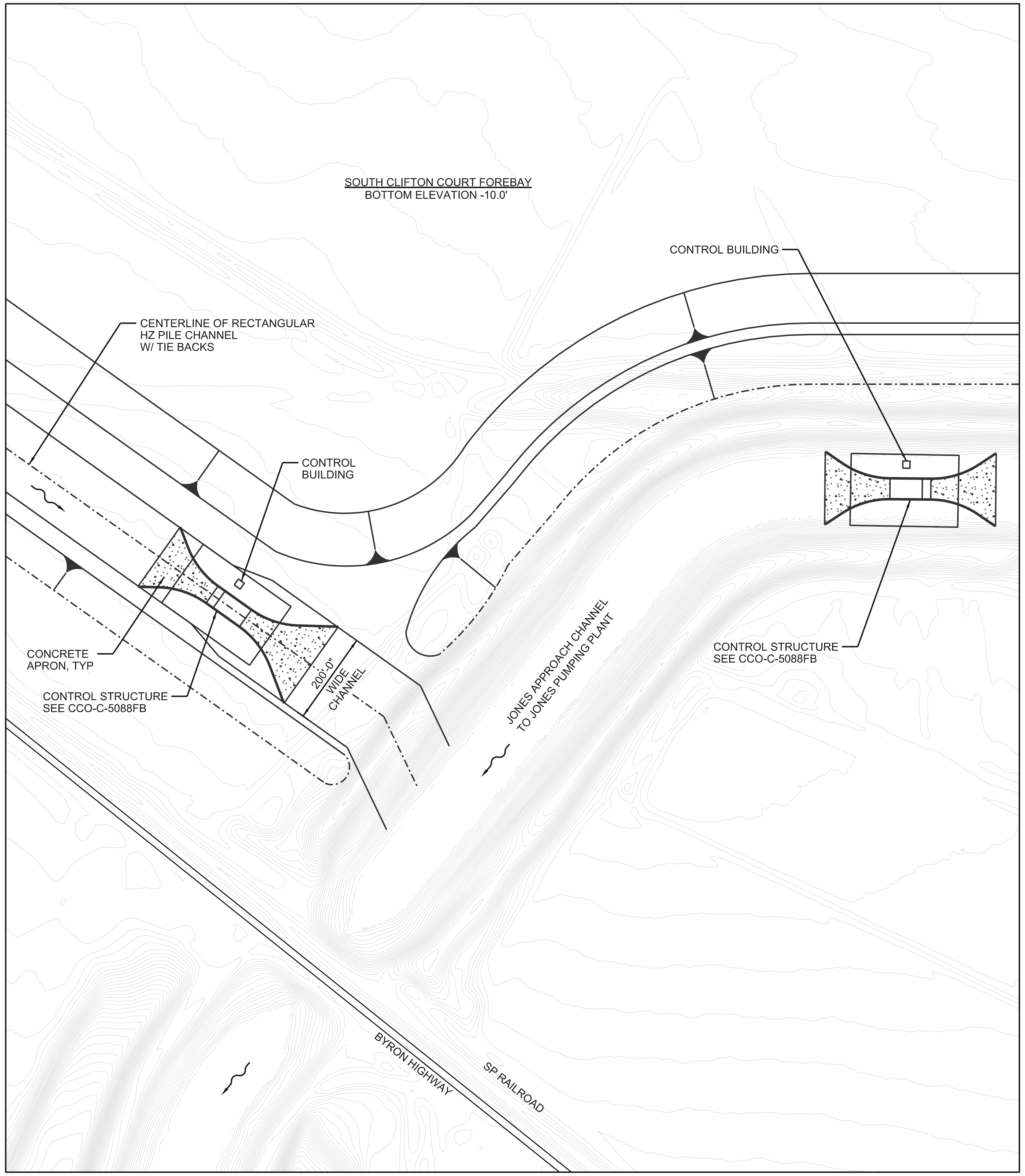
PROJECT NO.
240010
SHEET NO.
CCO - C - 1083FB
REV
0
SEQUENCE NO.
83 OF 96



APPROACH TO BANKS PUMPING PLANT

PARTIAL PLAN
1"=300'

N



APPROACH TO JONES PUMPING PLANT

PARTIAL PLAN
1"=200'

N

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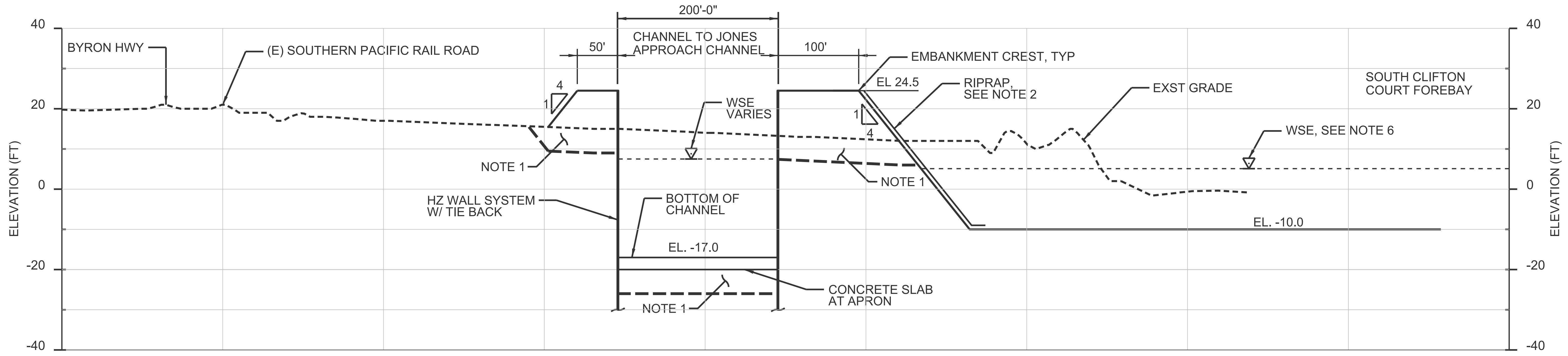
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CHANNEL APPROACHES TO PUMPING PLANTS
SITE PLANS

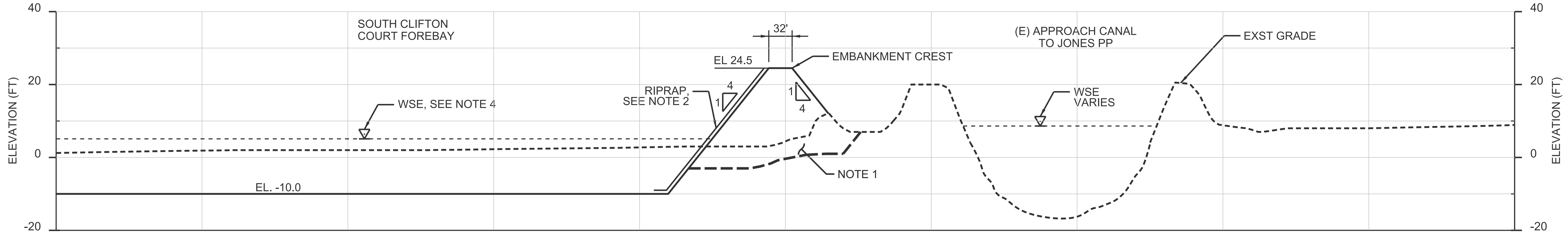
PROJECT NO. 240010
SHEET NO. CCO - C - 1084FB
REV 0
SEQUENCE NO. 84 OF 96



(A) SECTION

SCALE AS SHOWN

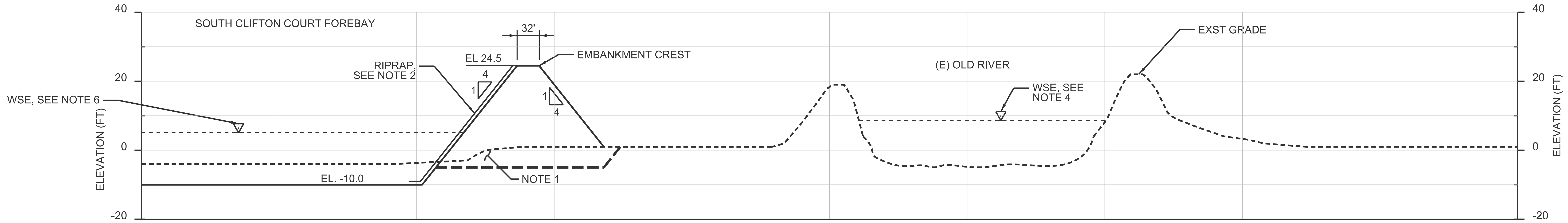
CCO-C-1089FB



(B) SECTION

SCALE AS SHOWN

CCO-C-1089FB



(C) SECTION

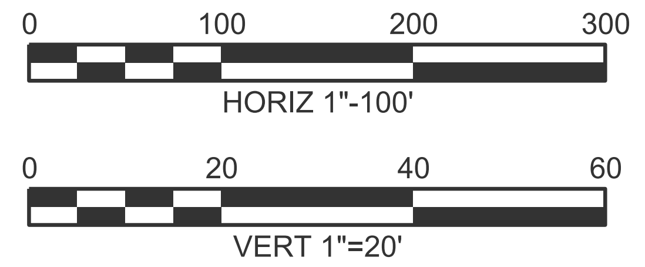
SCALE AS SHOWN

CCO-C-1089FB

NOTES:

1. EXCAVATION OF UNSUITABLE FOUNDATION MATERIAL WILL VARY. AN AVERAGE OF 6 FEET IS ASSUMED.
2. PLACE RIPRAP OVER FILTER LAYER.
3. FOREBAY FILLS CONSTRUCTED FROM COMPACTED EMBANKMENT MATERIAL.
4. OUTSIDE SLOPE AT ORIGINAL GROUND ELEVATION TO INCLUDE TOE DRAIN STARTING 38' FROM CREST AND EXTENDING TO TOE OF SLOPE.
5. CONSTRUCT SLURRY TRENCH CUTOFF TO EL. -50.0
6. SEE WATER SURFACE ELEVATION TABLE FOR OPERATING WSE LEVELS

OPERATION SCENARIO	OPERATED FOREBAY	WATER SURFACE ELEVATIONS												
		SWP						CVP			SWP & CVP			
		HISTORICAL MINIMUM WSE	HISTORICAL MAXIMUM WSE	TYPICAL MINIMUM OPERATING WSE	TYPICAL MAXIMUM OPERATING WSE	NORMAL MAXIMUM OPERATING WSE	ABSOLUTE MAXIMUM OPERATING WSE	DESIGN MINIMUM OPERATING WSE	DESIGN MAXIMUM OPERATING WSE	LOWEST WSE FOR PUMP OPERATION	MINIMUM WSE	MAXIMUM WSE	PREFERRED MINIMUM OPERATING WSE	PREFERRED MAXIMUM OPERATING WSE
ISOLATED SOUTH DELTA OPERATION	SOUTH CCF	1.1	12.6	1.1	3.1	8.1	9.1	-1.43	9.57	-2.43	-	-	-	-
ISOLATED NORTH DELTA OPERATION	NORTH CCF	-	-	-	-	-	-	-	-	-	1.1	11.07	3.1	7.07
DUAL OPERATON	NORTH CCF	-	-	-	-	-	-	-	-	-	5.6	14.7	-	-
	SOUTH CCF	1.1	12.6	1.1	3.1	5.1	-	-	-	-	-	-	-	-
	OLD RIVER	-	-	-	-	-	-	-1.43	5.1	-2.43	-	-	-	-



FINAL DRAFT
DATE: APRIL 1, 2015

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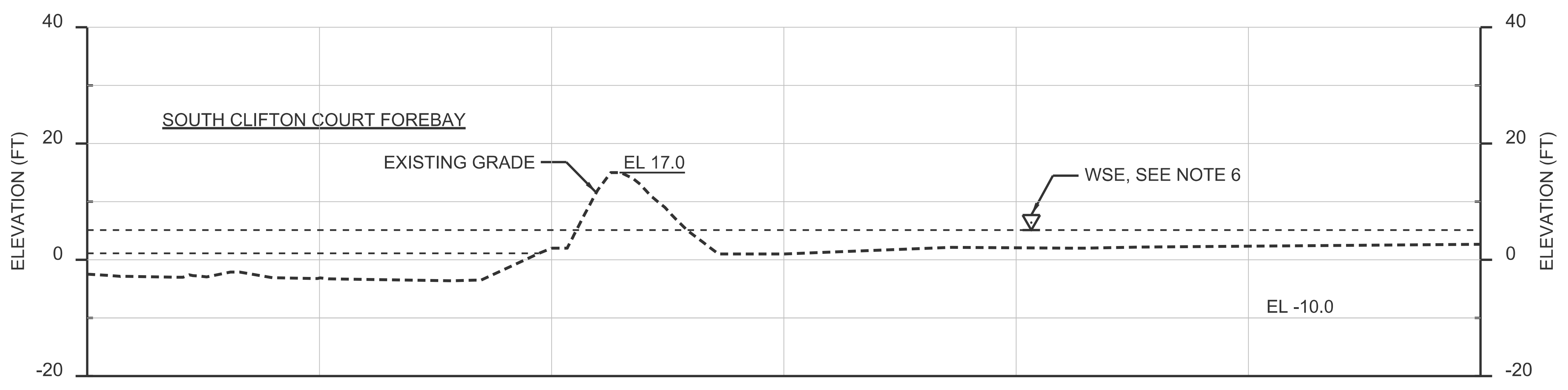
CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

MODIFIED CLIFTON COURT FOREBAY EMBANKMENT SECTIONS I

PROJECT NO. 240010
SHEET NO. CCO - C - 3085FB
REV 0
SEQUENCE NO. 85 OF 96

NOTES:

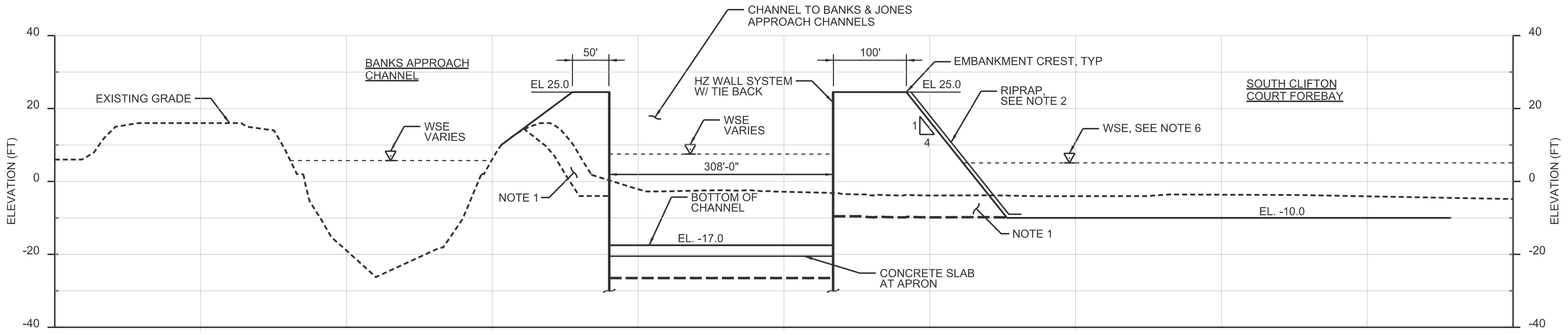
1. EXCAVATION OF UNSUITABLE FOUNDATION MATERIAL WILL VARY. AN AVERAGE OF 6 FEET IS ASSUMED.
2. PLACE RIPRAP OVER FILTER LAYER.
3. FOREBAY FILLS CONSTRUCTED FROM COMPACTED EMBANKMENT MATERIAL.
4. OUTSIDE SLOPE AT ORIGINAL GROUND ELEVATION TO INCLUDE TOE DRAIN STARTING 38' FROM CREST AND EXTENDING TO TOE OF SLOPE.
5. CONSTRUCT SLURRY TRENCH CUTOFF TO EL -50.0.
6. SEE WATER SURFACE ELEVATION TABLE ON CCO-C-3085FB FOR WSE OPERATING LEVELS.



D SECTION

SCALE AS SHOWN

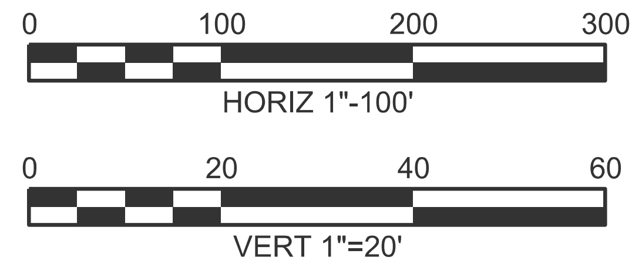
CCO-C-1082FB



E SECTION

SCALE AS SHOWN

CCO-C-1082FB



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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

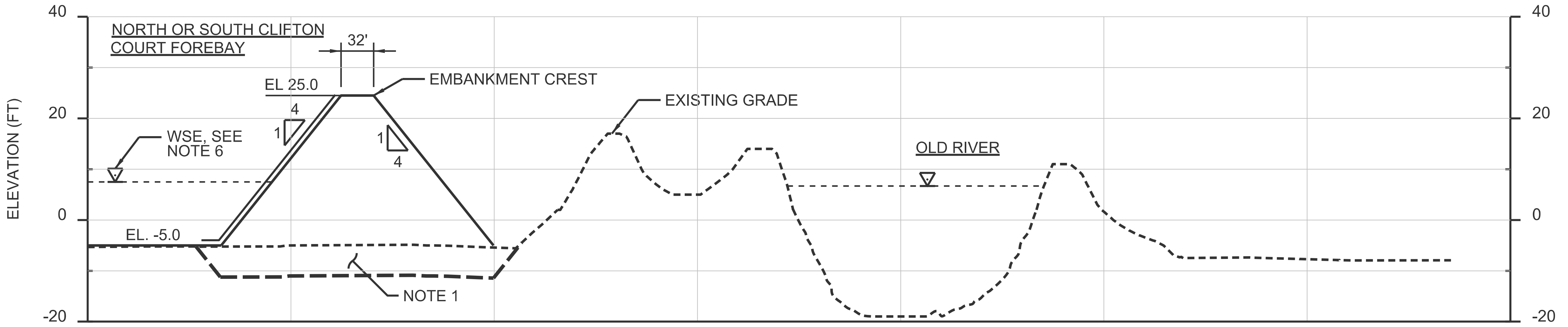
MODIFIED CLIFTON COURT FOREBAY EMBANKMENT SECTIONS II

PROJECT NO. 240010

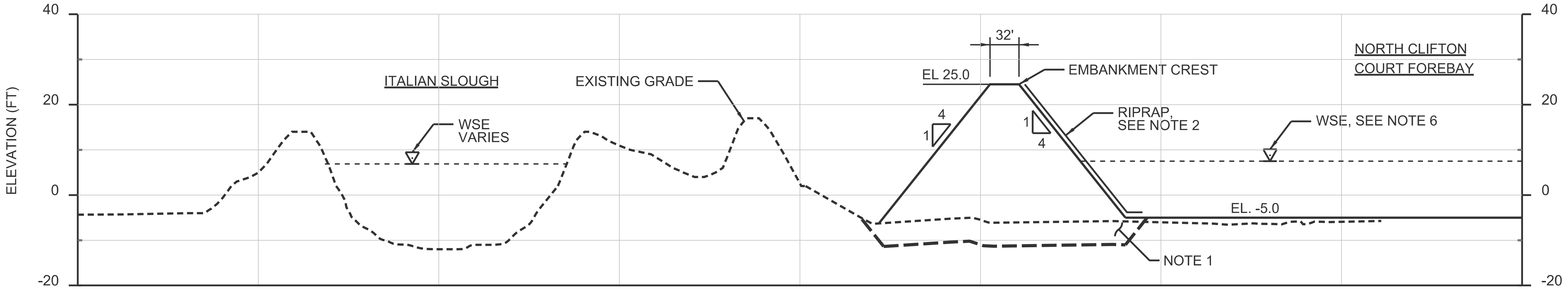
SHEET NO. CCO - C - 3086FB

REV 0

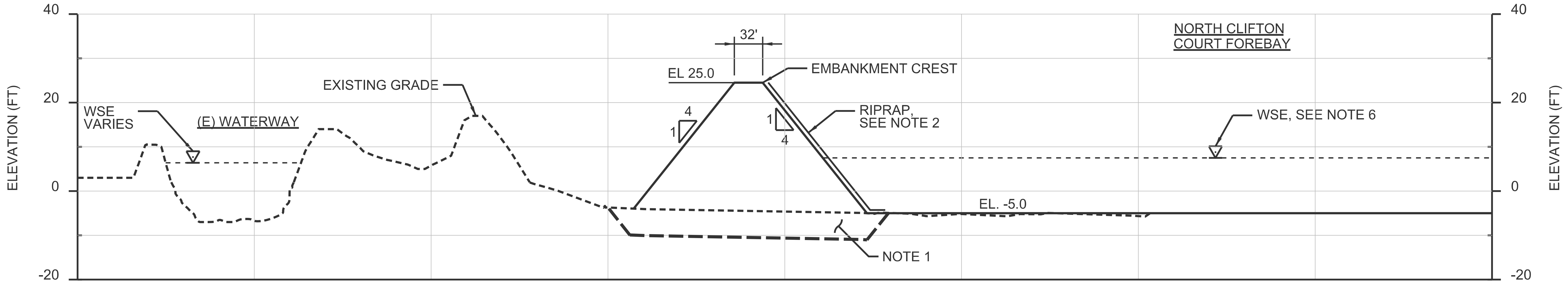
SEQUENCE NO. 86 OF 96



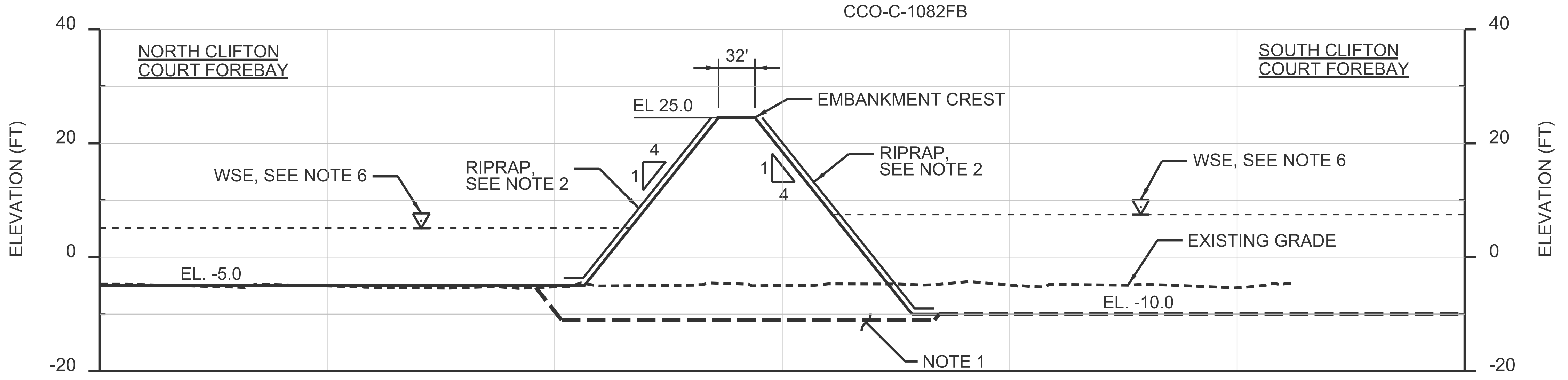
F SECTION
SCALE AS SHOWN
CCO-C-1082FB



G SECTION
SCALE AS SHOWN
CCO-C-1082FB



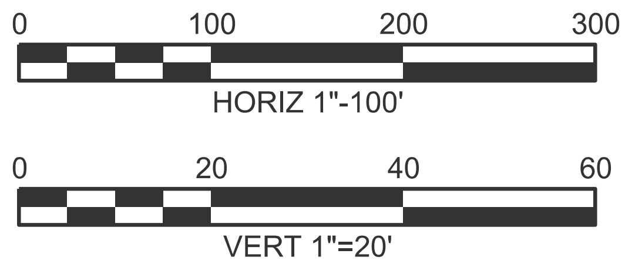
H SECTION
SCALE AS SHOWN
CCO-C-1082FB



I SECTION
SCALE AS SHOWN
CCO-C-1082FB

NOTES:

1. EXCAVATION OF UNSUITABLE FOUNDATION MATERIAL WILL VARY. AN AVERAGE OF 6 FEET IS ASSUMED.
2. PLACE RIPRAP OVER FILTER LAYER.
3. FOREBAY FILLS CONSTRUCTED FROM COMPACTED EMBANKMENT MATERIAL.
4. OUTSIDE SLOPE AT ORIGINAL GROUND ELEVATION TO INCLUDE TOE DRAIN STARTING 38' FROM CREST AND EXTENDING TO TOE OF SLOPE.
5. CONSTRUCT SLURRY TRENCH CUTOFF TO EL -50.0.
6. SEE WATER SURFACE ELEVATION TABLE ON CCO-C-3085FB FOR WSE OPERATING LEVELS.



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0 1"

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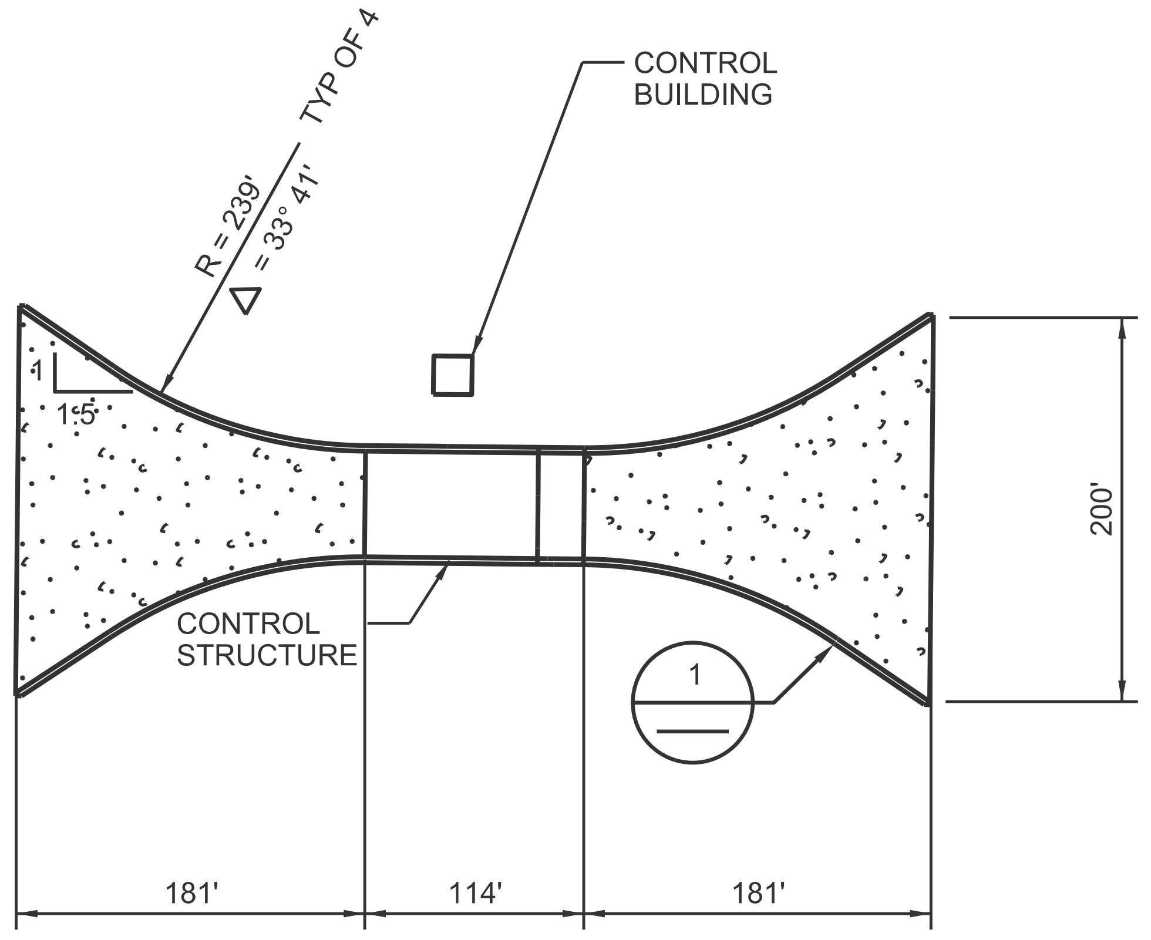
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

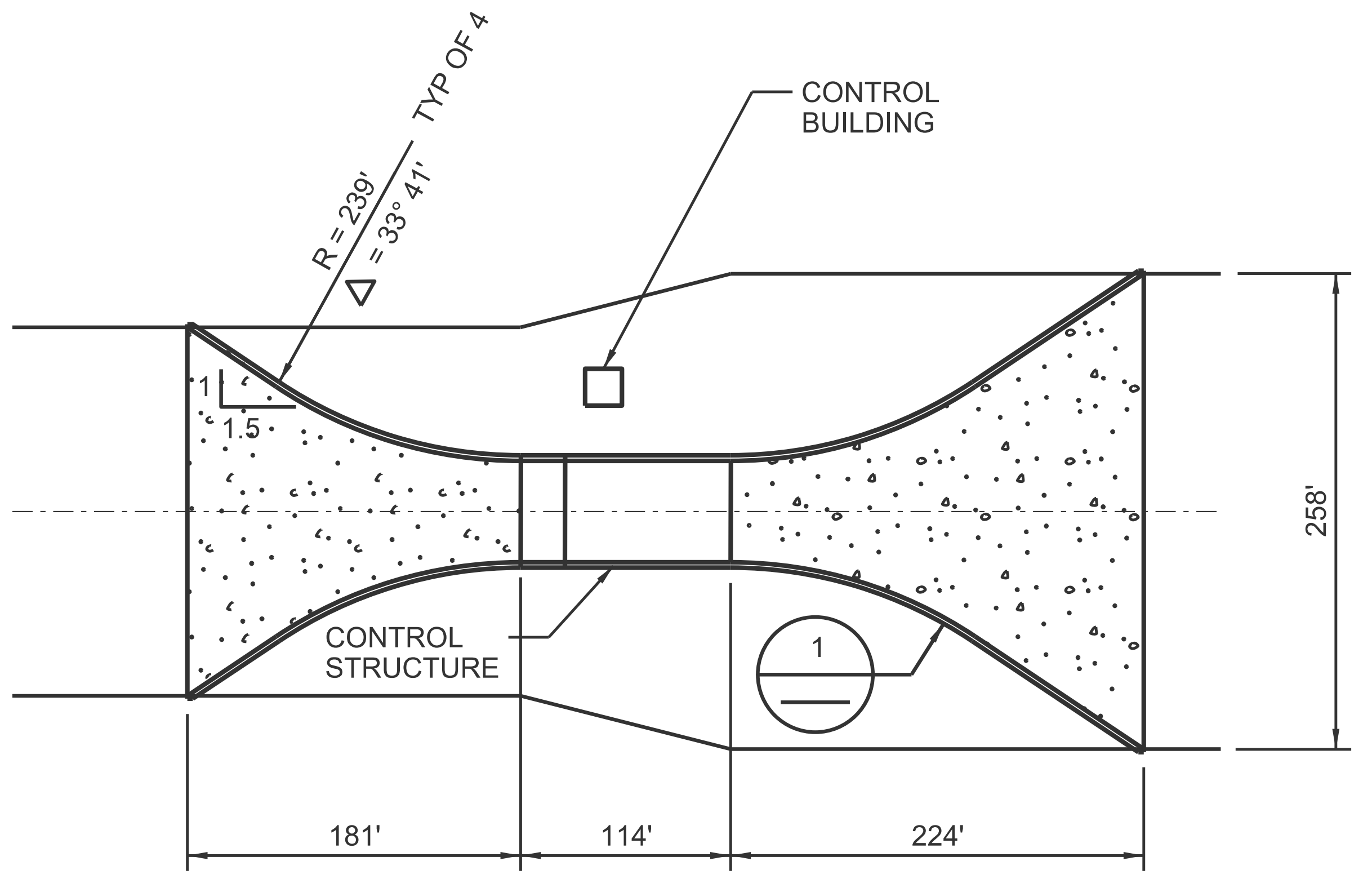
MODIFIED CLIFTON COURT FOREBAY
EMBANKMENT SECTIONS III

PROJECT NO. 240010
SHEET NO. CCO - C - 3087FB
REV 0
SEQUENCE NO. 87 OF 96



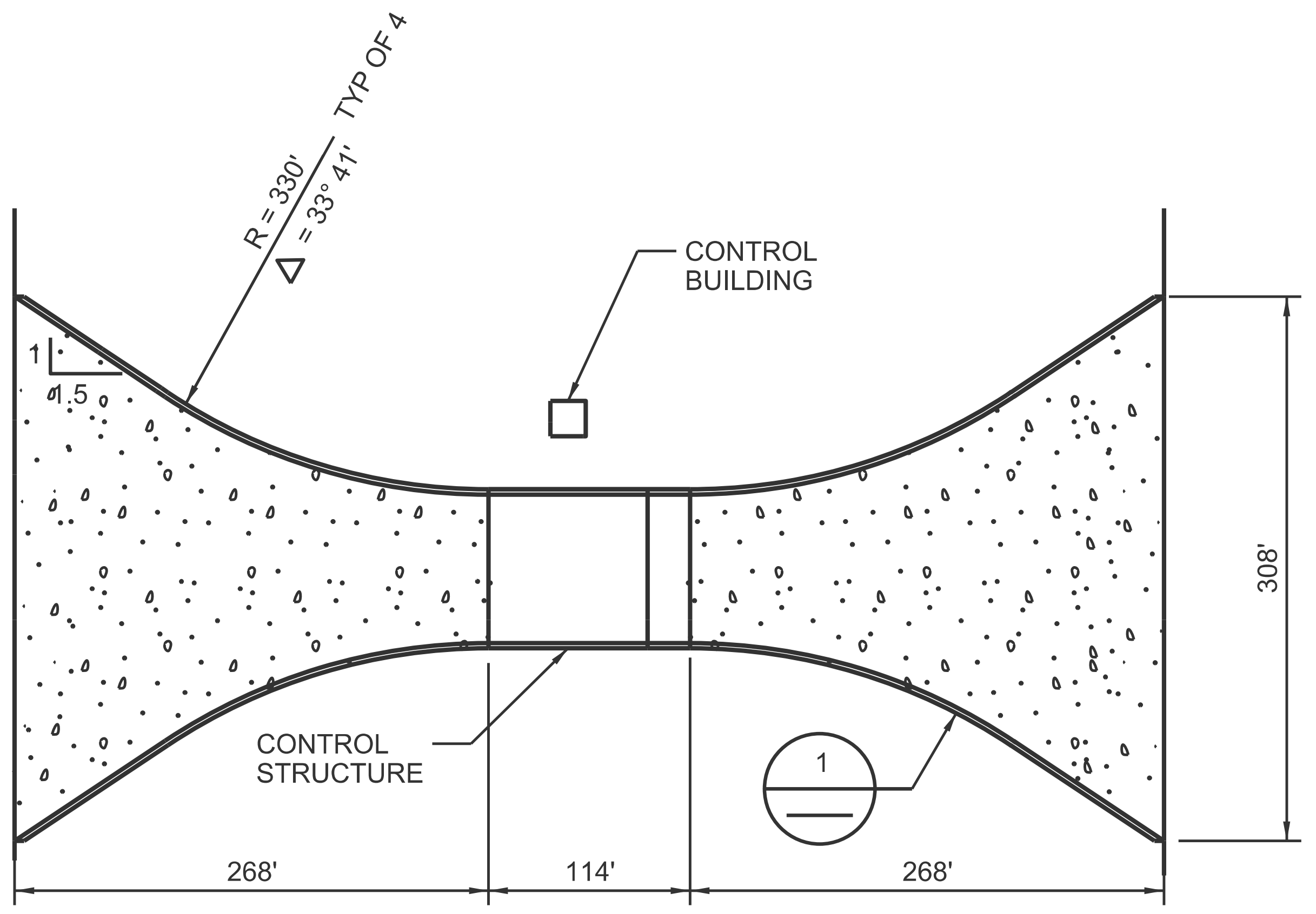
@ JONES APPROACH CHANNEL

PLAN
NTS



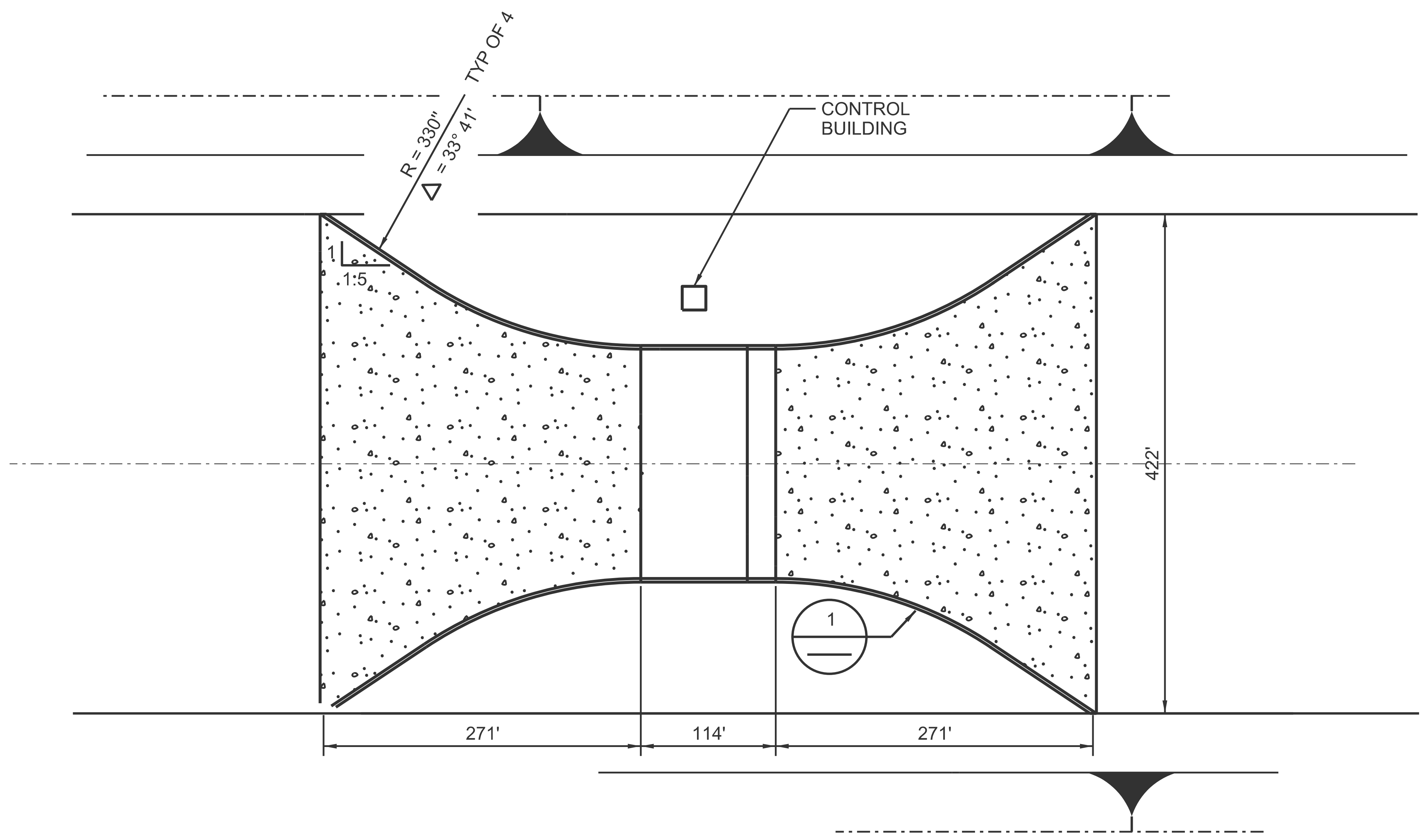
@ CHANNEL FROM NCCF TO JONES APPROACH CHANNEL

PLAN
NTS



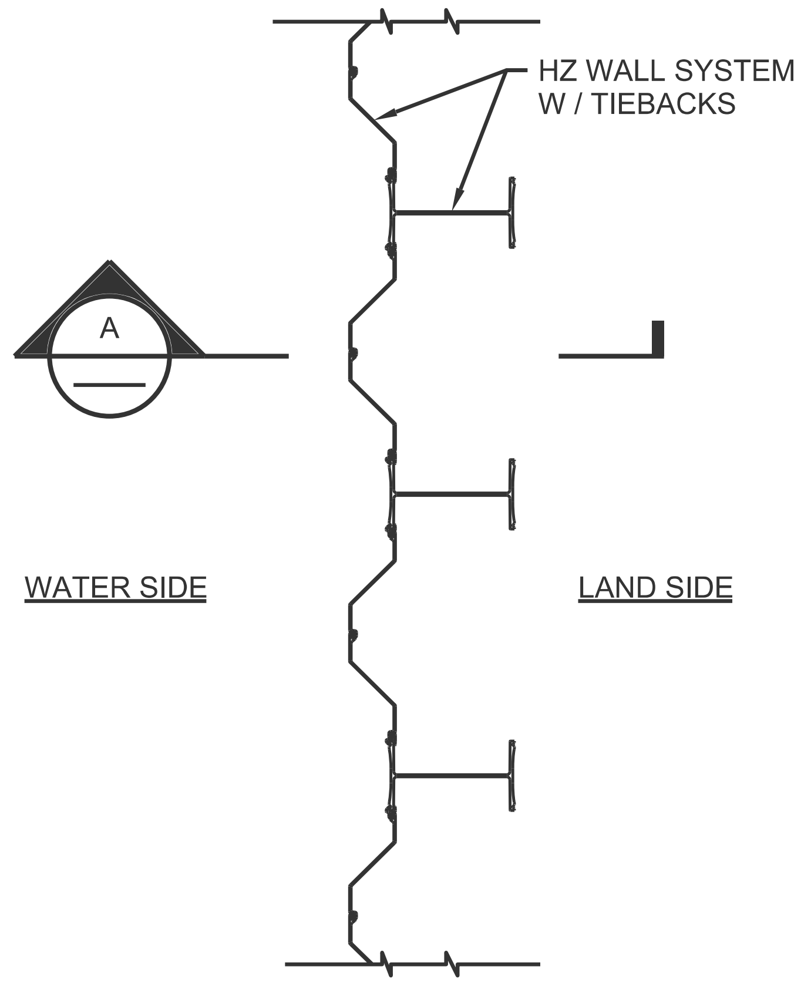
@ BANKS APPROACH CHANNEL

PLAN
NTS

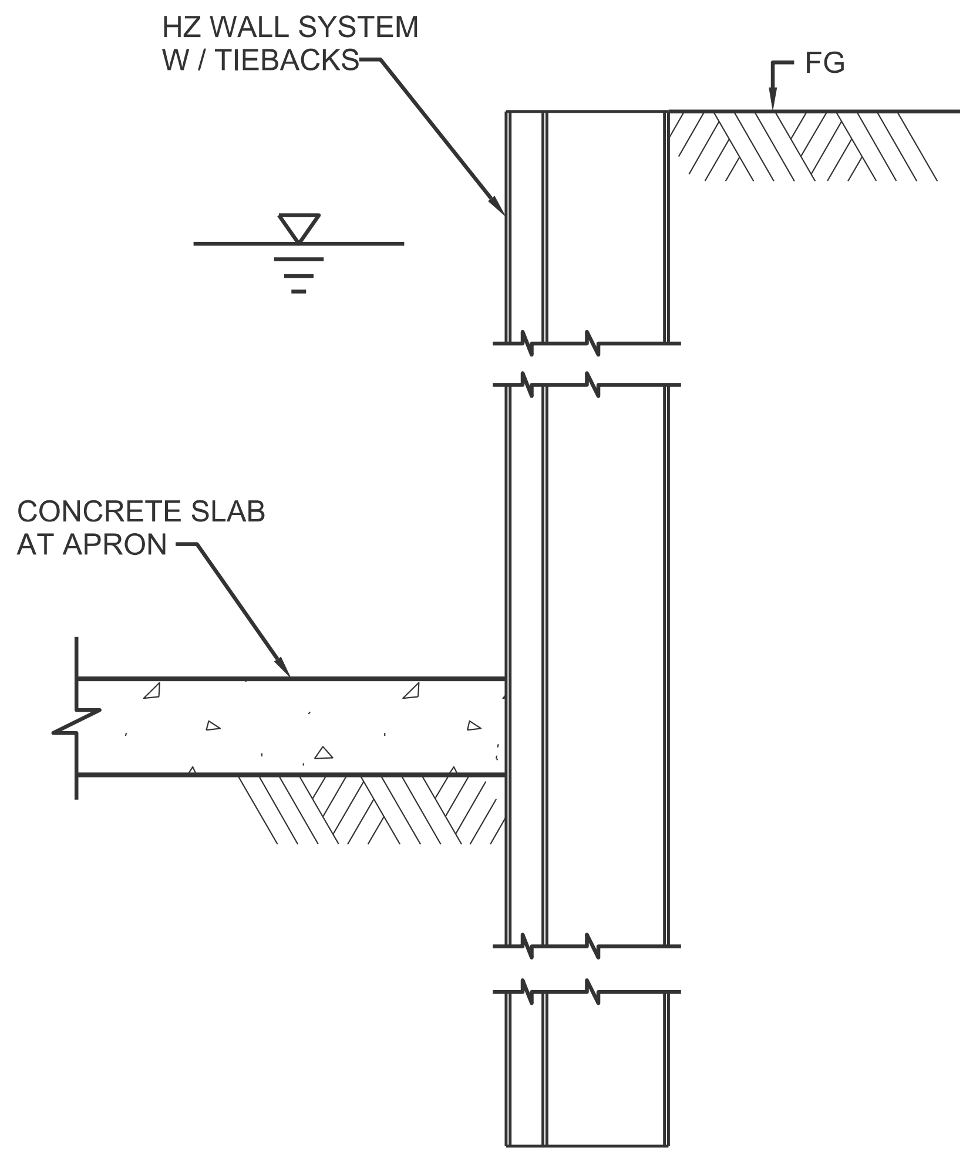


@ CHANNEL FROM NCCF TO BANKS APPROACH CHANNEL

PLAN
NTS



① **DETAIL**
1/4"=1'-0"



Ⓐ **SECTION**
1/4"=1'-0"

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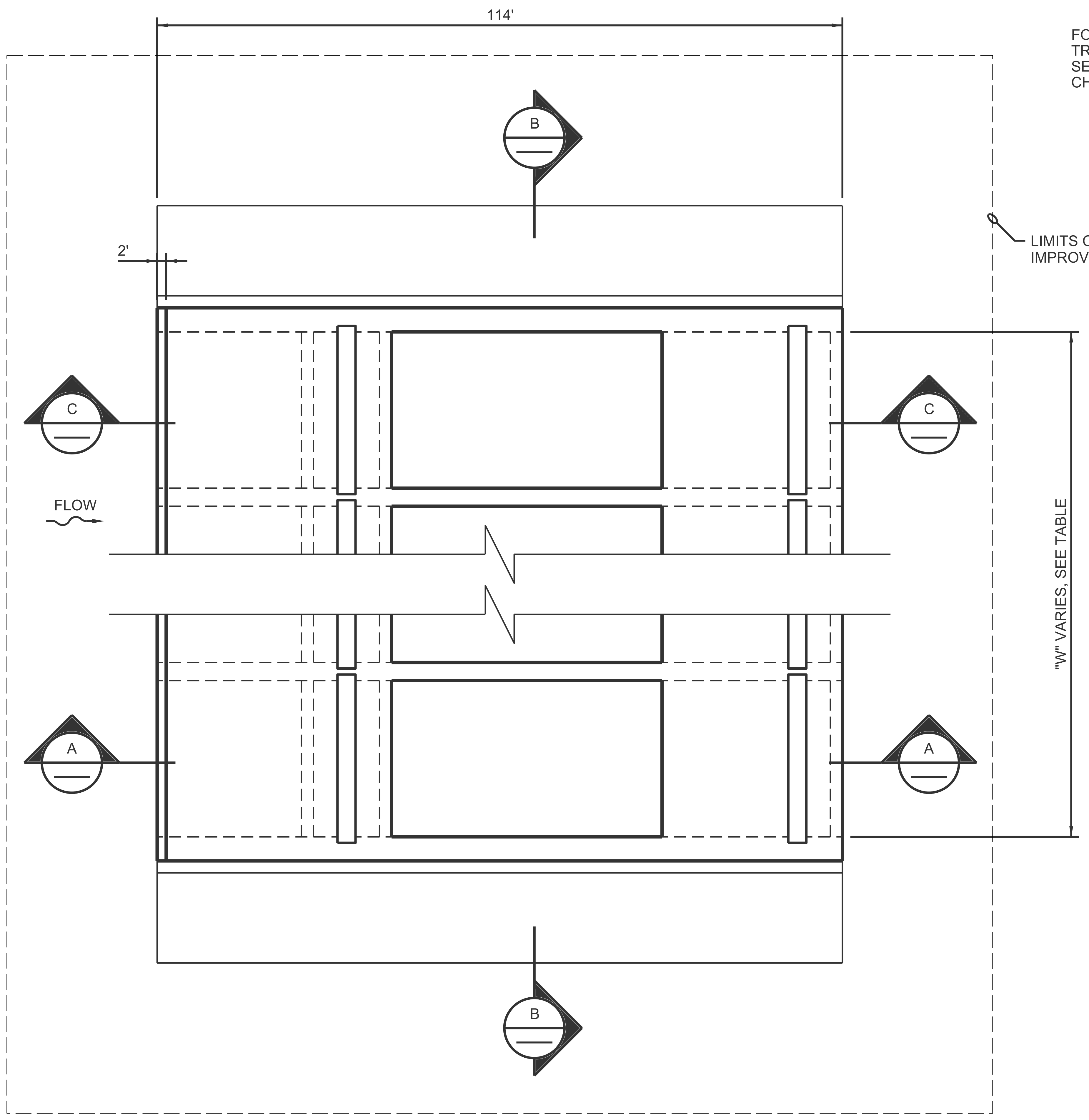
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CHANNEL CONTROL STRUCTURES
PLANS, SECTION, AND DETAIL

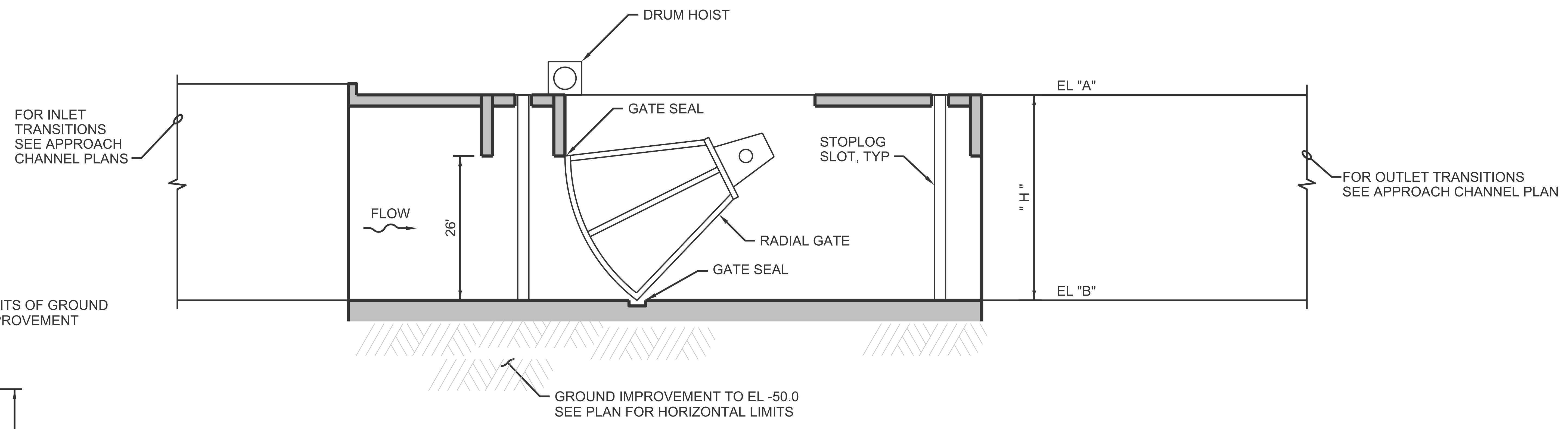
PROJECT NO. 240010
SHEET NO. CCO - C - 4088FB
REV 0
SEQUENCE NO. 88 OF 96



PLAN

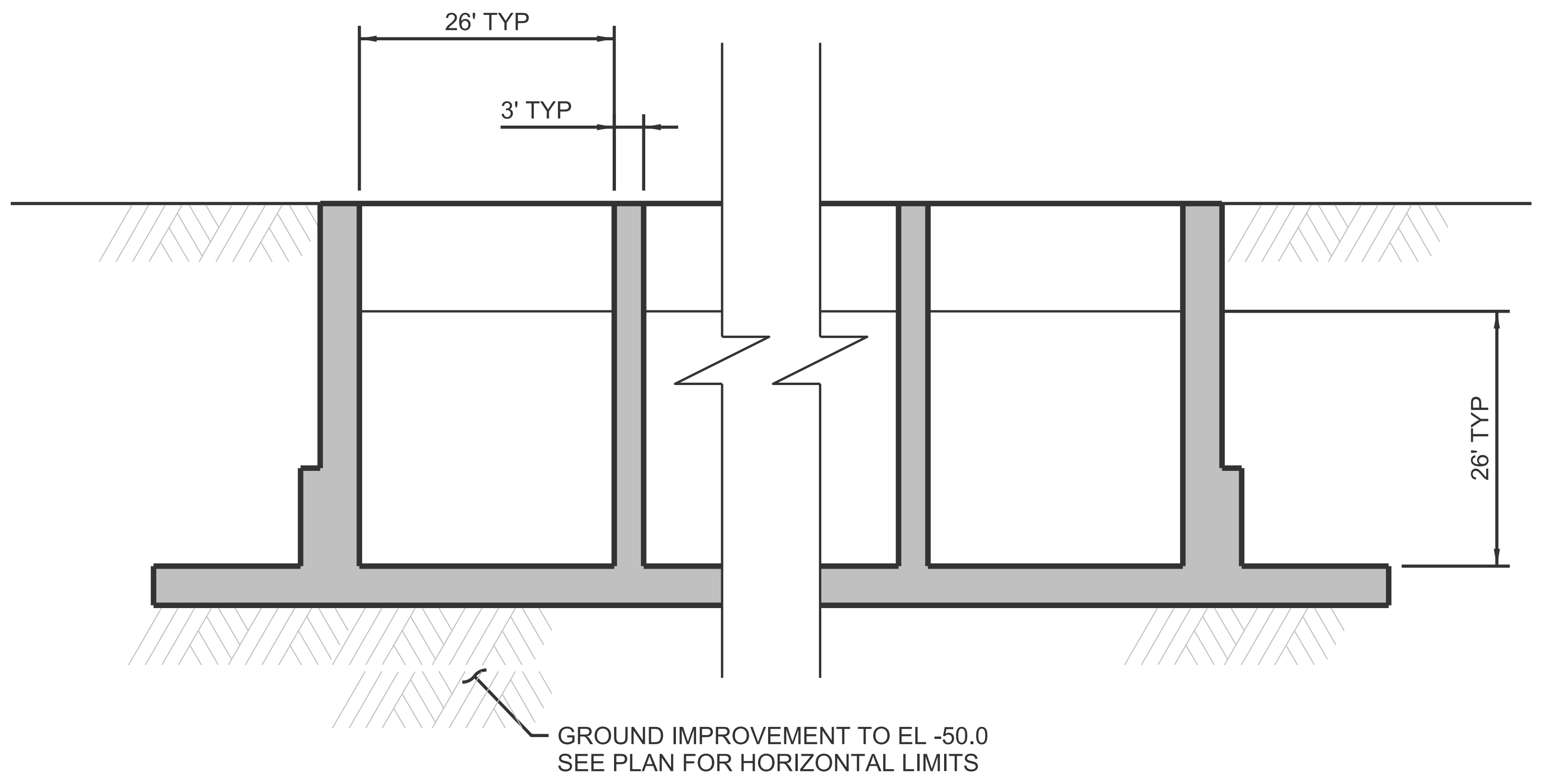
1/16"=1'-0"

ITEM	BANKS APPROACH CHANNEL	NCCF TO BANKS APPROACH CHANNEL	JONES APPROACH CHANNEL	NCCF TO JONES APPROACH CHANNEL
Q (CFS)	10,600	10,600	4,600	4,600
NUMBER OF GATES	3	3	2	2
EL "A"	20.0	20.0	20.0	20.0
EL "B"	-27.9	-27.9	-17.0	-17.0
"H"	47.9	47.9	37.0	37.0
"W"	84.0	200.0	55.0	113.0



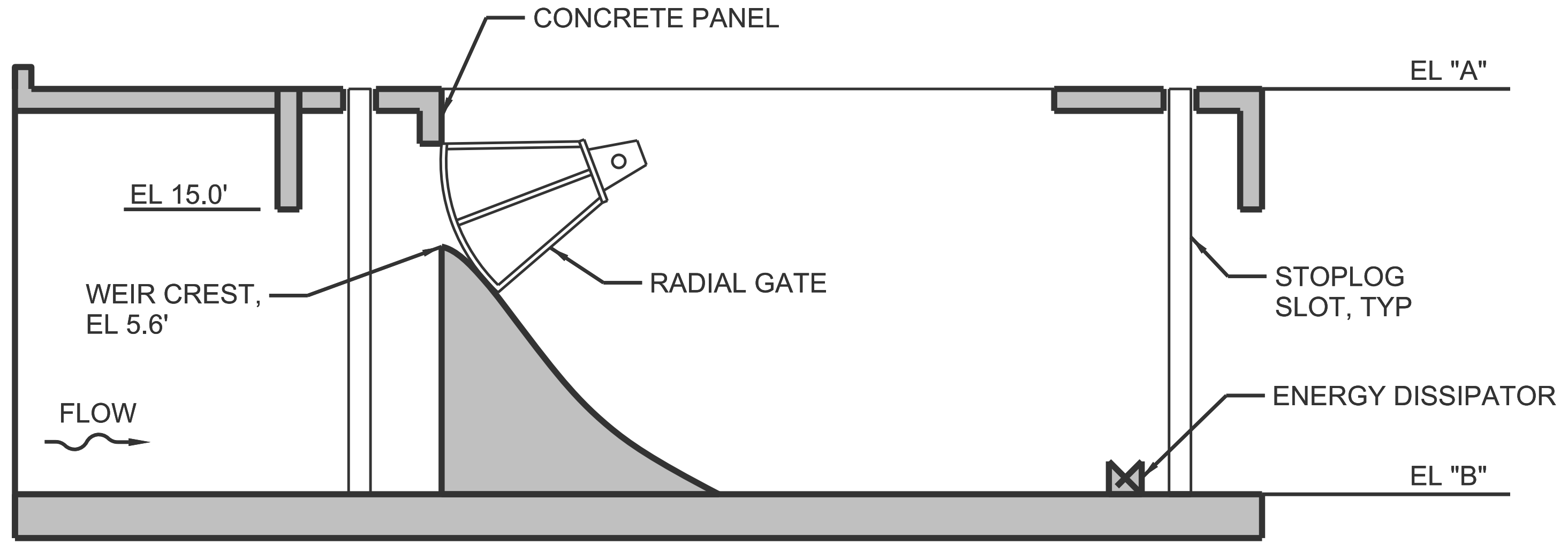
A SECTION

1/16"=1'-0"



B SECTION

1/16"=1'-0"



C SECTION

1/16"=1'-0"

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0 1"

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DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CHANNEL CONTROL STRUCTURES
PLAN AND SECTIONS

PROJECT NO. 240010
SHEET NO. CCO - C - 5089FB
REV 0
SEQUENCE NO. 89 OF 96

PLAN

1/16"=1'-0"

N

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0 1 2 3
1/16"=1'-0"

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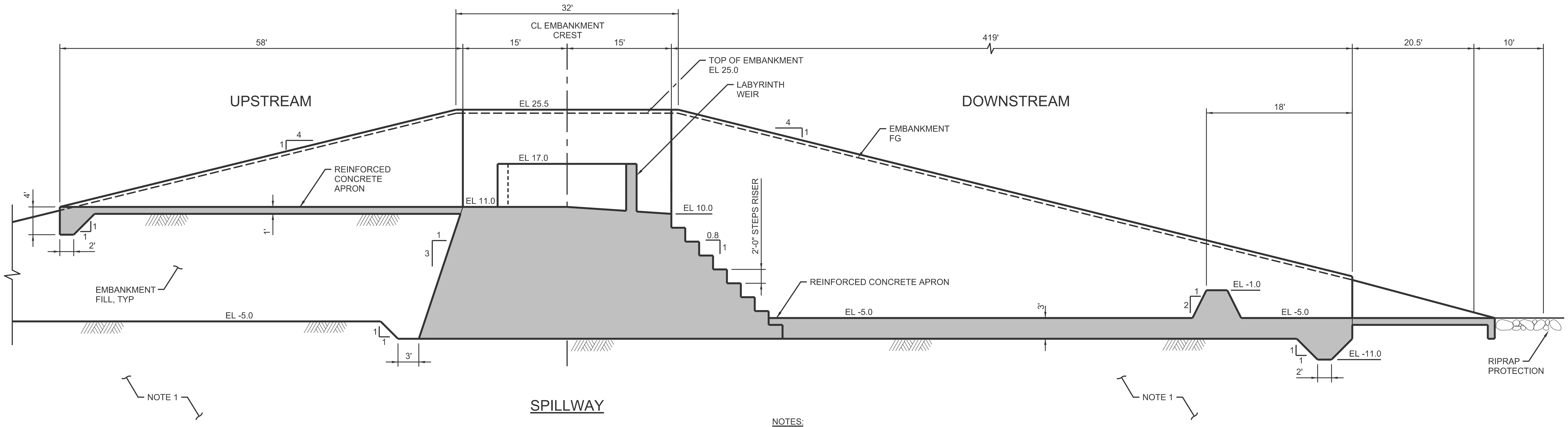
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH CLIFTON FOREBAY SPILLWAY PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 5090FB
REV 0
SEQUENCE NO. 90 OF 96

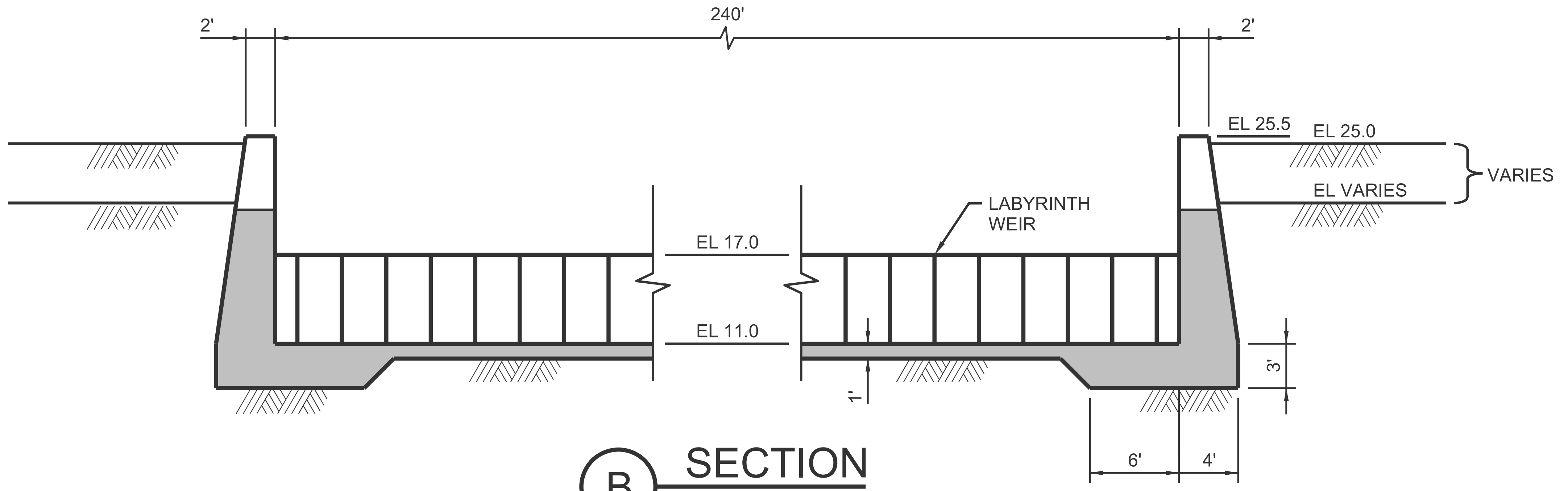


SPILLWAY

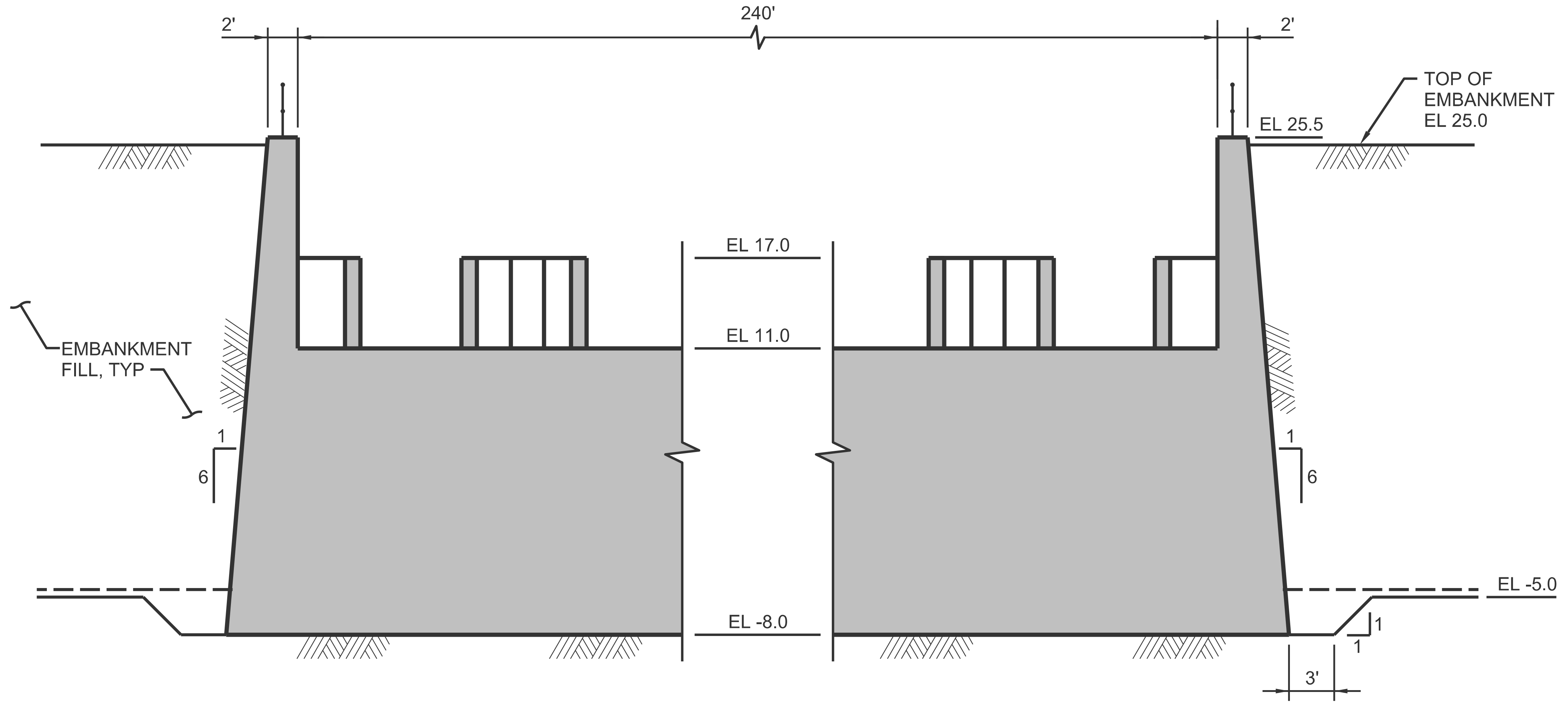
A SECTION
1/8"=1'-0"
CCO-C-4090FB

NOTES:

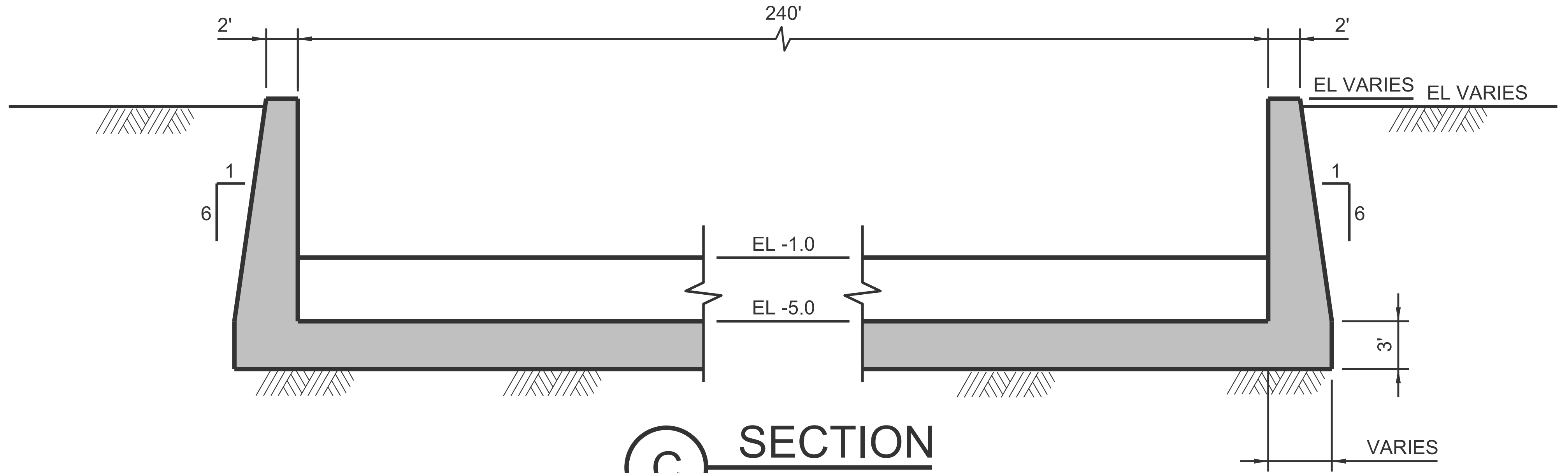
1. GROUND IMPROVEMENTS TO EL -50.0, SEE PLAN FOR HORIZONTAL LIMIT.
2. CONSIDER REINFORCED CONCRETE SPILLWAY DURING PRELIMINARY ENGINEERING.



B SECTION
1/8"=1'-0"
CCO-C-4090FB



D SECTION
1/8"=1'-0"
CCO-C-4090FB



C SECTION
1/8"=1'-0"
CCO-C-4090FB

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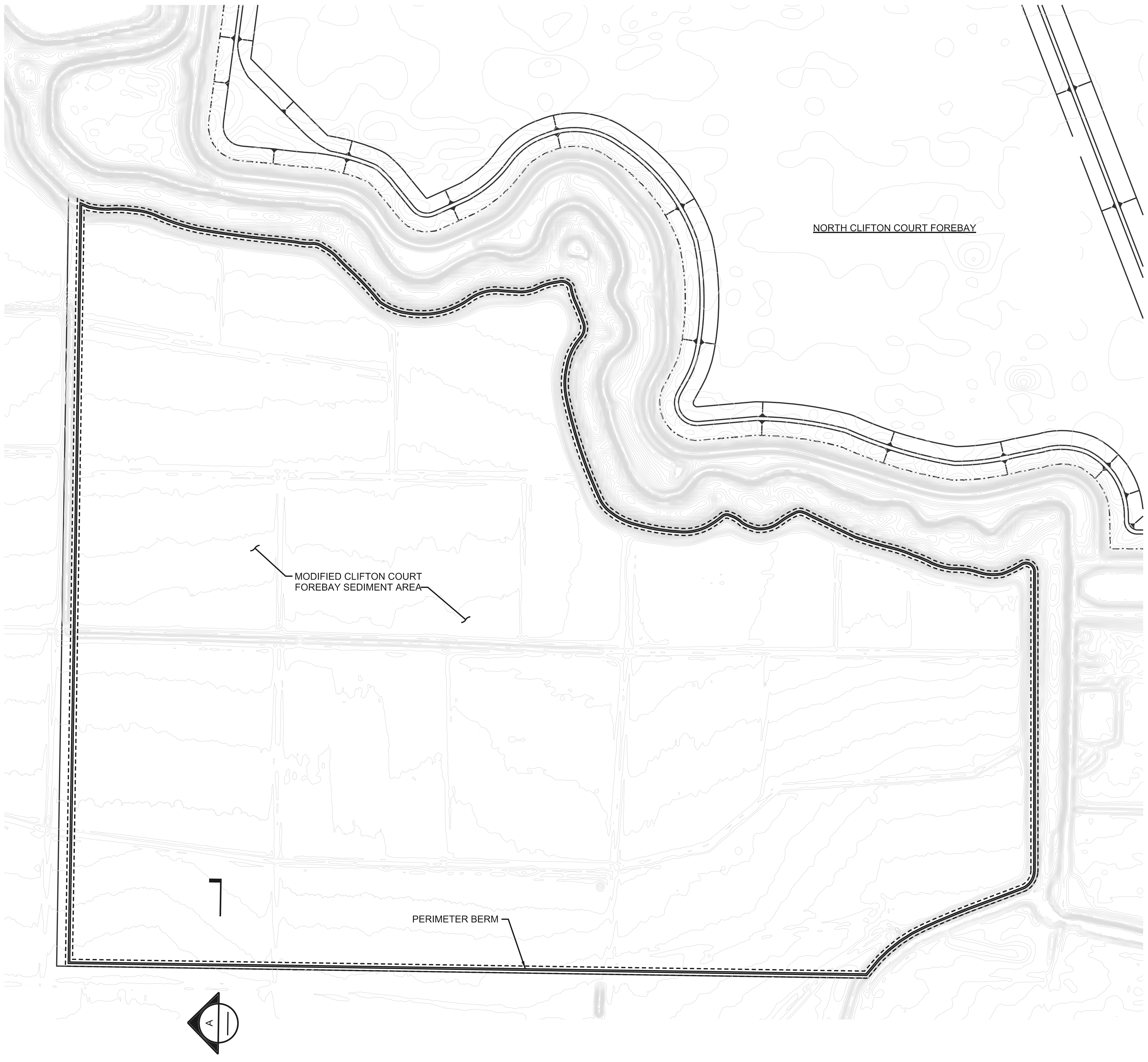
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

NORTH CLIFTON COURT FOREBAY
SPILLWAY SECTIONS

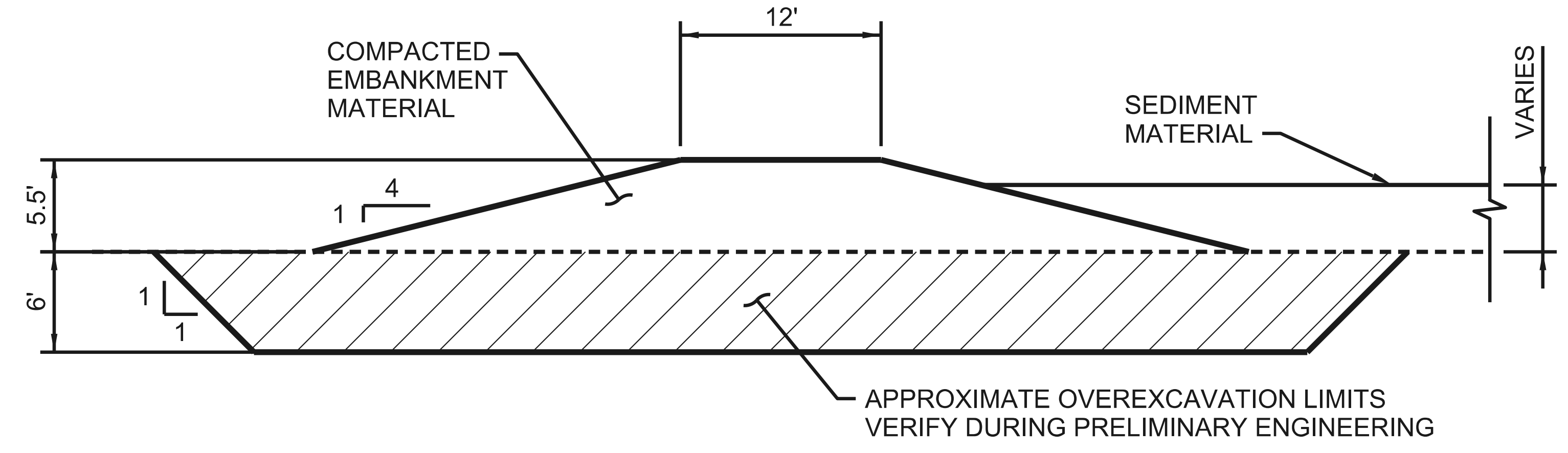
PROJECT NO. 240010
SHEET NO. CCO - C - 3091FB
REV 0
SEQUENCE NO. 91 OF 96



1 PLAN

1"=500'

CCO-C-1082FB



A SECTION

1"=10'

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MODIFIED CLIFTON COURT FOREBAY SEDIMENT AREA PLAN AND SECTION

PROJECT NO. 240010

SHEET NO. CCO - C - 1092FB

REV 0

SEQUENCE NO. 92 OF 96

NOTES:

1. FIBER OPTIC CABLE SYSTEM (IN-TUNNEL OR SURFACE INSTALLED), MICROWAVE, AND LEASED TELECOMMUNICATION LINES ARE ALTERNATIVES THAT WILL BE STUDIED FURTHER IN PRELIMINARY ENGINEERING.
2. FIBER OPTIC CABLE IN CONDUIT AROUND INTERMEDIATE FOREBAY.

PLAN
1"=2 MILES

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CONCEPTUAL ENGINEERING REPORT
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

CONTROL AND COMMUNICATIONS OVERVIEW

PROJECT NO. 240010
SHEET NO. CCO - E - 1093GN
REV 0
SEQUENCE NO. 93 OF 96

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Printed By: Mendez,Martin

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0 1"

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MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

POWER SUPPLY AND GRID CONNECTIONS VICINITY MAP

PROJECT NO.
240010
SHEET NO.
CCO - E - 1094GN
REV
0
SEQUENCE NO.
94 OF 96

Edited By: Bautista,Jav
Printed By: Bautista,Jav

PLAN
1"=40'-0"

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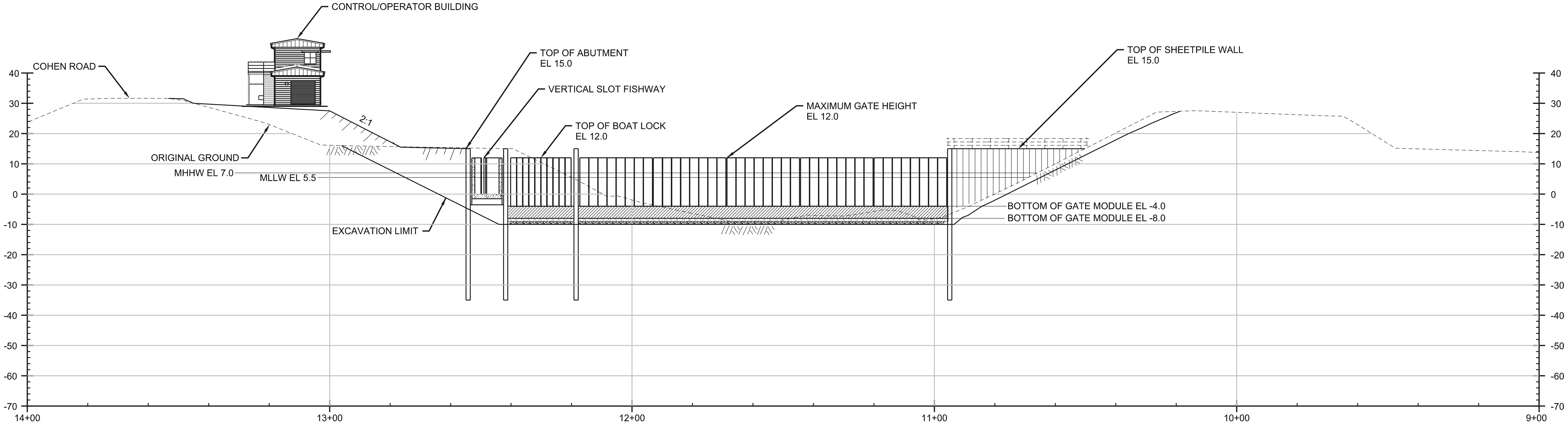
DUAL CONVEYANCE FACILITY
MODIFIED PIPELINE / TUNNEL OPTION - CLIFTON COURT FOREBAY PUMPING PLANT

OLD RIVER FISH CONTROL STRUCTURE
PLAN

PROJECT NO. 240010
SHEET NO. CCO - C - 1095OP
REV 0
SEQUENCE NO. 95 OF 96

pw:\\USMWD224.mwd.h2o:MWD_Projects\MWD_Projects\Projects\Active Projects\601053 (DWR) Delta Habitat & Conservation & Conveyance Program\02 Study (CER-Conceptual Engineering Report)\07-CER-CCO FINAL DRAFT - April 1, 2015\Drawings\02 Civil\CCO-C-095-240010

Edited By: Baghdassarians,Albert
Printed By: Bautista,Jav



A STRUCTURE PROFILE

1"=20'-0"

CCO-C-1095OP

NOTES:

1. VERTICAL DATUM - 1988 NAVD

2. SHEETPILE NOT SHOWN FOR CLARITY.

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OLD RIVER FISH CONTROL STRUCTURE
PROFILE

PROJECT NO. 240010

SHEET NO. CCO - C - 4096OP

REV 0

SEQUENCE NO. 96 OF 96