DRAFT REPORT OF THE INITIAL ANALYSIS & OPTIMIZATION OF THE PIPELINE/TUNNEL OPTION

Delta Habitat Conservation and Conveyance Program (DHCCP)
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The views contained in this report do not necessarily reflect the views of all participants.

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** Note – December 8, 2010 report of the outside review panel (24p) is included as Appendix D.*

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* *By Outside Reviewers*

** *By Division of Engineering, DWR*

Acronyms

AF	Acre-Feet
ASTM	American Society for Testing and Materials
ATO	All Tunnel Option (obsolete; now PTO for Pipeline/Tunnel Option)
AWS	American Welding Society
AWWA	American Water Works Association
BDCP	Bay Delta Conservation Plan
BTF	Byron Tract Forebay
CAISO	California Independent System Operator Corporation
CCF	Clifton Court Forebay
CER	Conceptual Engineering Report (including Addendum)
cfs	cubic feet per second
CM	Construction Management
CVP	Central Valley Project
Delta	Sacramento River and San Joaquin River Delta
DHCCP	Delta Habitat Conservation and Conveyance Program (also, the Program)
DOE	Division of Engineering
DOGGR	Division of Oil, Gas and Geothermal Resources
DSM2	Delta Simulation Model II
DWR	State of California Department of Water Resources (also, the Department)
EIR	Environmental Impact Report
EIR/S	Environmental Impact Report/Statement
EIS	Environmental Impact Statement
ELT	Early Long Term
EPB	Earth Pressure Balance
EPC	Engineering Procurement and Construction
fps	feet per second
GBR	Geotechnical Baseline Report
GIS	Geographic Information System
GPM	Gallons per Minute
HAZ38	Computer program for calculating seismic hazards
HCP	Habitat Conservation Plan
HGL	Hydraulic Grade Line

HP	Horsepower
HVAC	Heating, Ventilating and Air Conditioning
ICF	Isolated Conveyance Facility
ID	Inside Diameter
IF (also IFB)	Intermediate Forebay
IPP	Intermediate Pumping Plant
JV	Joint Venture
k/ft	Kips per foot
LLT	Late Long Term
MDE	Maximum Design Earthquake
MWD	Metropolitan Water District
NGA	Next Generation Attenuation (empirical ground motion models)
NTP	Notice To Proceed
O&M	Operations and Maintenance
OCIP	Owner-Controlled Insurance Program
PARO	State Water Project Power and Risk Office
PEER	Pacific Earthquake Engineering Research
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PLAXIS	A set of geotechnical software tools
PP	Pumping Plant
psf	Pounds per square foot
psi	Pounds per square inch
PTO	Pipeline/Tunnel Option
PVC	Poly-Vinyl Chloride
PW	Present Worth
Q	Flow Rate
R&R	Repair and Replacement
RD	Reclamation District
Reclamation	United States Bureau of Reclamation; also USBR
RFP	Request for Proposal
ROM	Rough order of magnitude
ROV	Remotely Operated Vehicle
ROW	Right of Way

SAP	Integrated software for structural analysis and design [formerly Structural Analysis Program]
SLR	Sea-Level Rise
SPT	Standard Penetration Test
SWC	State Water Contractors
SWP	State Water Project
TBM	Tunnel Boring Machine
TDH	Total Dynamic Head
TO	Task Order
US	United States
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USGS	United States Geologic Survey
WAPA	Western Area Power Administration
WGNCEP	Working Group on Northern California Earthquake Potential
WSE	Water Surface Elevation
24/7	24 hours per day, 7 days a week

1.0 INTRODUCTION

The Delta Habitat Conservation and Conveyance Program (DHCCP) has developed various concepts to convey water from the Sacramento River in the north around the Delta to the existing export pumping plants in the south part of the Delta through an isolated conveyance facility. This effort supports the Bay-Delta Conservation Plan (BDCP) which proposes to revise the current means of conveyance which is limited to the existing Delta channels.

The DHCCP is responsible for engineering conveyance options and preparing an environmental impact report/statement (EIR/S) describing alternatives to achieve the objectives of the BDCP. The DHCCP is a Department of Water Resources (DWR) program that includes participation of the US Bureau of Reclamation (USBR or Reclamation) and state water contractors. The proposed conveyance would become an integral part of the State Water Project (SWP) and the federal Central Valley Project (CVP) by providing water directly to each Project's respective export pumping plant.

DHCCP prepared engineering concepts for a surface canal aligned east of the Delta and, alternatively, a combined canal/tunnel conveyance aligned west of the Delta under Task Order WGI-04. A pipeline/tunnel conveyance option was added to the scope of work for Task Order WGI-04 and facilities for an "all-tunnel" conveyance under the Delta were conceived and documented in the Conceptual Engineering Report Isolated Conveyance Facility All Tunnel Option (CER) and its Addendum. (See References DHCCP 2010a, DHCCP 2010c)

The subject of this report is DHCCP Task Order WGI-34, Focused Analysis of Pipeline/Tunnel Option (PTO) formerly the All Tunnel Option – Initial Optimization. The purpose was to conduct initial optimization of the pipeline/tunnel facilities to a level on par with the canal conveyance options. This initial optimization was limited to a 12-week period with the intention of providing additional guidance, beyond the CER, to the preliminary engineering effort should the pipeline/tunnel conveyance alternative be selected as the proposed project for the BDCP. This initial optimization involved engineers in the DHCCP including the DHCCP consultant, DWR, USBR, state water contractors, and outside tunnel.

The analysis was divided into separate subject areas. Teams of engineers, led by engineering managers, were established for each area as further described in Section 3. Outside tunnel experts were engaged to review the recommendations. Work was conducted from September through November 2010 with the final review meeting conducted in December 2010. The results of this initial optimization effort, including the outside reviewers' comments, are contained in this report and its appendices.

The report begins with background of the Pipeline/Tunnel Option and describes the initial optimization process. Specific subject areas follow, grouped by team, with each sub-issue addressed and results offered. Recommendations for further study are provided as a guide to any follow-on engineering effort.

References used in the report are included in the last section. Appendices consisting of more detailed analysis of certain subject areas or providing relevant information to support the conclusions and recommendations of this report are also provided.

2.0 BACKGROUND

The State Water Project (SWP) and the Central Valley Project (CVP) are each systems of reservoir storage and river conveyances that bring water from snow-packed Sierra Nevada Mountains to cities and farms in the central valley, the coast, and southern California. Both projects use the Sacramento River and San Joaquin River Delta (the Delta) as a primary means of conveyance of this water to each Project's export pumping plant at the south end of the Delta. The export pumping plants send water into the respective conveyance canals (Delta-Mendota for CVP and the California Aqueduct for SWP) for delivery to central valley and southern California users.

The SWP and CVP facilities were completed in the 1960s and early 1970s although a planned peripheral canal to convey water around the Delta was not built. Over time, conditions in the Delta have deteriorated. In response, Governor Schwarzenegger requested various conveyance options be investigated as solutions to help improve conditions in the Delta and improve the reliability of the water it supplies.

An isolated conveyance facility (ICF) is an option under consideration which diverts water from the Sacramento River through screened intakes via pumping and conveys it via pipeline, canal, tunnel (or combination of these) around or under the existing Delta delivering it to the SWP and CVP export pumping plants in the South of the Delta. The conveyance remains isolated from the Delta's rivers, channels, and estuaries allowing restoration of habitat there and protecting the water supply from the consequences of levee failures. Isolating the conveyance of water from the Sacramento River in the north to the export pumping plants in the south is part of the developing Bay Delta Conservation Plan (BDCP). A Habitat Conservation Plan (HCP) for the BDCP project and an Environmental Impact Report/Statement (EIR/EIS) for DHCCP are considering conveyance alternatives, including isolated conveyances via surface routes (canal) and subsurface routes (pipeline/tunnel).

Engineering of conveyance facilities is at the conceptual level to provide sufficient description for analysis of the environmental impact and the preparation of the EIR/EIS. Conceptual Engineering Reports (CERs) were first prepared for isolated conveyance facility options for two surface routes, around the Delta to the east (ICF East Option) and around the Delta to the west (ICF West Option) in early 2009. The latter option includes surface canals on the west side of the Delta in the north and the south joined by a 17-mile 3-bore x 27-ft diameter tunnel in the middle. The East and West options were analyzed and optimized in the summer of 2009 to confirm their facility concepts and better understand their capital costs. During this optimization effort, the 3-bore x 27 ft. ID tunnel concept was revised to a 2-bore x 33 ft. ID tunnel, the larger diameter bores being recommended to save construction time and cost compared to the three-bore tunnel configuration. At that same time, land use impacts and other environmental impacts attributed to above-ground surface conveyance came into sharp focus as the Program met significant challenges to its requests for property access to collect geotechnical soil borings and conduct environmental surveys required to assess impact.

From this setting a third isolated conveyance option—an "all tunnel" (all underground) was conceived and a CER prepared for what is now referred to as the Pipeline/Tunnel Option (PTO).

The Pipeline/Tunnel Option is depicted on Figure 2-1. The PTO proposes a new isolated conveyance for Sacramento River water diverted through multiple fish-screened intakes located between Freeport and Courtland and conveyed by pipelines and tunnels to an Intermediate Forebay (IF) protected from flood, earthquake, and sea level rise (SLR). Water collected in the IF would flow by pumping or gravity through the Intermediate Pumping plant (IPP) into a two-bore 34 mile tunnel system to a new forebay, the Byron Tract Forebay (BTF), located adjacent to and south of the Clifton Court Forebay (CCF). Water would then be conveyed by short canals from the Byron Tract Forebay to the existing pumping plants serving the State Water Project and Central Valley Project.

At the time of this writing (November 2010) a "proposed project" has not been finally identified from among the various conveyance alternatives. Upon designation of a proposed project, preliminary engineering would begin. In the meantime, analysis of the options can provide better understanding of construction costs and likely construction schedules as well as provide important input to the preliminary engineering phase. It is to this end that the analyses in this report have been conducted.

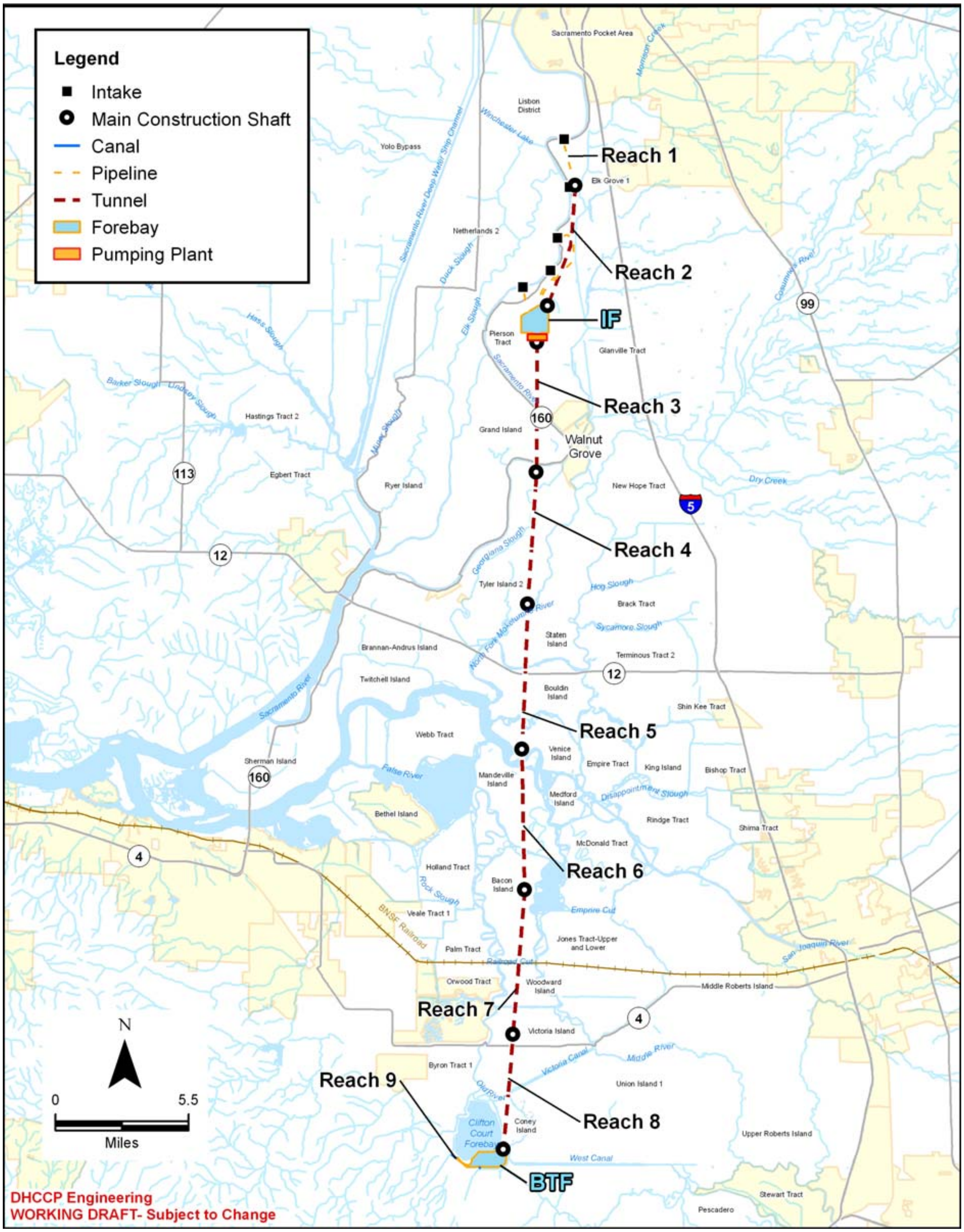

Figure 2-1 Overall Pipeline/Tunnel Option Alignment

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3.0 ANALYSIS OBJECTIVES AND OPTIMIZATION APPROACH

The objective of this analysis was to initially optimize the size and configuration of the Pipeline/Tunnel Option facilities for a specific capacity resulting in a conveyance or range of conveyance options that are reliable, constructible, cost effective, and practicable.

The specified capacity for the PTO is 15,000 cubic feet per second (cfs).

The analyses herein delve deeper into certain aspects of the pipeline/tunnel conveyance than were possible during the conceptual engineering period. The two surface conveyance options were analyzed and optimized in 2009, prior to the PTO concept being completed (DHCCP 2009a and 2009b). As a viable project option, the PTO was in need of additional analysis and optimization to bring its definition on par with the canal options.

The conceptual engineering effort dealt mostly with canal conveyance issues; tunneling was only addressed as a means to cross the deeper watercourses and rivers along the East and West alignments. As the estimated project capital costs for canal options increased to $7-$8 billion, the cost of extending tunnels the full length of the conveyance ($10-$11 billion) became viable to some, especially if the land requirements and associated environmental mitigation challenges could be avoided. The all tunnel option conceptual engineering effort was conducted in a relatively short period and did not receive the optimization scrutiny from some project participants, e.g. State Water Contractors and USBR, as was afforded the canal options.

In late January 2010, the BDCP Steering Committee requested a "focused analysis" of the tunnel conveyance approach "to effectively compare it with canal alignments, which have already received significant review and analysis." The Committee indicated that focused analysis included optimization.

One specific objective of this analysis and optimization was to address concerns of Program participants regarding the tunnel liner. The conceptual tunnel lining relies exclusively on pre-cast concrete lining segments (no inner shell) to carry internal pressure loads due to pumping. Some have suggested that a "2nd pass" liner may be needed, with its increase in capital cost and extension of the construction schedule.

In addition to liner, tunnel diameter required further analysis as the baseline inside diameter (ID) of 33 ft was derived merely from a workable target flow velocity (8 fps at 7,500 cfs) used in the canal options. Consideration of larger tunnel diameters and their corresponding cost tradeoffs, capital vs. operating, were other key questions.

Implementation—constructing the tunnels, both in terms of access shafts (portals) through which to begin tunnel boring and conducting the boring itself—varies in complexity with depth, requires contract packages that industry will bid on, and requires plans that reflect the unique site conditions of the Delta (e.g. existing ground below sea level protected only by aging levees, thick layers of organic/peat soils, gas wells, high water table, and limited highway and rail access.)

The DHCCP work plan included representatives from DWR, the USBR, and State Water Contractors as direct participants in the work.

Separate teams identified to address specific topic areas in the task order are shown below:

Team A – Design Criteria

- Liner design
- Evaluation of recent geotechnical data
- Input to profile
- Shaft loading
- Seismic displacement analysis
- Eastern alignment subsurface condition assessment

Michael Forrest, PE – Team Lead DHCCP		Sam Gambino, Geotech Engineer, DHCCP
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Galen Klein, Tunnel Engineer DHCCP		Susan Olig, Geotech Engineer, DHCCP
Michael Monaghan, Liner Engineer, DHCCP		State Water Contractors
DWR DOE		USBR

Team B – Implementation

- Shaft arrangements
- Contract packages, bonding, and schedule (construction phasing)
- Profile
- Alignment
- Protection of shafts from flooding

Michael Cherry, Tunnel Option Manager – Team Lead DHCCP		Robert Goodfellow, PE – Tunnel Engineer and Peer Reviewer, DHCCP
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Sergio Valles, PE – SWC/MWD		Carlos Jaramillo, Lead Tunnel Engineer, DHCCP
Ryan Irlmeier – Cost/Schedule, DHCCP		Paul Kneitz, PE - DHCCP

Team C – Tunnel Conveyance System Optimization

- Tunnel diameter optimization
- Configuration recommendations
- Intakes-to-intermediate forebay optimization
- Capital cost, energy/O&M costs, present worth calculations

Paul R. Kneitz, PE – Team Lead DHCCP		Jesse Wallin – Engineer, DHCCP
Craig Galuska – Team Assistant Lead, DHCCP		Chris Nielsen – Engineer, DHCCP
Stephane Lecina, PE – Hydraulics Engineer, DHCCP		Madhavan Jayakumar – Engineer, DHCCP
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Gordon Enas, PE – DWR DOE		Alan Stroppini, PE – Engineer, USBR
Ganesh Pandey – DWR DOE		Sergio Valles, PE – SWC/MWD

Team D – Gas Wells and Barge Facilities

- Gas well interferences and approach
- Barge facility considerations

Nekane Hollister, PE – Team Lead, DWR/DOE	John Hooper, PE – Engineer, DWR/DOE
Anna Gutierrez, PE – Engineer, DWR/DOE	Hoang Le, PE – Engineer, DWR/DOE

F. Thomas Young, PE – Task Order Mgr, DHCCP	

Team E – Operations and Maintenance (Inspection using ROVs; Tunnel de-watering issues)

- Applicability of remotely operated vehicles
- De-watering issues

Nekane Hollister, PE – Team Lead, DWR/DOE	Shah Adil, PE – Engineer, DWR/DOE
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Tunnel Experts – Outside Reviewers

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Resumes for the above-listed tunnel experts/outside reviewers are provided in Appendix C.

The task order work plan was approved at the end of August 2010 and work began in September 2010. The first outside review meeting was conducted in Sacramento, CA on October 5, 2010. See Appendix A for notes of that meeting. Appendix B contains responses and concurrence from the reviewers. Analysis and optimization continued through the months of October and November. The initial draft report of recommendations and optimization (this report, Revision A) was issued for outside review in late November, 2010. The second and final outside review meeting was conducted in Sacramento on December 7 and 8. The report by the Outside Reviewers is provided in Appendix D. The final issuance of this Draft Report is Revision B, December 17, 2010.

It is the intention of this work effort to provide input and guidance for follow-on engineering efforts, specifically preliminary engineering, in expectation of an isolated conveyance facility using the pipeline/tunnel option becoming DHCCP proposed project.

4.0 TUNNEL DESIGN CRITERIA AND COMPONENT ANALYSIS

4.1 Introduction

This section presents the work performed by Team A – Tunnel Design Criteria and Component Analysis. Specifically, further analysis of the tunnel conveyance has been performed in accordance with the scope of work for Team A to:

- Confirm hydraulic friction factor(s) for tunnels.
- Analyze tunnel lining structural requirements and revise tunnel lining design concepts as needed.
- Analyze size, shape and structural requirements for the shafts. Update shaft design requirements as needed.
- Review the recently acquired geotechnical data (including borings over water along the alignment) and confirm basic tunneling concepts.
- Evaluate seismic loading and potential range of seismic displacement of the tunnels and propose concepts for mitigating seismic displacement.

4.2 Review of Geotechnical Data and Preliminary Ground Characterization

The purpose of Section 4.2 is to review the currently available geotechnical data and to confirm basic tunneling concepts.

4.2.1 Summary of Geotechnical Data

Team A reviewed currently available geotechnical data for the project. Limited subsurface information was compiled from data collected from 5 borings conducted in 1993 and 1994 for the Delta Seismic Stability Study and 8 borings conducted in October 2009 by the DHCCP for evaluation of conveyance. This data is included in a report entitled Draft Phase I Geotechnical Investigation – Geotechnical Data Report – Pipeline/Tunnel Option – Rev 0, dated July 12, 2010. (DHCCP 2010b).

4.2.2 Preliminary Ground Characterization

The tunnel is anticipated to be constructed in soft ground conditions below the groundwater table. Ground conditions anticipated to be encountered include interbedded deposits of silty sand, clean sand, silt, clay, and gravel. Earth Pressure Balance (EPB TBM) or Slurry Tunnel Boring Machines (Slurry TBM) will be required for construction of the tunnels. Additional ground characterization of the soils related to the type of machine was not part of the scope of TO WGI-34. This will need to be done during preliminary engineering, and final design.

Preliminary ground characterization was performed in order to estimate ground hydraulic conductivity, to prepare preliminary load diagrams for the shafts and to make preliminary estimates of the loading on the tunnels. This ground characterization was performed based on a weighted average of each of the materials found in the boreholes. The type of materials considered were gravel, sand (<5%, 5-12%, >12%), silt and clay. The most recent borings were reviewed and empirical correlations (Terzaghi et al, 1996) were used to estimate geotechnical parameters at different depths. In order to make these preliminary estimates, averages were taken across layers at different depths. The estimated geotechnical parameters which were used in the initial evaluations performed for this optimization task are shown in Table 4-1.

Table 4-1 Summary of Geotechnical Parameters Assumed

	Depth (ft)		
Parameter	80 ft	120 ft	160 ft
Water Unit Weight, γ_w (lb/ft^3)	62.4	62.4	62.4
Soil Unit Weight, γ_{soil} (lb/ft^3)	130	125	125
Soil Poisson's Ratio, ν_{soil}	0.3	0.35	0.35
Soil Cohesion, c (psi)	0	0	0
Soil Friction Angle, ϕ (°)	33	30	28
Soil Modulus of Elasticity, E_{soil} (ksf)	450	600	900
Soil Hydraulic Conductivity, k_{soil} (cm/sec)	2×10^{-3}	6×10^{-4}	10×10^{-4}

It should be noted that the parameters assumed and shown in Table 4-1 reflect an apparent increase in clay content with depth.

4.2.3 Preliminary Estimate of Spring Stiffness for Structural Analysis

One of the methods to model the soil-structure interaction of the tunnel lining and the surrounding soils uses soil springs to simulate this elastic interaction. The stiffness of these springs can be estimated following the procedure proposed by USACE (1997). The radial spring stiffness, (k_r) and tangential (k_t) stiffness can be estimated by the following formulas:

- Radial Spring: $k_r = E_r b\theta/(1+v)$ = 130 k/ft
- "E_r": Young's modulus for the soil = 4×10^5 psf
- "b": The width of the segmental tunnel ring = 5 ft
- "θ": The arc subtended by the segment "beam" in the finite element model= 5° (0.087266 radians)
- "v": Poisson's ratio for the soil = 0.35
- Tangential Spring: $k_t = 0.5k_r/(1+v)$ = 48 k/ft

The values of the spring stiffness were estimated based on a conservative version of the geotechnical parameters shown in Table 4-1.

These springs are input as compression only elements into the finite element model because soil cannot act in tension. Impact of variation in the spring stiffness should be evaluated in more detail during future phases of the project.

4.3 Tunnel Lining Evaluation

4.3.1 Description of Precast Concrete Segmental Lining

The tunnel lining is anticipated to be constructed of precast concrete segments configured in a universal pattern. For the purposes of these analyses, each ring of segments is assumed to be 5 feet wide and would consist of 8 segments plus a key. The scope of the optimization task did not include the optimization of the size of the segments or looking into a different number of segments, but generally, the technical literature indicates that tunnels with diameters of 30 ft or larger tend to use at least 8 segments. A schematic of the tunnel lining is shown in Figure 4-1. The joints between each segment will contain synthetic rubber gaskets to minimize exfiltration and infiltration. In order to minimize exfiltration a double gasketed joint is proposed for the tunnel. The segments would be assembled forming a ring, and these rings would be joined by doweled circumferential joints. The segments of one ring would be bolted together within an over-excavated space, leaving an empty annulus which would be filled with grout after installation of the segment ring. The segments will be designed to resist a combination of loads, including external loads during construction and operation, and internal pressures that are anticipated to be developed during operation of the tunnel.

For the Conceptual Engineering Report (CER) (DHCCP, 2010), a 33-foot ID tunnel with 24-inch thick lining was assumed. For this Tunnel Optimization Task, tunnels ranging in internal diameter from 20 to 50 feet and tunnel depths (to spring line) ranging from 80 feet to 200 feet have been evaluated. These evaluations indicate that the preliminary tunnel lining thickness could range from 10-inches to 18-inches depending on the depth and diameter of the tunnel. These thicknesses are based on compression of the rings at the given depths and loading conditions. Considerations for gasket placement, additional reinforcement to withstand internal pressure and other miscellaneous elements required for complete design of the tunnel liner will likely control lining thickness. It is recommended that 24-inches continue to be assumed as the required thickness of the tunnel segments.

4.3.2 Preliminary Structural Design Criteria

For the purposes of performing preliminary structural analyses and calculations, the structural design criteria shown below have been used.

4.3.2.1 General

Unless otherwise specified, the structural design would be in accordance with the following documents:

- ACI 318-05, Building Code Requirements for Reinforced Concrete
- AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Building, Thirteenth Edition
- EM 1110-2-2000, Standard Practice for Concrete for Civil Works Structures Change 2
- EM 1110-2-2104, Strength Design for Reinforced - Concrete Hydraulic Structures
- EM 1110-2-2901, Tunnels and Shafts in Rock
- EM 1110-2-6053, Engineering and Design - Earthquake Design and Evaluation of Concrete Hydraulic Structures

4.3.2.2 Materials

Segments f'c: 5,000 psi

Grout f'c: 2,500 psi

Reinforcing Steel: ASTM A615, Grade 60

Structural Steel: ASTM A36

High-Strength Bolts: ASTM A325

Anchor Bolts, Misc. Metal: ASTM A307

Welding: AWS D1.1-82; E70 XX

4.3.2.3 Design Loads

All parts of the structure would be designed to support and resist the loads and forces listed below.

Dead Load: The vertical load due to weight of all permanent elements of the structure and equipment. Unit weights shall be assumed as follows:

Concrete: 145 lb/ft^3

Steel: 490 lb/ft^3

Water: 62.4 lb/ft^3

Load Combination: The design would include evaluation of several combinations of the following loads:

U_h = factored loads for a hydraulic structure

H_f = hydraulic factor = 1.3, except for members in direct tension. For members in direct tension, H_f =1.65.

D = Dead load

L = Live load

F = Hydrostatic pressure

W = Wind pressure

E = Seismic load

H = Static soil pressure

4.3.3 Preliminary Loading Cases for Precast Concrete Segmental Lining

During final design the evaluation of the precast concrete lining would need to consider the loading cases presented in this section; however, for the purposes of this optimization task, these loading cases were not applied to size the precast segmental lining for this task, but are presented for information. For this task, the preliminary thickness of the precast lining was based on earth pressures and groundwater pressures (compression loading). The additional loading cases that would need to be considered are shown in Figures 4-2 through 4-11 and listed below:

- Final Condition External Pressure (Dewatered) (Fig. 4-2)
- Final External and Internal Pressure (Fig. 4-3)
- Final Condition at Manhole Internal Pressure (Fig. 4-4)
- Final Condition at Manhole External Pressure (Dewatered) (Fig. 4-5)
- Parallel Tunnel - No Side Confinement Construction Condition (Fig. 4-6)
- EPBM/Muck Car Track Loading on Segments During Construction (Fig. 4-7)
- Grout Load on Segments During Construction (Fig. 4-8)
- Storage Load on Segments in Yard (Fig. 4-9)
- Handling and Lifting Load on Individual Segments (Fig. 4-10)
- TBM Jacking Loads During Construction (Fig. 4-11)

4.3.4 Preliminary Lining Evaluation

4.3.4.1 Anticipated Lining Thickness

A family of curves has been prepared showing the preliminary estimate of lining thickness for varying tunnel diameters for a range of depths. The family of curves is presented in Figure 4-12. The thickness of the liner presented in the curves is based on external loads, and was prepared using an elastic closed form solution. Examination of the curves indicates that the thickness increases approximately linearly both in response to increased diameter, and increased depth, and ranges from 10 inches (defined as minimum thickness) to 30 inches for the largest diameter considered and maximum depth.

It should be noted that these thickness curves are based only on the compression loading of the segments, and other loading cases have not yet been analyzed. It is likely that handling loads, erection loads, and TBM jacking loads, as well as geometrical requirements due to the use of two gaskets could control the thickness of the segments.

4.3.4.2 Preliminary Evaluation of Tension on Lining

In order to resist the internal pressures that are anticipated during operation of the tunnels and to minimize leakage, a preliminary review of several design concepts has been performed. Reinforced concrete linings have been used successfully in many projects operating under internal pressure, including the San Diego South Bay Ocean Outfall with a precast segmental lining designed for 89 ft of internal differential head. Cast in place tunnel linings rely on the reinforcement steel to resist internal pressures and control the size of resulting cracks. Precast segmental linings rely on the reaction of the surrounding ground and external water pressure, or structural restraints. Some of the mechanisms that could be used to handle internal pressures in precast segmental linings and that were considered for the optimization task include passive steel reinforcement, post-tensioned steel cables and shear cones. These methods are described below. For this optimization task, a finite element model was run assuming that shear cones are selected as the mechanism to resist the internal pressures. External hydrostatic loads and ground loads were assumed for the modeling. The results are described below. The loading case where ground loads acting on the liner are neglected is currently being evaluated based on comments from the first outside reviewers meeting.

Passive Reinforcement

Full rings of steel can be embedded in the segments and then attached (bolted together) after the segments are installed. The steel rings would be nominally tensioned so any internal pressure larger than the external pressure would engage the steel immediately, and maintain the gasket compression, and thus control the joint opening.

Post Tensioning

Post-tensioned cables can be installed within the segments and then tensioned after the segments are installed. This is a viable design alternative that has been applied successfully to several tunnels in Japan. Post-tensioned cables are usually considered a corrosion risk and because of this risk post-tensioned cables were not recommended by the outside review panel as the preferred method of resisting internal pressures.

Shear Cones

A relatively new concept, installing shear cones or dowels, was suggested at the outside reviewers meeting. Shear cones, such as those manufactured by SOFRASAR, can be installed to transfer normal forces from one ring to an adjacent ring via radial and tangential shear. This is a relatively new concept and before final design is completed based on the concept of utilizing shear cones, we recommend that a full scale testing program be implemented to validate the ability of the shear cones to effectively resist internal pressures of the magnitude anticipated during operation of the facility.

For the purposes of this optimization, a SAP finite element structural model was run assuming shear cones were installed. Preliminary finite element analyses were performed using internal loading corresponding to the hydraulic head at the intermediate pumping station, and assuming no tension capacity between segments (e.g. no passive steel or post-tensioned cables were assumed in addition to the shear cones). For the shear cone analysis, two cases were evaluated: 34-foot ID and 45-foot ID tunnels at 120 feet in depth.

A third load case evaluated using the 34-foot ID tunnel, and considered an extreme case, was internal pressure and external hydrostatic load without any contribution of the ground surrounding the tunnel. In this load case, the segmental lining would expand due to the tension loading caused by the internal pressure being greater than the external pressure. This expansion would need to be controlled by structural means, and the one considered in this case was shear dowels or cones only. This case was examined using theory of elasticity formulations, as discussed below.

The load in the lining would consist of two stress components, hoop stress along the lining, and radial stress across the lining. Hoop stress would be carried by the internal reinforcement within the segment, and transferred from one segment to the adjacent segment by the shear dowels located close to the segment-to-segment joints. Radial stress would be carried by shear in the segment concrete, and by the bolts at the joints.

This load condition was evaluated assuming a 34-ft ID tunnel, with a 24-inch lining thickness, nine segments per ring, each ring 5 feet wide, a differential internal hydrostatic pressure equivalent to 120 ft of head, equivalent to a pressure of 7,500 psf (52 psi), and approximating the tunnel lining to a "thin walled cylinder." Using this approximation the following was calculated:

Hoop stress acting on the concrete segments	470 psi
Hoop tension load in each ring	675 kips

The tension load could be accommodated with reinforcement in the concrete segment, as the segment would most likely have two layers of reinforcement with bars spaced at about 6 inches.

To transfer the load across the joint, dowels at each side of the segment would be used. The standard load case concept calls for shear dowels at 5° or about 1.5 ft, resulting in eight dowels per segment, per side. Four dowels at each side of the joint would be available to transfer the load as shear, at about 84 kips per dowel. Commercially available synthetic dowels are designed to resist 10 to 20 kips per dowel, steel core dowels are designed for 60 kips, while the SOFRASAR shear cones are rated at 56 kips per dowel.

Reducing the spacing of the dowels can provide the required shear strength, and/or custom made dowels with the required capacity, can provide the required shear strength. Reducing the spacing to about 3°, or about 1 ft center-to-center would bring the load per dowel to 56 kips per dowel, within the range of commercially available dowels at a factor of safety which may be appropriate for this extreme load case.

As shown in Figure 4-1, the segment rings are placed rotated one to the next, so the joints are not aligned. Assuming this geometry, when one ring tries to expand due to internal pressure, the shear cones would transfer the load to the adjacent ring. The finite element model indicated that for a 45-foot ID tunnel, nearly 15 inches of cover of plain concrete (f'c=5,000 psi) over the shear cone would be required. For a 34-foot ID tunnel the concrete thickness required would be 10 inches over the shear cones. The thickness requirement could be reduced by using shear and/or fiber reinforcement in the segments. The use of passive or active (post-tensioned) reinforcement in the rings would also reduce the required thickness of the liner.

4.3.4.3 Steel Liner

The tunnel shown in the CER uses a precast segmental lining with gaskets for control of leakage and infiltration. This type of design is acceptable if some amount of exfiltration and infiltration are acceptable during operation of the facility. However, if no exfiltration or infiltration is permissible, then the precast segmental lining would be used only for preliminary ground support and a second pass internal impervious liner would be required. The options for this impervious liner include steel, geomembranes, or sprayed-on membranes or a second pass reinforced concrete lining, but the most commonly used, and the one presented in this task for comparison, is a steel liner.

The steel liner was dimensioned assuming that the steel would take the entire internal load without contribution from the backfill concrete, segmental lining, or surrounding ground, resulting in a 1-inch thick plate. Considering the large diameter of the liner, external loads could buckle it, as confirmed using the Jacobsen procedure (USACE 1997, Tunnels and Shafts in Rock, EM 1110-2-2901, Dept. of the Army, Washington), so steel stiffener rings were added. These rings are 1-inch thick, 10-inches high and spaced 10 feet apart. These dimensions are based on conservative estimates, and using ASTM A516, fy=38,000 psi steel, assuming an internal pressure equivalent to 220 ft of water head, and external pressure equivalent to 120 ft of water head. The steel plate thickness and steel stiffeners could be reduced by using a higher steel grade, but this refinement was not performed for this task.

4.3.5 Preliminary Evaluation of Gasket Capabilities

A preliminary evaluation of anticipated tunnel exfiltration through the segmental tunnel liner gaskets was performed assuming that no structural means were provided to counteract the internal pressure. The calculations considered the interdependence between the liner's permeability and its loading. These calculations were performed using assumptions that represent the conditions that would be observed at the beginning of exfiltration shortly after the tunnel is pressurized, and represent the point when the highest anticipated exfiltration would occur. These conditions would only apply before a steady-state flow regime is reached. Once a steady state condition is reached, the gradient acting across the gaskets would be reduced to the hydraulic head losses across the gasket, resulting in lower amounts of exfiltration. In that sense, the exfiltration values estimated are conservative.

The analyses were performed using PLAXIS to evaluate the stress conditions in the lining and on the gaskets, and Seep/w to evaluate the impact of leakage on the groundwater.

The analyses showed that a tunnel at a depth to springline of 80 ft would have a considerable amount of exfiltration (Table 4-2), and the impact on groundwater could reach the surface, while a tunnel at depth of 160 ft would experience a moderate amount of exfiltration and would have a negligible impact on the groundwater.

Table 4-2 Summary of Tunnel Exfiltration Estimates

Depth, d (ft)	Exfiltration from tunnel at maximum internal pressure, q (ft^3/sec/ft)	Total exfiltration (ft3/s)	Exfiltration (acre-ft/day)
80	1.4×10^{-2}	1100	2200
120	2.9×10^{-3}	180	360
160	4.5×10^{-4}	24	48

Exfiltration (leakage) criteria will need to be developed and adopted during Preliminary Engineering. Search of technical literature identified criteria used for sewer tunnel exfiltration (0.008 gpm x 100 ft x ft ID resulting on 515 gpm or 2.3 acre-ft/day) and infiltration, or the AWWA criteria for leakage from pressure pipes (3,700 gal/hr or 0.3 acre-ft/day). Depending on the exfiltration criterion adopted for this project, a tunnel at a depth of 160 feet could be acceptable without additional structural restraints. All of these exfiltration estimates are preliminary because there is very little geotechnical information available to characterize the ground around the tunnel.

4.3.6 Tunnel Lining – Initial Recommendation

Based on the studies performed as part of this task, a precast segmental lining continues to be favored as originally proposed in the CER. Internal pressure could be resisted using any of the systems described above (passive reinforcement, post-tensioning or shear cones) and exfiltration could be controlled using a double gasket system. Passive reinforcement and post-tensioning have been successfully used in other tunnel projects using precast segmental lining with internal pressure (San Diego South Bay Outfall, several tunnels in Japan, and the Thun Flood Relief Tunnel in Switzerland.) The use of shear cones to transfer shear from one ring of segments to adjacent rings appears viable, subject to rigorous prooftesting and demonstrated performance in completed tunnels. A second pass system using a steel liner installed in the areas of higher pressure should be maintained as an option until development of the design and testing prove the feasibility of the favored lining option.

4.4 Friction Factor

For the CER, hydraulic calculations were performed assuming a Manning's "n" value of approximately 0.0145. This optimization has evaluated the assumed friction factor and linings to the interior of the tunnel.

4.4.1 Methods to Reduce Friction Factor

Lining the interior of the tunnel can reduce head losses by reducing friction in the tunnel and subsequently reducing the energy costs needed to overcome that friction during operation. The following types of interior liners have been considered and compared with an unlined tunnel:

- Polyurethane applied prior to segment installation
- Epoxy applied prior to segment installation and applied after segment installation
- Ameron T-Lock PVC Liner

Table 4-3 shows the anticipated costs and benefits of potential tunnel coatings.

Table 4-3 Potential Tunnel Coatings

Interior coating	Manning's Coefficient	Added Cost $/SF	Limitations
T-Lock	0.0100	9	Limited to 100 feet of pressure head differential on the outside of the pipe.
Epoxy applied in tunnel	0.0100	14	Sensitive to moisture in tunnel but can be formulated to compensate
Epoxy applied above ground	0.0115	8	Could fill in joints below ground later.
Polyurethane applied in tunnel	0.0100	12	Surface must be dry. May be difficult to keep the tunnel dry.
Polyurethane applied above ground	0.0115	6	Moisture can be controlled above ground
No coating	0.0120	0	Smooth concrete forms assumed for precast segments.
Other tunnel surfaces			
Formed concrete	0.0120	NA	Equivalent to precast liner but with fewer joints – *for reference only.*

4.4.2 Recommendations for the Use of a Friction Reduction Liner

A preliminary cost/benefit analysis indicates that the use of liners to offset pumping costs alone is not recommended (Section 6.6.5, Table 6-12, Line 6). There may be additional benefits in reduced maintenance and extended design life that have not been taken into account. Further study of lining potential is recommended during preliminary engineering. Additional types of liners may be considered during preliminary design. See Appendix E for discussion on friction reducing liners.

4.5 Preliminary Seismic Analysis

The purpose of this section is to make a preliminary evaluation of the expected displacements from a seismic event and to recommend the approach to design for these displacements.

4.5.1 Assumptions

Peak ground velocity for a 975-year return period, which is the Maximum Design Earthquake (MDE) selected for the DHCCP, was computed for the DHCCP tunnel pipeline option at the southern end of the project near Clifton Court. Limited shear wave velocity data was available. A shear wave velocity in the upper 30 meters (V_S30) was chosen to be 304 m/sec based on nearby boreholes.

There are relatively few predictive equations for peak ground velocities (PGV) in comparison to the large number of equations for estimating peak ground acceleration (PGA) and spectral accelerations. Prior to the development of the Next Generation Attenuation models (NGA), developers did not routinely provide predictive relationships for PGV. For engineering purposes, PGV was estimated based on PGA or another spectral acceleration value. The NGA models of Abrahamson and Silva (2008), Boore and Atkinson (2008) and Campbell and Bozorgnia (2008) provide predictive equations for PGV given magnitude, distance, fault type and site conditions. These three predictive equations allow for computation of PGV within a probabilistic seismic hazard analysis program.

For this project, the seismic source characterization of the San Francisco Bay Area and Delta regions developed by URS for the Delta Risk Management Study (URS, 2008) were utilized. These models are based partially on the USGS Working Group on Northern California Earthquake Potential (WGNCEP, 1996), the USGS Working Group on California Earthquake Probabilities (WGCEP, 2003) and the California Geological Survey's seismic source model used in the USGS National Hazard Maps (Cao *et al.*, 2003) and revised based on recent research (Wong *et al.*, 2008). The three NGA models were weighted equally. The hazard calculations were made using the computer program HAZ38 developed by Norm Abrahamson. The program has been validated in the PEER Center-sponsored "Verification of Probabilistic Seismic Hazard Analysis Computer Programs" Project (Thomas *et al.*, 2010).

4.5.2 Peak Ground Acceleration and Velocity

In addition to PGV, PGA was also computed. PGV and PGA for 975-year return period are provided in Table 4-4.

Table 4-4 Median Peak Ground Velocity and Acceleration

	975-Year Return Period	
	PGV (cm/sec)	**PGA (g)**
Weighted Mean Total	49	0.49

4.5.3 Preliminary Evaluation of Tunnel Performance during Earthquake

The seismic behavior of the tunnels was studied using closed-form solutions. The performance of the tunnel was analyzed for axial and curvature and ovaling.

These analysis methods only evaluate the response of the tunnel lining to ground shaking. Evaluation of potential impact due to liquefaction and fault displacement is discussed below. Satisfactory seismic performance of the conveyance system implies its remaining in operation following the design earthquake.

The results of the analyses indicate that the performance of the tunnel lining is controlled by the ovaling deformations, and they, as well as axial, and curvature deformations generated by the design earthquake can be tolerated by the tunnel lining used for the CER. The compressive stresses induced by the earthquake are less than the assumed compressive strength of the concrete; however, temporary de-stressing of segment joints could occur, resulting in temporary increase in exfiltration.

The results indicate that the tunnel, given the geometry assumed for the opening and the liner, will remain structurally stable when subjected to the ovaling deformations caused by the design earthquake.

It should be noted that the Thorton Arch area has been identified as a seismically active area, but no detailed studies of its potential deformation have been performed.

4.5.4 Potential for Liquefaction

A preliminary evaluation of liquefaction potential has been performed for the pipeline/tunnel along the current alignment. The procedure used is presented in a paper by Seed et al. (2003). The following eight borings along the tunnel alignment were analyzed to assess the potential for liquefaction:

- DCR-DH-011
- DCA-DH-003
- DCA-DH-004
- DCA-DH-005
- DCR-DH-015
- DCR-DH-016
- DCT-DH-010
- DCE-DH-014

SPT information provided in the Geotech Package dated July 27, 2010 (DHCCP, 2010) was used to assess the potential for liquefaction. SPT values reported in the logs were already normalized to an efficiency of 60% (N_{60}), therefore no energy ratio correction was applied. For those boreholes where the water level was above the ground surface (DCR-DH-011, DCR-DH-015 & DCR-DH-016), the total stress used to estimate the cyclic shear did not included the weight of the water above the ground water surface. For the other boreholes (DCA-DH-003, DCA-DH-004, DCA-DH-005, DCT-DH-010 & DCE-DH-014) it was assumed that the groundwater level is at the ground surface.

It was assumed that clays (CL, CH, OH), high plasticity silts (MH) and peats (PT) are not susceptible to liquefaction (cyclic mobility). These soils might be subject of potential loss of strength due to sensitivity. Potential sensitivity of such soils was not evaluated as part of this liquefaction analysis. Liquefaction susceptibility of "dirty" granular soils (SM, SC) and low plasticity silts (ML) was evaluated following the recommendations by Seed et al. (2003). When information available (water content, fines content, Atterberg limits, etc) was not sufficient to conclude whether or not these soils were liquefiable, they were assumed as potentially liquefiable; however, these soils were flagged as requiring further testing to determine if they are indeed potentially liquefiable. The seismic event assumed for the liquefaction analysis had a magnitude of 7.5 and a peak ground acceleration of 0.49 g. The average shear wave velocity for the uppermost 40 feet of soil ($Vs,_{40}$) was assumed to be 500 ft/sec.

All of the borings analyzed included soils that are potentially liquefiable, although to different extents. Substantial, continuous liquefaction of the soil column can be expected down to elevation -100 feet, based on the borings analyzed. Below this depth only isolated pockets of liquefaction are observed.

4.6 Preliminary Load Diagrams for Shafts

Temporary construction shafts and permanent access shafts will be required along the tunnel alignment. The purpose of this section is to present preliminary loading diagrams for the shafts.

4.6.1 Assumed Shaft Configuration and Construction Methods

The size and shape of the shaft excavations and the type of excavation support system for the shafts will be determined by the Contractor subject to the limitations shown on the Drawings and the requirements stated in the Specifications. Shafts are assumed to be cylindrical and non-cylindrical.

4.6.2 Preliminary Loading Diagrams for Shafts (including uplift)

Preliminary earth pressure calculations have been performed for the construction shafts based on borings DCA-DH-004 and DCR-DH-016 and are shown in Figures 4-13 through 4-16. The following assumptions were made in developing preliminary loading diagrams for the shafts:

- A shaft depth of 120 ft.
- Groundwater was assumed at ground surface and no dewatering performed outside the shaft during construction.
- Earth pressures were calculated for two possible alternatives: braced wall and yielding wall (free to move to achieve active condition).
- Two possible shapes of shaft were considered: cylindrical shaft and non-cylindrical shaft.
- The braced wall calculations were based on Peck et. al. (1973) and the yielding wall calculations were based on Rankine and Cheng & Hu (2005) for non- cylindrical shaft and cylindrical shaft, respectively.

Idealized soil profiles were developed at two locations based on boring DCA-DH-004 and DCR-DH-016 provided in the Geotech Package dated July 27, 2010 (DHCCP, 2010). Boring DCR-DH-016 was drilled on a ship over river. The top elevation of shaft near boring DCR-DH-016 is assumed as the same as river surface on the landside. These two different soil profiles were selected to show a range of potential conditions that can be found along the tunnel alignment, and illustrate the type of loading conditions that the shafts would need to be designed for. The ground was characterized based on the testing performed on samples from the two boreholes listed above. However, design of the shafts will require additional exploration and testing and a review and re-evaluation of the ground characterization.

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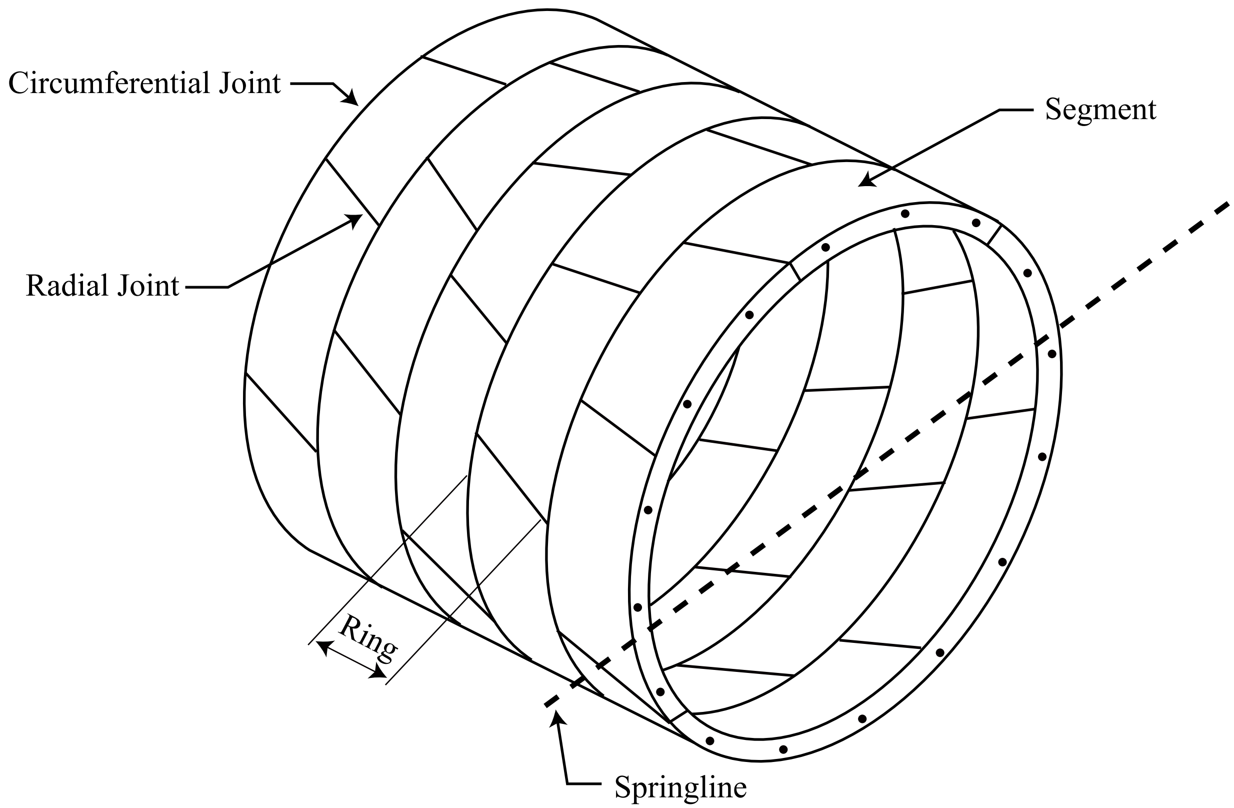

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Schematic of Precast Concrete Segmental Tunnel Lining

FIGURE 4-1

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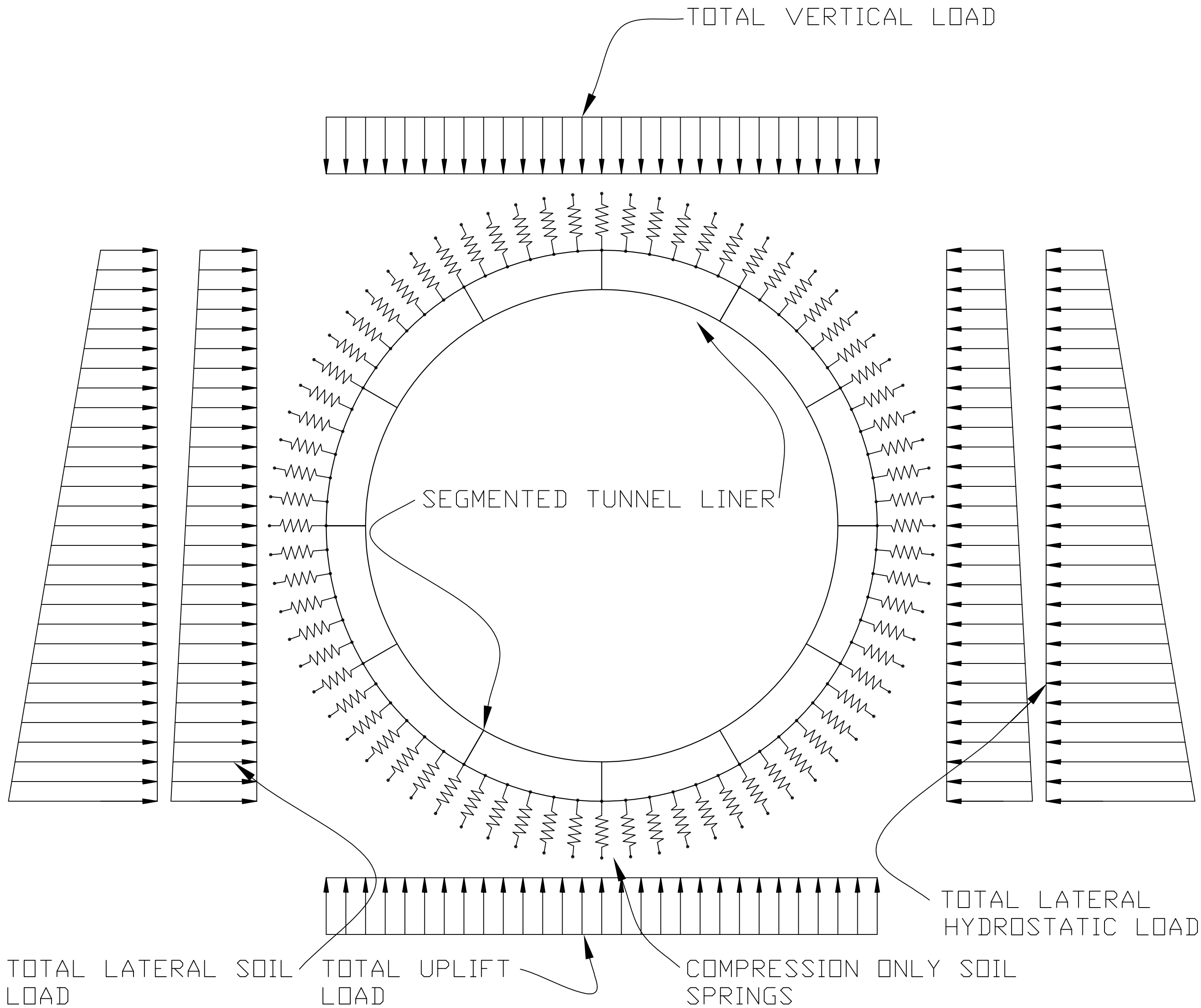

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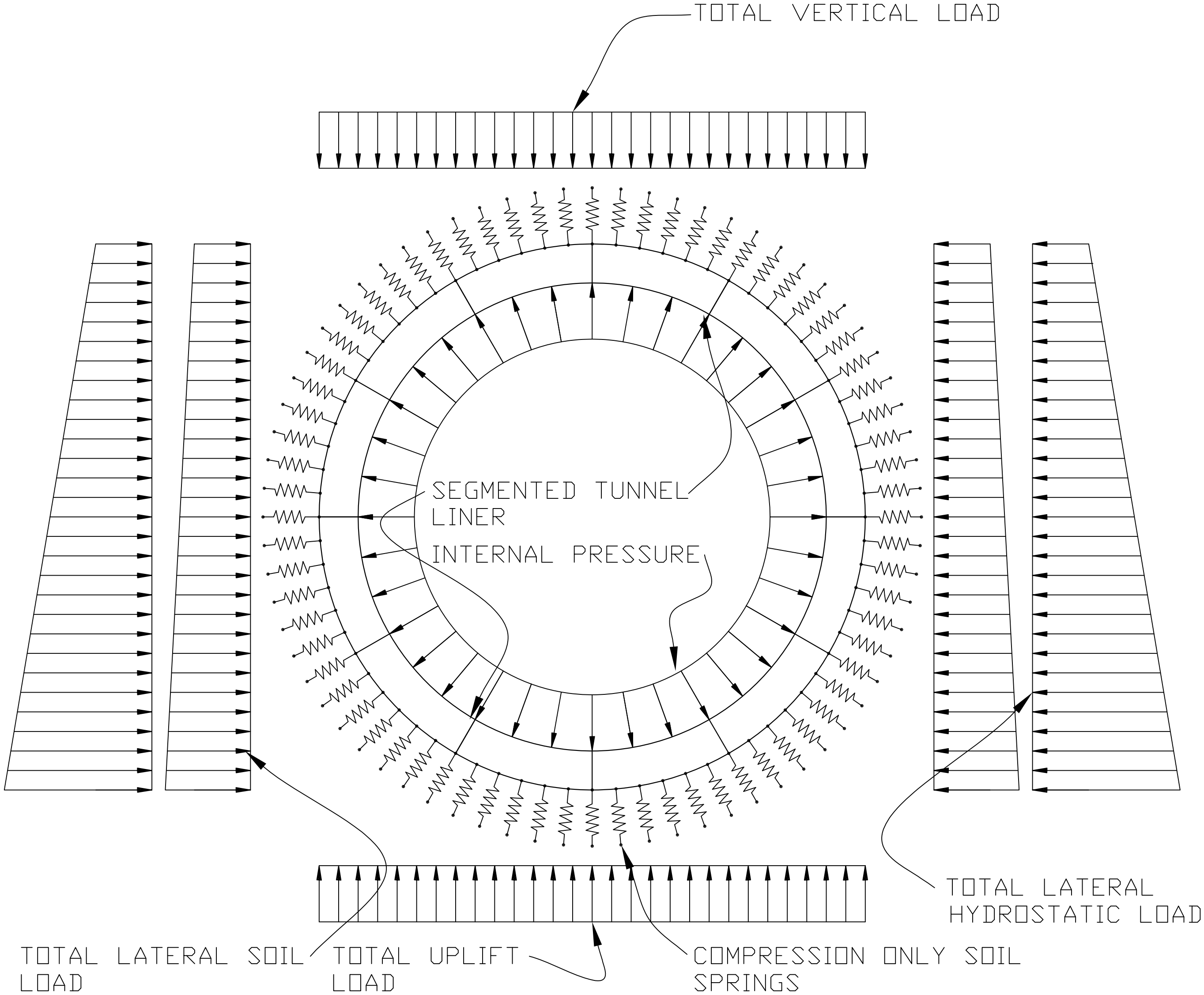
Final Condition External Pressure (Dewatered)

FIGURE **4-2**



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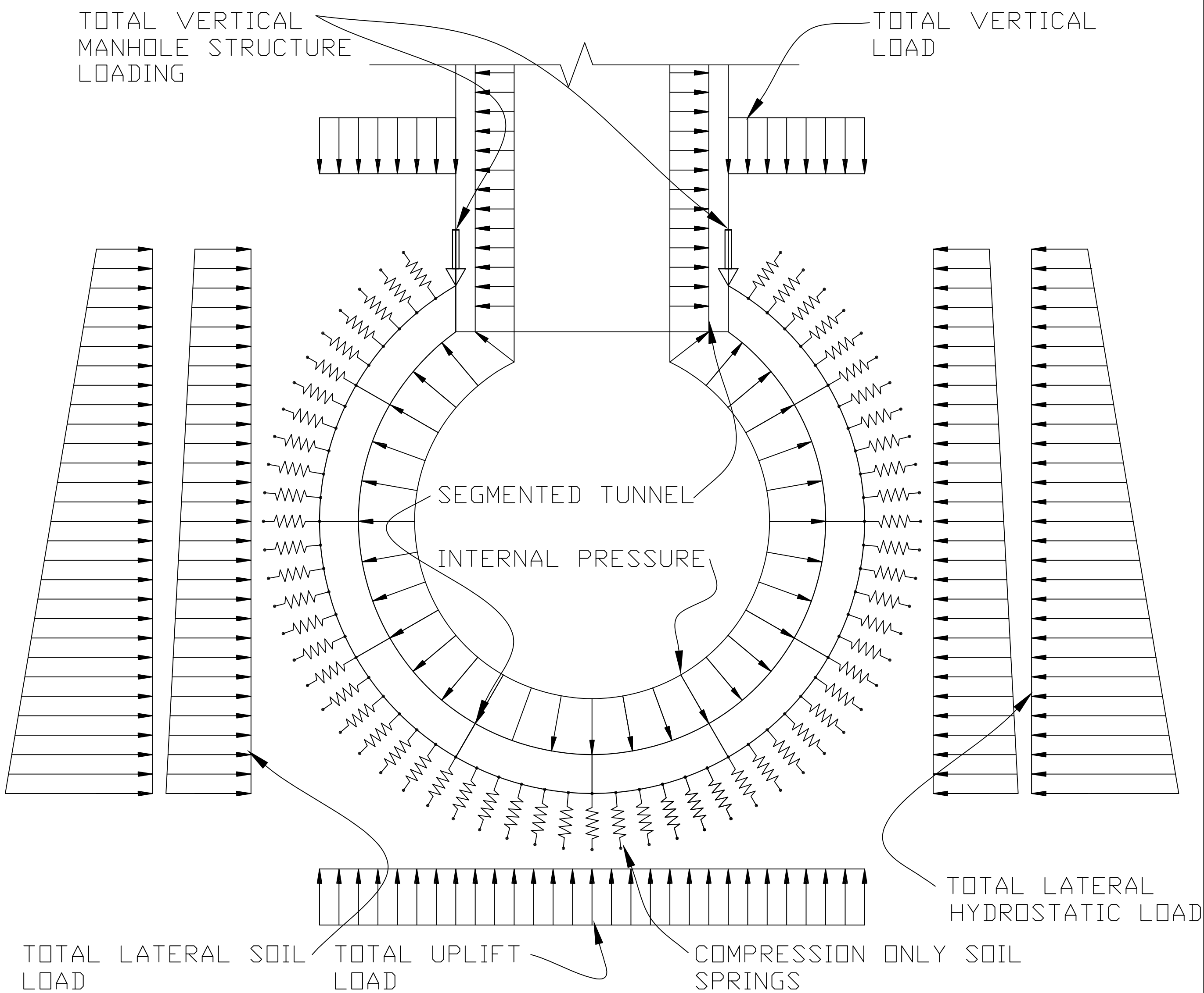
Final Condition External Pressure

FIGURE
4-3



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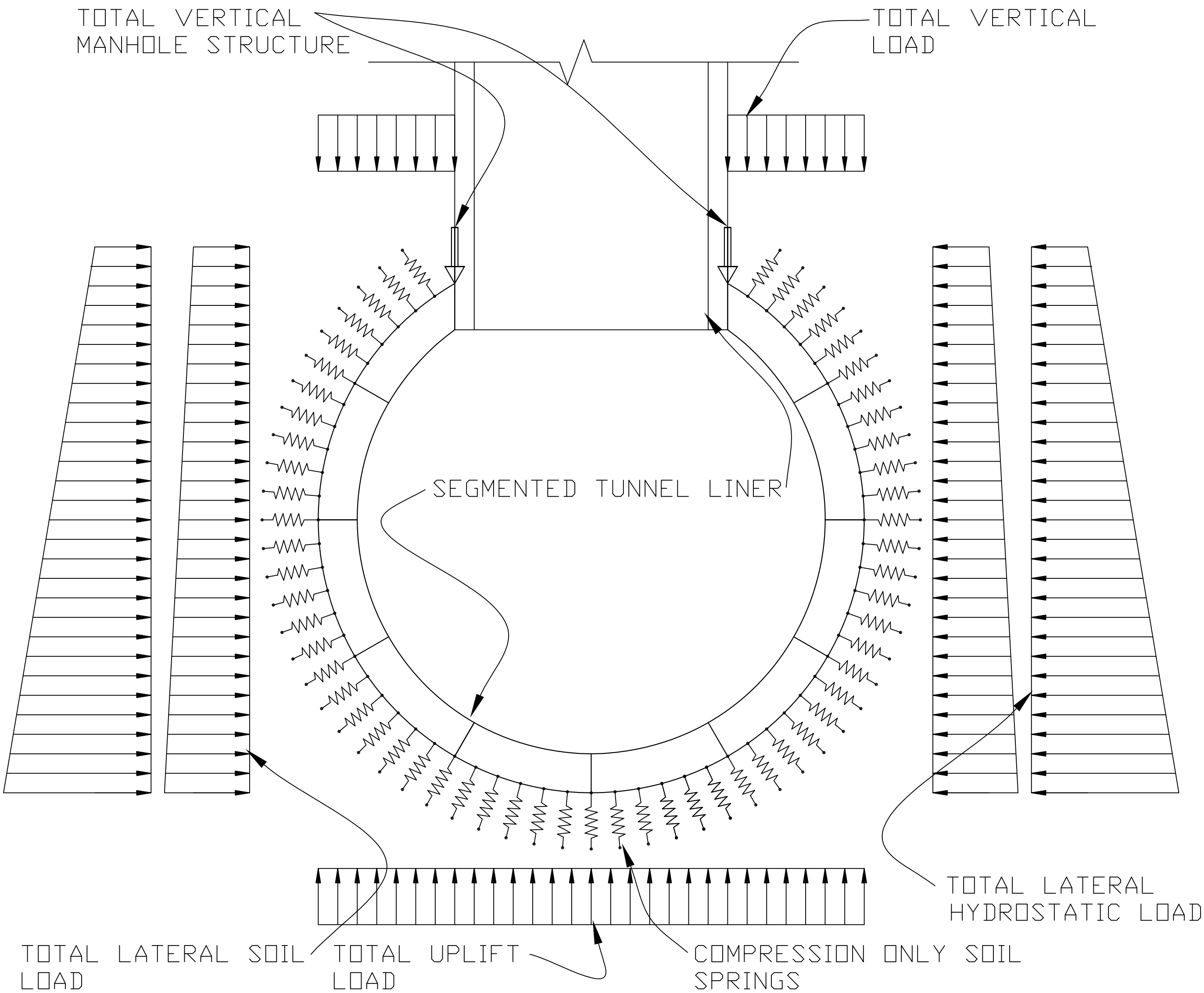
Final Condition at Manhole Internal Pressure

FIGURE
4-4



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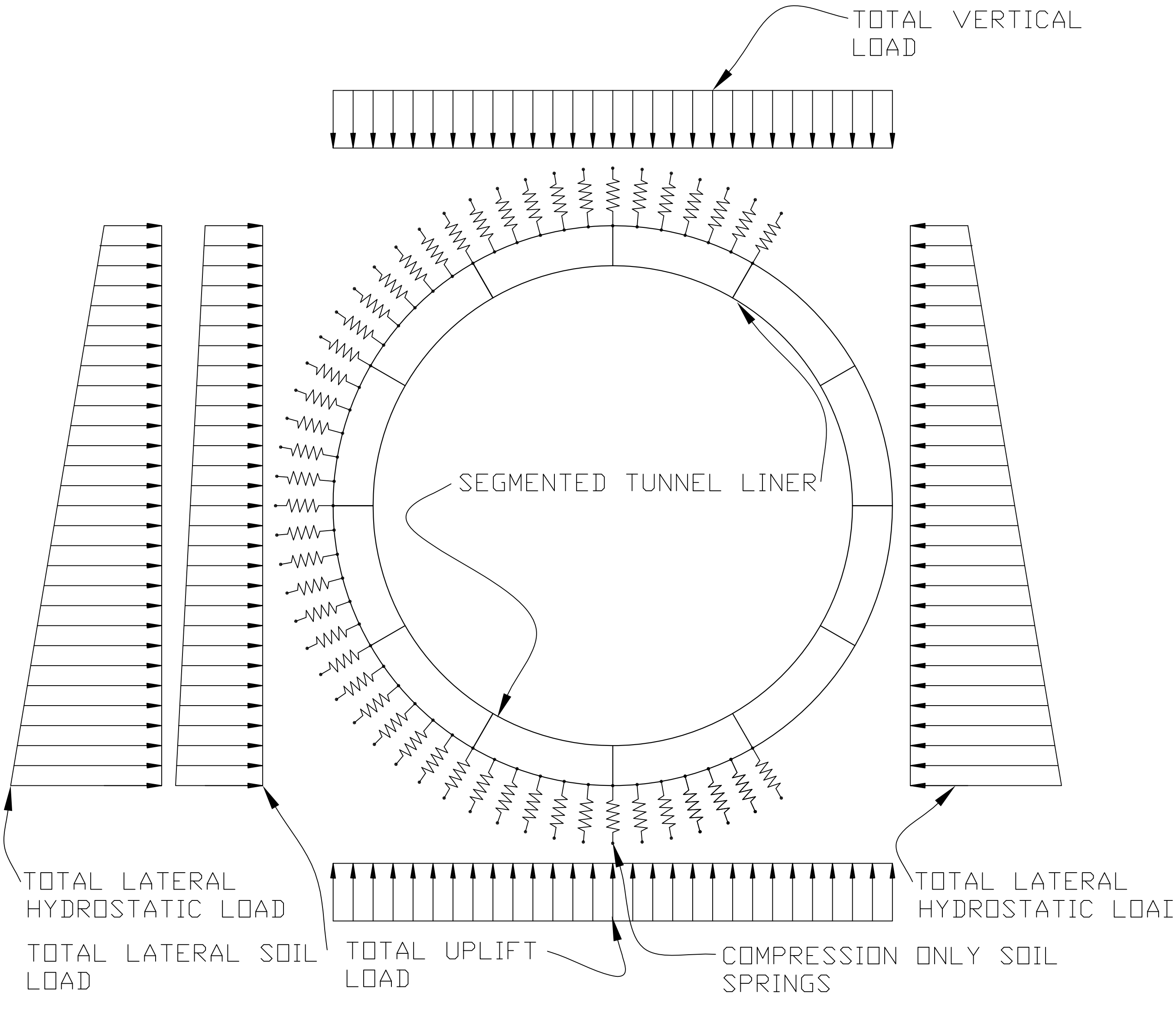
Final Condition at Manhole External Pressure (Dewatered)

FIGURE
4-5



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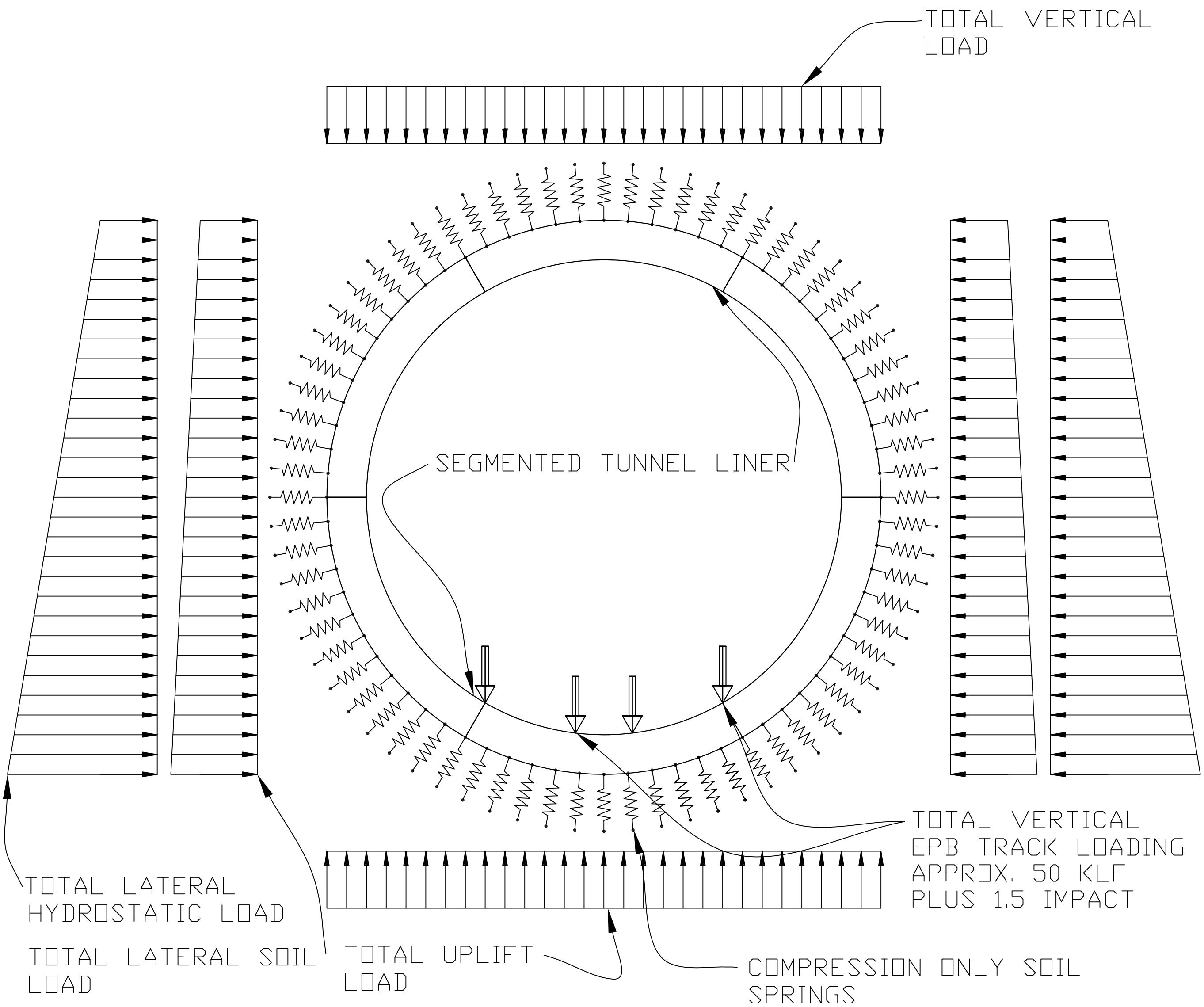
Parallel Tunnel—No Side Confinement Construction Condition

FIGURE **4-6**



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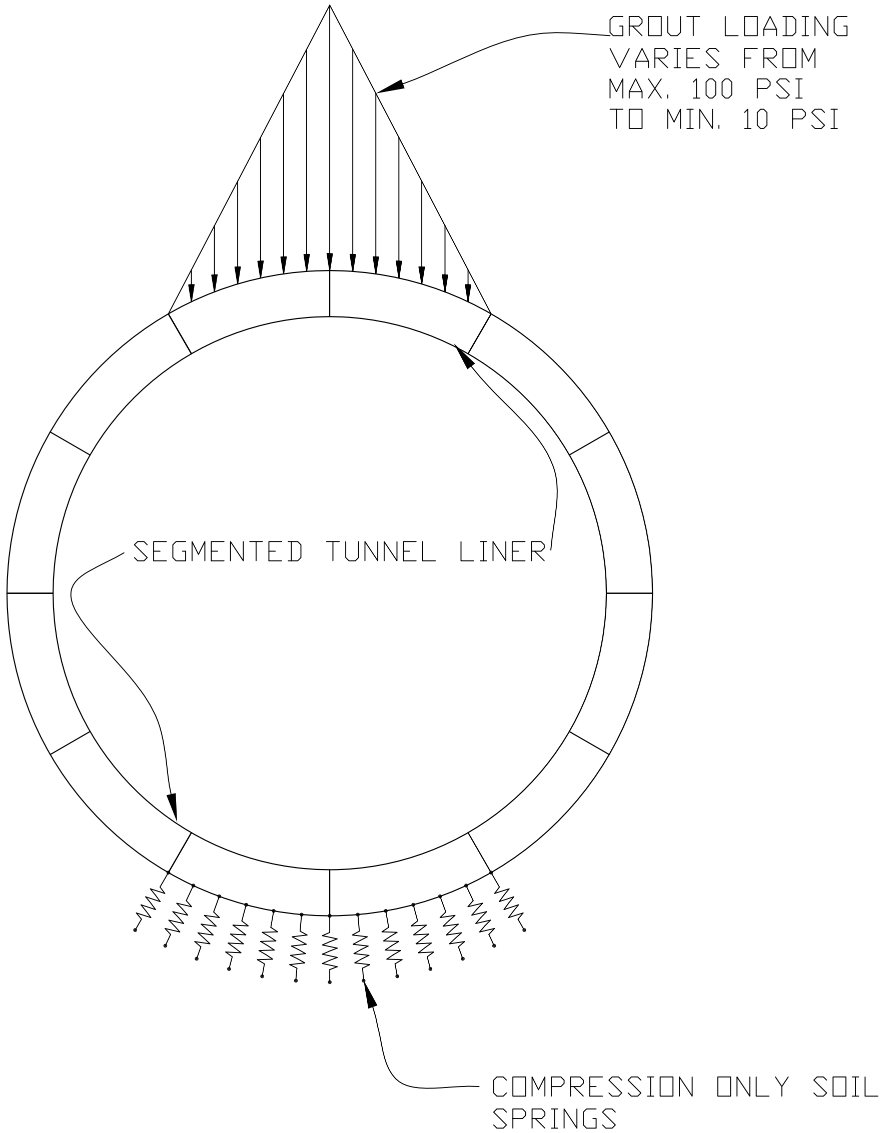
EPBM/Muck Car Track Loading on Segments During Construction

FIGURE
4-7



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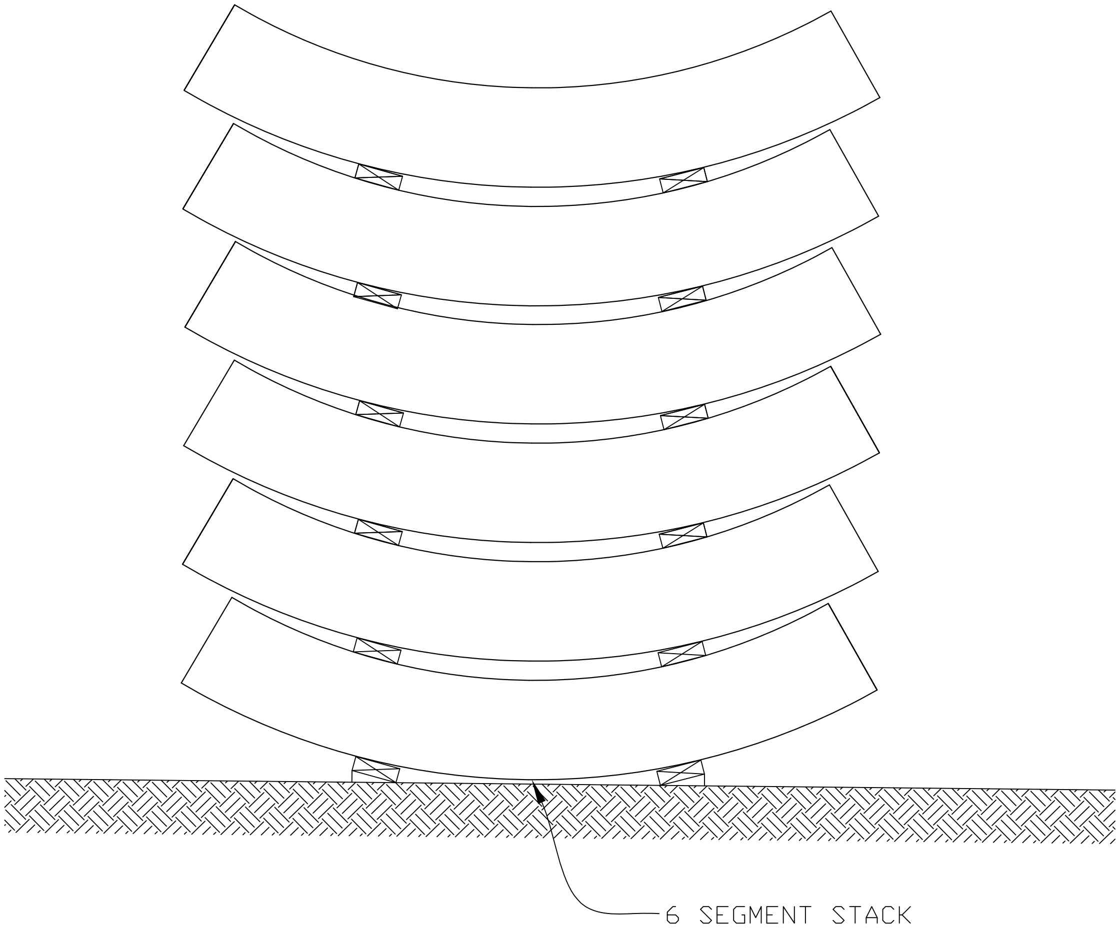
Grout Load on Segments During Construction

FIGURE
4-8



NOTES:
1. SEGMENT f'c~=2000 psi WHEN STORED AT PLANT
   f'c~=5000 psi WHEN STORED AT SITE
2. INCLUDE A 1.5 IMPACT FACTOR FOR PLACEMENT
   OF INDIVIDUAL SEGMENTS ON STACK.

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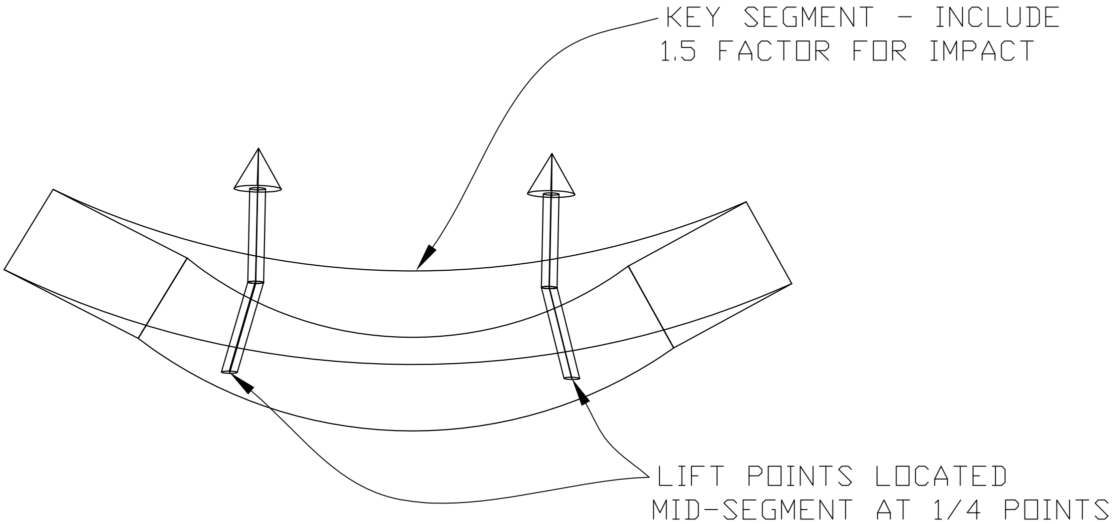
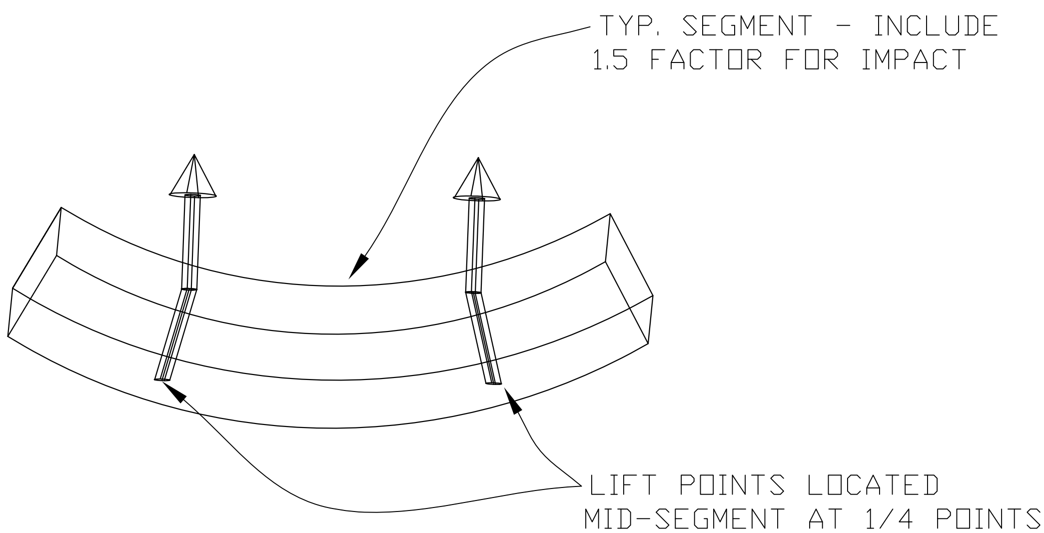
Storage Load on Segments in Yard

FIGURE
4-9



KEY SEGMENT HANDLING



TYPICAL SEGMENT HANDLING

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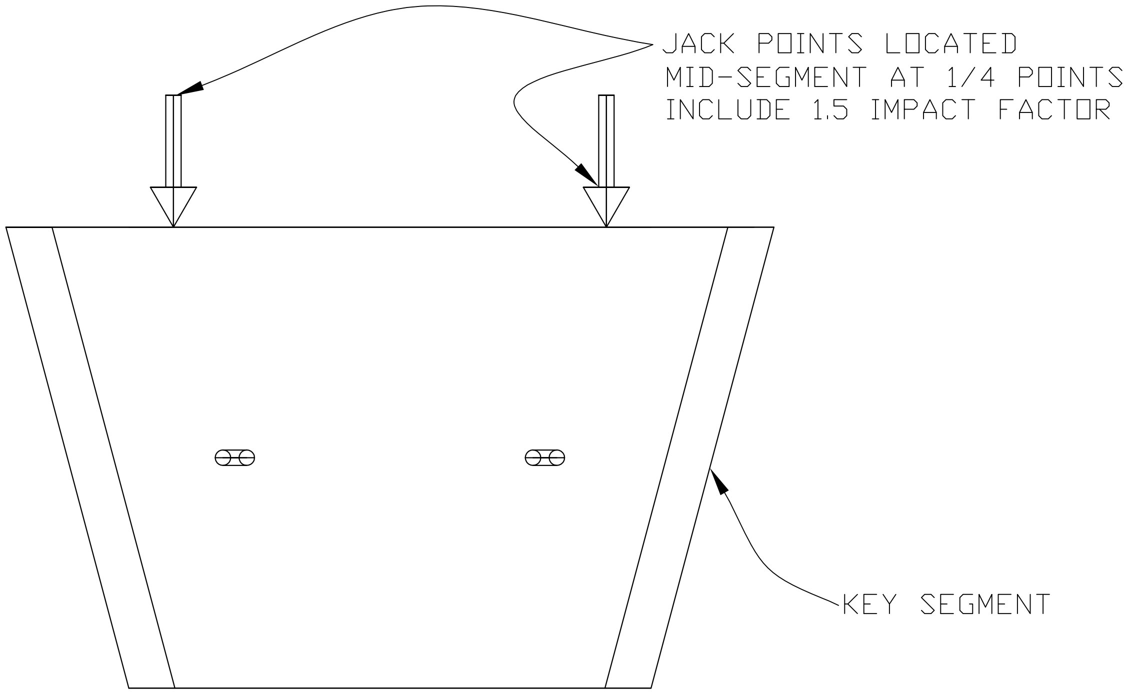
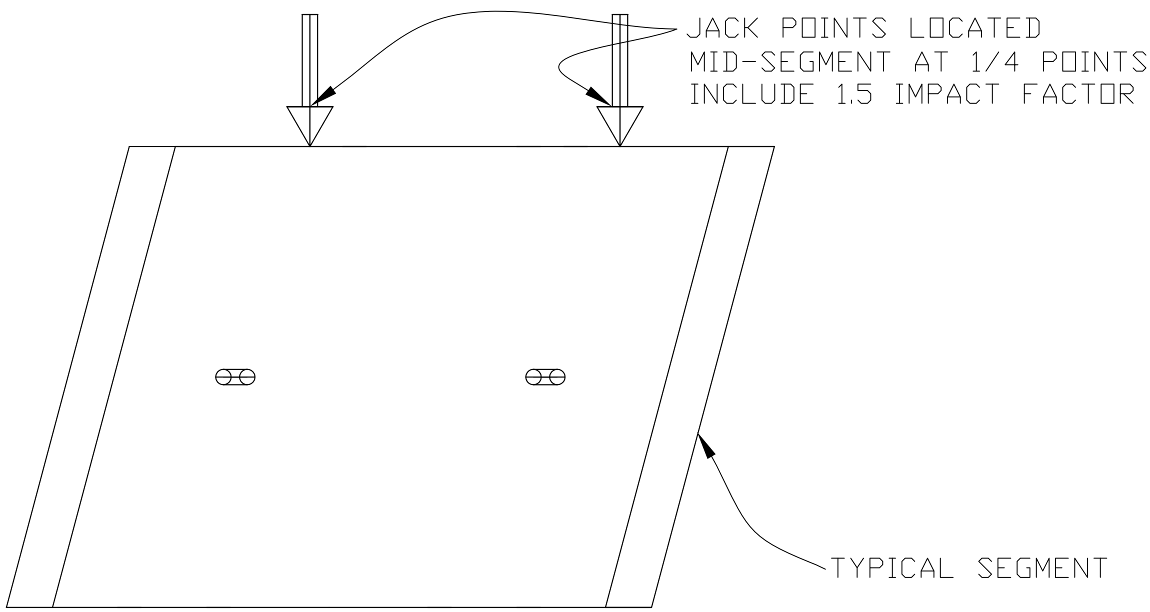
Handling and Lifting Load on Individual Segments

FIGURE

4-10



KEY SEGMENT HANDLING



TYPICAL SEGMENT HANDLING

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TBM Jacking Loads During Construction

FIGURE
4-11

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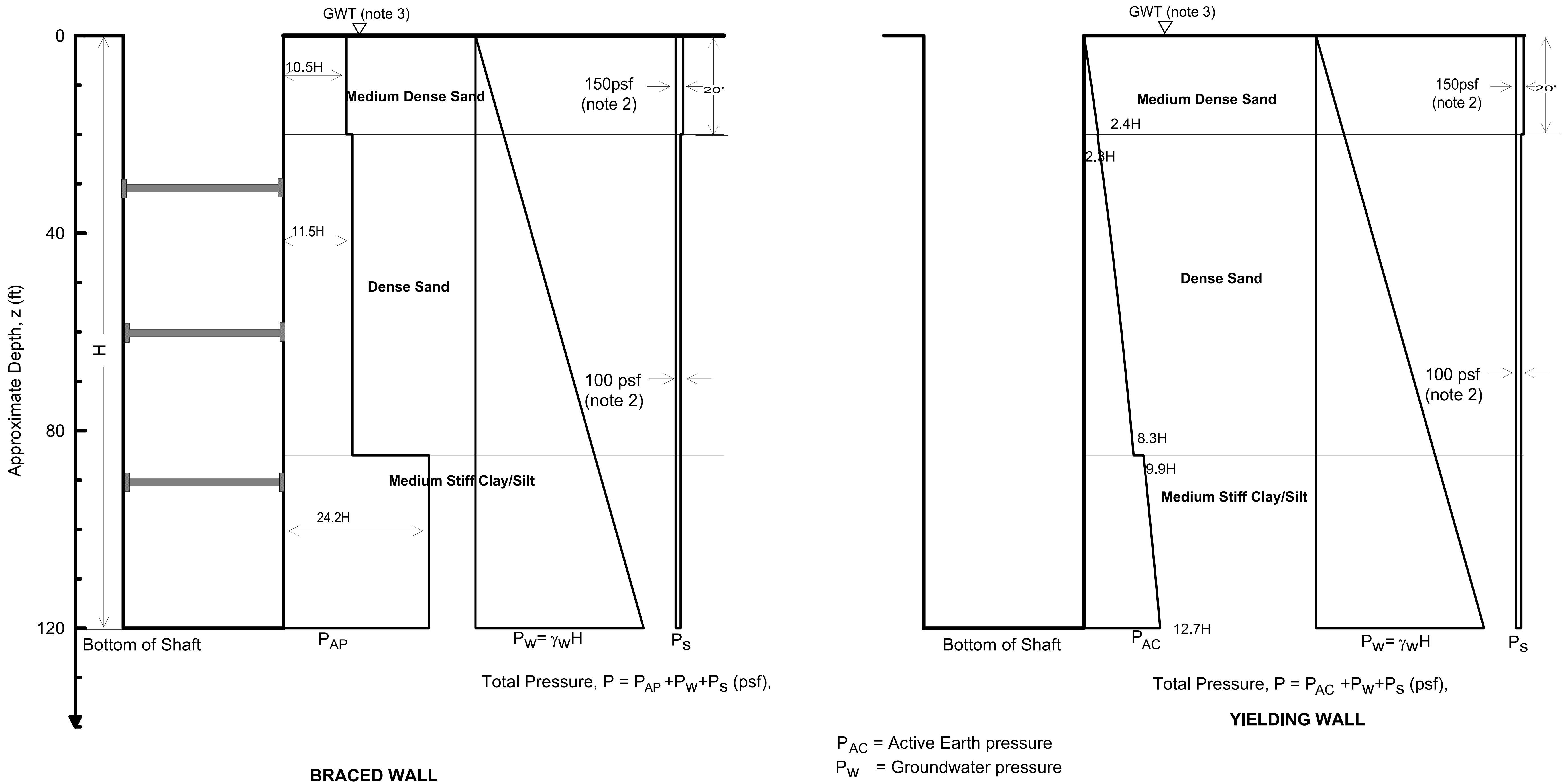
Preliminary Lining Thickness for Varying Tunnel Diameters and Depths

FIGURE
4-12



P_{AC} = Active Earth pressure
P_W = Groundwater pressure
P_S = Surcharge pressure
P_{AP} = Apparent Earth pressure

Notes:

1. Interpreted subsurface conditions and corresponding earth pressure diagrams are preliminary in nature and are based on generalized subsurface data.
2. Surcharge pressure is based on a uniform adjacent vertical surcharge of 400 psf. This surcharge pressure should be increased if large equipment or stockpiled soils are adjacent to the excavation support system.
3. The design groundwater depth (H = 0 ft.) assumes that no exterior dewatering will be conducted and that the shaft does not have an adequate drainage system. Actual groundwater levels are expected to vary from the design condition shown.

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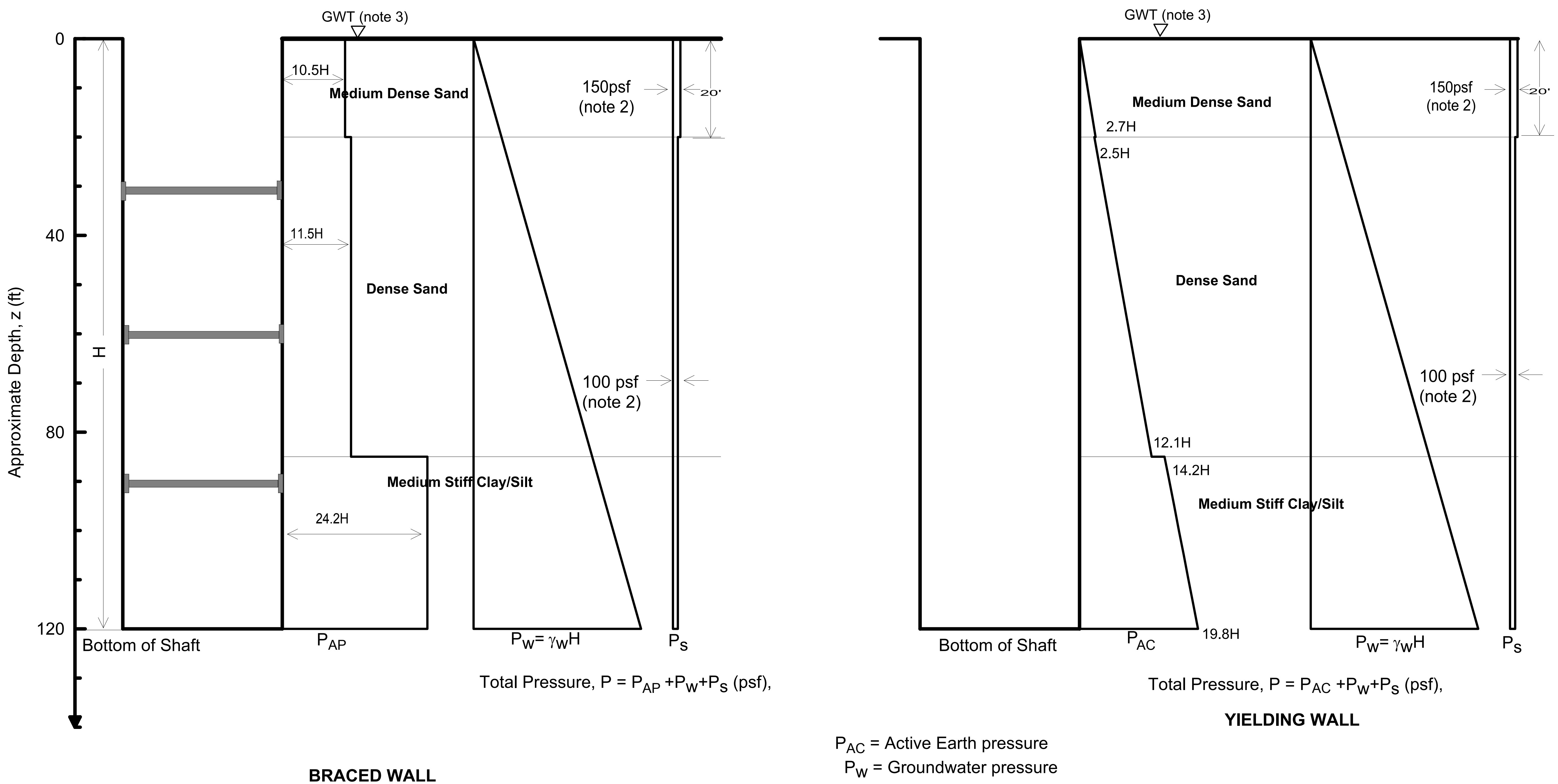
Typical Lateral Pressures for Design of Cylindrical Shaft Excavation Support at Boring DCA-DH-004

FIGURE 4-13



P_{AC} = Active Earth pressure
P_W = Groundwater pressure
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Notes:

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2. Surcharge pressure is based on a uniform adjacent vertical surcharge of 400 psf. This surcharge pressure should be increased if large equipment or stockpiled soils are adjacent to the excavation support system.

3. The design groundwater depth (H = 0 ft.) assumes that no exterior dewatering will be conducted and that the shaft does not have an adequate drainage system. Actual groundwater levels are expected to vary from the design condition shown.

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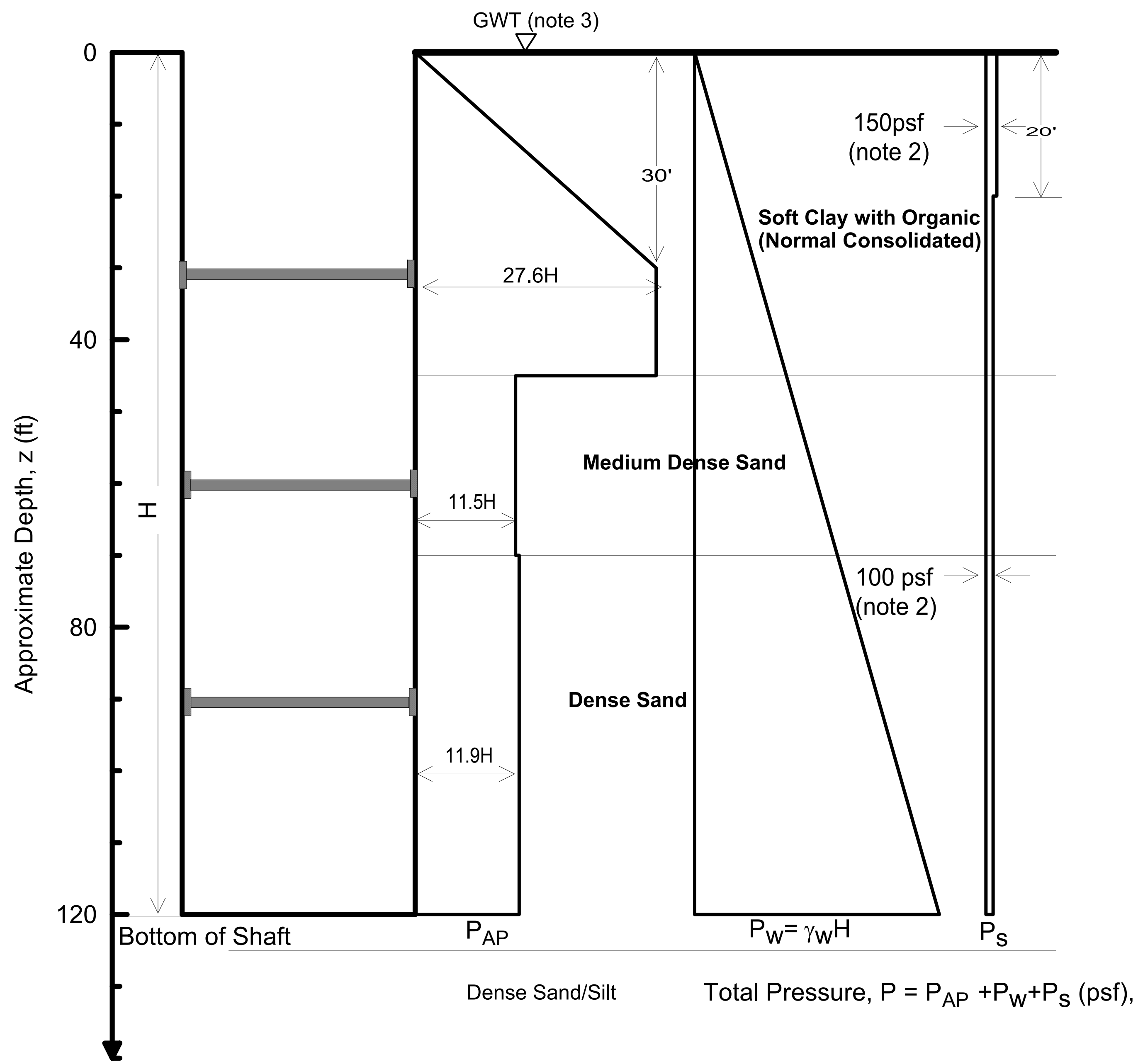
California Department of Water Resources
Advancing the Bay Delta Conservation Plan
Delta Habitat Conservation & Conveyance Program

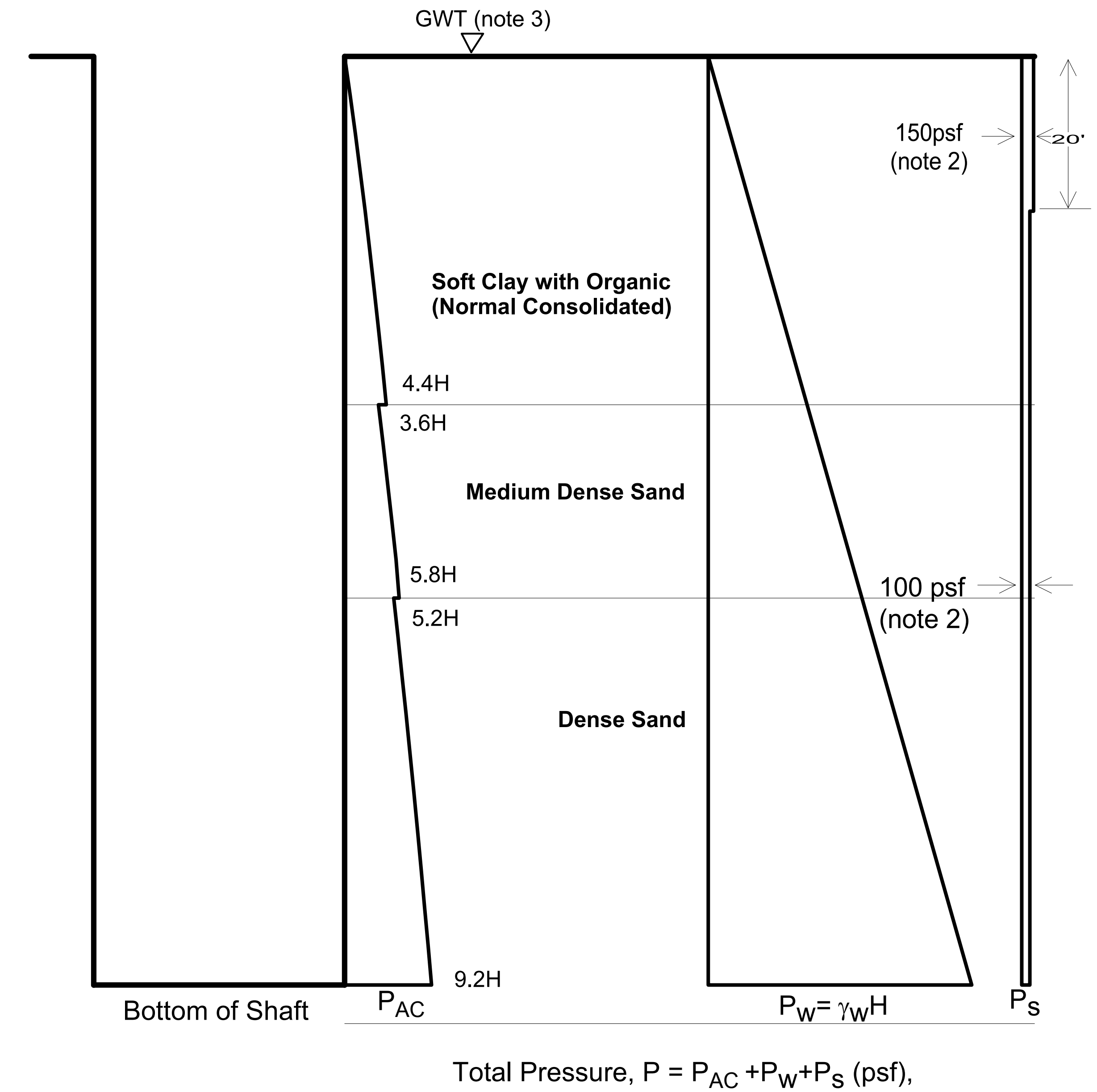
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Typical Lateral Pressures for Design of Non-Cylindrical Shaft Excavation Support at Boring DCA-DH-004

FIGURE
4-14



BRACED WALL



YIELDING WALL

P_{AC} = Active Earth pressure
P_W = Groundwater pressure
P_S = Surcharge pressure
P_{AP} = Apparent Earth pressure

Notes:

1. Interpreted subsurface conditions and corresponding earth pressure diagrams are preliminary in nature based on generalized subsurface data.
2. Surcharge pressure is based on a uniform adjacent vertical surcharge of 400 psf. This surcharge pressure should be increased if large equipment or stockpiled soils are adjacent to the excavation support system.
3. The design groundwater depth (H = 0 ft.) assumes that no exterior dewatering will be conducted and that the shaft does not have an adequate drainage system. Actual groundwater levels are expected to vary from the design condition shown.

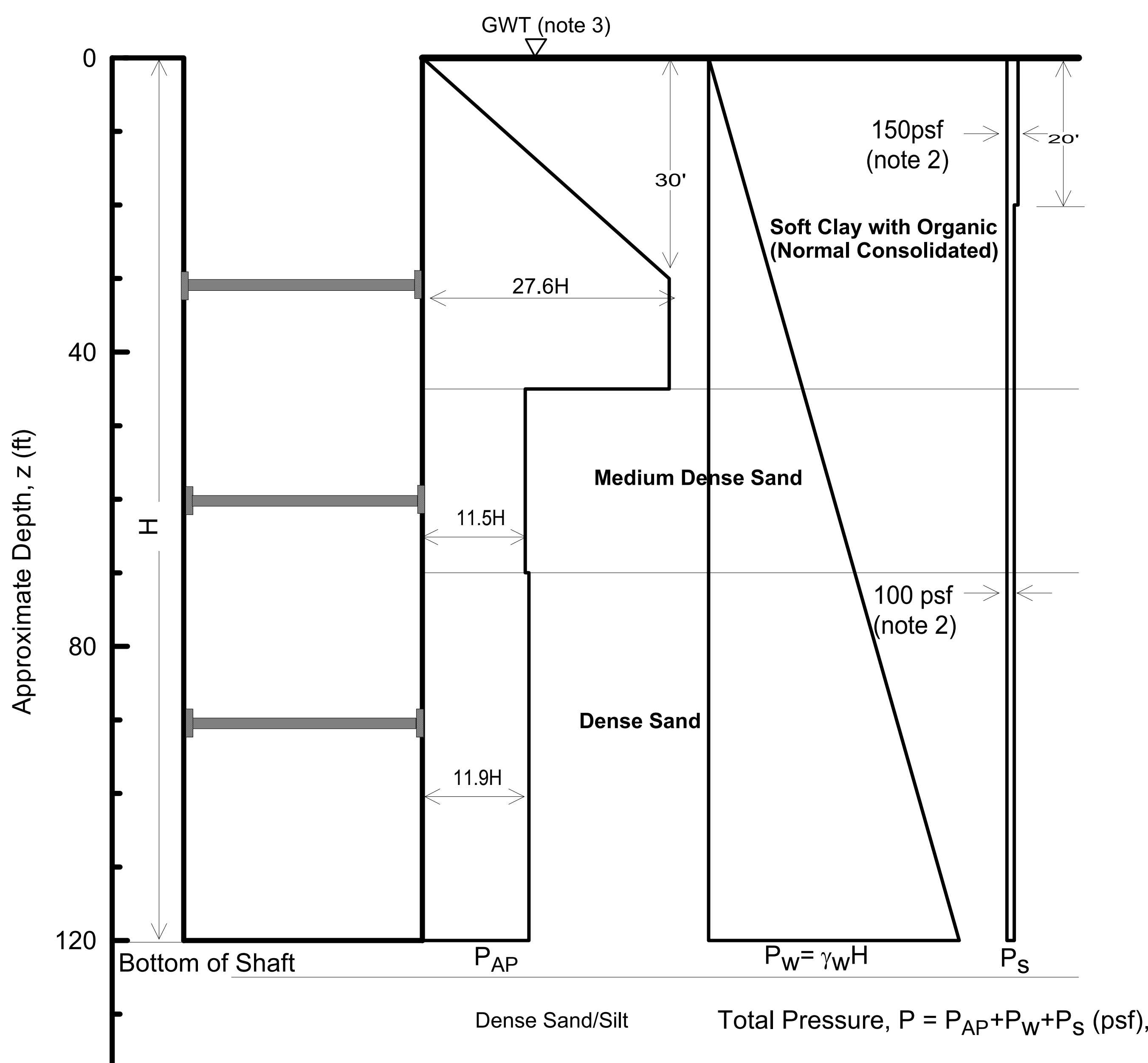
DHCCP Engineering
WORKING DRAFT – Subject to Change

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

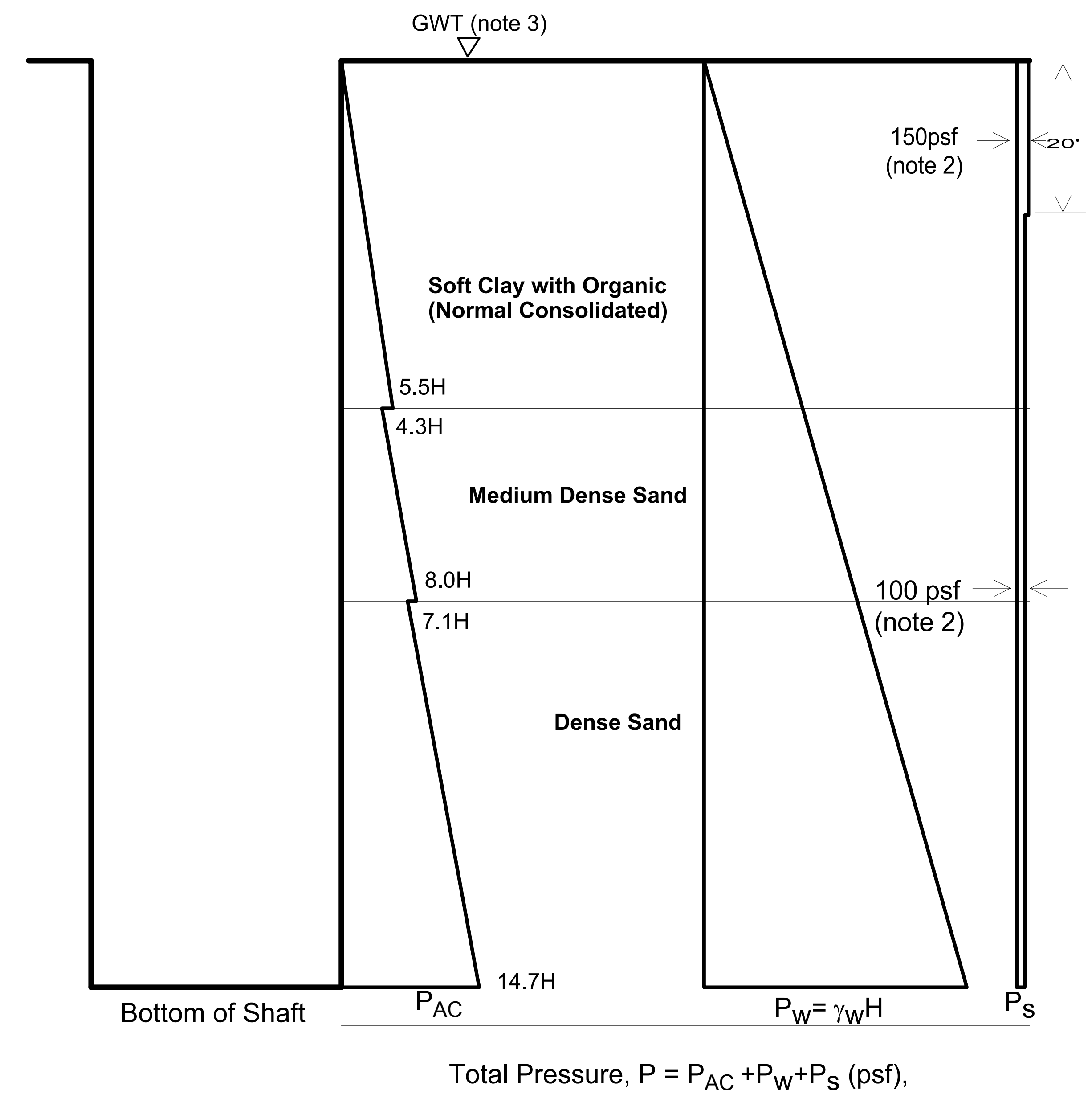
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Typical Lateral Pressures for Design of Cylindrical Shaft Excavation Support at Boring DCR-DH-016

FIGURE
4-15



BRACED WALL



YIELDING WALL

P_{AC} = Active Earth pressure
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Notes:

1. Interpreted subsurface conditions and corresponding earth pressure diagrams are preliminary in nature and are based on generalized subsurface data.
2. Surcharge pressure is based on a uniform adjacent vertical surcharge of 400 psf. This surcharge pressure should be increased if large equipment or stockpiled soils are adjacent to the excavation support system.
3. The design groundwater depth (H = 0 ft.) assumes that no exterior dewatering will be conducted and that the shaft does not have an adequate drainage system. Actual groundwater levels are expected to vary from the design condition shown.

DHCCP Engineering
WORKING DRAFT – Subject to Change

California Department of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	ANALYSIS & OPTIMIZATION REPORT PIPELINE/TUNNEL OPTION **Typical Lateral Pressures for Design of Non-Cylindrical Shaft Excavation Support at Boring DCR-DH-016**	FIGURE **4-16**

5.0 TUNNEL IMPLEMENTATION, ALIGNMENT AND PROFILE

Tunnel Implementation (Team B), essentially the "constructability" team, was charged with analyzing and optimizing implementation (construction) of the main conveyance tunnel. This analysis focused on five areas:

- Profile
- Construction Program Packaging
- Shaft Arrangement
- Flood Protection
- Potential Realignments

Team B gathered data, ideas and opinions through a series of meetings and teleconferences and presented these to the Outside Reviewers on October 5, 2010. (Appendix A) After reviewing the input from the Outside Reviewers, Team B continued its analysis and reached consensus on the recommendations in this section and presented the findings to the Outside Review Team at the final review meeting on December 7th and 8th.

5.1 General

Participants in the team represented a wide variety of backgrounds and points-of-view and are listed in Section 3.0. Refer to Figure 2-1 for overall tunnel alignment and shaft locations.

This diverse group came with a wide range of experience and expertise, which provided the team a solid basis from which to make recommendations.

5.2 Profile

Tunnel profile has an effect on cost of construction. Construction shafts become more expensive as the depth becomes greater, and interventions out to the face of the TBM become more expensive and time consuming as depth increases. Team B sought to reduce construction costs by optimizing profile.

Some of the questions to consider included:

- Can tunnel depth be decreased (from the CER Profile), in order to reduce cost?
- What tunnel profile is most advantageous for future maintenance dewatering?
- Are vertical drop shafts (to clear below the San Joaquin River) acceptable, or are vertical curves or other profiles preferable?

- Are seismic and/or liquefiable soils a concern with various profiles?

The Conceptual Engineering Report (CER) lists the tunnel centerline at -80 ft (elevation - NAVD88), in general to stay below peat layers and is suggested as the minimum tunnel depth. At the San Joaquin River, the depth is suggested to be approximately El -120 ft to provide some depth of cover below the line of flow of the river. The change in elevation from El -80 ft to El -120 ft is shown as a vertical drop shaft on the north side of the river (Venice Island Shaft), sloping up at a 2% grade until reaching El -80 ft on the south side. The depth of the San Joaquin River is shown as El -40 ft, giving the tunnel approximately 60 ft of cover.

In October of 2009, members of the City of Stockton Port Commission voted to oppose any conduit crossing the Stockton Deep Water Ship Channel buried less than 110 feet below the channel's average low tide level. In the San Joaquin River, average low tide is around 0 ft, which would limit the crown of the tunnel to no higher than El -110 ft with the centerline at El -130 ft.

Team B members expressed concern about the forces acting on the shaft and tunnel invert at the 90-degree changes in direction (horizontal-to-vertical and vertical-to-horizontal) at the San Joaquin River. Alternatives to vertical drops should be analyzed.

From an operations standpoint, tunnel dewatering becomes more difficult with the low spot at the San Joaquin River. Water pumped out of the tunnel in the middle of the Delta might have to be wasted (pumped back into the river system). Dewatering the tunnel at either Byron Tract Forebay or Intermediate Forebay would be preferred.

Team A will be optimizing tunnel elevation to account for internal operating pressure. This will have an effect on the Team B profile.

In preparation for the October 5 first review meeting, three potential profiles were prepared.

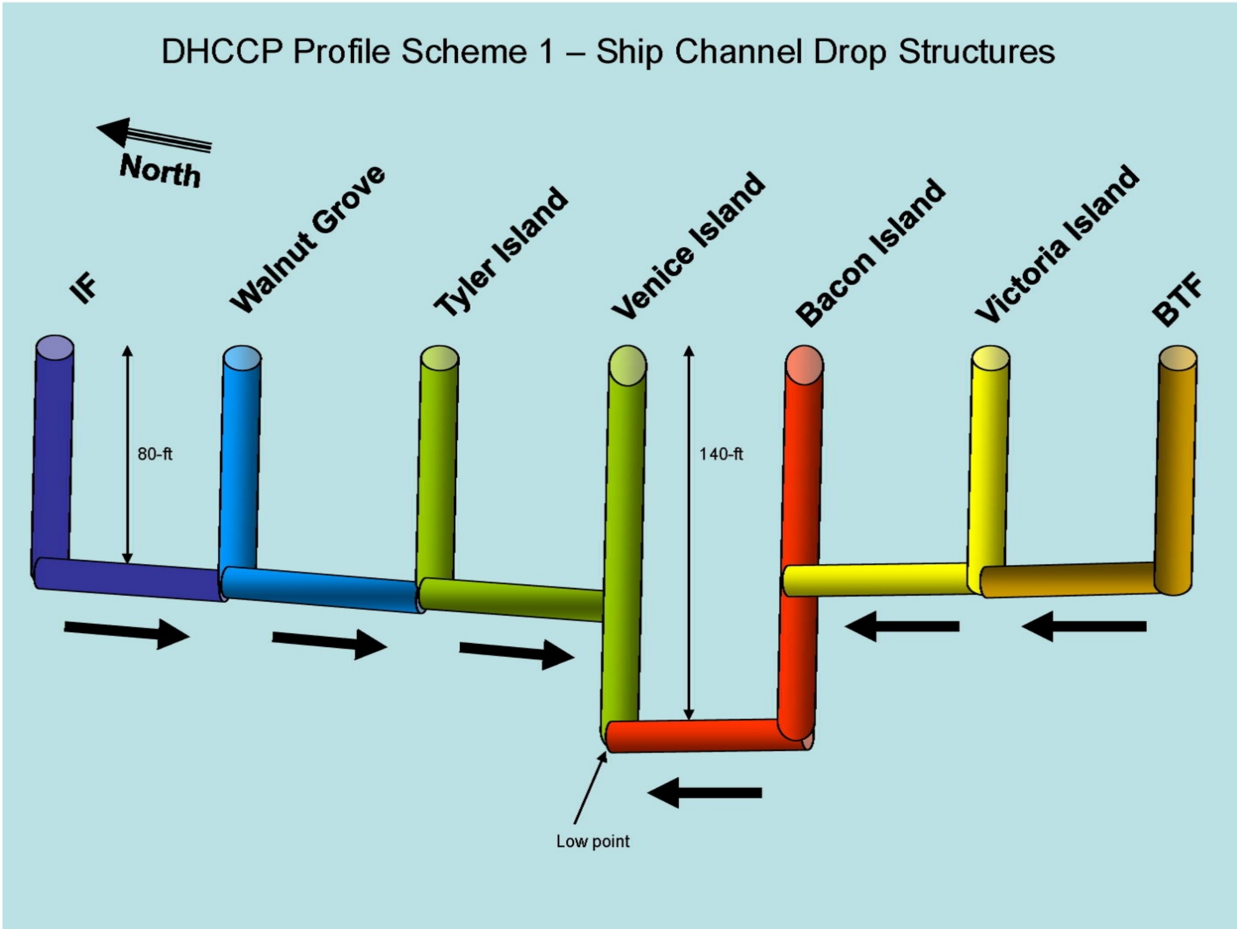


Figure 5-1a – DHCCP Profile Scheme 1

Profile Scheme 1 (all depths shown are elevation NAVD88) maintains most of the tunnel at the CER level of El -80 ft and makes use of two drop structures to deepen the profile at the San Joaquin River. This has the advantage of minimum depth for most of the alignment and greater depth between Venice Island and Bacon Island only.

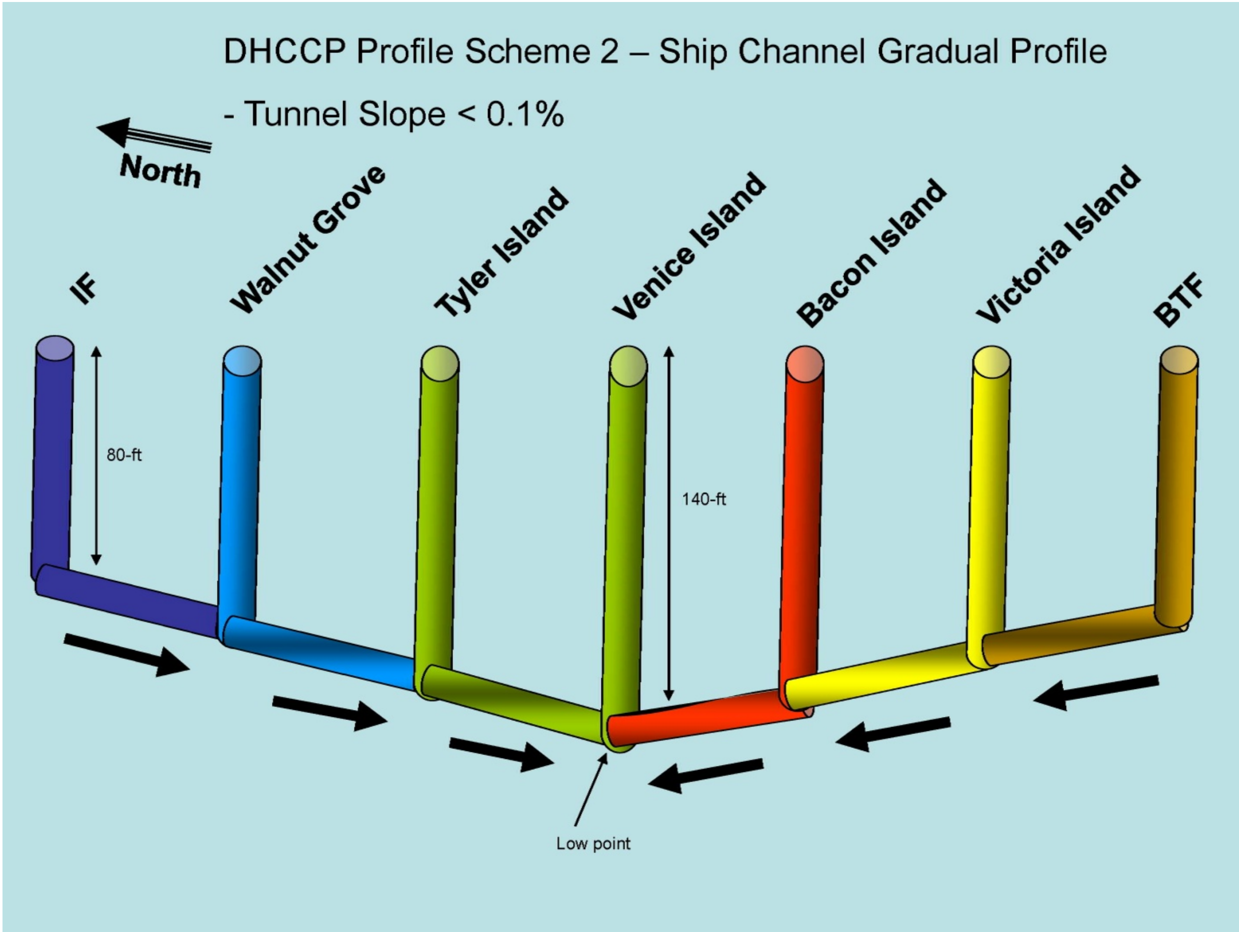


Figure 5-1b – DHCCP Profile Scheme 2

Profile Scheme 2 (all depths shown are elevation NAVD88) makes use of a straight grade from both ends, sloping down to a low point below the San Joaquin River. This allows the tunnel to pass under the San Joaquin River at the required depth and avoids any vertical transition structures. A disadvantage is that all dewatering would be done in the middle of the Delta, and water would either be pumped into the second tunnel or spilled into the river.

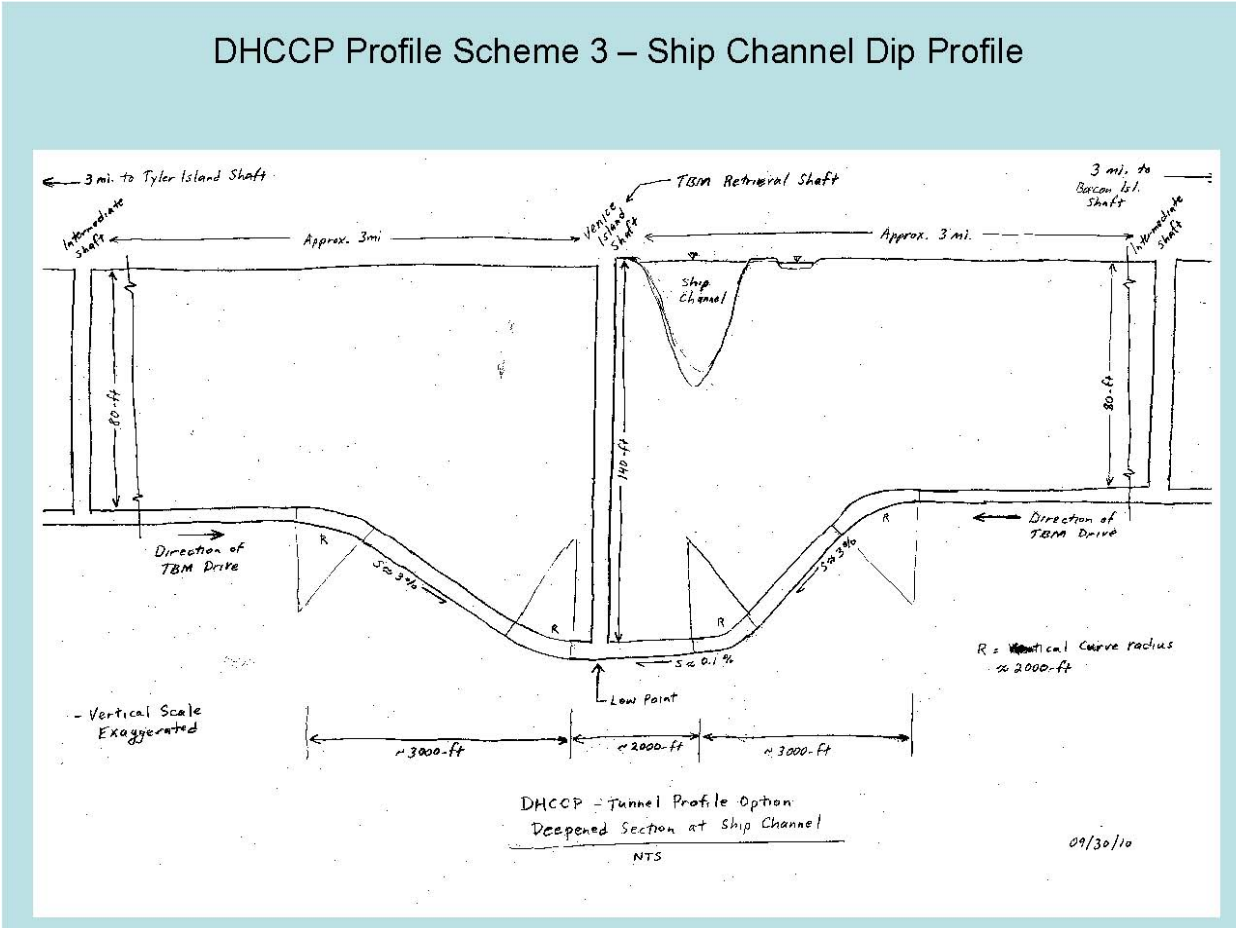


Figure 5-1c – DHCCP Profile Scheme 3

Profile Scheme 3 (all depths shown are NAVD88) is a hybrid, maintaining minimum depth for the greatest portion of the alignment and making use of the turning capabilities of the tunnel boring machine to create a 2,000-ft radius and a 3% slope to get under the San Joaquin River. This alignment generally slopes down from the San Joaquin River to the Intermediate Forebay in the north and from the San Joaquin River to Byron Tract Forebay in the south. The siphon portion could be as short as 8,000 ft. In Scheme 1, the siphon portion between the drop structures (which would have to be dewatered in the middle of the Delta) is approximately 6 miles. In Scheme 2, the entire 33.5 miles slopes down to the middle and would have to be handled at the San Joaquin River. Scheme 3 reduces the amount of water that would be trapped in the siphon under the San Joaquin River that would need to be handled in the middle of the Delta.

The Outside Review Team had several recommendations related to the optimization of the tunnel profile:

- Vertical drop siphon configuration within shafts is common; this does not pose a significant hydraulic loss.

- For shaft construction, shallower is better.
- 3% slope and 2,000 ft radius to get under San Joaquin River is workable.
- Consider starting at El -160 at the Intermediate Forebay and basically straight grade up to El -80 at Byron Tract Forebay
- Create a matrix to help with a structured decision making process to determine profile since so many factors play into the profile decision (internal pressure, lining, soils, operational concerns, constructability, etc.)

A set of profiles was then created (see Appendix F) and reviewed with the team. Team A provided information related to liquefaction depth. A concern of the Outside Review Team was that the tunnel needed to be below the potential liquefaction zone of the soil. Based on the very limited geotechnical information, Team A estimated the liquefaction depth to be elevation -100 ft (tunnel crown depth). Subsequently, all profiles were edited to comply with the liquefaction limit (see Section 4.5.4). At this time, a matrix was created (See Appendix H) to identify the various parameters that feed into the decision making process.

Finally, the eight potential profiles (with the liquefaction limit) were presented to the team and evaluated based on the criteria listed in the matrix.

Based on input from Team A, the issue of internal tension spreading the joints of the precast segmental liner is only minimally helped by increased tunnel depth since confining soil pressure potentially will not be counted on to counteract internal pressure (for more information, see Section 4.3.4.2). Therefore, as far as the construction group is concerned, shallower is better.

After reviewing the matrix (Appendix H) and considering the above information, the following profiles were eliminated:

- Profile "B" was considered to be similar to Profile "C", but slightly deeper at the Intermediate Pumping Plant.
- Profile "D" trapped water at the San Joaquin River, and the vertical transitions had additional construction difficulties.
- Profile "E" is a flat profile, which doesn't allow for efficient dewatering.
- Profile "G" trapped all water at the San Joaquin River.

The preferred profiles were a modified "C" and a modified "H", with a modified "A" as an alternate (see Figures 5-2 a through c).

Profile H was preferred because this profile was straight with no transitions, curves or changed slopes; the depth is close to the minimum and all dewatering can be done at the Byron Tract Forebay. It also complies with the -110 ft depth limit at the San Joaquin River (see Figure 5-2b).

Profile "C" was also preferred, because there is no trapped water at the San Joaquin River, no vertical transitions and minimum depth. A disadvantage is that a dewatering scheme would be needed at each end and a venting mechanism would be needed at Bacon Island. In general, the tunnel slope would be slightly downhill from Bacon Island to Byron Tract Forebay and from San Joaquin River to Intermediate Pumping Plant (see Figure 5-2c).

Profile "A" is identical to Profile "H", except the slope is uphill from the IPP to the BTF. With Profile "A", dewatering would occur at the IPP. This has the advantage of the power supply being immediately adjacent and the disadvantage of pumping water out of the tunnel back into the Intermediate Forebay (from which it was previously pumped – see Figure 5-2a).

The recommendation of Team B is to proceed into Preliminary Engineering using Profile "H" as a basis. This profile has the best dewatering scheme, constructability and cost (see Matrix in Appendix H). One point to consider is that when the dewatering equipment is defined, a power supply should be considered. Profile "H" needs the dewatering equipment at the Byron Tract Forebay, where no major power substation is planned. The power substation is planned for the Intermediate Pumping Plant. If the power supply for the dewatering equipment becomes a major factor, Profile "C" could be reconsidered, since most of the dewatering would be done at the IPP.

5.3 Construction Program Packaging

Team B was assigned the task of reviewing how to best to accomplish construction of the PTO to allow the Construction Industry (contractors, bonding and insurance companies) to respond effectively to request for proposals.

This subsection includes input from the team and from knowledgeable outsiders leading to recommendations for:

- Contract Packaging
- Bonds and Insurance
- Segments, Shafts, Power and Early Site Work
- Construction Schedule
- Cost Impacts

A Risk Assessment Workshop in early 2010, identified a high risk issue to be the potential inability of the construction industry to respond to invitations-to-bid to construct tunnels as large and as long as these. The baseline concept constructs the entire project (2 – 33 ft tunnels) at one time, allowing some lag time between contracts to account for administrative functions. The following assumptions apply to the baseline concept for the main conveyance:

- Six contracts, each approximately $950 million, each tunneling two 6-mile parallel 33 ft ID bores;
- Each contract to be let approximately 1 month apart;
- Pre-cast concrete segmental liner is included in each tunneling contract.

The issue of Program packaging was presented to the Outside Review team in their October 5, 2010 meeting. The Review team expressed several concerns related to this subject including the following comments:

- The worldwide bonding market may not have sufficient capacity to support 100% bonding; the Program may pay as much as a 20% to 25% premium for bonding at 100%. Reduced bonding limits are in place at similar projects at Lake Mead, the Port of Miami, Alaskan Way, and in Canada.
- Attempting to contract for both bores of the tunnel at one time, potentially could saturate the bidding marketplace. Consider starting and completing one full tunnel bore and begin conveying water; then proceed with the second tunnel. This has the added benefit of allowing the Program to apply Lessons Learned to the second bore.
- Two bores may unnecessarily increase costs. Consider a single large-bore tunnel solution and avoid twin tunnels.
- Very large contract values may reduce potential bidding competition. Consider contracting for the manufacturing of pre-cast segments separately from the tunneling contracts.

The Outside Review team recommended:

- Begin now to engage management and sureties to examine and consider other than 100% bonding requirements for the program, given the expected size of the tunneling contracts.
- Develop a means to pre-qualify bidders for tunnel contracts.
- Spread contracts out—six (6) $950 million contracts or twelve (12) $475 million contracts—to avoid market saturation.
- Have Owner procure and furnish tunnel lining segments to all contractors.

Not wanting to merely extemporize on contract packaging, Team B reached out to the industry and solicited opinions from potential bidders and other industry leaders. Through its considerable network of contacts, individual team members interviewed several industry leaders asking what is possible, what is probable and what won't work at all. Some of the industry leaders contacted were:

- Executive Vice-President of a major international heavy-civil construction company
- President of a major US tunneling contractor
- Senior Vice-President of a major US heavy-civil construction company
- District Manager of a major international heavy-civil construction company
- Vice-President/Operations Manager of a major US heavy-civil construction company
- Bond Manager of a major international EPC contractor
- Bonding and Insurance Broker for an international surety company

The complete set of meeting conversation notes is included in Appendix I and will be discussed in the appropriate sections that follow.

Industry leaders were contacted regarding contractor outreach, prequalification, and design-build versus design-bid-build.

A consistent theme of the Outside Review Team as well as several of the contracting industry leaders was "do extensive contractor outreach and pre-qualify the teams." The basic question we asked each individual was "What can DHCCP do to make it more likely that the Construction Industry will respond positively to our RFP?" A brief description of the potential project (using values and descriptions that have been previously presented to the BDCP Steering Committee) was provided. Several key concepts became themes among the responders:

- There is only so much tunneling expertise in the world. Expertise to physically put together bids (proposals) for projects of this size (either $950 million each or $475 million each) is limited. Contractors need time to do a thorough job, otherwise the Owner will pay a premium because the proposers have not had enough time to become comfortable with the information; this results in the addition of more risk money to the bid. Also, given that the expertise to construct these tunnel projects is limited, putting out twelve (12) $475 million or six (6) $950 million projects, basically at one time, will strain the contracting community to the point that they will not be able to respond. In the end we can expect to get inexperienced/unqualified contractors if we let $5.7 billion worth of tunneling all at one time.

- There needs to be a pre-qualification process that generates, preferably, a short list joint venture teams. A 'rip-n-read' style procurement based solely on low bid pricing will likely result in one or more unqualified contractors.

- An extensive Geotechnical Baseline Report (GBR) that allows bidders to quantify their risk is critical. This pays dividends in the bids, as contractors feel like they know what to expect and therefore can take some of the risk money out of their bid.

- The contracting community can handle $2.85 billion (if the spacing between procurement is closer to 3 months). They can handle three (3) $950 million projects or six (6) $475 million projects. It is likely the contracting community could not handle the entire $5.7 billion worth of tunnels at one time.

- The bonding industry cannot handle full 100% payment and performance bonds for $5.7 billion on one program. Bonding companies want to spread their risk—not too much work with one contractor, not too much work in one location, not too much work with one owner. The bonding companies may request a penal sum limit of 10% to 20% of the contract amount. There is precedent for reducing the bonding amounts, but this would require legislative approval.

- Contractual language must be fair. Lop-sided risk sharing on the part of the Owner causes problems with the contracting community.

- Unrealistic schedules discourage competition.

- The contractors did not universally prefer the pre-cast concrete segments be contracted separately and provided as "owner-supplied material." While separating out the segments would reduce the size of the tunnel contracts substantially (see Section 5.3.6), the contracts would still be part of the same Program and, thus, may not make much difference to the bonding community. Leaving pre-cast segments in the tunnel contracts reduces the risk for the Owner, because the coordination between the segment manufacturer and the tunneler is eliminated. One Contractor preferred separating the segments from the tunneling contract. He believed that the costs could be lower overall and that the competition for the segments would be sufficient. The benefit is that the competition for the tunneling contracts may be greater, given the lower contract values.

- Contractors prefer some latitude to use their creativity and past experience in providing solutions to project challenges. Allowing some form of design-build encourages contractor innovation, which can benefit the Owner.

5.3.1 Contract Packaging Recommendations

Based on the experience of Team B members and the input from the contracting and bonding communities, the recommendations for contractor packaging are:

- Focus on one bore of the tunnel first and aggressively pursue bidding the second bore as market conditions allow.
- Break up each bore into six contracts.
- Consider removing pre-cast segments from tunneling contracts.
- Separate the award of contracts by three months.

One of the goals of the Program is to construct the project as quickly as possible. Opinions differ on the ability of the contracting industry to perform this much tunneling work at one time. $5.7 billion represents the total construction cost of the main conveyance tunnels, including contingency. The other construction work contemplated in the program (pumping plants, intakes, pipelines, etc.) is estimated to be approximately $3.5 billion (including contingency).

Considering that all $9.2 billion must be executed in relatively the same time period, this represents a considerable challenge for the construction industry.

It is clear that the construction community is creative and resourceful and can be a huge ally to the Program. Extensive contractor outreach and industry review, risk-sharing and fairness in contracting documents, and thorough geotechnical investigations will go a long way towards helping the Program reach the goal of constructing the project in the shortest amount of time.

The team agrees that twelve $475 million contracts would allow more firms to participate, due to the lower construction value. It may be that a construction joint venture could be awarded more than one contract (with the lead contractor for the JV changing from one contract to the other).

Removing precast concrete segments from tunneling contracts presents a higher risk position for the Owner, but has its advantages, including:

- Better quality control
- More consistent finished product
- Reduced tunneling contract size

- Possible early start on segment manufacturing.

In addition to higher risk, removing the precast segments includes these disadvantages:

- Additional owner interface between segment manufacturer and tunnel contractor.
- Increased potential claims issues

Additional contractor outreach in the coming years is needed to verify this recommendation as the landscape of the community may change between now and then.

Waiting 3 months between procurements gives each contractor time to provide a complete bid and to become comfortable with the procurement documents. The 3-month period also allows the contractors to remove some risk contingency from their bids and gives the bonding community and the Owner's administrative team time to react.

5.3.2 Bonds and Insurance Recommendations

The current bonding market is limited. Only a few sureties currently offer bonding: Travelers, Zurich, Chubb, Liberty and possibly CNA. AIG is in the business, but not currently capable of bonding a project of this magnitude. In its current state, the capacity in the market cannot handle $6 billion for one Program. Recommendations are as follows:

- Begin working with executive management to reduce the penal sum of the bonds to something like 10% or 20% of the contract value. This will require state legislative action. As an example, the Bay Bridge (American Bridge contract) had a bond sum of approximately $500 million, while the contract amount was about $1.4 billion. The Little Miller Act in California stipulates that contracts over $25,000 provide a 100% payment and 50% performance bond.
- Begin working on an Owner-Controlled Insurance Program (OCIP). On a program of this size, an OCIP can save significant cost.
- As previously stated bid, award, and construct one bore of the tunnel first and then aggressively pursue bidding the second bore.

It may take some time to work through the legal process of obtaining relief from the Little Miller Act, but there is precedent in California and other parts of the United States for such relief. The bonding industry will likely not be able to respond to a 100%/50% bonding requirement on $6 billion worth of work. Additional bonding industry outreach should occur, as the bonding experts can help with navigating through the legal process.

Owner-Controlled Insurance Programs are used regularly in the market today and should be straightforward to institute.

Like the contracting community, the bonding group feels that it would not be able to respond to $6 billion worth of bonds on one program at one time. Bonding companies remain obligated until notice of final completion of a contract.

It is important to stay in touch with the bonding community as insurance companies are dynamic and change over time. By the time of construction, things may indeed have changed.

5.3.3 Segments, Shafts, Power and Early Site Work Recommendations

For segments, shafts, power, and early site work, the following recommendations are made:

- Separate pre-cast concrete liner segments from the tunnel contracts, as previously discussed. As contractor outreach continues, depending on the market conditions nearing bid time, revisit this recommendation.

- Keep the construction shafts in the tunnel contracts. See Section 5.4.3 for more discussion.

- Contract an early works package to provide temporary power to each launch shaft location. This work should start relatively early to allow time for the permit process with each power provider. The early site work package could also include constructing barge facilities and temporary roads if desired.

5.3.4 Construction Schedule

The main conveyance tunnel baseline configuration is two 37 ft diameter tunnel bores beginning at the Intermediate Forebay (IF) and Pumping Plant and terminating at the Byron Tract Forebay (BTF), a length of 33.4 miles. Each of the two tunnel bores is divided into 6 sections by access shafts (portals) making a total of 12 tunnel sections to be bored, each approximately 5.5 miles long (see Figure 5-3).

Baseline Schedule

The Baseline Construction Schedule (Figure 5-4) assumes 12 TBMs are used, one at each tunnel section with both bores being mined simultaneously and completing within 6 months of each other. An alternative baseline schedule employs 6 TBMs, one at each tunnel section of a single bore, and that one conveyance tunnel is completed before the second parallel bore is started. See "Baseline Schedule" in Appendix J.

Other baseline schedule assumptions are:

- The PTO CER Addendum alignment was used:
  - Two tunnel bores.
  - Six reaches for each bore.

- Six Construction Contracts are awarded:
  - Each contract includes both the East and West bores for a single reach.
  - Contracts are awarded one month apart.
- Tunnel boring machines can be manufactured to meet demand.
- Average tunnel production rate is 40 ft per day.
- Tunneling shift schedule: two 10 hour shifts/ per day, 5 days/week; the 6th day is for maintenance.
- Second pass liner is not included.
- A TBM can be refurbished in 6 months.

Baseline Schedule Summary

- Tunnel Construction Duration – 60 months (5.0 years)

Baseline Schedule Review

Team B reviewed the Baseline Schedule and recommended several changes to the list of assumptions.

- Assigning a single contractor to both bores of a reach was used in the baseline schedule on the assumption that both bores would encounter similar geology and difficulties. It was expected the contractor would begin one bore slightly before the other and derive benefits of experience that could be applied to the second bore. During Team B discussions, there was overwhelming support to put a single contractor working from a single construction shaft to limit contractor interactions (See Section 5.4 Shaft Arrangement). Therefore the decision was made to rearrange the packaging to have a single contractor to work both North and South out of a single shaft on one bore of the tunnel.

- Due to the large size of contracts, the team recommended using 3 months between contract awards rather than 1 month to allow the un-successful contractors to prepare their next offer and the owner/program manager to complete award activities and be ready for next award. Reduction to 2 months was considered possible when arranged and advertised as such in advance.

- TBMs were discussed. Given past experience, the group concluded that eighteen months represented a fair amount of time to order, manufacture, deliver, and install a TBM of the size anticipated (between 37 ft and 45 ft diameter.) It was also assumed that one TBM could be delivered from a given manufacturer every three months. MWD shared a project in which the TBM was ordered and turning the cutting head within sixteen months. Herrenknecht representative, Dr. Gerhard Lang, said they have the facilities and manpower to build 18 large diameter TBMs at a time and can build them in twelve months. The team's research identified at least six TBM manufacturers currently building EBP machines: Herrenknecht, Robbins, Lovat, Mitsubishi, Wirth, and Hitachi-Zosen. Several other manufacturers may also have capability including Kawasaki, Seli, Komatsu, Technicor, and IHI.

- The baseline average 40 ft/day tunneling advance rate was discussed, which was based on a 5-day-a-week, 2 -10 hour shifts-per-day with the sixth day for maintenance. The group recommended using a weekly advance rate and leaving the shifts-per-day and hours-per-shift up to the contractor, with provision that contractor be allowed to work 24/7. Consequently, an advance rate of 200 ft/wk was adopted by the group. This is equivalent to the baseline, but allows the contractor to be more flexible with resources.

- The group discussed planned maintenance and unplanned interventions along the tunnel route noting the CER has planned two safe havens and a vent shaft in each tunnel reach. Given the need to conduct cutter head maintenance at the planned safe havens and vents, the team recommend two weeks be included in the schedule for each of these locations.

These revised assumptions were applied to the Baseline Schedule to create the Revised Schedule that was presented to the Outside Review team.

Revised Schedule

The Revised Schedule (Figure 5-5) incorporates the results of team discussions on shafts, contract size, TBM manufacturers, advance rates, and interventions. See "Revised Schedule" in Appendix J.

Assumptions for the Revised Schedule are:

- The PTO CER Addendum was used (no change)
    - Two tunnel bores
    - Six reaches for each bore
- 12 Construction Contracts
    - 1 contract is awarded every <u>three</u> months
- Tunnel Boring Machines can be manufactured, delivered, and installed <u>within eighteen months.</u>
- Average TBM Advance Rate of <u>200</u> feet/<u>week</u>

- Contractor determines work schedule (e.g. # of shifts, hours/shift, days worked per week)

- Second pass liner is not included

Revised Schedule Summary:

- Tunnel Construction Duration – 96 months (8.0 years)

Revised Schedule Review

In general, the Outside Review team concurred with the assumptions made in the Revised Schedule, notably:

- 18 months from NTP to tunneling is reasonable for tunneling contractors and their TBM manufacturers.

- 200 ft/week average planned TBM advance rate is reasonable.

- There was acknowledgement among some reviewers that contracting for all twelve $475 million tunnel contracts at one time may strain the limits of the construction industry.

Conservative Schedule

The Revised schedule was modified to incorporate minor alignment revisions (some sections made longer, some shorter), contract sizing, and experience on other recent large tunneling programs including Portland CSO, Seattle's Alaskan Way Viaduct, and New Jersey's recently cancelled Trans-Hudson Tunnel. The Conservative Schedule (Figure 5-6) relies on the same assumptions as the Revised Schedule and includes the revisions mentioned above.

If the Construction community were unable to respond to all $5.7 billion worth of tunneling work at one time, the Program has the option of delaying completion of the second bore of the main conveyance tunnel until the first bore is substantially complete, allowing the market to recover before bidding the remainder.

This would only be done if, at the time, the Program felt that not enough contractors were available to create healthy competition and thus reasonable bid prices. See "Conservative Schedule" in Appendix J.

Conservative Schedule Summary

- Tunnel Construction Duration – 143 months (12.3 years)

First Bore Complete – 75 months (6.3 years)

Optional Schedules

Three Optional schedules are presented. These schedules are devised to take advantage of specific market conditions that may exist during the construction period. Each of these involve larger contracts than anticipated in the "Revised Schedule." See "Optional Schedules" in Appendix J.

Optional Schedule 1 (Figure 5-7) would award a total of six contracts for the main conveyance tunnel. Each contract would be awarded three months apart and would be for two sections of the tunnel. This process would include a "phased" notice to proceed. Each contractor would be awarded one segment in the first bore and also the parallel segment in the second bore. The emphasis would be on the first bore. Once the TBM finishes the first bore a second NTP would be given and the TBM would be removed and refurbished and mining on the second bore would begin immediately. The total mining period for Optional Schedule 1 would be approximately 10 years. This option could be considered if the construction market is weaker (not enough workers) and not all 12 segments can be started as discussed in the "Revised Schedule."

Optional Schedule 2 (Figure 5-8) is nearly identical to Optional schedule 1, however each contractor would purchase two TBMs. The second TBM would not begin until mining was complete on the first bore. This option would cost more (additional cost for a second TBM in each of six contracts), but has the advantage of finishing approximately six months faster than Optional Schedule 1. Optional Schedules 1 and 2 would only be viable if the market could not provide enough qualified workers to execute all segments at the same time.

Optional Schedule 3 (Figure 5-9) would take advantage of a specific set of market conditions to reduce construction time. If the construction market was very healthy, especially in the larger contractor arena, six contracts could be awarded with each contractor performing two segments simultaneously. Each contractor would use one shaft to launch and drive two TBMs (one working toward the north and one working south). This has the advantage of an earlier completion (potentially less than eight years) and some cost savings (contractors can spread overhead costs over two tunnel segments. This Optional Schedule has the potential to providing the fastest completion.

Construction Schedule Recommendation

One of the goals of the Program is to complete the construction in the shortest possible time. Team B recommends using the Revised Schedule as the preferred plan. This has the advantage of smaller contract amounts, providing more contractors access to provide bids. As stated previously, separating the precast segments and the tunneling contracts reduces the twelve (12) tunneling contracts to approximately $315 million each (including contingency). Larger contractors and joint ventures have the option of executing more than one segment if desired. If the larger contractor/joint venture community was especially healthy at the time, Optional Schedule 3 provides some advantages in shared overhead costs and slightly improved completion time.

Only if the construction market were especially weak or saturated would either the Conservative Schedule or either Optional Schedule 1 or 2 be considered.

5.3.5 Cost Impacts

Cost Basis

The initial cost basis for the group discussions was the CER Rev. 1 Cost Estimate. The total value of this estimate for the main conveyance tunnel is $5.7 billion, including 35% contingency. There are 6 reaches and 2 tunnel bores in the main conveyance totaling 12 reaches. Initially when only the schedule was considered we focused on grouping reaches and work together to minimize contractor interfaces and the total number of contracts to the Owner. When costs were considered the group quickly felt that this much work may saturate the tunnel contractor, bonding, and insurance markets, causing reduced response to RFPs. As a result the team considered options which extend the schedule, but created better opportunities for competitive bidding and contractor involvement.

Cost Alternatives

Consideration was given to removing the tunnel segments and shafts from the tunneling contracts. Although the Team would prefer the tunnel segments be included in the tunneling contract, removing the segments was recommended as it would lower the contract values and allow for more competitive bidding. This is potentially an additional risk to the Owner and represents another contractor interface, but the benefit of creating smaller contracts is believed to outweigh these additional risks. The tunnel segments were estimated at approximately $160 million per reach or approximately $1.9 billion in total. The tunnel contract per reach with the removal of segments is approximately $315 million. This still represents a significant tunnel contract, but should be more approachable by U.S. based and joint venture contractors. Also, this could allow for a strong tunneling contractor to be awarded several reaches.

Discussion was also given to removing the tunnel shafts from the tunneling contracts. The shaft costs are approximately $20 million per reach. Although this would be a measurable reduction in the tunneling contract, the team preferred that the contractor be allowed to design and build their own shaft. Allowing the contractor to design and build their own shaft would reduce Owner risk and potentially allow the Owner to recognize efficiencies in contractor innovation.

Costs associated with tunneling at various depths and slope were briefly discussed within the team. There is agreement that there are cost impacts for these items, but the impact is very minor and not significant enough to consider at this time.

Recommendation

The recommendation of the team is that the contracts be awarded for each reach including the tunnel shafts but excluding tunnel segments. The tunnel segments should be awarded as separate contracts.

Lost Water Revenues

One of the issues to consider when deciding on the final construction phasing plan is the potential of lost revenue due to delaying the construction of the second bore of the main conveyance tunnel. The team recognizes that there is a cost associated with lost water revenues (for the Department of Water Resources, Bureau of Reclamation and the State and Federal water contractors). The early long term operating parameters for the system have not been set and diversions in the South Delta might be increased during the construction of the second bore of the tunnel to compensate for the inability to divert the full amount through the tunnel. Using the BDCP 15 minute data, and assuming that south diversions are not increased to compensate, the amount of water that is “lost” during the 5 year construction period (of the second bore) ranges from 1.2 million acre-feet (if those five years are “dry”) to 7.8 million acre-feet (if those five years are “wet”). Based on this analysis, it was assumed that 5 million acre-feet could be lost for those 5 years. Certain water contractors sell water at a rate of $700 per acre-foot. In this scenario, the lost revenue represents $3,500,000,000. Obviously, water is sold at greatly varying rates by different contractors and Federal contractors are different from State contractors. Also, $700 per acre-foot is revenue, not profit. More study should be done to more accurately estimate the potential value of lost water.

Escalation

Delaying the second bore until after the first would cause the Program to incur additional escalation costs, potentially on the order of $500 million for an additional 5 years of construction.

Additional Program Management Support

If it became necessary to delay the construction of the second bore until after the first is complete (due to market conditions), extended Program Management would be required. The annual cost for this additional support is estimated at approximately $15,000,000. The total additional cost is $71,321,000. See Appendix K for detailed Program Management cost information.

5.4 Shaft Arrangement

Team B also investigated the CER assumptions related to construction shaft arrangement in conjunction with other tunneling studies. Two main areas related to the shafts were reviewed:

- Contractor Interfaces
- Construction Options

Prior to the first Outside Reviewer meeting in October, Team B reviewed the CER assumptions to determine where to focus the effort. The CER assumed separate launch and retrieval shafts, located adjacent to each other, with a hand-mined section of tunnel in-between. This arrangement has the advantage of limited contractor interfaces, which reduces potential claims issues that could arise when more than one contractor is trying to occupy the same space. The disadvantage is cost. Two shafts are more expensive than one and hand-mining between shafts is difficult and expensive. It was clear that shaft arrangement was one issue to study.

The second issue was constructability options. The CER assumed a slurry wall arrangement, using 60 ft inside diameter shafts for launch and 45 ft inside diameter for retrieval. All shafts were assumed to be round in shape.

Team B prepared a short presentation to the outside reviewers for the October meeting, which outlined the areas that would be studied. For Contractor Interfaces, several options were presented to the reviewers:

- Separate launch and retrieval shaft
- Combined launch and retrieval shafts
- Combined shafts with abandoned shield skin

For Construction Options:

- Slurry wall or secant pile construction methods
- Jet grouted or tremie plug
- Ground freezing
- Permeation grouting
- Round, oval, figure-eight, rectangular and dumbbell shaped shafts

The reviewers were in general consensus with the team with a few added recommended areas of study:

- Deep soil mixed walls may also be viable
- Consider caisson-style construction
- For tunnel break-out and break-in, ground freezing may be viable. Concerns remain over freezing in this soil condition.
- Permeation grouting may not be viable.

- Driving to retrieval shaft and leaving skin in place is viable

The reviewers did recommend leaving the method of construction up to the contractor; provide a viable baseline scheme, but leave final decision to the contractor to redesign within defined performance parameters.

5.4.1 Contractor Interfaces

Several options were considered to cover a wide range of possible configurations. The focus of this section is on the main shafts that will be used by the tunnel contractors for staging construction of the tunnels. There is a brief discussion of mid-line shafts, such as ventilation shafts, which will not be used for launching or retrieving Tunnel Boring Machines (TBMs), but may be used to support tunnel and/or lining construction.

Typical Construction Shaft

A key issue is the interface between shaft-tunnel contractors and, even more importantly, between adjacent tunnel contractors. The goal is to minimize the number of shafts required without adding undue interfaces, which could increase the likelihood for delays and associated claims.

Based on the most recent shaft-tunnel configurations, and considering shaft locations and tunnel driving direction, it may be possible to limit the most critical tunnel contractor interfaces to only four shaft locations i.e. where a shaft might be used for both a launch and retrieval. In these cases, the interface between contractors can either be managed and/or significantly reduced.

Option 1 - Separate Launch and Retrieval Shafts

Separate launch and retrieval shafts for every heading would essentially eliminate the need for interface between contractors (depending on the spacing between adjacent shafts). However, the construction costs and schedule impacts of approximately eight additional shafts far outweigh the relatively modest benefits of reduced contractor interfaces. An additional negative aspect of separate shafts would be the need for construction of short tunnel sections (adits) to connect adjacent shafts. These adits can be risky work unless the pressurized-face TBMs can be used for excavation and lining.

Based on an informal comparison of similar multi-contract tunnel programs, the use of separate launch and retrieval shafts is very rare. On this basis, the evaluations herein focused on combined-use shafts and related options, which would avoid the construction of added shafts that would serve no long-term purpose on the project.

Option 2 - Combined Launch and Retrieval Shafts

Combined launch and retrieval shafts would require close management of contractor interfaces using the contract/schedule, or to construct larger and/or less conventionally shaped shafts. Claims might be more likely than with Option 1, but these could be offset with schedule constraints and liquidated damages. This system of "shared usage" shafts is now becoming common based on cost efficiency and is has been used in numerous large multi-contract programs.

This arrangement would require the construction of fewer shafts than Option 1, thus saving costs. Further details on combined shafts are included in the discussion on shaft configurations (shapes).

Option 3 - Hybrid Combined Shaft Option

Another option that eliminates the costs of additional shafts and significantly reduces the contractor interfaces would be to use combined shafts but include the option to leave the TBM skins buried just before break-in to the "retrieval shaft". In this scenario, a contractor would mine to just short of a retrieval shaft and stop in a zone of pre-prepared (grouted/soil mixed) ground, then remove the internal TBM components back through the tunnel and leave the shield skin in place. The tunnel contractor would abandon only the "outer skin" of the TBM after all salvageable components are removed. The cost of a shield skin (and less efficient machine breakdown and demobilization) would still be far less than the construction of a separate TBM retrieval shaft. This approach is becoming more common in modern tunneling.

Furthermore, for locations where two headings will start from the same shaft going opposite directions, interface is eliminated if one contractor was awarded the contract to construct the shaft and both tunnels. This again would eliminate additional shafts (Option 1). The size of such a tunnel contract would still be within the capabilities of many tunnel contractors.

Contractual requirements would define the work associated with salvage and shield skin abandonment and identify each window for the arriving contractor to accomplish his work. Alternatively, the contractors could be required to reach agreement and handle the work between themselves, but this is likely to lead to disputes and delays which might be avoided altogether or reduced by incorporating proper language into each tunnel contract.

As part of Team B's review of cost and schedule, one possibility was presented that provided for separating the shaft construction from the tunneling contract. This could be done to reduce the size of the tunneling contracts. However, unless there will be a significant time between shaft completion and the start of the TBM drives, allowing another contractor to sink the shafts may create more problems than it would solve. Major tunneling contractors have experience in shaft construction and they can efficiently use the time to assemble their work crews, equipment and management while they await the TBM delivery. If they elect to subcontract the shaft construction (i.e. the retrieval shafts) then they are in control and late completion, problems with ground support, water inflows, etc. are unlikely to be acceptable reasons for delaying the work. On this basis, it is considered preferable to include the shaft construction within the tunnel construction contracts.

5.4.2 Construction Options

Shaft Shape

Based on a review of similar projects, the following shaft configurations (shapes) were evaluated and considered.

Circular (Round) Shafts - This is a common shaft configuration and has advantages with ease of engineering (symmetry) and ability to "self-support" without struts. Various types of ring supports will likely be needed given the depths and ground conditions expected. However, ring supports do not take up much of the shaft footprint, thus leaving the center core of the shaft open. This maximizes vertical space availability for construction access. The circular shape is often less costly to construct due to the inherent engineering benefits afforded by shape, i.e. lack of stress concentration points. For large diameter tunnels such as proposed for this project, circular shafts can be quite large, usually over 50-ft diameter. This large diameter often means that space efficiency is reduced.

Portland CSO Project Shaft

Elliptical (Oval) Shafts - Many of the same principles for circular shafts also apply for elliptical shapes. Although the shape is not as ideal from an engineering support standpoint, it is usually more space efficient for the tunnel contractor i.e. the value for each cubic yard (CY) of excavated material is maximized. This elongated shape also has advantages when sharing the shaft for TBM driving and/or recovery because more open space is provided along the tunnel axis. With this method a portion of each shaft would be "handed over" to the arriving contractor and there can be adequate room for two cranes to site on opposite ends of the oval. If necessary, a temporary bulkhead could be built to keep tunnel construction operations separate until the arriving TBM was completely removed.

Typical Elliptical-shape Shaft

"Figure 8" Shafts - A further variant of the elliptical shape is to employ a "Figure 8" configuration, which are essentially two truncated circular shafts combined together. The shape shares many of the space efficiencies of the elliptical designs but affords even greater separation for cases when two contractors would be working out of the same shaft, or when tunnels are being driven in opposite directions out of the same shaft. This layout would allow an adjacent TBM to be moved into one side when mining reached that limit. Prior to this, the contractor could use the opposite side of the "Figure 8" as a short tail track to support the tunneling operations. The middle of the shaft would be strutted for support and a temporary bulkhead can easily be erected.

Note: Both Elliptical and "Figure 8" shafts need careful consideration in deep soft-ground conditions as they can present unique engineering challenges to design efficiently. Finite element analysis is needed and careful sequencing is required to avoid extremely thick walls that would eliminate any cost savings from the space efficiency.

Barbell/Yin-Yang Shafts - This is a more unusual method in which two circular shafts are constructed and connected with a conventional rectangle section. This is complex from an engineering standpoint and would likely only be considered where two different tunnel contractors are required to drive tunnel concurrently in opposite directions (even then, the "Figure 8" would be likely be preferred).

Rectangular Shaft - This design can be the most space efficient but is likely the most expensive due to the engineering shortcomings, which would require extensive struts and internal bracing. In addition to the added expense from the bracing, vertical access is limited, which can result in a larger shaft footprint being needed.

Shaft Construction Methods

The construction methods discussed below make general assumptions based on the ground water and soil conditions based on the provided boring logs. It is also assumed that the work site has no significant overhead or underground obstructions and the site size will not be limited.

The main assumption is that dewatering the shaft areas prior to construction is not practical, because soils at shaft locations range from clay to gravel with some layers having an assumed high permeability interbedded with others have very a low permeability. It is also expected that extensive site dewatering would be environmentally unacceptable.

The construction of each shaft can be divided into two main parts, the shaft walls and the bottom "plug". Assuming shafts will be approx 100 to 125-ft deep and the water table is almost at the ground surface, a shaft wall must be designed to have a low permeability prior to excavation of the interior soil.

The method of shaft wall construction best suited for the site conditions would likely be slurry walls (diaphragm walls) or secant pile walls. Newer technology can be specified in order to accurately construct panels or secant pile walls at greater depth than in the past (i.e. computer guidance, etc.). Constructed properly, these walls would have a very low permeability and would not need additional treatments such as grouting. The depth of shafts is about the efficient working limit of even specialized drilling/mixing equipment. Panels constructed using grabs (clamshells) is also an option.

Less conventional methods for wall construction such as deep soil mixing should not be ruled out, particularly with the evolving technology. Other methods such as tangent piles, soldier piles and lagging or a segmented lining installed during excavation would not be practical assuming the high water table and highly permeable soils. Due to the depth of excavation and the soil/groundwater conditions, a sheet pile wall may be problematic to install and maintain.

Bottom "plug" construction depends on the method of excavation. If a wet excavation is proposed (clamshell), a bottom plug made of a thick concrete slab can be tremied into place underwater. The issue of heave would have to be thoroughly investigated before using this method. If a seal is desired prior to excavation, a jet grouted plug can be constructed. This is a common practice and produces an engineered result with fewer unknowns. Other, less desirable options could include an extensive grouting program using permeation/fracture/ compaction grout, or ground freezing of the lower portion of the shaft during excavation until a permanent slab with incorporated waterstop is constructed.

At these depths, the thickness of the required "plug" can be substantial – as much as 20 to 40-ft thick. Even then, the friction between the "plug" and the shaft walls is critical. The use of soil anchor tie-downs may be required and would need to be studied further to maximize the efficiency of the "plug" design. The use of temporary relief wells through the "plug" might also be considered. Although allowing water to be relieved from below the "plug" could temporarily help alleviate heave problems, the environmental issues associated with this dewatering could severely limit this alternative.

The option of construction shafts using caissons is not thought to be practical. Due to water pressures, a pneumatic caisson would likely be necessary.

TBM Break-out and Break-In

Ground improvement for TBM entry and exit must be incorporated into the design of shafts so that additional lateral pressures are provided for. This ground-improved zone is often accomplished by jet grouting an area slightly greater than the TBM shield length beyond the shaft wall for TBM launching, with a thinner zone required for TBM retrieval. Use of ground freezing or similar methods can also be considered.

In addition to ground improvement, the shaft walls will require additional reinforcement prior to TBM launching or retrieval. This could include ring supports or other forms of internal bracing.

Because the above elements are essential to the successful launch and retrieval of the TBMs, the tunnel break-in/break-out is typically part of the tunnel construction contract, so that the tunnel contractor controls this work.

Brightwater Shaft, Seattle

Mid-line Shafts

In addition to the tunnel construction access shafts, mid-line shafts will be required for ventilation and for intermediate tunnel access points. These shafts present many of the same technical challenges discussed above, but will not be used for launching or retrieving the TBMs. Because of this, it may be possible to build a single mid-line shaft to serve both adjacent tunnels, i.e. in lieu of a separate shaft for each tunnel. If a single shaft were employed, it would require separate adits leading from the shaft to the adjacent tunnels.

This will require further study as the cost savings from the single shaft can be eroded by the cost and risks involved in excavating short adits, which cannot be excavated using the pressurized-face TBMs.

5.4.3 Recommendations

The following are recommendations to optimize construction shaft arrangement:

- Construct combined launch and retrieval shafts and leave TBM skin shield in place outside the retrieval shaft if conditions require.
- Keep shaft construction in the tunneling contracts.
- Use secant pile or slurry wall construction with jet grouted or tremie plug.
- Leave final decision up to the contractor; provide a baseline scheme that works and allow the contractor to redesign to meet his needs.

In general, there is no need to construct a separate TBM receiving shaft where a combined or hybrid shaft configuration can be done.

The alternative (Option 3) of driving up to the TBM retrieval shaft and leaving the TBM skin in place is viable. At combined launch/retrieval shafts, provisions should be considered that would allow the tunnel contractor to leave the TBM skin buried if needed. This could include a provisional bid item to allow the CM to exercise this option if the planned retrieval shaft is not ready based on delays by the preceding contractor.

At single headings where one tunnel is to be driven from a shaft, construct the smallest possible diameter shafts and combine the tunnel and shaft contract. At locations where two headings are to be driven from the same shaft, it would be up to the contractor to manage both headings as they choose.

The use of separate stand-alone contracts for the shaft construction is not favored. It is seen as beneficial to have the responsible tunnel contractor build the shaft(s) needed, particularly for the TBM launching.

Shaft construction based on secant pile or slurry wall construction is preferred, with jet grouted or tremie plug. Deep soil-mixed walls may also be viable. A jet grouted bottom "plug" (or equivalent) is optimal, with the option of a wet-poured bottom slab possible. The use of tie-down anchors will need to be evaluated.

For tunnel break-in/break-out, jet grouting and/or ground freezing may be viable (although there are some technical concerns with freezing in clays). Permeation grouting is likely not viable.

Shaft construction methods should be left up to the contractor, as long as key performance criteria are established. These criteria will include the permeability of the shaft walls and "plug", including allowable seepage rates and groundwater drawdown limits outside the shafts, as well as the tolerances for the shaft walls (verticality, etc.).

Adequate engineering and design should be done prior to contract formation. The engineer's design should provide a viable baseline scheme, while leaving reasonable flexibility for the temporary shaft design up to the contractor (i.e. within defined performance parameters).

5.5 Shaft Flood Protection

5.5.1 Need and Current Estimate

The Problem, part I – Existing Grade below Sea Level

With the tunnel alignment through the center of the Delta, the existing grade at available land areas (islands) where access shafts can be placed is below sea level. See Figure 5-3. Elevation of existing grade on these islands is generally -12 ft.

Flood Protection of construction and ventilation shafts is a concern. With workers and equipment underground at the tunnel heading (and other places inside the shaft and tunnel), a levee breach during construction could have catastrophic consequences. A means of protecting the shaft opening from flood waters during construction is essential.

Protection of each island is afforded solely by the levee constructed to create the island which is maintained by local land holders through their respective "reclamation district" (RD). Delta islands have flooded from clear day levee failures such as the Upper Jones Tract which was fully inundated within 8 hours following a levee break attributed to burrowing rodents that weakened the levee structure to the point of failure.

An inventory of the proposed main shaft locations, the Reclamation District responsible for levee integrity, and the approximate ground surface elevations are provided in the table below.

Table 5-1 Reclamation Districts and Ground Elevations for Main Tunnel Shafts

Island	*Reclamation District*	*Existing Ground Elev**
Intermediate Pumping Plant	RD 551, Sacramento County (Pearson District)	- 5
Upper Andrus (Walnut Grove)	RD 556, Sacramento County (Upper Andrus Island)	- 5
Tyler	RD 563, Sacramento County (Tyler Island)	-12
Bouldin	RD 156, San Joaquin County (Bouldin Island)	-15
Bacon	RD 2028, San Joaquin County (Bacon Island)	-15
Victoria	RD 2040, San Joaquin County (Victoria Island)	-11
Byron Tract Forebay (no island)	Contra Costa County (no RD, not an island)	- 4

** Source – DHCCP GIS Database LIDAR approximate average.*

The Problem, part II – Flooding from Storm Events and Sea Level Rise

While shafts on Delta islands are at daily risk of inundation from the surrounding sea levels, storm events and sea level rise due to climate change over time create increased threats to shafts over longer periods of time. Water surface elevations may increase 10 ft to 30 ft from major storm events. Over 50 years, sea levels are predicted to rise 2 ft to 4 ft.

Not all locations experience the same rise in water surface elevation from storm events. Islands in the northern parts of the Delta see elevations several feet higher from storm-induced events than do those locations nearer the center and southern parts of the Delta. Further, with storm events come wind-induced waves that increase water surface elevations at levees where long clear fetches allow greatest wave build up, anywhere from an additional 2 ft to 7 ft.

Baseline Concept of Flood Protection

The Conceptual Engineering Report contemplated constructing a large elevated pad around the shaft to prevent flood water from entering. These pads vary in size according to their use (launch/retrieve, ventilation). The CER used EL 27 ft as a top of pad elevation south of Walnut Grove. The estimated total cost of placing soil in this scenario is approximately $160 million (direct cost plus contingency and Program Management, Construction Management and Final Design).

This topic area was not offered to the Outside Review team on October 5, 2010 so no comments are addressed.

5.5.2 Construction Period Protection

Protection of the tunnel shaft from inundation by the surrounding waters must be a minimum contract requirement during construction. The elevation should be established in consideration of the risk of occurrence and addressed in preliminary engineering for incorporation into the final design and contract documents. The recommendation of Team B is to start at an elevation that would protect from a levee break at normal water surface elevation. Each shaft site should be individually studied and designed for the risk and flood level associated with that site. Some levees may be at greater risk of failure than others and may require more robust protection. Some sites would not necessarily need a pad built to flood protection level, and a small ring levee may suffice. Any flood protection above that level would be left up to the contractor.

Some options for flood protection above the minimum could include gasketed liner plates, berms or additional fill.

5.5.3 Permanent Protection

For the permanent structures, a similar but distinct risk analysis process should be done. Design criteria for permanent facilities involves protection from a 200-year flood event. Once the permanent flood protection level is determined, that level should be compared to the temporary flood requirement. If the permanent level is higher than the temporary requirement, the Contractor could be mandated to build to the higher permanent level. In order to perform emergency maintenance at a permanent shaft location during a flood event, a pad of sufficient height to be above flood waters would be needed. Also, the permanent pad should be of sufficient size to allow the permanent facility and temporary equipment to have enough space. A total area of 2 acres should be sufficient.

5.5.4 RECOMMENDATIONS

- Investigate conditions at each shaft location, including existing ground elevation, river water surface elevation under normal conditions, and levee integrity.
- Determine which shaft locations are to be permanent; set location-specific parameters commensurate with water surface elevation and flood risk predicted levels.
- Establish minimum protective standards for temporary, construction-only protections expected from a levee break; conditions that may occur without warning.
- Establish site specific flood protection levels for permanent structures; allow contractor to establish means and timing for non-levee flood event protections during construction and prior to completion of permanent protective measures.

5.6 Potential Realignments

Regarding potential realignments, Team B focused its efforts on three areas:

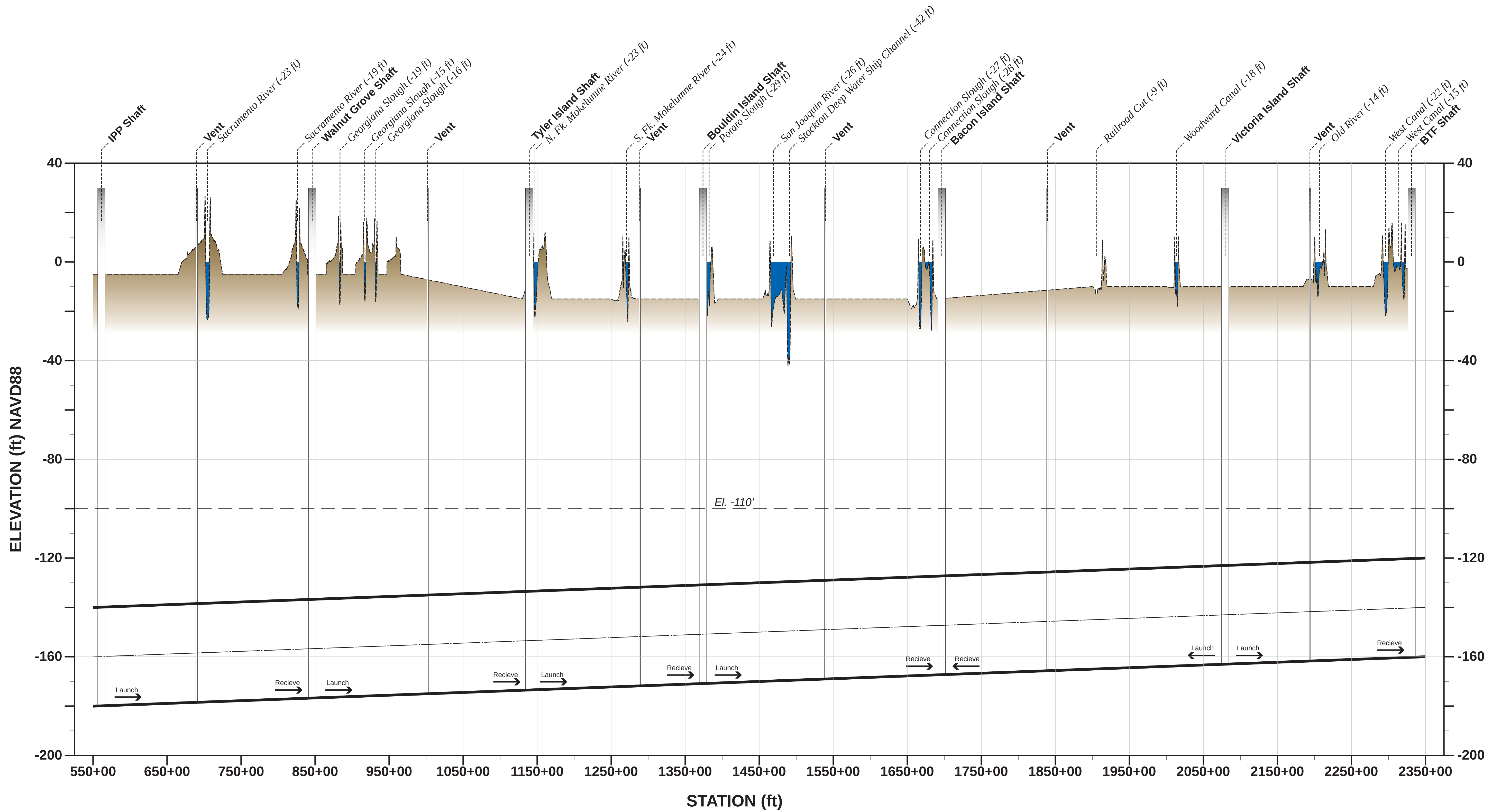
- Team B was asked to explore the possibility that a realignment of the tunnel conveyance to the east (Figure 5-10) would yield better conditions both for constructing tunnel shafts (portals) and for boring the tunnels. *(Reference Independent Review Committee Report of February 19, 2010 p 15).* Relocating the tunnel to the eastern side of the Delta would put it outside the Conveyance Planning Area and would in all probability cause a delay in the environmental process. An eastern alignment for the tunnel would also be much longer and would cost between $1 billion and $1.5 billion more than the current alignment. The Team agreed that unless the geotechnical conditions were very favorable in an eastern alignment, we should leave it where it is.
- One comment that came from the Outside Review team in October 2010 was that construction shafts should be located such that both barge (water) and road (land) access were possible. One of the construction shafts located on Venice Island had no land access, due to the lack of bridges to Venice Island.
- The Outside Review team was very concerned with potential gas and oil wells located in the tunnel alignment. These wells would pose difficulties in the tunneling operation (see Section 7.0 Gas Wells) for certain portions of the alignment.

Recommendations

See Appendix L for a complete discussion of the geologic evaluation of an alternative alignment located on the eastern side of the Delta. The current alignment is located within the "Conveyance Planning Area." This is the area in which the EIR/EIS team is analyzing impacts. Moving the alignment outside the Conveyance Planning Area would involve a large delay to the environmental process. The recommendation of the Team is to leave the alignment where it is in the middle of the Delta. Not enough is known about the geology in the eastern alignment to say that it much better than the current alignment. The eastern alignment would be approximately 10 miles longer than the current alignment, which increases cost. The cost and delays associated with moving it to the east outweigh the potential of improved geological conditions.

Team B recommends moving the Venice Island shaft north to Bouldin Island. This would put the shaft close to Highway 4, while maintaining river access for barges. Bouldin Island is a known Greater Sandhill Crane roosting site and coordination with the environmental scientists should occur to best site the shaft (Figure 5-11).

See the discussion in Section 7 for the relocation of two shafts to avoid gas wells in the alignment (Figure 5-11).



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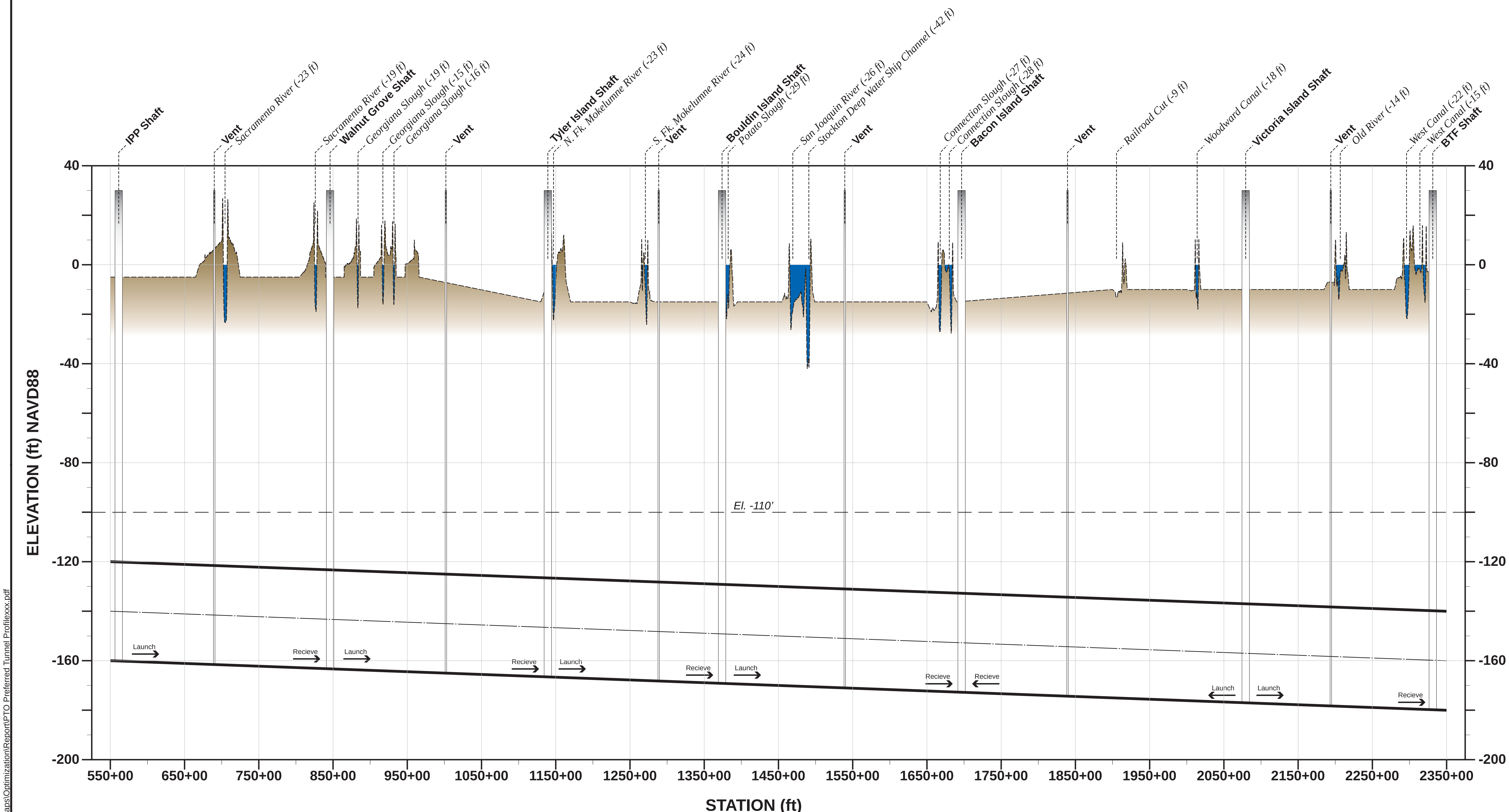
DHCCP Engineering
WORKING DRAFT-Subject to Change

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

ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Preferred Tunnel Profile (A)

FIGURE
5-2a



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Vert. Scale: ¼" = 10'
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Delta Habitat Conservation & Conveyance Program
DHCCP Team

ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Preferred Tunnel Profile (H)

FIGURE
5-2b

DHCCP Engineering
WORKING DRAFT-Subject to Change

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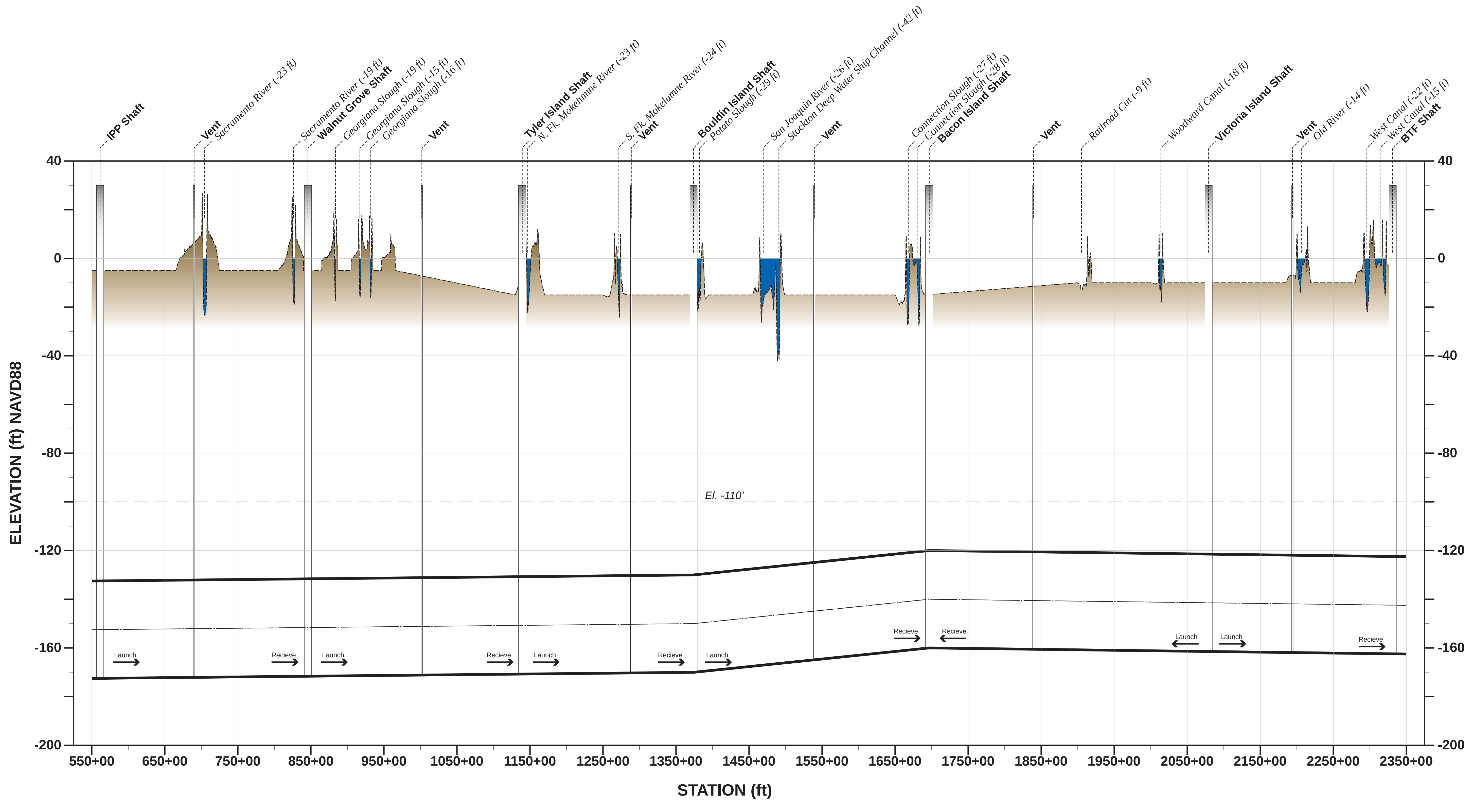

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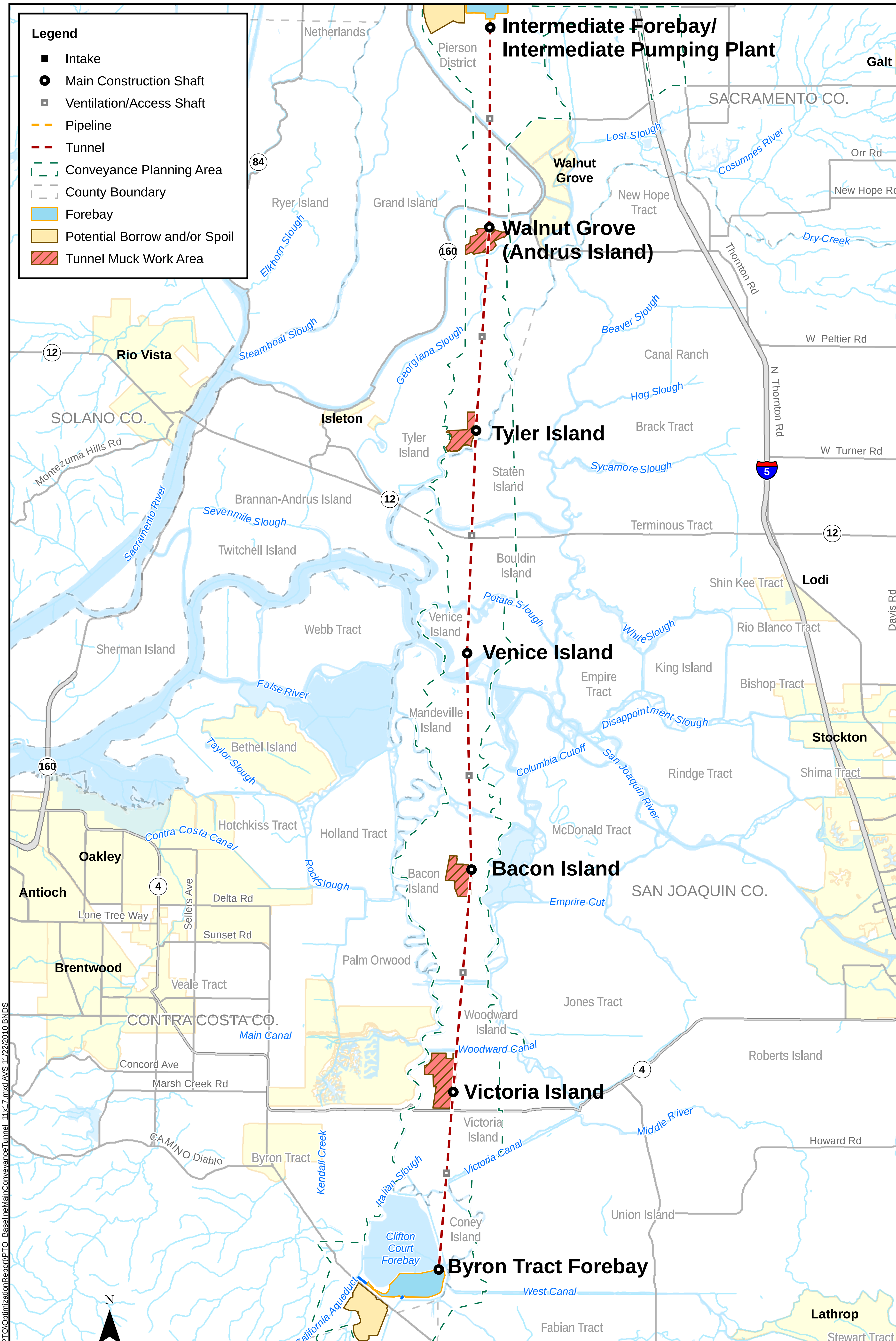
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Preferred Tunnel Profile (C)

FIGURE
5-2c



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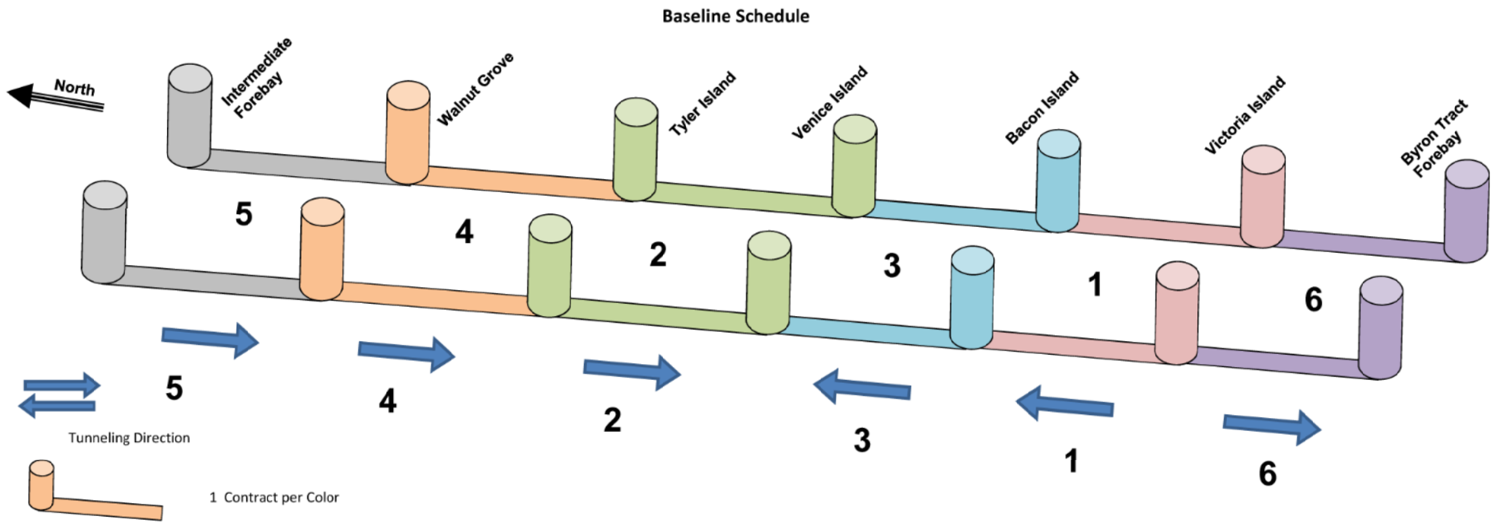
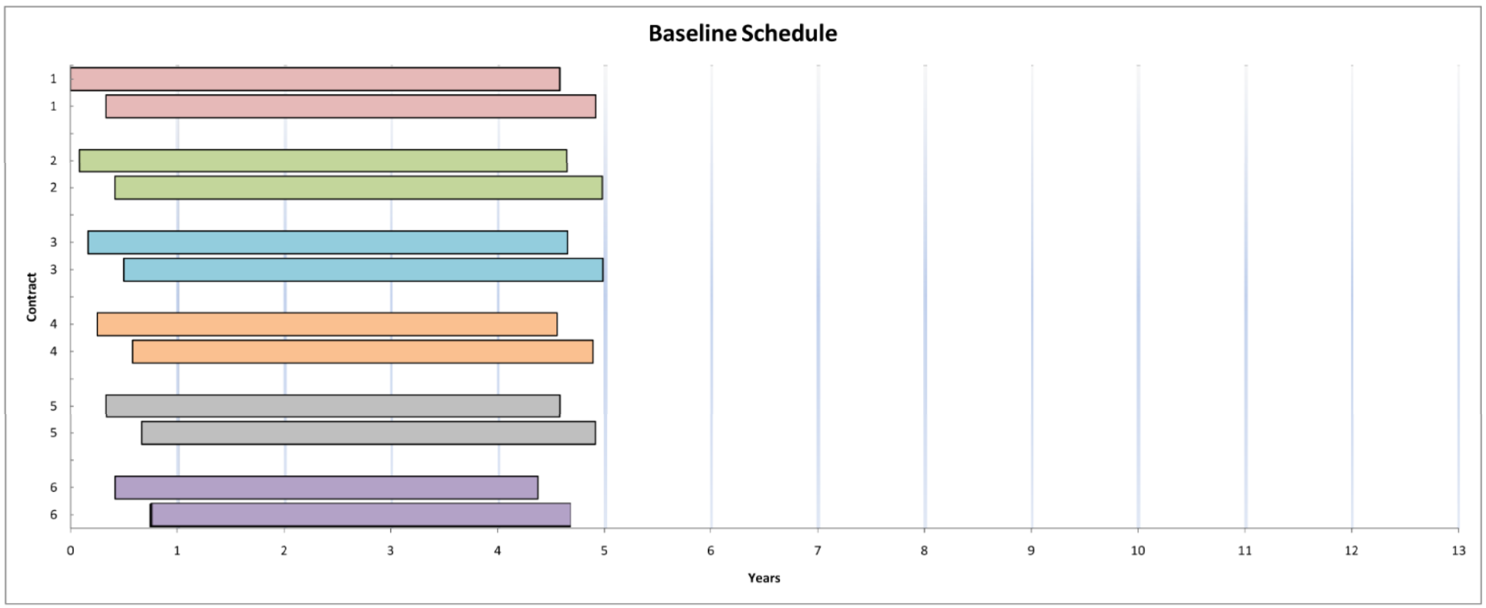
PIPELINE/TUNNEL OPTION

Baseline Main Conveyance Tunnel

FIGURE
5-3





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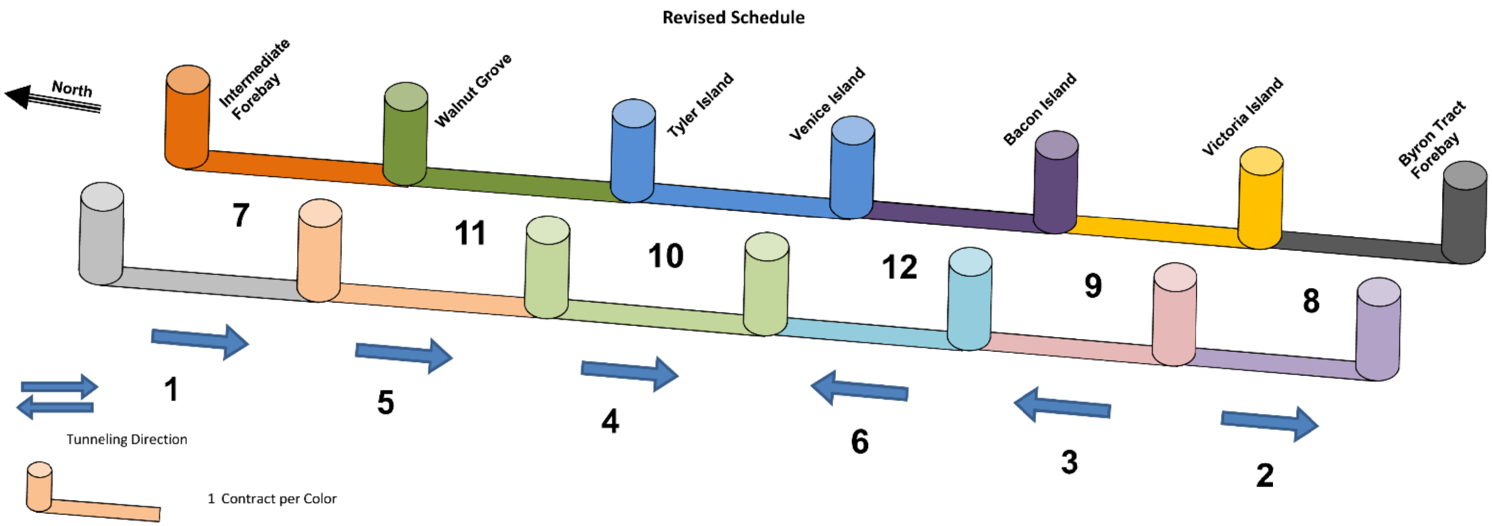

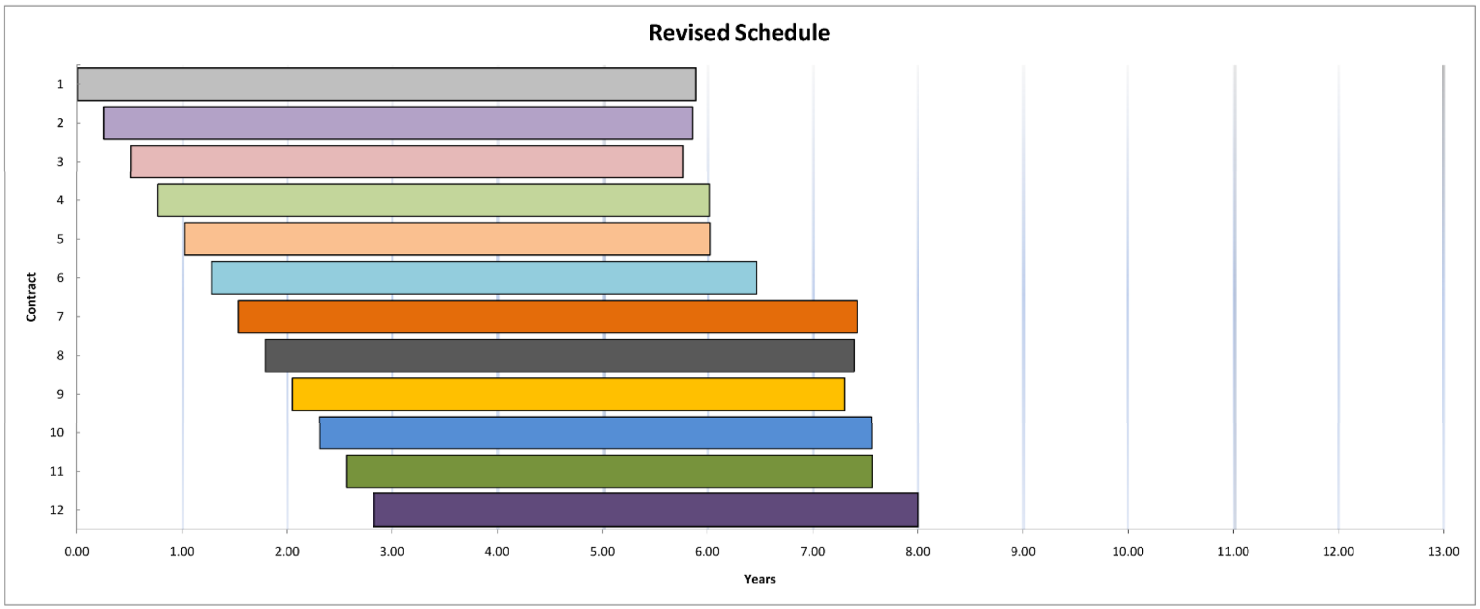

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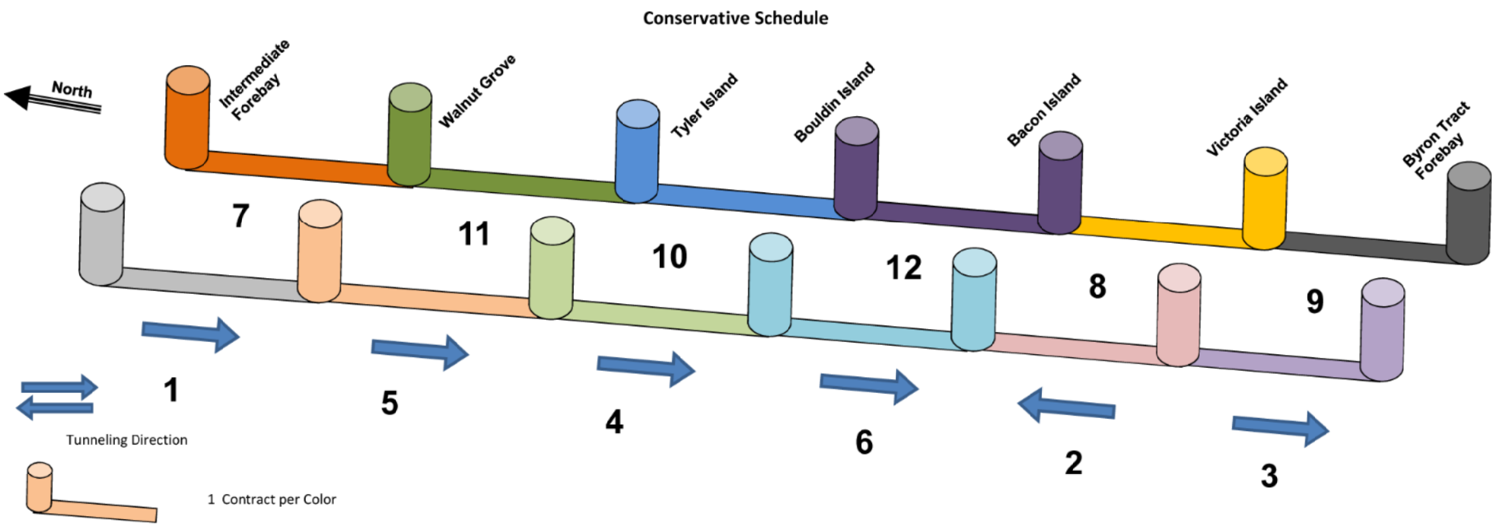
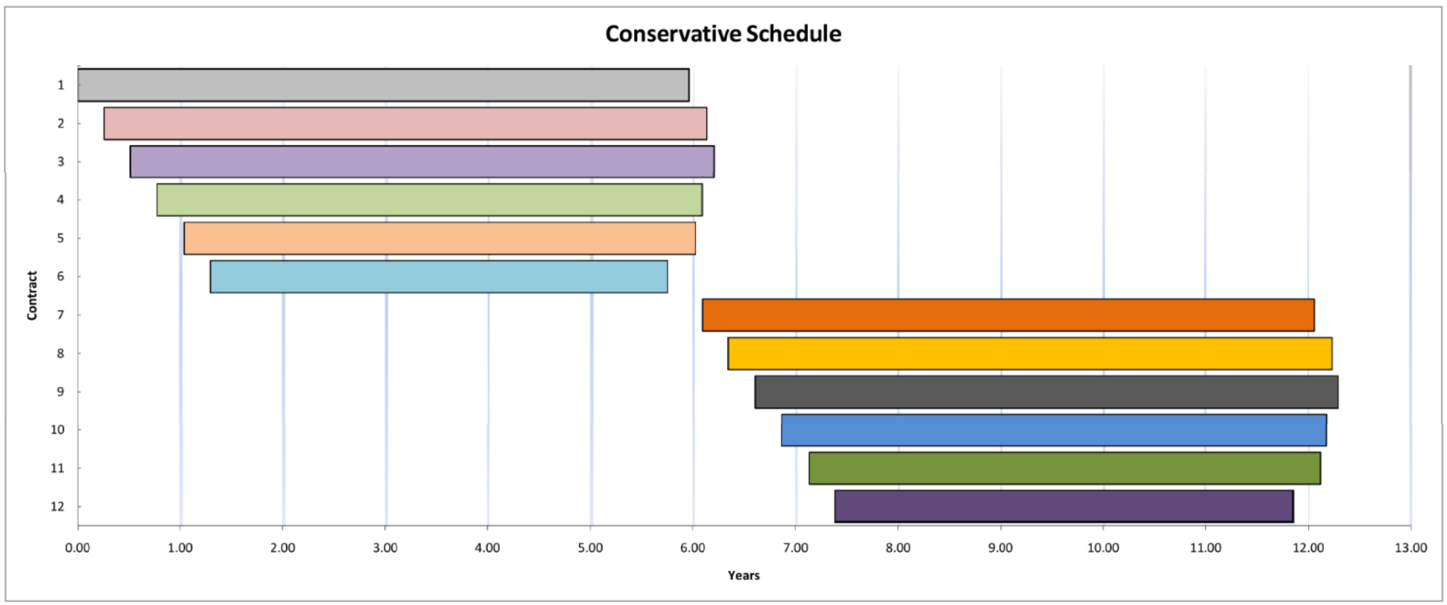
PIPELINE/TUNNEL OPTION

Revised Schedule

FIGURE
5-5





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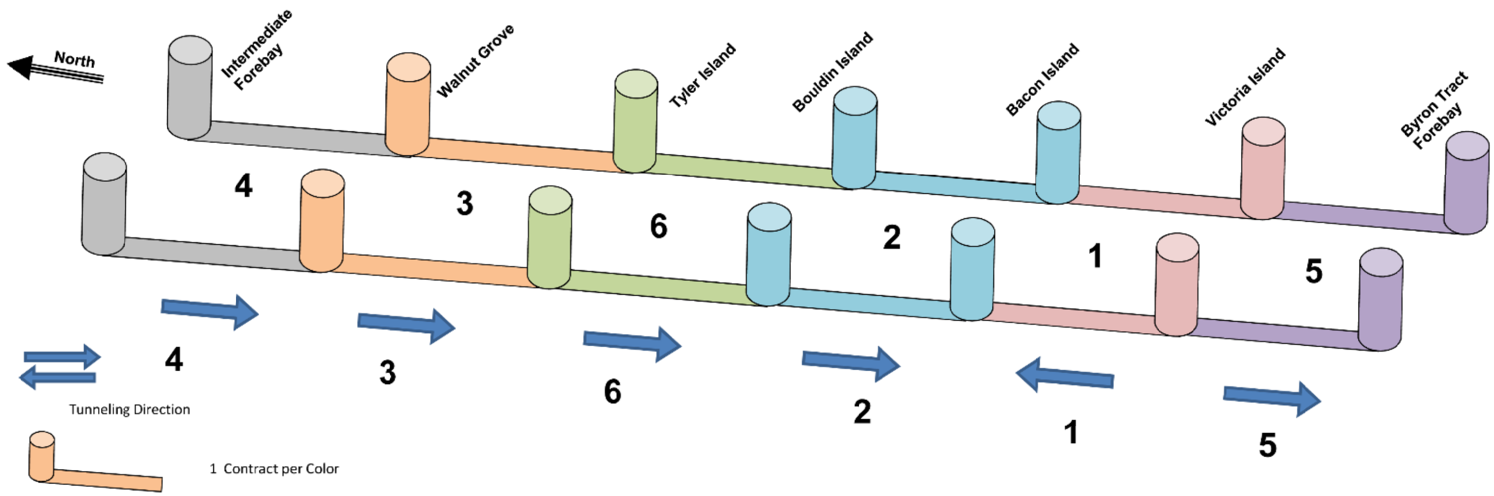
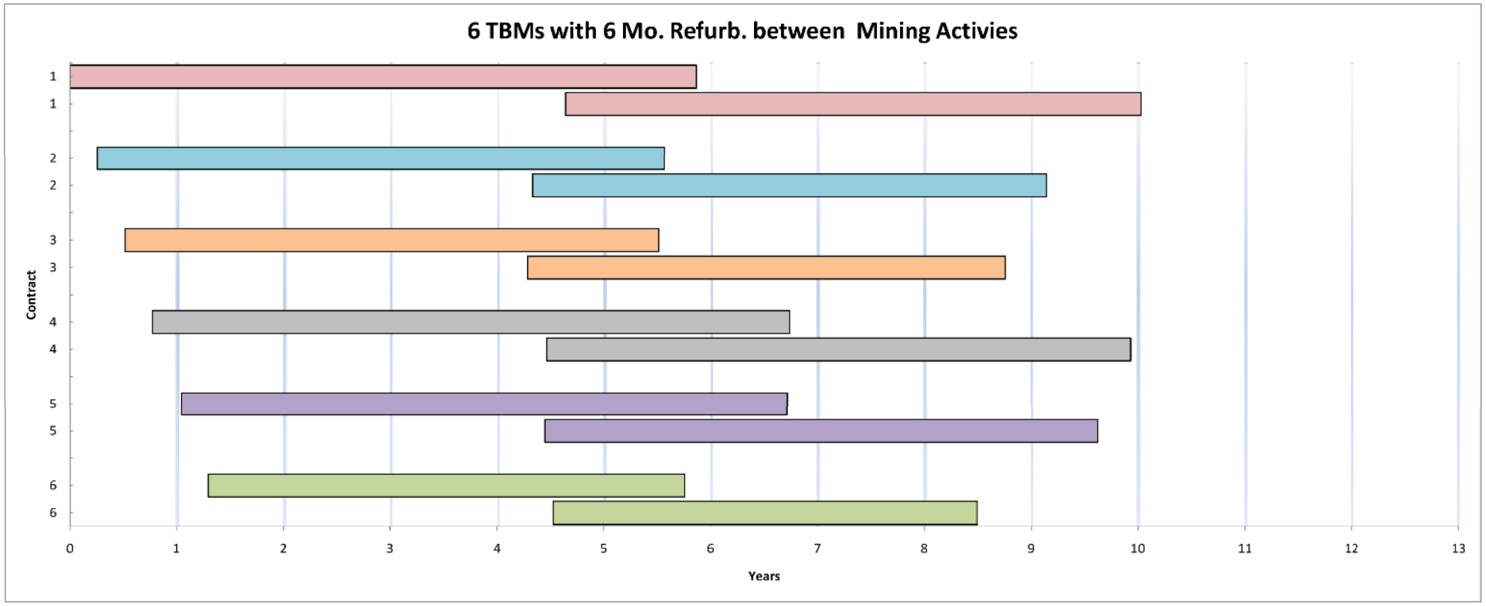
PIPELINE/TUNNEL OPTION

Conservative Schedule

FIGURE
5-6

6 TBMs with 6 mo. Refurb. Between Mining Activities





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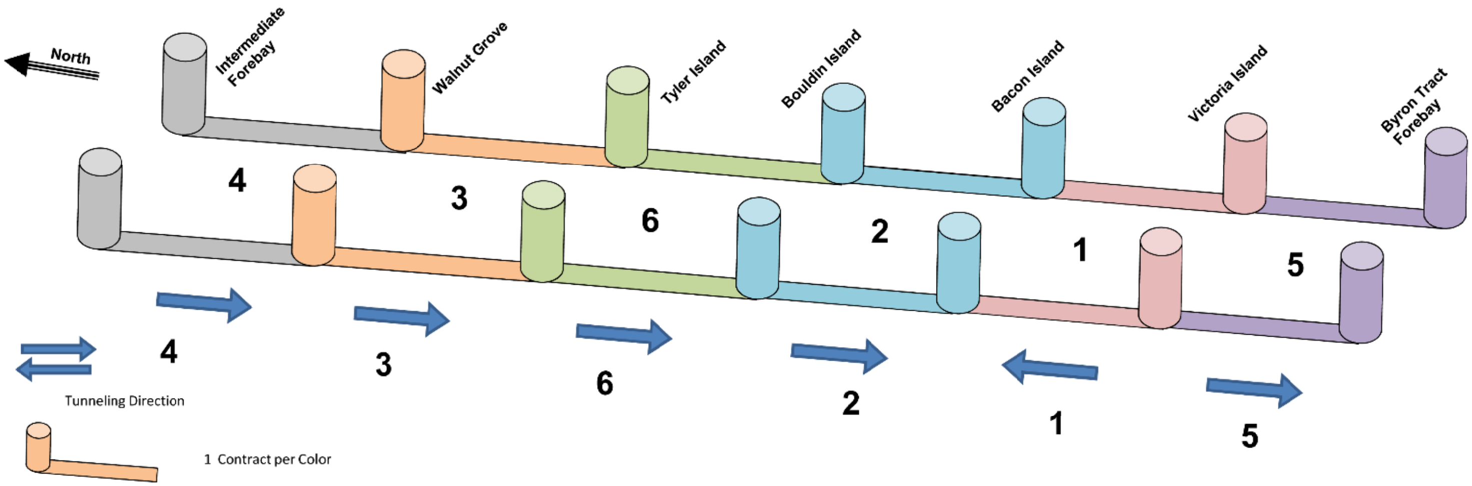
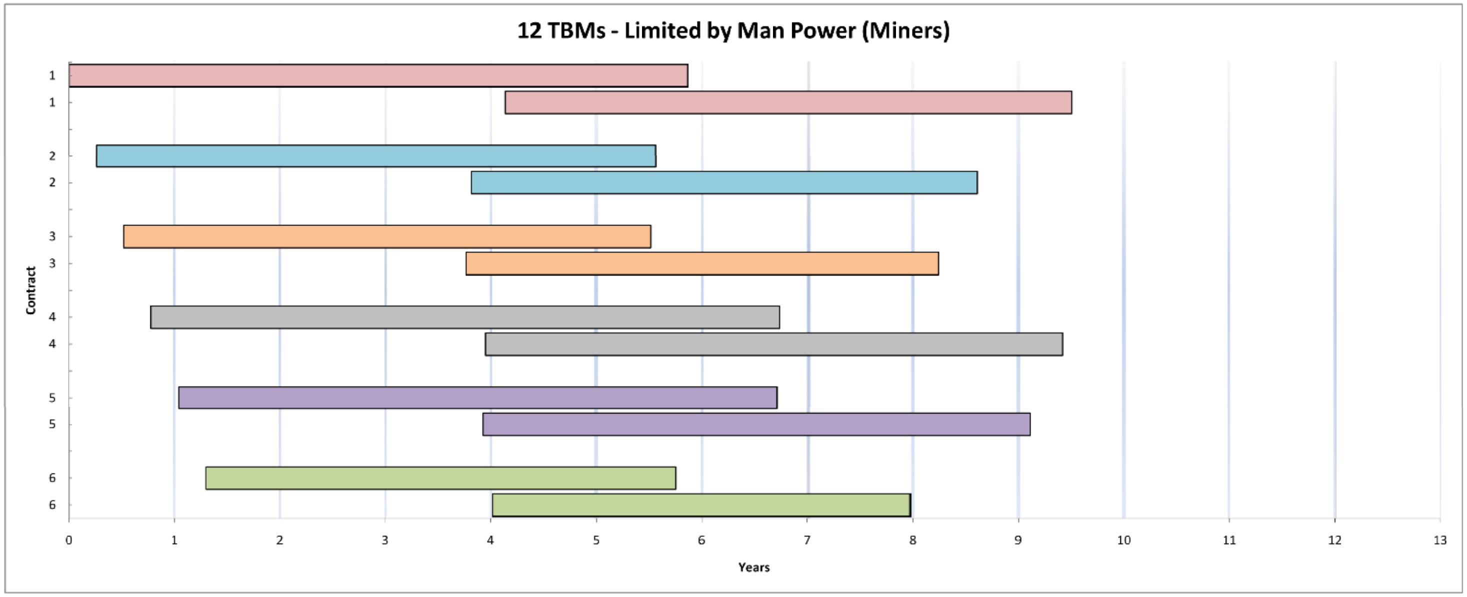
Optional Schedule 1

FIGURE
5-7

12 TBMs - Limited by Man Power (Miners)





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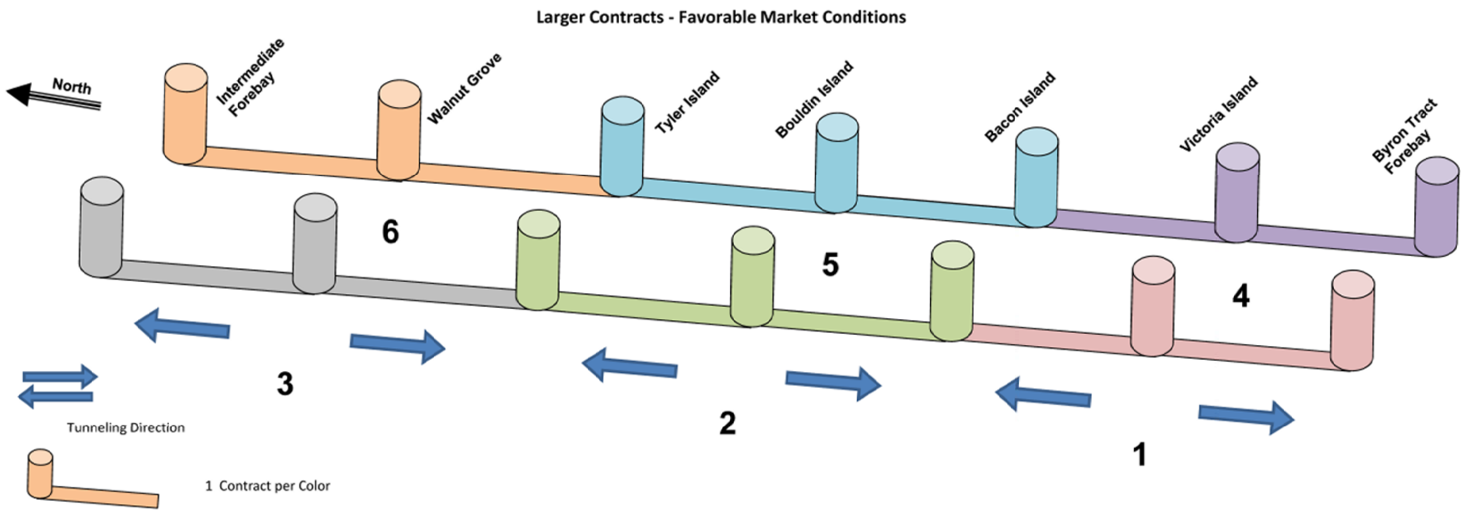
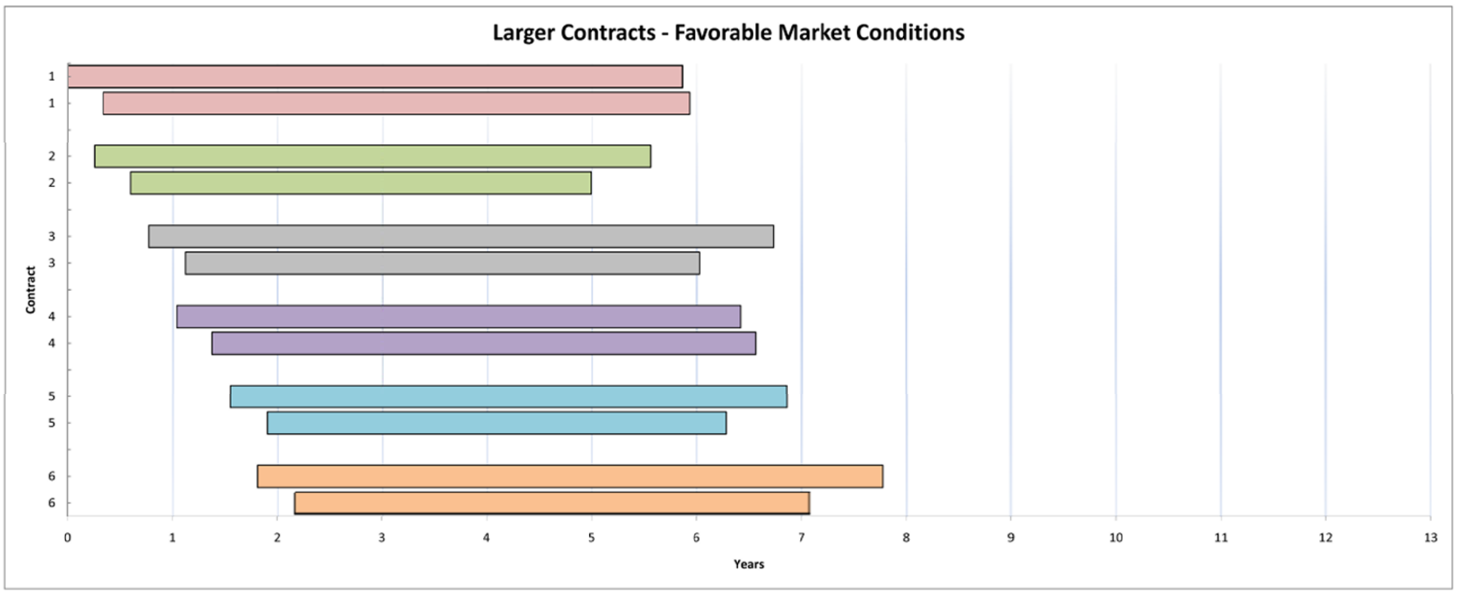
PIPELINE/TUNNEL OPTION

Optional Schedule 2

FIGURE
5-8





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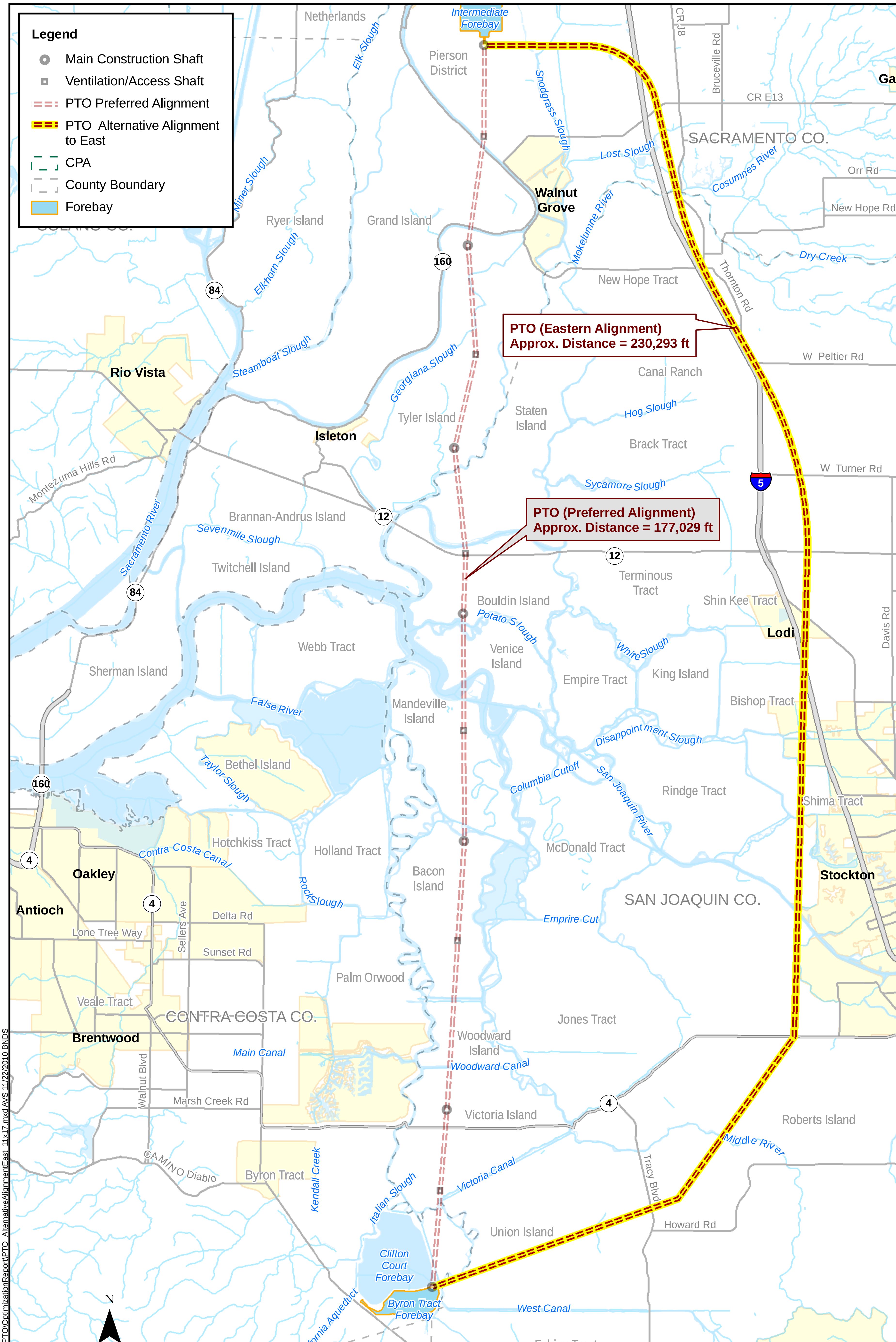
PIPELINE/TUNNEL OPTION

Optional Schedule 3

FIGURE
5-9



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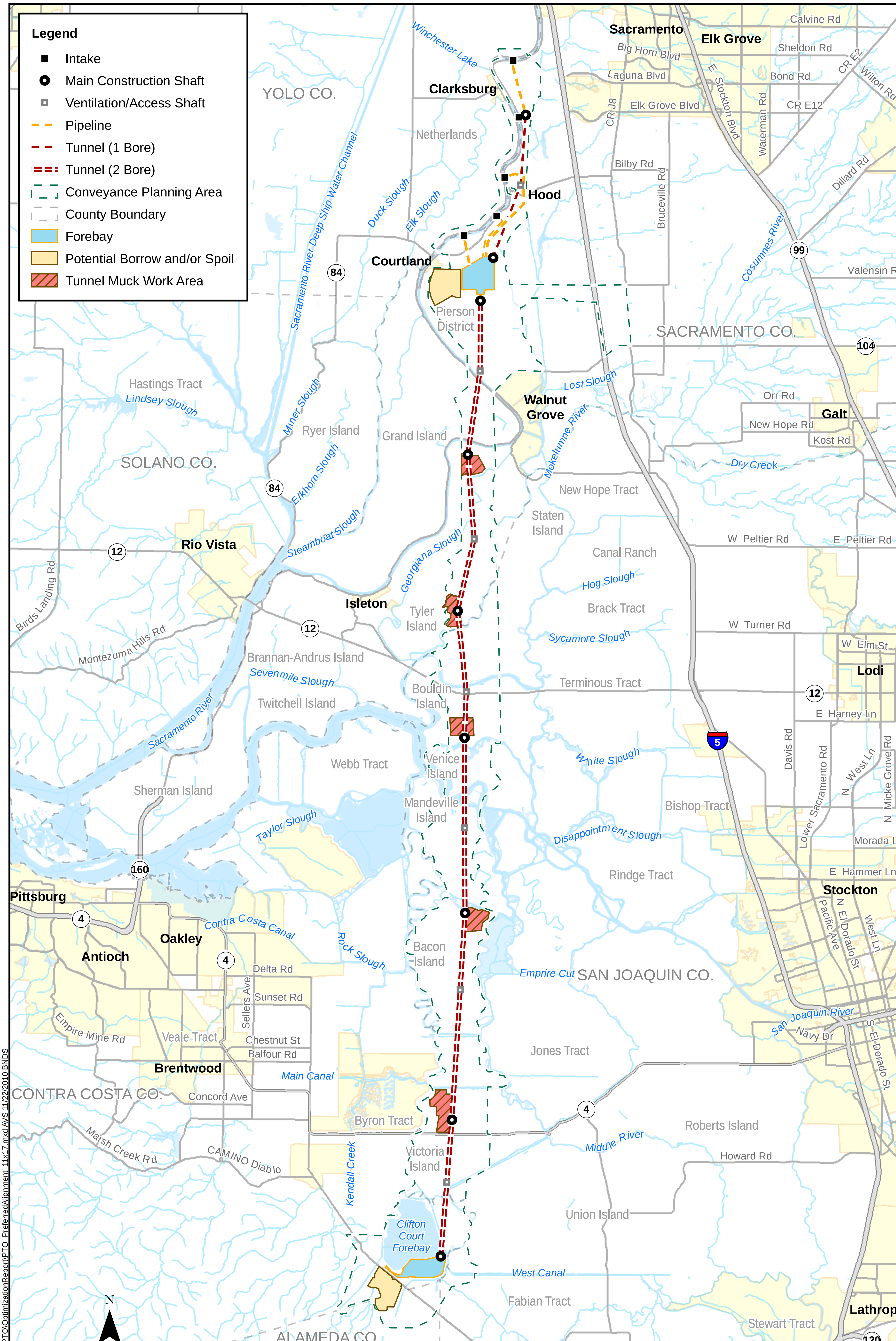
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Alternative Alignment to East

FIGURE
5-10



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ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Preferred Alignment

FIGURE
5-11

6.0 INTAKE AND TUNNEL CONVEYANCE SYSTEM OPTIMIZATION

6.1 Introduction

This section presents the identification and evaluation of conveyance alternatives identified for the Intake Conveyance and Tunnel Conveyance Systems. The Intake Conveyance System represents the conveyance system interconnecting the Sacramento River Intakes with the Intermediate Forebay (IFB), and the Tunnel Conveyance System is the conveyance system interconnecting the IFB with the Byron Tract Forebay (BTF). Team C Tunnel Conveyance System Optimization led this evaluation, and the team participants are identified in Section 3.0.

Specifically, Team C performed an initial optimization of the conveyance systems in accordance with the scope of work as follows:

- Apply the BDCP diversion requirements to the conveyance operations and identify a range of alternatives for evaluation.
- Evaluate the intake conveyance alternatives (See Section 6.5), which looked at different approaches to connect the intakes to the IFB.
- Evaluate tunnel conveyance alternatives, including one or more alternatives operating under gravity flows (See Section 6.6), which looked at different approaches to convey 15,000 cfs from the IFB to the BTF.
- Evaluate the feasibility of providing greater operational flexibility at the Intermediate Forebay.

6.2 Overall Approach to Evaluation and Optimization

The general step-by-step approach for conducting the alternatives evaluations is illustrated in the flow chart Figure 6-1. For each alternative listed in the "alternatives matrix," team members performed a sizing exercise that included building system curves, superimposing pump curves onto the system, verifying that pumps are available for the system being considered, and confirming with Team A the corresponding tunnel diameters that match the system requirements. Once this step was accomplished, the alternative could be carried forward to the cost evaluation stage. If it was found that pumps were not available to operate on the system, the alternative was eliminated or modified for re-evaluation.

The cost evaluation phase focused on the comparison of relative costs. The relative costs considered for the intake conveyance were different from the costs considered for the tunnel conveyance.

- The intake conveyance costs were based on the relative pipeline and tunnel construction costs, and the present worth of energy costs; most of the other costs were considered to be in-common (refer to Table 6-8).

- The tunnel conveyance costs were based on the relative construction costs of pipelines, tunnels, forebays, pumping plants and pumping plant features, plus the present worth of energy, OPERATIONS AND MAINTENANCE (O&M) and pump replacement costs; most of the other costs were considered to be in-common (refer to Table 6-11).

6.3 Evaluation and Optimization Factors

The factors used to perform the evaluation and initial optimization included the proposed Sacramento River flow diversions from the North Delta and the flow diversions from Old River in the South Delta, the energy cost for pumping these diversions, the operations and maintenance of the pumping plants, the variables used in the present worth calculations, and the sensitivity analysis. BDCP has developed three different flow diversion scenarios; Dual, Isolated, and Dual with OMR Flow Restrictions. Dual was used for the majority of the optimization work as it was assumed to be the most likely operational scenario. Isolated was only used during the sensitivity analysis to "bracket" the range of energy costs for the project knowing that larger diversions from the North Delta would have to be pumped. The Dual with OMR Flow Restrictions was not provided.

6.3.1 North and South Delta Diversions (Dual Operations)

Sacramento River diversion flows and stages at the proposed five northern intakes were used to compare alternatives on the same basis. Diversion flows and river stage along with several other operations criteria allowed the calculation of the energy use and the estimated cost of pumping for each alternative.

Diversion flows for each of the five intakes and corresponding river stage data were supplied by BDCP (refer to Appendix M for location and definition). This data was based on output from CALSIM II from October 1974 through September 1991 for a total of 17 years of record (water year October through September). Diversion flows were dynamically simulated using operating rules in DSM2, tidal conditions and sweeping velocity rules were included in the model and results included output points every 15 minutes. The 17 year period includes all five year types (hydrologic classification): wet, above normal, below normal, dry and critical. Hydrologic classifications are established by DWR and are based on annual precipitation in the watershed.

The BDCP data set projected diversion volumes from the river for the Early Long Term (ELT) and Late Long Term (LLT) scenarios. These scenarios have differing flow bypass requirements resulting from habitat restoration projects and water quality. The ELT scenario was used for this evaluation since it represents the proposed flows at the time the conveyance is expected to begin operating.

The "Dual Operation" BDCP data set assumed diversions from both the North Delta (five northern intakes) and the South Delta (from Old River and Clifton Court Forebay) to supply the two export plants (Central Valley Project and State Water Project) as shown in Table 6-1. The South Delta operation is similar to the current operation where the export plants are supplied from Old River and Clifton Court Forebay in the South Delta.

Table 6-1 Hydrologic Classification and Estimated Dual Diversions (North & South)

Year	Year Type (DWR)	ELT North Diversions (AF)	ELT South Diversions (AF)
1975	Wet	4,175,404	3,504,500
1976	Critical	1,385,702	3,392,211
1977	Critical	181,975	2,176,992
1978	Above Normal	4,040,443	2,924,910
1979	Below Normal	2,325,861	3,380,797
1980	Above Normal	2,800,213	3,173,789
1981	Dry	2,064,741	3,982,048
1982	Wet	5,860,823	2,831,751
1983	Wet	5,606,904	714,864
1984	Wet	4,611,062	1,824,759
1985	Dry	1,359,252	5,354,830
1986	Wet	2,870,842	3,392,632
1987	Dry	1,022,704	3,459,984
1988	Critical	677,191	2,482,961
1989	Dry	1,937,477	3,017,122
1990	Critical	453,106	2,707,155
1991	Critical	487,794	1,933,225

Flow Data Smoothing:

The 15-minute flow data provided by the BDCP showed extreme flow variations. Based on discussions with the team performing the DSM2 modeling, the model operates the intake pumps at their peak capacity (with some ramping up and down) as soon as allowed by the sweeping velocity criteria, and maintains this operation until the daily allocation is reached; then the pumps may be turned off for the rest of the day. The DSM2 model did not consider the fact that the sweeping velocity may have allowed more pumping or overall pumping at a lower capacity to divert the same volume using less energy and with less wear and tear on the pumps and motors.

To prevent overestimating pumping energy cost and to account for a more likely mode of operation, smoothing the flow data was considered.

The smoothing approach consisted of replacing every flow point by the average of the upcoming flow over a specific duration. Durations of 1 hour, 3 hours, 6 hours and 9 hours were evaluated. The 9 hour smoothing and the 6 hour smoothing were rejected since they would potentially interfere with pumping during the tidal cycle. The 3 hour smoothing was chosen over the 1 hour since it had more impact upon cost reduction. Table 6-2 presents an annual cost comparison of the preferred alternative between the smoothing options for 1979, which is an average year in terms of cost.

Table 6-2 Comparison of the Smoothing Options

Year	ELT Annual Pumping Power (MW-hr)	ELT Annual Operation Cost ($)	% change from the DSM2 15 minute data set
1979-15min	283,793	$ 13,419,978	0%
1979-1hr	266,369	$ 12,528,713	-7%
1979-3hr	223,403	$ 10,356,031	-23%
1979-6hr	182,710	$ 8,424,284	-37%
1979-9hr	157,940	$ 7,384,040	-45%

Following discussions with the DSM2 modeling team, the cost impact of a different method of operation was briefly evaluated. The daily duration of acceptable sweeping velocity was identified and the daily diversion volume was pumped over this period to minimize pumping flows. The impact of spreading the pumping time was evaluated for years 1979 (average year) and 1983 (wet year) at Intake 1 Pumping Plant (PP). Results showed a significant reduction in pumping cost when compared to the original BDCP data (15 minute data set), prior to "smoothing". The energy cost reduction was 47% in year 1979 and 36% in year 1983. These numbers were more aligned with the 6 hour smoothing operation, discussed above. This method of operation should be further evaluated in the future (see Section 11.3).

6.3.2 North Delta Diversions (Isolated Conveyance Facility Operations)

The "Isolated Conveyance Facility Operation" BDCP data set assumed diversions from the North Delta only. This scenario is not the preferred mode of operation but it would represent operation in the event the existing Old River diversion capability was lost, for example by salinity intrusion (levee failure or from sea level rise). In this scenario, the northern intakes and intake pumping plants would pump the entire volume required at the export plants; therefore the operating cost of the proposed facilities would be higher than for the dual operation. For this reason, the "Isolated Conveyance Facility Operation" was only used in the sensitivity analysis.

Table 6-3 presents the forecasted diversion volumes estimated through the BDCP model simulation of the "Isolated Conveyance Facility Operation". For comparison, Table 6-3 includes the diversion volumes from the dual operation shown in Table 6-1.

Table 6-3 Hydrologic Classification and Estimated North Diversions (Isolated Conveyance and Dual Operation)

Year	Year Type (DWR)	Isolated Conveyance Operation, ELT, North Diversions (AF)	Dual Operation, ELT, North Diversions, from Table 6-1 (AF)	Difference
1975	Wet	5,129,463	4,175,404	954,059
1976	Critical	2,024,519	1,385,702	638,817
1977	Critical	811,651	181,975	629,676
1978	Above Normal	5,642,236	4,040,443	1,601,793
1979	Below Normal	3,066,486	2,325,861	740,625
1980	Above Normal	5,063,205	2,800,213	2,262,992
1981	Dry	2,454,570	2,064,741	389,829
1982	Wet	6,767,682	5,860,823	906,859
1983	Wet	7,114,437	5,606,904	1,507,533
1984	Wet	5,612,268	4,611,062	1,001,206
1985	Dry	2,367,238	1,359,252	1,007,986
1986	Wet	3,896,272	2,870,842	1,025,430
1987	Dry	1,856,060	1,022,704	833,356
1988	Critical	1,529,153	677,191	851,962
1989	Dry	2,870,049	1,937,477	932,572
1990	Critical	1,626,543	453,106	1,173,437
1991	Critical	1,392,499	487,794	904,705

Table 6-3 shows a noticeable increase of diversion volume of the isolated over the dual conveyance operation. However, this increase remains moderate and is not expected to alter significantly the selection of the preferred alternative. The sensitivity analysis will evaluate this effect in more details.

6.3.3 Basis of Hydraulic Analysis

The hydraulic analysis was based on a system conveyance capacity of 15,000 cfs. For this hydraulic evaluation, the proposed/assumed operating WSE (or static conditions) at the three open surfaces in the intake and tunnel conveyance systems were:

- Sacramento River: 2.5 to 16 feet NAVD88
- Intermediate Forebay (IFB): 10 to 25 feet NAVD88

- Byron Tract Forebay (BTF): 0.5 to 7.5 feet NAVD88

The estimation of the hydraulic losses through the conveyance systems were calculated using a spreadsheet model incorporating losses associated with pressurized conduits (pipelines and tunnels) and weirs. The hydraulic model was updated with a set of input data for each alternative calculation in order to accommodate differences in system configurations, weir heights, and conduit sizes. Each of the alternative system configurations are explained in later sections of this report. The model was developed utilizing the Darcy Weisbach approach characterizing the internal surface condition of the conduits with the absolute roughness. These calculations included both headlosses from friction losses and fitting (minor) losses. A range of absolute roughness values were used in the hydraulic analysis in order to establish system operating ranges that could possibly occur throughout the life of the project. Roughness values tend to increase with the aging of the infrastructure. A higher absolute roughness value represents a rougher internal conduit surface resulting in higher friction losses throughout the conveyance system when compared to lower absolute roughness values. The absolute roughness values are explained below.

- In the tunnels, an absolute roughness of 1.0 millimeter (mm) was assumed for design; representing the internal surface conditions of pre-cast concrete segments, with filled-in bolt boxes, but with no lining. (Ackers and Pitt, 1996). A smaller absolute roughness of 0.6 mm was also used in the hydraulic analysis for tunnels for determining the minimum system curve of the conveyance system.

- In the concrete pipelines, an absolute roughness of 0.6 mm was assumed for design; representing the internal surface conditions of pre-cast concrete segments, with no lining (Wallingford and Barr, 1998). A smaller absolute roughness of 0.15 mm was also used in the hydraulic analysis for these pipelines for determining the minimum system curve of the conveyance system.

- In the pump branches, an absolute roughness of 0.3 mm was assumed for design; representing internal surface of steel pipe with cement mortar lining (Wallingford and Barr, 1998). A smaller absolute roughness of 0.15 mm was also used in the hydraulic analysis for pump branches for determining the minimum system curve of the conveyance system.

During sensitivity analysis of the selected alternative, several of the factors used in the hydraulic model were modified. These factors are explained in more detail in Section 6.3.8 and the results are presented in Sections 6.5.4 and 6.6.4 of this report.

6.3.4 Energy Cost Calculation

The Energy Cost was determined for all considered alternatives. The determination of Energy Costs resulted from a three step calculation:

- Hydraulic Power was calculated at all pumping plants using the BDCP Dual Operations Model outputs
- Annual Energy Costs were established assuming 2010 Western Area Power Administration (WAPA) rates (see Table 6-4)
- Present Worth was calculated for variable discount and energy escalation rates in the sensitivity analysis

Power Calculation:

In this evaluation, the power consumption was determined every 15 minutes (each time step of the data set provided) as a function of the Sacramento River stage and of the diversion flow rate, which were presented in the BDCP Dual Operation data set. The power consumption was calculated at the five Intake Pumping Plants and also at the Intermediate Pumping Plant (IPP). An initial assumption considered the operating flow rate at the IPP was the sum of the diversion flow rates of all five Intake PP (assuming only short-term operational storage in the IFB).

The power consumption through a pump is a function of the pumping flow rate and the Total Dynamic Head (TDH); see Equation 1 below. The TDH describes the energy or lift added to the Hydraulic Grade Line (HGL) by the pump. At a given flow rate, the TDH is a function of the headlosses in the system and the static lift (lift from the upstream to the downstream static water surface elevations).

For each Intake PP and IPP within each optimization alternative, a mathematical function was determined to estimate the TDH as a function of flow, and assuming an average roughness value. These mathematical functions were derived from the system curves previously established for the selection of the pumping units at the Intake and IPP.

The static lift is a function of the Sacramento River stage (15 minute interval data), the WSE in the IFB and weir crest elevations (proposed downstream of pumping plants). For the purpose of this evaluation the following WSE were assumed:

- Intermediate Forebay: 25 ft NAVD 88
- Byron Tract Forebay: 5 ft NAVD 88

The level assumed in the IFB may not have a significant impact upon power calculation since the effect is balanced between the intake static lift and the intermediate static lift. For example a high WSE in the IFB results in a high intake static lift but a low intermediate static lift (and vice versa for low WSE).

At the IPP, when the diversion flow rates added to a total of less than the possible gravity flow, the power consumption was considered zero. The possible gravity flow through the proposed gravity bypass system and tunnels was calculated to be 6,400 cfs for the assumed gradient of 20 ft between the IFB at 25 ft NAVD 88 and the BTF at 5 ft NAVD 88.

The total power (P) requirement in Watts (metric system) for each 15-minute flow interval was calculated as:

$$P = (\rho * g * Q * TDH/\eta) \quad \text{(Equation 1)}$$

Where,

- ρ = density of water = 1,000 kg/m^3
- g = gravity = 9.81 m/s^2
- Q = flow = m^3/s
- TDH = Total Dynamic Head in meters
- η = combined efficiency of the pumps and motors ($\eta=\eta_{pump}\eta_{motor}$)

Motor efficiencies were assumed to be 0.92. The motor efficiency was based upon data provided by a manufacturer (96% - 97%), but reduced to 92% to account for a loss in efficiency due to the use of variable frequency drives and to be conservative.
Pump efficiencies were based upon pump curves (intakes and IPP) provided by manufacturers. DWR Year Type cases were evaluated to identify where, on average, the duty point would be on the efficiency curves. An average pump efficiency of 0.86 was assumed at the intakes. This value accounts for losses in efficiency resulting from variable speed pumping. Average pump efficiency of 0.88 was assumed at the IPP. These values result from a sensitivity analysis considering the variations in pumps at intakes, and variations in resulting efficiencies under various hydrologic conditions. The pump curves are based on the pumps presented in the CER for each option.

The system power requirement (total in Megawatt), which includes all power uses (intake pumps, IPP pumps, side stream pumps, HVAC, O&M, lighting, etc) has been estimated for each DHCCP option by assuming an additional 15% on the calculated pumping power requirement for intake and IPP pumps.

The power was then converted to an energy consumption expressed in MW-Hr, by assuming the calculated power consumption remained constant over the 15 minute interval.

Summary of Assumptions:

- 15 minute time step power calculations
- IPP flow is sum of 5 Intake PP flows at the considered time step
- No storage assumed in forebays
- WSE of 25 ft NAVD 88 assumed in the IFB
- WSE of 5 ft NAVD88 assumed in the BTF

- Motor efficiency: 0.92
- Total system power = total pumping power + 15%

Annual Energy Cost Calculation:

Annual energy costs for each of the alternatives have been estimated utilizing WAPA Unit Energy Costs. WAPA provides two sets of rates based on time of usage during the week and day. Table 6-4 presents WAPA unit energy costs for on-peak and off-peak usage for each month of the year. The unit costs presented in Table 6-4 reflect the estimated energy costs for 2010 based on using the WAPA transmission system, and the wholesale energy cost estimates for North of Path 15 (NP15).

The only real differences between energy purchased off the California Independent System Operator Corporation (CAISO) grid and energy purchased off the WAPA grid are the transmission charges associated with delivery of the energy. For the CAISO, those charges include ancillary service charges, miscellaneous market rates, the CAISO Grid Management Charge, and the CAISO Transmission Wheeling Access Charge. Together, for 2010, the CAISO charges total about $5.89/MWHr, which is slightly more than 10% added to the market price for energy.

For WAPA, energy can be acquired at the same rates as the CAISO, so there is no difference there. But WAPA's transmission charges are about $4/MWHr less than the CAISO charges. The WAPA Transmission Rate comes from their website as of June 11, 2010, and reflects rates in place through Sept 30 of this year for use of the CVP transmission system (which is what we would be using to move power to the DHCCP facilities). Per WAPA's website, the WAPA charges for ancillary services are consistent with (which was assumed to be the same as) the CAISO rates, so the CAISO ancillary services rate estimate was used. The Miscellaneous Market rates (which were assumed to represent energy balancing charges) were assumed to be consistent with the assumptions for CAISO (so as to not create artificial differences between the CAISO and WAPA rates). Additionally, the FERC/NERC Rate was included to be consistent with the CAISO rate (again to avoid creating artificial differences). The composite WAPA Energy Cost Adder is about $4/MWHr less than the Adder for CAISO.

Table 6-4 WAPA Unit Energy Costs

Month	WAPA Estimated Unit Energy Costs ($/MWHr)	
	On-Peak	Off-Peak
Jan-10	58.38	45.63
Feb-10	56.13	43.38
Mar-10	53.88	38.38
Apr-10	47.92	36.85
May-10	49.49	32.89
Jun-10	50.75	34.07
Jul-10	64.03	42.69
Aug-10	61.68	44.51
Sep-10	58.76	44.48
Oct-10	57.36	44.95
Nov-10	58.89	48.19
Dec-10	62.34	49.62

The definitions for on-peak and off-peak are those used for energy transactions in the NP15 area, which includes the project area. The on-peak and off-peak definitions are as follows:

- On-Peak Power Cost: Hour Ending 0700 - 2200, Monday through Saturday excluding Holidays recognized by the North American Electric Reliability Corporation (NERC). This translates to on-peak rates from 6:00AM until 10:00 PM, Monday through Saturday.

- Off-Peak Power Cost: Hour Ending 0100-0600, and 2300 - 2400 Monday through Saturday, excluding NERC Holidays, plus Hour Ending 0100-2400 on Sundays and NERC Holidays. This translates to off-peak rates from 12:00 AM to 6:00 AM and 10:00 PM to midnight Monday through Saturday. Off-peak also applies all day on Sundays and Holidays.

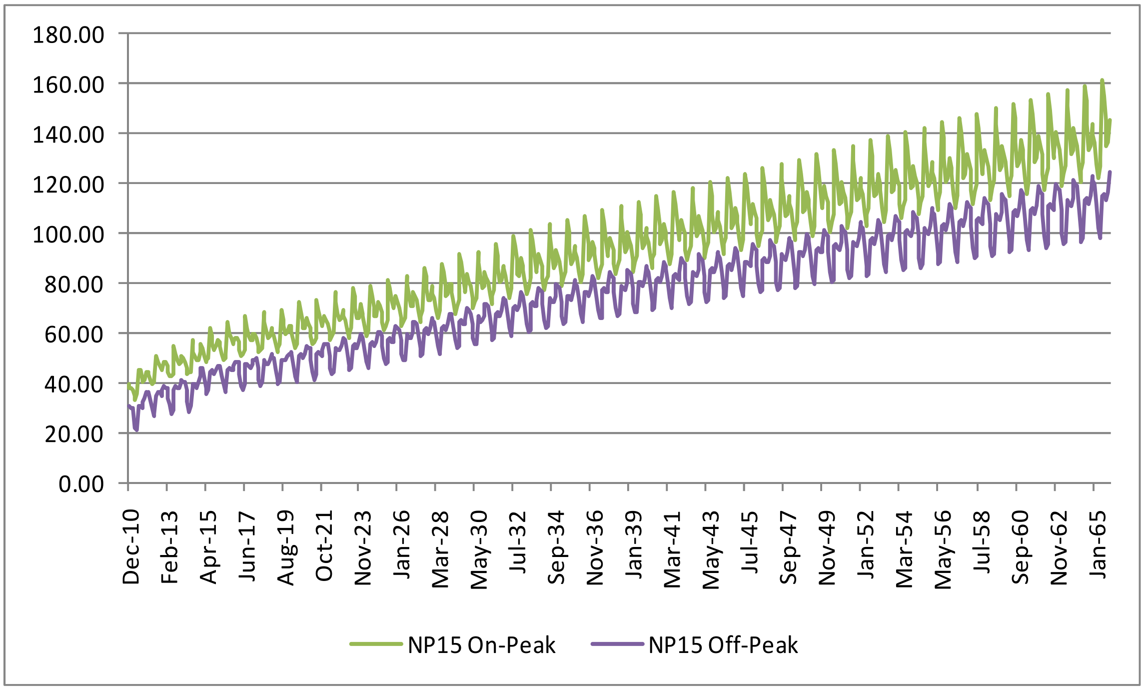
There are eight recognized NERC Holidays. These are:

New Years Day (January 1)
Memorial Day (last Monday in May)
Independence Day (July 4)
Labor Day (1st Monday in September)
Thanksgiving (4th Thursday of November)
Day after Thanksgiving (4th Friday of November)
Christmas Eve (December 24)
Christmas Day (December 25)

Total energy requirements were then determined for the 17-year of data provided by BDCP (from year 1975 to year 1991). The calculations were performed one 15 minute time step at a time, integrating On-Peak and Off-Peak periods; they allowed the comparison between alternatives and the identification of the sensitivity of several parameters.

The State Water Project Power and Risk Office (PARO) energy rates were also provided as an alternative to the WAPA rates. Figure 6-2 presents the estimated 2010 rates and expected escalation for the upcoming 55 years. The PARO rates were considered as part of the sensitivity analysis. However, varying the discount and escalation rates of the WAPA rates was also identified as a means of matching the WAPA rates more closely to the PARO rates.

Figure 6-2 PARO Energy Rates



6.3.5 Relative Construction Costs

As part of the conveyance optimization analysis, various arrangements were evaluated from a construction cost perspective. The basis of construction costs is the Conceptual Engineering Report Rev. 1 Pipeline/Tunnel Option Cost Estimate, dated 09/09/10.

In the Rev. 1 Cost Estimate, Direct Construction Costs were estimated to be $6.99 billion. Construction contingencies were applied based on type of work: contingency on tunneling work was 35% and contingency on all other work was 25%. Program management, construction management and final design costs were estimated to be 18% of the combined direct cost including construction contingency.

As part of the Intake Conveyance system optimization analysis, the direct construction costs of conveyance pipelines and tunnels were estimated. For this analysis, the cost elements associated with the conveyance pipelines include transition structures and cut and cover pipelines, as well as land acquisition costs. The cost elements associated with tunnels include launching, retrieval and ventilation shafts as well as TBM mining and precast concrete segmental lining installation, but no tunnel easement costs.

The Tunnel Conveyance system was a study of different IPP arrangements as well as different tunnel sizes and IFB volumes. IPP costs included adjustments to the plant size (concrete, rebar, structural steel, etc.) and size and number of pumps. Forebay costs included the increases in earthwork and riprap to account for the increased levee height and flatter water-side slopes.

To support the analysis of the alternatives, transition structure costs per each and pipeline costs per foot were provided. This way, intake conveyance arrangements could be mixed and matched and pipeline costs could be applied with the various pipeline lengths as required.

For tunnel diameter and length changes, the costs were provided to match each arrangement. For different diameters, tunneling costs varied due to assumptions in mining production rates, cost of tunnel boring machine and trailing gear, number of workers on the production mining crews, duration of contract (salaried staff overhead) and cost of precast concrete segmental lining. Other activities that contribute to cost were not affected by diameter changes.

In the Baseline cost estimate (Rev. 1 PTO Cost Estimate) for the 33 ft inside diameter tunnels, the estimated TBM mining production rate was assumed to be 40 feet per day. The cost of TBM and backup system for each tunnel segment was assumed to be $40 million. The cost of the precast lining was $4,000 per LF ($491.11 per cubic yard of concrete). The number of workers on the production mining crew was 24.

For tunnels up to and including 36 ft inside diameter, the above assumptions related to production and number of workers on the mining crew did not change. Costs for equipment and the precast concrete lining increased with each diameter increase.

For tunnels 37 ft inside diameter and larger, the assumption for production mining rate decreased to 35 ft per day. This increased the cost of the mining and also the length of the contract, which increased the salaried overhead costs. The number of workers on the production mining crew increased to 27 workers for 37 ft and larger. This change in assumption is due to the increased time required to erect the segmental lining and increased quantity of muck that has to be removed each mining cycle. Costs for equipment and segmental lining continued to increase incrementally per diameter increase.

6.3.6 Present Worth Factors

An equivalent present worth (PW) economic analysis was performed to compare the present worth of each alternative to the baseline. The present worth comparison was based on the relative direct construction costs, energy costs, major equipment repair and replacement costs, and pumping plant operation and maintenance costs.

The factors that were used in the present worth analysis include a) Period of Analysis; b) Discount Rate; and c) Rate of Escalation.

USBR, DWR and MWD were asked to provide input to the selection of present worth factors. The information received was used to set the initial PW factors as follows:

- Period of Analysis. Most participants agreed that a 50-year period of analysis was appropriate for this evaluation to serve as the economic life of these facilities. Although the service life of the project with preventative maintenance may be closer to 100 years, accounting for the present value of future cash flows at 50 years or less will more closely match the probable funding mechanism for the project which is likely to be through the sale of bonds.

- Discount Rate. The URL listed below will access the Project Interest Rate that Reclamation is to use for planning purposes in FY11. This URL directs to a Treasury Department web page, and Table 5 is applicable to Reclamation. The Rate being used for FY11 is 4-1/8 %. Note that this rate is allowed to vary over time, but for purposes of this evaluation, the 4-1/8% value was kept constant for the period of analysis.

  http://www.treasurydirect.gov/govt/rates/tcir/tcir_fy2011_opdirannual.htm

  The USBR rate was used for the evaluation with the concurrence of DWR.

- Rate of Escalation. The rate of escalation to be used for energy and other costs in the evaluation was taken from MWD's experience and information received from PARO. MWD stated that it would use 2.5% for the average annual rate of escalation of energy costs. PARO provide a summary of its energy planning information and it showed that a 2.4% average annual rate of escalation for electrical energy is expected for the period 2010 – 2065. For reference, the November 18, 2010 version of BDCP Chapter 8 (Cost and Funding) uses a long-term average rate of 2.1% for both conveyance and restoration. For this analysis, an average annual 2.5% escalation rate was used for the evaluation.

6.3.7 Pumping Plant O&M Costs

Non-energy pumping plant O&M costs include the base cost of operating a PP, as well as the major equipment repair and replacement costs that vary with the number of pumps installed.

None of the alternatives evaluated included elimination of Intake Pumping Plants, so the base O&M cost for Intake Pumping Plants was assumed to be the same for all alternatives. To determine annual O&M base cost for the IPP, a three year average of the actual O&M costs for the SWP Banks Pumping Plant was used. The Banks Pumping Plant, which began operation with seven pumps in 1969 and four additional pumps added in 1986, was used as it is comparable in size and capacity to the Intermediate Pumping Plants in this evaluation.

For the three year average, it was assumed that the Banks O&M costs did not include the major equipment Replacement and Repair (R&R) costs. Since the alternatives evaluated had different numbers of pumps, the R&R costs were calculated. To determine the R&R costs for the Intake and Intermediate Pumping Plants, the following assumptions were made:

Intake PP Pumps

- Annual – routine inspection and performance monitoring (the pumps need not be pulled out), included in O&M costs.
- Every Five years – replace all the pump bearings, packing etc. Assume material cost of $5,000 per pump.
- Every Twenty-five years – replace pump impeller, wear rings, bearings etc. Assume 15% of the initial pump cost per supplier recommendation.

IPP Pumps

- Annual – routine inspection and performance monitoring (the pumps need not be pulled out), included in O&M costs.
- Every Five years – replace all the pump bearings, packing etc. Assume material cost of $15,000 per pump.
- Every Twenty-five years – replace pump impeller, wear rings, bearings etc. Assume 12% of the initial pump cost per supplier recommendation.

It was assumed that the pumps will have a useful life of 50 years if this maintenance schedule is followed.

Intake and IPP Pump Motors

- Annual – routine inspection; lube oil change would be a routine operation procedure. Included in O&M costs.
- Every Ten years – replace all bearings. Assume 2% of the motor cost.
- Every Twenty-five years – either replace the motor or rewind the motor. Assume rewinding will cost 50% of a new motor. Assume and additional $2,000 for auxiliary system maintenance (cooling water system, lubrication system etc.) per pumping plant.

It was assumed that the motors will have a useful life of 50 years if this maintenance schedule is followed.

Additional details can be found in Appendix O.

6.3.8 Factors for Sensitivity Analysis

A sensitivity analysis incorporating multiple factors was performed on the lowest present worth alternatives identified. Each of the sensitivity factors listed below was analyzed for cost and operational advantages and disadvantages. The results of the sensitivity analysis for the Tunnel Conveyance are presented in Section 6.6.4. A similar approach was applied to the Intake Conveyance least present worth alternative; however, only the tunnel diameter and present worth factors were analyzed. The results for the sensitivity analysis for the Intake Conveyance are presented in Section 6.5.4.

IFB Storage / IFB Size / IFB Water Depth: This sensitivity factor analyzes the impact of increasing the volume in the IFB for greater storage to offset differences in flow being pumped into the IFB by the intake pumping plants and out of the IFB by the IPP/gravity bypass.

This sensitivity factor analysis uses the same BDCP diversion data for flow entering the IFB; however, it differs from the least present worth alternative by assuming a daily average flow leaving the IFB. This mismatch between flow in and flow out would create the need for IFB storage and would cause water surface elevations to fluctuate in the IFB throughout the day.

This factor was analyzed in order to understand the impacts that IFB storage would have on the IFB design (need for increasing the bank height), operational power costs, impacts to system operation, and to determine the expected fluctuations in IFB water surface elevations during daily operation.

- Friction Factors: This sensitivity factor analyzed the impact of adding an epoxy coating to the interior surface of the conveyance tunnels. The epoxy coating would reduce the hydraulic friction factor of the tunnels which would decrease the dynamic losses and internal pressures throughout the system and would reduce pump and motor size requirements and resultant energy cost.

  This factor was analyzed in order to understand how a different tunnel design and friction factor may affect mechanical operation, capital cost and operational costs for the conveyance system.

- Tunnel Diameters: This sensitivity factor analyzed the effects of changing tunnel diameters of the least present worth alternative. As tunnel diameters increase and decrease, the dynamic friction losses through the tunnel conveyance would be affected, which would impact pump and motor selections.

  This factor was analyzed in order to compare and contrast impacts that changes in tunnel diameters would have on capital costs, operational costs, pump selection, and overall system operation.

- Weir Elevation: The weir downstream of the IPP has a baseline top elevation of 30-feet and provides back pressure to the intermediate pumps to aid with pump operation. This sensitivity factor raised the weir elevation of the least present worth alternative. A higher downstream weir elevation will increase the back pressure on the pumps during start up and low flow conditions and would allow pumps to meet lower conveyance flow conditions at reduced speeds. Additionally, if the IPP gravity bypass was eliminated or if the WSE in the IFB was raised, the proposed pumping units would have more difficulties to operate at low flow and low head conditions without a raised weir. This factor was analyzed in order to obtain a better understanding of the potential energy cost of raising the weir and potential benefits to pumping plant operation.

- Operation without Gravity Bypass: Four of the five tunnel conveyance system alternatives include a gravity bypass system to convey water by gravity from the IFB to the beginning of the tunnel system that terminates at the BTF. If the least present worth alternative includes a gravity bypass, this sensitivity factor includes removing the gravity bypass and associated gravity piping and valves between the IFB and the beginning of the conveyance tunnels. The gravity bypass system eliminates the need for pumping at lower flow conditions and therefore decreases the overall required pump operating envelope. Conversely, elimination of the gravity bypass would increase the range required for the pumping system to meet low flow export conditions. In order for pumps to meet these conditions, lower pump speeds, increased tunnel diameters, and/or other factors may need to be incorporated.

  This factor was analyzed to compare and contrast the trade-offs between the annual cost and initial capital cost of the gravity bypass.

- Energy Unit Cost Variation: Present worth for each of the alternatives have been calculated utilizing the on-peak and off-peak 2010 WAPA energy rates. This sensitivity factor included using the PARO energy rates for determining alternative present worth values for the least present worth alternative.

  This factor was evaluated in order to understand how alternative power supply sources might affect the projected 50 year present worth calculations for the least present worth alternative.

- Factors for Present Worth Calculation: The Present Worth calculations for all alternatives were determined using a 4.125% discount rate and a 2.5% escalation rate over a 50-year period to arrive at the tentative solution. This sensitivity factor also evaluated a 100-year period and various other discount and escalation rates for calculating present worth.

  These factors were evaluated to understand how different factors would affect the present worth calculations of the least present worth alternative.

- Comparison of BDCP Operational Scenarios: The BDCP has provided several operation scenarios that could be utilized for analyzing the tunnel conveyance system. The dual operation scenario has been initially used on all of the alternatives and represents allowable diversion data from the years 1975 to 1991. The dual operation assumes diversion occurs from both the north Delta and the south Delta to meet the maximum export flow of 15,000 cfs.

For the least present worth alternative, the team analyzed the effects on the tunnel conveyance system using the isolated conveyance flow operation scenario which differs from the dual operation in that all diversion is assumed to occur along the north Delta with no supply coming from the south. This sensitivity factor would increase the volume of water required to be conveyed from the north Delta through the tunnel conveyance system when compared to the dual scenario and would have impacts to the operational costs that would need to be analyzed and discussed.

6.4 Method of Present Worth Analysis

A present worth analysis was performed to compare the intake and tunnel conveyance alternatives to the baseline. The present worth analysis uses a 50-year time period for the project, and applies an escalation rate of 2.5% and a discount rate of 4.125% to each year of cost data.

Figure 6-3 shows the present worth calculation for the Baseline Alternative. The present worth calculation, used for alternative comparison, considered a 50 year operation and a start of operation in 2022. The 2010 WAPA rates were used with an annual escalation rate of 2.5%; the discount rate assumed was 4.125%. These rates are expected to vary depending on the agency; therefore the sensitivity analysis evaluated the impact of these rates.

The calculated annual costs in 2010 dollars, for the 17 years of the BDCP diversion, were used in the following manner:

- The annual cost in 2010 dollar of year 1975 diversion was assigned to year 2022
- The annual cost in 2010 dollar of year 1976 diversion was assigned to year 2023
- A three time repeat of the 17 years of provided diversion data (minus one year: 1991), were used to assign the 17 calculated costs to the 50 years from 2022 to 2071.
- Escalation factors and discount factors were escalated from 2010 to 2022 to account for the offset of the estimated start of operation.

Refer to Appendix N for additional Present Worth Calculations.

Figure 6-3: Present Worth Calculation for the Baseline Alternative

Baseline Alternative: 33 ft diameter tunnel, 3hr smoothing flow, dual operation

Assumptions	
Discount rate (%)	4.125
Escalation rate (%)	2.5
Years	50

Presented Costs are Pumping Costs only, Increased by 15%

Present Worth

	year	actual year	Power Costs (2010)	Power Costs (w/escalation)	Present Worth (2010 dollars)	Escalation Factor $(1+i)^n$	Discount Factor $1/(1+i)^n$
	0			-	$0		
1975-wet	1	2022	$ 23,933,753	$32,188,236	$19,816,959	1.34	0.62
1976-critical	2	2023	$ 6,271,523	$8,645,363	$24,928,691	1.38	0.59
1977-critical	3	2024	$ 292,412	$413,170	$25,163,307	1.41	0.57
1978-above normal	4	2025	$ 24,598,629	$35,626,149	$44,592,004	1.45	0.55
1979-below normal	5	2026	$ 11,358,460	$16,861,697	$53,423,231	1.48	0.52
1980-above normal	6	2027	$ 14,377,207	$21,876,621	$64,427,089	1.52	0.50
1981-dry	7	2028	$ 10,148,005	$15,827,424	$72,072,837	1.56	0.48
1982-wet	8	2029	$ 36,660,200	$58,606,835	$99,262,448	1.60	0.46
1983-wet	9	2030	$ 34,696,545	$56,854,330	$124,594,085	1.64	0.45
1984-wet	10	2031	$ 28,658,258	$48,133,890	$145,190,691	1.68	0.43
1985-dry	11	2032	$ 5,623,601	$9,681,431	$149,169,282	1.72	0.41
1986-wet	12	2033	$ 16,573,016	$29,244,920	$160,711,389	1.76	0.39
1987-dry	13	2034	$ 3,722,262	$6,732,553	$163,263,264	1.81	0.38
1988-critical	14	2035	$ 2,896,424	$5,369,808	$165,217,979	1.85	0.36
1989-dry	15	2036	$ 10,005,906	$19,014,151	$171,865,299	1.90	0.35
1990-critical	16	2037	$ 1,279,260	$2,491,742	$172,701,898	1.95	0.34
1991-critical	17	2038	$ 2,204,292	$4,400,859	$174,120,946	2.00	0.32
1975-wet	18	2039	$ 23,933,753	$48,978,208	$189,288,218	2.05	0.31
1976-critical	19	2040	$ 6,271,523	$13,154,943	$193,200,575	2.10	0.30
1977-critical	20	2041	$ 292,412	$628,687	$193,380,144	2.15	0.29
1978-above normal	21	2042	$ 24,598,629	$54,209,398	$208,250,252	2.20	0.27
1979-below normal	22	2043	$ 11,358,460	$25,657,066	$215,009,393	2.26	0.26
1980-above normal	23	2044	$ 14,377,207	$33,287,866	$223,431,397	2.32	0.25
1981-dry	24	2045	$ 10,148,005	$24,083,297	$229,283,210	2.37	0.24
1982-wet	25	2046	$ 36,660,200	$89,177,231	$250,093,277	2.43	0.23
1983-wet	26	2047	$ 34,696,545	$86,510,587	$269,481,308	2.49	0.22
1984-wet	27	2048	$ 28,658,258	$73,241,405	$285,245,297	2.56	0.22
1985-dry	28	2049	$ 5,623,601	$14,731,443	$288,290,384	2.62	0.21
1986-wet	29	2050	$ 16,573,016	$44,499,605	$297,124,347	2.69	0.20
1987-dry	30	2051	$ 3,722,262	$10,244,375	$299,077,472	2.75	0.19
1988-critical	31	2052	$ 2,896,424	$8,170,798	$300,573,548	2.82	0.18
1989-dry	32	2053	$ 10,005,906	$28,932,279	$305,661,196	2.89	0.18
1990-critical	33	2054	$ 1,279,260	$3,791,480	$306,301,503	2.96	0.17
1991-critical	34	2055	$ 2,204,292	$6,696,427	$307,387,597	3.04	0.16
1975-wet	35	2056	$ 23,933,753	$74,526,136	$318,996,146	3.11	0.16
1976-critical	36	2057	$ 6,271,523	$20,016,801	$321,990,541	3.19	0.15
1977-critical	37	2058	$ 292,412	$956,622	$322,127,976	3.27	0.14
1978-above normal	38	2059	$ 24,598,629	$82,486,010	$333,509,086	3.35	0.14
1979-below normal	39	2060	$ 11,358,460	$39,040,261	$338,682,318	3.44	0.13
1980-above normal	40	2061	$ 14,377,207	$50,651,425	$345,128,253	3.52	0.13
1981-dry	41	2062	$ 10,148,005	$36,645,585	$349,607,045	3.61	0.12
1982-wet	42	2063	$ 36,660,200	$135,693,702	$365,534,410	3.70	0.12
1983-wet	43	2064	$ 34,696,545	$131,636,088	$380,373,395	3.79	0.11
1984-wet	44	2065	$ 28,658,258	$111,445,460	$392,438,652	3.89	0.11
1985-dry	45	2066	$ 5,623,601	$22,415,632	$394,769,265	3.99	0.10
1986-wet	46	2067	$ 16,573,016	$67,711,411	$401,530,501	4.09	0.10
1987-dry	47	2068	$ 3,722,262	$15,588,028	$403,025,360	4.19	0.10
1988-critical	48	2069	$ 2,896,424	$12,432,835	$404,170,410	4.29	0.09
1989-dry	49	2070	$ 10,005,906	$44,023,885	$408,064,334	4.40	0.09
1990-critical	50	2071	$ 1,279,260	$5,769,185	$408,554,404	4.51	0.08
NET PRESENT WORTH (50 YEARS)					**$408,554,404**		

* All costs calculated assumed 2010 on-peak and off-peak WAPA rates

6.5 Evaluation and Optimization of Intake Conveyance System

The Intake Conveyance System as described in the CER consists of a total of 5 intake structures on the Sacramento River, each capable of drawing up to 3,000 cfs. Intakes 3 through 5 convey flow from the Sacramento River through respective dual 16 ft inside diameter pipelines to the Intermediate Forebay (IFB). As Intakes 1 and 2 are located farthest from the IFB, the discharge from these two intakes is routed through dual 16 ft diameter pipelines from their respective locations to a single 29 ft interior diameter conveyance tunnel that starts near Scribner and River Road east of Highway 160 and continues south to the IFB.

The evaluation and optimization of the Intake Conveyance System was conducted according to the flowchart shown in Figure 6-4.

North Delta diversions were analyzed as described in Section 6.3.1. The energy cost of these diversions was calculated as described in Section 6.3.4. Relative construction costs were calculated as described in Section 6.3.5. Pumping plant O&M costs were calculated as described in Section 6.3.7. Present worth was calculated in accordance with Section 6.4 using the present worth factors described in Section 6.3.6. The lowest present worth alternative was then selected for a sensitivity analysis as described in Section 6.3.8.

6.5.1 Identification of Alternatives

Two primary alternatives were identified for evaluation; one was adding Intake No. 3 to the CER intake tunnel and one was adding both Intakes No. 3 and No. 4 to the CER intake tunnel. These two alternatives were selected to potentially reduce construction costs while maintaining similar hydraulic performance in the conveyance tunnel and pipelines and at the intake structures. The basic approach is to add additional flows from intake structures to a larger baseline tunnel, specifically Intakes 3 and 4, utilizing the tunnel lining system as described in the CER[1]. The baseline intake conveyance facilities can be seen in Figure 6-5.

Alternatives 1A and 1B modify the baseline by adding flows from Intake 3 into a larger conveyance tunnel. Alternative 1A keeps the same tunnel shaft location as in the baseline for Intakes 1 and 2, and adds another tunnel shaft and connection near Intake 3. Alternative 1A intake conveyance facilities can be seen in Figure 6-6. Alternative 1B modifies the tunnel shaft location for Intakes 1, 2, and 3 and brings all three intakes to one common tunnel shaft. Alternative 1B intake conveyance facilities can be seen in Figure 6-7.

Alternatives 2A and 2B modify the baseline by adding flows from Intakes 3 and 4 into a larger conveyance tunnel. Alternative 2A keeps the same tunnel shaft location as in the baseline for Intakes 1 and 2, and adds another tunnel shaft and connection near Intake 3 and also near Intake 4. Alternative 2A intake conveyance facilities can be seen in Figure 6-8. Alternative 2B keeps the modified intake shaft location described for Alternative 1B, and

[1] Pending verification that the proposed tunnel lining system can resist internal water pressure, and completion of a preliminary surge analysis, optimization of the tunnel diameter should be re-evaluated prior to start of Preliminary Engineering, or, to reduce internal pressure on the tunnel lining system for this tunnel segment, an evaluation into the feasibility to gravity-flow from each intake should be performed prior to start of Preliminary Engineering.

has a separate tunnel shaft connection near Intake 4. Alternative 2B intake conveyance facilities can be seen in Figure 6-9.

The tunnel diameters in the intake conveyance alternatives were preliminarily sized based on minimizing impacts to the hydraulic grade line (HGL). HGL is one of the key factors to evaluate the pump selection, to determine the number of pumps and their capacity. Due to the large size of these intake facilities (maximum 3,000 cfs or 1.3 million gpm), the pumping units are considered custom, so a dramatic change in HGL can affect the pump selection, which can increase the size and cost of individual pumps. However, a small change in HGL, up to approximately 5 feet, should not alter the pump selection significantly. As indicated in Table 6-5, the HGL calculated at the pump discharge increases for the alternatives due to increased friction and minor losses as more flow (higher velocities) has been added to the conveyance tunnel connecting the intakes to the IFB. In the case of Intakes 1 and 2, the increase in HGL is relatively minor, up to approximately 5 feet of additional headloss. For Intake 3, the increase in HGL is relatively minor for Alternatives 1A and 2A, but more substantial for Alternatives 1B and 2B, primarily due to friction losses from the longer conveyance tunnel and the longer pipelines that connect to this intake. Intake 4 has an increase in headloss just for Alternatives 2A and 2B, since this intake is added to the conveyance tunnel.

Table 6-5 Intake Conveyance Alternative HGL

	HGL AT PUMP DISCHARGE (ELEV NAVD 88)				
	Intake 1	Intake 2	Intake 3	Intake 4	Intake 5
Baseline	58.6	51.8	45.6	38.3	34.8
Alt 1A	58.5	52.3	48.5	38.3	34.8
Alt 1B	60.8	53.8	55.6	38.3	34.8
Alt 2A	59.4	53.2	51.2	43.6	34.8
Alt 2B	62.0	54.9	56.7	43.6	34.8

Table 6-6 shows the initial tunnel sizes evaluated based on minimizing the HGL impact and the resultant flow velocity by intake conveyance alternative.

Table 6-6 Intake Conveyance Tunnel Size Evaluation

Alternative	Description	Intake Conveyance Tunnel ID (ft)	Intake Conveyance Tunnel Flow (cfs)	Maximum Tunnel Velocity (fps)
0	Baseline (CER w/ Addendum) Tunnel Connecting Intakes 1 and 2 to IFB	29	6,000	9.1
1A	Add Intake 3 Pipeline to Tunnel 1, 2, Larger Tunnel for 1, 2, 3	33	9,000	10.5
1B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3	33	9,000	10.5
2A	Add Intake 3 and 4 Pipelines to Tunnel, Larger Tunnel for 1,2,3,4	36	12,000	11.8
2B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3, 4	36	12,000	11.8

Table 6-7 summarizes the pipeline lengths reflected in Figures 6-5 through 6-9 as well as the tunnel diameters shown in Table 6-6.

Table 6-7 Evaluation of Pipeline/Tunnel Configuration from Intake Pumping Plants to IPP

Alt	Description	Intake 1 Pipeline					Intake 2 Pipeline					Intake 3 Pipeline				
		Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)	Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)	Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)
0	Baseline (CER w/ Addendum)	(2) 16'	10,507	3,000	7.46	10.2	(2) 16'	1,439	3,000	7.46	3.4	(2) 16'	21,200	3,000	7.46	16.4
1A	Add Intake 3 Pipeline to Tunnel 1, 2, Larger Tunnel for 1, 2, 3	(2) 16'	10,507	3,000	7.46	9.7	(2) 16'	1,439	3,000	7.46	3.5	(2) 16'	3,505	3,000	7.46	4.9
1B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3	(2) 16'	15,400	3,000	7.46	13.0	(2) 16'	5,043	3,000	7.46	5.9	(2) 16'	7,787	3,000	7.46	7.8
2A	Add Intake 3 and 4 Pipelines to Tunnel, Larger Tunnel for 1,2,3,4	(2) 16'	10,507	3,000	7.46	9.7	(2) 16'	1,439	3,000	7.46	3.5	(2) 16'	3,505	3,000	7.46	4.9
2B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3, 4	(2) 16'	15,400	3,000	7.46	13.0	(2) 16'	5,043	3,000	7.46	5.9	(2) 16'	7,787	3,000	7.46	7.8

Alt	Description	Intake 4 Pipeline					Intake 5 Pipeline				
		Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)	Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)
0	Baseline (CER w/ Addendum)	(2) 16'	9,255	3,000	7.46	9.1	(2) 16'	5,641	3,000	7.46	5.5
1A	Add Intake 3 Pipeline to Tunnel 1, 2, Larger Tunnel for 1, 2, 3	(2) 16'	9,255	3,000	7.46	9.1	(2) 16'	5,641	3,000	7.46	5.5
1B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3	(2) 16'	9,255	3,000	7.46	9.1	(2) 16'	5,641	3,000	7.46	5.5
2A	Add Intake 3 and 4 Pipelines to Tunnel, Larger Tunnel for 1,2,3,4	(2) 16'	2,574	3,000	7.46	4.3	(2) 16'	5,641	3,000	7.46	5.5
2B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3, 4	(2) 16'	2,574	3,000	7.46	4.3	(2) 16'	5,641	3,000	7.46	5.5

Alt	Description	Combined Tunnel (1 + 2)					Combined Tunnel (1 + 2 + 3)					Combined Tunnel (1 + 2 + 3 + 4)				
		Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)	Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)	Diameter (ft)	Length (ft)	Capacity (cfs)	Velocity (fps)	Head Loss (ft)
0	Baseline (CER w/ Addendum)	29'	27,539	6,000	9.08	19.1	--	--	--	--	--	--	--	--	--	--
1A	Add Intake 3 Pipeline to Tunnel 1, 2, Larger Tunnel for 1, 2, 3	33'	11,431	6,000	7.02	5.2	33'	15,911	9,000	10.52	14.4	--	--	--	--	--
1B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3	--	--	--	--	--	33'	22,524	9,000	10.52	18.6	--	--	--	--	--
2A	Add Intake 3 and 4 Pipelines to Tunnel, Larger Tunnel for 1,2,3,4	36'	11,431	6,000	5.90	3.4	36'	9,536	9,000	8.84	7.0	36'	6,375	12,000	11.79	10.1
2B	Alternate Shaft Location for Intake 1, 2, 3 Pipelines, Larger Tunnel for 1, 2, 3, 4	--	--	--	--	--	36'	16,150	9,000	8.84	9.6	36'	6,375	12,000	11.79	10.1

6.5.2 Evaluation of Alternatives

Table 6-8 indicates the results of the present worth analysis for the intake conveyance alternatives.

All of the alternatives indicate some construction cost savings compared to the baseline due to variations in total pipeline length and tunnel lengths and diameters and number of tunnel shafts. Alternative 1A indicates slightly more construction cost savings than Alternative 2A, due to the smaller increase in tunnel diameter more than offsetting the decrease in Intake 4 pipeline length. Alternatives 1B and 2B have the least savings since each alternative has more pipeline costs than 1A and 2A, and in the case of 2B, there is some additional cost with the larger conveyance tunnel.

Table 6-8 Intake Conveyance System Present Worth Analysis

	Construction				PW O&M		Total PW Cost	
Alternative	Construction Costs with Contingency		Construction Total		Energy	Pump R&R		
	Pipeline	Tunnel	Total Costs	Add / Deduct	50 Year Cost	50 Year Cost	PW Cost	Add/ Deduct
Baseline	700	447	1,147	0	175	10	1,332	0
Alternative 1A	499	525	1,024	(123)	181	10	1,215	(117)
Alternative 1B	705	427	1,132	(15)	189	11	1,332	0
Alternative 2A	433	595	1,028	(119)	190	11	1,229	(103)
Alternative 2B	638	500	1,138	(9)	197	11	1,346	14

All of the alternatives showed additional energy cost due to the increased HGL at the intakes over the baseline alternative. All of the alternatives showed slightly higher pump R&R costs due to the pump cost increase over the baseline alternative.

Finding: Alternative 1A has the lowest total present worth.

6.5.3 Lowest Present Worth Intake Alternative

Alternative 1A was selected for further evaluation, based on the overall lower present worth. Alternative 1A would simplify the longest pipeline route from Intake 3 to the IFB by bringing flows into a larger conveyance tunnel.

6.5.4 Sensitivity Analysis

This section presents the results of the sensitivity analysis that was performed for Alternative 1A as part of defining minor refinements to the lowest present worth alternative described in Section 6.5.3 above. The sensitivity factors that were used focused primarily on tunnel diameter and variations in the discount rate and energy escalation.

Table 6-9 presents a summary of the sensitivity analysis results.

The table shows the following principal costs; the basis for these costs is defined in Section 6.3: Construction Cost (Tunnel and Pipeline); Energy Cost; and Total. The total is the sum of the cost values in the construction and energy columns to the left of it.

For the intake conveyance, construction contingency values for pipeline (25%) and tunnel (35%) are already included in the construction cost values.

All energy values shown are for the basic PW criteria that include 4-1/8% discount rate, 2.5% escalation, 50-year period of analysis, and 3-Hr diversion flow smoothing, unless otherwise noted.

Intake Tunnel Diameters. This analysis studied the feasibility of increasing or decreasing the tunnel diameters from the 33-foot ID lowest present worth alternative that would have the effect of changing the hydraulic friction factor and therefore changing energy costs. For this analysis, tunnel diameters of 32-foot ID, 34-foot ID, 35-foot ID and 36-foot ID were analyzed.

For the 32-foot ID tunnels, the reduction in construction cost from the lowest present worth alternative was estimated to be $19 million and the increase in present worth of energy cost was approximately $7 million. The net present worth benefit of implementing this facility design option is approximately $12 million.

For the 34-foot ID tunnels, the additional construction cost from the lowest present worth alternative was estimated to be $10 million and the reduction in present worth of energy cost was approximately $5 million. The net present worth cost of implementing this facility design option is approximately $5 million.

For the basic assumption that annual escalation of energy is 2.5%, decreasing the tunnel diameter to 32-foot ID would appear to be economically feasible. It should also be stated that the economics of changing to a 34-foot ID tunnel are very close as well and within the accuracy of the cost estimates being generated. This could mean that the tunnel diameter selection could be influenced by tunnel work to be performed on the main conveyance, in case that standardizing on a single diameter for all tunnels on the program might generate cost savings not currently accounted for here.

Tunnel Hydraulics and Pump Selection. The tentative decision to go to a 32-foot ID tunnel was checked to verify whether it could have an effect on intake pumping plant pump selection. The result of that review showed that the maximum flow velocity increases to 11.2 feet per second, and the maximum steady-state HGL increases by about 5% at Intakes 1, 2, and 3. These increases are not expected to adversely affect pump selection.

Table 6-9 PW Sensitivity Analyses Intake Conveyance

BASELINE	Construction	Pipeline	Tunnel	Energy	TOTAL	Difference to 33 ft Alternative 1A
Alternative 1A, 33 ft ID Tunnel	$1,024	$499	$525	$181	$1,205	$0
Alternative 2A, 33 ft ID Tunnel	$1,028	$433	$595	$190	$1,218	$13

Alternative 1A	Construction	Pipeline	Tunnel	Energy	TOTAL	Difference to 33 ft Alternative 1A
Tunnel Diameters (ID - feet):						
32	$1,005	$499	$506	$188	$1,193	($12)
34	$1,034	$499	$535	$176	$1,210	$5
35	$1,046	$499	$547	$171	$1,217	$12
36	$1,058	$499	$559	$167	$1,225	$20
Alternative 1A, 33 ft ID Tunnel	$1,024	$499	$525	$181	$1,205	$0
PW Criteria = 4-1/8% Discount, 3.0% Escalation	$1,024	$499	$525	$213	$1,237	$32
PW Criteria = 4-1/8% Discount, 3.5% Escalation	$1,024	$499	$525	$251	$1,275	$70
PW Criteria = 4-1/8% Discount, 4.0% Escalation	$1,024	$499	$525	$297	$1,321	$116
PW Criteria = 4-1/8% Discount, 5-1/2% Escalation	$1,024	$499	$525	$503	$1,527	$322
PW Criteria = 3% Discount, 2.5% Escalation	$1,024	$499	$525	$261	$1,285	$80
PW Criteria = 3% Discount, 3.0% Escalation	$1,024	$499	$525	$310	$1,334	$129
PW Criteria = 3% Discount, 3.5% Escalation	$1,024	$499	$525	$369	$1,393	$188
PW Criteria = 3% Discount, 4-3/8% Escalation	$1,024	$499	$525	$506	$1,530	$325
PW Criteria = 2.5% Discount, 2.0% Escalation	$1,024	$499	$525	$261	$1,285	$80
PW Criteria = 1% Discount, 5% Escalation	$1,024	$499	$525	$1,437	$2,461	$1,256
Alternative 1A, 32 ft ID Tunnel	$1,005	$499	$506	$188	$1,193	($12)
PW Criteria = 4-1/8% Discount, 3.0% Escalation	$1,005	$499	$506	$221	$1,226	($11)
PW Criteria = 4-1/8% Discount, 3.5% Escalation	$1,005	$499	$506	$260	$1,265	($10)
PW Criteria = 4-1/8% Discount, 4.0% Escalation	$1,005	$499	$506	$308	$1,313	($8)
PW Criteria = 4-1/8% Discount, 5-1/2% Escalation	$1,005	$499	$506	$522	$1,527	$0
PW Criteria = 3% Discount, 2.5% Escalation	$1,005	$499	$506	$271	$1,276	($9)
PW Criteria = 3% Discount, 3.0% Escalation	$1,005	$499	$506	$321	$1,326	($8)
PW Criteria = 3% Discount, 3.5% Escalation	$1,005	$499	$506	$382	$1,387	($6)
PW Criteria = 3% Discount, 4-3/8% Escalation	$1,005	$499	$506	$524	$1,529	($1)
PW Criteria = 2.5% Discount, 2.0% Escalation	$1,005	$499	$506	$270	$1,275	($10)
PW Criteria = 1% Discount, 5% Escalation	$1,005	$499	$506	$1,489	$2,494	$33

PW Criteria (Discount Rate & Escalation) Variations. This analysis of Alternative 1A studied the sensitivity of variations in discount rate and energy escalation on the decision to choose between a 32-foot ID and a 33-foot ID tunnel.

For this evaluation, as shown in Table 6-9, the present worth of energy was re-calculated assuming different discount rate and energy escalation assumptions.

The results of this evaluation are presented in Figure 6-10. The results show that two solutions are available; either Alternative 1A with a 32-foot ID tunnel or Alternative 1A with a 33-foot ID tunnel, depending on the values of discount rate and energy escalation used.

Reading the figure, the approximate breakeven is defined by the diagonal dashed line labeled "3-Hr Diversion Flow Smoothing." As shown, for higher cost of capital and lower energy escalation, the 32-foot ID tunnel project with the lower initial construction cost is favored. Conversely, the 33-foot ID tunnel project with higher initial construction cost is favored for lower cost of capital and higher energy escalation.

Reliability. Based on the findings of TO 20, TM 5, selecting Alternative 1A over Alternative 2A will provide a higher level of water conveyance reliability in the case of planned outages of the intake tunnel system. Under Alternative 1A, during a tunnel outage, Intakes 4 and 5 can continue to be operated and convey up to 6,000 cfs of river diversions to the IFB. Under Alternative 2A, the outage period conveyance flowrate would have been limited to 3,000 cfs.

6.5.5 Results of Analysis

The Intake Conveyance System can be one of two different configurations, depending on the eventual selection of present worth factors to be considered in confirming the economic feasibility of the project. To select only one alternative, the cost of capital should first be determined based on the method of financing the project and then the expected escalation of energy and goods and services can be found on the optimization chart, Figure 6-10, to then select the optimized alternative.

Optimized Alternative. The optimized alternative, based on a cost of capital and energy escalation located left of the breakeven line shown on Figure 6-10, is likely to be the Alternative 1A with a 32-foot ID tunnel, resulting in the following changes to the project components[2]:

[2]This recommendation is subject to verification of pump selection, verification that the proposed tunnel lining can resist the tunnel internal water pressure, and completion of a preliminary surge analysis, all of which should be performed as a first step in the Preliminary Engineering phase.

- Connect Intake PP 3 to the tunnel system rather than construct a pipeline from Intake PP 3 to the IFB.

- Change the tunnel diameter to 32-foot ID.

6.6 Evaluation and Optimization of Tunnel Conveyance System (15,000 cfs)

The Tunnel Conveyance System as described in the CER consists of two 33 foot diameter parallel tunnels, both of which have a gravity bypass around the IPP. The IPP was engineered for a total capacity of 15,000 cfs with the IFB water elevation varying between Elev. 10 and Elev. 25. Two sets of pumps are required to handle the flow ranges.

Figure 6-11 indicates the evaluation flowchart for the Tunnel Conveyance System alternatives.

North Delta diversions were analyzed as described in Section 6.3.1. The energy cost of these diversions was calculated as described in Section 6.3.4. Relative construction costs were calculated as described in Section 6.3.5. Pumping plant O&M costs were calculated as described in Section 6.3.7. Present worth was calculated in accordance with Section 6.4 using the present worth factors described in Section 6.3.6. The lowest present worth alternative was then selected for a sensitivity analysis as described in Section 6.3.8.

6.6.1 Prescreening of Alternatives

This section describes the prescreening process used to develop the list of alternatives identified in Section 6.6.2 for detailed evaluation.

The following "strawman" list of tunnel conveyance system alternatives was identified in the TO34 Work Plan and prescreened in this section (Alts 11 and 12 are not shown as they were evaluated by Team B):

Alt	Description
0	Baseline (CER w/ Addendum)
1	Eliminate Intermediate Forebay
2	All Low Head Pumps
3a	All Low Head Pumps; Gravity Tunnel A and Pump Tunnel B
3b	All Low Head Pumps; Pump Tunnels A and B - no gravity bypass
4	Eliminate Intermediate Pumping Plant
5	All High Head Pumps
6a	All High Head Pumps; Gravity Tunnel A and Pump Tunnel B
6b	All High Head Pumps; Pump Tunnels A and B - no gravity bypass
7	Relocate Intermediate Pumping Plant
8	Use Two (2) Intermediate Pumping Plants
9	Alternate Sizing of Intermediate Forebay
10	Use of PVC (polyvinylchloride) Liner in Tunnel

From the list above, a matrix was developed that included the size and capacity of all facility components for each configuration. This matrix is provided in Appendix P entitled "Strawman Matrix and Pump Selection".

The Strawman Matrix was reviewed and modified as follows to become the Screened Alternatives Matrix that is presented in the next section:

- Alt 1 from the Strawman Matrix was eliminated from future evaluation because a free water surface is required for the system to operate and alternating the size of the forebay is covered by Strawman Matrix Alt 9.
- Alt 2, 3a, 3b, 5, 6a, and 6b from the Strawman Matrix were combined into Alt 2.1, 2.a1, and 2.b1 in the Screened Alternatives Matrix as all are trying to reduce the number of pumps required for the Intermediate Pumping Plant.
- Alt 4 from the Strawman Matrix was carried forward as Alt 1 in the Screened Alternatives Matrix.
- Alt 7 and 8 from the Strawman Matrix were eliminated because Team A found that reducing internal tunnel pressure did not provide a cost benefit.
- Strawman Matrix Alt 9 became a sensitivity analysis criterion as it could benefit any of the Screened Alternatives.
- Alt 10 from the Strawman Matrix became a sensitivity analysis criterion as it could benefit any of the Screened Alternatives.

One possible alternative not included in the original strawman list of tunnel conveyance system alternatives is a single 45-foot inside diameter tunnel conveyance (compared to the other alternatives all using a two-tunnel arrangement). The single tunnel conveyance could be considered further in the future but, has been screened out at this time due to its lower level of operational reliability and challenges associated with structural design of the tunnel liner.

6.6.2 Identification of Alternatives

Table 6-10 identifies the alternatives remaining after prescreening. Two general types remain; pumped systems represented by Alternatives 2.1, 2.a1, and 2.b1; and a gravity system, represented by Alternative 1.

Baseline. The Baseline Alternative, as presented in the CER, consists of two 33-foot diameter parallel tunnels, both of which have a gravity bypass around the IPP. The IPP would be designed for a total capacity of 15,000 cfs with the IFB water elevation varying between Elev. 10 and Elev. 25. Two sets of pumps are required to handle the flow ranges. Ten high-head pumps with 18,000 Hp motors are required at high system head, and six low-head pumps with 8,000 Hp motors are required at low system head. The IFB has a bottom area of 760 acres, a circumference along the water side levee toe of 25,310 LF, and a levee top elevation of 32.2. The corresponding volume of the operating range is 11,860 AF and the full volume is 25,300 AF. The Baseline Alternative is shown in Figure 6-12.

Alternative 1. Same as Baseline except the IPP is eliminated and all flow is conveyed by gravity through the tunnels between the IFB and BTF. This alternative consists of increasing the two parallel tunnels to 45-ft diameter in order to decrease the headloss through the tunnels so that the maximum 15,000 cfs required output would be conveyed completely by gravity without the need for intermediate pumping. Alternative 1 is shown in Figure 6-13.

Alternative 2.1. Same as Baseline except it utilizes two 34-foot diameter parallel tunnels to convey flow from the IPP to the BTF. This increase in tunnel size would reduce the friction losses through the system and would reduce the intermediate pump size necessary to convey the maximum required output of 15,000 cfs. Similar to the baseline, both the tunnels would be provided with a gravity bypass to handle flow rates during lower flow demand. The IPP would be designed for a maximum capacity of 15,000 cfs utilizing a weir on the pumping plant discharge side to ensure a minimum backpressure to the pumps. A total of ten pumps, each rated for 1,500 cfs @ 76.5-ft, was determined to sufficiently meet all flow ranges not covered by the gravity bypass and to successfully eliminate the need for "low flow pumps" as was provided in the baseline. Alternative 2.1 is shown in Figure 6-14.

Alternative 2.a1. Alternative 2.a1 consists of two 40-foot diameter parallel tunnels, one of which is dedicated to gravity flow (Tunnel A) and the other tunnel (Tunnel B) dedicated to pumped flow. Tunnel A would have a capacity of 4,600 cfs (approximate demand of CVP) at a gradient of 14.5 ft. Both the tunnels are provided with gravity bypass to handle flow rates during low demand. The IPP would be designed for a total capacity of 10,500 cfs (approximate capacity of SWP) with the IFB water elevation varying between Elev. 10 and Elev. 25. A weir with a top of weir elevation of 30.00 ft will be installed on the pumping plant discharge side to ensure a minimum backpressure to the pumps. A total of seven pumps, each rated for 1,500 cfs @ 64 ft will be able to handle the flow rates. The pumping system is capable of handling lower flows than gravity flows. The pumps can be started at low speeds and depending on the system head, different pump combinations have to be used to cover the required operating range. The estimated pump motor HP is 15,000. Alternative 2.a1 is shown in Figure 6-15.

Alternative 2.b1. Alternative 2.b1 consists of two 36-foot diameter parallel tunnels, both of which are pumped flow tunnels without any gravity bypass. Capacity of each tunnel would be 7,500 cfs. The IPP would be designed for a total capacity of 15,000 cfs with the IFB water elevation varying between 10.0 and 25.0. A weir with a top of weir elevation of 30.0 ft will be installed on the pumping plant discharge side to ensure a minimum backpressure to the pumps. A total of ten pumps, each rated for 1,500 cfs @ 58.4 ft will be able to handle the flow rates. With a single type of pump, it is possible to cover the system flow range of 500 cfs to 15,000 cfs at low system head and 600 cfs to 15,000 cfs at high system head. The pumps can be started at low speeds and depending upon the system head, different pump combinations have to be used to cover the required operating range. The estimated pump motor HP is 14,000. Alternative 2.b1 is shown in Figure 6-16.

Table 6-10 Tunnel Conveyance System Alternative Facilities after Prescreening

Alt	Description	Intermediate Forebay						Tunnel Conveyance			Intermediate PP (IPP)						Gravity Bypass				
											High Head Pumps		Low Head Pumps		Medium Head Pumps			Capacity Each (cfs)			
		Surface Area, empty (acres)	Berm Height (ft NAVD88)	Total Storage (AF)[1]	Active Storage (AF)	Min. Elev (ft)	Max. Elev (ft)	Tunnel A Diameter (ft)	Tunnel B Diameter (ft)	Tunnel max HGL (ft NAVD88)	Number	HP	Number	HP	Number	HP	No. of Pipes	Max[2]	Int[3]	Min[4]	Diameter (ft)
0	Baseline (CER w/ Addendum)	760	32.2	25,300	11,850	10	25	33	33	90	10	18,000	6	8,000	0	0	2	4,000	3,260	1,200	26
1	Eliminate Intermediate Pumping Plant, Gravity Option	760	32.2	25,300	11,850	10	25	45	45	25	0	0	0	0	0	0	N/A	0	0	0	N/A
2.1	Single Pump Size in IPP	760	32.2	25,300	11,850	10	25	34	34	80	0	0	0	0	10	18,000	2	4,290	3,480	1,300	26
2.a1	Single Pump Size in IPP; Gravity Tunnel A and Pump Tunnel B; Gravity Bypass to both	760	32.2	25,300	11,850	10	25	40	40	70	0	0	0	0	7	15,000	1 (B)	6,210	5,050	1,900	30
2.b1	Single Pump Size; Pump Tunnels A and B - no gravity bypass	760	32.2	25,300	11,850	10	25	36	36	60	0	0	0	0	10	14,000	N/A	0	0	0	N/A
3	Combine Elements of Alt 1 and 2, Gravity one tunnel, pump second with single pump size	760	32.2	25,300	11,850	10	25	45	34	25/80	0	0	0	0	5	18,000	1	A: 9,350 B: 4,290	A: 7,500 B: 3,480	A: 2,850 B: 1,300	26

Notes:

1. Volume with no freeboard
2. Intermediate Forebay at Max Elevation 25', Byron Tract Forebay at 0.5'
3. Intermediate Forebay at Max Elevation 25', Byron Tract Forebay at 7.5'
4. Intermediate Forebay at Min Elevation 10', Byron Tract Forebay at 7.5'

Alternative 3. Alternative 3 combines elements of Alternative 1 and 2.1. This alternative consists of two parallel tunnels with one tunnel being 45-foot diameter and being utilized for gravity bypass flow only. The second tunnel would be 34-ft in diameter and would be primarily used for pump discharge with an option for gravity bypass operation. The gravity bypass would be designed for a total capacity of 7,500 cfs to be conveyed through the 45-ft diameter tunnel and would be utilized to primarily meet lower export flow conditions. Additional gravity flow capacity would be provided through the 34-ft diameter tunnel if necessary. The IPP would be designed for a total capacity of 7,500 cfs to be pumped through the 34-ft diameter tunnel. As previously discussed, a weir would be installed on the pumping plant discharge side to ensure a minimum backpressure to the pumps. A total of 5 pumps, each rated for 1,500 cfs @ 76.5-ft, was determined to sufficiently meet all flow ranges not covered by the gravity bypass and to successfully eliminate the need for "low flow pumps" as was provided in the baseline. The pump motor for this alternative was estimated to be 18,000 hp. Alternative 3 is shown in Figure 6-17.

6.6.3 Evaluation of Alternatives

Table 6-11 indicates the results of the present worth analysis for the tunnel conveyance alternatives. At this time, alternatives for both pumped and gravity systems can be carried forward to the preliminary engineering phase, if desired, given observations made by the Outside Review Panel regarding the possible range of operating pressures allowable for the tunnel liner system described in Section 4.3.6.

Finding - Gravity System: If it is later determined that the tunnel liner cannot withstand the operating pressures associated with the optimized pumped system, or if a second-pass liner becomes necessary, then the gravity system may become feasible and Alternative 1 could become the alternative having the lowest present worth cost.

Finding - Pumped System: Alternative 2.1 showed some direct construction savings compared to the baseline, and all of the alternatives showed a reduction in energy cost. For the pumped system, Alternative 2.1 has the lowest total present worth cost.

Table 6-11 Tunnel Conveyance System Present Worth Analysis

	Construction Costs										PW O&M			Total PW Cost	
Alternative	Direct Construction Costs								Construction Total		Energy	Pump R&R	Other O&M		
	Tunnel Other	Liner	Shaft	IPP	IPP Pipeline	Gravity Bypass	Surge Towers	Power	Total Costs	Add / Deduct	50 Year Cost	50 Year Cost	50 Year Cost	PW Cost	Add/ Deduct
Baseline	2,516	1,449	246	400	42	83	19	65	4,820	0	409	24	140	5,398	0
Alternative 1	3,334	1,986	246	0	83	0	0	55	5,704	884	175	0	0	5,879	486
Alternative 2.1	2,523	1,520	246	319	42	83	19	65	4,817	(3)	370	20	140	5,346	(47)
Alternative 2.a1	3,211	1,773	246	247	42	83	19	65	5,686	866	266	12	140	6,104	711
Alternative 2.b1	2,682	1,591	246	319	42	0	19	65	4,964	144	316	15	140	5,435	42
Alternative 4	3,095	1,753	246	199	42	83	19	65	5,502	682	266	10	140	5,918	525

6.6.4 Lowest Present Worth Alternative

In the November 10, 2010 Workshop, for the pumped systems, Alternative 2.1 was selected as the lowest present worth alternative for purposes of performing the sensitivity analysis.

6.6.5 Sensitivity Analysis of Pumped System

This section presents the results of the sensitivity analysis that was performed for Alternative 2.1 as part of defining minor refinements to the lowest present worth alternative described in Section 6.6.4 above. The sensitivity factors used were those generally described in Section 6.3.8.

Table 6-12 presents a summary of the sensitivity analysis results. The table shows the following principal costs; the basis for these costs is defined in Section 6.3: Construction Cost; Energy Cost; Pump R&R; Other O&M; and Total. The total is the sum of the cost values in the four columns to the left of it.

In addition, there are columns showing costs taken as differences between Alternative 2.1 and the alternatives to the baseline. Where the values are negative, it means the total cost of Alternative 2.1 is less than the other alternative by the amount shown. Where the values are positive, it means the total cost of Alternative 2.1 is greater than the other alternative by the amount shown.

All energy values shown are for the basic PW criteria that include 4-1/8% discount rate, 2.5% escalation, 50-year period of analysis, and 3-Hr diversion flow smoothing, unless otherwise noted.

Table 6-12 PW Sensitivity Analyses Tunnel Conveyance

BASELINE	Construction	Energy	Pump R&R	Other O&M	TOTAL		Difference to 33 ft Baseline	
Baseline Project - 50 Yr PW / Dual Operations, 33 ft ID Tunnel	$4,820	$409	$24	$141	$5,394		$0	
Modified Baseline Project - 50 Yr PW / Dual Operations, 32 ft ID Tunnel	$4,689	$459	$24	$141	$5,313		($81)	
Modified Baseline Project - 50 Yr PW / Dual Operations, 32 ft ID Tunnel; 6-Hr Smoothing	$4,689	$391	$24	$141	$5,245		($149)	
Alternative 2.1	**Construction**	**Energy**	**Pump R&R**	**Other O&M**	**TOTAL**	**Difference to Alt. 2.1**	**Difference to 33 ft Baseline**	**Difference to 32 ft Baseline**
Tentative Solution - Nov. 10	$4,817	$370	$20	$141	$5,348	$0	($45)	$36
IFB TOU Operation	$4,836	$282	$20	$141	$5,279	($70)		
Friction Factor Options (Epoxy Coating)	$5,043	$315	$20	$141	$5,519	$170		
Tunnel Diameters (ID - feet):								
35	$4,988	$337	$20	$141	$5,486	$138		
36	$5,168	$311	$20	$141	$5,640	$291		
Weir Elevation (Elevation - feet):								
IPP Weir 30 ft	$4,817	$370	$20	$141	$5,348	$0		
IPP Weir 35 ft	$4,817	$374	$20	$141	$5,352	$4		
IPP Weir 40 ft	$4,817	$379	$20	$141	$5,357	$9		
IPP Weir 45 ft	$4,817	$384	$20	$141	$5,362	$14		
Without Gravity Bypass	$4,734	$377	$20	$141	$5,272	($76)		
PW Criteria (Discount Rate & Escalation) Variations (3-Hr Smoothing):								
PW Criteria = 4-1/8% Discount, 3.0% Escalation	$4,817	$435	$28	$165	$5,445	$97	($48)	$25
PW Criteria = 4-1/8% Discount, 3.5% Escalation	$4,817	$512	$34	$196	$5,559	$211	($56)	$6
PW Criteria = 4-1/8% Discount, 4.0% Escalation	$4,817	$606	$40	$233	$5,696	$348	($65)	($16)
PW Criteria = 3% Discount, 2.5% Escalation	$4,817	$533	$35	$204	$5,589	$241	($58)	$1
PW Criteria = 3% Discount, 2.9% Escalation	$4,817	$611	$41	$235	$5,704	$356	($66)	($18)
PW Criteria = 3% Discount, 3.5% Escalation	$4,817	$753	$50	$291	$5,911	$563	($81)	($52)
PW Criteria = 3% Discount, 4.0% Escalation	$4,817	$901	$60	$349	$6,127	$779	($95)	($87)
PW Criteria = 2.5% Discount, 2.0% Escalation	$4,817	$533	$35	$204	$5,589	$241	($58)	$2
PW Criteria = 5% Discount, 4.5% Escalation	$4,817	$535	$35	$205	$5,592	$244	($58)	$1
Analyze Using 100 Yr Present Worth								
Baseline Project - 33 ft Tunnels	$4,820	$592	$98	$205	$5,715		$0	
Alternative 2.1 - 34 ft Tunnels	$4,817	$537	$80	$205	$5,639		($76)	
Comparison of BDCP Operating Scenarios - Isolated Conveyance Facility (ICF) Operations:								
Baseline Project - 33 ft Tunnels	$4,820	$629	$24	$141	$5,614		$0	
Alternative 2.1 - 34 ft Tunnels	$4,817	$568	$20	$141	$5,546		($68)	
Add Contingency to Construction Costs:								
Baseline Project - 33 ft Tunnels	$6,447	$409	$24	$141	$7,021		$0	
Alternative 2.1 - 34 ft Tunnels	$6,451	$370	$20	$141	$6,982		($38)	
PW Criteria (Discount Rate & Escalation) Variations (6-Hr Smoothing):	$4,817	$319	$20	$141	$5,297			$52
PW Criteria = 4-1/8% Discount, 3.0% Escalation	$4,817	$375	$28	$165	$5,385	$37		$45
PW Criteria = 4-1/8% Discount, 3.5% Escalation	$4,817	$442	$34	$196	$5,489	$141		$30
PW Criteria = 4-1/8% Discount, 4.25% Escalation	$4,817	$559	$37	$254	$5,667	$319		($4)
PW Criteria = 3% Discount, 2.5% Escalation	$4,817	$460	$35	$204	$5,516	$168		$26
PW Criteria = 3% Discount, 3.125% Escalation	$4,817	$564	$37	$254	$5,672	$324		($5)
PW Criteria = 3% Discount, 3.5% Escalation	$4,817	$649	$50	$291	$5,807	$459		($17)
PW Criteria = 3% Discount, 4.0% Escalation	$4,817	$777	$60	$349	$6,003	$655		($45)
PW Criteria = 2.5% Discount, 2.0% Escalation	$4,817	$459	$35	$204	$5,515	$167		$25
PW Criteria = 5% Discount, 5.125% Escalation	$4,817	$564	$37	$254	$5,672	$324		($5)
PW Criteria = 4-1/8% Discount, 5.25% Escalation	$4,817	$809	$35	$205	$5,866	$518		($235)

IFB Storage / IFB Size / IFB Water Depth. This analysis studied the feasibility of adding storage volume to the IFB that would be used to store peak river diversions occurring on tidal cycles for conveyance downstream of the IFB at a daily average rate. It used the diversion data from the dual operations scenario, but assumed a daily average flow pumped south from the IFB, rather than peak instantaneous flow from the intake pumping plants. Starting at a water surface elevation of Elev 25 (the high water level of the baseline IFB), this analysis increased the operating water depth in the IFB by 9 feet to Elev 34, which increased the IFB levee height to Elev 36 from the Elev 32.2 needed for flood protection. The IFB total water storage is increased by 7,500 AF (from 19,500 AF to 27,000 AF).[3]

Frequent daily water level changes in the IFB would have an adverse effect on the IFB embankment stability. As a result, for this option, the design of the embankment would be changed to a flatter water-side finished slope of 3.5:1 (from the 3:1 side slopes in the baseline IFB).

In addition, to operate the IFB water levels up to Elev 34, it will be necessary to raise the IPP weir elevation to Elev 40 feet or higher in order to provide adequate backpressure on the pumps so that they will always operate on the system curve (see "Weir Elevation" below).

The overall feasibility of this potential change was estimated as follows:

- Additional construction cost of IFB embankment	$11 million
- Additional construction cost to account for TDH increases to the pumps, motors and switchgear at the intake pumping plants	$7 million
- Reduction in present worth of energy cost due to operational changes	($88 million)
Subtotal	($70 million)
- Addition in present worth of energy cost due to change of IPP weir elevation to Elev 40	$9 million
- Net PW benefit	($61 million)

[3] It is recommended that prior to start of Preliminary Engineering that a variation of this option be studied that increases the IFB storage by increasing the area / footprint instead of increasing the maximum operating water level.

The net present worth benefit of implementing this facility design and operational scenario is approximately $61 million.

Finding: This change to the PTO configuration is therefore economically feasible.

Friction Factor Options. This analysis studied the feasibility of using alternative tunnel coatings or tunnel linings that would have the effect of reducing the hydraulic friction factor and therefore reducing energy costs. Team A provided several possible options for this analysis, including polyurethane or epoxy coatings, for example (Section 4). For this analysis, an epoxy coating was assumed to be applied to the interior surfaces of the conveyance tunnels.

At an average construction cost of $6 per square foot for the epoxy coating of both tunnels, the additional construction cost was estimated to be $226 million and the reduction in present worth of energy cost was approximately $55 million. The net present worth increase of implementing this facility design option is approximately $170 million.

Finding: This change to the PTO configuration is therefore not economically feasible.

Tunnel Diameters. This analysis studied the feasibility of increasing the tunnel diameters that would have the effect of reducing the hydraulic friction factor and therefore reducing energy costs. For this analysis, tunnel diameters of 35 foot ID and 36 foot ID were analyzed.

For the 35 foot ID tunnels, the additional construction cost was estimated to be $171 million and the reduction in present worth of energy cost was approximately $33 million. The net present worth increase of implementing this facility configuration option is approximately $138 million.

For the 36 foot ID tunnels, the additional construction cost was estimated to be $351 million and the reduction in present worth of energy cost was approximately $59 million. The net present worth increase of implementing this facility configuration option is approximately $291 million.

Finding: For the basic assumption that annual escalation of energy is 2.5%, increasing tunnel diameters is not economically feasible.

Weir Elevation. This analysis studied the energy cost impacts associated with changing the IPP weir height. In the baseline case, the weir downstream of the IPP has a rim elevation of 30 feet to provide back pressure to the intermediate pumps that aids pump operation. A higher downstream weir elevation will increase the back pressure on the pumps during start up and low flow conditions and will allow the pumps to meet lower conveyance flow conditions at higher reduced speeds. Additionally, if the IPP gravity bypass was eliminated or if the WSE in the IFB was raised, the proposed pumping units would have more difficulties to operate at low flow and low head conditions without a raised weir. This factor was analyzed in order to obtain a better understanding of the potential energy cost of raising the weir and potential benefits to pumping plant operation.

Finding: As shown in Table 6-12, the present worth of energy for raising the weir elevation from 30 feet to 40 feet is relatively small, only $9 million based on a total of $370 million. As a result, it is recommended that the elevation of the weir be adjusted higher as needed for proper pump operation.

Operation Without the IPP Gravity Bypass. This analysis studied the feasibility of eliminating the IPP gravity bypass and associated gravity piping and valves between the IFB and the beginning of the conveyance tunnels. It is essentially a test of the economic feasibility of installing a gravity bypass system.

Elimination of the gravity bypass would reduce the project construction cost but at the same time increase the range required for the pumping system to meet low flow conveyance conditions, and it will increase pumping energy. To account for this, it will be necessary to raise the IPP weir elevation; for this evaluation, it is assumed to be raised to Elev. 40.

The overall feasibility of this potential change was estimated as follows:

- Reduction in Construction Cost due to elimination of Gravity Bypass	($83 million)
- Increase in present worth of energy cost due to operational changes	<u>$7 million</u>
Subtotal	($76 million)
- Addition in present worth of energy cost due to change of IPP weir elevation to Elev 40	$9 million
- Net PW benefit	($67 million)

Finding: The net present worth benefit of implementing this facility configuration and operational scenario is approximately $67 million. For the basic assumption that annual escalation of energy is 2.5%, eliminating the gravity bypass system is therefore economically feasible.

PW Criteria (Discount Rate & Escalation) Variations. This analysis studied the sensitivity of variations in discount rate and energy escalation on the selection of Alternative 2.1 as the tentative solution. It is essentially a test of whether there should be a second optimized solution under certain economic conditions.

For this evaluation, as shown in the table, the present worth of energy, pump R&R and O&M costs were re-calculated assuming different discount rate and energy escalation assumptions.

The results of this evaluation are presented in Figure 6-18. The results show that either a baseline project configuration with two 32-foot ID tunnels or Alternative 2.1 with 34-foot ID tunnels would be optimal depending on the values of discount rate and energy escalation used.

Reading from the figure, the approximate breakeven points are defined by the diagonal dashed line labeled "3-Hr Diversion Flow Smoothing." As shown, for higher cost of capital and lower energy escalation, the 32-foot ID tunnel project with the lower initial construction cost is favored. Conversely, the 34-foot ID tunnel project with higher initial construction cost is favored for lower cost of capital and higher energy escalation.

Figure 6-18 also shows the influence of operational enhancements on the outcome. The dashed line labeled "6-Hr Diversion Flow Smoothing" indicates the breakeven points for the 6-Hr running average of river diversions is moved slightly to the right. The 6-Hr running average approximates a more operator-friendly, energy efficient operating scheme that allows the peak river diversions to be spread out over longer durations, thus reducing the overall energy consumption.

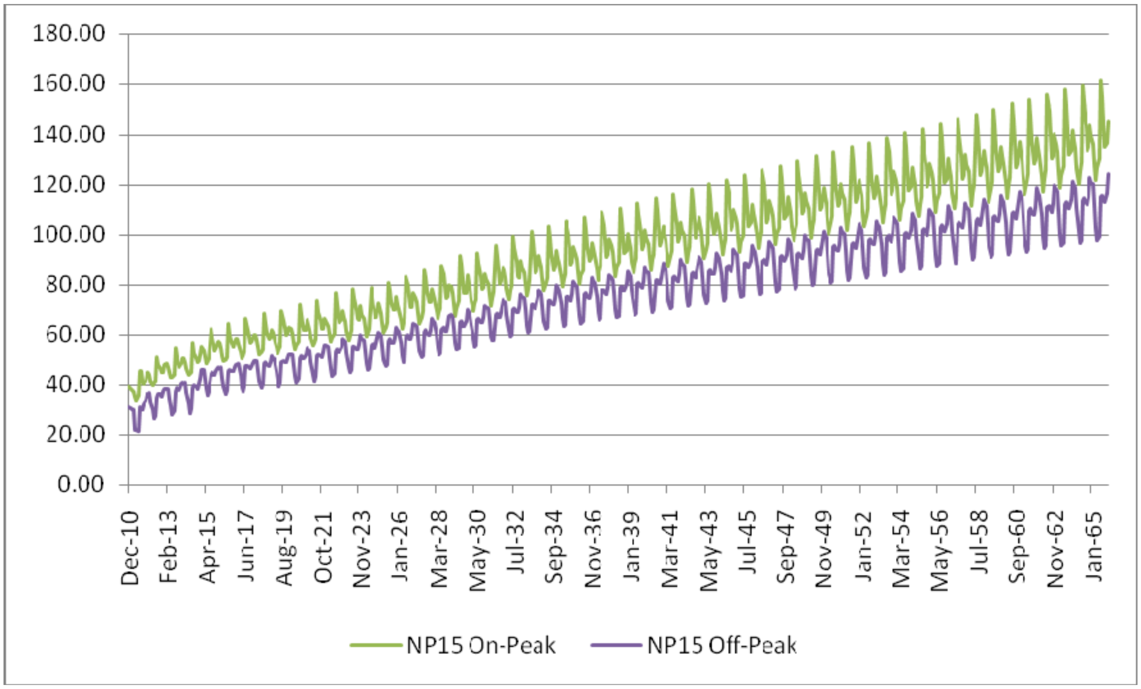
Finding: The result on the optimized project suggests it will take a greater value of energy escalation to justify the larger tunnel diameters if the Department is able to operate the project in this manner.

Energy Unit Cost Variation: As stated earlier, energy costs were based on WAPA energy rates.

Forecasted energy rates for the period 2010 – 2065 were also received from the State Water Project PARO, as shown in Figure 6-19, Forecasts for Power and Power-Related Costs, with $/MW-hr values shown for the vertical axis.

Upon comparison with WAPA rates, the PARO rates for 2010 are slightly lower overall than the WAPA rates, and the average annual escalation was calculated to be approximately 2.4%.

Figure 6-19: Forecasts for Power and Power-Related Costs (PARO)



Finding: As a result, the WAPA rates used in the present worth analysis are providing a slightly higher energy cost than if PARO rates were used. In the future, if the electrical power for the project is not supplied by WAPA and PARO rates are used, then it means the dashed lines shown on Figure 6-18 would shift slightly to the right.

Present Worth Calculation – 100-Yr Analysis Period. This analysis studied the sensitivity of the PW period of analysis on the selection of Alternative 2.1 as the tentative solution.

Finding: The result shows that increasing the PW period of analysis to 100 years does not change the outcome of selecting Alternative 2.1 compared to the baseline project.

Comparison of BDCP Operating Scenarios - Isolated Conveyance Facility (ICF) Operations. This analysis studied the sensitivity of the BDCP Operating Scenario on the selection of Alternative 2.1 as the tentative solution.

Finding: The result shows that using the ICF Operating Scenario does not change the outcome of selecting Alternative 2.1 compared to the baseline project.

Add Contingency to Construction Costs. This analysis studied the sensitivity of including the construction cost contingency as part of the evaluation on the selection of Alternative 2.1 as the tentative solution.

Finding: The result shows that including the construction cost contingency does not change the outcome of selecting Alternative 2.1 compared to the baseline project.

6.6.6 Results of Analysis

The optimized alternative for the Tunnel Conveyance, pumped system, can be one of two different projects, depending on the eventual selection of present worth factors to be considered in confirming the economic feasibility of the project. To reach the conclusion, the cost of capital should first be determined based on the method of financing the project, and then the expected escalation of energy and goods and services can be found on the optimization chart to select the optimized alternative.

Optimized Alternative. The optimized alternative, based on a cost of capital and energy escalation located left of the breakeven line shown on Figure 6-18, is likely to be the Baseline Project with 32-foot ID tunnels, resulting in the following changes to the project components [4]:

- Revise the tunnel diameters from 33-foot ID to 32-foot ID.
- Raise the IFB embankments and flatten the water-side slope to 3.5:1 to achieve a maximum water surface operating level of Elev. 34; this will allow for the storage of peak river diversions during tidal cycles in the IFB. The IPP can then operate more on the basis of conveying average daily flows that are more uniform than the peak diversions.
- Delete the gravity bypass system as a cost savings measure. Establish the system minimum system conveyance flowrate value to be approximately 755 cfs.
- Raise the IPP weir to Elev. 40 in response to raising the IFB operating level.
- Raise the top rim elevation of the IPP surge tower by approximately 15 feet.

A hydraulic profile of the optimized alternative, pumped system, is shown as Figure 6-20.

6.7 Evaluation and Optimization of Tunnel Conveyance System (Export-Driven Scenario)

The Department of Water Resources Division of Engineering (the Department) developed a scenario to be evaluated for optimization of Pipeline/Tunnel Option facilities. Specifically, the Department sought to optimize the main conveyance tunnel diameter, Intermediate Pumping Plant size, Intermediate Forebay size, and Byron Tract Forebay size for diversions

[4]This recommendation is subject to verification of pump selection, verification that the proposed tunnel lining can resist the tunnel internal water pressure, and completion of a preliminary surge analysis, all of which should be performed as a first step in the Preliminary Engineering phase.

and exports that were more closely scheduled to meet the historical operation of the Central Valley Project pumping plant (Jones) at 4,600 cfs capacity and a preference for 24/7 operation and the State Water Project pumping plant (Banks) at 10,300 cfs capacity and a preference for off-peak pumping [10pm to 6am].

The Department's report entitled "Optimization Study of the Southern Conveyance" is presented in Appendix Q.

6.8 Dual Operations

The facilities in the CER were engineered on the basis of Isolated Conveyance Facility Operation, that is, no future diversions from Old River into either Clifton Court Forebay to supply the SWP or into the Delta-Mendota Canal to supply the CVP. However, flow diversions are also being modeled as a dual diversion scenario as described in Section 6.3.1. This section will evaluate the adequacy of utilizing the baseline facilities under this new operational scenario.

6.8.1 Baseline Facilities

The facilities that could be utilized during dual operations were divided geographically into north and south. The northern facilities include the proposed intake pumping plants, the Intermediate Pumping Plant and the IFB, as discussed previously. Southern facilities include the existing Clifton Court Forebay (CCF) and the proposed Byron Tract Forebay (BTF). Figure 6-21 shows the potential water levels and storage volumes available to operate the new system. Figure 6-22 shows the existing and proposed southern facilities.

Intermediate Forebay Storage. The IFB is required to balance the mismatch between the rate water can be withdrawn from the Sacramento River and the rate water would be conveyed south via the IPP. This mismatch in rates will vary during the course of a day and is dependent upon Sacramento River flow, tidal conditions, and the operation of Jones and Banks Pumping Plants. Appendix H of the CER showed a need to store up to 10,400 cfs for a period of six hours. This storage equates to approximately 5,200 AF. The normal operating range of 7 feet at the Banks and Jones Pumping Plants was also applied to the proposed IFB. An active storage volume of 5,200 AF would require a water surface area of approximately 750 acres (i.e., 5,200 AF divided by 7 feet). The IFB provided in the CER actually had a bottom area of 760 acres, a circumference along the water side levee toe of 25,310 LF, an operating range between EL 10 and 25, and a levee top elevation of 32.2. The corresponding volume of the CER operating range is 11,860 AF and the full volume is 25,300 AF.

Clifton Court Forebay Storage. Section 4.1.1.1 of the CER identified a maximum design operating storage of 28,653 AF at the maximum design operating WSE of +8.1 feet. Figure 6-22 shows the existing and proposed CCF facilities. The existing radial gates are in the southeast portion of the CCF. The new gates are on the west side of the CCF, just upstream of the Skinner Fish Facility.

Byron Tract Forebay Storage. Appendix H of the CER showed a need to store up to 1,933 AF. With an operating range of 7 feet at the Banks and Jones Pumping Plants, approximately 276 acres of storage was needed. The BTF provided in the CER actually had a bottom area of 600 acres and a levee top elevation of 24.5 feet. The corresponding volume of the CER operating range is 4,200 AF and the full volume is 20,700 AF. Figure 6-22 also shows the new BTF facilities. New gates are provided prior to discharge from BTF to the SWP to the west and the CWP Delta-Mendota Canal to the south. An additional gate structure is required in the Delta-Mendota Canal just upstream of the new discharge from the BTF.

This analysis shows that much more storage capability exists in the new forebays than was required for the CER assumptions. The CER identified the need for 7,133 AF, while the proposed forebays provide 16,000 AF within the operating ranges currently proposed.

6.8.2 Dual Operations Scenario

In order to evaluate the adequacy of the existing and proposed facilities for dual operations, several operating scenarios were developed. These scenarios are presented in Table 6-14. The Diversion Category shows the origination of the flow and the Operating Rules identify how all of the facilities might be operated.

Categories 1 and 2 include only north Delta diversions and are differentiated by whether they are long-term or short-term, respectively. The remainder of the Diversion Categories includes a south Delta diversion component. Category 3 includes short-term diversions from the north and only diversions into CCF from the south. Category 4 includes both diversions from the south, which is how the system is currently operated. Category 5 includes diversions from both the north and south.

Table 6-13 Operations Diversion Categories

Diversion Category			Operating Rules								
No.	**North Delta Diversions**	**South Delta Diversions**	**Intakes / Intake Pumping Plant**	**Intermediate Forebay**	**Intermediate Pumping Plant / Gravity Bypass**	**Byron Tract Forebay**	**Clifton Court Forebay (diversion from Old River)**	**Delta-Mendota Canal (diversion from Old River)**	**SWP Export Pumping Plant (Banks)**	**CVP Export Pumping Plant (Jones)**	**Remarks**
1	0 – 15,000 cfs Daily, or Extended Duration	CCF 0 cfs DMC 0 cfs	Pumps On NORMAL	Water Surface Constant Level Water In = Water Out	Flow Pumped = Flow Diverted	Water In = Water Out	Gates Closed	Gates Closed	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	
2	0 – 15,000 cfs Short-Term Diversion	CCF 0 cfs DMC 0 cfs	Pumps On TIDAL CYCLE	Water Surface Constant Level Water In = Water Out	Flow Pumped = Flow Diverted	Water In = Water Out	Gates Closed	Gates Closed	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	
3	0 – 15,000 cfs Short-Term Diversion	CCF Tidal DMC 0 cfs	Pumps On TIDAL CYCLE	Water Surface Constant Level Water In = Water Out	Flow Pumped = Flow Diverted	Water In = Water Out	Gates Open at Tidal Periods / Store Water Until Later	Gates Closed	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	Water stored at CCF is released for export later when North Delta Diversions are reduced such that Banks pumping capacity is not exceeded.
4	0 cfs	CCF Tidal DMC 0 – 4600 cfs	OFF	OFF	OFF	Gates Closed	Gates Open at Tidal Periods	Gates Open	Flow Pumped = Diverted Flow (Diverted Tidally & Stored)	Flow Pumped = Diverted Flow	Same as Current Operation
5	0 – 15,000 cfs	CCF Tidal DMC 0 – 4600 cfs	CURTAIL NORTH DELTA DIVERSIONS?	STORE EXCESS FLOWS AT IFB?	Flow Pumped = Flow Diverted	Water In = Water Out	Gates Open at Tidal Periods / Store Water Until Later	Gates Open	Flow Pumped from BTF = Diverted Flow Split for SWP/CVP	Flow Pumped = Deliveries from BTF & Diversions from Old River	

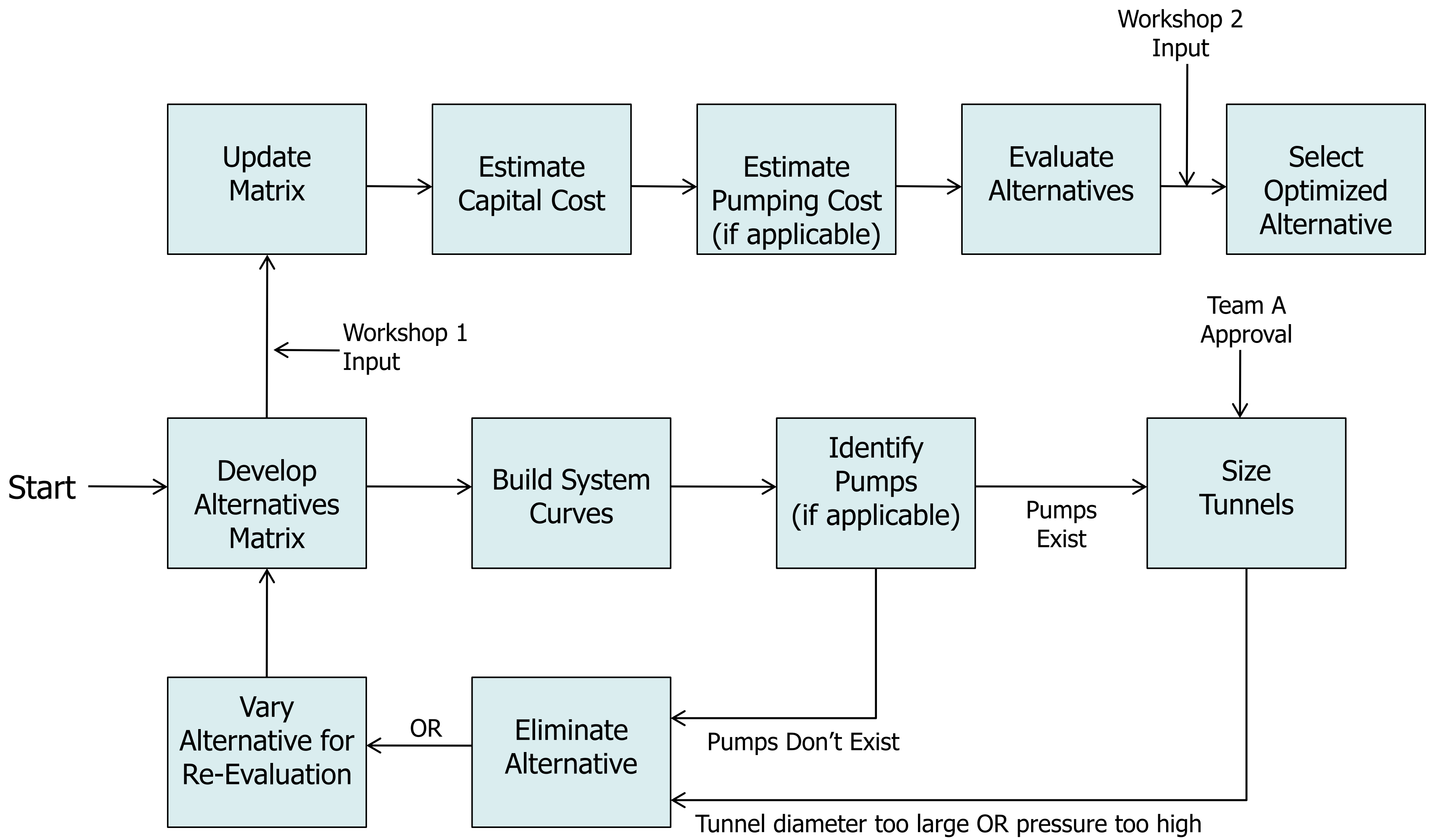
An attempt was made to identify what percentage of time flow diversions occurred in each operating category. Table 6-14 shows the results of placing every 15 minute data point from the BDCP Dual Operating Scenario described in Section 6.3.1 into each of the categories described in Table 6-13. It shows that the northern diversions occur more frequently during wet years and the southern diversions occur more frequently during dry years. The maximum mismatch between total diversions and export was a nine hour period (April 4, 1986 from 00:45 to 09:30) when 4,560 AF was diverted in excess of what could be exported. This was well within the maximum 53,500 AF capacity of the new forebays and the CER minimum of 7,133 AF identified previously. During discussions with the BDCP modelers, it was learned that the daily maximum diversion capacity is capped at 14,900 cfs, the maximum combined capacity of the export facilities. Since there is more forebay storage available than required by this modeling effort, additional modeling could show the ability to divert more flow when conditions are favorable than to limit diversion to the capacity of the export facilities.

Table 6-14 Proposed Dual Operating Scenario Percent Occurrence by Diversion Category

Year	Year Type (DWR)	Diversion Category				
		1	2	3	4	5
1975	Wet	16%	2%	14%	44%	23%
1976	Critical	0%	0%	9%	75%	17%
1977	Critical	0%	0%	0%	95%	5%
1978	Above Normal	23%	5%	13%	45%	14%
1979	Below Normal	3%	1%	14%	60%	22%
1980	Above Normal	17%	8%	0%	55%	21%
1981	Dry	0%	0%	1%	73%	26%
1982	Wet	37%	5%	8%	28%	23%
1983	Wet	60%	22%	1%	8%	9%
1984	Wet	20%	5%	0%	41%	34%
1985	Dry	0%	0%	0%	80%	20%
1986	Wet	16%	0%	10%	60%	13%
1987	Dry	0%	0%	0%	84%	16%
1988	Critical	0%	0%	0%	88%	12%
1989	Dry	1%	0%	5%	77%	18%
1990	Critical	0%	0%	0%	91%	9%
1991	Critical	0%	0%	0%	91%	9%
Average		11%	3%	4%	64%	17%
Wet 1983		60%	22%	1%	8%	9%
Normal 1980		17%	8%	0%	55%	21%
Dry 1989		1%	0%	5%	77%	18%

Criteria/method used to identify categories:

1. Category 1 and 2: CCF at 0 for more than 8 hrs and DMC at 0 (e.g. it is a Category 1 or a Category 2)
2. Category 2: it is a Category 1 and 2 as above described and north diversion at 0 for 4 hrs
3. Category 1: remainder of Category 12 that is not a Category 2
4. Category 3: DMC at 0 and other than Category 1 and 2
5. Category 4: other than Category 1 and 2, other than Category 3 and north diversion at 0
6. Category 5: all the rest



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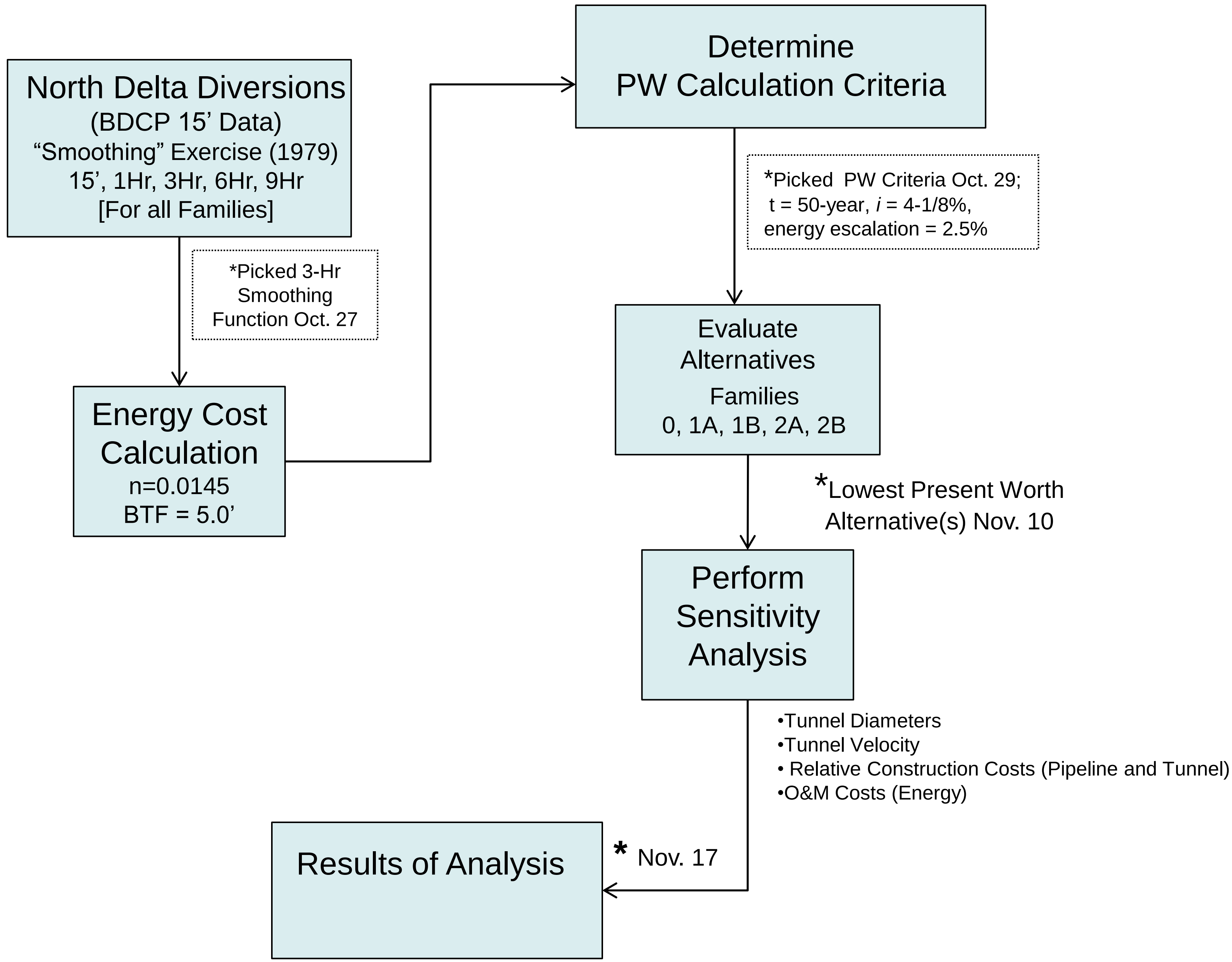
PIPELINE/TUNNEL OPTION

Sizing Alternatives and Definition Flow Chart

FIGURE
6- 1



* Decision

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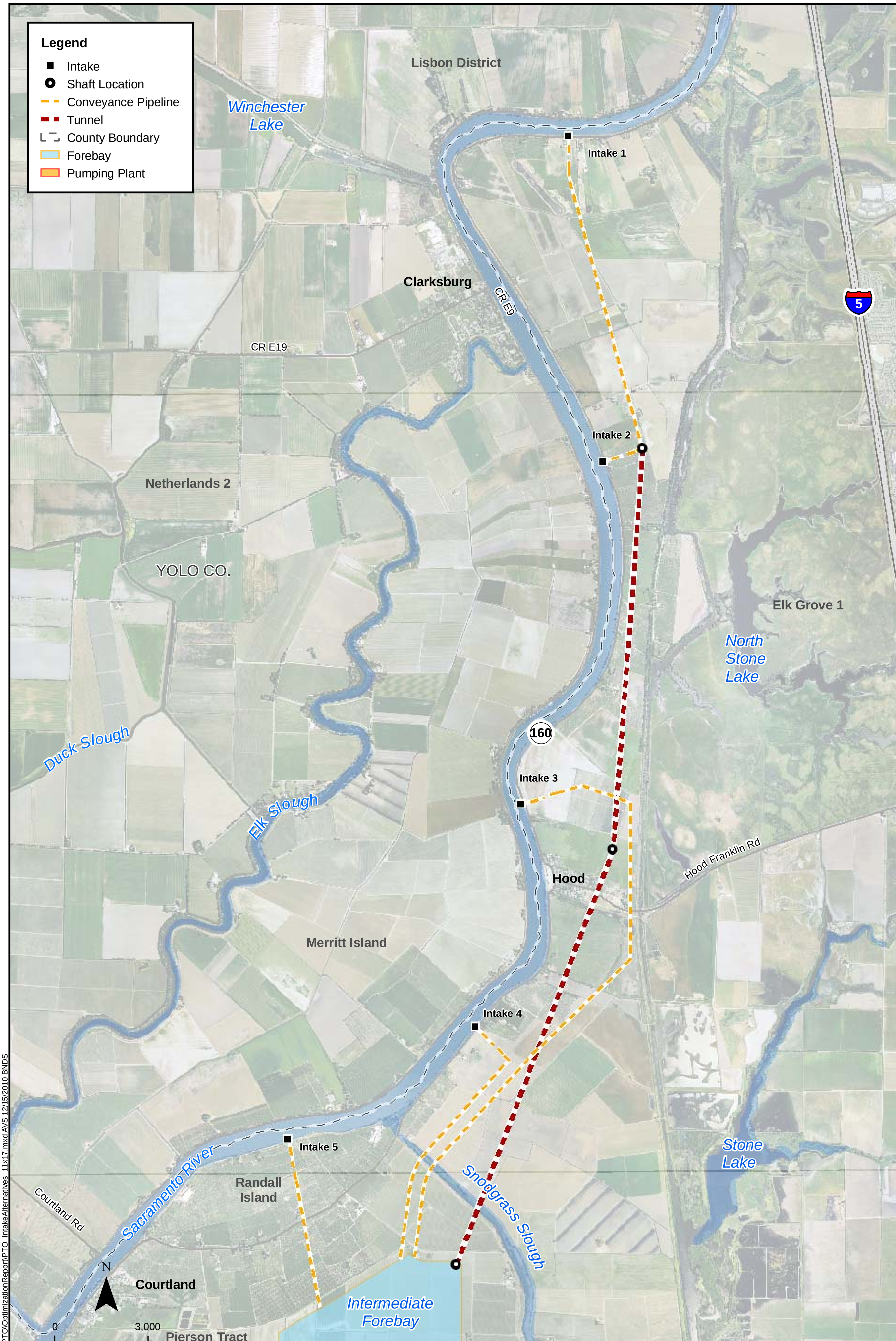
Intake Conveyance Options Optimization Evaluation Flowchart

FIGURE

6- 4



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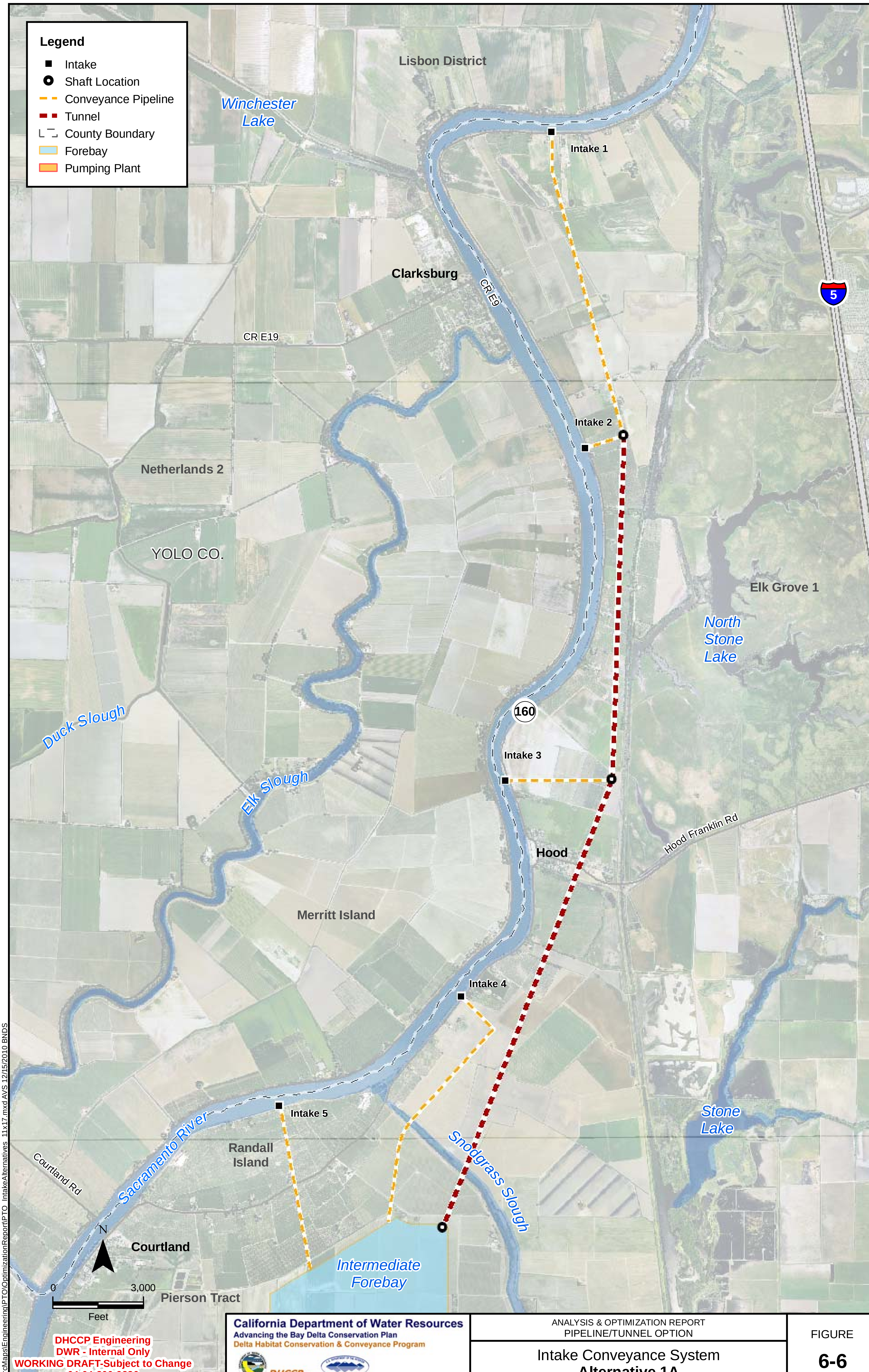
PIPELINE/TUNNEL OPTION

Intake Conveyance System
Baseline

FIGURE
6-5



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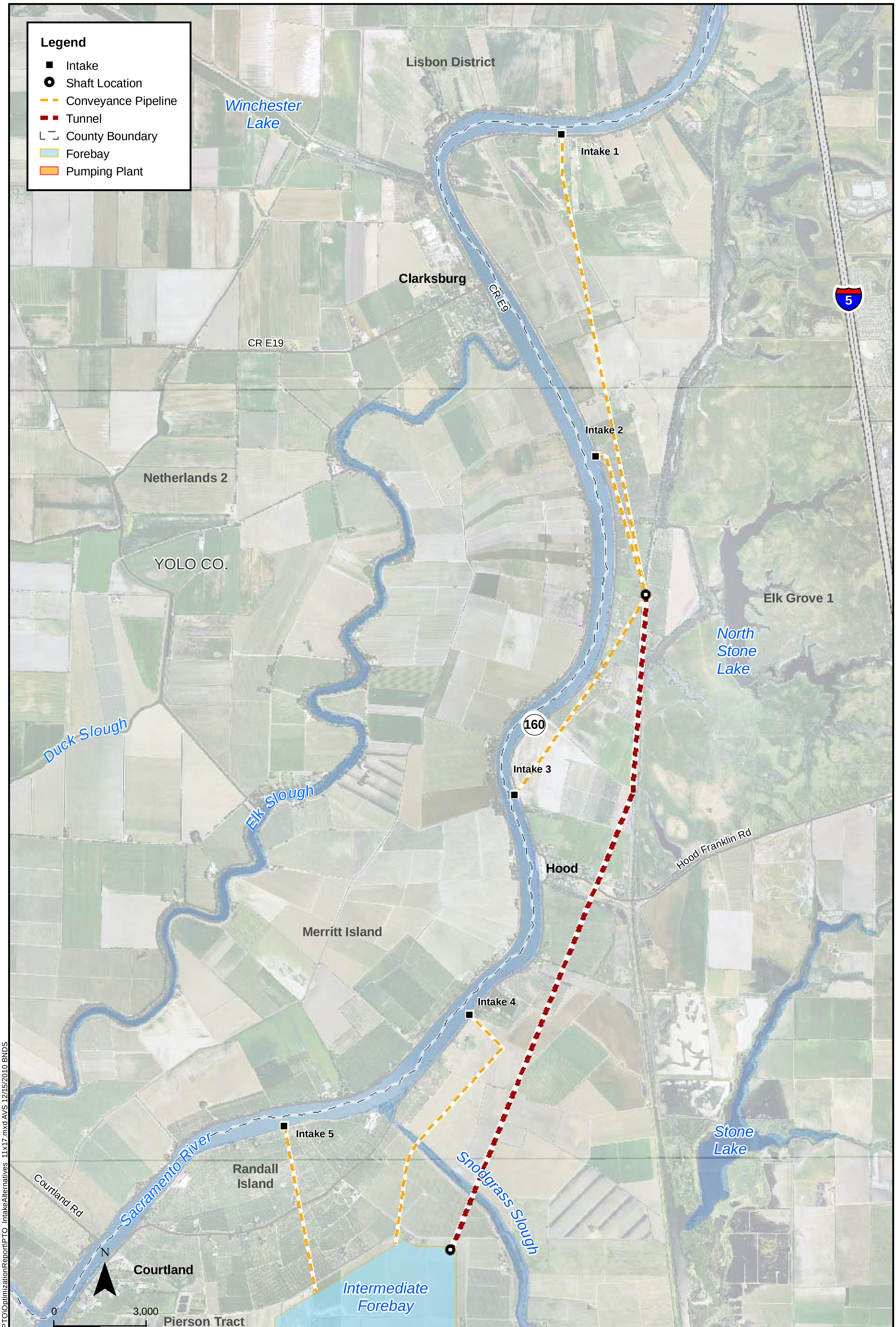
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Intake Conveyance System
Alternative 1A

FIGURE
6-6



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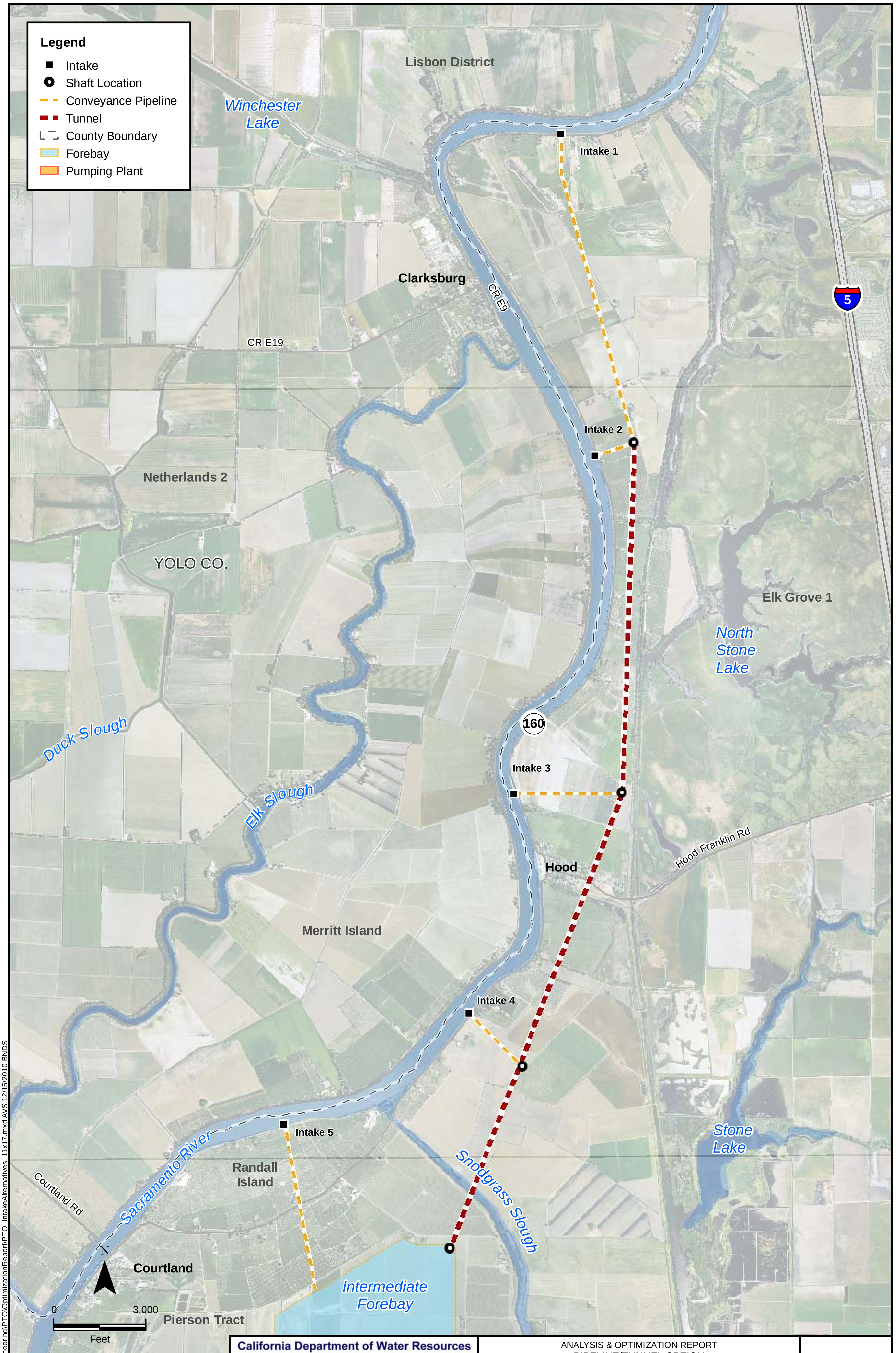
PIPELINE/TUNNEL OPTION

Intake Conveyance System
Alternative 1B

FIGURE
6-7



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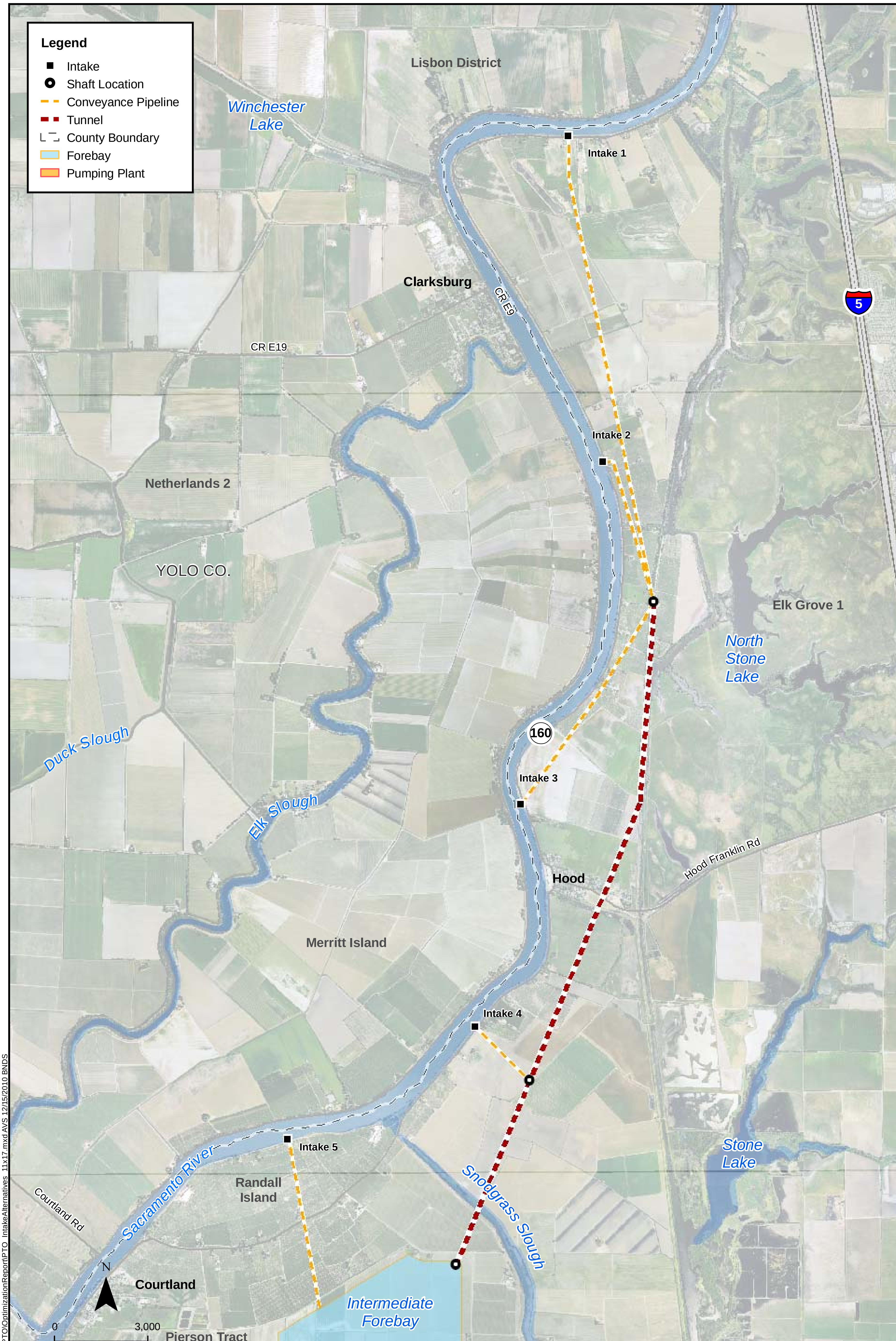
PIPELINE/TUNNEL OPTION

Intake Conveyance System
Alternative 2A

FIGURE
6-8



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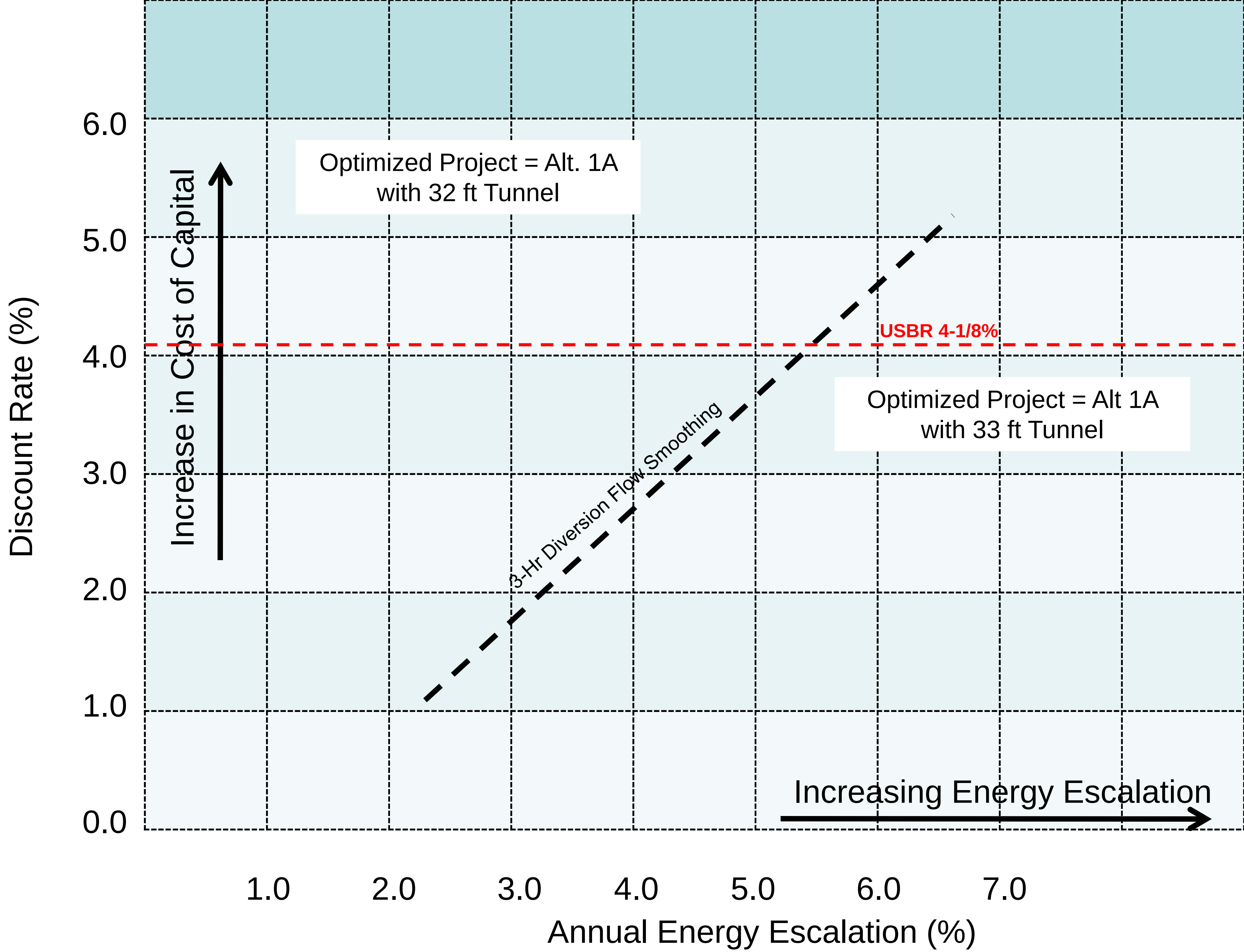
PIPELINE/TUNNEL OPTION

Intake Conveyance System
Alternative 2B

FIGURE
6-9



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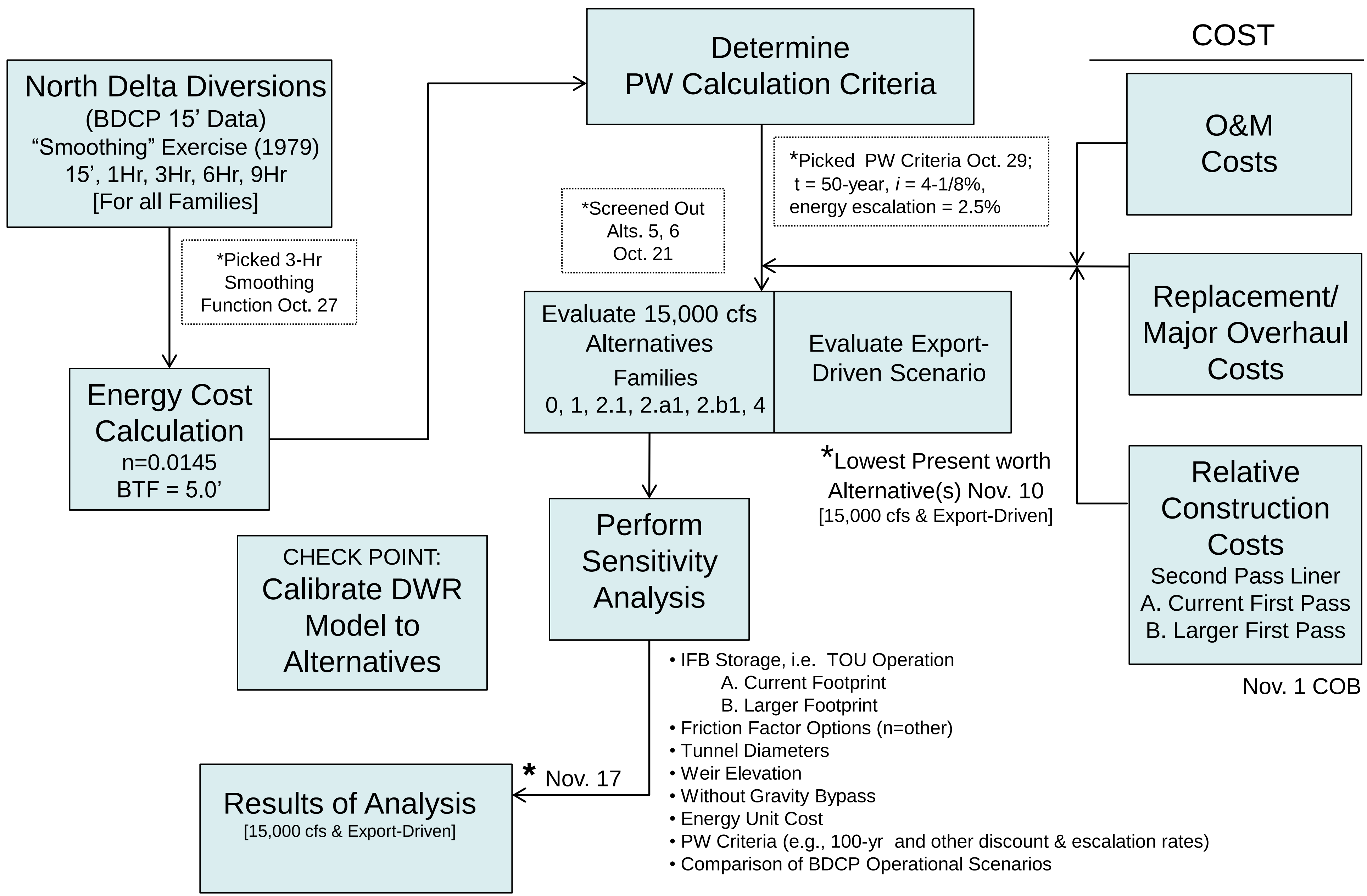

* Decision

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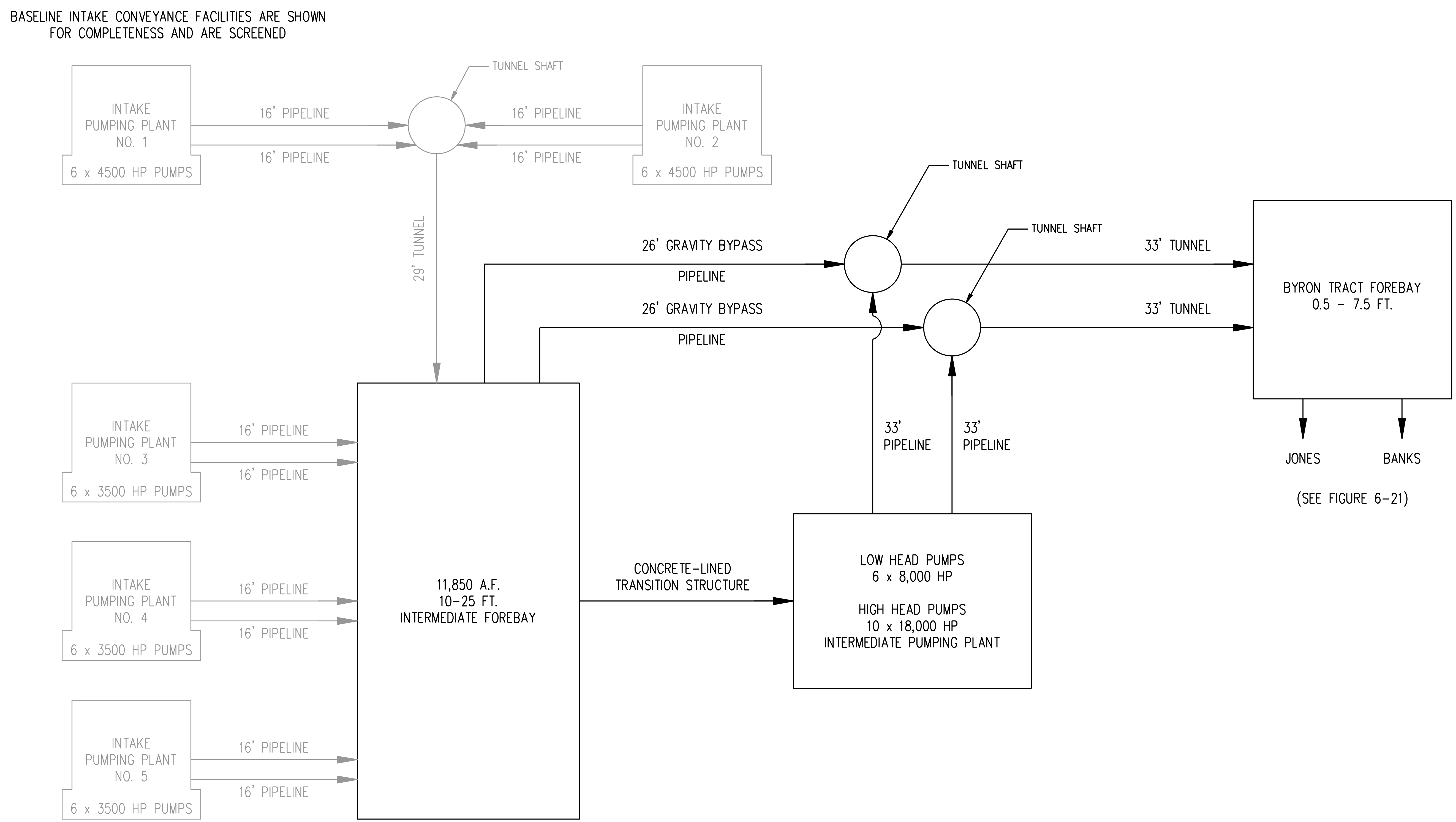
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Tunnel Conveyance Options Optimization Evaluation Flowchart

FIGURE

6-11



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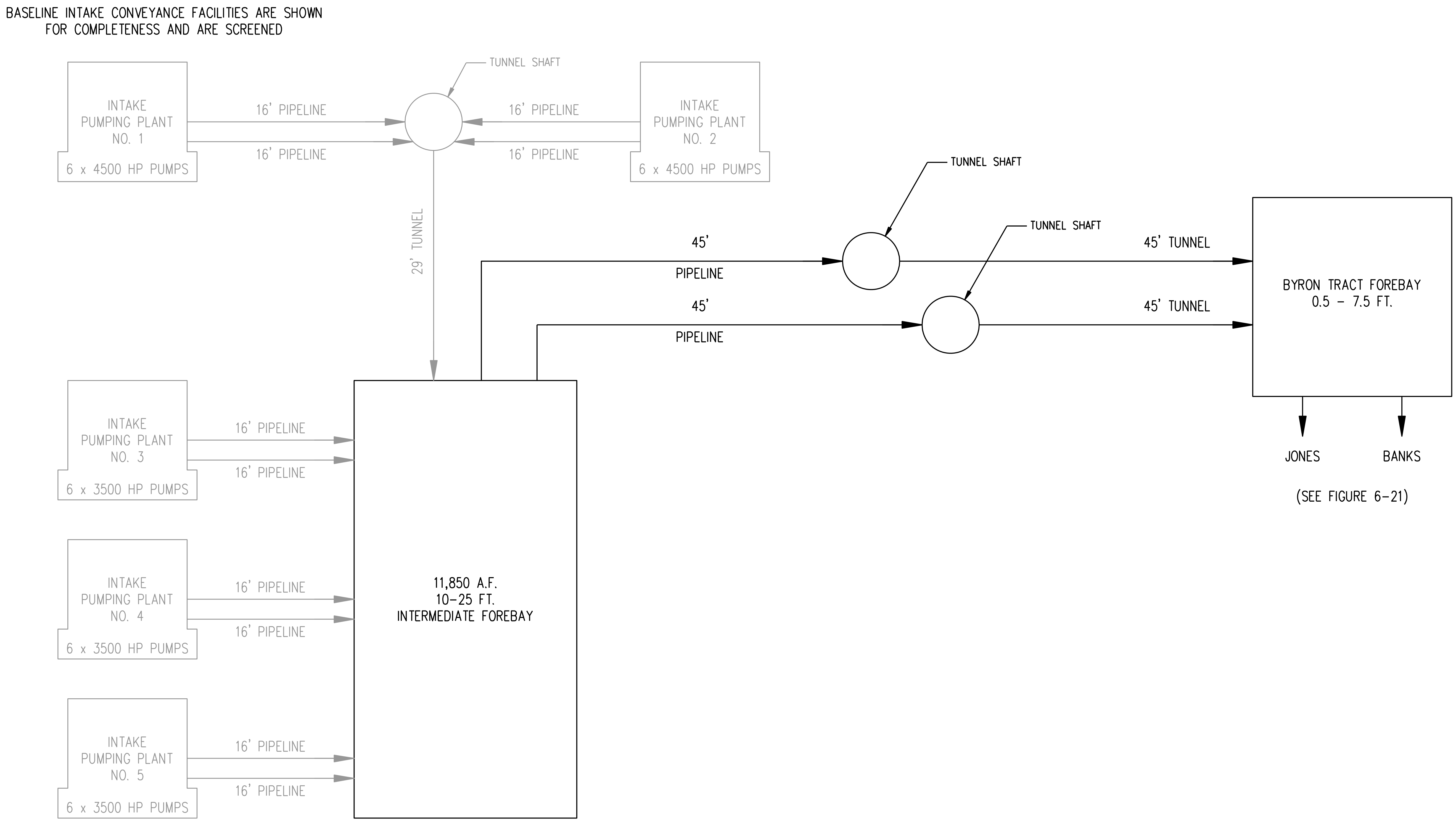
ANALYSIS & OPTIMIZATION REPORT
Pipeline/Tunnel Option

Tunnel Conveyance System - Baseline

FIGURE
6-12



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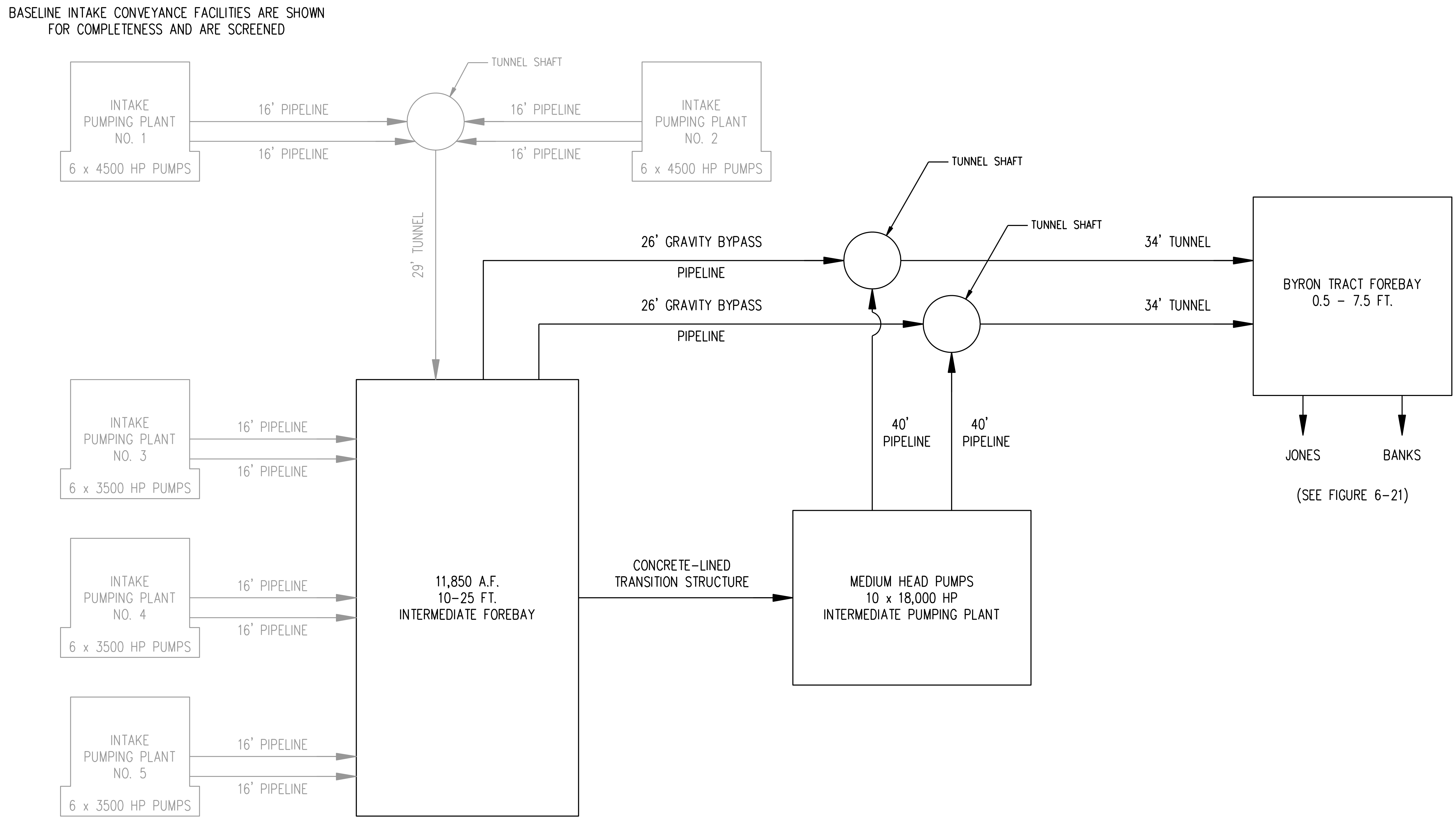
Pipeline/Tunnel Option

Tunnel Conveyance System - Alternative 1
Eliminate Intermediate Pumping Plant

FIGURE
6-13



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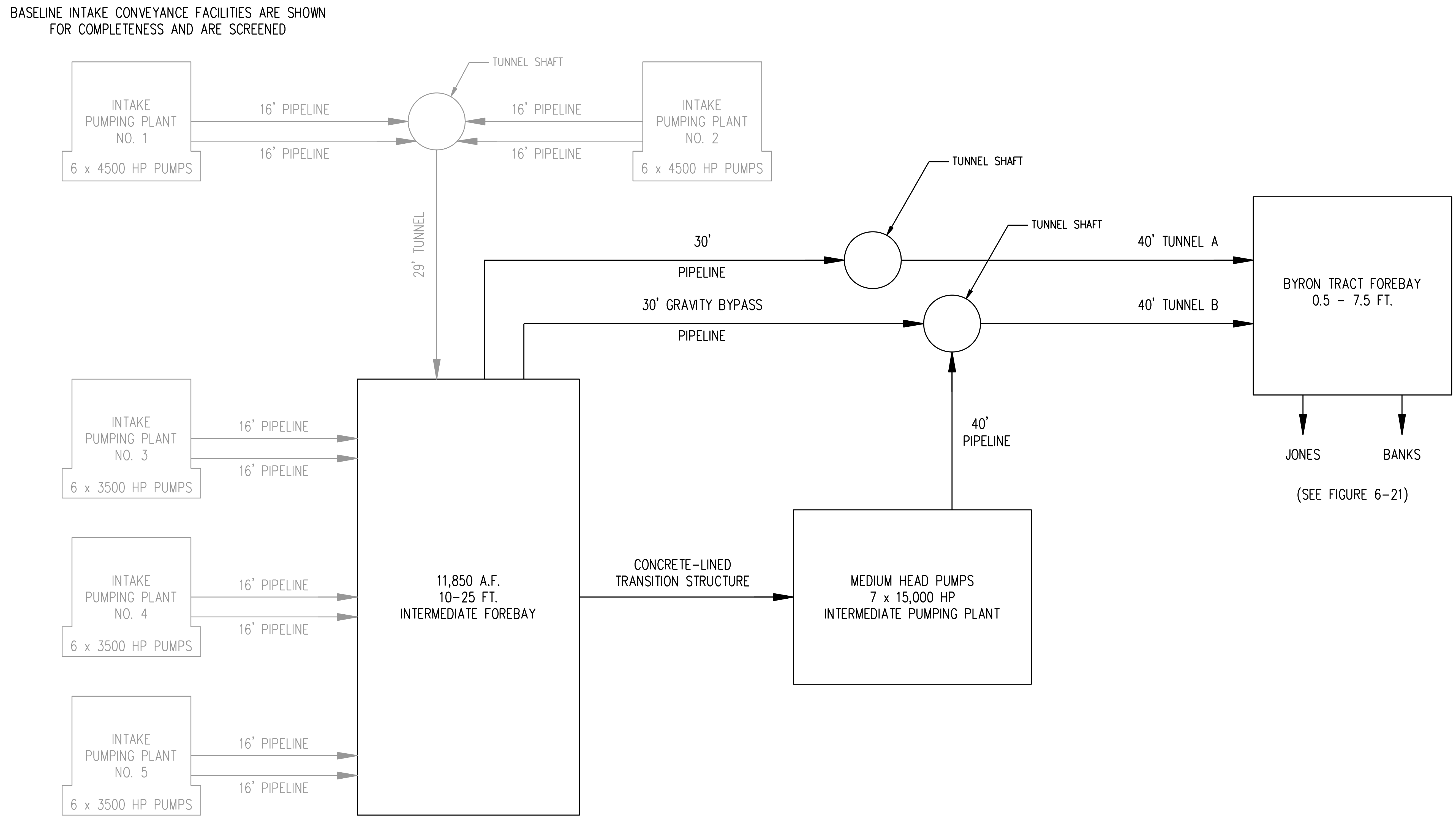
ANALYSIS & OPTIMIZATION REPORT
Pipeline/Tunnel Option

Tunnel Conveyance System - Alternative 2.1
Single Pump Size

FIGURE
6-14



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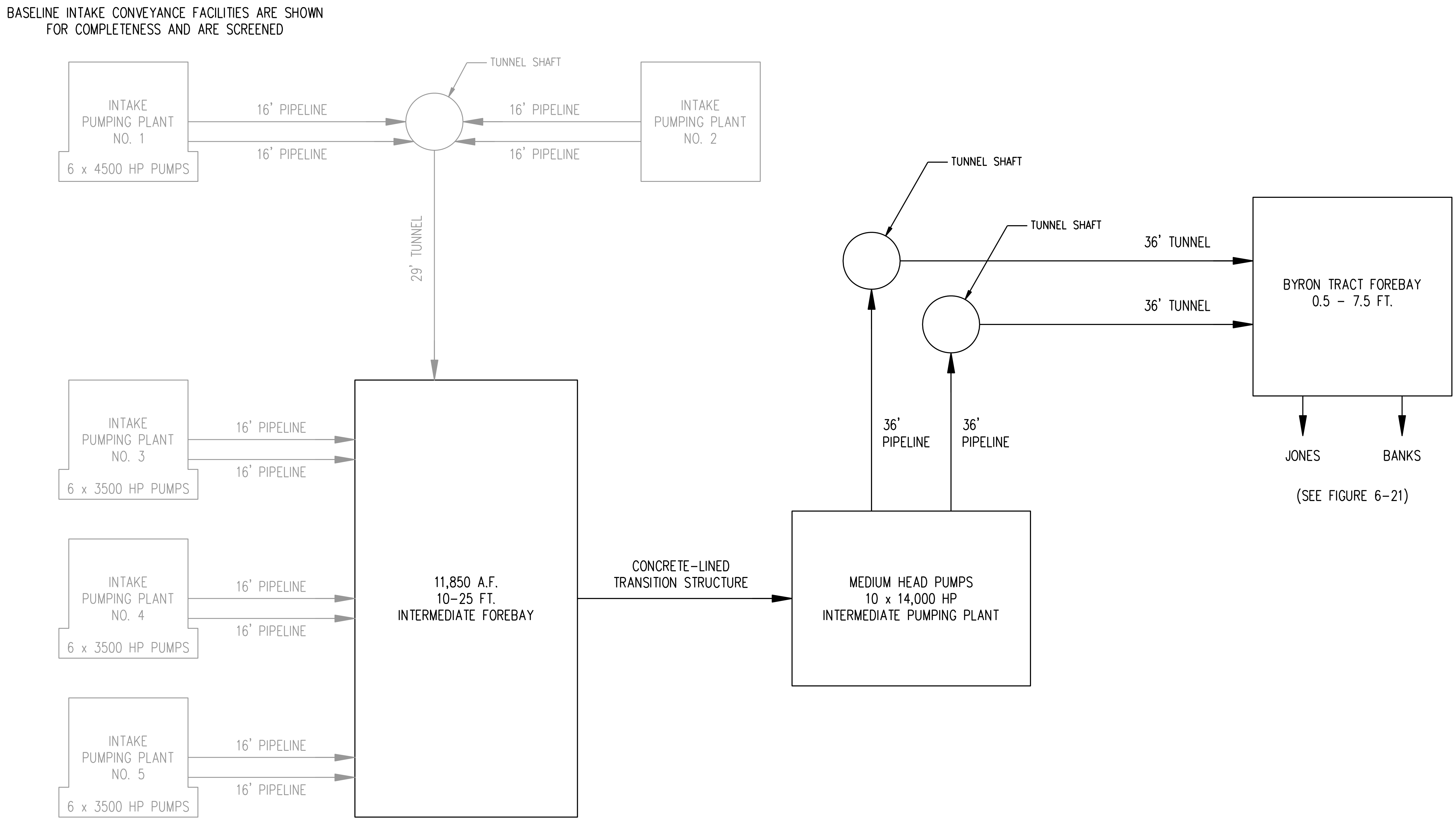
ANALYSIS & OPTIMIZATION REPORT
Pipeline/Tunnel Option

Tunnel Conveyance System
Alternative 2.A1 - Single Pump Size
(Tunnel A = Gravity, Tunnel B = Pumped)

FIGURE
6-15



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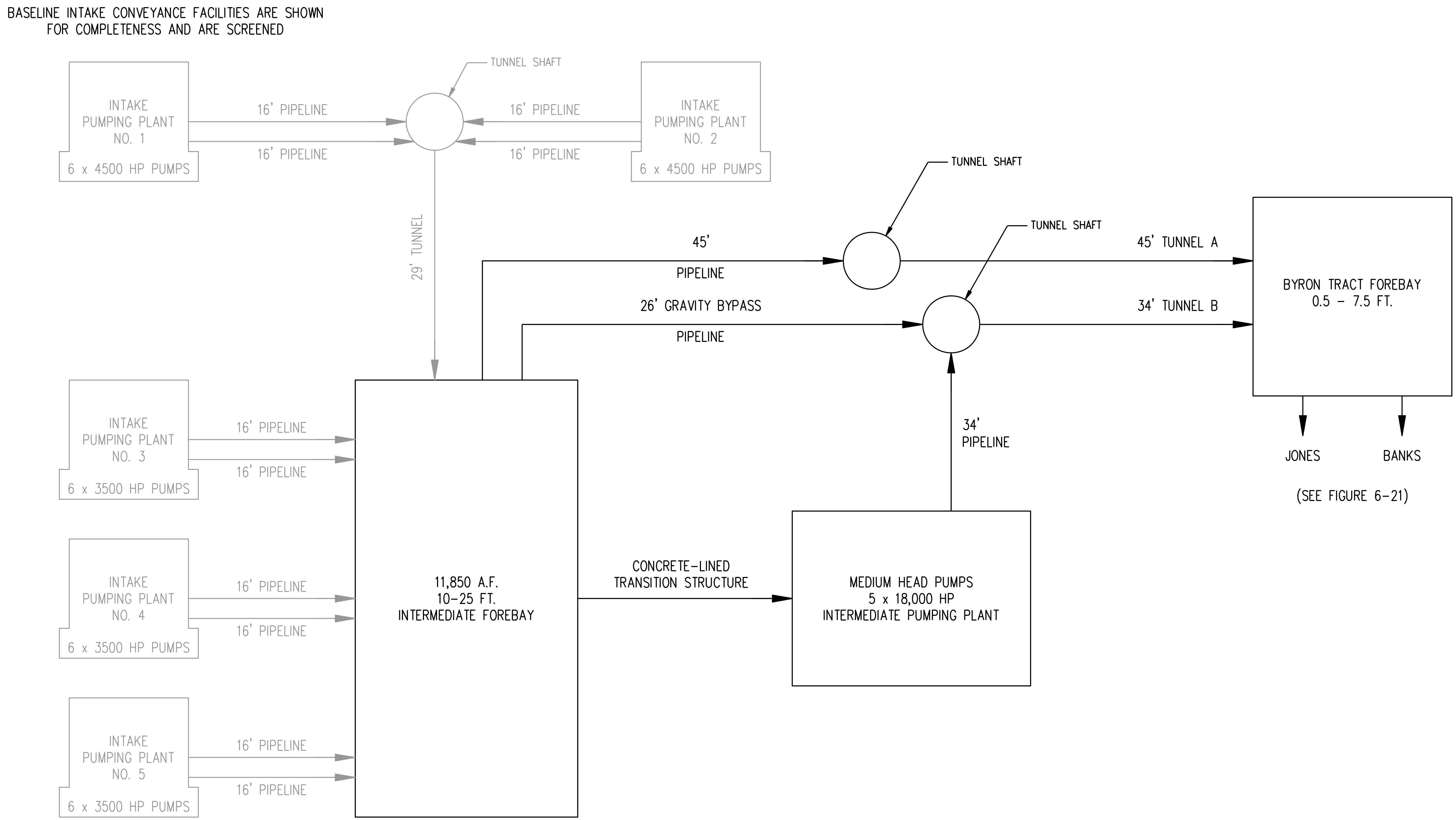
ANALYSIS & OPTIMIZATION REPORT
Pipeline/Tunnel Option

Tunnel Conveyance System - Alternative 2.B1
Single Pump Size (No Gravity Bypass)

FIGURE
6-16



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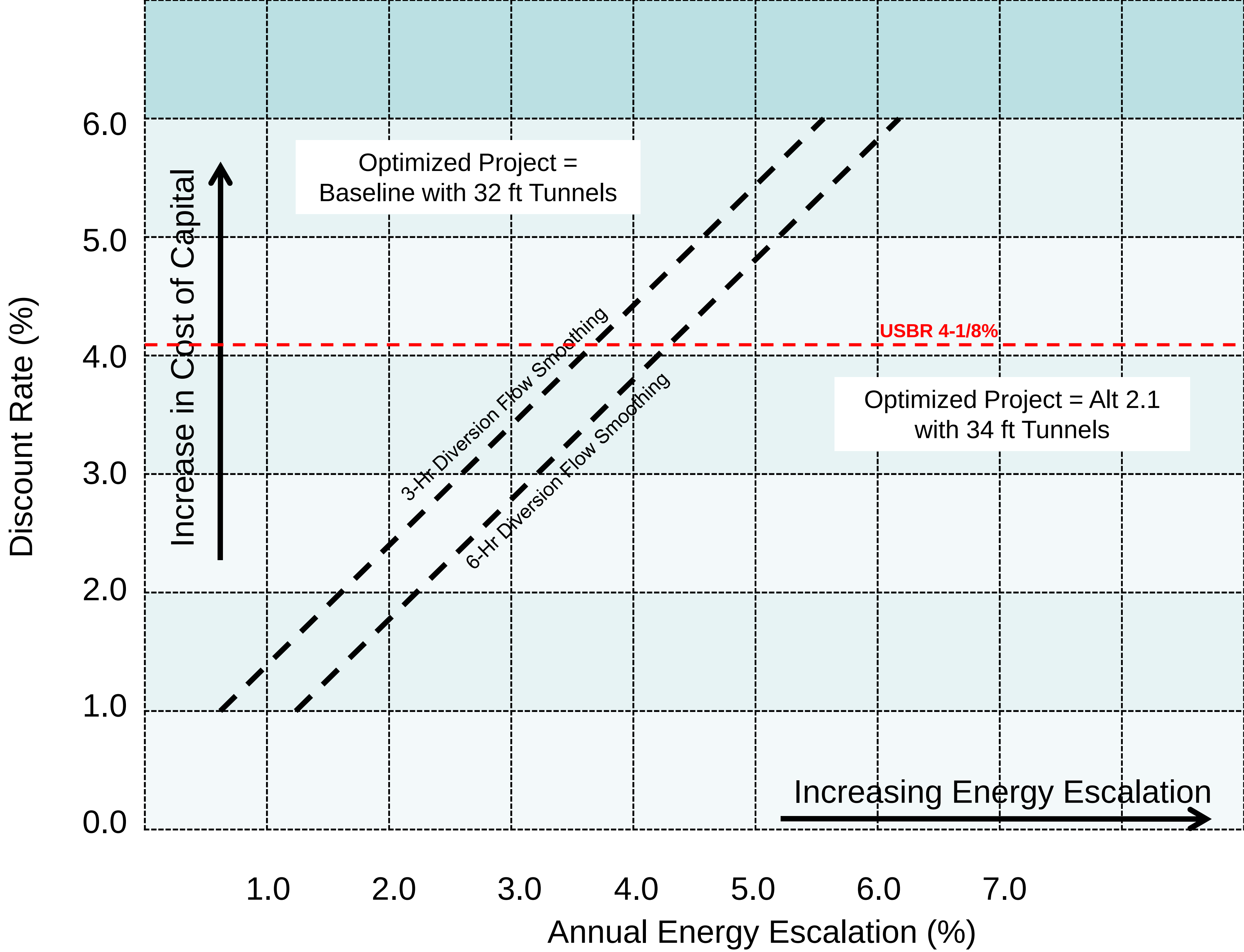
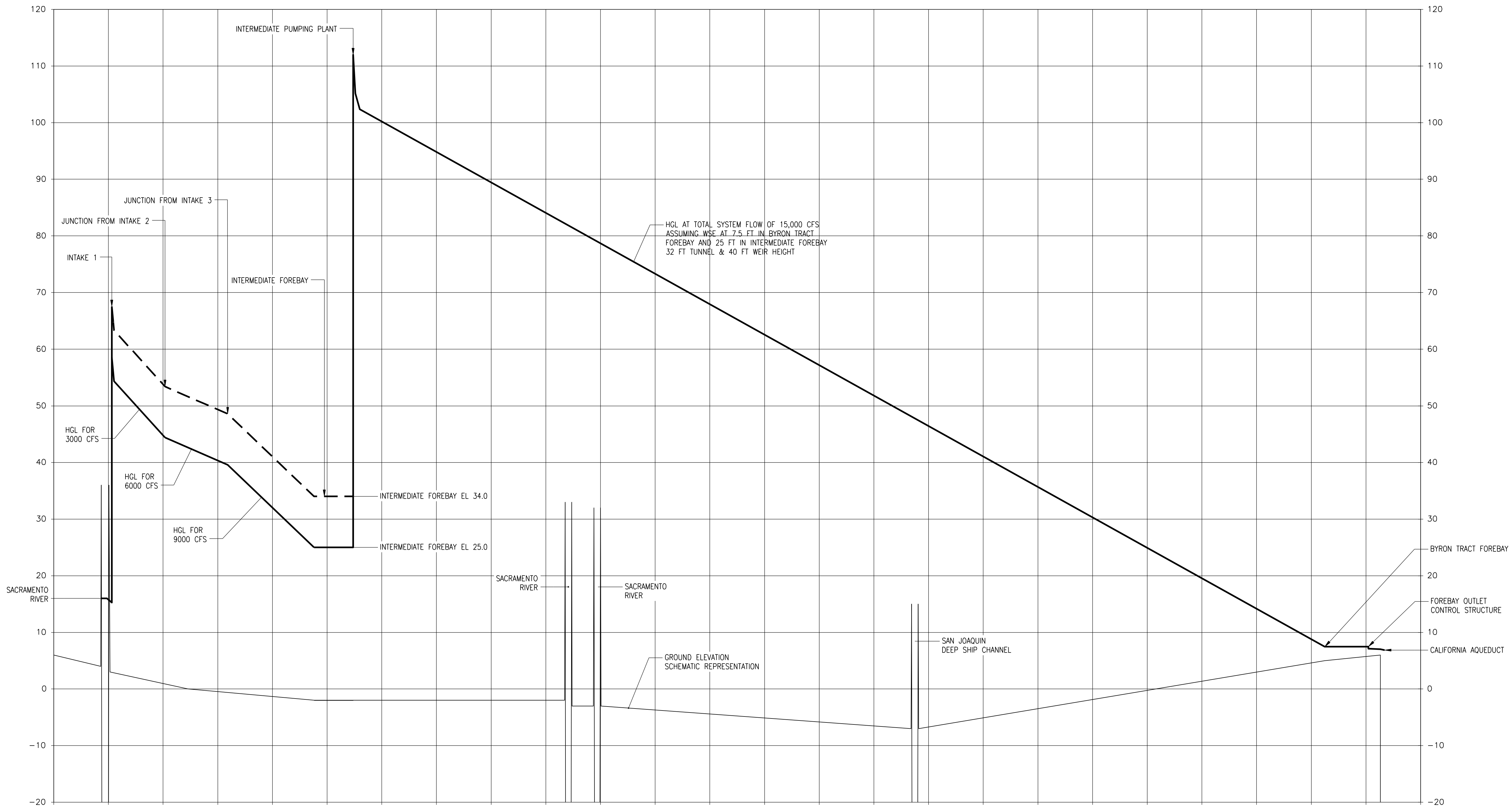
ANALYSIS & OPTIMIZATION REPORT
Pipeline/Tunnel Option

Tunnel Conveyance System
Alternative 3 - Single Pump Size
(Tunnel A = Gravity, Tunnel B = Pumped)

FIGURE
6-17



California Department Of Water Resources Advancing the Bay Delta Conservation Plan Delta Habitat Conservation & Conveyance Program DHCCP Team	ANALYSIS & OPTIMIZATION REPORT **PIPELINE/TUNNEL OPTION**	**FIGURE**
	Tunnel Conveyance System **Optimized Project 15,000 cfs**	**6-18**



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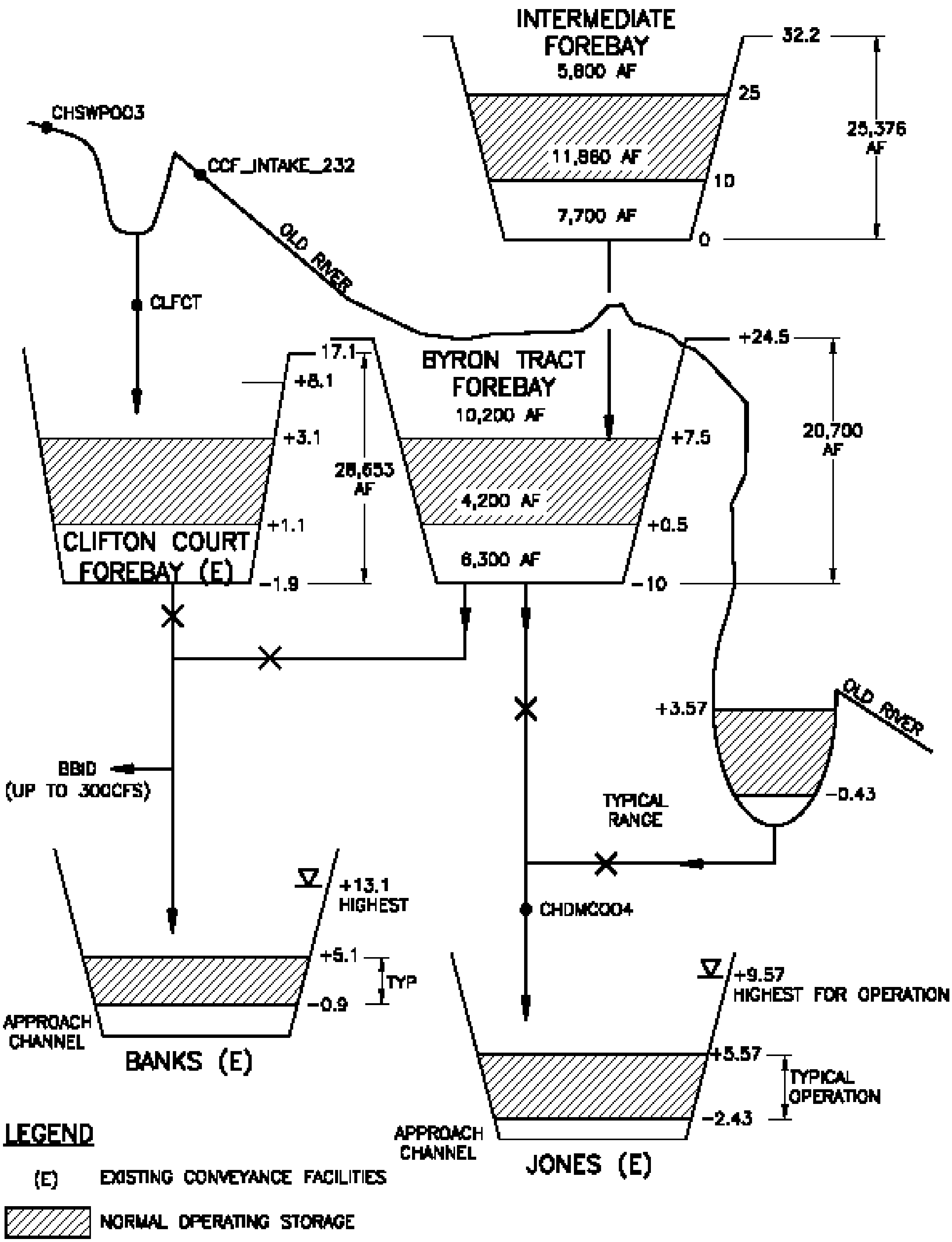
ANALYSIS & OPTIMIZATION REPORT
PIPELINE / TUNNEL OPTION

Hydraulics
Hydraulic Profile of Optimized Alternative

FIGURE
6-20



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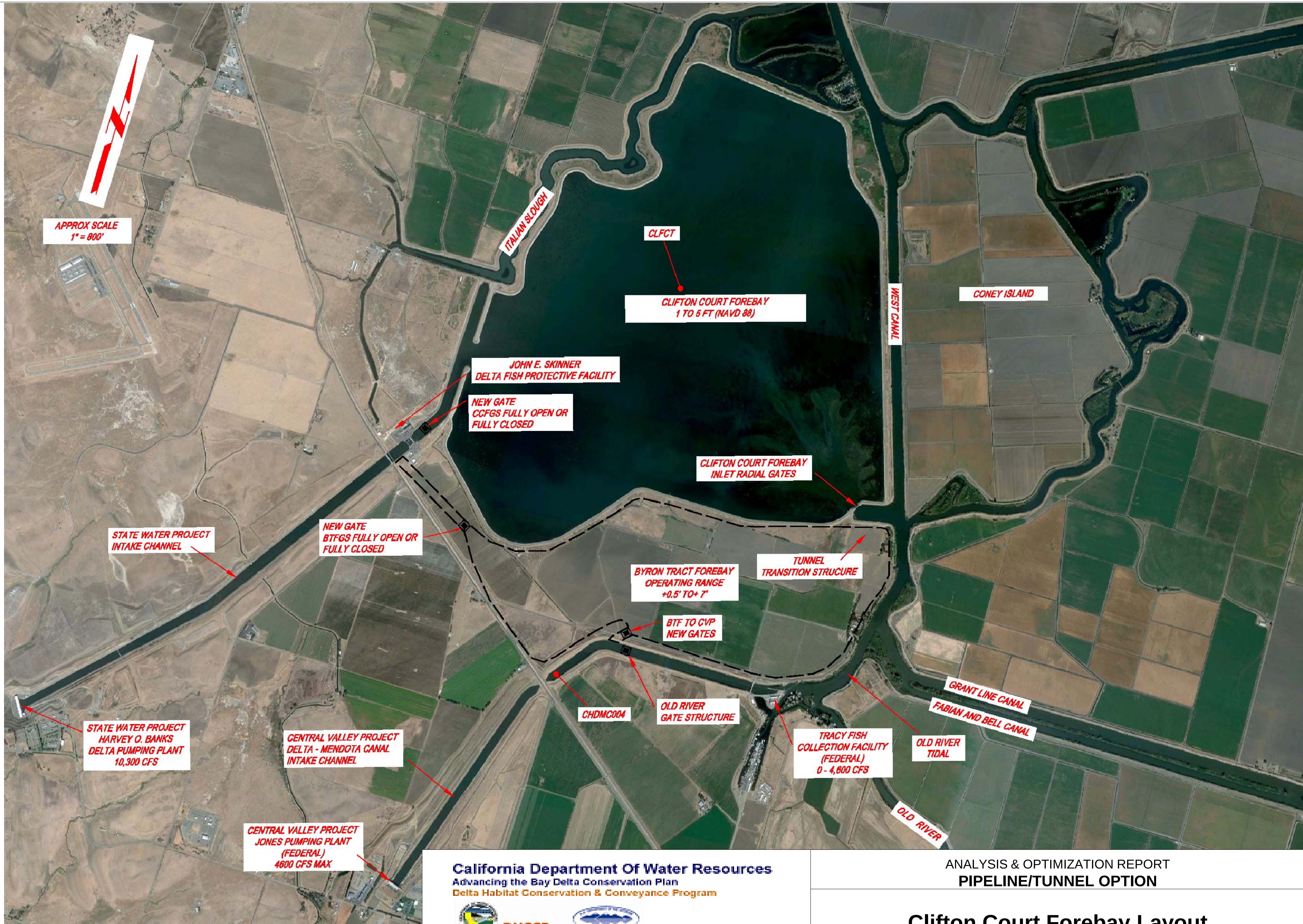
ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Dual Operations
Schematic Layout

FIGURE

6-21



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ANALYSIS & OPTIMIZATION REPORT
PIPELINE/TUNNEL OPTION

Clifton Court Forebay Layout

FIGURE
6- 22

7.0 GAS WELLS

There are numerous gas wells and gas fields that are located near or within the proposed right-of-way (ROW) as shown in the CER. (See Table 7-1) The data used to create Table 7-1 were obtained from the Department of Conservation's website (Division of Oil, Gas & Geothermal Resources, DOGGR) at: http://owr.conservation.ca.gov/WellSearch/WellSearch.aspx, in conjunction with DHCCP's GIS file, which contains the relative location (latitude and longitude coordinates) of the gas wells, gas fields, and proposed ROW (See Appendix S).

There are eighteen gas wells located within, or directly adjacent to the proposed ROW. Of the eighteen wells, ten are located within the proposed ROW and eight are located adjacent to the boundaries of the ROW. Five of the eighteen wells are currently active and producing gas, one is currently idle, and the rest have been plugged and abandoned. The wells were abandoned as early as 1957 and as recently as 2004.

Proposed Tunnel Alignment Revision

The Outside Reviewers recommended the tunnel alignment avoid any active or idle gas wells and minimize intersection with plugged wells due to the potential for damage to the wells by the tunnel boring machines during mining operations. Accordingly, working within the previously established Conveyance Planning Area boundary, re-alignment at the Walnut Grove (Andrus Island) and the Tyler Island shafts is proposed as shown on Figure 7-1 and in greater detail in Appendix S.

Recommendations

In summary:

- Participate in the DOGGR Well Review Program;
- Obtain permits for any well work (active or abandoned);
- Given that well coordinates on DOGGR website are not necessarily accurate, conduct a survey to determine their exact location;
- Avoid all wells to the extent practical; avoid tunneling over wells;
- Given that DOGGR makes no guarantee that wells are properly abandoned or will not leak after abandonment, address each proximate well specifically;
- DWR has neither designed nor constructed a project that passes through a gas field or near existing gas wells, either active or abandoned. Accordingly, and as recommended by the Outside Reviewers, engage the services of a petroleum engineering consultant with experience in the installation and abandonment of gas wells (ideally one familiar with the Delta and its gas wells and fields) to advise the DWR and the DHCCP.

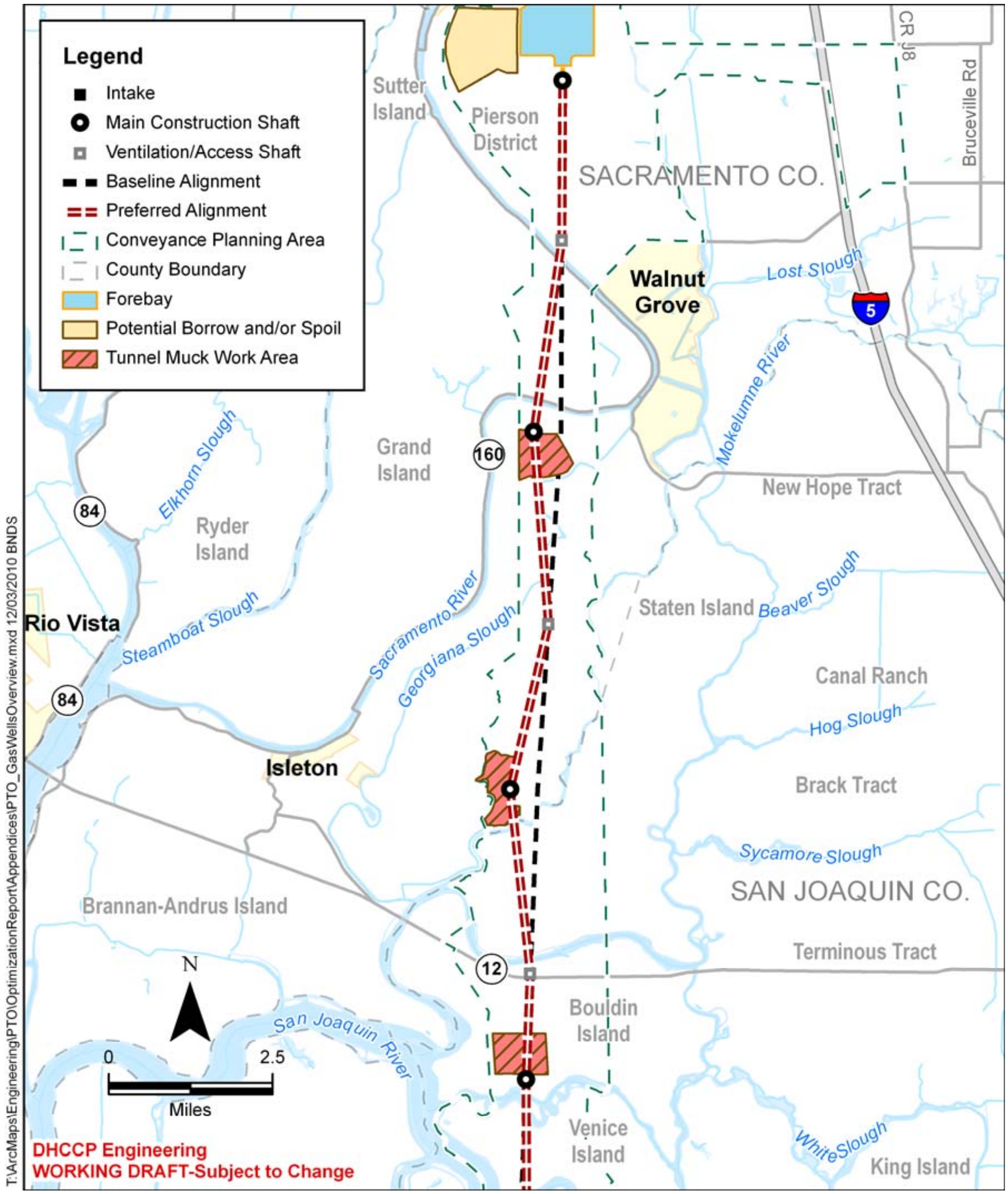

Figure 7-1 Proposed Realignment to Avoid Active/Idle Gas Wells

Table 7-1 Gas Wells in Pipeline / Tunnel Right-of-Way

Affected Area / Structure	Wells (qty)	API #	Well Status	Total Depth	Field Name	Operator	Operator Status	Lease	Spud Date	Abandonment Date	County	Comments
Intake 1 Pipeline	1	06700001	Plugged	7,938'	N/A	UMC Petroleum Corp.	InActive	Scribner	12/16/66	12/26/66	Sacramento	Well is within proposed ROW. The status of the well is 'Uncompleted Abandoned.' Well was plugged in 1966. The cement plug was placed from Elev. 568' to Elev. 418'. Cut off 9 5/8" casing, installed 10 foot cement plug at surface and welded steel plate over stub.
Tunnel	1	06720082	Plugged	8,050'	N/A	Montara Petroleum Co.	N/A	Wurster	6/19/75	6/30/75	Sacramento	Well is just outside the proposed ROW. Well was plugged in 1975. The status report of the well suggests that the well is dry and abandoned. Cut casing 5' below ground, plugged casing with 25 lineal feet of cement, welded steel plate on stub and abandoned well.
Tunnel	3	06720029	Plugged	12,648 '	N/A	Union Oil Co. of Calif.	InActive	Wurster Unit	8/17/69	10/12/69	Sacramento	Well is within proposed ROW. Well was plugged in 1969. The status report of the well suggests that the well is dry and abandoned. The cement plug was placed from Elev. 10,301' to Elev. 10,111', Elev. 1,969' to Elev. 1,609' and Elev. 25' to surface. Welded on steel plate capped and abandoned,
Intake 5					Merritt Island Gas							Gas Field
Int. Forebay	1	06720324	Plugged	8,250'	N/A	Vern Jones Oil & Gas Corp.	Active	Jonson	11/6/97	11/13/97	Sacramento	Well is within proposed ROW. Well was plugged in 1997. The status report of the well suggests that the well is dry and abandoned. The cement plug was placed from Elev. 1,516' to Elev. 1,223' and Elev. 871' to Elev. 671'. Cut casing 5' below ground, plugged casing with 45 lineal feet of cement, welded steel plate on stub and abandoned well.
Int. Forebay	2	06720325	Plugged	8,615'	N/A	Vern Jones Oil & Gas	Active	Jonson	11/17/97	11/29/97	Sacramento	Well is within proposed ROW. Well was plugged in 1997. The status report of the well suggests that the well is dry and abandoned. The cement plug was placed from Elev. 1,680' to Elev. 1,288' and Elev. 871' to Elev. 720' .Cut casing 5' below ground, plugged casing with 25 lineal feet of cement, welded steel plate on stub and abandoned well.
Int. Forebay	1	06720089	Plugged	6,710'	N/A	Santa Fe Energy Resources, Inc.	InActive	Hamatani	4/28/75	5/8/75	Sacramento	Well is within proposed ROW. Well was plugged in 1975. The status report of the well suggests that the well is dry and abandoned. The cement plug was placed from Elev. 1,275' to Elev. 1,109' and Elev. 752' to Elev. 552' .A cement plug was also placed at the surface.
Tunnel	1	06700358	Plugged	9,117'	N/A	ARCO Western Energy	InActive	KCYSunray Union Vorden Farms	11/26/60	12/11/60	Sacramento	Well is just outside the proposed ROW. Well was plugged in 1960. The cement plug was placed from Elev. 1,505' to Elev. 1,394'. A 10' cement plug was also placed near the surface and a plate was welded over it.
Tunnel					Snodgrass Slough Gas							Gas Field

Affected Area / Structure	Wells (qty)	API #	Well Status	Total Depth	Field Name	Operator	Operator Status	Lease	Spud Date	Abandonment Date	County	Comments
Shaft	3-2	06720438	**Active**	6,048'	Thornton, W.-Walnut Grove Gas	Stream Energy Inc.	Active	Wilson Lands	4/29/06	Active	Sacramento	Well is just outside the proposed ROW. Well is currently active and producing gas.
Tunnel	1	06700214	Plugged	4,295'	River Island Gas	Chevron U.S.A. Inc.	Active	Georgiana Unit Two	7/26/57	8/3/57	Sacramento	Well is within proposed ROW. The status of the well is 'Uncompleted/Abandoned.' Well was plugged in 1957. Placed a 10' lineal cement plug in the 9 5/8" casing at the surface and welded a steel plate on the casing.
Tunnel	1	06720432	**Active**	3,562'	River Island Gas	Royale Energy Inc.	Active	Elliott	2/24/06	Active	Sacramento	Well is just outside the proposed ROW. Well is currently active and producing gas.
Tunnel	1	06720420	**Active**	4,517'	River Island Gas	Royale Energy Inc.	Active	Andrus Island West	9/4/05	Active	Sacramento	Well is just outside the proposed ROW. Well is currently active and producing gas.
Tunnel	1	06720335	Plugged	5,400'	River Island Gas	Capitol Oil Corp.	Active	Pollard	7/17/98	8/27/04	Sacramento	Well is just outside the proposed ROW. Well was plugged in 2004. The status report of the well suggests that the well is abandoned. The cement plug was placed from Elev. 4,418' to Elev. 3,906' , Elev. 3,531' to Elev. 3,094' and Elev. 1,668' to Elev. 1,383' .Cut off casing 5' below grade, placed 25 linear feet cement plug in casing and annulus. RDMO.
Tunnel	3-10	06720412	**Active**	5,530'	River Island Gas	Towne Exploration Company	Active	Mello	11/3/04	Active	Sacramento	Well is within proposed ROW. Well is currently active and producing gas.
Tunnel	1-15	06720321	**Idle**	8,955'	River Island Gas	Jim Graham	Active	Furth	3/26/97	Idle	Sacramento	Well is within proposed ROW. Well is currently idle.
Tunnel	6	06720003	Plugged	9,709'	River Island Gas	Chevron U.S.A. Inc.	Active	Brandt	4/15/67	4/1/68	Sacramento	Well is just outside the proposed ROW. Well was plugged in 1968. Cement plugs were placed at 4,496' to 4,700', 850' to1,177', 1,177 to 1,741', 8,795' to 9,141' and 9,148' to 9,418.' Cut off 9 to 5/8" casing 5' below ground level, plugged top 10' of casing with neat cement and abandoned well.
Tunnel	5	06720495	**Active**	10,227'	River Island Gas	Towne Exploration Company	Active	Jensen	12/13/07	Active	Sacramento	Well is within proposed ROW. Well is currently active and producing gas.
Tunnel	1	06700184	Plugged	3,912'	River Island Gas	Len Owens, Operator	N/A	R.D. Graham	7/15/59	12/27/63	Sacramento	Well is just outside the proposed ROW. Well was plugged in 1963. The status of the well is 'Completed Abandoned.' Cement plug was placed from 21' to 4.' Welded cap on 9-5/8" and 5-1/2" casings and abandoned.
Forebay	1-30	01320129	Plugged	11,622'	N/A	UMC Petroleum Corp.	InActive	Danielson Moore	4/6/78	5/17/78	Contra Costa	Well is within proposed ROW. Well was plugged in 1978. The status report of the well suggests that the well is dry and abandoned. The cement plug was placed from Elev. 1,621' to Elev. 1,421', Elev. 350' to Elev. 250' and Elev.5' to Elev. 30'. Cut off casing 5' below ground level, welded on plate and abandoned well.

8.0 BARGE FACILITIES

The PTO alignment under the Delta affords limited opportunities for accessible shaft or portals from which to access the subsurface and conduct the tunnel mining work. The area is rural and few roads are in place. For the shaft locations in the CER, land areas nearby have been proposed for barge facilities to handle equipment and material deliveries.

The Outside Review Panel supported the concept of water transport of equipment and materials to the shaft sites. The Panel also encouraged the relocation of shafts where possible to areas supported by existing highways.

Barge facilities are recommended given the proximity of the various construction staging areas to rivers and canals. None of the tunnel conveyance shafts have barge or dock facilities nearby at this time. Several factors must be considered in the planning for this critical component of the construction effort. Some of those factors are addressed in Appendix T.

9.0 LONG TERM MAINTENANCE

9.1 Inspection

Future maintenance and inspection needs for the Program were examined by the Division of Engineering of DWR to identify features to be considered in preliminary engineering of the tunnel project. These features would facilitate maintenance and inspection activities, and dewatering activities as necessary to adequately maintain the tunnel. Use of remotely operated vehicles (ROVs) is proposed to minimize the need to dewater the tunnels to inspect them. DOE's consideration of inspection issues are contained in Appendix U.

9.2 Dewatering

The DOE also studied dewatering the tunnels to determine possible schemes to be considered in the planning and engineering of the tunnel project. Dewatering may be necessary during the life of the tunnels.

The dewatering was assumed to be conducted in one bore while the other remains in operation.

As proposed in the CER, maintaining one bore in operation will permit up to 7,500 cfs conveyance capacity.

Details of DOE's study are contained in Appendix V.

10.0 SUMMARY OF FINDINGS

10.1 Design Criteria

- Friction Factor – reduction of friction by installation of additional liner, coating or other means is not supported by corresponding savings in pumping costs; continued use of Manning's n = 0.0145 in hydraulic calculations is reasonable.

- Lining – a precast concrete segmental lining installed as part of tunnel boring operations is the favored lining system. The structural performance of the precast segmental lining under internal pressure would be enhanced by the use of a stress transfer system knitting together the precast segments. A second pass system using a steel liner installed in the areas of higher internal pressures should be maintained as an option until development of the design and testing during preliminary engineering prove the feasibility of the favored lining system.

- Geotechnical Data – much more data is needed for preliminary engineering; however, the few data points are reasonably consistent and indicate tunneling as conceived in the CER is viable.

- Seismic Displacement - the results of limited analysis indicate that the tunnel, given the geometry assumed for the opening and the liner, will remain structurally stable when subjected to the ovaling deformations caused by the design earthquake.

- Shaft Loading – some initial estimates of loading based on the very limited geotechnical information are provided in Figures 4-13 through 4-16.

10.2 Implementation

- Profile - essentially straight-line, slightly sloped to either the Intermediate Pumping Plant or to the Byron Tract Forebay are the preferred profiles with an alternate having a single highpoint at Bacon Island shaft and slightly sloping both ways (to IPP and BTF). (See Figures 5-2 a through c).

- Construction Phasing (Construction Program Packaging):
    - Focus on one bore of the tunnel first and aggressively pursue bidding the second bore as market conditions allow.
    - Break up each bore into six contracts.

- Consider removing pre-cast segments from tunneling contracts.
- Separate the award of contracts by three months.

- Bonds and Insurance:
  - Begin working with executive management to reduce the penal sum of the bonds to something like 10% or 20% of the contract value. This will take legislative action as reportedly done for the Bay Bridge (American Bridge contract.)
  - Begin working on an Owner-Controlled Insurance Program (OCIP). On a program of this size, an OCIP can save significant money.

- Shafts –to optimize construction and shaft arrangement:
  - Construct combined launch and retrieval shafts and leave TBM skin shield in place outside the retrieval shaft if conditions require.
  - Keep shaft construction in the tunneling contracts.
  - Use secant pile or slurry wall construction with jet grouted or tremie plug.
  - Leave final decision up to the contractor; provide a baseline scheme that works and allow the contractor to redesign to meet his needs.

10.3 Tunnel Conveyance System Optimization

- Present Worth of Energy and O&M – energy and O&M costs are secondary relative to construction costs using current assumptions; minimizing capital cost takes precedence.
- Intake Conveyance – connect Intakes 1, 2 and 3 to a single 32 ft ID conveyance tunnel.
- Tunnel Conveyance Range of Configurations – twin bore tunnel diameters and pumping plants ranging from 32 ft ID with high and low head pumps (CER configuration) to 34 ft inside diameter with single-size medium head pumps are recommended.
- Gravity By-Pass at IPP – does not pay for itself strictly in power (energy) and O&M cost savings alone using current assumptions; other benefits

(e.g. operational flexibility when Intermediate Pumping Plant is out of service) may support such a facility.

- Forebays – It appears feasible to add operating storage to the Intermediate Forebay to reduce peak pumping flowrates when possible. This will require armoring of waterside embankment slopes and slight increase in embankment height. Such optimization requires a Program-wide facility operations plan before proceeding further. An operations plan that limits export pumping of the SWP Banks Pumping Plant to 12-hours per day or less requires an Intermediate Forebay size that would exceed the conveyance planning area boundaries.

10.4 Gas Wells

- Avoid wells (slight re-alignment proposed); conduct comprehensive survey and inspection of final alignment; engage the services of a petroleum engineering consultant with experience in the installation and abandonment of gas wells (ideally one familiar with the Delta and its gas wells and fields) to advise the Department and the Program.

10.5 Barge Facilities

- Use of barge facilities is recommended.

Refer to Appendix D for comments on this by the Outside Review Panel

11.0 AREAS FOR FURTHER STUDY

The analysis and optimization activities of this task order have led to a number of specific areas requiring further study in subsequent engineering efforts, preferably before entering into preliminary engineering. These areas are offered here according to the team making the recommendation.

11.1 Tunnel Design Criteria

The analyses and evaluations performed used assumptions regarding materials, ground behavior, construction methods, and other issues, and only provide a general idea of dimensions required for the project. As the tunnel conveyance option moves toward preliminary engineering, the following issues will need to be studied in more detail:

- A more detailed study of the final alignment.
- Develop and adopt leakage criteria.
- Review tunnel loading cases.
- Refine the segment thickness and reinforcement.
- Determine preferred method for handling internal pressure on segments, and devise a comprehensive testing program for the selected method.
- Study gasket capabilities and devise a testing program for the gaskets.
- Perform a more detailed seismic evaluation, considering other locations along the tunnel alignment.
- Refine liquefaction analyses once more geotechnical data becomes available.
- Refine shaft loading once shaft location specific geotechnical data becomes available.

11.2 Tunnel Implementation

- Prepare a risk register; develop risk mitigation plan;
- Contracting – develop:
  - Outreach Plan
  - Method for prequalification of contractors
- Bonding
  - Address the areas where legislative action is required for 50% or 25% bonds; draft language and start pursuing.

11.3 Tunnel Conveyance System Optimization

- Prepare a Program-wide facility operations plan that integrates the proposed diversions with sensible pumping regimes. Such a plan is essential for further engineering of the proposed conveyance option.
- Confirm the pump selections for the proposed optimized facility sizing.
- Refine preliminary surge analysis of the system using conventional pipe flow modeling software.
- Agree on the cost of capital to be applied to the evaluations by exploring financing options and risks.
- Revise the intermediate forebay embankment geometry as proposed and confirm slope stability and seepage safety factors.
- Reconsider a gravity system solution if tunnel liner costs associated with a pumped system increase.

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