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1. Purpose

The purpose of this technical memorandum is to outline the criteria, guidelines, and requirements for hydraulic computational approaches used for steady-state and transient surge hydraulic analyses for the Delta Conveyance System Project (project). These analyses extend from the intakes located on the Sacramento River to the project discharge point at the California Aqueduct, just upstream of the existing State Water Project (SWP) Harvey O. Banks Pumping Plant (Banks). The major hydraulic elements include the following:

- River intakes
- Conduits (open channel and pressurized)
- Drop shafts
- Tunnels
- Reservoirs
- Pumping Plant
- Various control structures

1.1 Background

The project configuration is a single main tunnel system. The project includes the following infrastructure:

- Up to three Sacramento River intake facilities
- A main tunnel connecting the river intakes to a new pumping plant
- A new Southern Forebay
- Tunnels and hydraulic structures connecting the Southern Forebay to the approach channel to Banks

An additional scenario would also be analyzed to deliver water to the existing Central Valley Project (CVP) C.W. “Bill” Jones Pumping Plant (Jones).

The DCO has established a range of 3,000 to 7,500 cubic feet per second (cfs) of diversion capacities. The DCA performed a systemwide hydraulic and capacity analysis for design flow capacities of 4,500, 6,000 and 7,500 cfs, which is described in the Capacity Analysis for Preliminary Tunnel Diameter Selection TM, (DCA, 2021a).

2. Design Codes, Standards, and References

The latest adopted version of the following codes, standards, and references apply to this TM unless otherwise noted:

- American National Standards Institute (ANSI) (2018): ANSI/HI Standard 9.8, Rotodynamic Pumps for Pump Intake Design
- ANSI (2017): ANSI/HI Standard 9.6.1, Rotodynamic Pumps Guideline for NPSH Margin
- Innovyze (2020): InfoWorks Integrated Catchment Modeling (ICM) version 11.0.2
- Idel'chik, I.E. (1960): Handbook of Hydraulic Resistance. Coefficients of Local Resistance and Friction
- Miller, D.S. (1990): Internal Flow Systems
- U.S. Department of the Interior Bureau of Reclamation (Reclamation) (1978): Design of Small Canal Structures

3. Hydraulic Criteria

This section describes the general modeling approach, as well as the governing equations used to evaluate the design of the facilities within the system.

3.1 Modeling Approach

The proposed project has many individual hydraulic elements with associated hydraulic losses that form the hydraulic and energy gradelines throughout the entire system. To replicate the interaction of these system components from the Sacramento River to the discharge point on the California Aqueduct, the modeling software InfoWorks ICM has been selected.

InfoWorks ICM uses a master database to store model and hydraulic data, with the tools necessary to create, edit, manage, and analyze the information to size elements and develop system configuration. The software facilitates execution of the following tasks:

- Manage and maintain a record of network models over time.
- Share model data among users, with audit trails and security mechanisms.
- Import model data from other systems.
- View a geographical representation of the network on screen, with the network displayed over the top of a detailed local map, two-dimensional (2D) and three-dimensional (3D), with comprehensive facilities to customize the network appearance.
- Analyze the results using a wide variety of graphical, textual, and statistical outputs.

InfoWorks ICM has a variety of hydraulic elements that can be used to model a system. The model uses the following major hydraulic elements and equations for the project:

- Gravity system closed conduits – Manning's equation
- Gravity system open conduits – Manning's equation
- Pressurized (pumping) system – Bernoulli's equation with Darcy friction factor "f" for head loss
- Submerged vertical and radial gates – Orifice equation
- Unsubmerged vertical and radial gates – Weir equation

- Minor losses – K $V^2/2(g)$, where:
  - K = hydraulic loss coefficient
  - V = velocity (foot per second [ft/s])
  - g = acceleration due to gravity (foot per square second [ft/sec^2])
- Intake screen sizing – VA=Q/As *FA, where:
  - V_A = approach velocity (ft/s)
  - Q = design flow in cubic feet per second (ft^3/sec) or cfs (1,500 cfs or 3,000 cfs)
  - A_s = the wetted area of screen required in square feet (ft^2)
  - FA = allowance factor

The use of these equations, definition of terms, and sizing criteria are outlined in further detail in the following sections.

3.2 Major System Facilities

Figure 1 shows the system's major hydraulic elements, listed herein, for the vertical flat pate intake screen option. These elements are discussed individually in the following sections:

- Intake facilities identified as Intakes C-E-2, C-E-3, and C-E-5
- Tunnels and shafts
- Pumping Plant
- Southern Forebay
- South Delta Conveyance Facilities (SDCF), comprising tunnels and hydraulic control facilities connecting the Southern Forebay to Banks and potentially to Jones

The general approach to the hydraulic analyses examined the following two subsystems:

- Hydraulic analysis between the Sacramento River and the Southern Forebay
- Hydraulic analysis between the Southern Forebay and the delivery points to Banks and Jones

The upstream system between the Sacramento River and the Southern Forebay is largely driven by the upstream boundary conditions: the water surface elevations (WSEs) in the river and the allowable hydraulic losses in the intakes, tunnels, and other appurtenances. The downstream system between the Southern Forebay and the Banks and Jones pumping plants is controlled by the lowest operational WSE feeding the pumps at the pumping plants. This downstream system would control the Southern Forebay WSE; which, in turn, establishes the downstream boundary conditions for the pumping plant or the gravity flow options into the Southern Forebay.

The remainder of this section discusses each major system facility and identifies the governing equations used for analysis, the governing criteria, and the methodology for sizing.

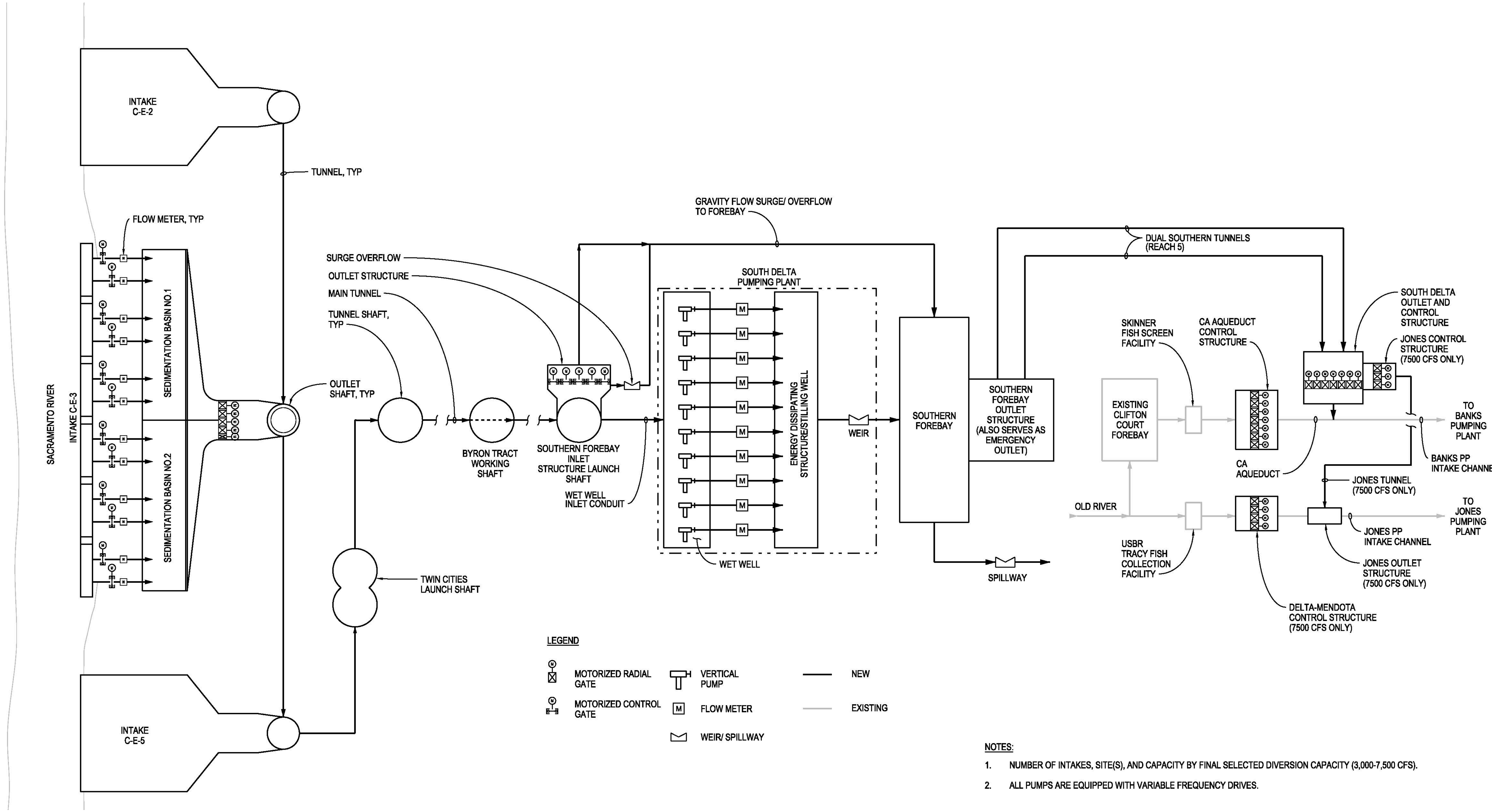

Figure 1. Overall Process Flow Diagram

3.3 Intake Facilities

The project would include up to three intakes (C-E-2, C-E-3, and C-E-5). Each intake facility's maximum river diversion would be limited to 3,000 cfs, as agreed upon with DCO and intake configuration criteria information. Each intake would include the following components:

- A fish screen structure
- Conduits or pipes to convey flow from the river intakes
- Sedimentation basins
- Gated structures connecting the sedimentation basins to the tunnel shaft

The conduits/pipes include isolation valves and flow meters. Downstream of the sediment basins would be a gated structure to control flow into the tunnel shaft and tunnel system. These radial gates could also serve as isolation for the tunnel system.

3.3.1 Governing Equations

The intakes, connecting conduits/pipes, sediment basin, and control structure would be modeled using InfoWorks ICM. The governing equations are provided here.

For screens sizing, the following equation would be used:

$$A_s = F_A \times (Q / V_A) \qquad \textbf{[1]}$$

Where:

V_A	=	Approach velocity (must not exceed 0.2 fps)
Q	=	Design flow (ft^3/sec or cfs [1,500 cfs or 3,000 cfs])
A_s (calculated)	=	The wetted area of screen required (ft^2)
F_A	=	Allowance factor (10 percent) to allow for variation in flow velocity (typically 5 to 10 percent)

InfoWorks ICM uses the Kirschmer formula to solve for flow and head loss through the screens. Head loss through the screens would be calculated using the loss formula:

$$H_L = k\, V^2/2g \qquad \textbf{[2]}$$

Where:

H_L	=	The head difference from upstream to downstream (feet)
k	=	Head loss coefficient
V	=	Velocity (fps)
g	=	Acceleration due to gravity (ft/sec^2)

$$k = C_k \cos\alpha \left(\frac{w}{S}\right)^{\frac{4}{3}} \qquad [3]$$

C_k = Kirschmer's coefficient representing bar shape
α = Screen angle to vertical
w = Bar width
S = Bar spacing

For the conduits, the project would use the Manning's equation provided here:

$$Q = (1.486/n) * AR^{2/3} * S^{1/2} \qquad [4]$$

Where:

n = Manning's roughness coefficient
A = Cross-sectional area of flow (ft^2)
R = Hydraulic radius (cross-sectional area divided by the wetted perimeter [feet])
S = Slope (head loss per unit length of tunnel [foot per foot{ft/ft}])

Roller gates would be used at the upstream end of the conveyance conduit that extends between the screen structure and the sedimentation basins. The gates would be modeled as an orifice. The orifice equation is applied when the gates are submerged. When the gates are unsubmerged, InfoWorks ICM applies the weir equation. For tee screens, butterfly valves are used to isolate the conduit from the tee screens. Head loss for the valves were calculated using the Bernoulli equation, as follows.

$$Q=C_d A(2*g*h)^{1/2} \qquad [5]$$

Where:

C_d = Coefficient of discharge = 0.6; per Reclamation's *Design of small Canal Structures*
A = Area of the orifice (ft^2)
g = Acceleration due to gravity (ft/sec^2)
h = Difference between upstream and downstream WSEs (ft)

Minor losses associated with entrances, exits, bends, expansions, and contractions would be based on the following standard equation normally associated with the Bernoulli equation:

$$h = K V^2/2g \qquad [6]$$

Where:

h = Head loss (feet)
K = Minor loss coefficient
V = Velocity (fps) (per the Cameron hydraulic handbook, velocity in the smaller pipe is used for contractions and expansions)
g = Acceleration due to gravity (32.2 ft/sec^2)

The size of the sedimentation basins would be calculated by applying Stokes Law (provided herein) and would be based on the length required to settle sand particles. Note, these calculations would be completed outside of the InfoWorks ICM model. The model would include final sizes for the basins model to provide hydraulic continuity and to calculate head loss to complete the systemwide hydraulic and energy gradelines.

Flow through the radial gates would be modeled with the following orifice equation.

$$Q=C_d * A * (2g*h)^{1/2} \qquad \textbf{[7]}$$

Where:

Cd = Coefficient of discharge =0.72; per Reclamation's *Design of small Canal Structures*

A = Area of the orifice (ft^2)

g = Acceleration due to gravity (ft/s^2)

h = Difference between upstream and downstream WSEs (feet)

The sediment drying lagoon high-density polyethylene (HDPE) piping would be modeled with Manning's equation. Note, this piping would be modeled separately, outside of the systemwide InfoWorks model, because it does not have a contribution to the overall systemwide hydraulic and energy gradelines.

$$Q = (1.486/n) * AR^{2/3} * S^{1/2} \qquad \textbf{[8]}$$

Where:

n = Manning's roughness coefficient

A = Cross-sectional area of flow (ft^2)

R = Hydraulic radius (pipe diameter/4)

S = Slope (head loss per unit length of tunnel [ft/ft])

Minor losses for the sediment during lagoon piping would be based on the following standard equation normally associated with the Bernoulli equation.

$$H = K\,V^2/2g \qquad \textbf{[9]}$$

Where:

h = Head loss (feet)

K = Minor loss coefficient

V = Velocity (fps)

g = Acceleration due to gravity (32.2 ft/sec^2)

3.3.2 Criteria

The following criteria would be used for modeling the intake screens:

- **River Elevations:** Table 1 provides the design Sacramento River elevations. All elevations are based on the North American Vertical Datum of 1988 (NAVD88).
- **Flow:** Design flow for each separate screening facility can range from 1,500 to 3,000 cfs.
- **Screen Approach Velocity:** Not to exceed 0.2 fps per the criteria generally applied during regulatory processes for fish screens in California waters with salmonids and Delta smelt. These criteria have been informed documents developed by National Marine Fisheries Service (NMFS) and California Department of Fish and Wildlife (CDFW) (NMFS 1997, NMFS 2018, CDFW 2010). The U.S. Fish and Wildlife Service (USFWS) generally concurs with these criteria for fish screens in areas with Delta smelt. No specific criteria have been developed for the Project intake sites along the Sacramento River.
- **Maximum Head loss Through Screens:** 1.0 foot.
- **Mannings n:** 0.013 for concrete box conduits, 0.014 for concrete lined channels, 0.009 for HDPE piping.
- **Coefficient of Discharge (Cd):** 0.6 for gates with vertical flat faces, 0.72 for radial gates (per Reclamation's *Design of Small Canal Structures*).
- **Minor Loss Coefficients from ICM Modeling:** The ICM model has standard rating curves for loss coefficients developed over time in a wide variety of gravity network systems, using pipes and manholes. These standard ratings would be used for initial modeling, and the results would be reviewed for reasonableness. User-defined curves can be used for elements that are not well-represented by the model.
- **Minor Loss Coefficients Sediment Drying Lagoon:** Entrance loss K = 0.5, exit loss K = 1.0.

Table 1. Water Surface Elevations for System Head Curves Development (feet)

Head Condition [a]	Sacramento River WSE at Intake C-E-2	Sacramento River WSE at Intake C-E-3	Sacramento River WSE at Intake C-E-5
High Pumping Head	3.8	3.72	3.61
Design Pumping Head	4.66	4.59	4.47
Normal Low Pumping Head	6.03	5.92	5.75
Extreme Low Pumping Head	28.2	27.3	26.3

[a] Pumping Head refers to the head conditions at the South Delta Pumping Plant

3.3.3 Methodology

The methodology for sizing the intake facilities can be summarized as follows:

- The sizing of the intake facility is determined by the number of fish screens required to deliver the maximum diversion rate, without exceeding the approach velocity in front of the screens. Both vertical flat plate screens and cylindrical tee screens have been examined.
- Each individual vertical flat plate screen bay or each individual cylindrical tee screen would have a downstream conduit connecting the screen facility to the sedimentation basis. The conduits for the vertical flat plate screens would be box sections. The conduits for the cylindrical tee screens would be

welded steel pipe. The conduits would be sized for a minimum velocity for scour, to keep sediment moving to the sedimentation basins. The lengths of the conduits would be determined by the space required to facilitate the construction of the temporarily relocated Highway 160.

- The hydraulic elements (intake screens, conduits, sedimentation basin, radial gate control structure) would be input into the InfoWorks ICM model to generate an envelope of system curves, as described in Sections 3.4 and 3.5.
- The sedimentation basins would be sized based on Stokes Law to determine the settling velocity of the smallest sand particle to be prevented from entering the downstream tunnels. Particles smaller than sand were deemed too small to settle out in the downstream conveyance. The steps are outlined here:
  1) Calculate the time taken for the sand particle to settle from the free surface elevation to the elevation below the outlet channel.
  2) Calculate the trajectory of the sand particle using the horizontal and vertical velocity components of a descending particle; the resulting distance provides the shortest sedimentation length required to retain the sediment.
  3) Provide additional basin depth to allow for the seasonal storage of sediment at the bottom of the basin.

3.4 Tunnels and Shafts

3.4.1 Governing Equations

The tunnels and shafts would be modeled using InfoWorks as pressurized circular conduits with large vertical shafts (large manhole structures). For the conduits, the program would use the following Manning's equation:

$$Q = (1.486/n) * AR^{2/3} * S^{1/2} \quad \textbf{[10]}$$

Where:

n = Manning's roughness coefficient
A = Cross-sectional area of flow (ft^2)
R = Hydraulic radius (cross-sectional area divided by the wetted perimeter [feet])
S = Slope (head loss per unit length of tunnel [ft/ft])

Minor losses associated with entrances, exits, bends, expansions, and contractions would be based on the following standard equation normally associated with the Bernoulli equation:

$$h = K\,V^2/2g \quad \textbf{[11]}$$

Where:

h = Head loss (feet)
K = Minor loss coefficient
V = Velocity (fps) (per the Cameron hydraulic handbook, velocity in the smaller pipe is used for contractions and expansions)
g = Acceleration due to gravity (32.2 ft/sec^2)

3.4.2 Criteria

The following criteria apply:

- **Manning's n Values:** A range was analyzed between 0.014 and 0.016. The range of n values were conservatively selected to size the facilities in support of the environmental evaluation and permitting. These values can be further refined in final design. From this range in roughness, maximum and minimum head losses can be calculated to size the tunnels and the associated inlet pumping conditions for the South Delta Pumping Plant.
- **Minor Loss Coefficients:** The ICM model has standard rating curves for loss coefficients that have been developed over time in a wide variety of gravity network systems, using pipes and manholes. These standard ratings would be used for initial modeling, and the results would be reviewed for reasonableness. User-defined curves can be used for elements that are not well-represented by the model.
- **Allowable Velocities:** The minimum allowable tunnel flow velocity is limited to 3.5 fps, to maintain a minimum scour velocity for cleaning sediment within the tunnel. The maximum tunnel flow velocity is limited to 8 fps. This velocity is generally an industry standard starting point for the analysis of large transmission systems, and provides a balance between pipe size, power cost, and reasonable transient (surge) pressure fluctuations. This velocity is also considered suitable for tunnels with segmental concrete lining systems.
- **River Elevations:** Table 1 provides the river elevations.
- **Project Flows:** Refer to Table 2 for the project flows analyzed.

Table 2. Proposed Project Capacity (cfs)

Intake	3,000	4,500	6,000	7,500
C-E-2	Not used	Not used	Not used	1,500
C-E-3	Not used	3,000	3,000	3,000
C-E-5	3,000	1,500	3,000	3,000

3.4.3 Methodology

The following methodology for analyzing and determining tunnel sizes would need to be coordinated with the work outlined in Section 3.5, which involves the analysis of the South Delta Pumping Plant. This analysis would use the results of the InfoWorks model extending from the Sacramento River to the pumping plant.

- The following steps would be used:
  - Develop a preliminary range of tunnel diameters based on the minimum and maximum flow criteria.
  - Conduct a system end-to-end hydraulic head loss analysis with varying tunnel diameters from the smallest to the largest, to develop an envelope of system curves for the facilities upstream of the pumping plant wet well.
  - Compare this envelope of upstream system curves with candidate pump selections to determine whether there are limitations or preferences on limiting upstream head loss (Section 3.5).

– Perform an economic cost analysis, considering both capital costs and pumping costs, to narrow in on the optimum size of the facilities. For example, a smaller tunnel saves capital costs but increases long-term pumping costs.

3.5 Pumping Plant

3.5.1 System Head Curves Development

The South Delta Pumping Plant would also be modeled as an element within the systemwide InfoWorks ICM model. The model provides the user with the ability to input pump curves, so real-time simulations can be performed under varying suction and discharge head conditions. The suction conditions are a result of the analysis described for the intakes and the tunnels and shafts (Section 3.4). The downstream boundary condition for the pump system head development would be the operating levels in the Southern Forebay. Note, two conditions are considered for the pump discharge: one includes a siphon condition, and the second does not. Section 3.5.4 describes the purpose and implementation of these conditions.

3.5.2 Governing Equations

System losses would be calculated using the industry standard Bernoulli equation. Friction losses within the Bernoulli equation would be calculated using the Darcy-Weisbach equation and the friction value *f*. Darcy's *f* value depends on a surface roughness, e, for the interior of the pipe. The value for *f* varies with the Reynolds number, and the correlation is provided with industry standards. For modeling purposes, the standard engineering practice is to use the Colebrook equation for determining *f*. The Colebrook equation is embedded within the InfoWorks ICM model.

$$H = f * L/D * V^2/2g \qquad \textbf{[12]}$$

Where:

f = Dimensionless friction factor

L = Pipe length (feet)

D = Pipe diameter (feet)

V = Average velocity (fps)

g = Acceleration due to gravity (ft/s^2)

Minor losses associated with entrances, exits, bends, expansions, and contractions would be based on the following standard equation normally associated with the Bernoulli equation.

$$H = K\, V^2/2g \qquad \textbf{[13]}$$

Where:

h = Head loss (feet)

K = Minor loss coefficient

V = Velocity (fps) (per the Cameron hydraulic handbook, velocity in the smaller pipe is used for contractions and expansions)

g = Acceleration due to gravity (32.2 ft/sec^2)

For the pumping plant wet well design, submergence requirements to suppress vortices would be calculated with the following equation from ANSI/HI Standard 9.8:

$$S = D^*(1.0 + 2.3F_D) \quad \textbf{[14]}$$

Where:

S = Minimum submergence depth (ft)

D = Outside diameter of bell or ID of pipe inlet (ft)

F_D = Froude number = $V/(gD)^{0.5}$

- V = Velocity at suction inlet = flow/area, based on D (ft/s)
- g = Acceleration due to gravity (32.2 ft/sec^2)

Roller gates would be used to allow gravity flow into the Southern Forebay when flow demands and differential head conditions are high enough. The orifice equation would be used for the head loss of the roller gates.

$$Q = C_d *A * (2gH)^{0.5} \quad \textbf{[15]}$$

Where:

Q = Flow (cfs)

C_d = Coefficient of discharge (0.6): Based on Reclamation's Design of Small Dams

A = Area of the gate opening (ft^2)

g = Acceleration due to gravity (ft/s^2)

H = Difference between upstream and downstream WSEs (feet)

An overflow weir would be incorporated above the roller gates to serve as surge relief. The following weir equation would be used for sizing the weirs:

$$Q = C_d *L * (H)^{3/2} \quad \textbf{[16]}$$

Where:

Q = Flow (cfs)

C_d = Coefficient of discharge (3.33 for broad-crested weir)

H = Total energy head above the weir crest height (feet)

L = Weir or crest length (feet)

3.5.3 Criteria

The following criteria apply:

- **River Elevations:** Table 1, located in Section 3.3.2, provides the river elevations.
- **Southern Forebay WSEs:** The required WSEs in the Southern Forebay would be established by the computed head conditions required to deliver flows into the South Delta Conveyance Facilities System (Section 3.6).
- **Absolute Roughness e:** The value for e is used to derive Darcy's *f* for pipe friction. A range of e from 0.1 to 1.5 millimeters (mm) would be used, per the *Handbook of Hydraulic Resistance Coefficients of Local Resistance and Friction* (Idel'chik, 1960).
- **Siphon and Non-siphon Conditions:** Each pump discharge would use a discharge siphon outside the pumping lant. The invert elevation at the crest of the siphon is designed to isolate the pumping plant from the Southern Forebay without the use of nonreturn (check) valves. In a siphon condition, the pumping head is identified as the difference in WSE between the pumping plant wet well and the Southern Forebay.

  Static head pumping requirements with pumping through an unprimed discharge siphon must also be calculated and incorporated into the pump sizing evaluation for steady-state operations, as this condition would occur each time the pump is started and when the siphon fails to generate. This non-siphon condition would assume no hydraulic connection between the pumping plant wet well and the Southern Forebay. That is, no vacuum would be established in the pump discharge piping arrangement, and the static head values associated with the unprimed condition would be established using the difference between the free WSE within the discharge piping's gooseneck arrangement and the Sacramento River WSE at zero-flow conditions or the pool elevation at the entrance to the intake's drop shaft, whichever is lower.

  The maximum vacuum pressure developed in the discharge siphon must not exceed 20 feet. As such, the new Southern Forebay must include provisions (such as a weir establishing a minimum discharge pool) to limit the minimum WSE to no more than 20 feet below the crown of the discharge piping gooseneck. For the condition when filling the forebay with the WSE below the weir, the vacuum breaker valves should be opened to prevent a siphon condition exceeding 20 feet.

3.5.4 Methodology

3.5.4.1 Pump Selection

The pumping plant maximum and minimum total dynamic head (TDH) conditions within the design flow range for the project would be determined as follows:

- Determine the maximum and minimum free WSEs within the pumping plant wet well by determining the maximum and minimum hydraulic gradeline (HGL) profile envelopes between the Sacramento River Intakes and the pumping plant wet well.
- Calculate the minimum WSE in the pumping plant wet well based on the minimum Sacramento River elevation and the tunnel head loss calculated with the highest Manning's n value at the maximum flow condition.
- Calculate the maximum WSE in the pumping plant wet well based on the maximum Sacramento River elevation and zero flow within the tunnel.

- Compute the TDH required to lift flow from the pumping plant wet well HGL elevation to the primed and unprimed discharge head of the pumps. The TDH calculation would also include both friction and minor losses developed within the vertical pump discharge column and discharge piping.
- Coordinate with candidate pump manufacturers to obtain pump curves that can cover the envelope of hydraulic conditions.
- Overlay the pump curves on the system curves and verify coverage of the desired operating ranges (flow and head conditions).
- Determine a reasonable number of pumps and combinations to cover the flow range.
- Determine whether there are limitations on upstream or downstream head losses that could affect pump selection.
- Perform an economic cost analysis that considers capital and operational costs to determine the optimal facility sizes. For example, smaller-diameter tunnels have lower capital costs but increased pumping costs due to increased head loss.

3.5.4.2 Pump Suction Configurations and Submergence Requirements

Two suction configurations would be examined. One is a formed suction intake (FSI), and the other is an open-top can, as shown on Figure 2. Two criteria would need to be examined for each configuration when considering the submergence of the pumps. The first criterion is to calculate the required submergence to suppress vortexing, as outlined in the ANSI/HI Standard 9.8. The second criterion is to examine the manufacturers' required net positive suction head (NPSH) for their particular pump design. The more stringent value (highest submergence) between these two criteria would be selected as the final submergence value.

For the FSI configuration, the vortex calculation would be applied to the horizontal inlet portion of the intake in accordance with ANSI/HI Standard 9.8.

For the open-top can configuration submergence calculations for vortexing would be performed both at the pump inlet and at the top of the can (invert of the pumping plant forebay). Calculations at the pump bell would use the previously identified equation from ANSI/HI Standard 9.8.

Vortex submergence calculations at the top of the can require the adaption of the standard vortex equation to account for the flow area between the can and the pump column. This is done by computing an equivalent circular diameter, based on the calculated cross-sectional area of the annular space between the pump column assembly and the ID of the pump intake can. The velocity in the annular space between the pump column assembly and the ID of the pump intake can must not exceed 5 fps, using the pump's maximum allowable region flow value. The adapted submergence equation is provided here.

$$S = 1.0*D + 2.3*[(12*0.409Q/D^2)/(12gD)^{0.5}]*D \qquad \textbf{[17]}$$

Where:

S = Submergence (inches)

D = Equivalent diameter of the annular space between the pump column and pump can (inches)

Q = Flow (gallons per minute [gpm] at the pump's maximum allowable region flow value)

g = Acceleration due to gravity (32.2 ft/s^2)

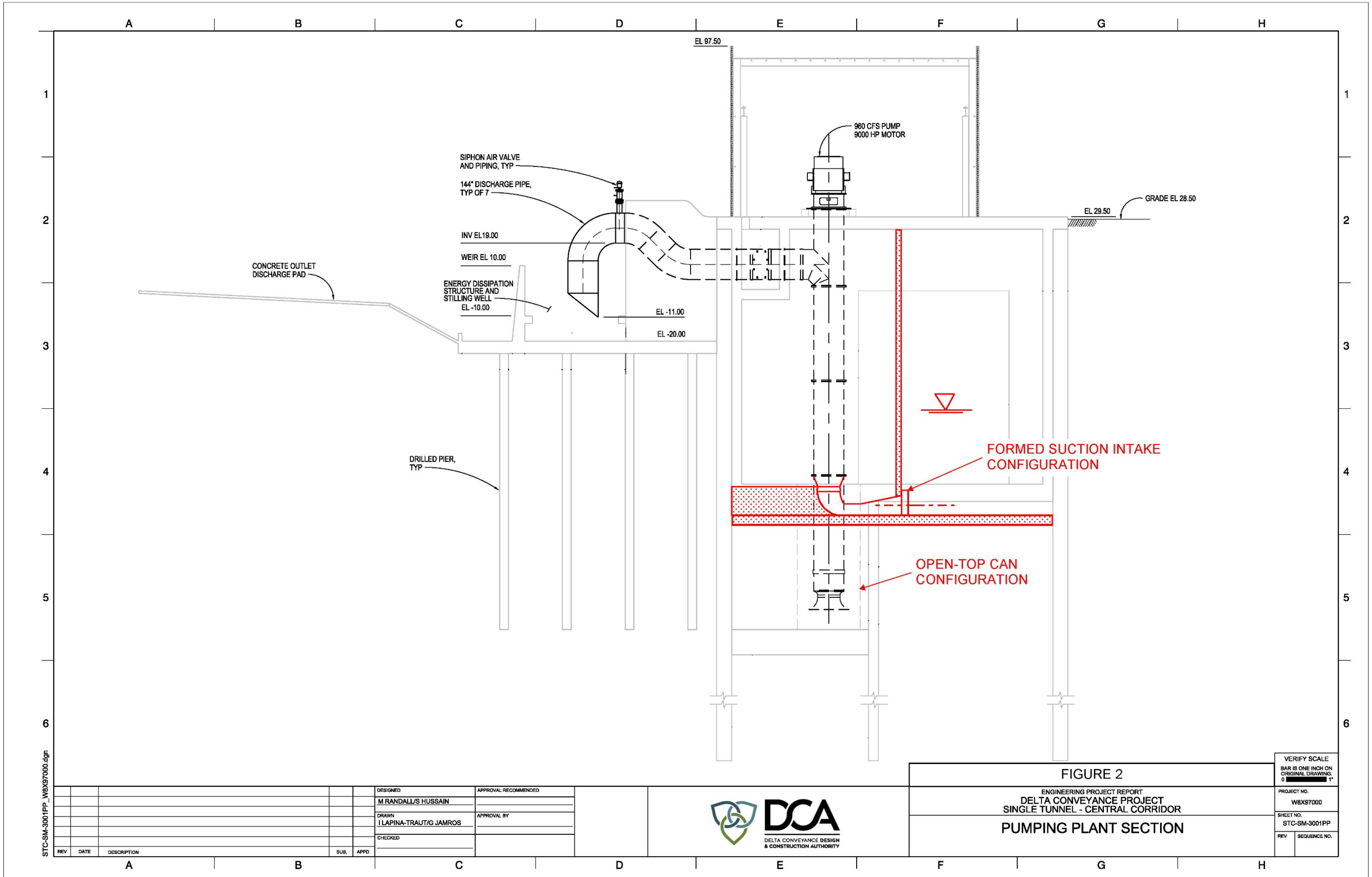

Figure 2. Pumping Plant Section

3.5.4.3 Net Positive Suction Head Evaluation

The NPSH evaluation compares the available NSPH ($NPSH_a$) of the system to the required NPSH ($NPSH_r$) as provided by the pump manufacturer. The minimum $NPSH_a$ to the main raw water pumps would be based on the calculated submergence depth between the free-water surface within the wet well and the vertical centerline of the first-stage impeller eye elevation. The ratio of the $NPSH_a$ to a pump's $NPSH_r$ must, as a minimum, comply with recommendations of ANSI/HI 9.6.1, the pump manufacturer's recommendation, or 1.35, whichever is higher, throughout the entire operating region of the pumps.

3.5.4.4 Gravity Flow Through the Delta Conveyance System

During periods when the Sacramento River stage is sufficiently high and the WSE in the Southern Forebay is sufficiently low, gravity flow between the Sacramento River intakes and the Southern Forebay can be achieved. Under these conditions, the main raw water pumps would be stopped, and flow control gates located within the repurposed Southern Forebay Inlet Structure Tunnel Launch Shaft (adjacent to the pumping plant) would be opened. The gravity flow to the Southern Forebay would vary based on the hydraulic conditions, but the maximum gravity flow must not exceed the design flow considered for the project, which is currently between 3,000 to 7,500 cfs.

The gravity flow gates would be sized based on the maximum and minimum HGLs of the upstream system. The target maximum head loss through the gates is 0.5 foot.

3.5.4.5 Surge Overflow Weir

To protect the project during transient surge events, overflow weirs would be incorporated into the tunnel outlet shaft. The wall openings above the gravity flow gates and the bottom of the concrete slab of the gravity outlet structure would act as surge overflow weirs. A freeboard of 0.5 feet would be provided between the surge overflow weir crest elevation and the maximum working WSE of the Southern Forebay. A transient surge analysis would be performed to determine the required flowrate and maximum allowable upstream head conditions for the weir. The required length of the weirs would be calculated with the weir equation previously identified.

3.6 Southern Forebay

The Southern Forebay would be designed to provide operational storage, to equalize and balance differences between inflow from the Sacramento River intakes and water exported by Banks and Jones. The Southern Forebay's primary functions include:

- Storing supply flows delivered from the upstream project facilities
- Balancing flows between the upstream supply facilities and the downstream Banks and Jones demand facilities
- Maintaining WSEs high enough to provide the hydraulic head required to drive demand flows by gravity from the Southern Forebay through the SDCF, the Delta-Mendota Canal, and the California Aqueduct to the existing pumping plants

The Southern Forebay embankment and spillway requirements would be as described in the Forebay Conceptual Design Criteria TM, (DCA, 2021b).

The Southern Forebay Siting Analysis TM (DCA, 2021d) provides Southern Forebay sizing, as well as controlling elevation requirements.

In the InfoWorks ICM system models, the Southern Forebay would be modeled as a reservoir and would be the downstream boundary condition for the upstream subsystem from the Sacramento River to the Southern Forebay. It would also be the upstream boundary conditions for the subsystem model from the Southern Forebay to the canals feeding the Banks and Jones pumping plants.

3.7 South Delta Conveyance Facilities

The South Delta Conveyance Facilities system consists of the following major components that would supply Banks:

- Southern Forebay Outlet Structure
- South Tunnels
- South Delta Outlet and Control Structure
- California Aqueduct Control Structure
- Existing California Aqueduct

In addition, an allowance for an extension to supply Jones would include the following structures:

- Jones Control Structure
- Jones Tunnel
- Jones Outlet Structure
- Delta-Mendota Control Structure
- Existing Delta-Mendota Canal

The entire South Delta Conveyance Facilities system sets the operational WSEs for the Southern Forebay, as described in Section 3.7.3.

3.7.1 Governing Equations

The hydraulic elements in the South Delta Conveyance Facilities include:

- Inlets
- Tunnels
- Outlets
- Radial gate control structures
- Open channels

The entire South Delta Conveyance Facilities system would be modeled with InfoWorks to develop a systemwide hydraulic and energy gradeline from the Southern Forebay, downstream to the canals feeding Banks and Jones. The governing equations are provided here:

For the tunnels, the InfoWorks ICM program would use the following Manning's equation:

$$Q = (1.486/n) * AR^{2/3} * S^{1/2} \qquad \textbf{[18]}$$

Where:

n = Manning's roughness coefficient

A = Cross-sectional area of flow (ft^2)

R = Hydraulic radius (cross-sectional area divided by the wetted perimeter [feet])

S = Slope (head loss per unit length of tunnel [ft/ft])

For the open channels, the program would use the following Manning's equation:

$$Q = (1.486/n) * AR^{2/3} * S^{1/2} \qquad \textbf{[19]}$$

Where:

n = Manning's roughness coefficient

A = Cross-sectional area of flow (ft^2)

R = Hydraulic radius (cross-sectional area divided by the wetted perimeter [feet])

S = Slope (head loss per unit length of tunnel [ft/ft])

Minor losses associated with entrances, exits, bends, expansions, and contractions would be based on the following standard equation normally associated with the Bernoulli equation:

$$h = K\ V^2/2g \qquad \textbf{[20]}$$

Where:

h = Head loss (feet)

K = Minor loss coefficient

V = Velocity (fps)

g = Acceleration due to gravity (32.2 ft/sec^2)

Flow through the radial gates would be modeled with the following orifice equation:

$$Q=C_d * A * (2g*h)^{1/2} \quad \textbf{[21]}$$

Where:

Cd = Coefficient of discharge =0.72; per Reclamation's *Design of Small Canal Structures*

A = Area of the orifice (ft^2)

g = Acceleration due to gravity (ft/s^2)

h = Difference between upstream and downstream WSEs (feet)

Minor losses for the lagoon piping would be based on the following standard equation normally associated with the Bernoulli equation:

$$h = K V^2/2g \quad \textbf{[22]}$$

Where:

h = Head loss (feet)

K = Minor loss coefficient

V = Velocity (fps)

g = Acceleration due to gravity (32.2 ft/sec^2)

3.7.2 Criteria

The following criteria apply:

- **Manning's n Values Tunnel:** A range would be analyzed between 0.014 and 0.016. This range is based on roughness values that have been used in the design of other tunnel systems of similar size and with similar lining systems.
- **Manning's n Values Open Channels:** n = 0.016.
- **Minor Loss Coefficients:** The ICM model has standard rating curves for loss coefficients that have been developed over time in a wide variety of gravity network systems, using pipes and manholes. These standard ratings would be used for initial modeling, and the results would be reviewed for reasonableness. User-defined curves can be used for elements that are not well-represented by the model.
- **Normal Low WSE – Banks**: The minimum normal operating water level in the California Aqueduct just upstream of the Banks Pumping Plant is 1.1 feet, as provided in the Conceptual Engineering Report (WaterFix BTO) dated 2018.
- **Normal High WSE – Banks**: The maximum normal operating water level in the California Aqueduct just upstream of the Banks Pumping Plant is 5.1 feet, as provided in the Conceptual Engineering Report (WaterFix BTO) dated 2018.
- **Normal Low WSE – Jones**: The minimum normal operating water level in the Delta-Mendota Canal just upstream of the Jones Pumping Plant is 0.43 feet, as provided in the Conceptual Engineering Report (WaterFix BTO) dated 2018.

- **Normal High WSE – Jones**: The maximum normal operating water level in the Delta-Mendota Canal just upstream of the Jones Pumping Plant is 5.6 feet, as provided in the Conceptual Engineering Report (WaterFix BTO) dated 2018.
- **Maximum Flowrate to California Aqueduct**: 10,670 cfs
- **Maximum Flowrate to Jones:** 1,500 cfs
- **Maximum Combined Flow between Southern Forebay and California Aqueduct:** 12,170 cfs
- **Targeted Maximum Control Structure Head loss:** 1.0 foot
- **Maximum Tunnel Velocity:** 8 fps
- **Minimum Southern Forebay WSE:** 5.24 feet
- **Minimum Height of South Delta Aqueduct Control Gates:** 26 feet
- **Maximum Radial Gate Width:** 24 feet

3.7.3 Methodology

The control for the South Delta Conveyance Facilities System is the minimum WSEs at Banks and Jones pumping plants. This is based on the minimum WSEs needed to maintain proper operations at each plant. The adopted procedure is to use these elevations to calculate head losses at the design flows through the upstream South Delta Conveyance Facilities System to determine the resulting head required in the Southern Forebay. This head is compared to the minimum allowable head provided in the criteria. The InfoWorks model would be used to link the individual hydraulic elements together to form a continuous HGL from the downstream pumping plants to the upstream Southern Forebay. Facility sizes would be adjusted in an iterative approach to obtain optimal sizes that fit within the allowable water surface boundary conditions at the Southern Forebay and the pumping plants.

The South Delta Outlet and Control Structure would control the discharge from the Southern Forebay. The height of this structure and control gates would be set by the maximum WSE computed for the Southern Forebay, plus 2.5 feet of freeboard.

The California Aqueduct Control Structure would control the discharge from Clifton Court Forebay (CCF) that feeds Banks. This structure would be used in tandem with the South Delta Outlet and Control Structure to either deliver water solely from the Southern Forebay, solely from CCF, or a combination of both. The California Aqueduct Control Structure height would be set at elevation 24 feet, minimum, to protect against the 200-year project flood event plus estimated sea-level rise plus 3 feet of freeboard into the Banks Pumping Plant Forebay.

4. Hydraulic Transient Surge Modeling

Systemwide hydraulic transient surge analyses would be performed for the project. These analyses would evaluate the maximum and minimum hydraulic grade profile throughout the systemwide tunnel alignment, and identify and size required surge control system components, including specific performance and operational requirements associated with project components and associated surge control features. The analysis would also determine whether further refinement is required for facility sizing to help control adverse surge conditions.

4.1 Hydraulic Transient Surge Analysis

This section provides a brief description of the hydraulic transient surge analysis that would be performed. Further details would be provided in Capacity Analysis for Preliminary Tunnel Diameter Selection TM, (DCA, 2021a), and Hydraulic Analysis of Delta Conveyance Alternatives-Main Tunnel System TM, (DCA, 2021c). The hydraulic transient surge analyses would include the following tasks:

- Definition of problem and boundary conditions
- Creation of relevant model
- Generation of applicable results and figures
- Validation through comparison to established results or methods, or both
- Presentation of relevant information

4.1.1 Problem Definition and Boundary Condition

The scenario would be described for each analysis run, including significant system configurations and boundary conditions. Surge conditions would also be performed, assuming lowest conduit friction losses to replicate the most conservative surge impacts.

4.1.2 Model Construction

The hydraulic transient surge model construction would be as follows:

- The modeled geometry would include features significant to the flow domain, including:
  - High and low points in the tunnel profile
  - Tunnel shaft sizes and locations
  - Sacramento River high and low WSEs at each intake
  - High and low wet well WSEs at each hydraulic control facility with a free-water surface
  - Tunnel overflow facilities
  - Main raw water pump sizing and operating capacities, gate operating parameters, and other key features specific to the run scenario
- Smaller details not affecting the flow solution could be disregarded, such as small grade breaks, horizontal bends, and other minor hydraulic features
- Mechanical items, such as pumps, control gates, vents, and other key features, would be described, including:
  - Pump curves
  - Motor and pump inertia
  - Orifice sizes
  - Loss characteristics
  - Gate closing and opening rates
- Critical tunnel data would be described to evaluate the risk to operations, including:
  - Lining material
  - Diameter
  - Thickness
  - Wave speed used in the calculation

- Surge mitigation options, such as surge chambers, would be described in detail, including:
  - Type
  - Inlet and outlet configuration
  - Stored volume
  - Water surface operating levels

4.1.3 Solution

The model would produce the following solutions:

- Initial Conditions – The model would run to establish steady-state initial conditions. The output must indicate steady-state conditions were achieved before the start of the transient surge.
- Transient Solution – The transient surge would be modeled over enough time to show the extent of the system response.
- Model Refinement – If additional control features are recommended, such as surge chambers, the model would be run with the additional features to show the relative improvement.
- Stability Criteria and Node Density – Solution variables, such as timestep and node density, would be shown to be sufficient for accurately modeling the system.

4.1.4 Sensitivity

The model would be evaluated for sensitivity to variables important to the transient characteristics. These could include:

- Timestep
- Node density
- Orifice size
- Pipe wave speed

4.1.5 Requirements

The maximum pressure surges must not exceed the rated pressure or top of structures for system components. Minimum pressure surges must not drop below the crown of the tunnel, the rated collapse pressure, or one-half the gage vapor pressure, whichever is most restrictive. In some cases, minor excursions below the crown of the tunnel could be allowed, provided the locations can easily refill and vent air from the tunnel.

4.1.6 Results and Presentation

The hydraulic transient surge analysis results would be presented as follows:

- Pressure Envelope – The pressure envelope would show the maximum and minimum transient pressures during the simulation at all locations of the modeled system. The envelope would show if and where the tunnel design pressure limits are exceeded.
- HGL Envelope – The HGL envelope would show the maximum and minimum transient HGL during the simulation at all locations of the modeled system. The envelope would show if and where the tunnel design pressure limits (high or low) are exceeded. The HGL would show the local hydraulic grade relative to the tunnel profile.

- Pressure versus Time Plots – Pressure versus time plots would be provided at all critical locations and would be over enough time to show the extent of the system response.
- Flow versus Time Plots – Flow versus time plots would be provided at critical boundary conditions and would be over enough time to show the extent of the system response.

4.2 Software

4.2.1 Transient Surge Analysis Software

The following software would be used for the transient surge analysis:

- For pressurized pipe systems, the selected software must analyze the system for fluid transient surges using the Method of Characteristics or the Wave Characteristic Method. The software would be comparable to products such as Bentley Systems' Hammer, or KYPipe. The selected transient modeling software must be accepted by the engineer before the simulation.
- For tunnel or pipe systems where flows can transition between open channel flow and pressurized flow, specialty models are required. The selected tunnel filling transient modeling software must be accepted by the engineer before the simulation.

5. References

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6. Document History and Quality Assurance

Reviewers listed have completed an internal quality review check and approval process for deliverable documents that is consistent with procedures and directives identified by the Engineering Design Manager (EDM) and the DCA.

Approval Names and Roles			
Prepared by	**Internal Quality Control review by**	**Consistency review by**	**Approved for submission by**
Ted Davis and Mark Draper / EDM Project Engineers Graham Bradner / EDM Forebay & Levee Lead	Tony Naimey / EDM Pumping Plant Lead and Kevin Nielsen / EDM Hydraulics Subject Matter Expert	Phil Ryan / EDM Engineering Manager Gwen Buchholz / DCA Environmental Consultant	Terry Krause / EDM Project Manager

This interim document is considered preliminary and was prepared under the responsible charge of Anthony M. Naimey, California Professional Engineering License M28450.

Note to Reader

This is an early foundational technical document. Contents therefore reflect the timeframe associated with submission of the initial and final drafts. Only minor editorial and document date revisions have been made to the current Conformed Final Draft for Administrative Draft Engineering Project Report version.