Appendix A1 Hydraulic Analysis Criteria (Final Draft)

1. Introduction and 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 Bethany Reservoir. The major hydraulic elements include the following:

- River intakes
- Conduits (open channel and pressurized)
- Drop shafts
- Main Tunnel
- Surge Basin
- Pumping Plant
- Bethany Reservoir Aqueduct
- Bethany Reservoir Discharge Structure

1.1 Organization

- Introduction and Purpose
- Hydraulic Criteria
- Major System Facilities
- Hydraulic Transient Surge Modeling
- References

1.2 Background

The Project configuration is a single main tunnel system as shown in the Engineering Concept Drawings. The Project includes the following infrastructure:

- Two Sacramento River intake facilities (C-E-3 and C-E-5)
- A main tunnel and surge basin structure connecting the river intakes to the Bethany Reservoir Pumping Plant (BRPP)_
- Bethany Reservoir Aqueduct connecting the BRPP to the Bethany Reservoir

The DCO has established 6,000 cubic feet per second (cfs) as the maximum diversion capacity for the Project. The DCA performed a systemwide hydraulic and capacity analysis for design flow capacities of 6,000 cfs, which is described in the Concept Engineering Report (CER) Appendix A2 Hydraulic Analysis of Delta Conveyance Options, Attachment 1. Capacity Analysis for Preliminary Tunnel Diameter Selection.

1.3 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
- Idelchik, 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

2. 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.

2.1 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 at the Bethany Reservoir, 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 the 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. Major System Facilities

The system's major hydraulic elements are discussed individually in the following sections:

- Intake facilities identified as Intakes C-E-3 and C-E-5
- Tunnels and shafts
- Surge Basin
- BRPP
- Bethany Reservoir Aqueduct and discharge structure connecting the BRPP to Bethany Reservoir

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

- Hydraulic analysis between the Sacramento River and the BRPP
- Hydraulic analysis between the BRPP and the delivery point to the Bethany Reservoir.

The upstream system between the Sacramento River and the BRPP is largely driven by the upstream boundary conditions: the water surface elevations (WSELs) in the river and the allowable hydraulic losses in the intakes, tunnels, and other appurtenances. The downstream system between the BRPP and the Bethany Reservoir is controlled by the lowest operational WSEL feeding the pumps at the pumping plants and the operational WSEL of the Bethany Reservoir. This upstream system would control the Surge Basin WSEL; which, in turn, establishes the inlet boundary conditions for the pumping plant.

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.

3.1 Intake Facilities

The Project would include two intakes (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 control and isolation gates, 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.

3.1.1 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:

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

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:

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

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)

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

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:

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

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.

$$Q=C_dA(2*g*h)^{1/2} \quad \textbf{[5]}$$

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 WSELs (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:

$$h = K\,V^2/2g \quad \textbf{[6]}$$

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.

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

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 WSELs (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.

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

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 drying lagoon piping would be based on the following standard equation normally associated with the Bernoulli equation.

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

Where:

h	=	Head loss (feet)
K	=	Minor loss coefficient
V	=	Velocity (fps)
g	=	Acceleration due to gravity (32.2 ft/sec^2)

3.1.2 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:** Maximum design flow for each separate screening facility is 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 WSEL at Intake C-E-3	Sacramento River WSEL at Intake C-E-5
High Pumping Head	3.72	3.61
Design Pumping Head	4.59	4.47
Normal Low Pumping Head	5.92	5.75
Extreme Low Pumping Head	27.3	26.3

[a] Pumping Head refers to the head conditions at the BRPP

3.1.3 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. Cylindrical tee screens were examined.
- Each individual cylindrical tee screen would have a downstream conduit connecting the screen facility to the sedimentation basis. 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.

3.2 Tunnels and Shafts

3.2.1 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)

3.2.2 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 BRPP.
- **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	6,000
C-E-3	3,000
C-E-5	3,000

3.2.3 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 BRPP. 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.
  – 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.

3.3 Pumping Plant

3.3.1 System Head Curves Development

The BRPP 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 3.1 and 3.2). The downstream boundary condition for the pump system head development would be the operating levels in the Bethany Reservoir.

3.3.2 Governing Equations

System losses for the pump discharge piping and would be calculated using the industry standard Bernoulli equation. Friction losses within the Bernoulli equation would be calculated using the Darcy-Weisbach equation (Equation 12) 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:

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

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)

An overflow weir would be incorporated at the top of the tunnel reception shaft within the Surge Basin to serve as surge relief for the main tunnel. The Surge Basin would be located immediately upstream of the BRPP wet well. The following weir equation would be used for sizing the weir:

$$Q = C_d * L * (H)^{3/2} \qquad \textbf{[15]}$$

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)

3.3.3 Criteria

The following criteria apply:

- **River Elevations:** Table 1 provides the river elevations.
- **BRPP wet well WSELs:** The WSELs in the wet well would be established by the computed head conditions required to deliver river diversion flows from the intakes into the BRPP wet well Bethany Complex (Section 3.5).
- **Absolute Roughness e:** The value for e is used to derive Darcy's *f* for pump suction and discharge 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).

3.4 Methodology

3.4.1 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 WSELs 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 WSEL 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 WSEL 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 Bethany Reservoir. The TDH calculation would also include both friction and minor losses developed within the pump discharge piping system and the Bethany Reservoir Aqueduct pipelines.
- 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.

3.4.2 Pump Suction Configuration and Submergence Requirements

Each pump suction configuration would include a formed suction intake (FSI). Two criteria would need to be examined for the suction 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 vortex calculation would be applied to the horizontal inlet portion of the intake. The second criterion is to examine the manufacturers' required net positive suction head ($NPSH_3$) for their particular pump design. The more stringent value (highest submergence) between these two criteria would be selected as the final submergence value.

3.4.3 Net Positive Suction Head Evaluation

The NPSH evaluation compares the available NSPH ($NPSH_a$) of the system to the required NPSH ($NPSH_3$) 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 $NPSH_3$ 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.

3.4.4 Surge Basin Overflow Weir

To protect the Project during transient surge events within the tunnel, a circular overflow weir would be incorporated above the tunnel reception shaft that would be located within the Surge Basin structure. The top of the overflow weir wall would be established to maintain the maximum HGL profile to within the design criteria limits established for each tunnel shaft and all hydraulic structures connected to the tunnel. A transient surge analysis would be performed to determine the required length and height of the circular weir and would be calculated with the weir equation previously identified.

3.5 Bethany Reservoir Conveyance Facilities

The Bethany Complex system consists of the following major components that would supply the Bethany Reservoir:

- Bethany Reservoir Pumping Plant
- Bethany Reservoir Aqueduct
- Bethany Reservoir Aqueduct Surge Tanks
- Bethany Reservoir Outlet Structure
- Bethany Reservoir

3.5.1 Governing Equations

The hydraulic elements in the Bethany Reservoir Conveyance Facilities include:

- Entrances into FSIs for each main pump within the BRPP
- Pump discharge piping systems
- Bethany Reservoir Aqueduct pipelines
- Bethany Reservoir Aqueduct exits into the Bethany Reservoir Discharge Structure

The entire Bethany Reservoir Conveyance Facilities would be modeled with InfoWorks to develop the hydraulic and energy gradelines from the BRPP to the Bethany Reservoir. The governing equations are provided here:

For the Bethany Reservoir Aqueduct pipelines, the InfoWorks ICM program would use 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.

$$H = f * L/D * V^2/2g \quad \textbf{[16]}$$

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.

$$H = K\,V^2/2g \quad \textbf{[17]}$$

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)

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

$$Q=C_d * A * (2g * h)^{1/2} \quad \textbf{[18]}$$

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 WSELs (feet)

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

$$H = K\,V^2/2g \quad \textbf{[19]}$$

Where:

h	=	Head loss (feet)
K	=	Minor loss coefficient
V	=	Velocity (fps)
g	=	Acceleration due to gravity (32.2 ft/sec^2)

3.5.2 Criteria

The following criteria apply:

- Maximum Combined Flow between BRPP and the Bethany Reservoir: 6,000 cfs
- **Maximum Combined Flow through each Bethany Reservoir Aqueduct pipeline:** 1,500 cfs
- Maximum Bethany Reservoir Aqueduct Pipeline Velocity: 10 fps

3.5.3 Methodology

The control for the Bethany Reservoir Conveyance Facilities System is to establish pumped flow capacities within the BRPP as required to maintain targeted WSELs within the outlet channels at Intakes C-E-3 and C-E-5 during river diversions into the Project. This is based on developing predetermined WSELs within the sedimentation basins and outlet channels at each intake needed to maintain proper operation of each intake for selected river diversion flow capacities. The InfoWorks model would be used to link the individual hydraulic elements of the Intakes C-E-3 and C-E-5, the tunnel and Surge Basin, and the Bethany Reservoir Aqueducts to the pumping plant to establish the pumping plant's inlet and outlet HGL profiles for properly sizing the main pumps.

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 the Bethany Reservoir Aqueduct, 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 in Appendix A2 Hydraulic Analysis of Delta Conveyance Options, Attachment 1. Capacity Analysis for Preliminary Tunnel Diameter Selection. 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 and pipeline profiles
  - Tunnel shaft sizes and locations
  - Sacramento River high and low WSELs at each intake
  - High and low wet well WSELs at the BRPP
  - Tunnel overflow facilities within the Surge Basin structure
  - Main raw water pump sizing and operating capacities, gate operating parameters at the intakes, 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
  - Pump control valve closing rates
- Critical tunnel and Bethany Reservoir Aqueduct pipeline 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 associated with the conceptual design. 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 conceptual 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 conceptual 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

American National Standards Institute (ANSI). 2012. *ANSI/HI Standard 9.8, Rotodynamic Pumps for Pump Intake Design*.

American National Standards Institute (ANSI). 2017. *ANSI/HI Standard 9.6.1, Rotodynamic Pumps Guideline for NPSH Margin.*

California Department of Fish and Wildlife (CDFW). 2010. *Fish Screen Criteria, June 19, 2000 version, Appendix S., California Salmonid Stream Habitat Restoration Manual, Fourth Edition.* July

California Department of Water Resources (DWR). 2018. "Table 5-2: Operational Water Surface Elevations at Banks Pumping Plan and Table 5-3: Operational Water Surface Elevations at Jones Pumping Plant." *Conceptual Engineering Report, California WaterFix, Byron Tract Forebay Option (WaterFix BTO)*. Volume 1/3. July.

Idel'chik, I.E. 1960. *Handbook of Hydraulic Resistance Coefficients of Local Resistance and Friction*. Moscow: Gosudarstvennoe Energeticheskoe Izdatel'stvo.

National Marine Fisheries Service (NMFS). 1997. *Fish Screening Criteria for Anadromous Salmonids, National Marine Fisheries Service Southwest Region*. January.

National Marine Fisheries Service (NMFS). 2010. *NOAA Technical Memorandum NMFS-NWFSC-1xx, NOAA Fisheries West Coast Region Anadromous Salmonid Passage Design Guidelines, National Marine Fisheries Service West Coast Region, Environmental Services Branch, Peer Review Draft*. August.

Miller, Donald S. 1990. *Internal Flow Systems*. 2nd Ed. Cranfield: BHRA (Information Services).

U.S. Department of the Interior Bureau of Reclamation (Reclamation). 1978. *Design of Small Canal Structures.*

Westaway, C. R., & Loomis, A. W. (1984). Cameron hydraulic data: *A handy reference on the subject of hydraulics, and steam*. Woodcliff Lake, NJ: Ingersoll-Rand.