Subject:	**Hydraulic Analysis of Delta Conveyance Options – Main Tunnel System (Final Draft)**
Project feature:	Projectwide
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1. Purpose

The purpose of this technical memorandum (TM) is to perform a preliminary hydraulic analysis of the Delta Conveyance Project (Project) main tunnel system in the Central and Eastern corridor alignment options and to document the hydraulic model build to perform this analysis. This TM considers a range of project design flow capacities, including 3,000, 4,500, 6,000, and 7,500 cubic feet per second (cfs).

Results of this analysis for each corridor (at each maximum design flow option) were used to:

- Develop the hydraulic grade line (HGL) envelopes for the 6,000-cfs design flow capacity under steady-state and hydraulic transient-surge conditions.
- Develop system head curves between the Sacramento River Intakes and the South Delta Pumping Plant (Pumping Plant) for the complete design flow range within the defined range of boundary conditions.
- Perform a simulated startup and shutdown of the Project and validate stable operation using real-time controls implemented in the hydraulic model for the 6,000-cfs design flow option.

This evaluation considered the following intake configurations for the range of Project design flow capacities:

- One 3,000 cfs intake (Intake C-E-5) for project design flow capacity of 3,000 cfs
- One 3,000 cfs intake (Intake C-E-3) and one 1,500 cfs intake (Intake C-E-5) for project design flow capacity of 4,500 cfs
- Two 3,000 cfs intakes (Intakes C-E-3 and C-E-5) for project design flow capacity of 6,000 cfs
- Two 3,000 cfs intakes (Intakes C-E-3 and C-E-5) and one 1,500 cfs intake (Intake C-E-2) for project design flow capacity of 7,500 cfs

Based on the results of a hydraulic and capacity analysis conducted for the Central Corridor, Capacity Analysis for Preliminary Tunnel Diameter Selection TM (DCA, 2021a), the maximum tunnel flow velocity was recommended to be limited to 6 feet per second (fps) at each maximum design flow capacity. This maximum flow velocity criterion of 6 fps resulted in the following tunnel inside diameter (ID) selections used in this analysis:

- 26-foot ID at the maximum design flow capacity of 3,000 cfs
- 31-foot ID at the maximum design flow capacity of 4,500 cfs
- 36-foot ID at the maximum design flow capacity of 6,000 cfs
- 40-foot ID at the maximum design flow capacity of 7,500 cfs

Figure 1 provides a schematic of the Project configuration considered in this analysis. This evaluation was conducted between the intakes and the new Pumping Plant wet well within the Figure 1 dashed boundary. Evaluation of the southern tunnels connecting the Southern Forebay to the existing State Water Project Harvey O. Banks Pumping Plant (Banks) and the Central Valley Project C.W. "Bill" Jones Pumping Plant (Jones) would be conducted as part of a subsequent hydraulic analysis of the South Delta Conveyance Facilities.

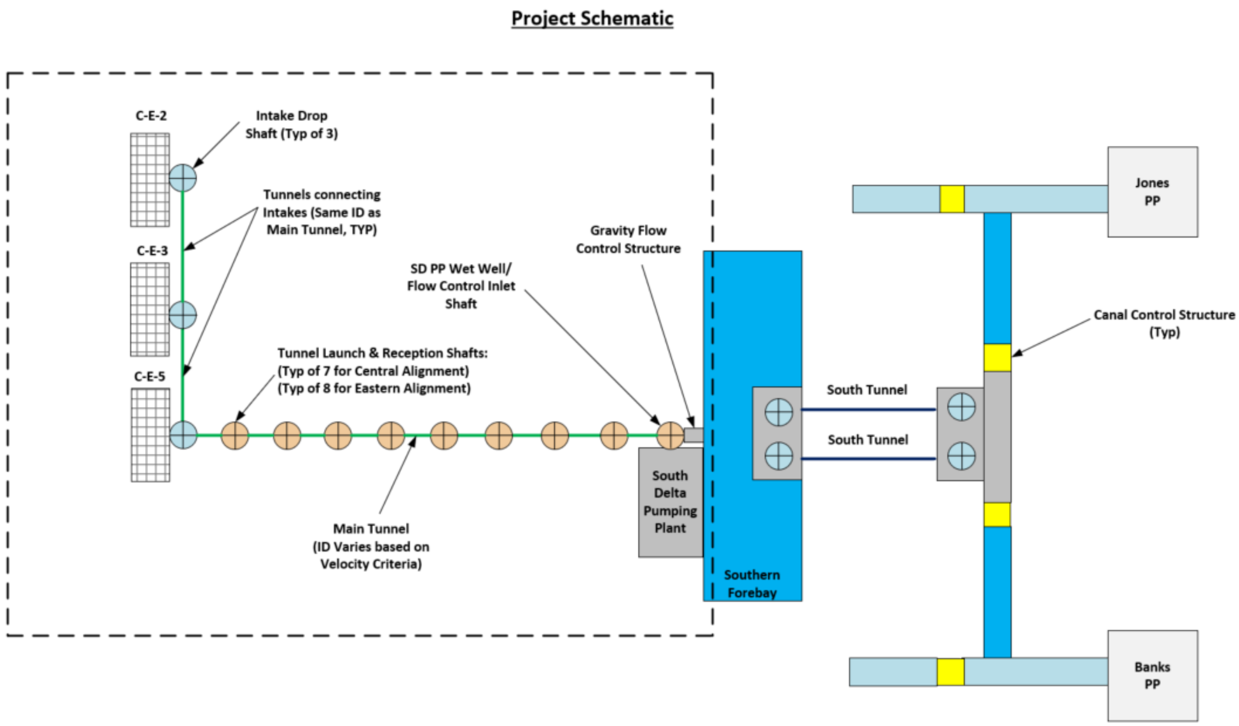

Figure 1. Delta Conveyance System Project Schematic

The following Project configuration was considered for this evaluation:

- Main tunnel alignments:
  - Central Corridor
  - Eastern Corridor
- Three intake locations on the east bank of the Sacramento River, as follows:
  - Intake 2 (Intake C-E-2), located at approximate River Mile (RM) 41.1
  - Intake 3 (Intake C-E-3), located at approximate RM 39.4
  - Intake 5 (Intake C-E-5), located at approximate RM 36
- Tunnels connecting the intakes per Figure 1
- Pumping Plant well and wet well inlet shaft structure per Figure 1

2. Methodology

The following describes the methodology for this evaluation:

- Selected the range of tunnel IDs to be evaluated for the Project that achieve the assigned maximum flow velocity criteria of 6.0 fps.
- Conducted system end-to-end hydraulic head loss for each tunnel corridor using candidate tunnel diameters and water surface elevations (WSELs) at each Sacramento River intake:
  - Determined the resulting WSELs within each tunnel shaft and at the Pumping Plant wet well
  - Developed the overall HGL profile for Project at each design flow capacity for each corridor
- Per request of the DCO, conducted evaluation of the Project system startup and shutdown based on a constant Sacramento River diversion rate of 1,000 cfs per 15 minutes.
- Conducted hydraulic transient-surge analysis using candidate tunnel diameters and developed the envelope of maximum and minimum HGLs for each transient-surge condition.

2.1 Criteria

The Hydraulics Analysis Criteria TM (DCA, 2021b) outlines the preliminary criteria used for this analysis. Additional criteria also utilized in this analysis are provided below:

- Intake Diversions for Steady-State Analysis:
  - The source of diversion flows from the Sacramento River was simulated from intake C-E-5 for diversion flow capacities up to 3,000 cfs.
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-3 and C-E-5 for design flow capacities above 3,000 cfs and up to 4,500 cfs.
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-3 and C-E-5 for design flow capacities above 4,500 cfs and up to 6,000 cfs.
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-2, C-E-3 and C-E-5 for design flow capacities above 6,000 cfs and up to 7,500 cfs.
  - Analysis was conducted at the maximum and minimum Sacramento River elevations at intakes C-E-2, C-E-3 and C-E-5.
- Transient-Surge Analysis:
  - Hydraulic transient-surge analysis was conducted between the intake C-E-3 drop shaft to the Pumping Plant wet well overflow facility at the maximum design flow capacity of 6,000 cfs and a tunnel diameter of 36 feet (ft).
  - Hydraulic transient-surge analysis was conducted between the intake C-E-2 drop shaft to the Pumping Plant wet well overflow facility at the maximum design flow capacity of 7,500 cfs and a tunnel diameter of 40 ft.
  - Simultaneous shutdown of the main raw water pumps in the Pumping Plant followed by closure of sediment basin outlet gates at each intake in operation was simulated for each transient-surge analysis.

- Hydraulic transient-surge analysis was conducted with steady-state boundary conditions using both the minimum and maximum predicted Sacramento River WSELs at intakes C-E-2, C-E-3 and C-E-5.

2.2 Assumptions and Boundary Conditions

The Hydraulics Analysis Criteria TM outlines the preliminary assumptions and boundary conditions used for this analysis. Additional assumptions and boundary conditions also utilized in this analysis are provided as follows:

- Central and Eastern tunnel alignments and locations of the intakes, shafts, Pumping Plant, and Southern Forebay are as shown in Figure 2.
- Tunnel ID connecting intakes C-E-2, C-E-3, and C-E-5 drop shafts match the main tunnel.
- These design flow sequences of operation were simulated in the hydraulic analysis of each tunnel corridor option:
  - 0 to 3,000 cfs; intake C-E-5 only.
  - 3,000 to 4,500 cfs; intake C-E-5 operates up to a maximum diversion rate of 1,500 cfs, and intake C-E-3 operates up to a maximum diversion rate of 3,000 cfs.
  - 4,500 to 6,000 cfs; intakes C-E-5 and C-E-3 each operate up to a maximum diversion rate of 3,000 cfs.
  - 6,000 to 7,500 cfs; intakes C-E-5 and C-E-3 each operate up to a maximum diversion rate of 3,000 cfs and intake C-E-2 operates up to a maximum diversion rate of 1,500 cfs.
- Sacramento River elevations (North American Vertical Datum of 1988 [NAVD88] assumed for the steady-state and transient-surge analysis based on information provided on the concept drawings for each intake (C-E-2, C-E-3 and C-E-5):
  - Intake C-E-2: min WSEL of 3.8 feet; design WSEL of 4.66 feet; normal WSEL of 6.03 feet; and high WSEL of 28.2 feet
  - Intake C-E-3: min WSEL of 3.72 feet; design WSEL of 4.59 feet; normal WSEL of 5.92 feet; and high WSEL of 27.3 feet
  - Intake C-E-5: min WSEL of 3.61 feet; design WSEL of 4.47 feet; normal WSEL of 5.75 feet; and high WSEL of 26.3 feet
- Overflow weir crest at the Pumping Plant wet well inlet shaft was set at 18 feet.
- Moment of inertia value for the Pumping Plant's large pumps was calculated using Bentley Systems' (Bentley's) HAMMER software as 825,500 pounds per square foot (lb-ft^2); this value was calculated based on the candidate pump's break horsepower (bhp) at the rated design point condition.
- Moment of inertia value for the Pumping Plant's smaller pumps was calculated using HAMMER as 187,000 lb-ft^2; this value was calculated based on the candidate pump's bhp at the rated design point condition.
- Wave speed for the main tunnel was calculated for the 36-foot ID tunnel as 1,647 fps. The 36-foot ID tunnel was used in this evaluation for the Project's maximum design flow capacity of 6,000 cfs.
- Wave speed for the main tunnel was calculated for the 40-foot ID tunnel as 1,589 fps. The 40-foot ID tunnel was used in this evaluation for the Project's maximum design flow capacity of 7,500 cfs.

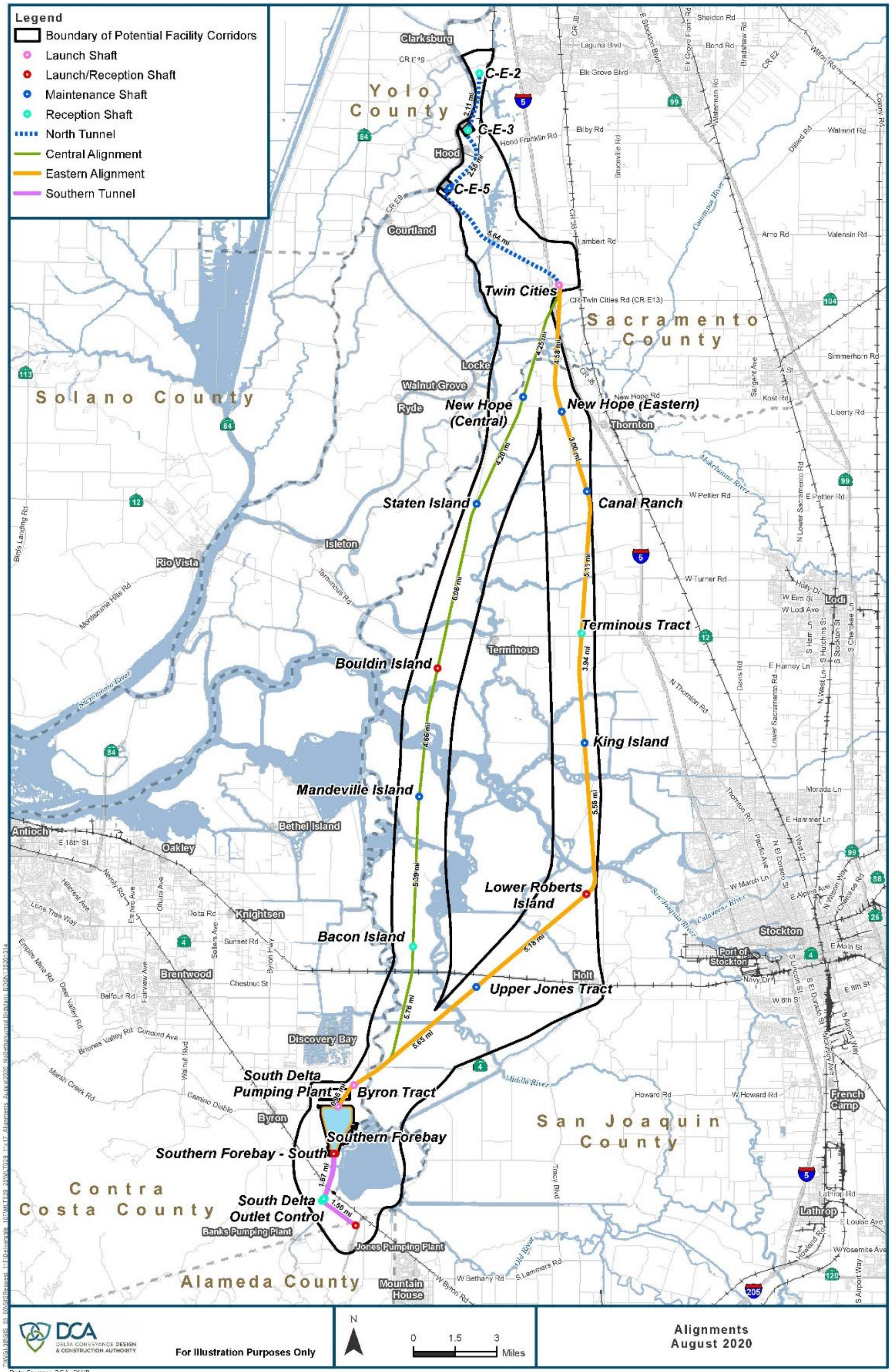

Figure 2. Delta Conveyance System Central and Eastern Alignments

2.3 Tools

The following are the tools used for the hydraulic analysis:

2.3.1 Conveyance System Hydraulic Model

A hydraulic model was constructed for the Project between the intakes and the Pumping Plant wet well, using Innovyze's InfoWorks Integrated Catchment Modeling (ICM) software, version 11.0.2.22016.

2.3.2 Transient-Surge Analysis

Bentley's HAMMER software was used to perform the transient-surge analysis. In addition to the steady-state pipe and hydraulic parameters, the HAMMER program uses the method of characteristics described by Wylie and Streeter (1993) to solve the pressure transients in the system. This method consists of deriving basic equations from physical principles (the continuity equation and conservation of energy and momentum). The equations are then solved along characteristic lines whose slope is dependent upon the acoustic wave speed.

3. InfoWorks ICM Model Development

3.1 Network Build

The hydraulic model was built to simulate the conveyance between the intakes and the Pumping Plant wet well and consists of the following components:

- Sacramento River intakes C-E-2, C-E-3, and C-E-5, including inlet structures, screens, control gates, sedimentation basins, radial gates, and outlet shafts to the tunnel.
- Pumping plant wet well and gravity flow structure.
- Tunnels connecting the intakes to the Pumping Plant wet well and gravity flow structure for the two main tunnel corridor options (Central and Eastern corridors).
- Vertical maintenance and reception shafts along the tunnel.

The modeled tunnel system network is presented on Figure 3.

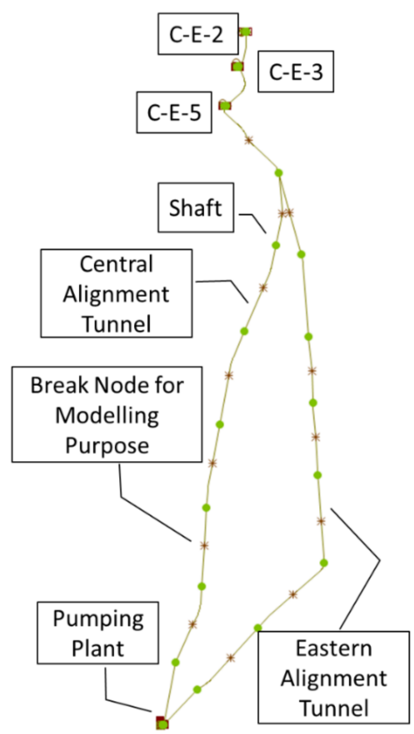

Figure 3. Overall Modeled Tunnel System Layout

3.1.1 Intakes

The representation of the intake facilities includes inlet structures, screens, baffle plates, drop gates, control gates, box conduits, sedimentation basins, radial gates, and outlet shafts to the tunnel. Each intake is modeled following the same methodology with an assumed maximum capacity of 3,000 cfs for intakes C-E-3 and C-E-5, and an assumed maximum capacity of 1,500 cfs for intake C-E-2. Figure 4 shows an overview of the InfoWorks ICM model from the intake structure to the inlet shaft at the Pumping Plant wet well, illustrating the complexity of the model representing this part of the system.

The intake model uses the vertical flat plate fish screen option. However, the model has been set up to always result in a fixed water surface elevation drop between the river and the sedimentation basins, regardless of flow. The model adjusts control gates between the river and the sedimentation basin to accomplish this elevation drop under the full range of flow conditions. If the cylindrical tee screen option is used, the screen losses would be slightly different, but by using the control gates the identical result would be achieved. Therefore, this preliminary model reflects the hydraulic behavior of the system for either screen type option.

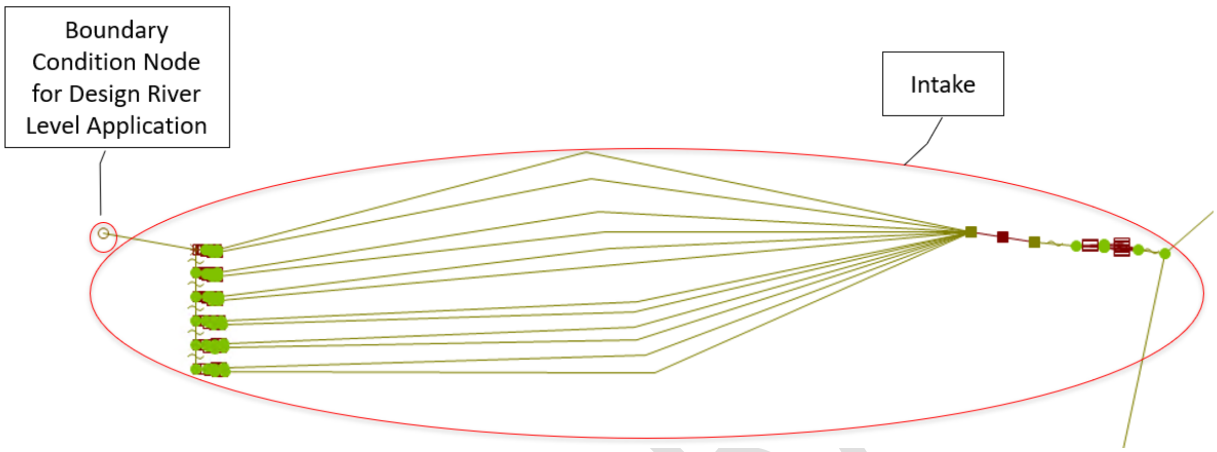

Figure 4. Model Representation from Intake Structure to Outlet Shaft

3.1.1.1 Intake Inlet Structure

The intake inlet structure is composed of six sections, each containing fish screens, baffle plates, a drop gate, a control gate, and two box conduits. For the analysis, fish screens are assumed to be vertical screens. The detailed model schematic from the screens to sedimentation basins are illustrated on Figure 5. The number of screens per intake section is documented in Table 1.

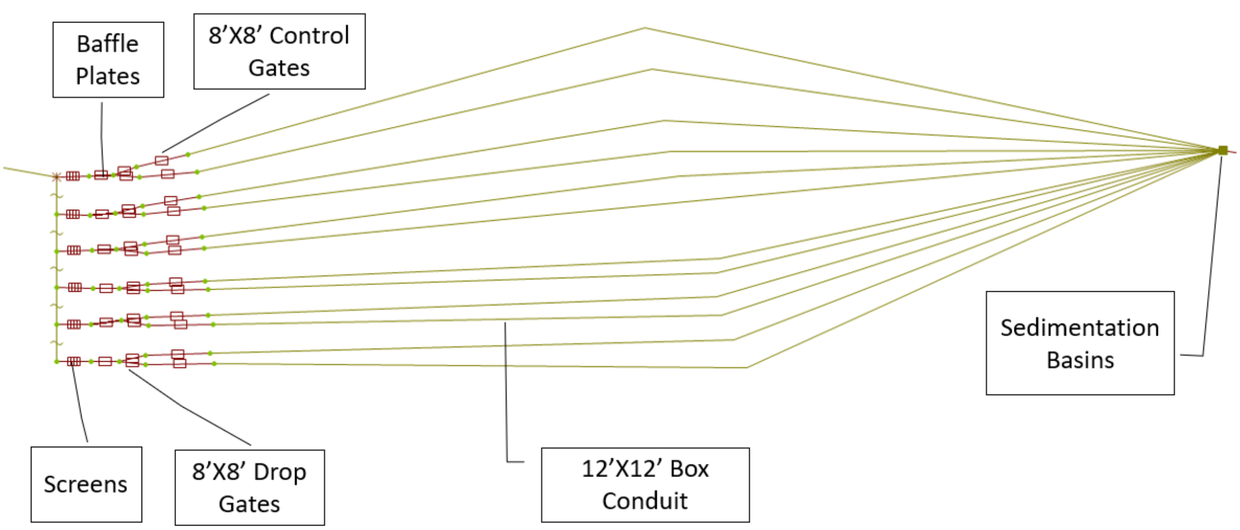

Figure 5. Detailed Model Representation Intake Screen to Sedimentation Basins

Table 1. Intake Fish Screens

Intake	Number of Sections	Screens per Section
Intake C-E-2	6	14
Intake C-E-3	6	11
Intake C-E-5	6	12

To simplify the model, the fish screens were modeled as one screen for each section instead of the individual screens. The total screen width was accounted for by multiplying the screens per section by the width of one individual screen. Other key fish screen model assumptions are the bar width, assumed to be 1.75 millimeters (mm), or 0.069 inches (in), a porosity of 50 percent, and the Kirchner's shape factor, assumed to be 2.42, representative of rectangular-edge bar screens.

Following the fish screens is a baffle plate. For modeling purposes, a head loss of 0.5 foot (ft) across the screens and baffle plate was used. To achieve the designed head loss in the model, the baffle plate is assumed to have 4-in by 4-in openings at a 4-in vertical spacing. InfoWorks ICM is limited to circular-shaped orifices, but sluice gates apply the orifice equation under submerged conditions and can be used to model noncircular orifices to achieve an equivalent head loss. Therefore, sluice gates were used to model the baffle plates. One sluice gate represents a row of openings on the baffle plate. The sluice gate width is sized to achieve the head loss of 0.5 feet (ft) in the model between the river and the end of the baffle plate at a design flow of 3,000 cfs per intake.

All gates were modeled in the intake structure, whether controlled or not, to account for the head loss that occurs through the gate in all conditions. The control gate incorporates real-time controls to control the opening height into the box conduits. The stop-log drop gate in the middle of the box conduit is not modeled for simplification, as it is not used for normal operations or controls and would introduce negligible head loss.

A gate can be directly attached to the entrance or exit of a channel or conduit. However, in the ICM model, a node is added to connect the channel/conduit and the gate. A volume equivalent to 1 ft of the channel/conduit is added to the node, and the channel/conduit's length is reduced by 1 ft to avoid a duplicate counting of the volume.

3.1.1.2 Sedimentation Basins and Radial Gates

The sedimentation basins are modeled as storage nodes using a storage area versus elevation curve to simulate volumetric retention. Storage areas were calculated using a trapezoidal cross section with a 3:1 slope. At the end of the sedimentation basin prior to the radial gate structure is an 8-ft-wide sill (modeled as a weir) that divides the sedimentation basin from the radial gate structure. The storage node following the sill is used to account for the section of the sedimentation basin prior to the radial gate structure channel, as seen on Figure 6.

The entrance to the radial gates is modeled as sluice gates, which use orifice and weir equations for surcharged and non-surcharged conditions, respectively. The openings are 40 ft by 30 ft and 15 ft by 8 ft for the large and small radial gates, respectively. The small radial gate is intended to fine tune the flow or level, which is not currently included in the modeling scenarios. The smaller radial gate would be used in future model runs when a dynamic river level is input into the InfoWorks ICM model.

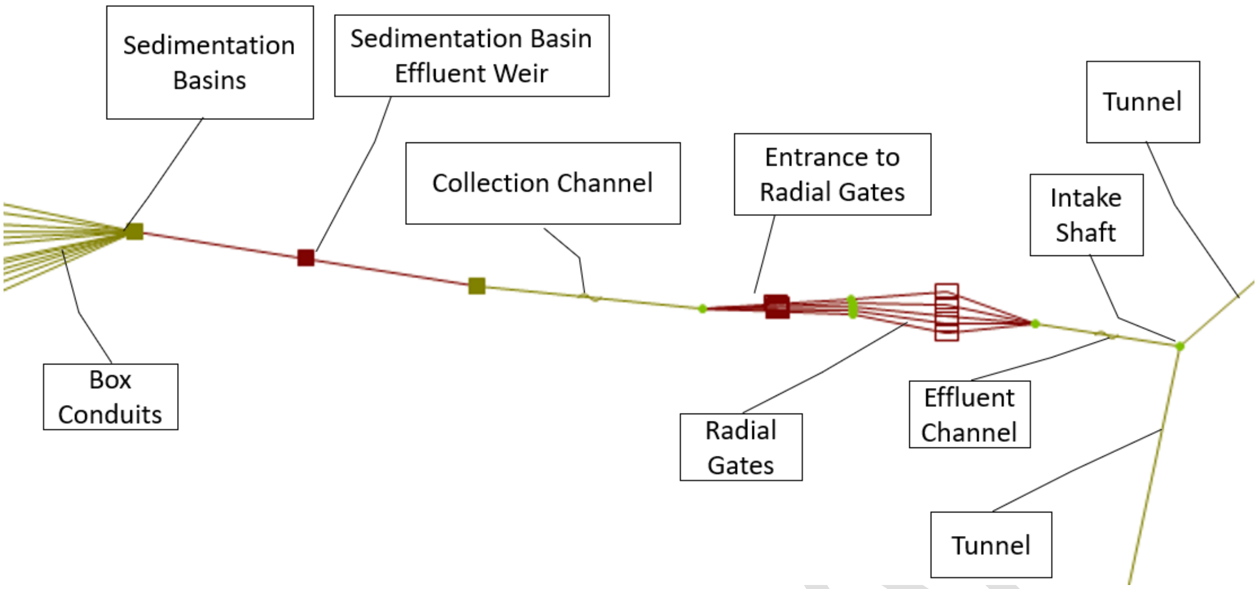

Figure 6. Model Representation of the Sedimentation Basin to Radial Gate Structure

3.1.2 Central and Eastern Tunnel Corridors

The Central and Eastern corridor were used for the analysis. The tunnel lengths and shaft locations are described in Table 2 for the Central Corridor and Table 3 for the Eastern Corridor. In Info Works ICM, dummy break nodes are required in the model when the tunnel length exceeds the maximum allowable length of 16,404 ft. The break nodes do not add additional storage or head loss to the system. The profiles of the main tunnel for a 36-ft ID tunnel are shown on Figures 7 and 8 for the Central and Eastern corridors, respectively. Profiles depict the upstream and downstream shafts shown in Tables 2 and 3 as gray vertical shafts.

Table 2. Central Corridor Tunnel Lengths

Upstream Shaft	Downstream Shaft	Length (ft)
Intake C-E-2	Intake C-E-3	11,113.3
Intake C-E-3	Intake C-E-5	13,254.0
Intake C-E-5	Twin Cities	29,332.1
Twin Cities	New Hope Tract	22,364.6
New Hope Tract	Staten Island	22,167.5
Staten Island	Bouldin Island	31,971.0
Bouldin Island	Mandeville Island	24,623.9
Mandeville Island	Bacon Island	28,480.7
Bacon Island	Byron Tract (Working Shaft)	30,401.4
Byron Tract (Working Shaft)	Southern Forebay Inlet Structure	5,069.2

Table 3. Eastern Corridor Tunnel Lengths

Upstream Shaft	Downstream Shaft	Length (ft)
Intake C-E-2	Intake C-E-3	11,113.3
Intake C-E-3	Intake C-E-5	13,254.0
Intake C-E-5	Twin Cities	29,332.1
Twin Cities	New Hope Tract Eastern	24,111.3
New Hope Tract Eastern	Canal Ranch	15,856.7
Canal Ranch	Terminus Tract	27,001.1
Terminous Tract	King Island	20,820.1
King Island	Lower Roberts Island	29,328.7
Lower Roberts Island	Upper Jones Tract	27,344.1
Upper Jones Tract	Byron Tract (Working Shaft)	29,800.5
Byron Tract (Working Shaft)	Southern Forebay Inlet Structure	5,069.2

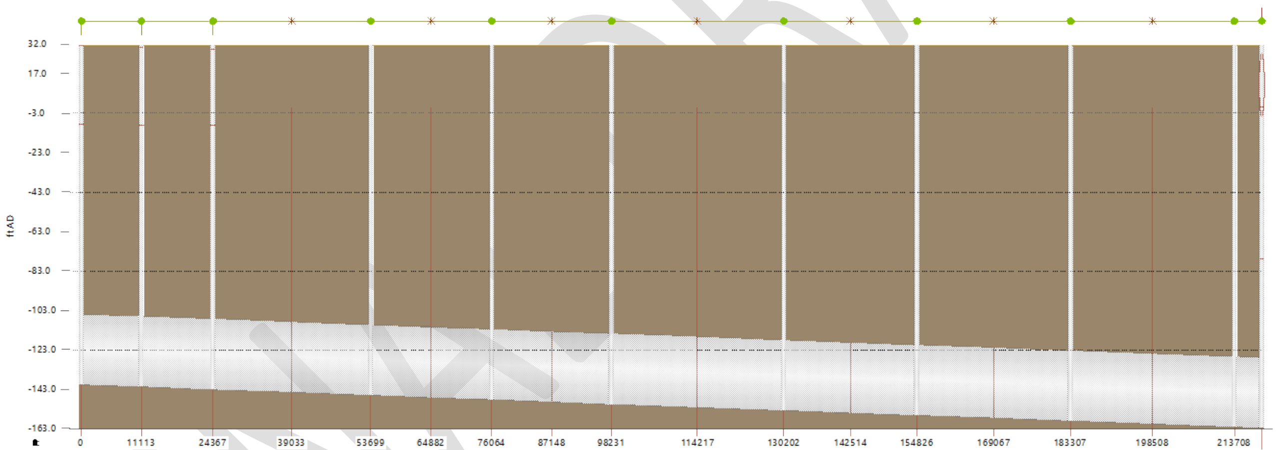

Figure 7. Modeled Tunnel Profile for Central Corridor

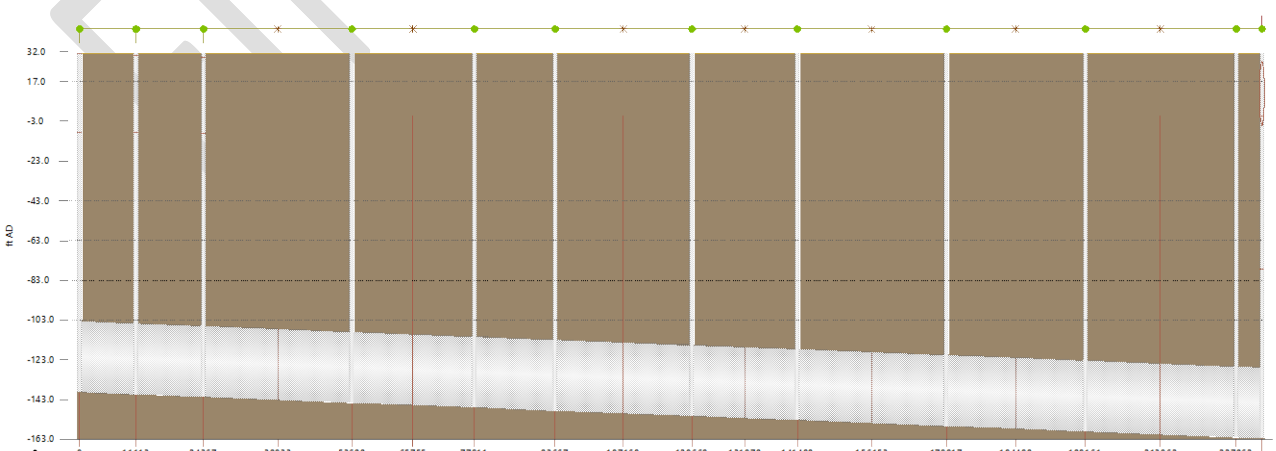

Figure 8. Modeled Tunnel Profile for Eastern Corridor

The Central and Eastern corridor models were built in the InfoWorks ICM model with a tunnel ID of 26 ft, 31 ft, 36 ft, and 40 ft. The intake outlet (drop) shafts have an ID of 83 feet. The ID of the reception and maintenance shafts are sized at 70 ft for each tunnel diameter. The Southern Forebay Inlet Structure and Southern Forebay Outlet Structure tunnel shaft ID varied per tunnel diameter and is shown in Table 4.

Table 4. Southern Forebay Shaft Diameters

Tunnel ID (ft)	Southern Forebay Inlet Structure and Southern Forebay Outlet Structure Shaft Finished ID (ft)
26	110
31	110
36	115
40	120

As shown on Figure 2, some tunnel sections have bends and approach angles relative to their connecting downstream tunnel sections, which introduce additional minor losses. These minor losses cannot be applied directly to the tunnel section in ICM. As such, they were combined and applied to the entrance or exit losses as applicable.

3.1.3 Pumping Plant and Southern Forebay

The Pumping Plant has four basic components: gravity flow and wet well inlet shaft, wet well inlet conduit, wet well, and pumps. The last shaft of the tunnel system, Southern Forebay Inlet Structure, includes the beginning of the gravity control structure. The gate entrances for the gravity control structure and wet well are modeled as sluice gates, similar to the gate structures in the intakes, which use orifice and weir equations for surcharged and non-surcharged conditions, respectively. There are three smaller-capacity pumps and six larger pumps included in the model. Each pump's performance curve was developed from a conceptual level pump selection for the Pumping Plant. The model representation of the Pumping Plant is shown on Figure 9.

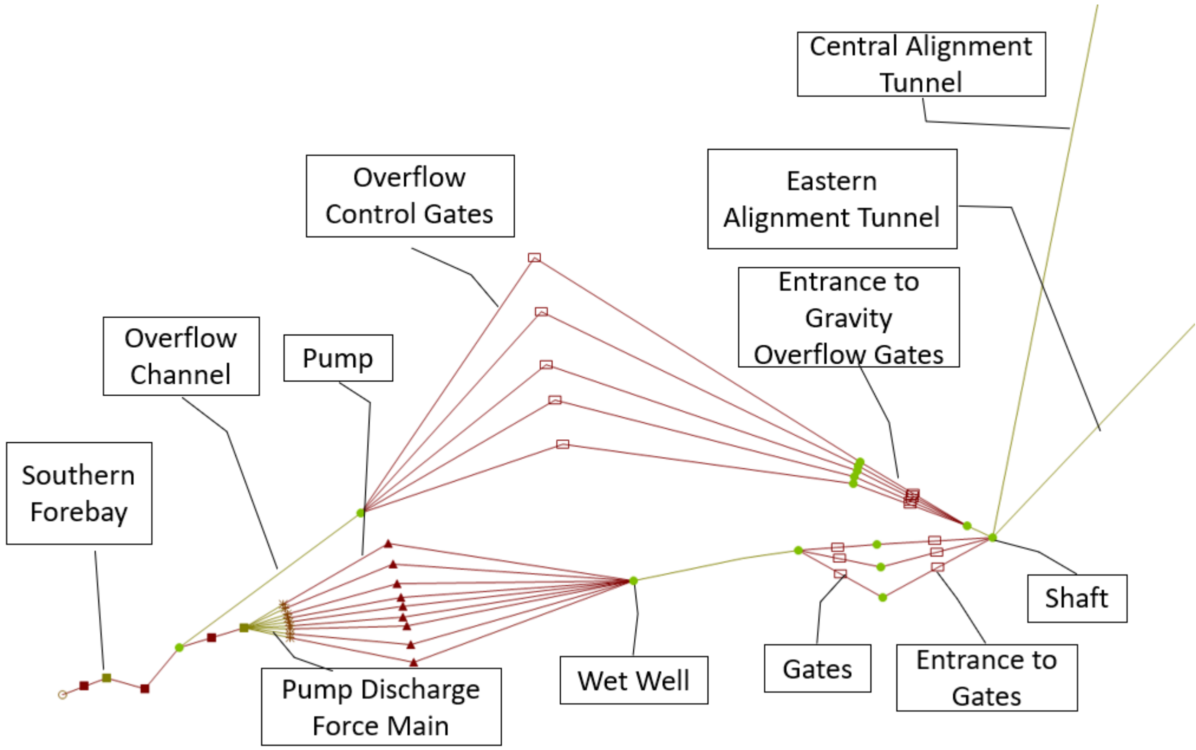

Figure 9. Model Representation of the Pumping Plant

The Southern Forebay is modeled as a storage node using the current design dimensions and a 4:1 slope for its embankments. The weir at the end of the Southern Forebay represents the outlet structure that can be controlled to specific elevations. The hydraulic analysis for the Southern Forebay and South Delta Conveyance Facilities would be the subject of a subsequent TM.

3.2 River Diversion Flow Sequences Modeled

Analysis of system head requirements and HGL was conducted for each tunnel diameter and flow scenario summarized as follows. Specific static head conditions were developed from the boundary condition WSELs defined in Section 2.2.

- 26-foot ID tunnel with a maximum flow capacity of 3,000 cfs: flow is diverted from 0 to 3,000 cfs at C-E-5 only.
- 31-foot ID tunnel with a maximum flow capacity of 4,500 cfs: flow is diverted from 0 to 3,000 cfs at intake C-E-3, and a diversion rate of 3,000 cfs is maintained by intake C-E-3. Once intake C-E-3 reaches 3,000 cfs, intake C-E-5 begins diverting flows up to 1,500 cfs until the maximum flow capacity of 4,500 cfs is achieved.
- 36-foot ID tunnel with a maximum flow capacity of 6,000 cfs: flow is diverted from 0 to 3,000 cfs at intake C-E-5, and a diversion rate of 3,000 cfs is maintained by intake C-E-5. Once intake C-E-5 reaches 3,000 cfs, intake C-E-3 begins diverting flows up to 3,000 cfs until the maximum flow capacity of 6,000 cfs is achieved.
- 40-foot ID tunnel with a maximum flow capacity of 7,500 cfs: flow is diverted from 0 to 3,000 cfs at intake C-E-5, and a diversion rate of 3,000 cfs is maintained by intake C-E-5. Once intake C-E-5 reaches 3,000 cfs, intake C-E-3 begins diverting flows up to 3,000 cfs, for a total flow of 6,000 cfs. Intakes C-E-

5 and C-E-3 maintain diversion rates of 3,000 cfs each, and intake C-E-2 begins diverting flows up to 1,500 cfs until the maximum flow capacity of 7,500 cfs is achieved.

3.3 Real-Time Controls

3.3.1 Controllable Devices

In the InfoWorks ICM model, real-time control (RTC) rules were developed to mimic the prospective operations of the controlled components, including intake control gates, intake radial gates, variable speed pumps at the Pumping Plant, and the control gates in the gravity flow/surge overflow structure.

Intake control gates are controlled by adjusting their opening heights from 0 to 8 ft. Their typical controlled states are as follows:

- Fully open at a maximum opening height of 8 ft.
- Fully closed at an opening height of 0 ft.
- Open gradually so the flow increases at a rate of 1,000 cfs/15 minutes for the intake
- Close gradually so the flow decreases at a rate of 1,000 cfs/15 minutes for the intake
- Slightly adjust the opening height so the flow sets at the targeted operation flow

Intake radial gates are controlled by adjusting the angles between the gate chord and the channel bottom. Their controlled states are as follows:

- An angle of 76.1 degrees represents a fully closed position when the radial gate touches the bottom of the channel.
- The radial gate is deemed fully open if its opening is above the maximum water surface through the gate.
- An opening of 115.3 degrees represents a vertical opening at its pivot height.
- Increase the angle gradually while the control gates are opening.
- Decrease the angle gradually while the control gates are closing. The radial gates fully close at the same time as the control gates.
- Slightly adjust the angle so the flow sets at the targeted normal operation.
- Slightly adjust the angle so the water level upstream of the radial gate does not fluctuate dramatically.
- Control the water level upstream of the radial gate to not more than 1.5 ft below the river WSEL.

Gravity flow control gates are controlled by adjusting their opening heights. Their controlled states are:

- Fully open with an opening height above the maximum water surface through the gate.
- Fully closed with an opening height of 0 ft.

Pumps are controlled by adjusting their operating speed (revolutions per minute [RPM]):

- Active.
- Inactive.
- Increase the RPM gradually so the total flow increases at a rate of 1,000 cfs/15 minutes.
- Decrease the RPM gradually so the flow decreases at a rate of 1,000 cfs/15 minutes.

- Slightly adjust the RPM so the flow sets at the targeted normal operation flow.
- The pumps must operate between the minimum and maximum RPM.
- Slightly adjust the RPM if the operating point is to the left side of the minimum preferred operating range (POR) line.
- Slightly adjust the RPM if the operating point is to the right side of the maximum POR line.
- The pumps can also be controlled by individual pump on/off levels.

Controlled pump operation examples for large and small pumps are illustrated on Figures 10 and 11, respectively.

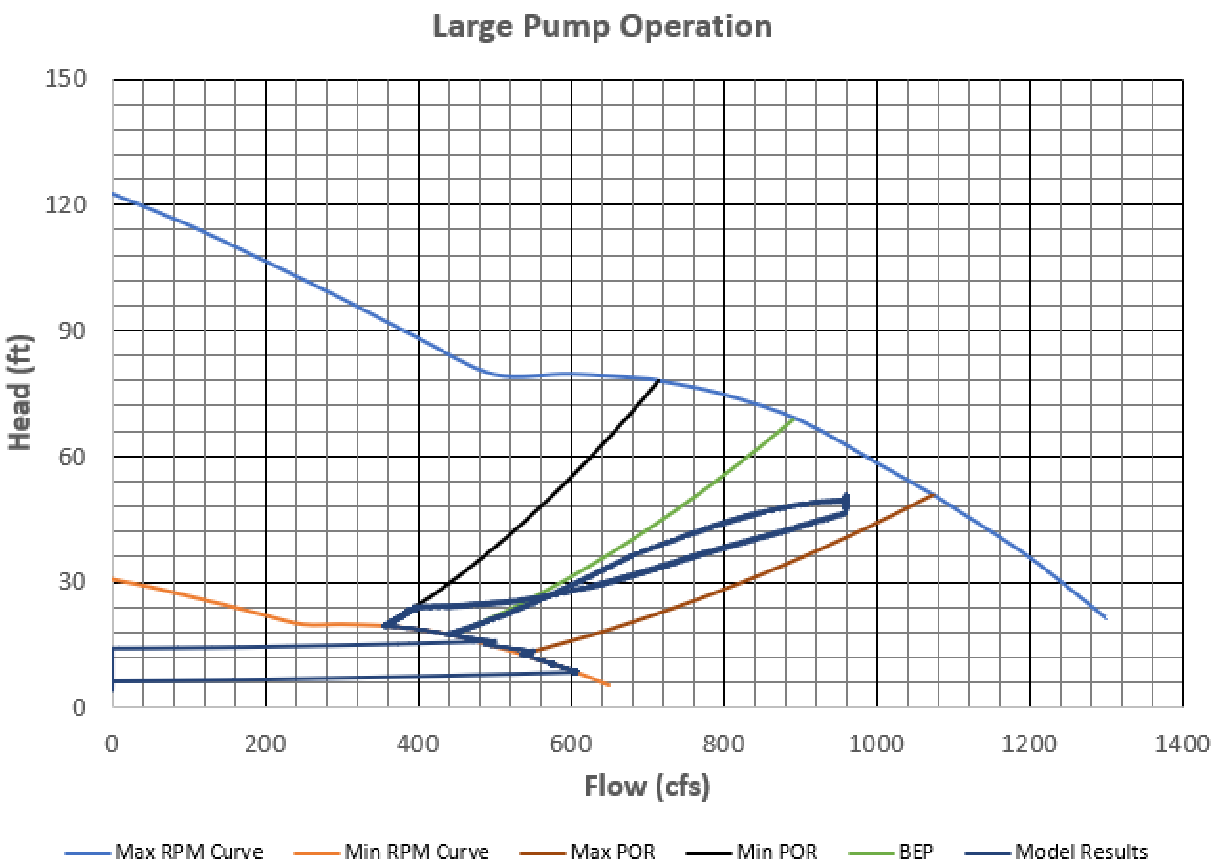

Figure 10. Large Pump Operation Scatter Plot

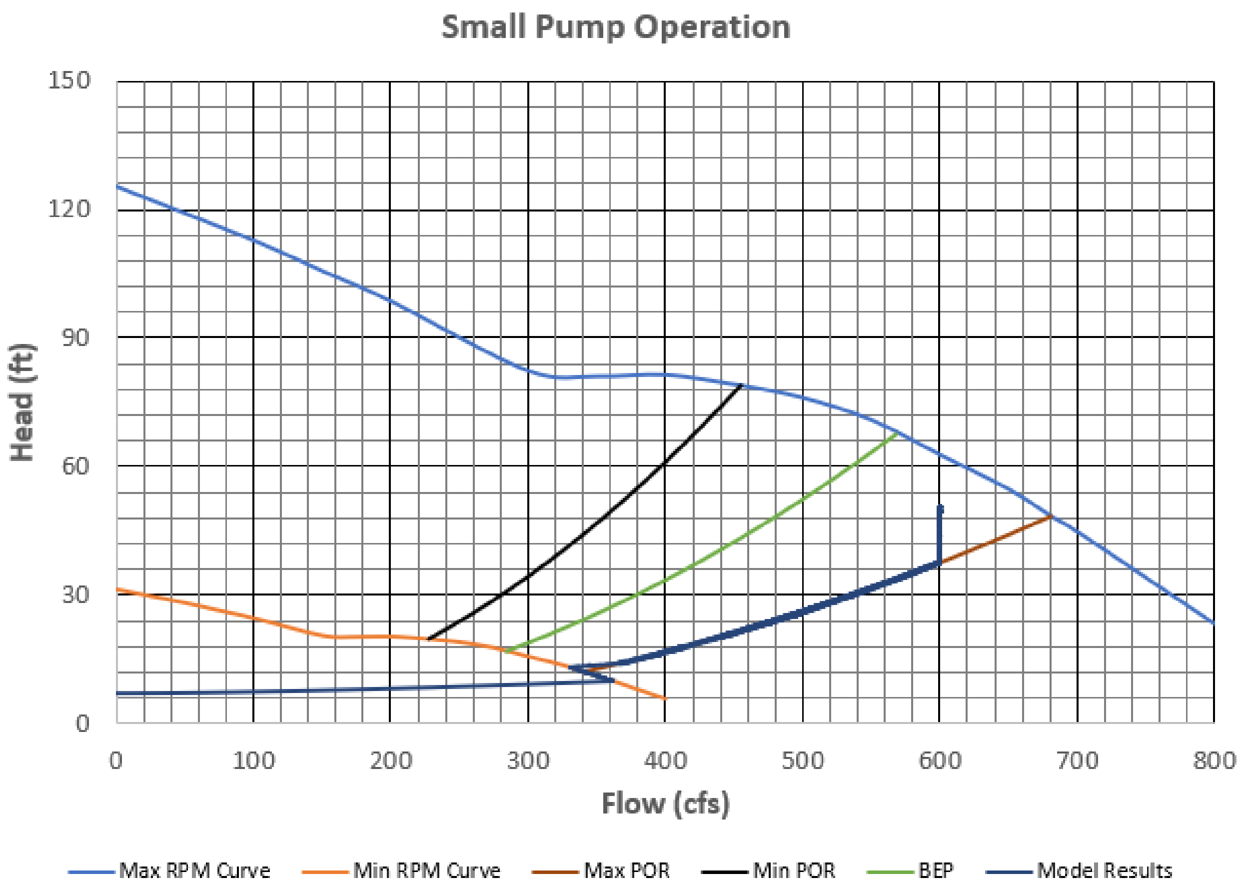

Figure 11. Small Pump Operation Scatter Plot

3.3.2 RTC Control Rules

3.3.2.1 Initialize the Tunnel to the Preoperational State

In the model, all the tunnels, shafts, channels, and basins are empty at the start of the simulation. It does not represent the actual preoperational state of the tunnel. Therefore, it is necessary to fill the tunnel to a predetermined water level to match the preoperational state for a specific model simulation. Inflow time series are loaded to the tunnel to fill it. RTC rules are used to confirm the water level stabilizes at a specific water level, and the correct river WSELs are loaded to the intakes.

3.3.2.2 Simulated System Startup Operation

For this scenario, the RTC controls represent the operation state of a device, time to operate, opening speed, interaction with other devices, and interaction with metering devices. The RTC rules enable all idle devices to be turned off or closed throughout the simulation. They also reset the devices' states from the initialization state to the preoperational state.

For the intakes, the controlling criterion is a flow increase rate of 1,000 cfs per 15 minutes, which is controlled by the intake control gates. A flow-time curve is developed with a flow increase rate of 1,000 cfs per 15 minutes. At each sampling time step, the RTC compares the actual flow through a gate with the flow-time curve. For example, if the actual flow is higher than the curve, the gate decreases its opening. The radial gates also have similar control rules to open gradually. The control gate is controlled by adjusting its vertical opening height, while the radial gate is controlled by adjusting its angle between the gate chord and the channel bottom.

The controlled flows of control gates at intakes C-E-3 and C-E-5 are illustrated on Figure 12 for a startup operation. It should be noted that a flow rate of 250 cfs through one control gate represents a total flow of 3,000 cfs for one intake with 12 control gates. In this plot, intake C-E-3 opens from 0 to 45 minutes to reach 3,000 cfs, and intake C-E-5 opens from 45 to 90 minutes reach another 3,000 cfs for a total system flow of 6,000 cfs at 90 minutes.

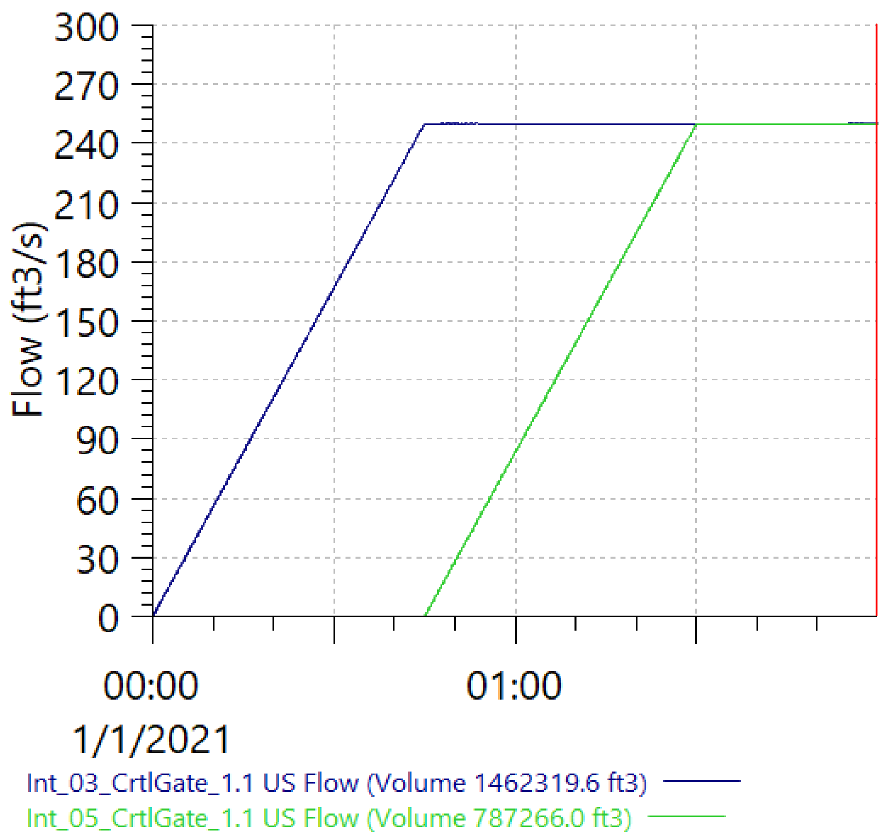

Figure 12. Intake Control Gate Startup Operation

The pumps are always controlled by four parameters: minimum revolutions per minute (RPM) line, maximum RPM line, minimum POR line, and maximum POR line. The optimal operational region is bounded by the four lines. At each sampling time, the pumps' flow-total-dynamic-head operational state is compared with the four lines. If the operational state is outside the optimal operational region, the RPM is increased or decreased until it falls inside the region.

In order to synchronize the operation between the intakes and the Pumping Plant, the pumping flow is also limited to a flow increase rate of 1,000 cfs within 15 minutes. A flow-time curve is developed with a flow rate increase of 1,000 cfs/15 minutes. At each sampling time step, the RTC compares the actual pumping flow with the flow-time curve. If the actual flow is higher than the curve, the pump decreases its RPM or maintains the minimum RPM, and vice visa. Five large pumps with a design flow of 960 cfs and 2 small pumps with a design flow of 600 cfs are utilized to achieve the 6,000 cfs design capacity.

At the beginning of the startup operation, only the lead pump (one of the smaller-capacity pumps) is started. The pump kick-on time is 60 seconds. The initial kick-on flow rate could be higher than the flow-time curve as the pump's speed increases from 0 RPM to its defined minimum RPM (within the POR), which is not controllable in the model. After 60 seconds, the pump is maintained at its minimum RPM, and flow moves closer to the flow-time curve. Eventually, the pumping flow becomes less than the flow-time curve, indicating the lead pump can no longer produce the pumping capacity required by the flow-time curve. The lag pump (the second smaller-capacity pump) is then started. After 60 seconds, both pumps would be adjusted to operate at identical speeds (minimum permissible speed within their POR).

Similarly, when both smaller-capacity pumps cannot keep up with the flow-time curve, the lag-lag pump (the first larger-capacity pump) is started. When both smaller-capacity pumps and the one higher-capacity pump are in operation, their speeds are controlled by their own optimal operational ranges. At first, all pumps operate at their minimum speed during the startup process until all operational pumps are on. Because the pump speeds are limited by their minimum permissible speed, the total flow is unable to match the linear flow-time curve exactly but follows the general trend. Eventually, the total pumping flow can keep up with the flow-time curve as larger-capacity pumps are in operation, and each pump's operating speed would continue increasing until the total pumped flow reaches the set-point total flow. It should be noted that the number of pumps on duty varies with the targeted design flows. The total controlled pumping flow is illustrated on Figure 13 for the startup operation.

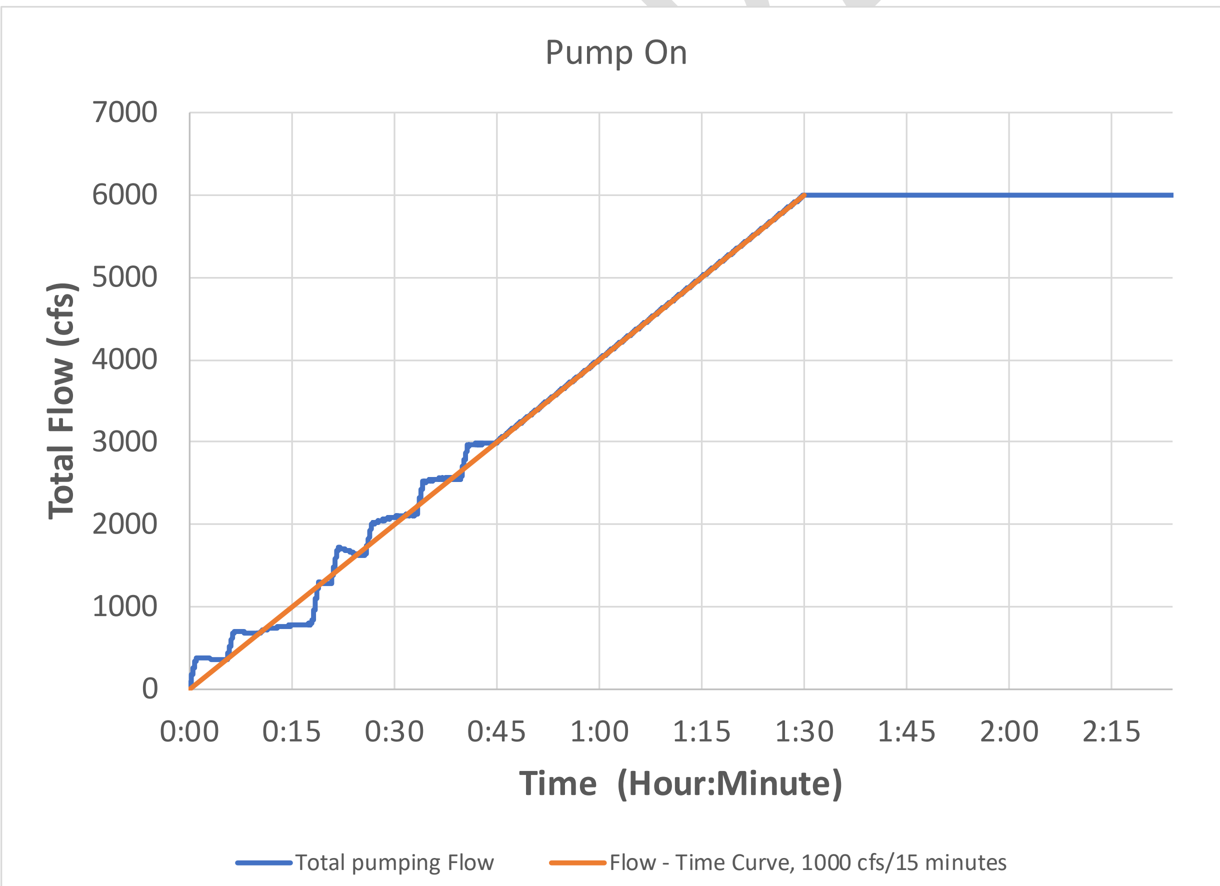

Figure 13. Real-Time Control on Pump Startup Operation

3.3.2.3 Simulated System Operation

During simulated system operation, the RTC controls the gates and pumps so the total system flow is maintained at its targeted flow. For the intake control gates, the flow through a gate is compared with its targeted normal flow rate at each sampling time, and the gate opening is adjusted if the flows are different. The radial gates also have similar controls. For the radial gates, the upstream water level (upstream of the gate) is compared with the river WSEL. If the differential level deviates between the river WSEL and the WSEL upstream of the radial gates, the gates are adjusted to achieve the required set-point. The controls are implemented so that the level differences do not fluctuate dramatically. The pumps compare flows with the targeted flows and adjust the RPM as necessary.

3.3.2.4 Simulated System Shutdown Operation

For the intakes, the controlling criterion is a flow decrease rate of 1,000 cfs per 15 minutes, controlled by the intake control sluice gates. A flow-time curve is developed with a flow decrease rate of 1,000 cfs/ 15 minutes. At each sampling time step, the RTC compares the actual flow through a gate with the flow-time curve. If the actual flow is higher, the gate decreases its opening. The radial gates and control gates fully close at the same time.

The controlled flows for intakes C-E-3 and C-E-5 are illustrated on Figure 14 for shutdown operation. It should be noted that a flow rate of 250 cfs through one control gate represents a total flow of 3,000 cfs for one intake with 12 control gates. In this plot, intake C-E-5 closes from 0 to 45 minutes to reach 0 cfs from 3,000 cfs linearly, while intake C-E-3 closes from 45 to 90 minutes to reach 0 cfs from 3,000 cfs linearly.

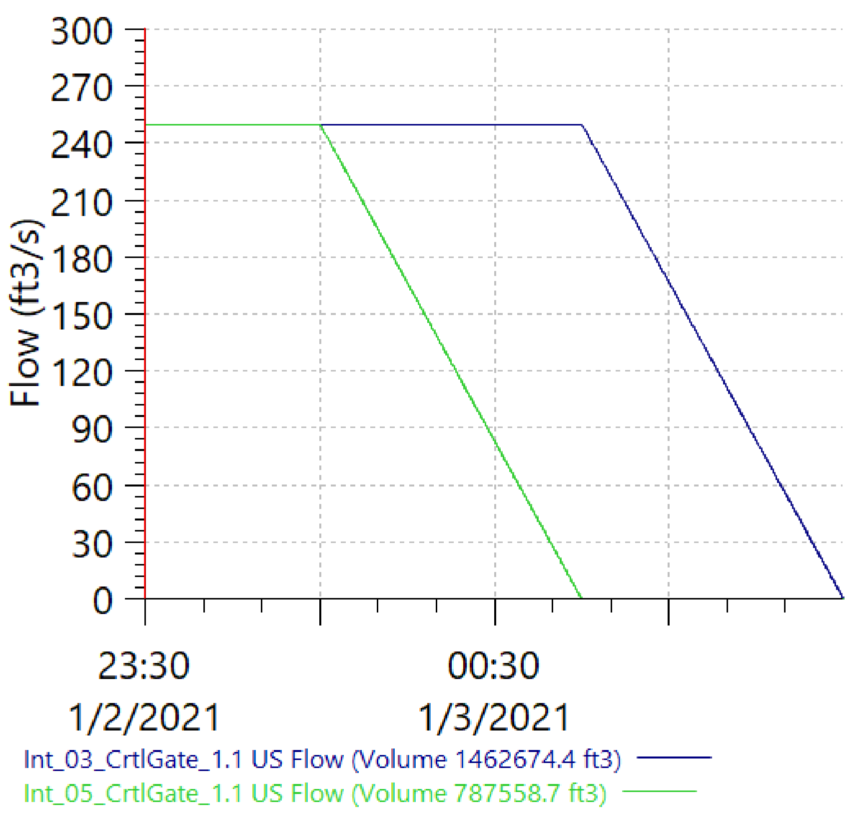

Figure 14. Intake Control Gate Shutdown Operation

In order to synchronize the operation between the intake and the Pumping Plant, the total pumping flow is also limited at a flow decrease rate of 1,000 cfs/15 minutes. A flow-time curve is developed with a flow decrease rate of 1,000 cfs/15 minutes. At each sampling time step, the RTC compares the actual pumping flow with the flow-time curve. If the actual flow is higher than the curve, the pump decreases its RPM or is maintained at its minimum RPM.

All pumps decrease their rotational speeds gradually, so the total pumping flow matches the flow-time curve. Eventually, all the pumps reach their minimum permissible speeds (within their POR), and the total pumping flow is more than the flow-time curve. The control rule is triggered to shut down one higher capacity pump. The pump shutdown time is 60 seconds. The RTCs are set to stop all higher pumps before stopping the smaller-capacity pumps. Because the pump speeds are limited by their allowable RPM (to maintain operation within their respective POR), the total flow cannot exactly match the flow-time curve but follows the general trend. The total controlled pumping flow is illustrated on Figure 15 for the shutdown operation.

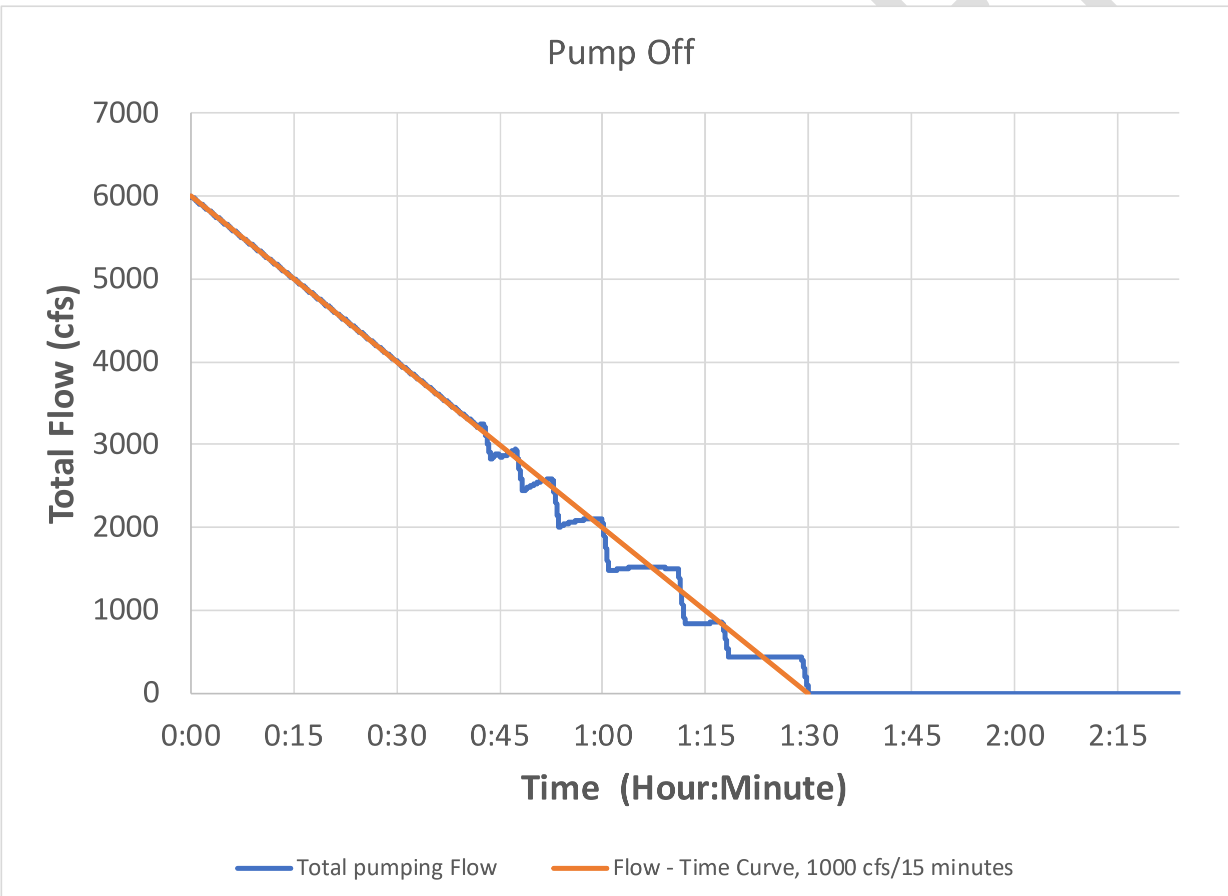

Figure 15. Real-Time Control on Pump Shutdown Operation

3.3.3 Operational Cycle Study

3.3.3.1 Model Setup

This study includes a combination of several RTC control scenarios under static boundary conditions, including preoperational initialization, startup operation for 90 minutes, stabilized normal operation for an extended period, and shutdown operation for 90 minutes. All scenarios are incorporated in one model simulation. The evaluations are performed for both the Central and Eastern corridors.

3.3.3.2 Model Simulation Results

The individual pump startup and the corresponding wet well water level variations are presented on Figures 16 and 17 for Central and Eastern corridors, respectively. The plots show a smooth transition between system startup and designated steady-state operating condition. Results were consistent for each tunnel corridor.

The individual pump shutdown and the corresponding wet well water level variations are presented on Figures 18 and 19 for the Central and Eastern corridors, respectively. The plots show the system shutdown operation at the Pumping Plant also has smooth transitions, which is minimally impacted by the tunnel corridors. After all pumps are shut down, the wet well water level does not stabilize immediately, but its water surface experiences slight oscillations for an extended period of time.

The tunnel shafts' water level variations during the startup operation is presented on Figures 20 and 21 for the Central and Eastern corridors, respectively. The plots show the system startup operation has smooth transitions in these shafts, which is minimally impacted by the tunnel corridors.

The tunnel shafts' water level variations during the shutdown operation is presented on Figures 22 and 23 for the Central and Eastern corridors, respectively. The plots show the system shutdown operation has smooth transitions in these shafts, which is minimally impacted by the tunnel corridors. After the system is shutdown, the water surfaces do not stabilize immediately but oscillate for an extended period of time. The extent of the water surface variations does not cause any concerns hydraulically.

The intake C-E-3 flow and water level variations during the startup operation is presented on Figures 24 and 25 for the Central and Eastern corridors, respectively. The plots show the diversion capacity set-point was achieved based on the diversion rate.

The intake C-E-3 flow and water level variations during the shutdown operation are presented on Figures 26 and 27 for the Central and Eastern corridors, respectively. The plots show the system shutdown operation has smooth transitions, which is minimally impacted by the tunnel corridors.

The intake C-E-5 flow and water level variations during the startup and shutdown operations are presented on Figures 28 through 31. C-E-5 flow is minimally impacted by the tunnel corridors.

The minimum and maximum HGL profiles along the tunnel are presented on Figure 32 and 33 for the Central and Eastern corridors, respectively.

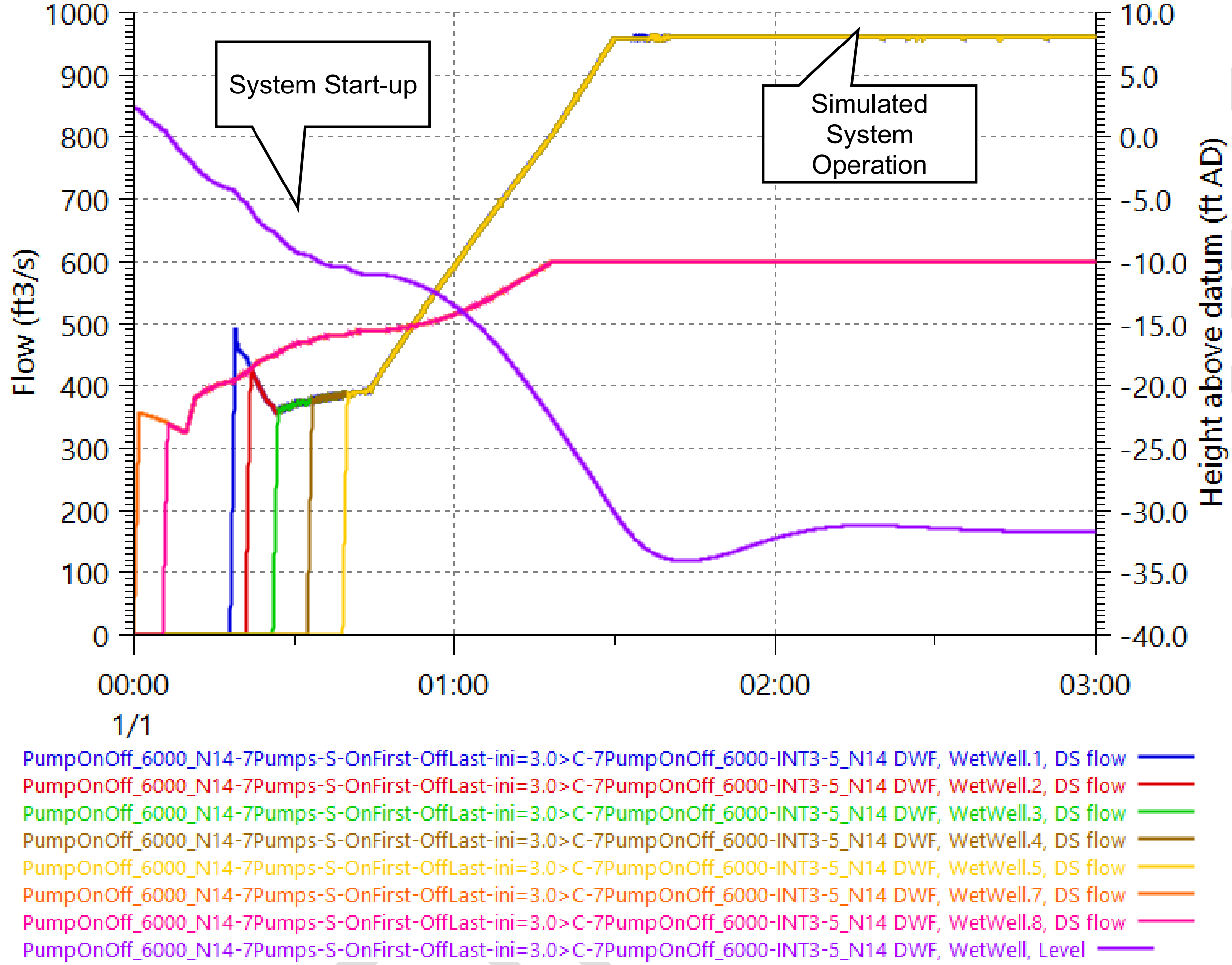

Figure 16. Pump Startup and Wet Well Water Level, Central Corridor, 6,000 cfs

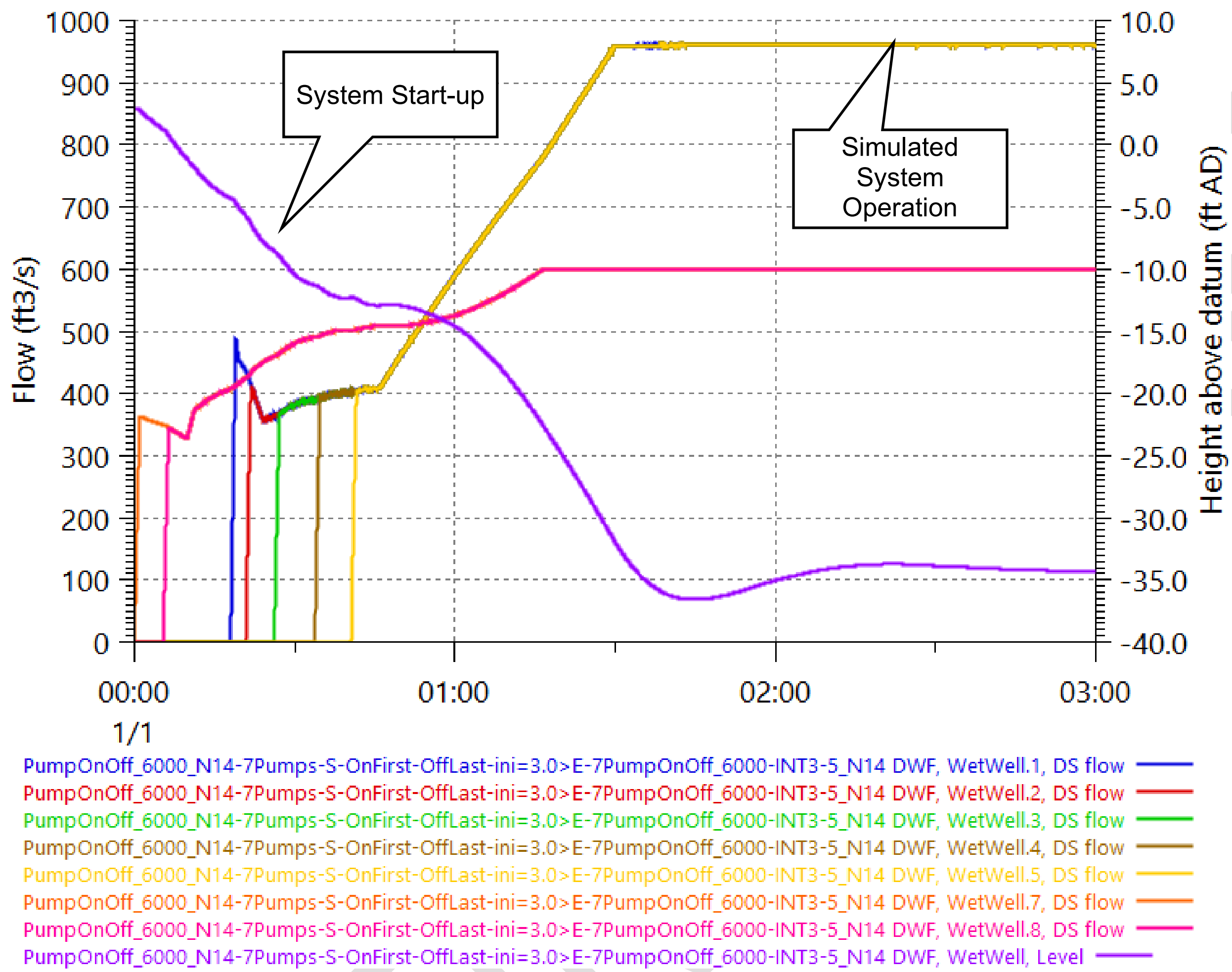

Figure 17. Pump Startup and Wet Well Water Level, Eastern Corridor, 6,000 cfs

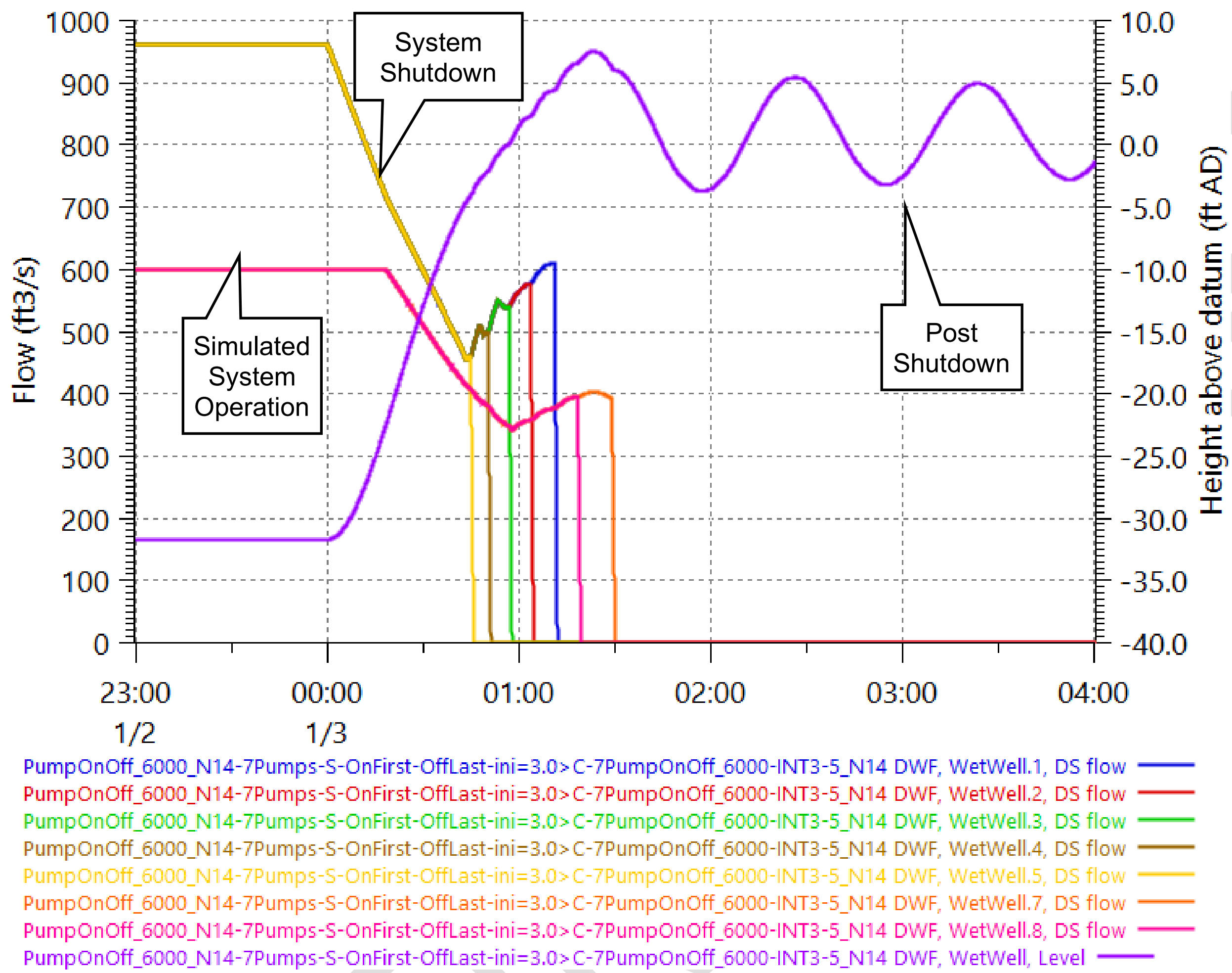

Figure 18. Pump Shutdown and Wet Well Water Level, Central Corridor, 6,000 cfs

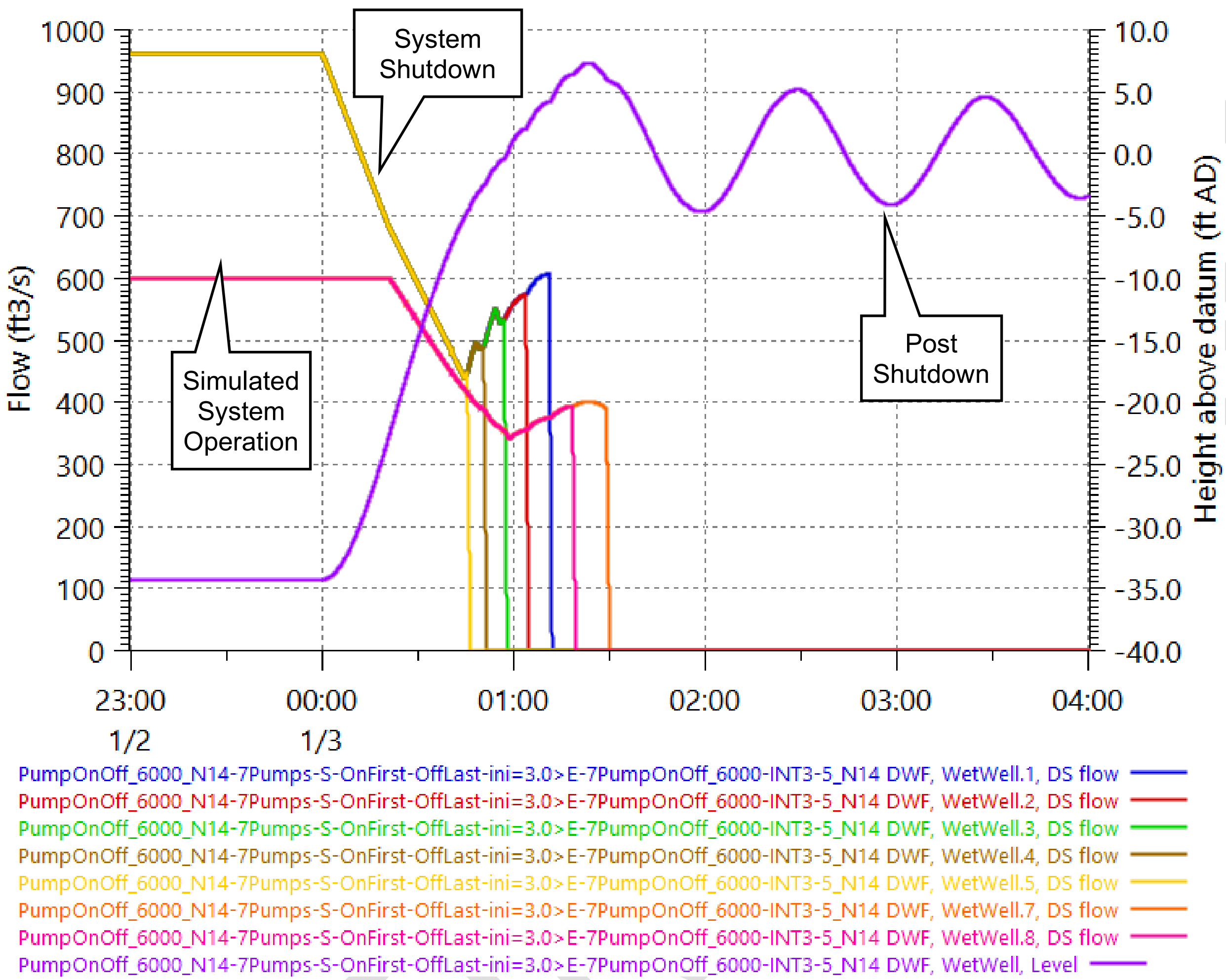

Figure 19. Pump Shutdown and Wet Well Water Level, Eastern Corridor, 6,000 cfs

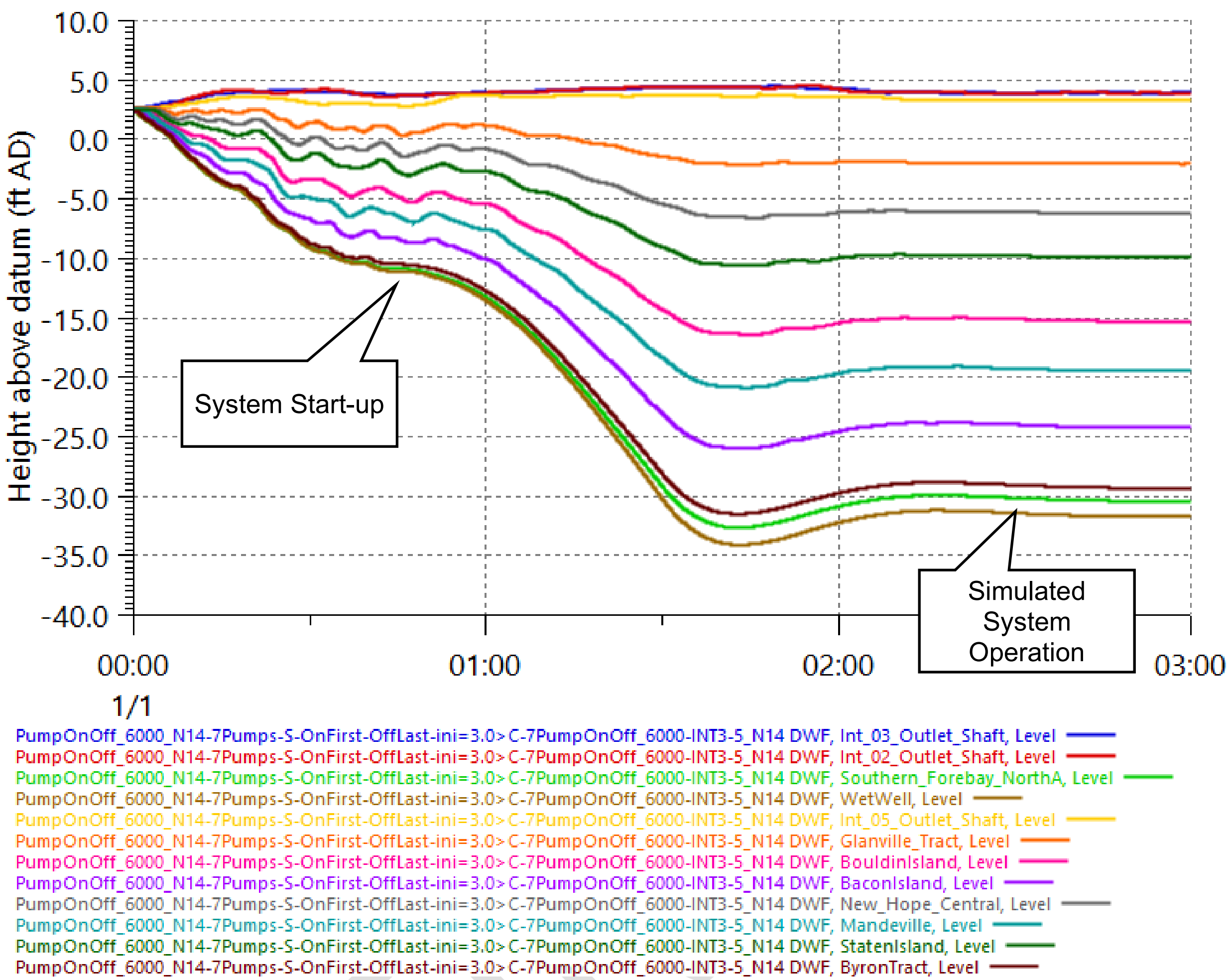

Figure 20. Tunnel Shaft Water Levels during System Startup Operation, Central Corridor, 6,000 cfs

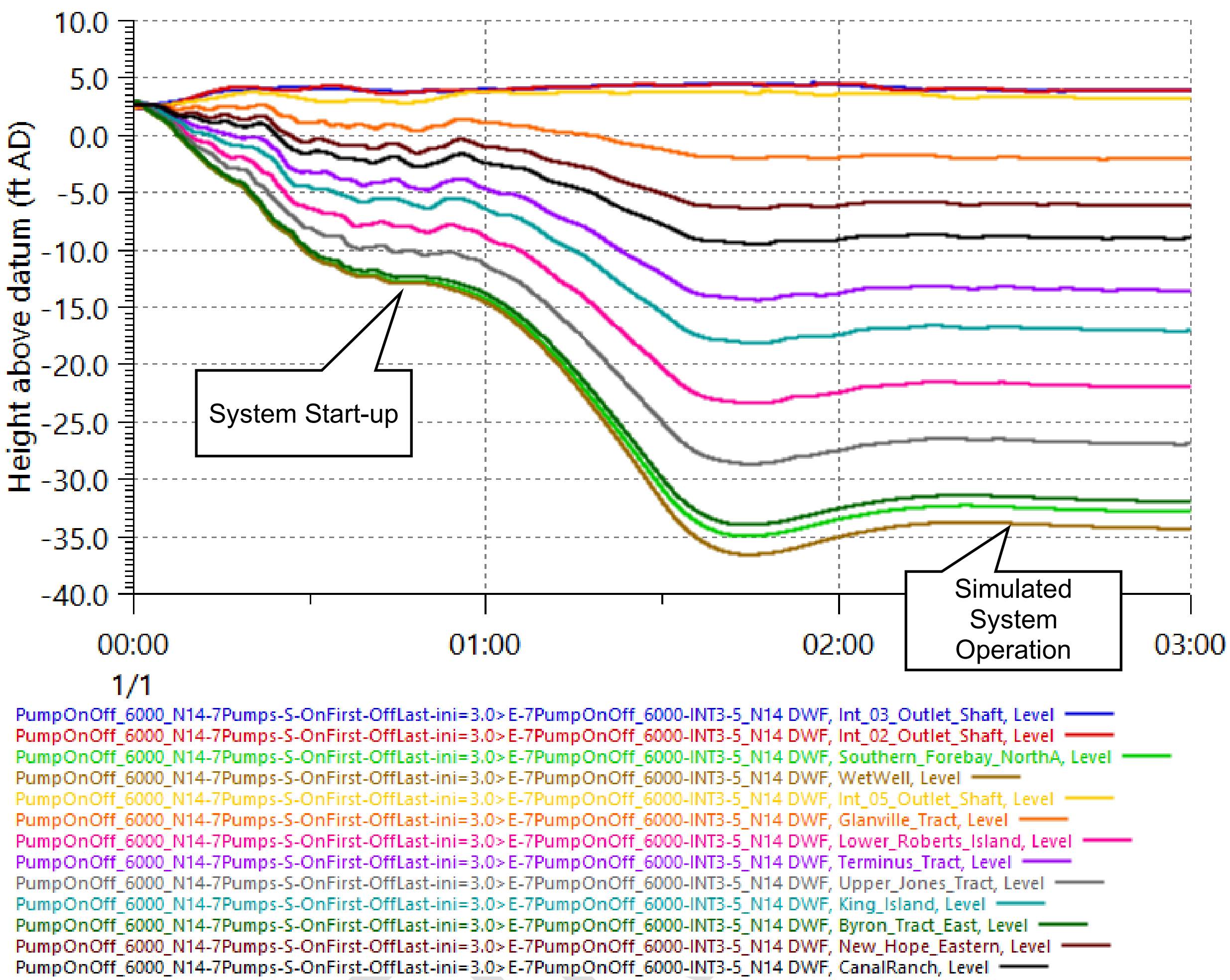

Figure 21. Tunnel Shaft Water Levels during System Startup Operation, Eastern Corridor, 6,000 cfs

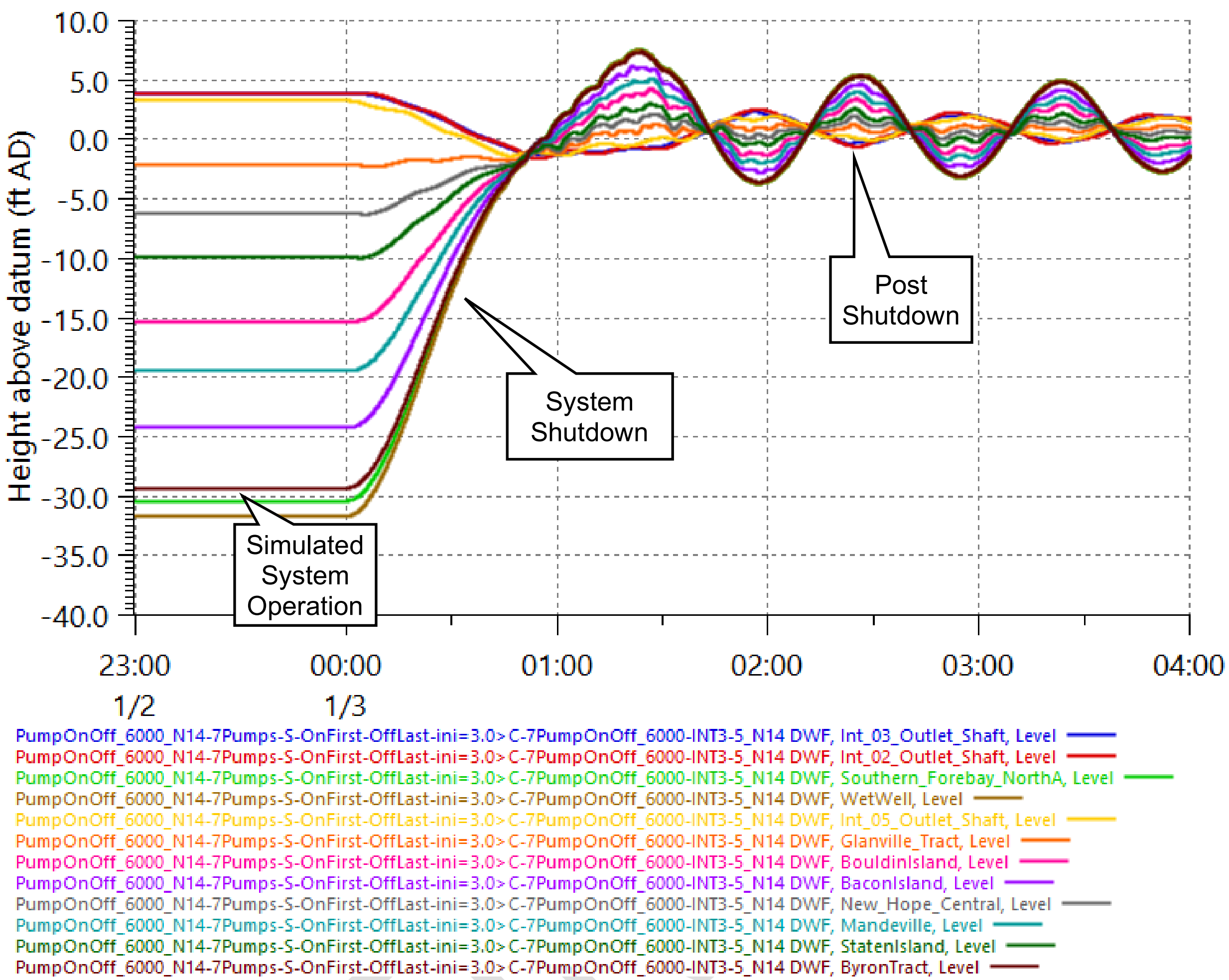

Figure 22. Tunnel Shaft Water Levels during System Shutdown Operation, Central Corridor, 6,000 cfs

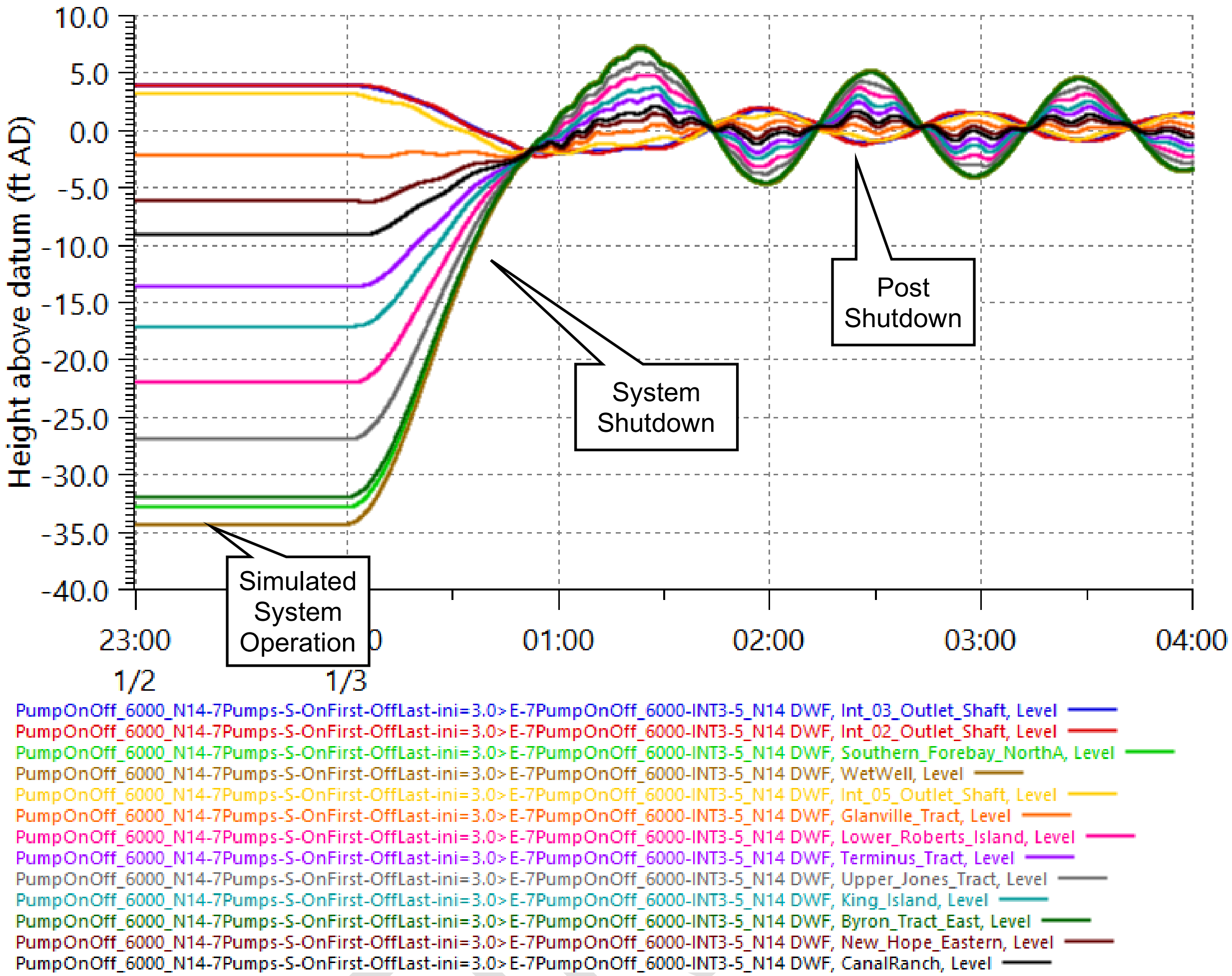

Figure 23. Tunnel Shaft Water Levels during System Shutdown Operation, Eastern Corridor, 6,000 cfs

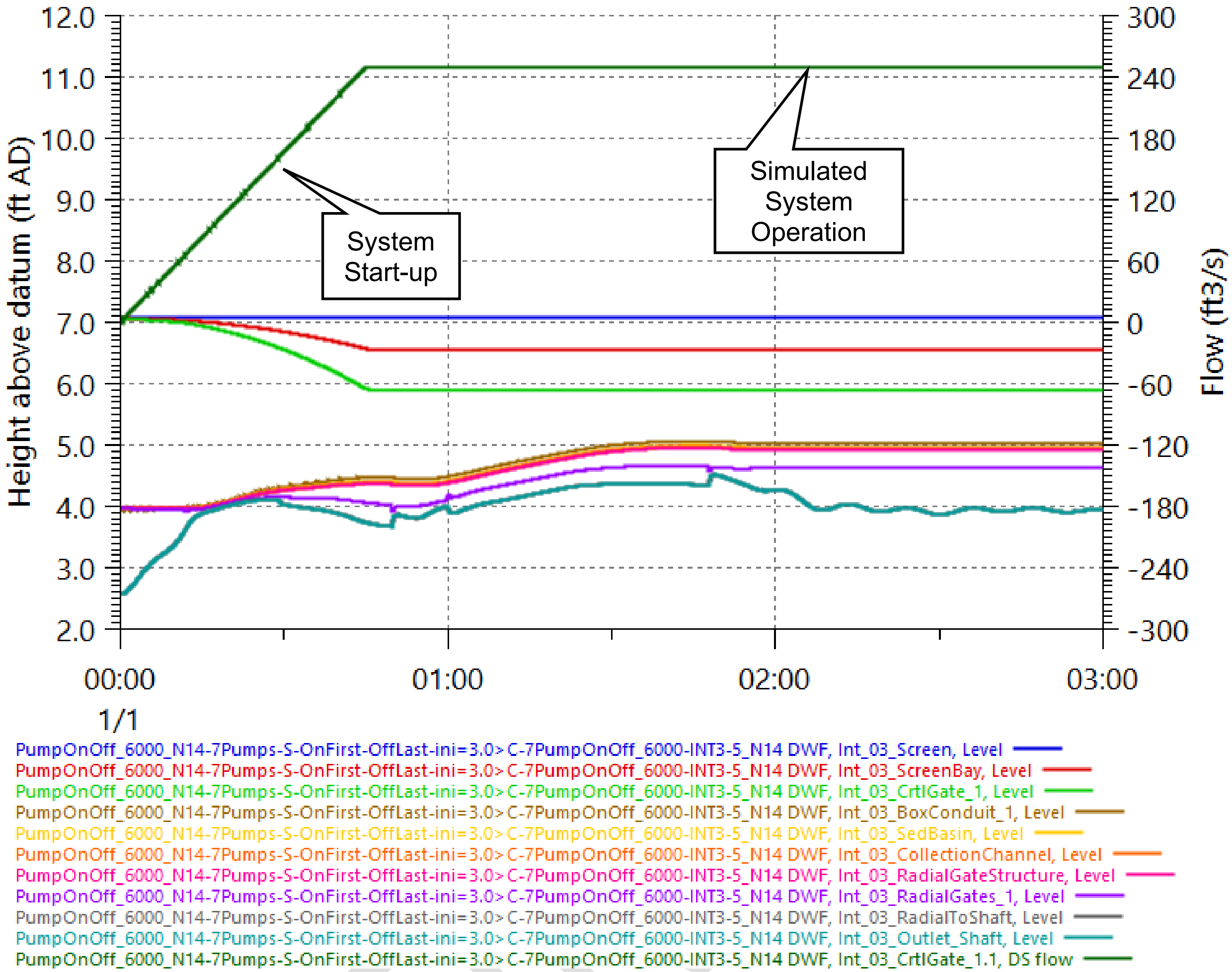

Figure 24. Intake C-E-3 Flow and Water Levels during System Startup Operation, Central Corridor, 6,000 cfs

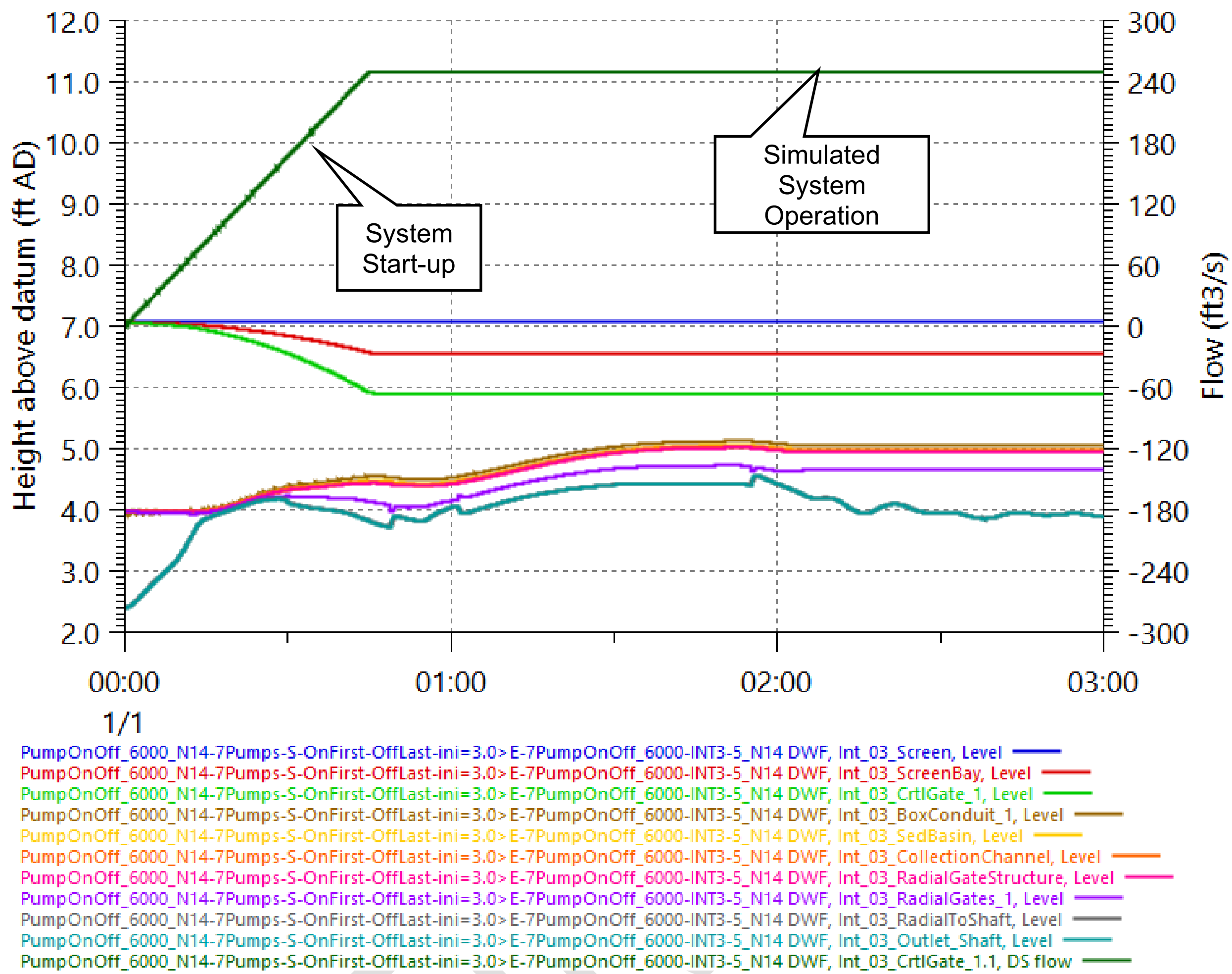

Figure 25. Intake C-E-3 Flow and Water Levels during System Startup Operation, Eastern Corridor, 6,000 cfs

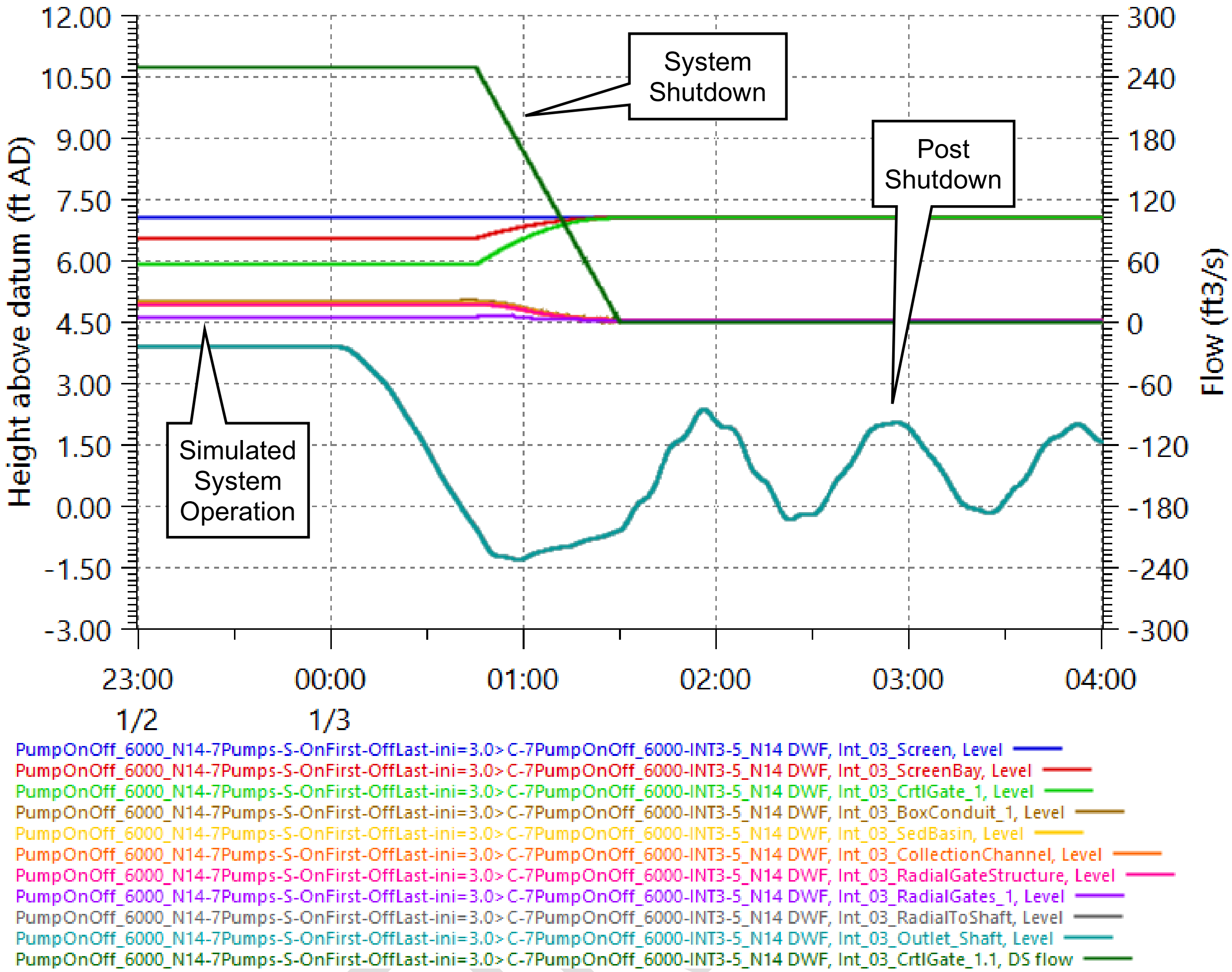

Figure 26. Intake C-E-3 Flow and Water Levels during System Shutdown Operation, Central Corridor, 6,000 cfs

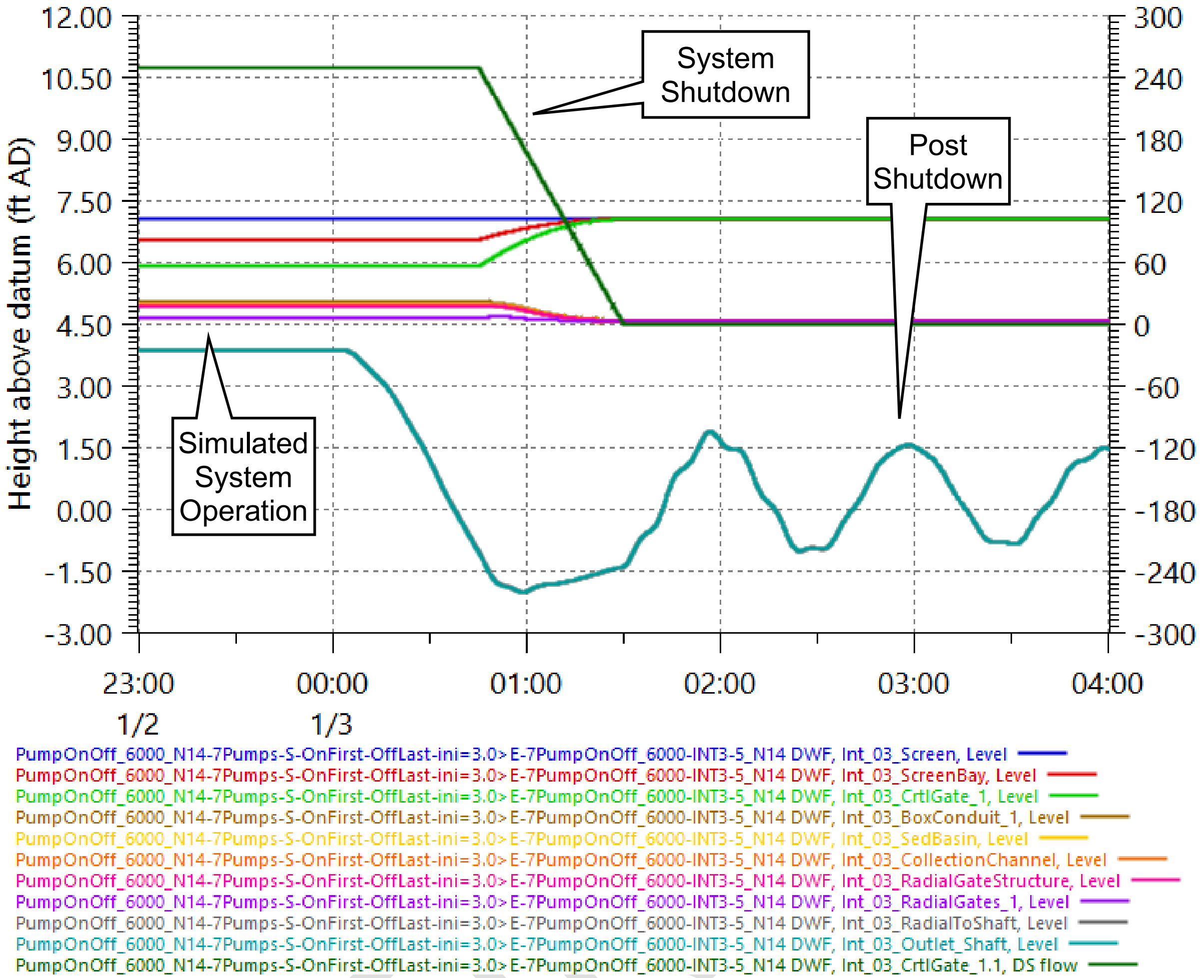

Figure 27. Intake C-E-3 Flow and Water Levels during System Shutdown Operation, Eastern Corridor, 6,000 cfs

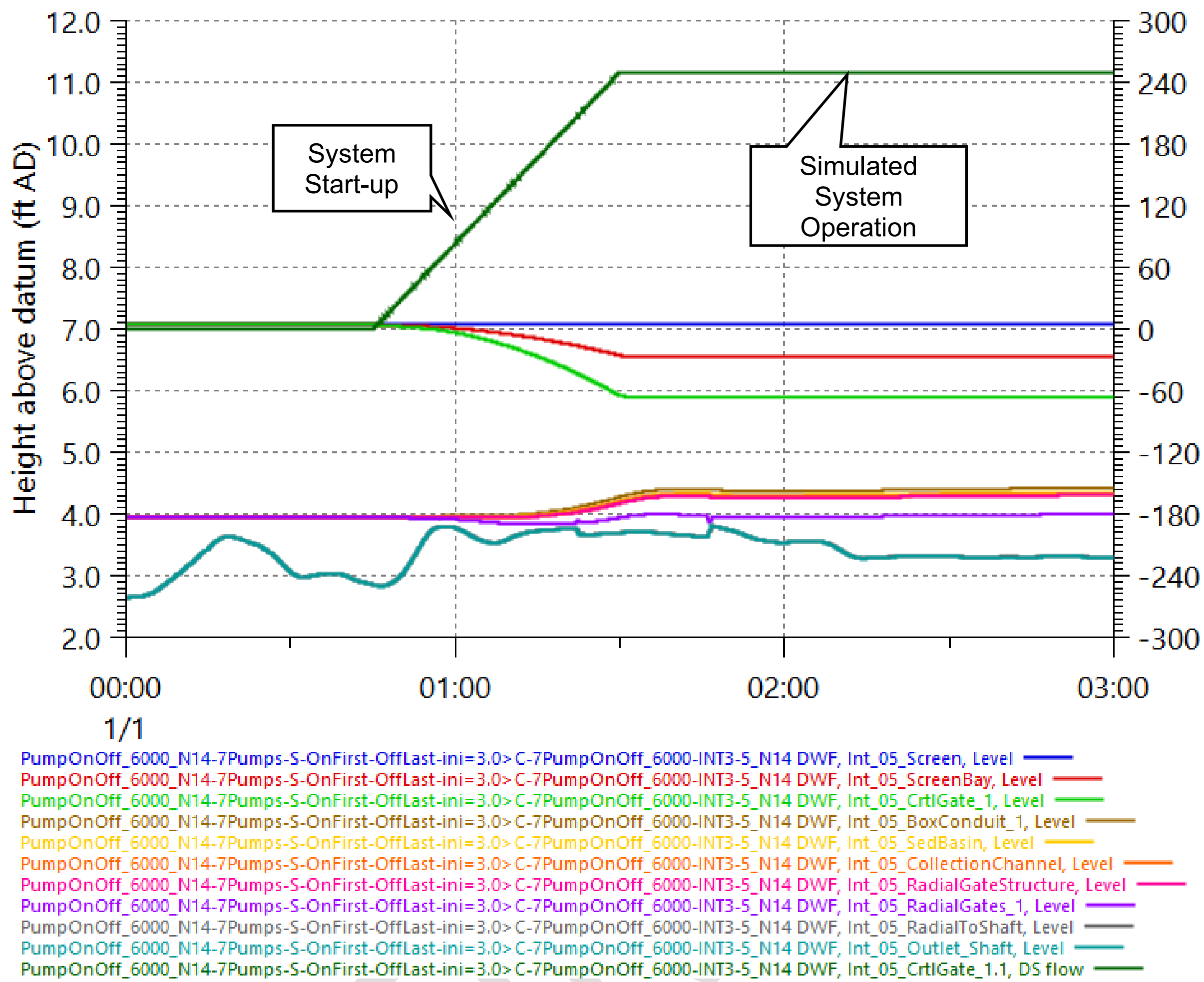

Figure 28. Intake C-E-5 Flow and Water Levels during System Startup Operation, Central Corridor, 6,000 cfs

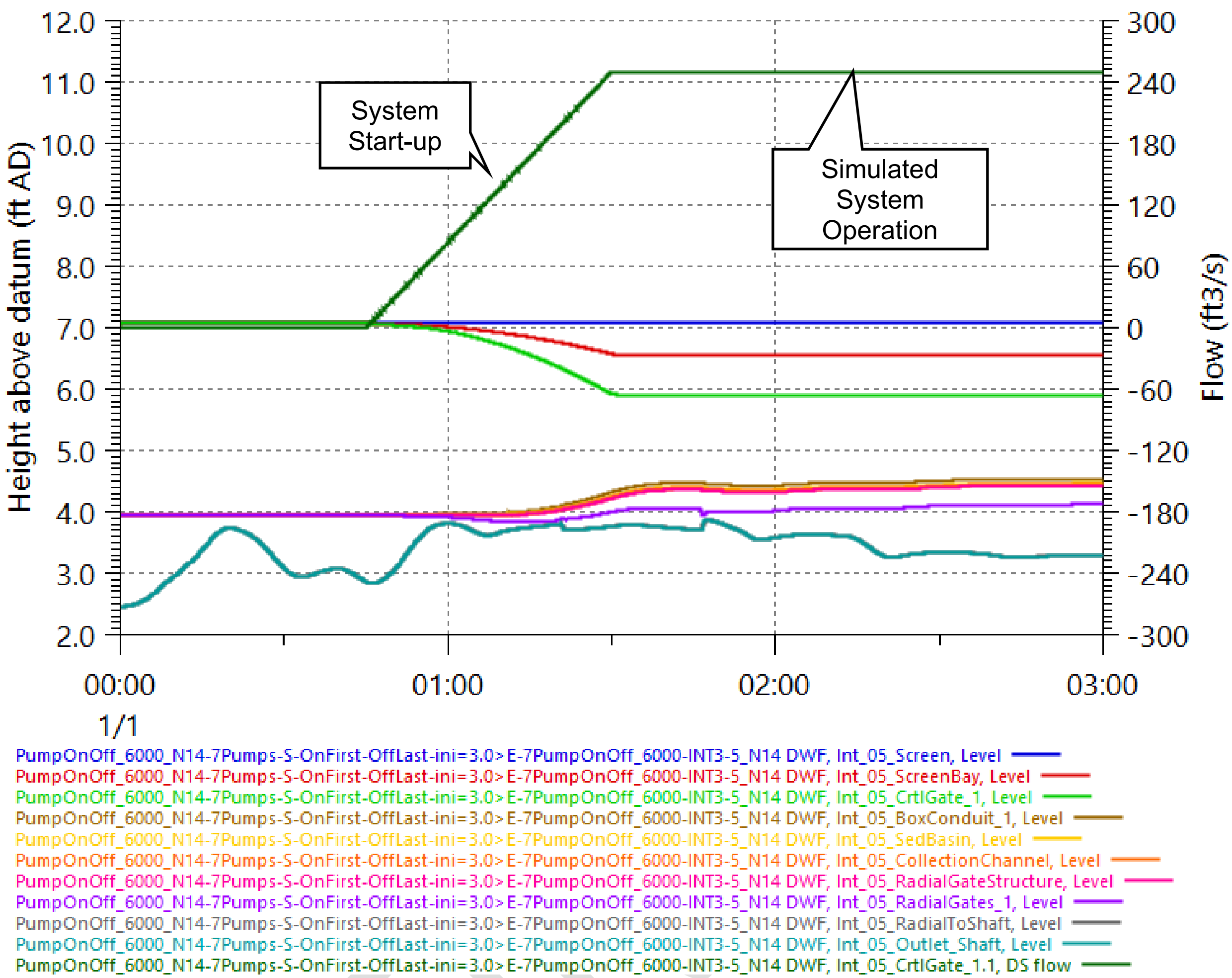

Figure 29. Intake C-E-5 Flow and Water Levels during System Startup Operation, Eastern Corridor, 6,000 cfs

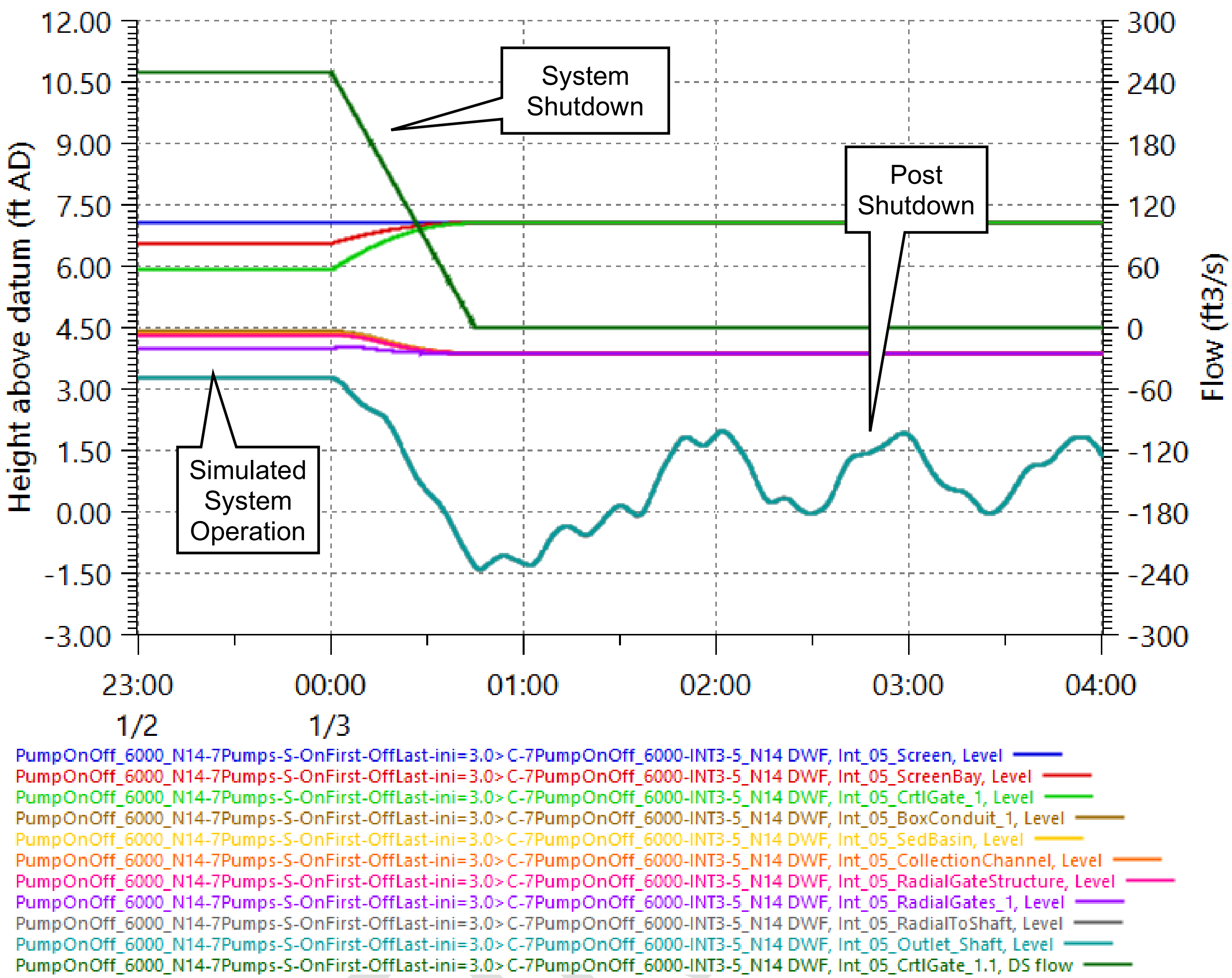

Figure 30. Intake C-E-5 Flow and Water Levels during System Shutdown Operation, Central Corridor, 6,000 cfs

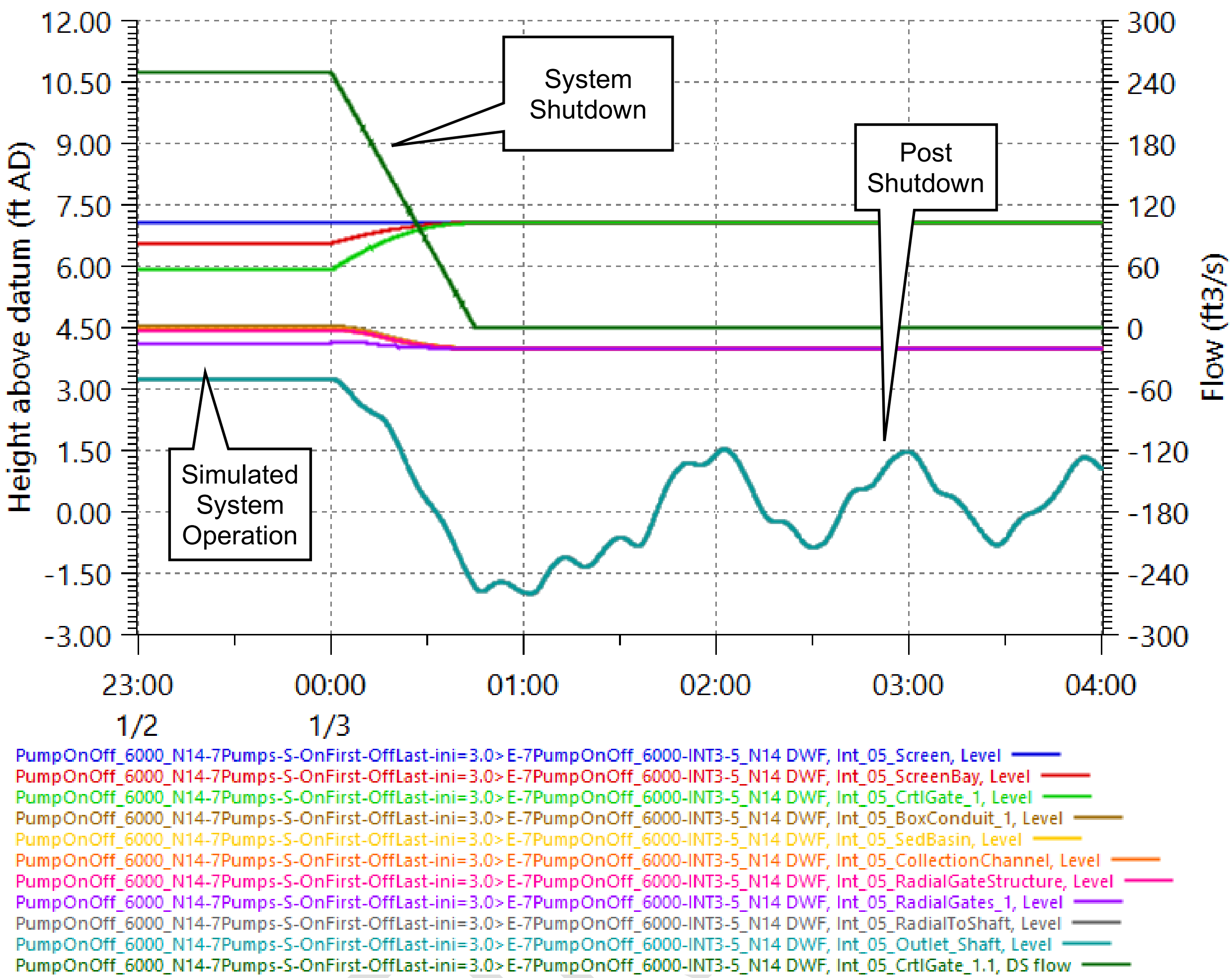

Figure 31. Intake C-E-5 Flow and Water Levels during System Shutdown Operation, Eastern Corridor, 6,000 cfs

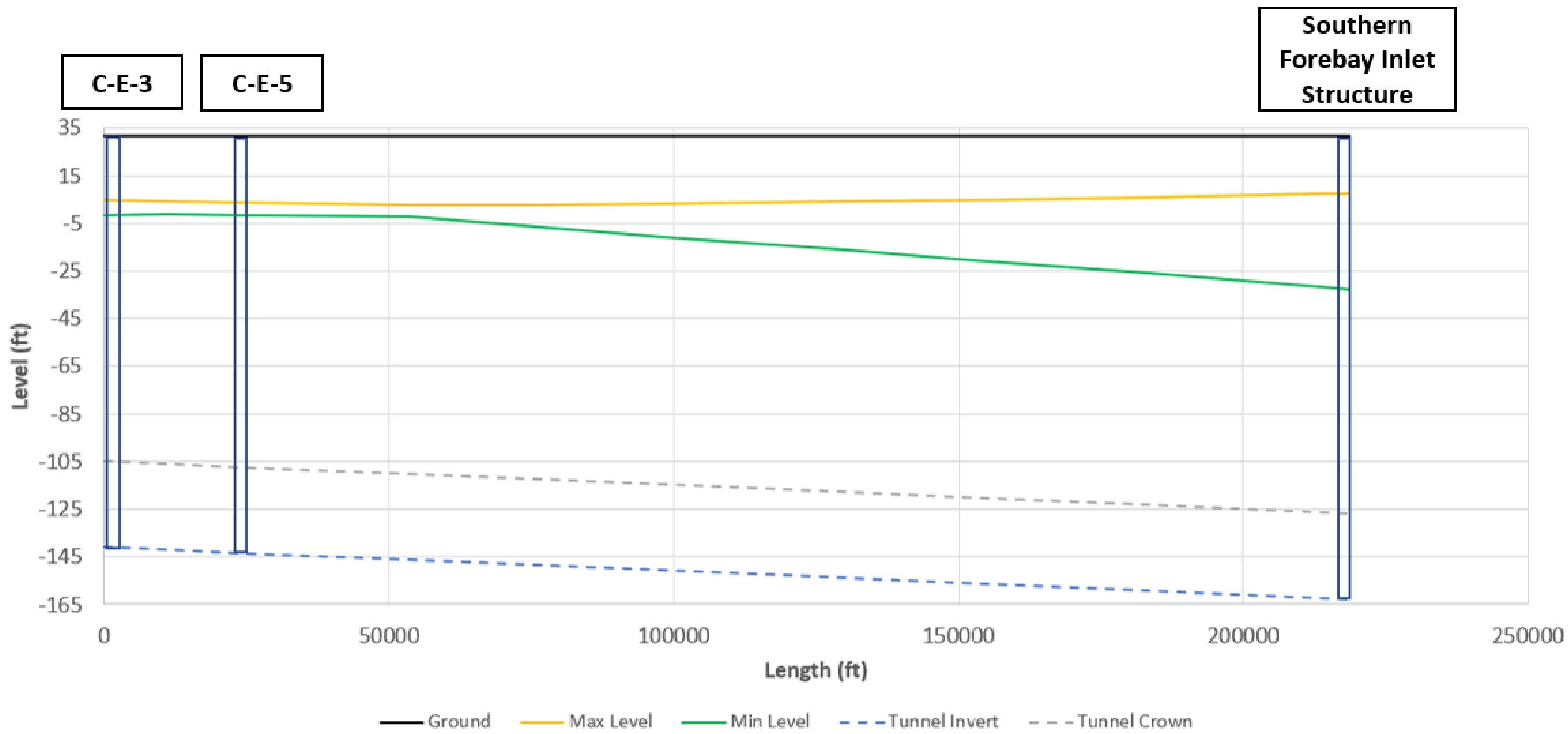

Figure 32. Minimum and Maximum Hydraulic Grade Line Profile, Central Corridor, 6000 cfs

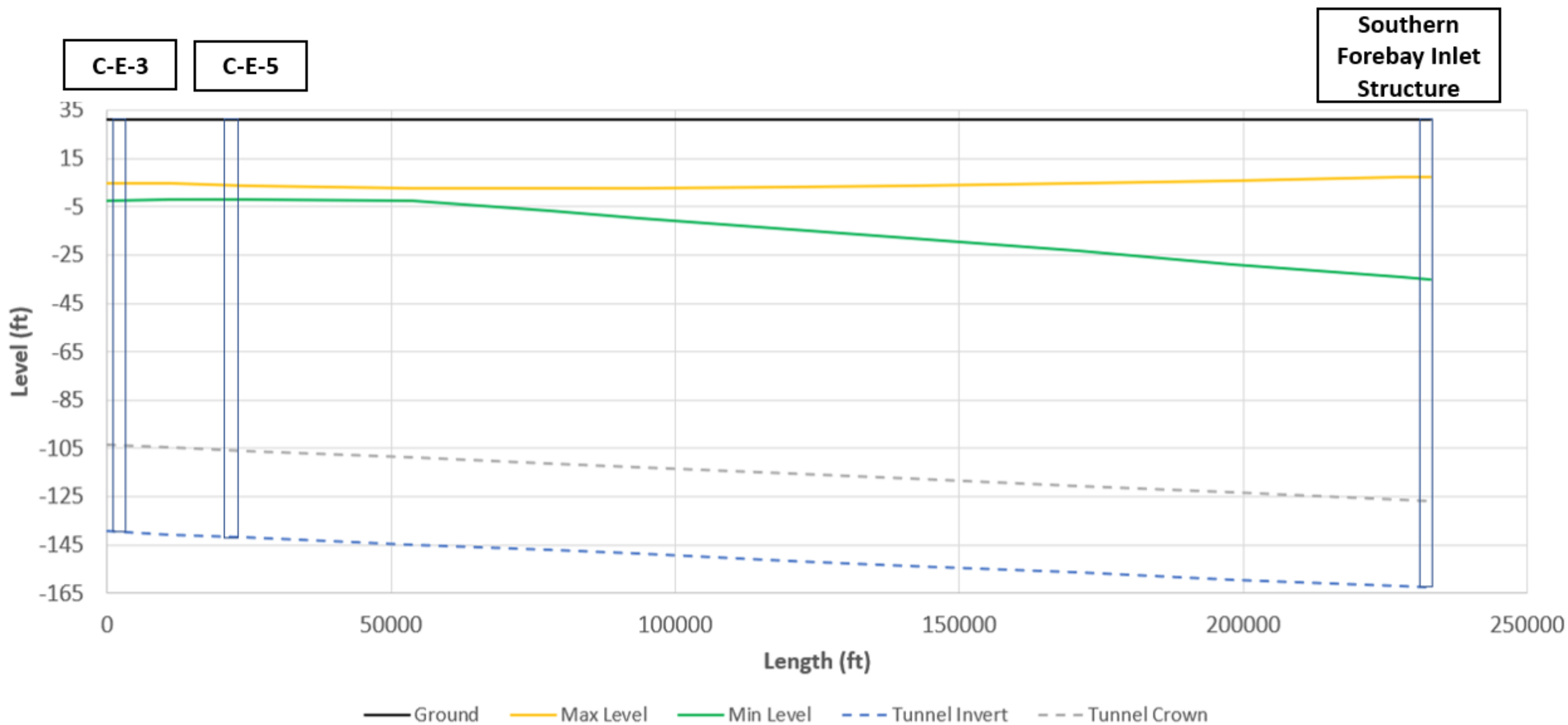

Figure 33. Minimum and Maximum Hydraulic Grade Line Profile, Eastern Corridor, 6000 cfs

3.3.3.3 Summary

The following summarizes the startup and shutdown operations for the Central and Eastern corridors:

- A steady-state diversion capacity of 6,000 cfs was established for startup and shutdown without the need of an additional hydraulic equalization facility, such as an intermediate forebay (IF).
- The system stabilized during the startup, shutdown and flow transitions to the set-point diversion capacity of 6,000 cfs.
  - Shafts along the tunnel acted as equalization chambers.
  - Pumps are operated with variable-frequency drives and changed gradually to match startup and shutdown rates at the intakes.
- System startup and flow transitions to the set-point steady-state operation would be achieved using the simulated diversion rate of 1,000 cfs per 15 minutes.
- System shutdown and flow transitions to the set-point steady-state operation would be achieved using the simulated rate of 1,000 cfs per 15 minutes.

The use of an additional hydraulic facility—the IF with a footprint of 500 ft by 500 ft—was previously evaluated for transient-surge conditions for the Central corridor at project design flow capacities of 6,000 and 7,500 cfs and is described in the Capacity Analysis for Preliminary Tunnel Diameter Selection TM. It was determined that the IF was not needed for surge mitigation. The system startup and shutdown were evaluated without the use of the IF, and it was determined that a smooth system startup from 0 to 6,000 cfs and a shutdown from 6,000 to 0 cfs is achievable with the tunnel conveyance system and diversion rates simulated, as previously described. As such, it is recommended that the IF is not needed for the Project.

4. Analysis and Evaluation

4.1 Steady-State Hydraulic Head Loss Analysis

In accordance with the methodology and criteria described, a steady-state, hydraulic head loss analysis was performed between the intake outlet shafts to the Pumping Plant wet well for the maximum capacity design flow capacities of 3,000, 4,500, 6,000, and 7,500 cfs for the Central corridor. The flow scenarios evaluated are documented in Table 6.

Table 6. Flow Scenario Descriptions Used for Central Corridor Steady-State Hydraulic Head Loss Analysis

Flow Scenario (cfs)	Tunnel ID (feet)	Manning's *n*	Intake Outlet Shaft ID (feet)	Intermediate Shaft ID (feet)	Pumping Plant Wet Well Inlet Shaft ID (feet)	River Level	Intake Order
3,000	26	0.016	83	70	110	Low & Design	C-E-5
3,000	26	0.014	83	70	110	Normal & High	C-E-5
4,500	31	0.016	83	70	110	Low & Design	C-E-3+5
4,500	31	0.014	83	70	110	Normal & High	C-E-3+5

Table 6. Flow Scenario Descriptions Used for Central Corridor Steady-State Hydraulic Head Loss Analysis

Flow Scenario (cfs)	Tunnel ID (feet)	Manning's *n*	Intake Outlet Shaft ID (feet)	Intermediate Shaft ID (feet)	Pumping Plant Wet Well Inlet Shaft ID (feet)	River Level	Intake Order
6,000	36	0.016	83	70	115	Low & Design	C-E-5+3
6,000	36	0.014	83	70	115	Normal & High	C-E-5+3
7,500	40	0.016	83	70	120	Low & Design	C-E-5+3+2
7,500	40	0.014	83	70	120	Normal & High	C-E-5+3+2

The steady-state hydraulic analysis incorporated the highest friction factor, Manning's *n* of 0.016, at the low and design Sacramento River WSELs to establish both the highest head loss between the intakes to the Pumping Plant wet well and the lower operating WSELs in the Pumping Plant wet well. The lower friction factor, Manning's *n* of 0.014, was combined with the normal and extreme high Sacramento River elevations to establish both the lowest head loss between the intakes to the Pumping Plant wet well and the higher WSELs in the Pumping Plant wet well. This analysis included head losses through the fish screens at each intake. Fish screens were assumed to be in clean condition.

Figures 34 through 37 plot the tunnel head loss results that were developed for each of the flow conditions. Tables 7 through 10 summarize the corresponding head loss to the Pumping Plant wet well WSEL for each maximum design capacity and assigned Manning's *n*.

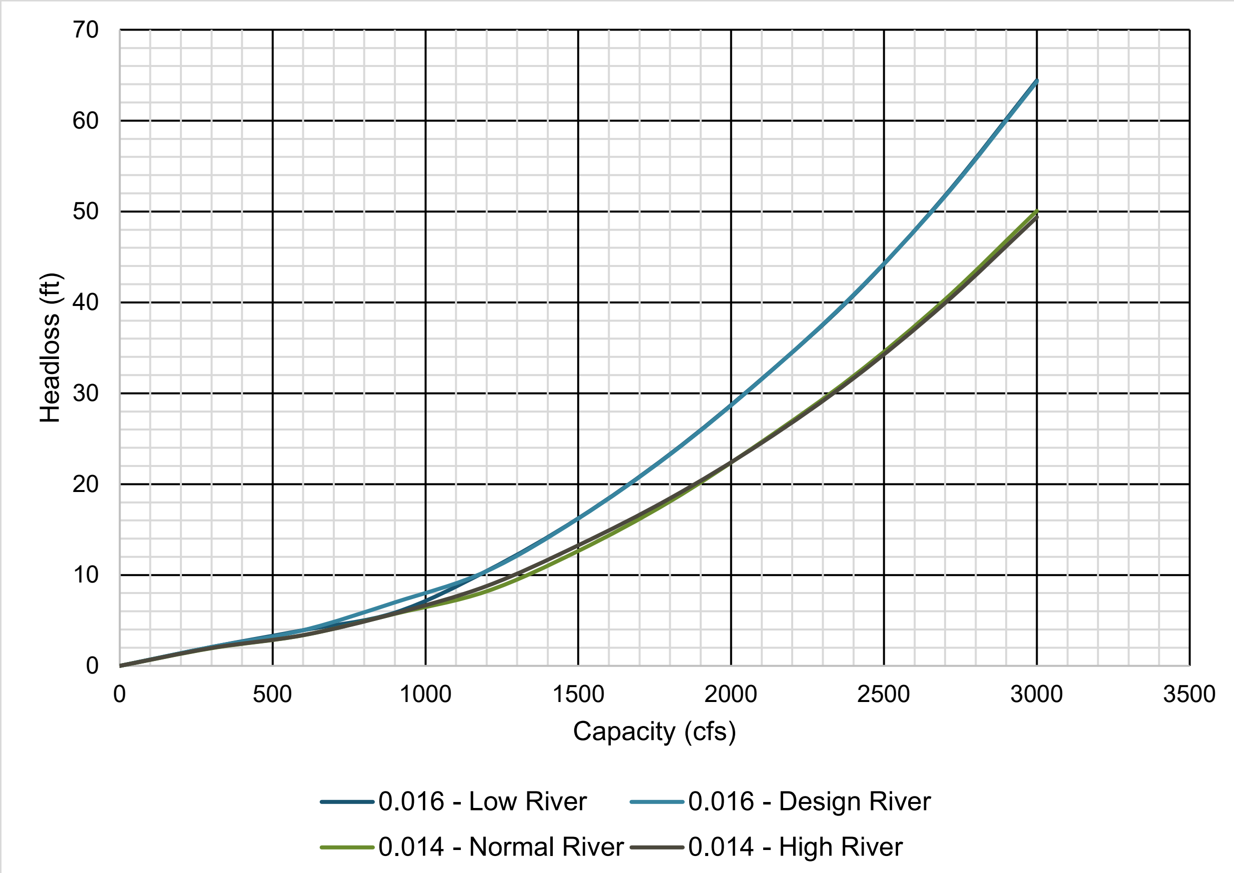

Figure 34. System Head Curves – 26-foot ID Tunnel with a Maximum Flow Capacity of 3,000 cfs

Table 7. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 3,000 cfs

River Level	Manning's *n*	Head Loss from Intakes to Pumping Plant Wet Well (ft)	Wet Well WSEL (ft)
Low River	0.016	64.4	-60.8
Design River	0.016	64.3	-59.8
Normal River	0.014	50.1	-44.3
High River	0.014	49.4	-23.1

Referring to Figure 34 and Table 7 for the 26-ft ID tunnel, at the maximum design capacity of 3,000 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 64.4 ft and result in a WSEL in the Pumping Plant wet well of -60.8 ft. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develop a steady-state head loss of 49.4 ft and result in a WSEL in the Pumping Plant wet well of -23.1 ft.

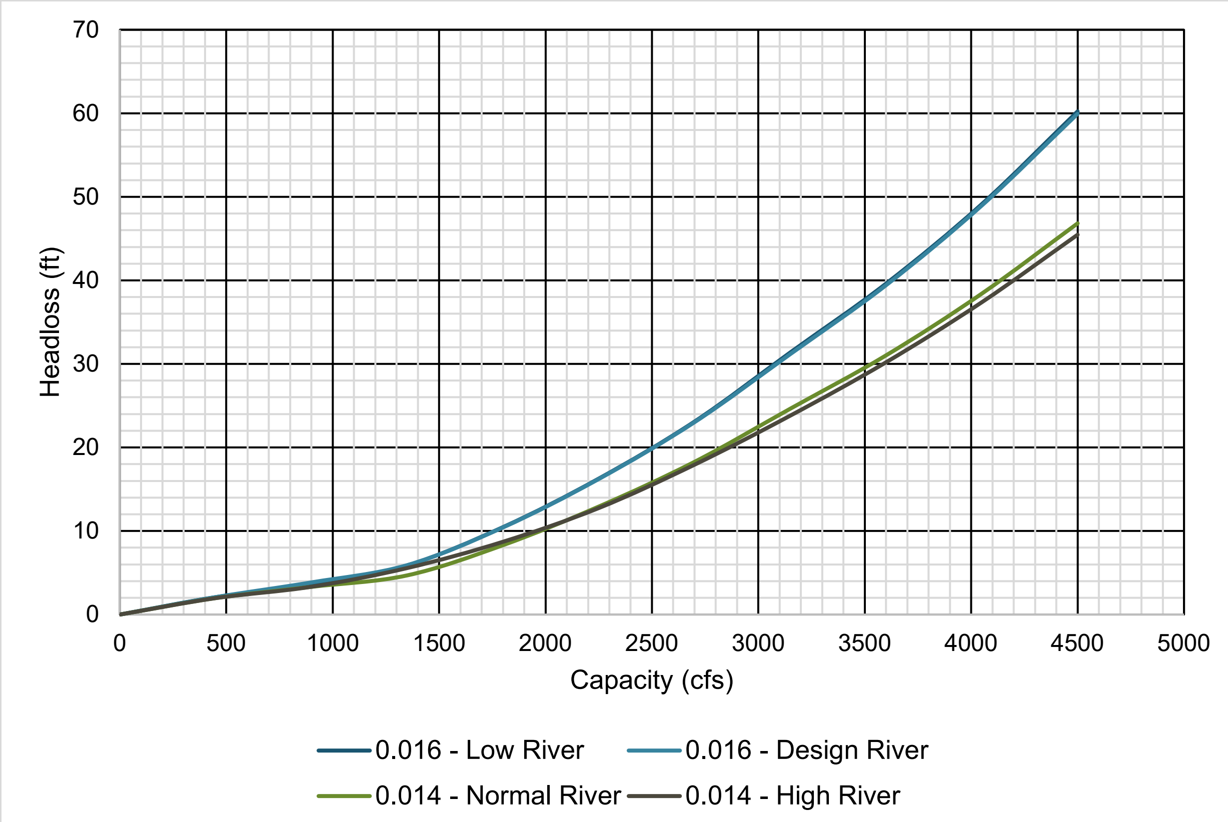

Figure 35. System Head Curves – 31-foot ID Tunnel with a Maximum Flow Capacity of 4,500 cfs

Table 8. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at 4,500 cfs Design Flow Capacity

River Level	Manning's *n*	Head Loss from Intakes to Pumping Plant Wet Well (ft)	Wet Well WSEL (ft)
Low River	0.016	60.2	-56.5
Design River	0.016	59.9	-55.4
Normal River	0.014	46.8	-40.9
High River	0.014	45.5	-18.2

Referring to Figure 35 and Table 8 for the 31-ft ID tunnel, at the maximum design capacity of 4,500 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 60.2 ft and result in a WSEL in the Pumping Plant wet well of -56.5 ft. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 45.5 ft and results in a WSEL in the Pumping Plant wet well of -18.2 ft.

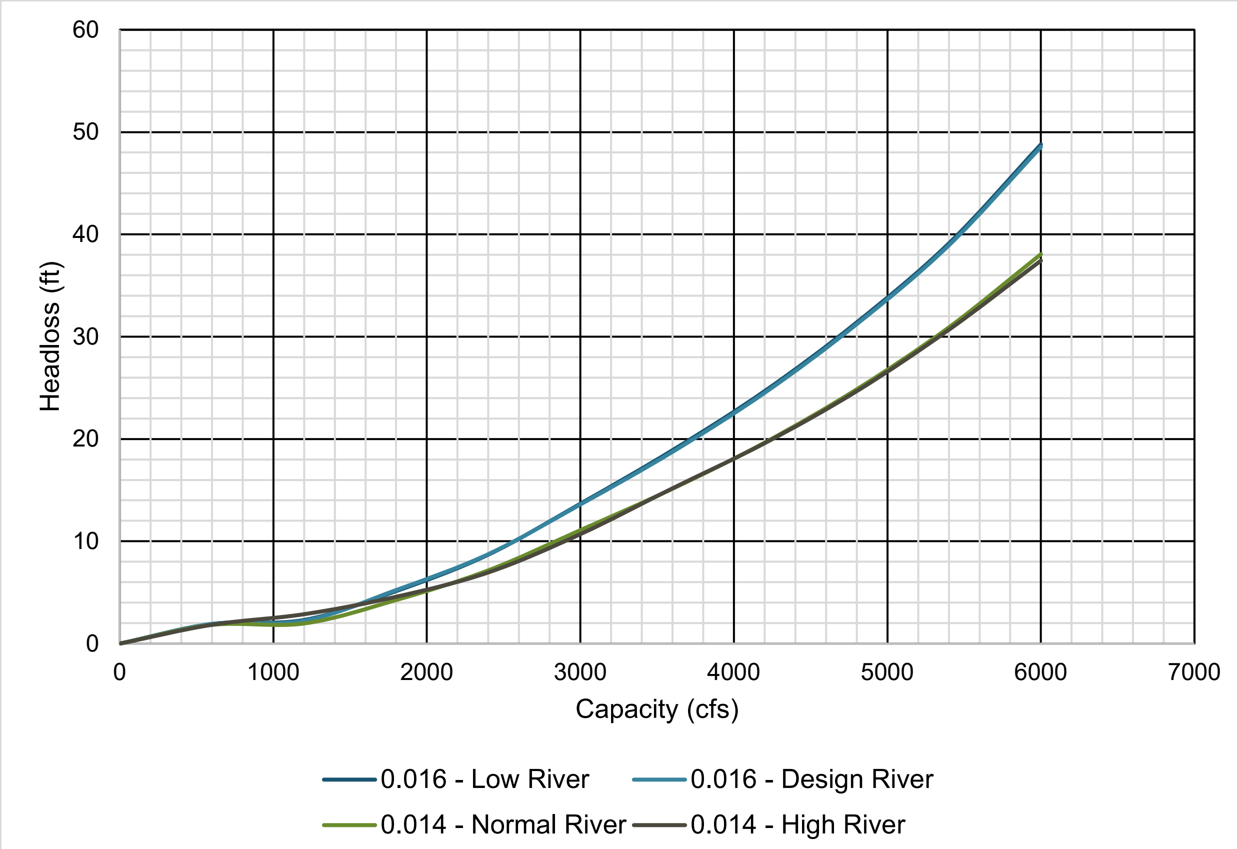

Figure 36. System Head Curves – 36-foot ID Tunnel with a Maximum Flow Capacity of 6,000 cfs

Table 9. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at 6,000 cfs Design Flow Capacity

River Level	Manning's *n*	Head Loss from Intakes to Pumping Plant Wet Well (ft)	Wet Well WSEL (ft)
Low River	0.016	48.8	-45.1
Design River	0.016	48.5	-44.0
Normal River	0.014	38.1	-32.1
High River	0.014	37.4	-10.1

Referring to Figure 36 and Table 9 for the 36-ft ID tunnel, at the maximum design capacity of 6,000 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 48.8 ft and result in a WSEL in the Pumping Plant wet well of -45.1 ft. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 37.4 ft and results in a WSEL in the Pumping Plant wet well of -10.1 ft.

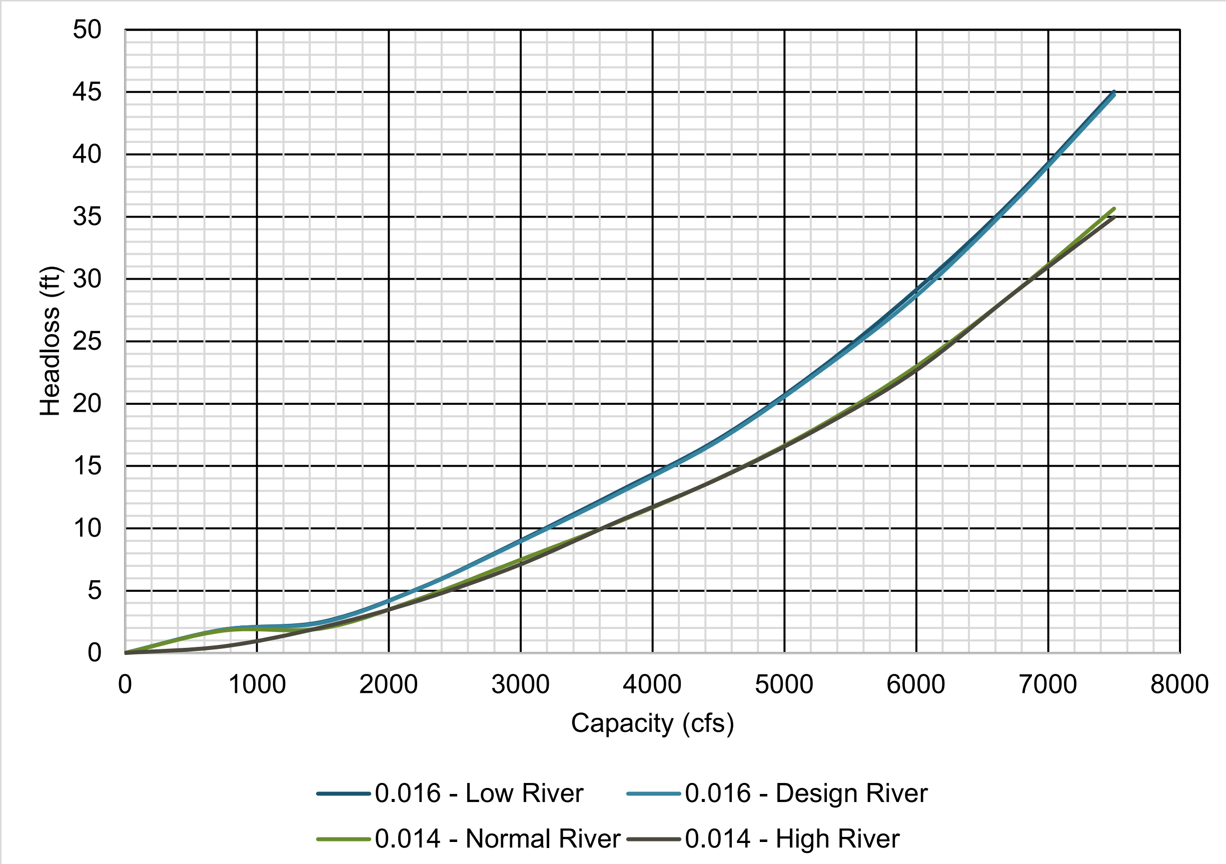

Figure 37. System Head Curves – 40-foot ID Tunnel with a Maximum Flow Capacity of 7,500 cfs

Table 10. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at 7,500 cfs Design Flow Capacity

River Level	Manning's *n*	Head Loss from Intakes to PP Wet Well (ft)	Wet Well WSEL (ft)
Low River	0.016	60.2	-60.8
Design River	0.016	64.3	-59.8
Normal River	0.014	50.1	-44.3
High River	0.014	49.4	-23.1

Referring to Figure 37 and Table 10, at the maximum design capacity of 7,500 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 60.2 ft and result in a WSEL in the Pumping Plant wet well of -60.8 ft. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 49.4 ft and results in a WSEL in the Pumping Plant wet well of -23.1 ft.

4.1.1 Hydraulic Grade Line Development

HGLs were generated using both the Central and Eastern corridors for a maximum design capacity of 6,000 cfs. The Central and Eastern corridors were evaluated at a Manning's *n* of 0.016 and 0.014 using the design Sacramento River levels as stated in Section 2.2. The HGL profiles are shown on Figures 38 and 41. Tables 11 and 12 show the WSEL for the Central and Eastern corridors, respectively.

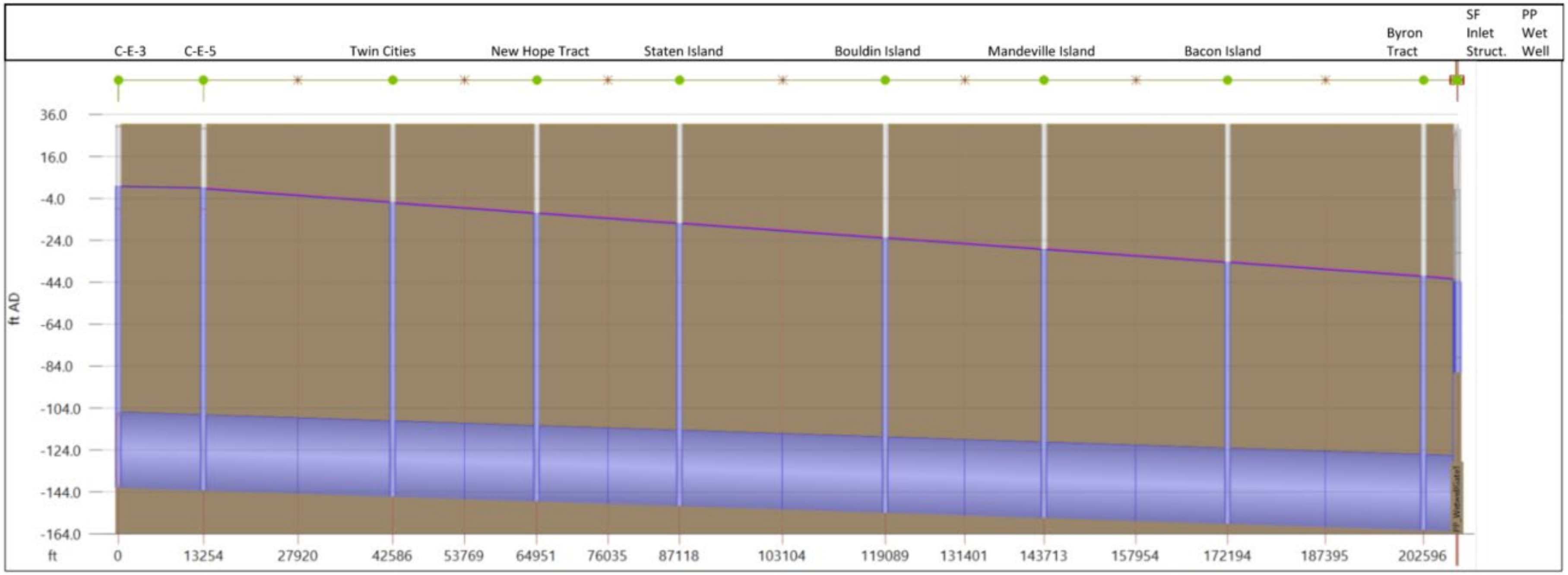

Figure 38. Central Corridor Hydraulic Grade Line – 36-foot ID, 6,000 cfs Maximum Design Capacity, Manning's *n* 0.016, Design River

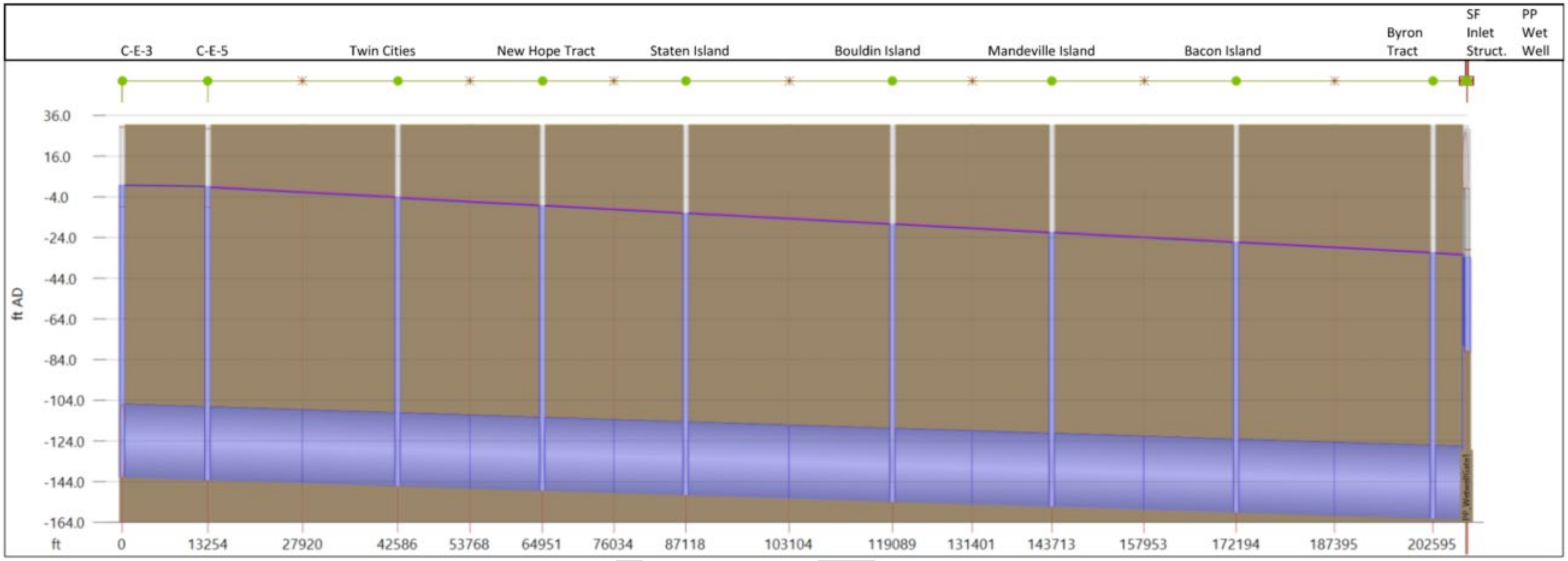

Figure 39. Central Corridor Hydraulic Grade Line – 36-foot ID, 6,000 cfs Maximum Design Capacity, Manning's *n* 0.014, Design River

Table 11. Central Corridor Hydraulic Grade Line WSEL at Design Flow Capacity 6,000 cfs

Node ID	WSEL (ft)	
	Manning's *n*: 0.016	Manning's *n*: 0.014
Intake C-E-3 Outlet Shaft	1.8	1.8
Intake C-E-5 Outlet Shaft	1.0	1.1
Twin Cities	-5.9	-4.2
New Hope Tract Central	-11.1	-8.4
Staten Island	-16.0	-12.1
Bouldin Island	-22.9	-17.4
Mandeville Island	-28.3	-21.6
Bacon Island	-34.5	-26.4
Byron Tract (Working Shaft)	-41.2	-31.5
Southern Forebay Inlet Structure	-42.5	-32.6
Wet Well	-44.0	-34.0

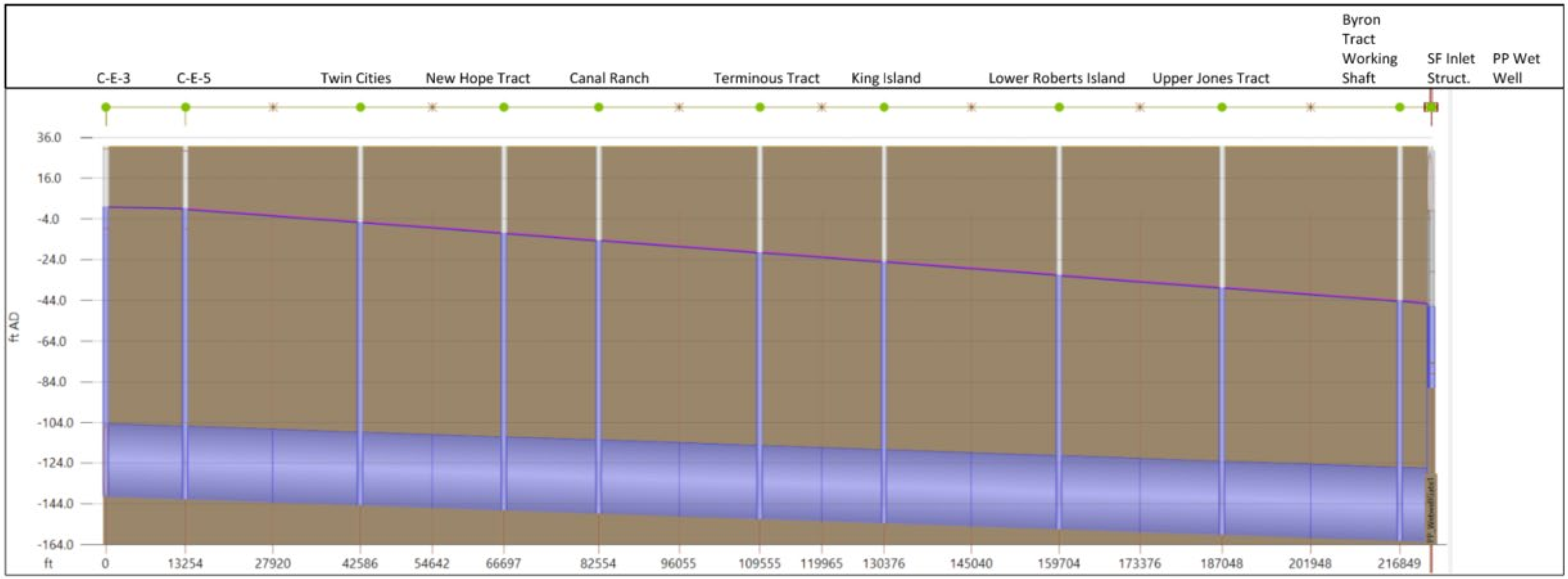

Figure 40. Eastern Corridor Hydraulic Grade Line – 36-foot ID, 6,000 cfs Maximum Design Capacity, Manning's *n* 0.016, Design River

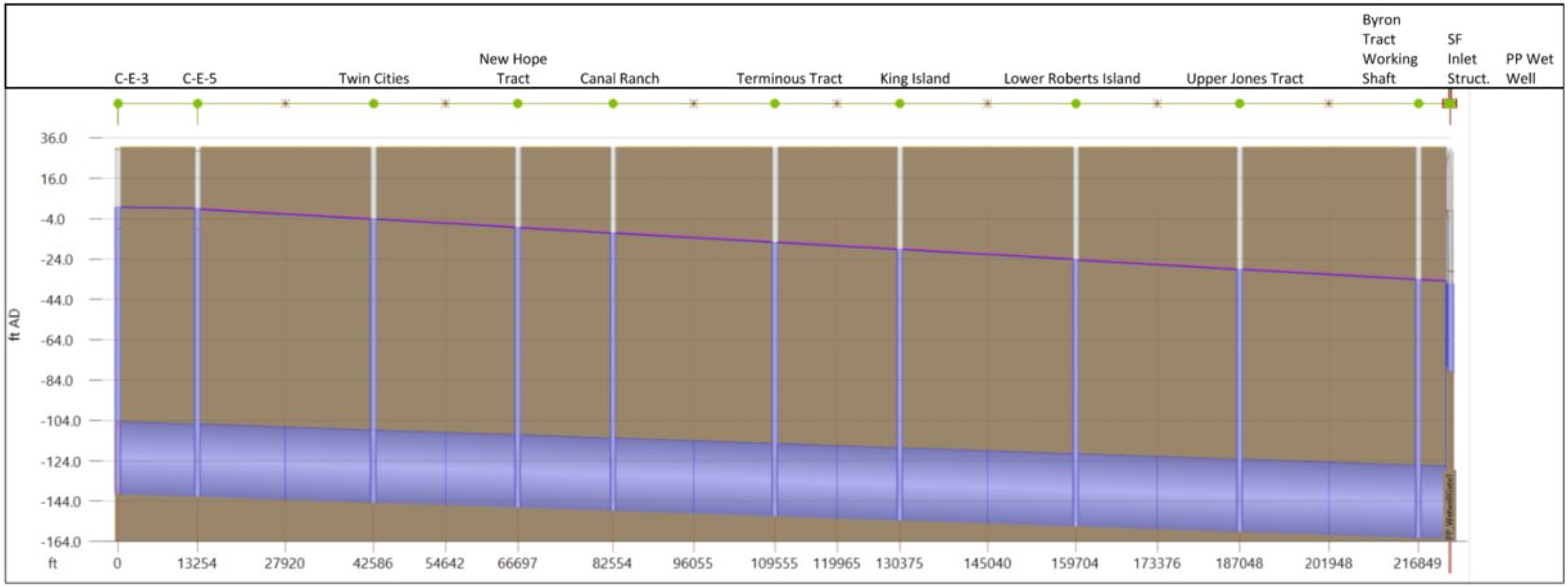

Figure 41. Eastern Corridor Hydraulic Grade Line – 36-foot ID, 6,000 cfs Maximum Design Capacity, Manning's *n* 0.014, Design River

Table 12. Eastern Corridor Hydraulic Grade Line WSEL at Design Flow Capacity 6,000 cfs

Node ID	WSEL (ft)	
	Manning's *n*: 0.016	**Manning's *n*: 0.014**
Intake C-E-3 Outlet Shaft	1.8	1.8
Intake C-E-5 Outlet Shaft	1.0	1.1
Twin Cities	-5.9	-4.2
New Hope Tract (Eastern)	-11.1	-8.3
Canal Ranch	-14.8	-11.1
Terminous Tract	-20.7	-15.7
King Island	-25.2	-19.2
Lower Roberts Island	-31.6	-24.1
Upper Jones Tract	-38.0	-29.1
Byron Tract (Working Shaft)	-44.5	-34.0
Southern Forebay Inlet Structure	-45.7	-35.0
Wet Well	-47.3	-36.6

5. Surge Analysis

5.1 Model Description

In accordance with the methodology and criteria described, a hydraulic transient-surge analysis was performed for the Project between the C-E-2 drop shaft and the Pumping Plant wet well. This analysis was conducted to establish the maximum and minimum HGLs along the entire tunnel system resulting from transient-surge events for each tunnel system and design flow conditions at maximum and minimum Sacramento River WSELs. This analysis includes system-wide configuration updates and intake gate closure rate adjustments applied to the previous hydraulic transient-surge analysis described in the Capacity Analysis for Preliminary Tunnel Diameter Selection TM.

The maximum tunnel flow velocity of 6 fps was recommended based on the hydraulic criteria previously established. For this analysis, a 36 ft and 40 ft tunnel are used in both the Central and Eastern corridors for the design flow conditions of 6,000 cfs and 7,500 cfs, respectively, to evaluate a simultaneous pump shutdown condition caused by power failure at the Pumping Plant. The vapor pressure was assumed to be -14.2 psi. The scenarios evaluated are shown in Table 13 for the two design flow capacities and tunnel corridors.

The tunnel design flow capacities of 3,000 and 4,500 cfs were not evaluated in this transient-surge analysis. The lower friction factor (Manning's *n* of 0.014) was used for this analysis to provide conservative transient-surge results.

Table 13. Scenarios for Central and Eastern Corridors for 6,000 and 7,500 cfs Project Design Capacities

Scenario	Corridor	Flow Rate [cfs]	Tunnel Diameter [feet]	River Elevation [feet]	Sedimentation Basins Gates Closing Time [minutes]
1	Central	6,000	36	3.6	12
2	Central	6,000	36	28.2	12
3	Central	7,500	40	3.6	12
4	Central	7,500	40	28.2	12
5	Eastern	6,000	36	3.6	12
6	Eastern	6,000	36	28.2	12
7	Eastern	7,500	40	3.6	12
8	Eastern	7,500	40	28.2	12

Transient-surge events were simulated by simultaneously stopping all pumps in operation at the Pumping Plant, followed by the closure of sediment basin outlet gates at each intake in operation. Closure of the sediment basin outlet gates prevented reverse flows into the Sacramento River from the intakes during the simulated transient-surge event.

5.2 Transient-Surge Results

5.2.1 Central Corridor

Results for the Central corridor, presented on Figures 42 through 45, show the hydraulic transient maximum and minimum HGL elevations that occur throughout the transient-surge events along the tunnel corridor, using 83 ft ID intake shafts, two 115 ft ID launch shafts at Twin Cities (double shaft), one 115 ft ID launch shaft at Bouldin Island, 115 ft ID overflow shaft at the Pumping Plant wet well entrance, and 70 ft ID for all other reception and maintenance shafts. The envelope of the maximum and minimum HGLs are plotted across the tunnel alignment between the intake C-E-3 drop shaft to the Pumping Plant overflow shaft for the steady-state flow conditions of 6,000 cfs and intake C-E-5 to the Pumping Plant overflow shaft for the steady-state flow condition of 7,500 cfs. For reference, the HGL elevation of 32 ft is shown by a green horizontal dashed line. The tunnel intake drop shafts and Pumping Plant overflow shaft are notated in each graph. Intermediate shafts along the alignment are indicated by vertical lines and are not notated. The tunnel crown elevation for each diameter evaluated is shown as a dashed blue line on each graph.

The control gates in each intake (in operation) were simultaneously closed. The gate closure rate was identical, and gates were closed at a linear rate of 12 minutes from their last position to fully closed. Additional runs were performed where control gates were left in their last open position resulting in continuous overflow causing flooding in certain shafts, which is an unacceptable condition. Therefore, controlled provisions must be implemented as the system design is further developed that prevents control gates from remaining open during significant hydraulic transient events.

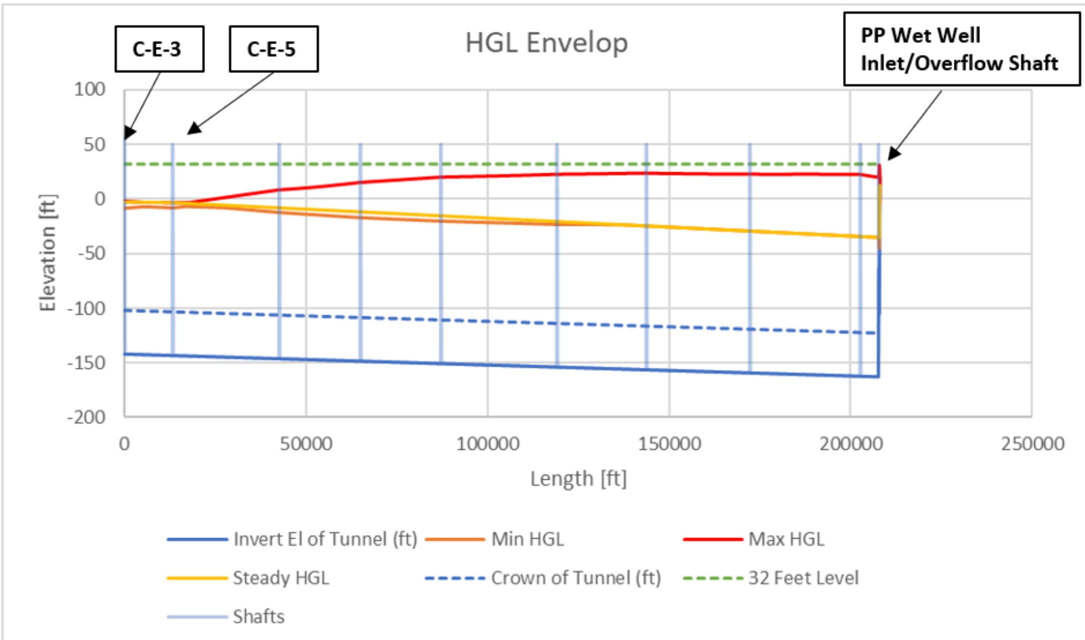

Figure 42. Scenario 1 – Central Corridor, Design Capacity 6,000 cfs, Low River Level, Minimum and Maximum Hydraulic Grade Line Profiles

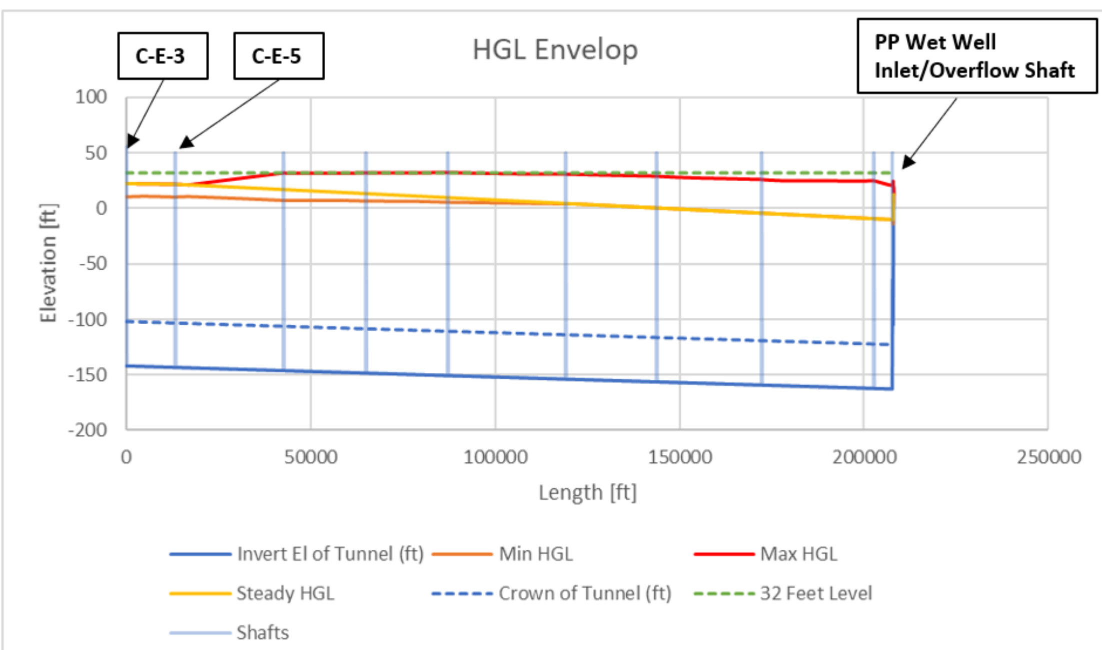

Figure 43. Scenario 2 – Central Corridor, Design Capacity 6,000 cfs, High River Level, Minimum and Maximum Hydraulic Grade Line Profiles

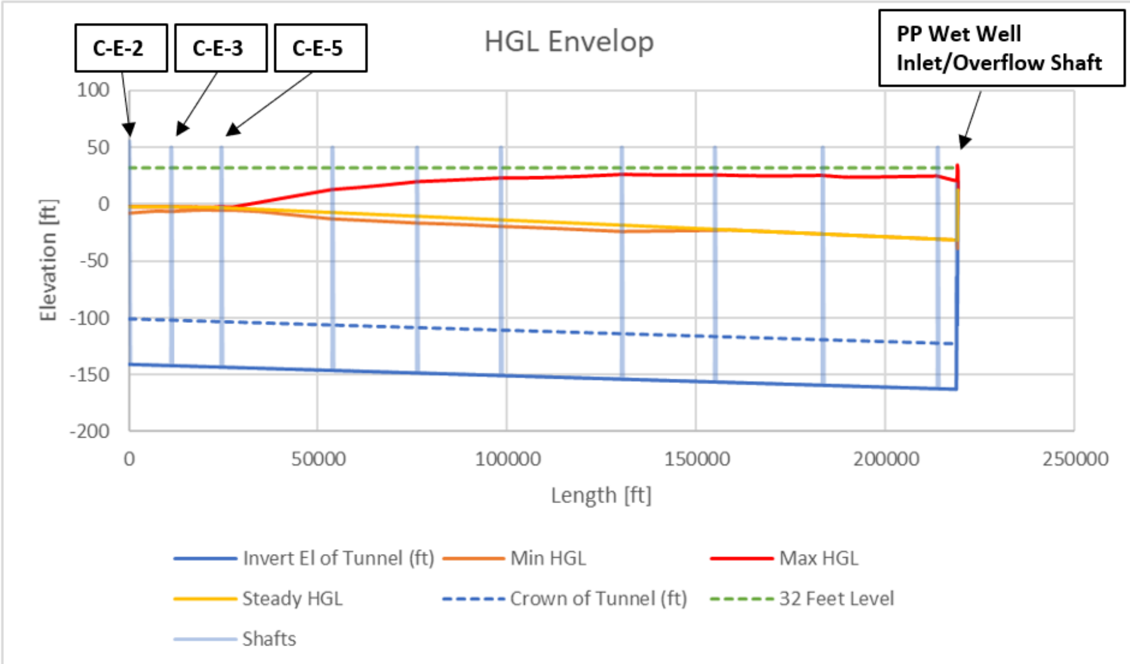

Figure 44. Scenario 3 – Central Corridor, Design Capacity 7,500 cfs, Low River Level, Minimum and Maximum Hydraulic Grade Line Profiles

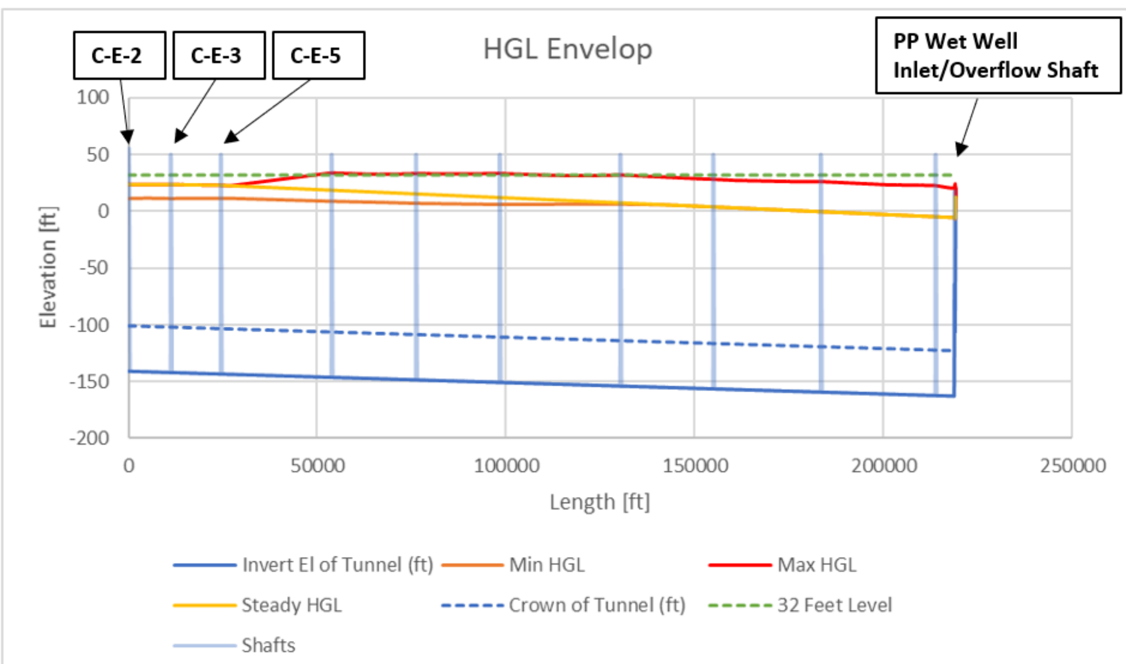

Figure 45. Scenario 4 – Central Corridor, Design Capacity 7,500 cfs, High River Level, Minimum and Maximum Hydraulic Grade Line Profiles

The results indicate that no negative pressures are developed along the entire length of the Central corridor tunnel alignment, and no overflow conditions occurred at any intake in operation. All pressures were within the conceptual design limits of the tunnel at either the maximum or minimum Sacramento River WSEL evaluated at each intake. See Table 14 below for exact HGL values for each scenario.

Table 14. Central Corridor – Minimum and Maximum HGL Values

Scenario	Minimum HGL [ft]	Maximum HGL [ft]
1	-35.11	23.82
2	-10.05	32.37
3	-31.35	26.23
4	-5.39	33.9

The maximum transient-surge HGL results at each tunnel shaft for the project design capacities of 6,000 and 7,500 cfs were compared against the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise at intake C-E-3 (for the 6,000 cfs design flow option) and intake C-E-2 (for the 7,500 cfs design flow option). Operation of the Delta Conveyance System during the 200-year flood event is highly unlikely, however since it is possible this condition was evaluated and serves as a conservative boundary condition for this analysis. To prevent overflow conditions at any shaft for each Project design capacity option, the height of each tunnel shaft was established from the greater WSEL between the 200-year flood with sea level rise HGL plus a 3-foot freeboard or the calculated maximum transient-surge HGL plus a 3-foot freeboard at each shaft location. The water surface elevations for the 200-year flood even with sea level rise used in this analysis were obtained from the *Preliminary Flood Water Surface Elevations (Not for Construction) Memorandum* (DWR, 2020c). For the project design capacity of 6,000 cfs, the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise at intake C-E-3 would be 27.3 feet. For the project design capacity of 7,500 cfs, the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise would be 28.2 feet.

Tables 15 and 16 summarize the calculated maximum surge HGL at each tunnel shaft developed for Scenarios 1 through 4, the Sacramento River's 200-year flood with sea level rise HGL (at the intakes) and the required top of shaft elevation (selected from the greater of the surge HGL versus the 200-year flood with sea level rise HGL) plus 3 feet added for freeboard for each 6,000 and 7,500 cfs project design capacity option. The top of shaft elevations shown in Tables 15 and 16 would prevent the occurrence of an overflow during either a transient-surge event or the Sacramento River 200-year flood with sea level rise WSEL at the intakes for the project design flow capacities of 6,000 and 7,500 cfs and are shown in the engineering concept drawings.

Table 15. Central Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 6,000 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-3 Reception shaft	83.0	-2.0	22.5	27.3	30.3

Table 15. Central Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 6,000 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-5 Maintenance Shaft	83.0	-3.0	21.9	27.3	30.3
Twin Cities Double Launch Shaft	115.0 (each shaft)	8.5	32.0	27.3	35.0
New Hope Tract Maintenance Shaft	70.0	15.3	32.0	27.3	35.0
Staten Island Maintenance Shaft	70.0	20.0	32.5	27.3	35.5
Bouldin Island Launch/Reception shaft	115.0	23.0	31.0	27.3	34.0
Mandeville Island Maintenance Shaft	70.0	24.0	29.0	27.3	32.0
Bacon Island Reception Shaft	115.0	23.5	26.5	27.3	30.3
Byron Tract Working Shaft	70.0	23.0	25.0	27.3	30.3
Southern Forebay Inlet Structure Launch shaft	115.0	20.0	21.0	27.3	30.3

Table 16. Central Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 7,500 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-2 Reception Shaft	83.0	-2.0	24.0	28.2	31.2
Intake C-E-3 Maintenance Shaft	83.0	-2.3	23.8	28.2	31.2
Intake C-E-5 Maintenance Shaft	83.0	-2.0	23.3	28.2	31.2

Table 16. Central Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 7,500 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Twin Cities Double Launch Shaft	115.0 (each shaft)	13.3	34.0	28.2	37.0
New Hope Tract Maintenance Shaft	70.0	20.0	33.5	28.2	36.5
Staten Island Maintenance Shaft	70.0	23.5	33.5	28.2	36.5
Bouldin Island Launch/Reception Shaft	115.0	26.3	32.0	28.2	35.0
Mandeville Island Maintenance Shaft	70.0	26.0	28.3	28.2	31.3
Bacon Island Reception shaft	115.0	25.3	26.3	28.2	31.2
Byron Tract Working Shaft	70.0	25.0	23.0	28.2	31.2
Southern Forebay Inlet Structure Launch shaft	115.0	21.0	22.0	28.2	31.2

5.2.2 Eastern Corridor

Results for Eastern Corridor, presented on Figures 46 through 49, show the simulated hydraulic transient maximum and minimum HGL elevations during the transient-surge events along the tunnel alignment, using 83 ft ID intake shafts, two 115 ft ID launch shafts at Twin Cities (double shaft), one 115 ft ID launch shaft at Lower Roberts, 115 ft ID overflow shaft at the Pumping Plant wet well entrance, and 70 ft ID for all other reception and maintenance shafts. The minimum and maximum HGL envelopes are plotted across the tunnel alignment between the intake C-E-3 drop shaft to the Pumping Plant overflow shaft for the steady-state flow conditions of 6,000 cfs and intake C-E-5 to the Pumping Plant overflow shaft for the steady-state flow condition of 7,500 cfs. For reference, the HGL elevation of 32 ft is shown by a green horizontal dashed line. The tunnel intake drop shafts, tunnel reception and maintenance shafts, and Pumping Plant overflow are depicted in each graph. The locations of the intake drop shafts and Pumping Plant overflow shaft are identified. The tunnel crown elevation for each diameter evaluated is shown as a dashed blue line on each graph.

The control gates in each intake (in operation) were simultaneously closed. The gates closure rate was identical, and gates were closed at a linear rate of 12 minutes from their last position to fully closed.

The results indicate that no negative pressures are developed along the entire length of the Eastern Corridor tunnel alignment, and no overflow conditions occurred at any intake in operation. All pressures were within the conceptual design limits of the tunnel at either the maximum or minimum Sacramento River WSEL evaluated at each intake. See Table 17 for exact HGL values for each scenario.

Table 17. Eastern Corridor – Minimum and Maximum HGL Values

Scenario	Minimum HGL [ft]	Maximum HGL [ft]
5	-36.31	24.08
6	-12.45	33.06
7	-32.84	25.61
8	-8.55	34.38

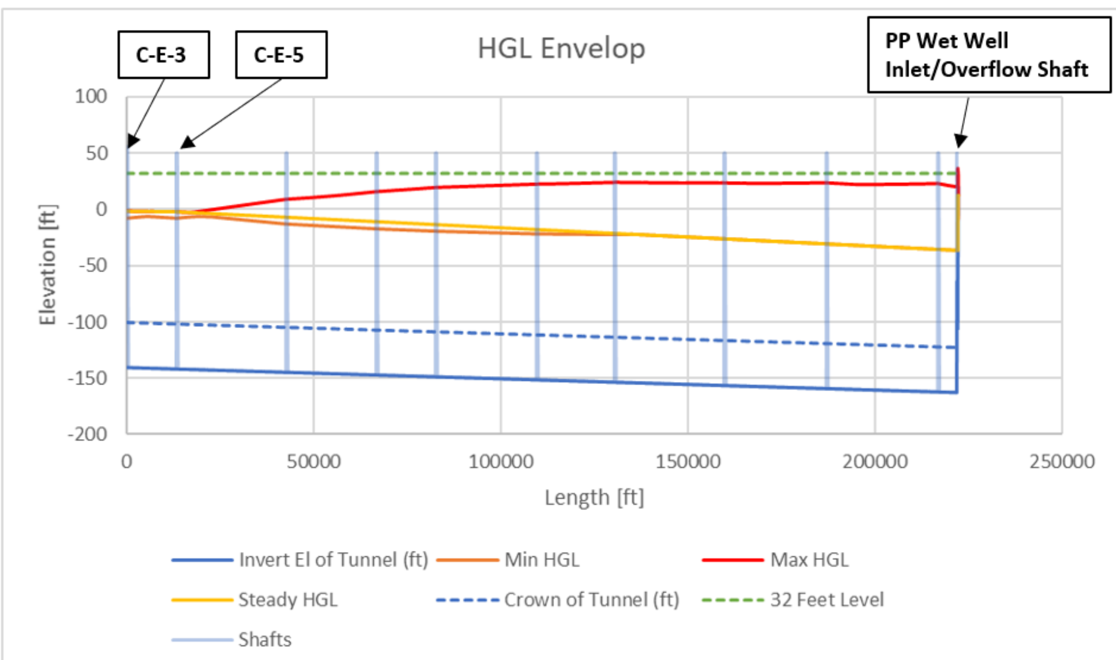

Figure 46. Scenario 5 – Eastern Corridor, Design Capacity 6,000 cfs, Low River Level, Minimum and Maximum Hydraulic Grade Line Profiles

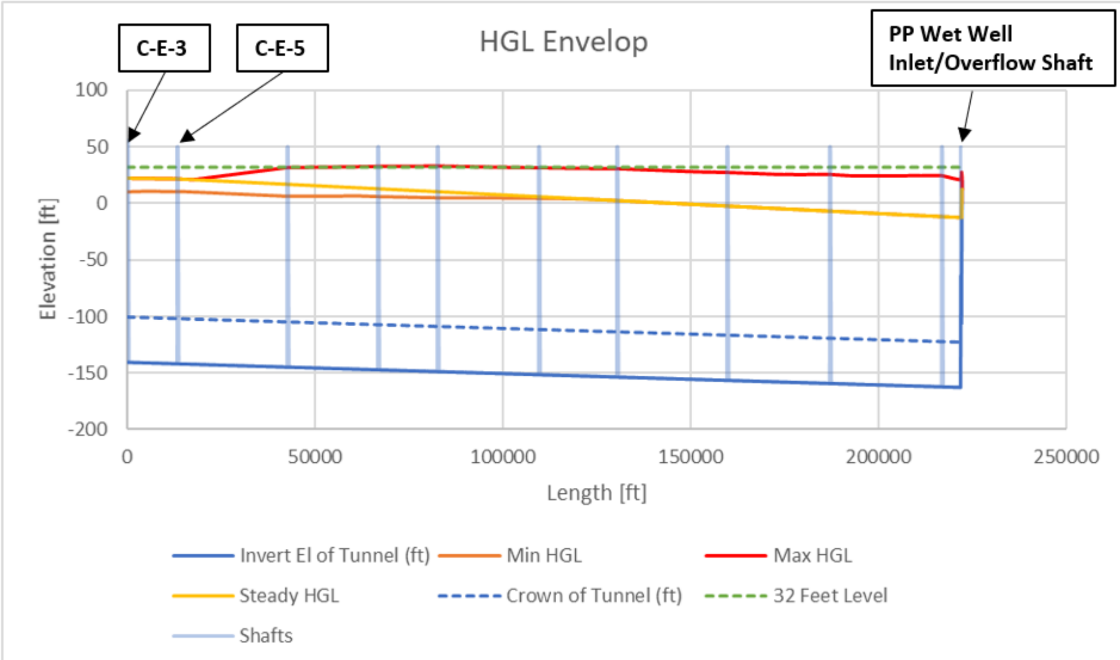

Figure 47. Scenario 6 – Eastern Corridor, Design Capacity 6,000 cfs, High River Level, Minimum and Maximum Hydraulic Grade Line Profiles

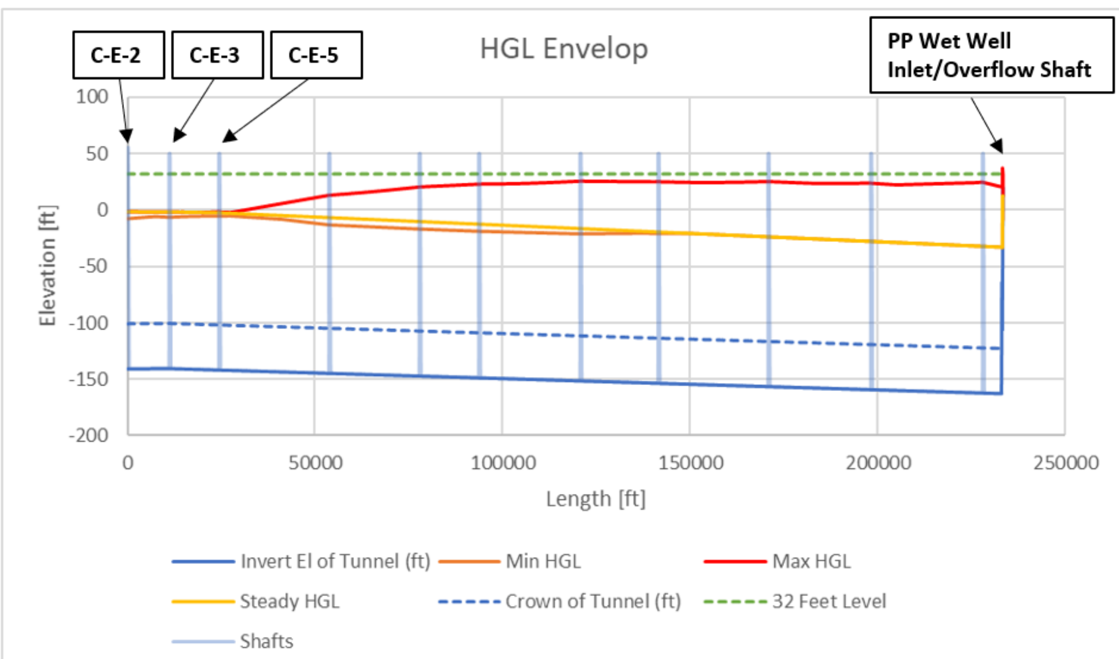

Figure 48. Scenario 7 – Eastern Corridor, Design Capacity 7,500 cfs, Low River Level, Minimum and Maximum Hydraulic Grade Line Profiles

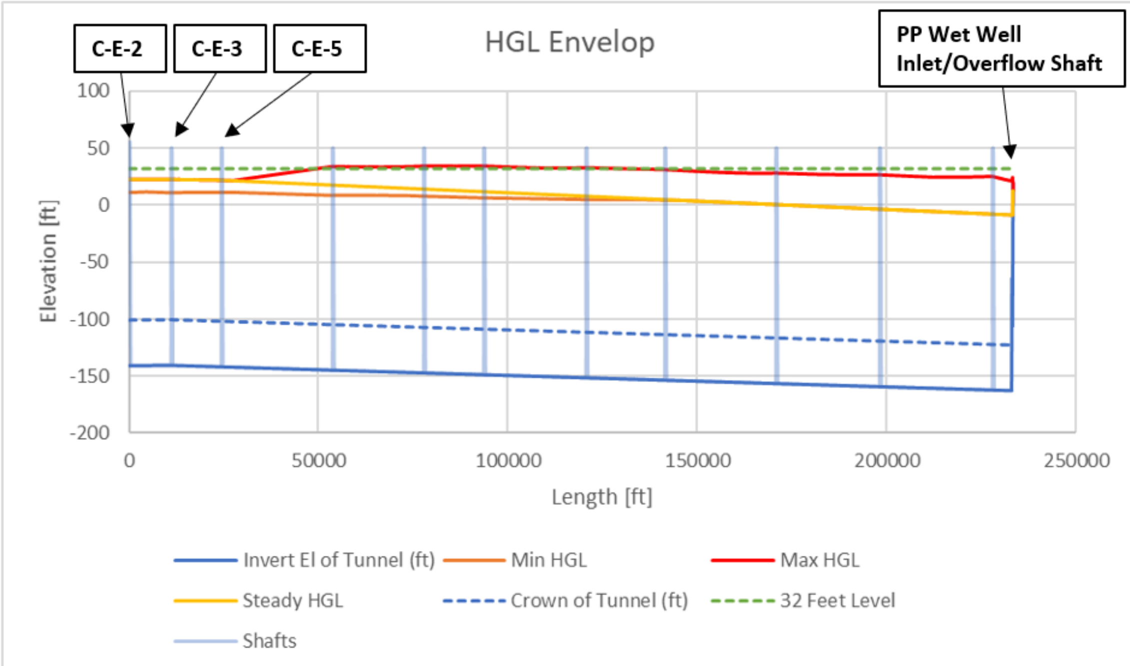

Figure 49. Scenario 8 – Eastern Corridor, Design Capacity 7,500 cfs, High River Level, Minimum and Maximum Hydraulic Grade Line Profiles

The maximum transient-surge HGL results at each tunnel shaft for the project design capacities of 6,000 and 7,500 cfs were compared against the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise at intake C-E-3 (for the 6,000 cfs design flow option) and C-E-2 (for the 7,500 cfs design flow option). To prevent overflow conditions at any shaft for each Project design capacity option, the height of each tunnel shaft was established from the greater WSEL between the 200-year flood with sea level rise HGL plus a 3-foot freeboard or the calculated maximum transient-surge HGL plus a 3-foot freeboard at each shaft location. For the project design capacity of 6,000 cfs, the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise at intake C-E-3 would be 27.3 feet. For the project design capacity of 7,500 cfs, the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise would be 28.2 feet.

Tables 18 and 19 summarize the calculated maximum surge HGL at each tunnel shaft developed for Scenarios 1 through 4, the Sacramento River's 200-year flood with sea level rise HGL (at the intakes) and the required top of shaft elevation (selected from the greater of the surge HGL versus the 200-year flood with sea level rise HGL) plus 3 feet added for freeboard for each 6,000 and 7,500 cfs project design capacity option. The top of shaft elevations shown in Tables 18 and 19 would prevent the occurrence of an overflow during either a transient-surge event or the Sacramento River 200-year flood with sea level rise WSEL at the intakes for the project design flow capacities of 6,000 and 7,500 cfs and are shown in the engineering concept drawings.

Table 18. Eastern Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 6,000 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-3 Reception shaft	83.0	-1.5	22.3	27.3	30.3
Intake C-E-5 Maintenance Shaft	83.0	-2.0	22.0	27.3	30.3
Twin Cities Double Launch Shaft	115.0 (each shaft)	9.0	32.0	27.3	35.0
New Hope Tract Maintenance Shaft	70.0	16.0	33.0	27.3	36.0
Canal Ranch Tract Maintenance Shaft	70.0	20.0	33.0	27.3	36.0
Terminous Tract Reception shaft	70.0	23.0	31.8	27.3	34.8
King Island Maintenance Shaft	70.0	24.0	31.0	27.3	34.0
Lower Roberts Launch/Reception Shaft	115.0	23.8	27.3	27.3	30.3
Upper Jones Tract Maintenance Shaft	70.0	23.8	26.0	27.3	30.3
Byron Ract Working Shaft	70.0	23.0	24.8	27.3	30.3
Southern Forebay Inlet Structure Launch shaft	115.0	20.0	21.0	27.3	30.3

Table 19. Eastern Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 7,500 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-2 Reception Shaft	83.0	-1.0	23.0	28.2	31.2

Table 19. Eastern Corridor – Required Tunnel Shaft Heights for the Project Design Capacity of 7,500 cfs

Tunnel Shaft	Shaft Finished Inside DIA (feet)	Min Sac River WSEL Surge HGL (feet)	Max Sac River WSEL Surge HGL (feet)	River 200-year Flood with SLR (feet)	Top of Shaft EL Plus Freeboard (feet)
Intake C-E-3 Maintenance Shaft	83.0	-1.0	23.0	28.2	31.2
Intake C-E-5 Maintenance Shaft	83.0	-1.0	22.0	28.2	31.2
Twin Cities Double Launch Shaft	115.0 (each shaft)	13.5	34.0	28.2	37.0
New Hope Tract Maintenance Shaft	70.0	21.0	34.5	28.2	37.5
Canal Ranch Tract Maintenance Shaft	70.0	23.0	34.5	28.2	37.5
Terminous Tract Reception Shaft	70.0	25.5	33.0	28.2	36.0
King Island Maintenance Shaft	70.0	25.0	31.5	28.2	34.5
Lower Roberts Island Launch/Reception shaft	115.0	25.0	28.0	28.2	31.2
Upper Jones Tract Maintenance Shaft	70.0	24.0	27.0	28.2	31.2
Byron Tract Working Shaft	70.0	25.0	25.5	28.2	31.2
Southern Forebay Inlet Structure Launch shaft	115.0	21.0	21.5	28.2	31.2

6. Conclusions and Recommendations

The operational study performed to analyze the need for an additional hydraulic facility determined that a smooth system startup from 0 to 6,000 cfs and a shutdown from 6,000 to 0 cfs is achievable with the tunnel conveyance system and diversion rates simulated, as previously described. As such, it is recommended that the IF is not needed for the Project.

Based on the results of the hydraulic transient-surge analysis, the maximum and minimum HGL envelope was found to be within the tunnel's conceptual design pressure limits for the boundary conditions evaluated for the Central and Eastern tunnel corridor configurations at the maximum design flow capacities of 6,000 and 7,500 cfs options.

7. References

Delta Conveyance Design and Construction Authority (DCA). 2021a. Capacity Analysis for Preliminary Tunnel Diameter Selection Technical Memorandum. Final Draft.

Delta Conveyance Design and Construction Authority (DCA). 2021b. Hydraulics Analysis Criteria Technical Memorandum. Final Draft.

Wylie, Benjamin E., and Victor L. Streeter. 1993. *Fluid Transients in Systems.* Englewood Cliffs, NJ: Prentice Hall.

California Department of Water Resources (DWR). 2020c. Preliminary Flood Water Surface Elevations (Not for Construction). September 30

8. 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**
Samantha Daigle / EDM Hydraulic Engineer	Tony Naimey / EDM Pumping Plant Lead	Gwen Buchholz / DCA Environmental Consultant Phil Ryan / EDM Design Manager	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.