Subject:	**Hydraulic Analysis of Delta Conveyance Options – Bethany Reservoir Alternative (Final Draft)**
Project feature:	Projectwide
Prepared for:	California Department of Water Resources (DWR) / Delta Conveyance Office (DCO)
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1. Purpose

The purpose of this technical memorandum (TM) is to perform preliminary hydraulic analyses of the Delta Conveyance Project (project) main tunnel and pressurized aqueduct systems for the Bethany Reservoir Alternative. This evaluation considered the intake configurations and range of Project design flow capacities as describe in *Hydraulic Analysis of Delta Conveyance Options – Main Tunnel System TM* (DCA 2021a).

Results of this analysis at each maximum design flow option were used to:

- Develop the hydraulic gradeline (HGL) envelopes for the 6,000-cubic-foot-per-second (cfs) project design capacity under steady-state and hydraulic transient-surge conditions.
- Develop system head loss curves between the Sacramento River Intakes and the Bethany Reservoir Pumping Plant (BRPP) 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 project design capacity.
- Perform a tunnel dewatering analysis of the project.

Figure 1 provides a schematic of the project configuration considered in this analysis. This evaluation was conducted between the intakes and the new Bethany Reservoir Discharge Structure.

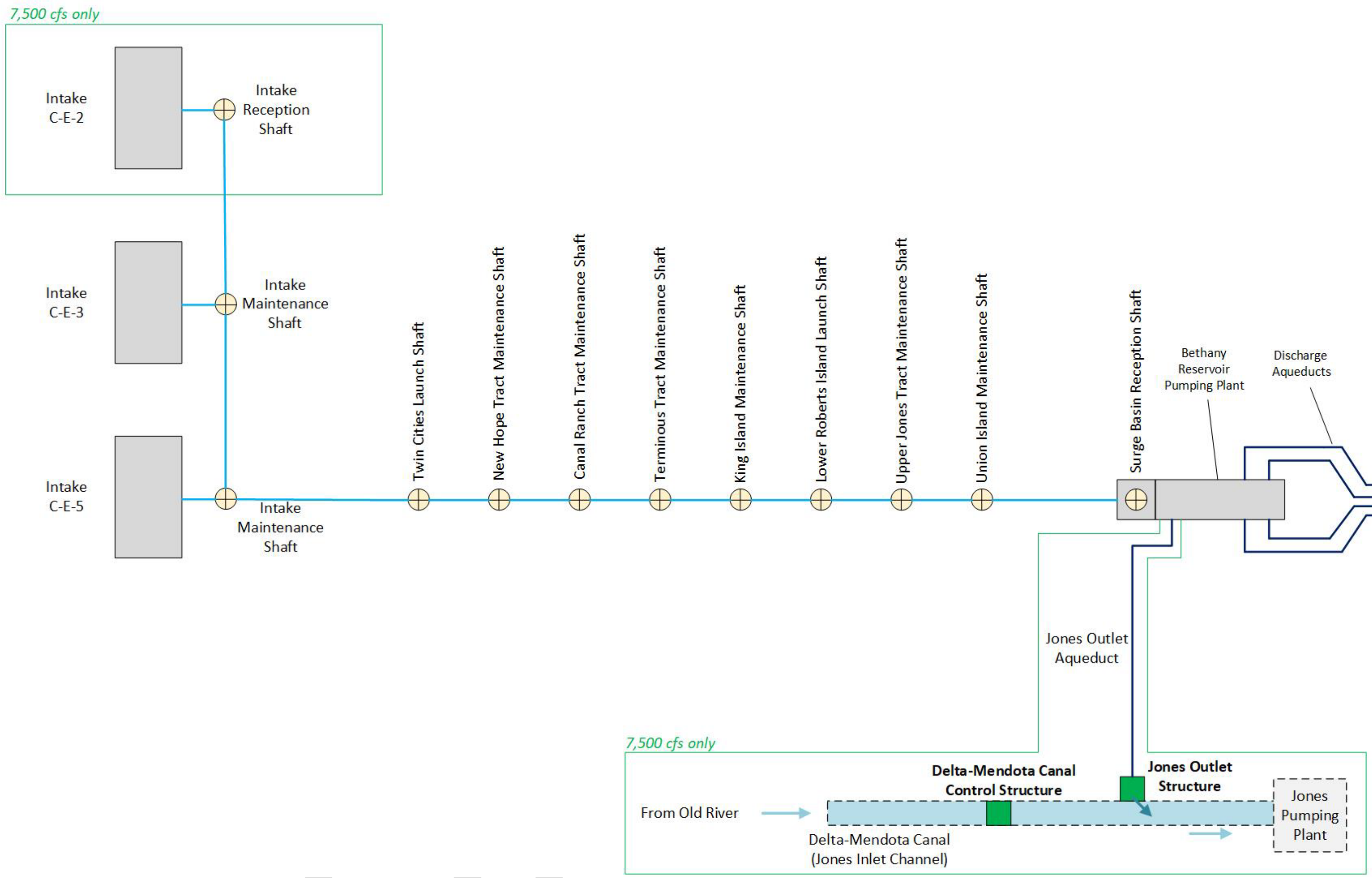

Figure 1. Delta Conveyance Bethany Reservoir Alternative Project Schematic

1.1 Organization

This TM is organized as follows:

- Methodology
- Analysis and Evaluation
- Surge Analyses
- Tunnel Dewatering
- Dewatering Duration
- Conclusions
- References
- Document History and Quality Assurance

2. Methodology

No changes were made in the methodology for the Bethany Reservoir alternative analysis, which are described in *Hydraulic Analysis of Delta Conveyance Options – Main Tunnel System TM* (DCA 2021a).

2.1 Criteria

The *Hydraulics Analysis Criteria TM* (DCA 2021b) outlines the preliminary criteria used for this analysis. Additional criteria also used in this analysis are provided in *Hydraulic Analysis of Delta Conveyance Options – Main Tunnel System TM* (DCA 2021a).

The tunnel dewatering criteria are used in the Bethany Reservoir alternative as previously documented in the *Tunnel Dewatering Pumping Facilities TM* (DCA 2021c).

2.2 Assumptions and Boundary Conditions

The *Hydraulics Analysis Criteria TM* (DCA 2021b) outlines the preliminary assumptions and boundary conditions used for this analysis. Additional assumptions, boundary conditions, and river diversion sequences also used in this analysis are provided in *Hydraulic Analysis of Delta Conveyance Alternatives – Main Tunnel System TM* (DCA 2021a). The water surface elevation (WSEL) of Bethany Reservoir was assumed to be 245 feet for this analysis. The normal working river WSEL is the current 50 percent annual exceedance and are shown in Table 1.

Table 1. Normal River WSEL At Intakes

Intake	Normal River WSEL (feet)
C-E-2	6.01
C-E-3	5.91
C-E-5	5.75

2.3 Tools

The following tools were used for the hydraulic analysis, as described in this section.

2.3.1 Conveyance System Hydraulic Model

A hydraulic model was constructed for the project between the intakes and the Bethany Reservoir Discharge Structure, using Innovyze's InfoWorks Integrated Catchment Modeling (ICM) software, version 11.0.2.22016. The hydraulic model build approach and detailed documentation is described in the *Hydraulic Analysis of Delta Conveyance Options – Main Tunnel System TM* (DCA 2021a). The hydraulic model for the Bethany Reservoir alternative remains the same as the Central and Eastern options defined per DCA (2021a) from the Sacramento River intakes south to the Lower Roberts Reception shaft, except the Lower Roberts Shaft would be a dual launch shaft. The same model build approach was applied to the tunnel and shafts south of Lower Roberts Reception shaft to the BRPP.

Following the BRPP is a pressurized aqueduct system with two or more pipelines, depending on the project conveyance capacity option. Each pipeline would convey a maximum capacity of 1,500 cfs. Four parallel pressurized pipelines were modelled using the criteria for pressure pipe documented in *Hydraulic Analysis Criteria TM* (DCA 2021b). The pipelines discharge to an outlet structure, modelled using previously described methods and criteria, and flows are discharged through the structure into Bethany Reservoir, modelled as an outfall with a static level boundary condition.

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.

2.4 Real-Time Controls

2.4.1 Controllable Devices

In the InfoWorks ICM model, real-time control (RTC) rules were developed to simulate the prospective operations of the controlled components, including intake control gates, intake radial gates, and variable speed pumps at the BRPP.

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

- Fully open at a maximum opening height of 8 feet
- Fully closed at an opening height of 0 feet
- Open gradually so the flow increases at a rate of 1,000 cfs per 15 minutes for the intake
- Close gradually so the flow decreases at a rate of 1,000 cfs per 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 the maximum vertical opening for the radial gate's pivot height providing a maximum vertical opening height of 31.6 feet.
- Under normal system operation radial gates would be modulated and would not have a vertical opening more than about 12 feet from each gate's bottom seal for any project design capacity range and range of Sacramento River WSEL's.
- 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.
- Control the water level upstream of the radial gate to not more than 1.5 feet below the river WSEL.

Pumps within the BRPP are controlled by adjusting their operating speed (revolutions per minute [rpm]) using variable-frequency drives (VFDs) and their flow capacity target:

- Active, when a pump is turned on.
- Inactive, when a pump is turned off.
- Increase the rpm gradually so the total flow increases at a rate of 1,000 cfs per 15 minutes using all active pumps.
- Decrease the rpm gradually so the flow decreases at a rate of 1,000 cfs per 15 minutes using all active pumps.
- Slightly adjust the rpm so the flow sets at the targeted normal operation flow of 500 cfs per pump in the 6,000 cfs scenario.
- The pumps must operate between the minimum and maximum rpm (established allowable range is between 50 and to 100 percent of the maximum rated speed of the pump).
- Adjust the pump's speed to maintain the pump's output flow and corresponding total dynamic head conditions within the pump's defined preferred operating range (POR). The calculated POR affinity curves establish the permissible minimum to maximum speed range (within the established speed range) based on the required total dynamic head of the pump.
- The pumps may also be controlled by individual pump on/off set-point levels within the wet well.

A controlled pump operation example is illustrated on Figure 2. The candidate pump has been plotted at its maximum rated speed and at its minimum operating speed. The minimum allowable speed (50 percent of its maximum rated speed) of the pump was not achieved due to the boundary head conditions at startup restricted the minimum permissible speed to a higher rpm based on the pump's POR, as shown on Figure 2. Pump affinity curves have been plotted along the pump's minimum and maximum flow POR curves between pump minimum and maximum speed. The affinity curve has also been plotted showing the pump's best efficiency point (BEP) at these speeds. Figure 2 shows that after the pump was started, the pump control logic maintained the pump performance within the maximum and minimum speeds (100 percent and 50 percent of rated speed) and within the POR. This is shown by the tracer plot identified as "model results" on Figure 2.

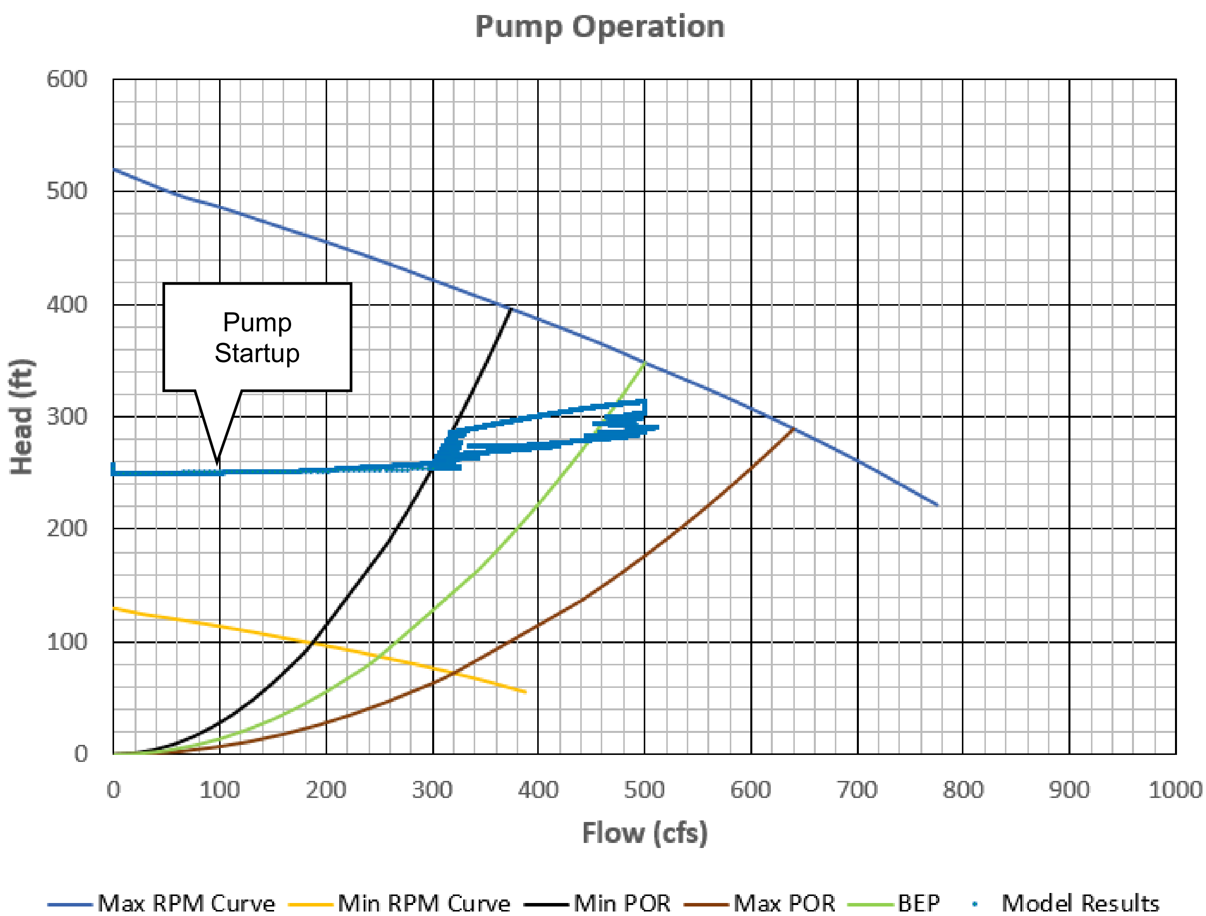

Figure 2. Example RTC Pump Operation Scatter Plot

2.4.2 RTC Rules

2.4.2.1 Initialize the Tunnel to the Preoperational State

In the model, all the tunnels, shafts, channels, and basins are assumed to be initially empty at the start of the simulation. Before the start of the operating simulations, the tunnel is filled to a predetermined HGL to match the preoperational boundary conditions for the specific model simulation. Inflow time series are loaded to the tunnel for the filling process. RTC rules are used to confirm the HGL stabilizes at the desired system water level, and the correct river WSELs are loaded to the intakes.

2.4.2.2 Simulated System Startup Operation

For this scenario, a total system-wide steady-state flow rate of 6,000 cfs was developed between the Sacramento River Intakes (C-E-3 and C-E-5) and the Bethany Reservoir. No flow was pumped to the C.W. "Bill" Jones (Jones) Pumping Plant Approach Channel. The RTCs for the project represent the operational state of any 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 on or off, and open or closed throughout the simulation. RTC rules 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 for this flow increase. 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 through intake control gates C-E-3 and C-E-5 are illustrated on Figure 3 for a system 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, C-E-3 opens from 0 to 45 minutes to reach 3,000 cfs, and C-E-5 opens from 45 to 90 minutes reach another 3,000 cfs, establishing the total system flow of 6,000 cfs in a total of 90 minutes.

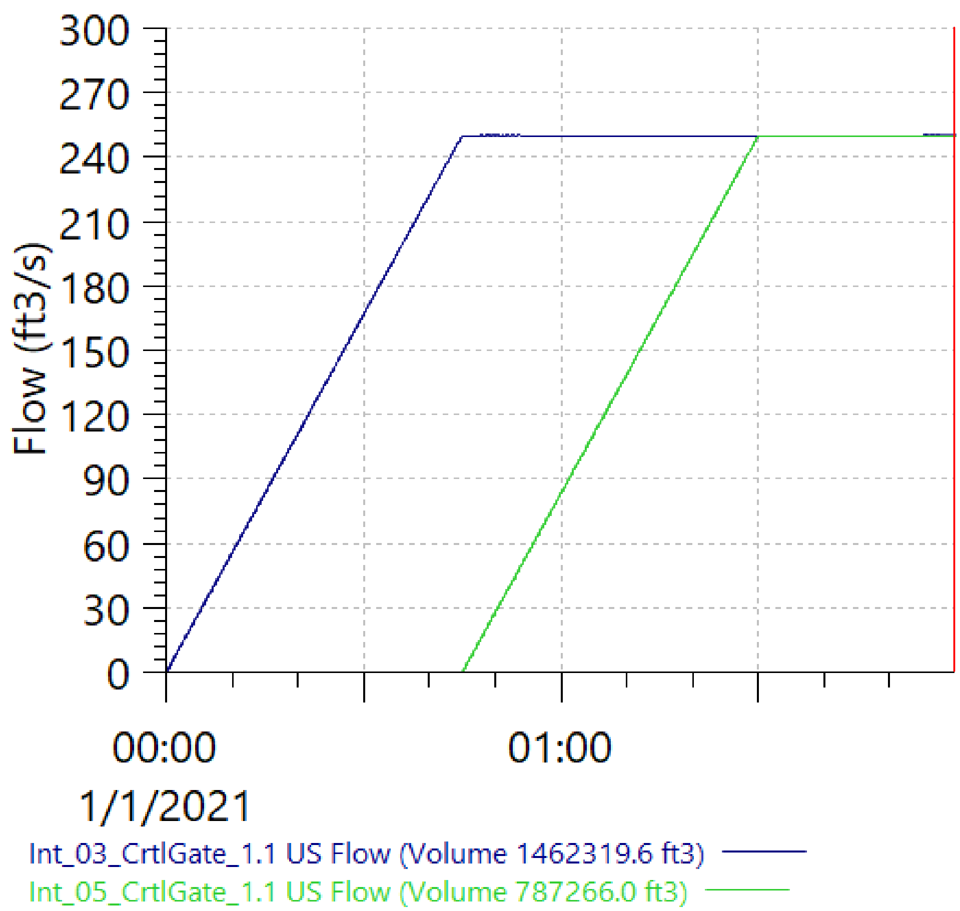

Figure 3. Intake Control Gate Startup Operation

For the main pumps (within the BRPP) the controlling criteria are based on four parameters: (1) the minimum permissible pump speed in revolutions per minute () line; (2) the maximum permissible pump speed rpm line; (3) the minimum flow POR affinity curve; and (4) the maximum flow POR affinity curve. These four parameters define the optimal performance envelop of the pumps. At each sampling time, the pumps' flow-total-dynamic-head operational state is compared for performance within this envelope (optimal operational region). 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 BRPP during the system startup, the BRPP's output flow is also limited to a flow increase rate of 1,000 cfs within 15 minutes. A flow-time curve was developed for this increase. 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 operating speed of the pump(s) is decreased, or the minimum permissible speed is maintained. Twelve identically sized pumps each with a design flow of 500 cfs at the pumps' rated head conditions were used to achieve the 6,000-cfs set-point capacity to the Bethany Reservoir.

At the beginning of the system startup operation, the lead pump in the BRPP 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 lowest permissible rpm (within the POR, and at or above the minimum established VFD speed). The minimum established VFD speed in the ICM model has been established as 50 percent of the maximum rated pump speed. After 60 seconds, the pump is maintained at its lowest permissible speed. 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 next pump in the lag pump sequence is then started. After 60 seconds, both pumps operate at identical speeds (lowest possible speed within their POR based on system head conditions).

Similarly, when the combined flow output of both pumps is below the flow-time curve, the next pump in the sequence is started. Eventually, up to twelve pumps would be in operation as required for the flow-time curve to establish the 6,000-cfs design flow condition to the Bethany Reservoir and the total pumped flow will match the flow-time curve. Each pump's operating speed would continue increasing in unison until the total pumped flow reaches the set-point total flow. It should be noted that the number of duty pumps will vary depending on the targeted set-point for Sacramento River diversion flow. The total controlled pumping flow to the Bethany Reservoir is illustrated on Figure 4 for the startup operation. The startup process for conveying pumped flows to the Jones Pumping Plant's approach channel is performed in an identical manner.

As can be seen in Figures 3 and 4 this control methodology produces a stable start-up of the intakes and BRPP.

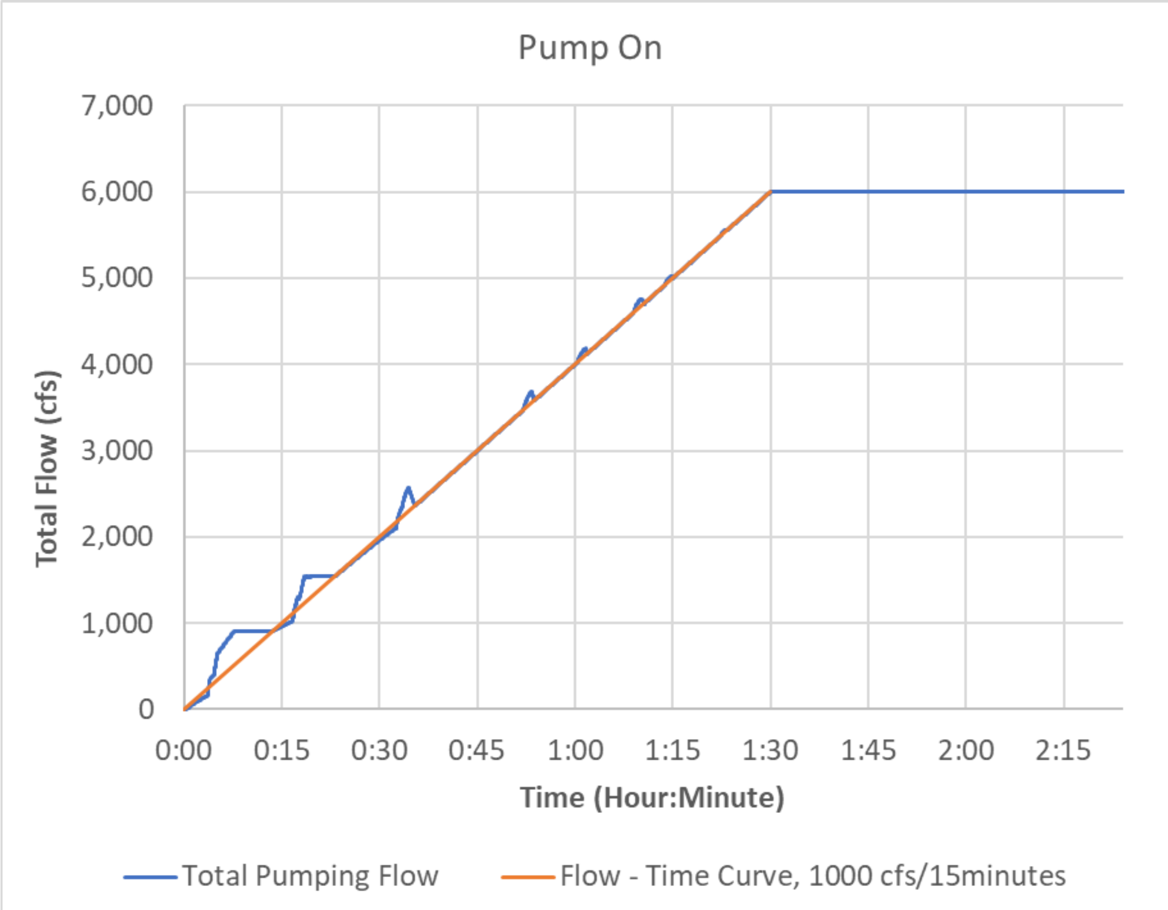

Figure 4. Real-time Control on Pump Startup Operation

2.4.2.3 Simulated System Operation

During simulated system operation, the RTCs operate the gates at each Sacramento River Intake (in operation) and the main pumps within the BRPP such that they are maintained within their required performance envelope and the target system flow set-point is achieved and maintained. 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 Sacramento River WSEL. If the differential level deviates between the Sacramento 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. For the pumps, the RTC compares output flows with the targeted flows and adjust each pump speed as necessary.

2.4.2.4 Simulated System Shutdown Operation

For the intakes, 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 opening is decreased.

The controlled flows for C-E-3 and C-E-5 are illustrated on Figure 5 for the systemwide shutdown operation, starting with a total system flow of 6,000 cfs. 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, C-E-5 closes from 0 to 45 minutes to reach 0 cfs from 3,000 cfs linearly, while C-E-3 closes from 45 to 90 minutes to reach 0 cfs from 3,000 cfs linearly.

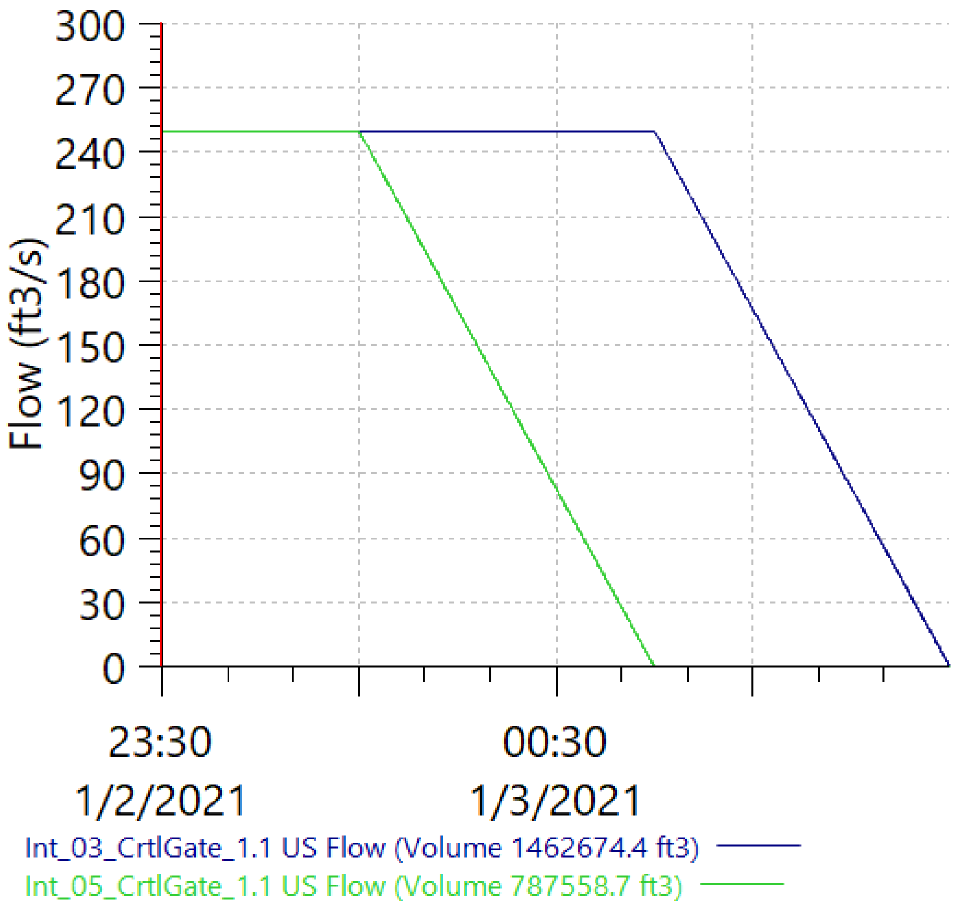

Figure 5. Intake Control Gate Shutdown Operation

In order to synchronize the operation between the intake and the BRPP, the total pumped flow is decreased at a 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 speeds of the pumps are decreased. All pumps decrease their rotational speeds gradually, so the total pumping flow matches the flow-time curve. Eventually, all the pumps will reach their lowest permissible speeds (within their POR), and the total pumping flow is more than the flow-time curve. When this condition occurs, a pump will be shut down. The pump shutdown time is 60 seconds. Because the pump speeds are limited by their permissible speed range (to maintain operation within their respective POR), the total flow cannot exactly match the flow-time curve but follows the general trend. The rate of change of the total flow versus time (1,000 cfs per 15 minutes) has been plotted against the output flow versus time for the controlled pump shutdown sequence is shown on Figure 6.

As can be seen on Figures 5 and 6, this control methodology produces stable synchronized, shutdown of the intakes and the BRPP.

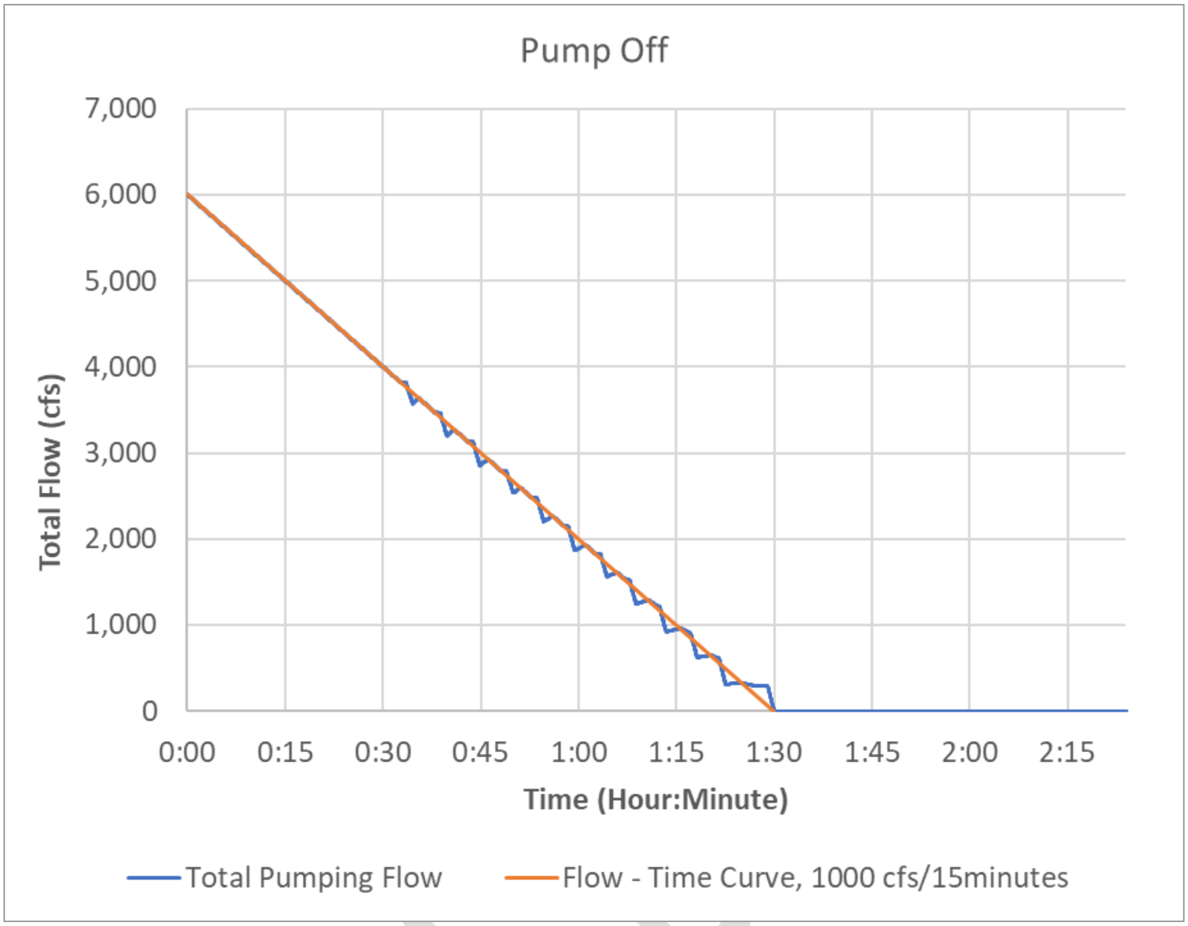

Figure 6. Real-time Control on Pump Shutdown Operation

2.4.3 Operational Cycle Study

2.4.3.1 Model Setup

This study includes a combination of several RTC scenarios under static boundary conditions, including pre-operational 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.

2.4.3.2 Model Simulation Results

The individual pump startup and the corresponding wet well water level variations are presented on Figures 7/8 and 9/10 for low and high river levels and tunnel friction factors using manning's *n* coefficients of 0.016 and 0.014, respectively. The figures show the downstream pump discharge pipe flow and wet well level. Wet well level is shown on the graph as "Height Above Datum (ft)", which is the wet well WSEL. For the RTC operation simulations, the initial tunnel level for low river was assumed to be 3 feet and a river level of 7 feet. For the high river scenario, the initial tunnel level is 22 feet and river level of 28.2 feet and the plots show a smooth transition between system start-up and the designated steady state operating condition. Results were consistent for all scenarios.

The individual pump shutdown and the corresponding wet well water level variations are presented on Figures 11/12 and 13/14 for low and high river levels at manning's *n* coefficients of 0.016 and 0.014, respectively. The plots show the system shutdown operation at the BRPP also has smooth transitions for all scenarios. After all pumps are shut down, the wet well WSEL oscillates for an extended period. The oscillations are minor and dissipate quickly upon restart of the BRPP pumps.

The tunnel shafts' WSEL variations during the startup operation is presented on Figures 15/16 and 17/18 for low and high river levels and tunnel friction factors using manning's *n* coefficients of 0.016 and 0.014, respectively. All shafts are represented at varying levels on the graph indicated in the legend. The shafts show slight increases during startup due to differential head difference between the shafts as pumps come online. The plots show the system start-up operation has smooth transitions in these shafts.

The tunnel shafts' WSEL variations during the shutdown operation is presented on Figures 19/20 and 21/22 for low and high river levels and tunnel friction factors using manning's *n* coefficients of 0.016 and 0.014, respectively. All shafts are represented at varying levels on the graph indicated in the legend. The plots show the system shutdown operation has smooth transitions in these shafts. After the system is shutdown, the water surfaces do not stabilize immediately and oscillate for an extended period, as described above.

Figure 23 through 38 show the water levels and flows at various points within C-E-3 and C-E-5. The figures include water levels at the fish screen, control gate, box conduit, sedimentation basin, radial gates, and intake outlet shaft. The flow shown on the graphs is at the downstream end of the control gate. Further descriptions of results are provided here.

The C-E-3 flow and water level variations during the start-up operation is presented on Figures 23/24 and 25/26 for low and high river levels and tunnel friction factors using manning's *n* coefficients of 0.016 and 0.014, respectively. The plots show diversion capacity set-point was achieved based on the diversion rate.

The C-E-5 flow and water level variations during the start-up operation is presented on Figures 27/28 and 29/30 for low and high river levels and tunnel friction factors using manning's n coefficients of 0.016 and 0.014, respectively. The plots show diversion capacity set-point was achieved based on the diversion rate.

The C-E-3 flow and water level variations during the shutdown operation are presented on Figures 31/32 and 33/34 low and high river levels and tunnel friction factors using manning's n coefficients of 0.016 and 0.014, respectively. The plots show the system shutdown operation has smooth transitions.

The C-E-5 flow and water level variations during the shutdown operations are presented on Figures 35 through 38 for low and high river levels and tunnel friction factors using manning's n coefficients of 0.016 and 0.014, respectively. The plots show the system shutdown operation has smooth transitions.

The minimum and maximum WSEL profiles along the tunnel are presented on Figure 39 and 40 for low and high river levels and tunnel friction factors using manning's n coefficients of 0.016 and 0.014, respectively.

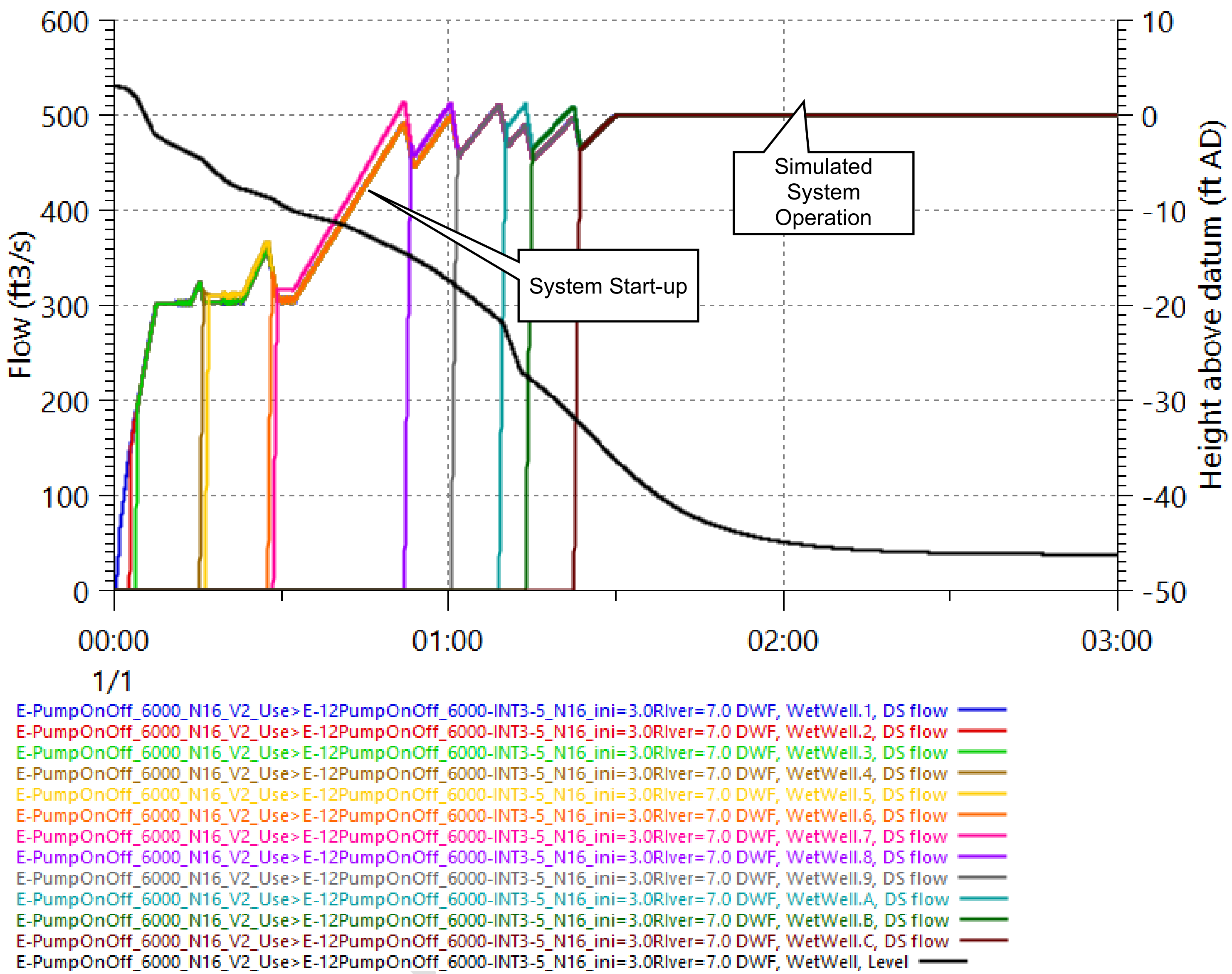

Figure 7. Pump Startup and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.016

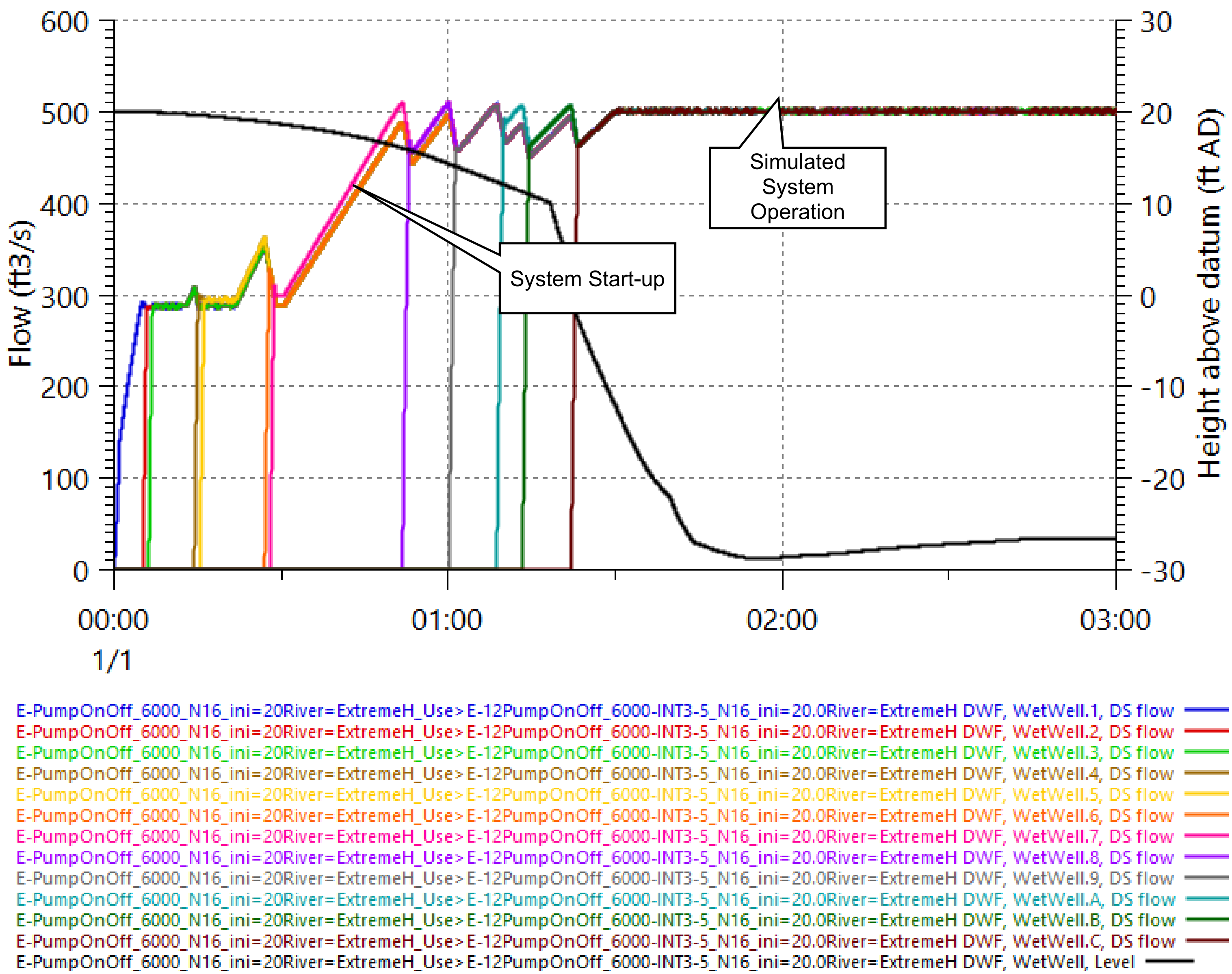

Figure 8. Pump Startup and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.016

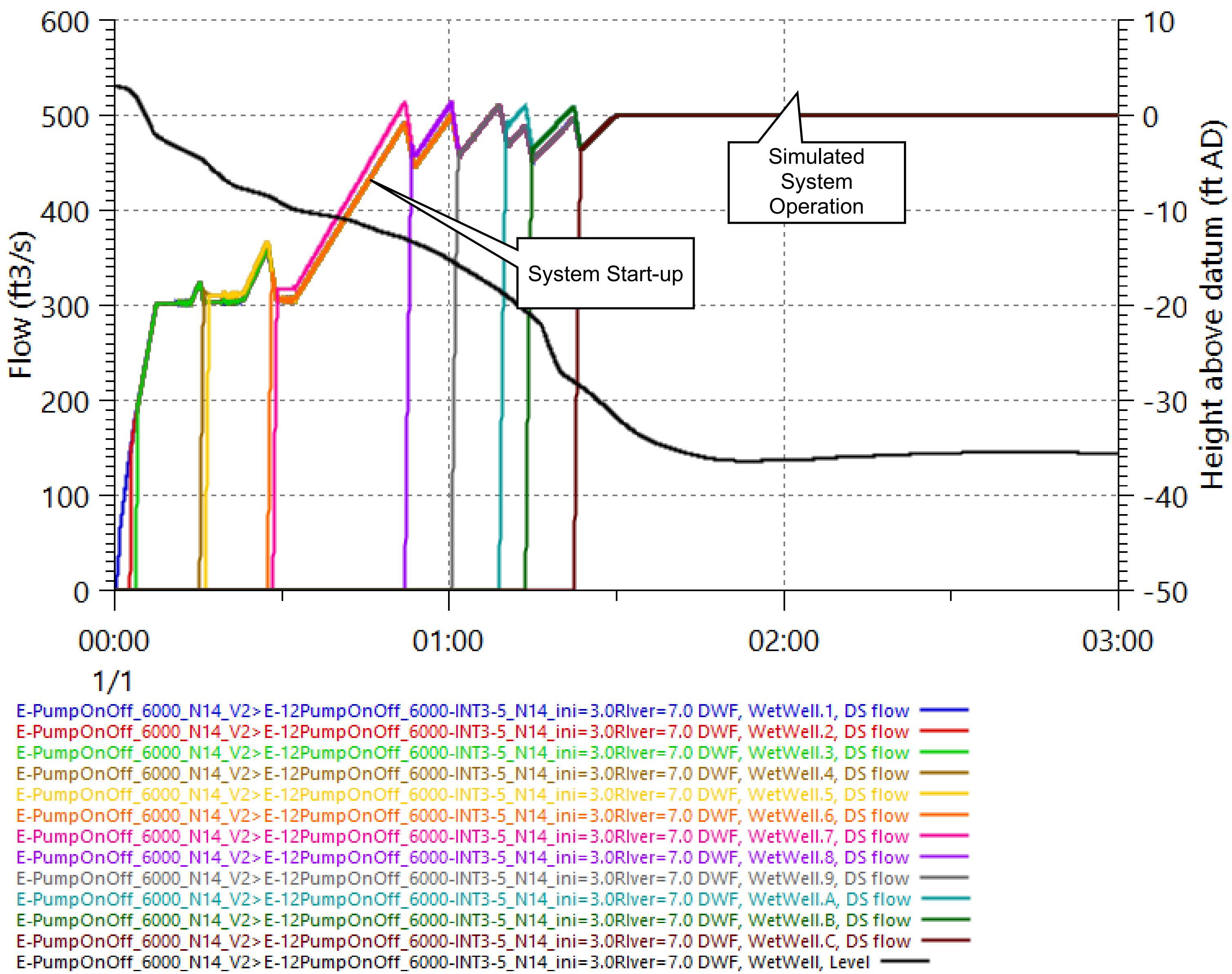

Figure 9. Pump Startup and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.014

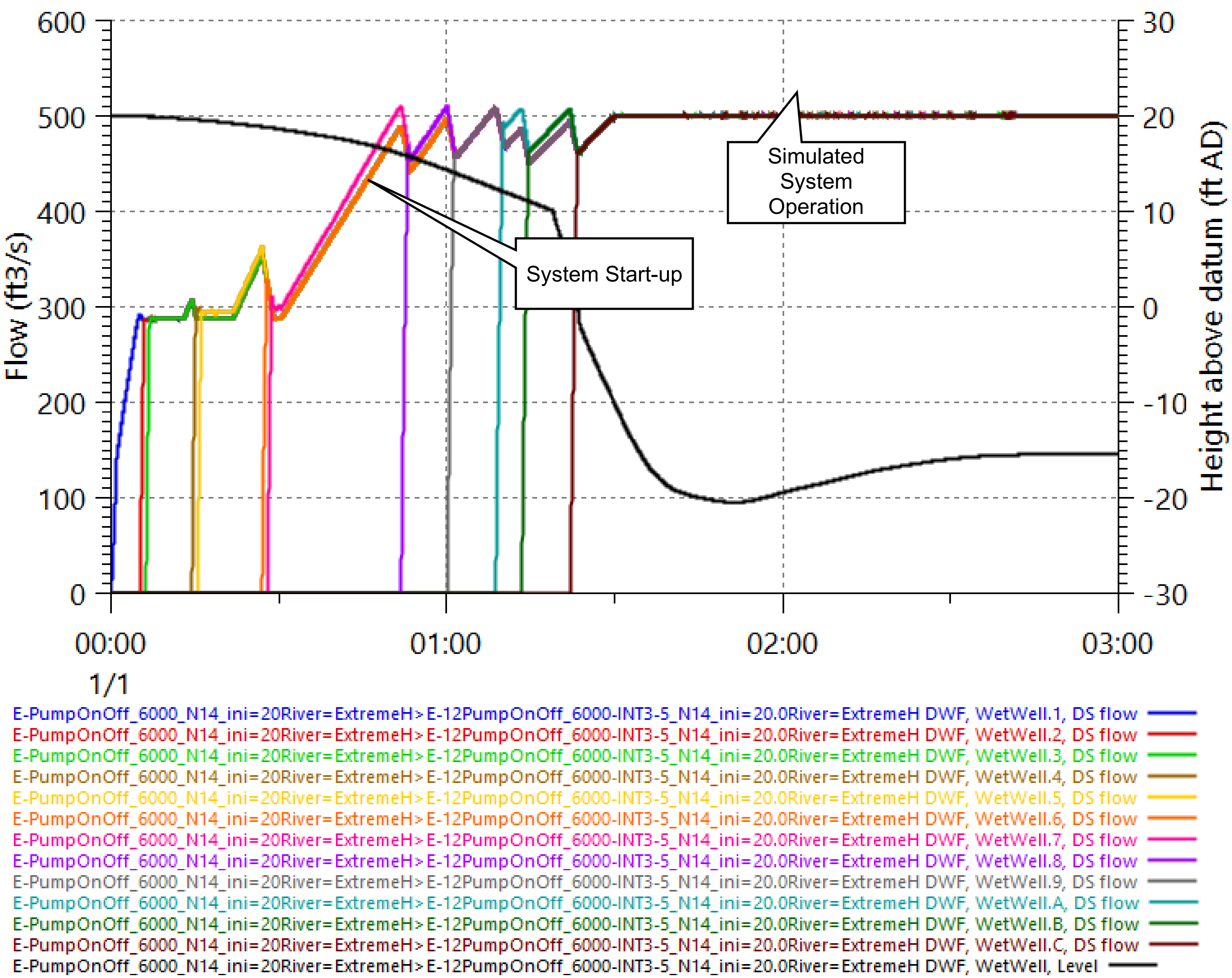

Figure 10. Pump Startup and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.014

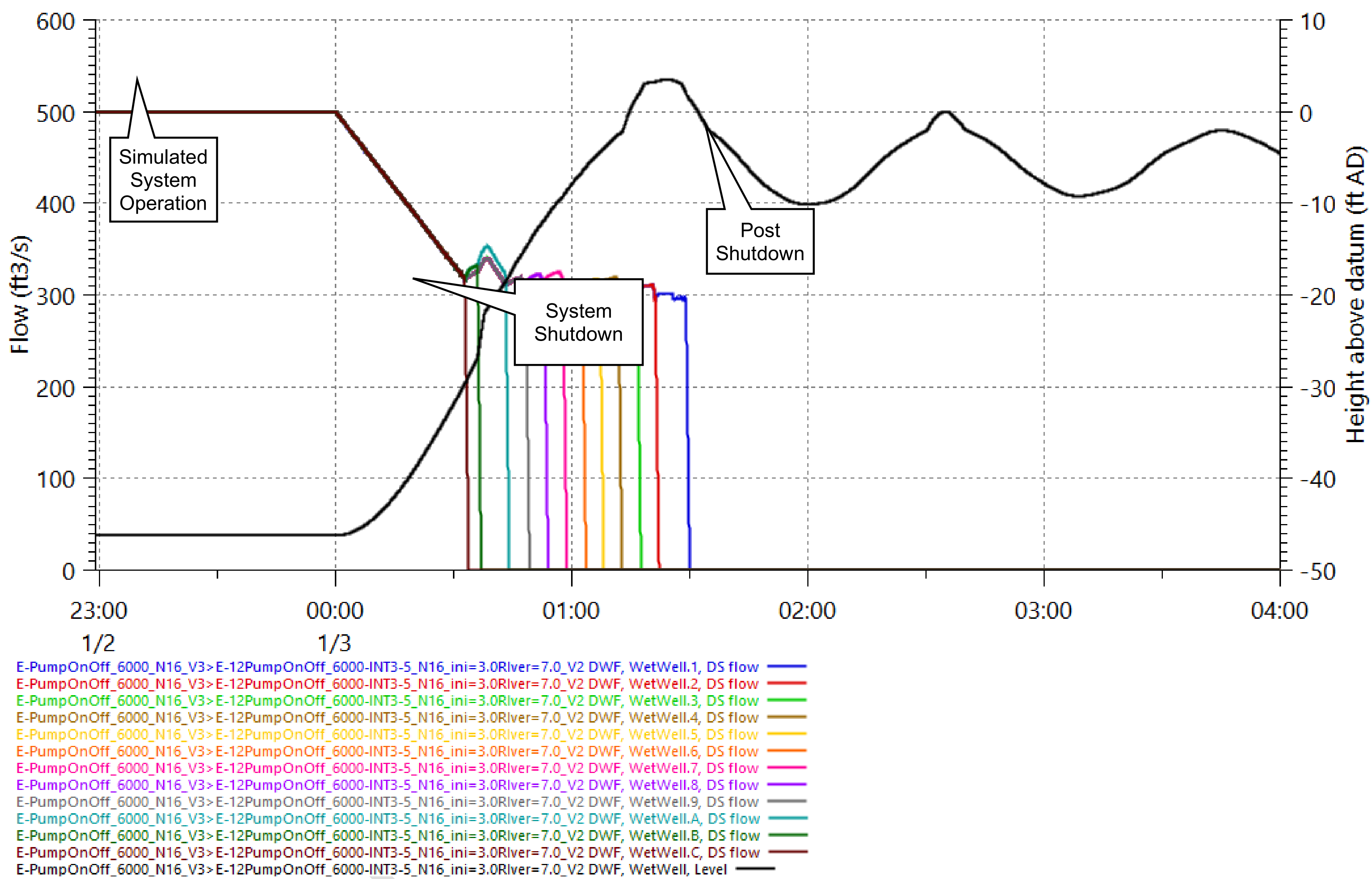

Figure 11. Pump Shut Down and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.016

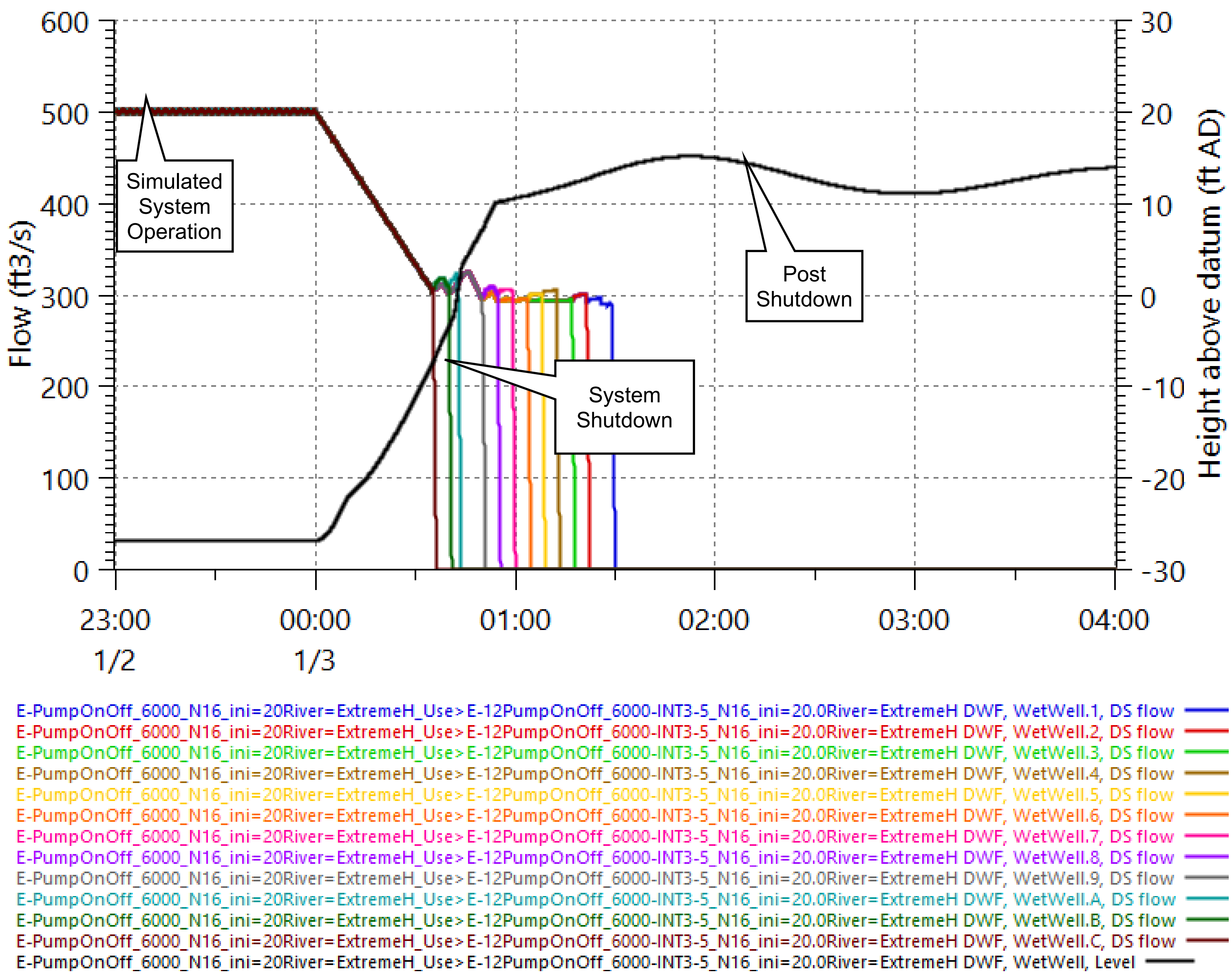

Figure 12. Pump Shut Down and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.016

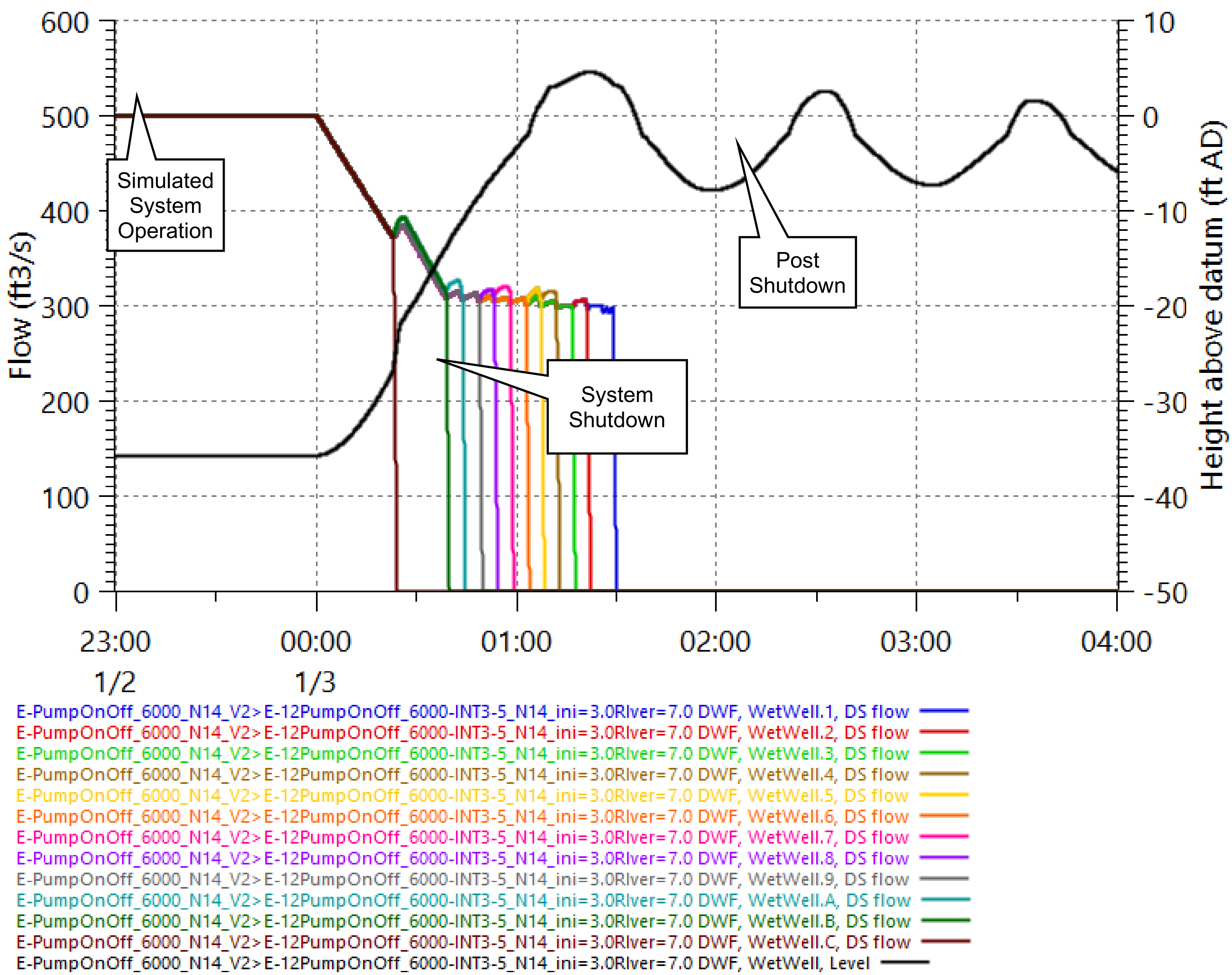

Figure 13. Pump Shut Down and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.014

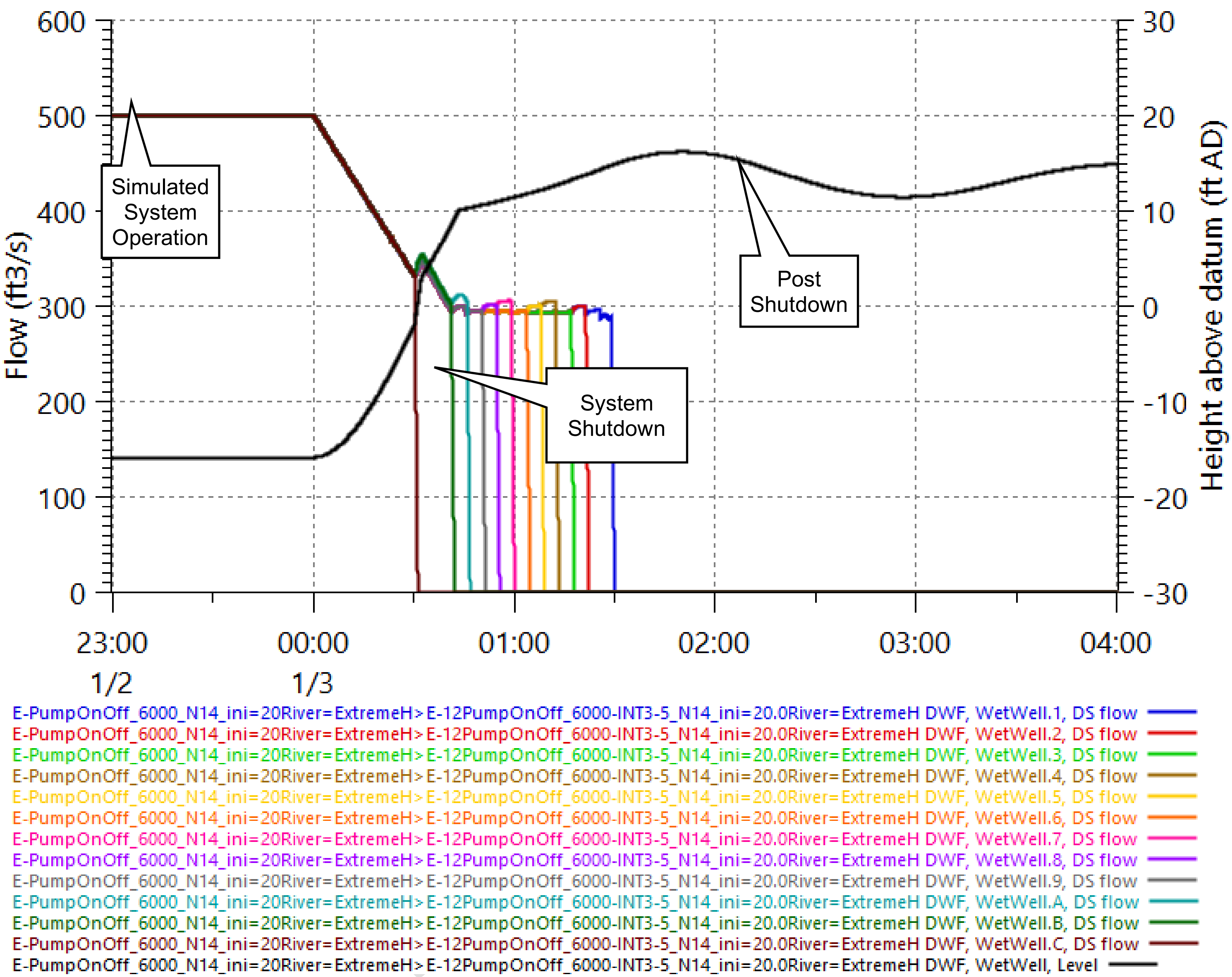

Figure 14. Pump Shut Down and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.014

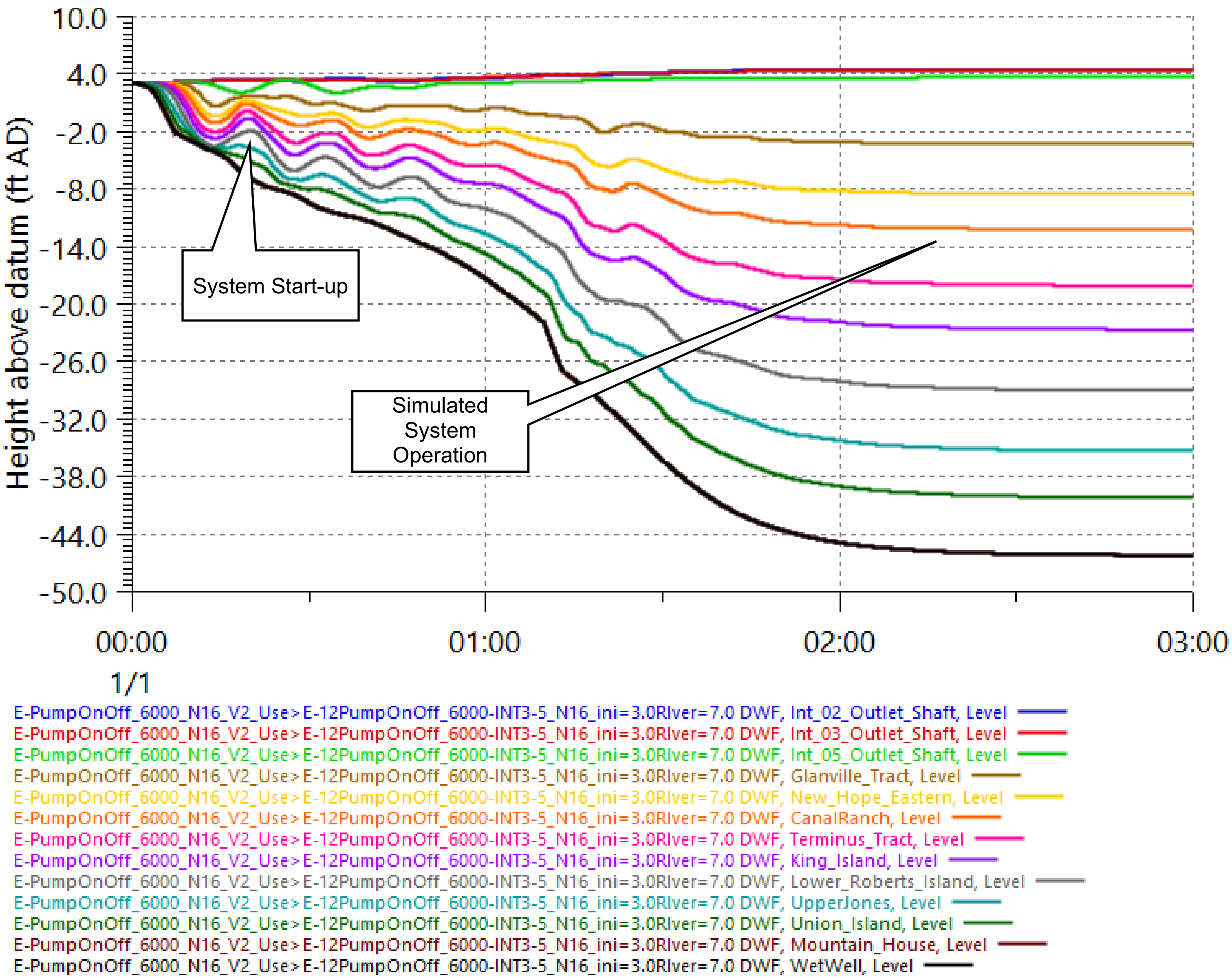

Figure 15. Tunnel Shaft Water Levels during System Startup, 6000 cfs, Low River, Manning's n= 0.016

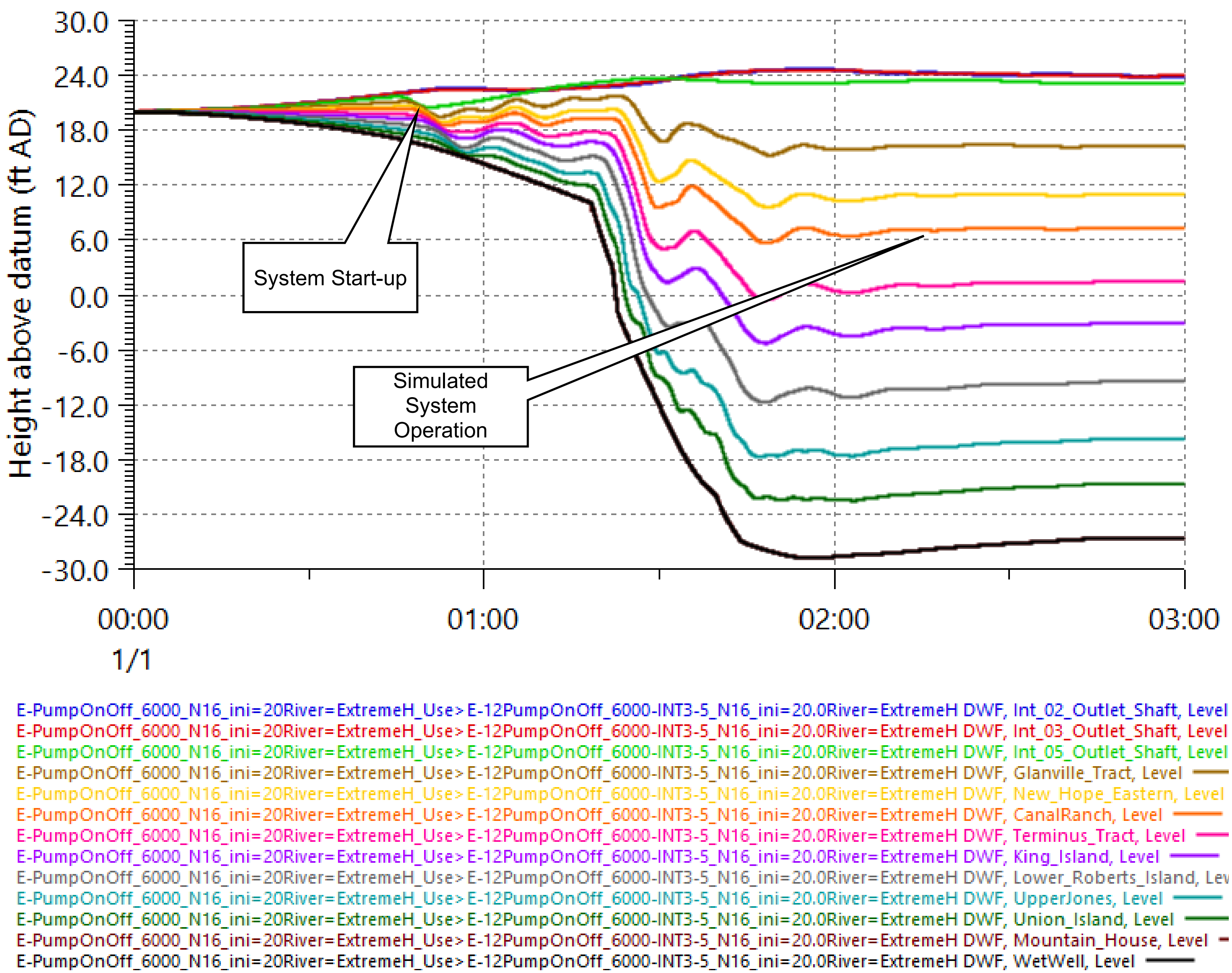

Figure 16. Tunnel Shaft Water Levels during System Startup, 6000 cfs, High River, Manning's n= 0.016

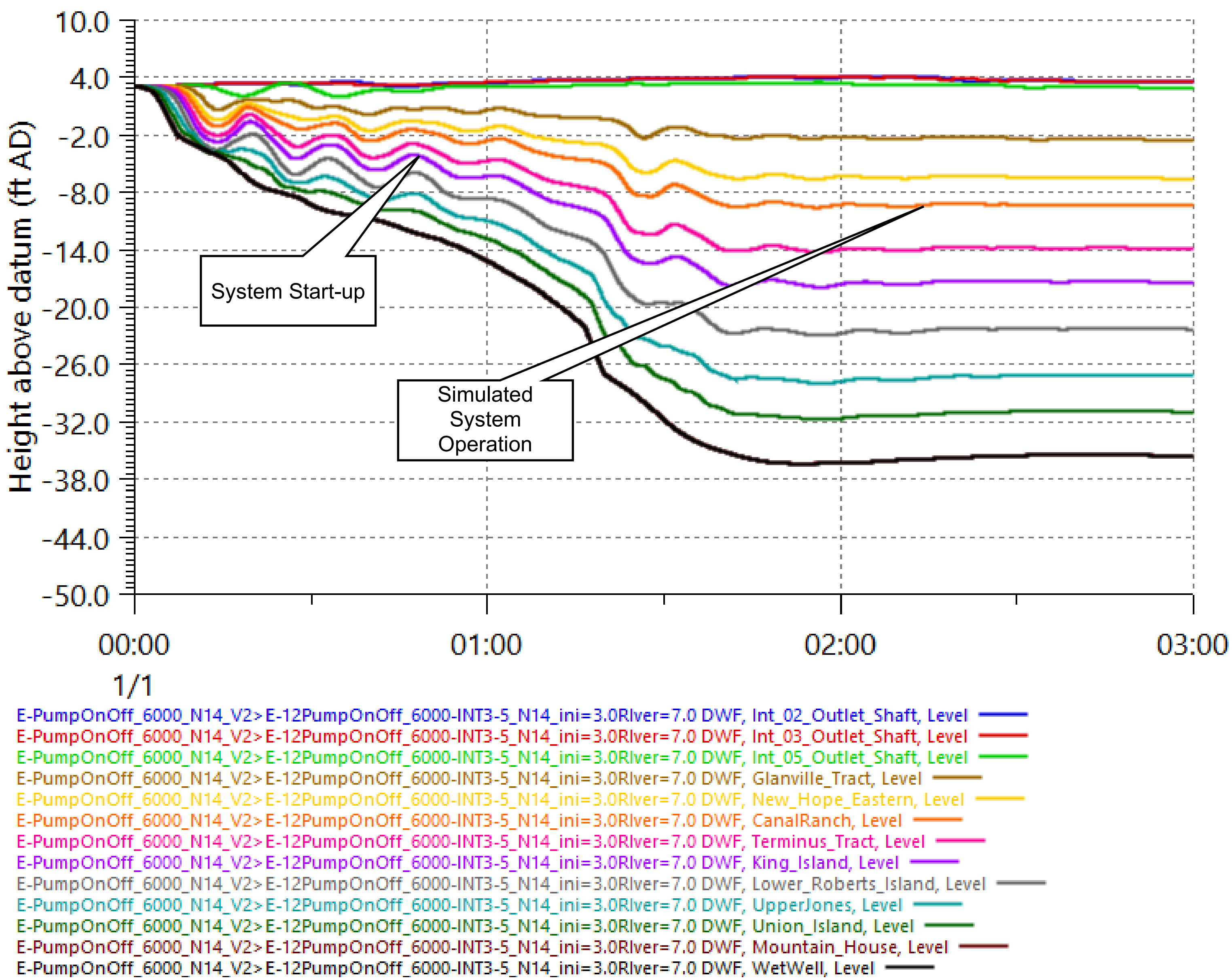

Figure 17. Tunnel Shaft Water Levels during System Startup, 6000 cfs, Low River, Manning's n= 0.014

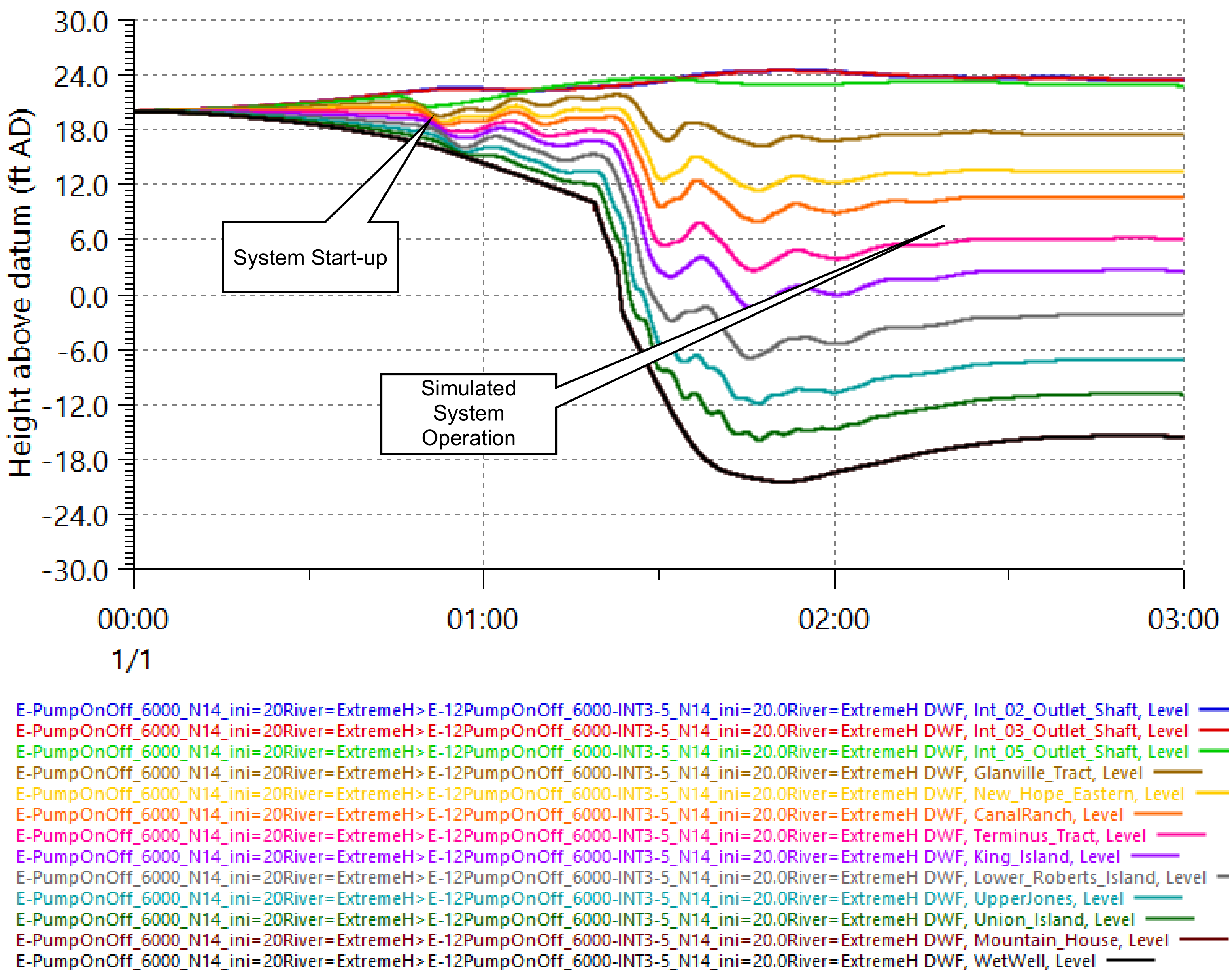

Figure 18. Tunnel Shaft Water Levels during System Startup, 6000 cfs, High River, Manning's n= 0.014

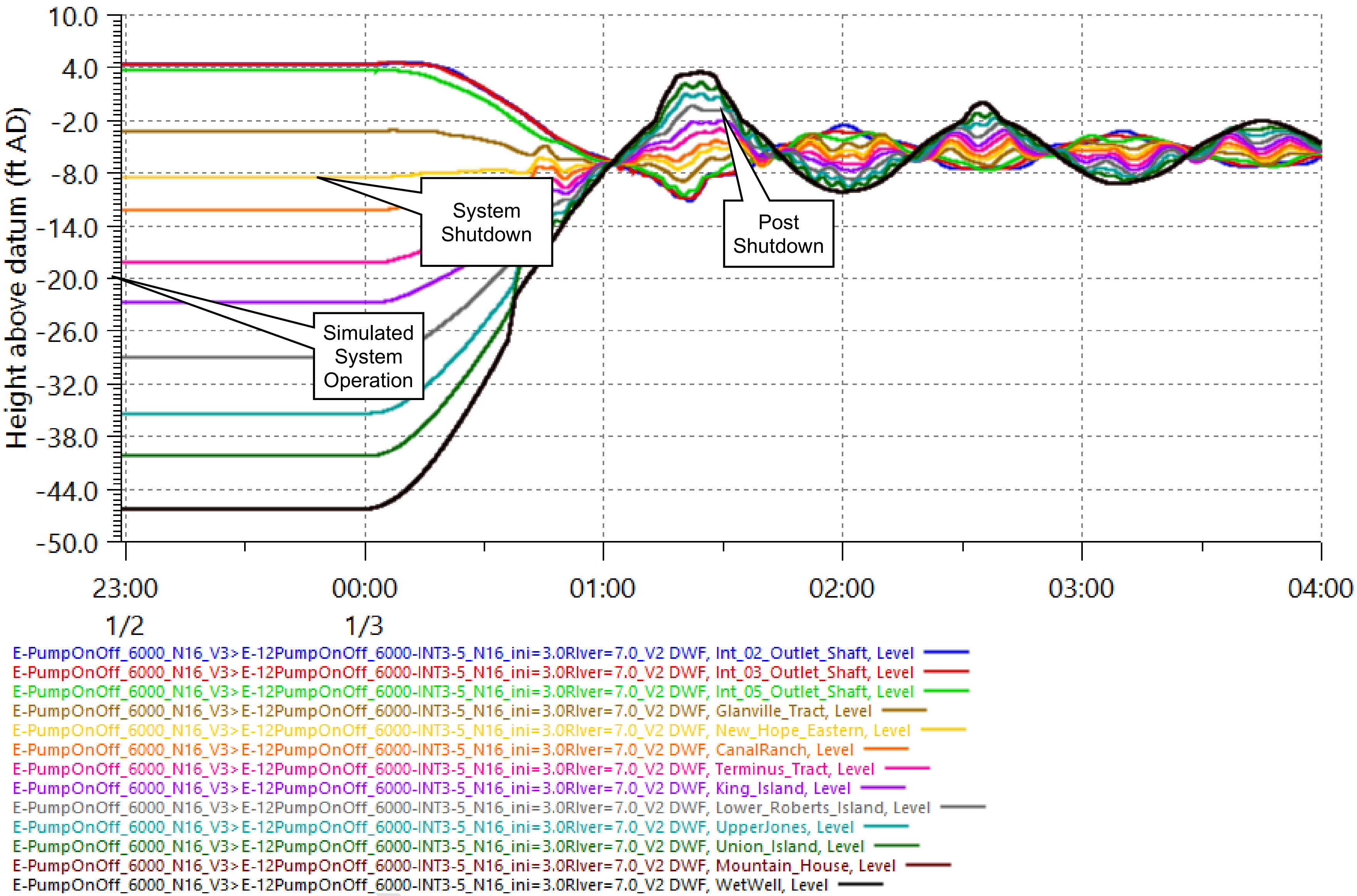

Figure 19. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, Low River, Manning's n= 0.016

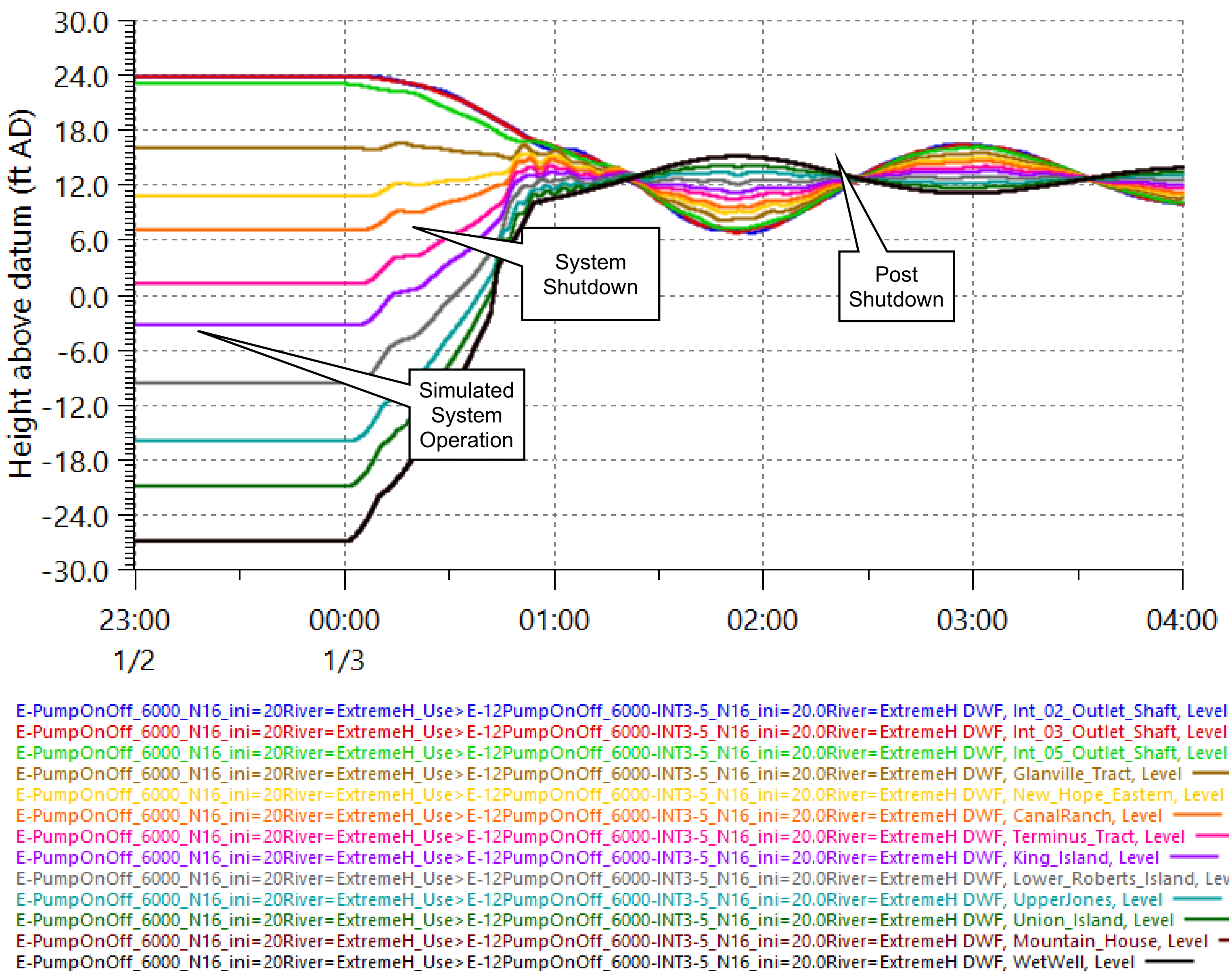

Figure 20. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, High River, Manning's n= 0.016

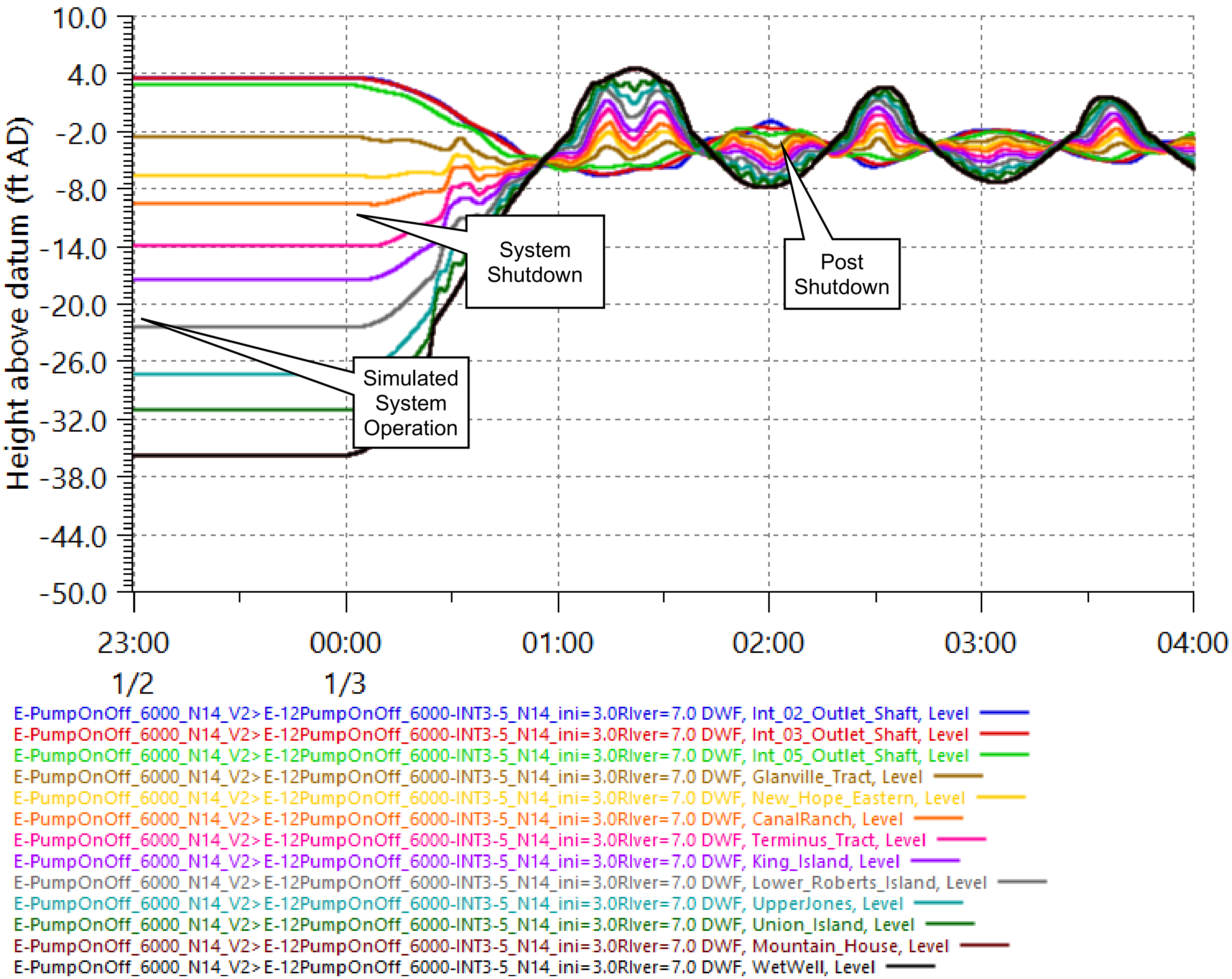

Figure 21. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, Low River, Manning's n= 0.014

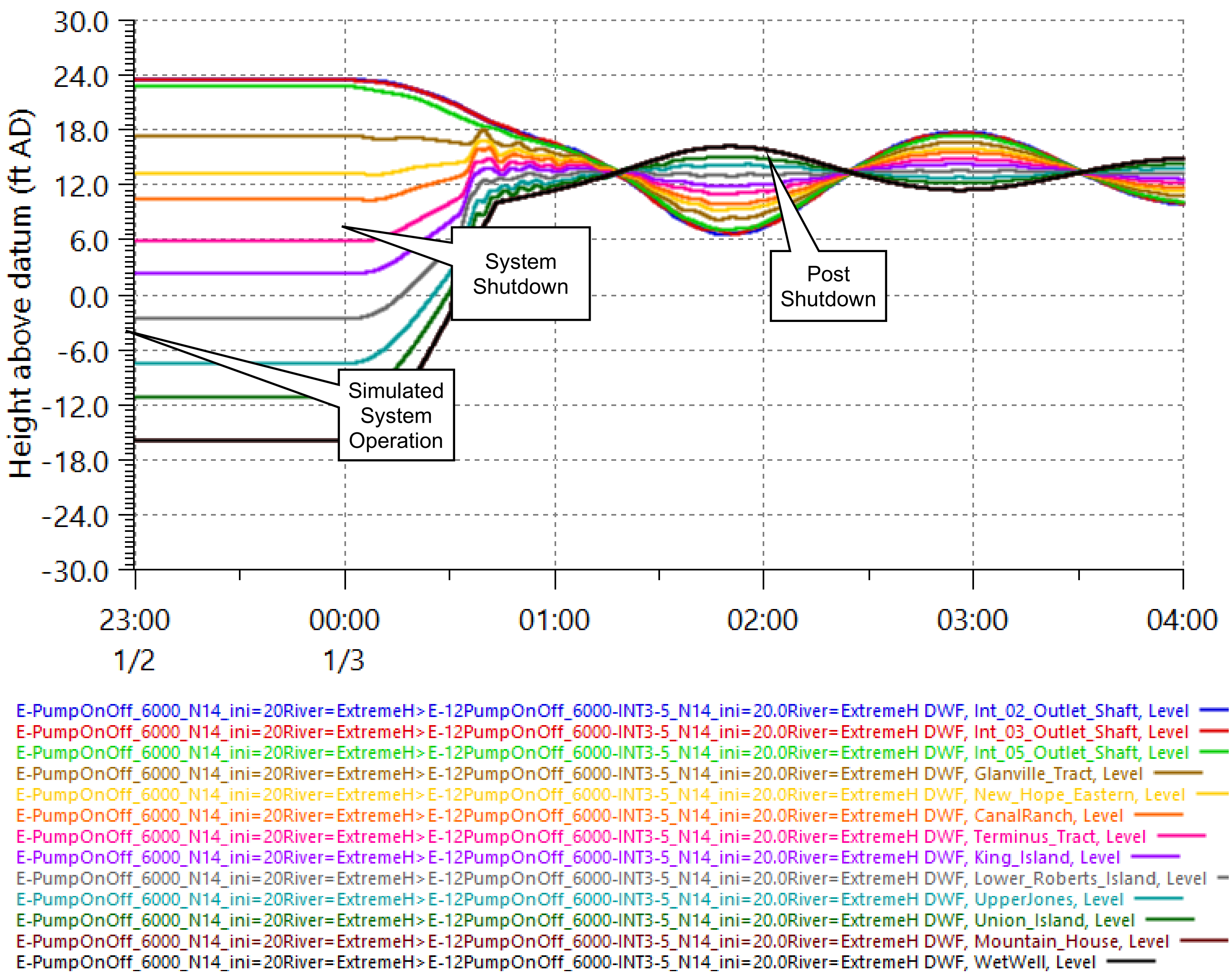

Figure 22. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, High River, Manning's n= 0.014

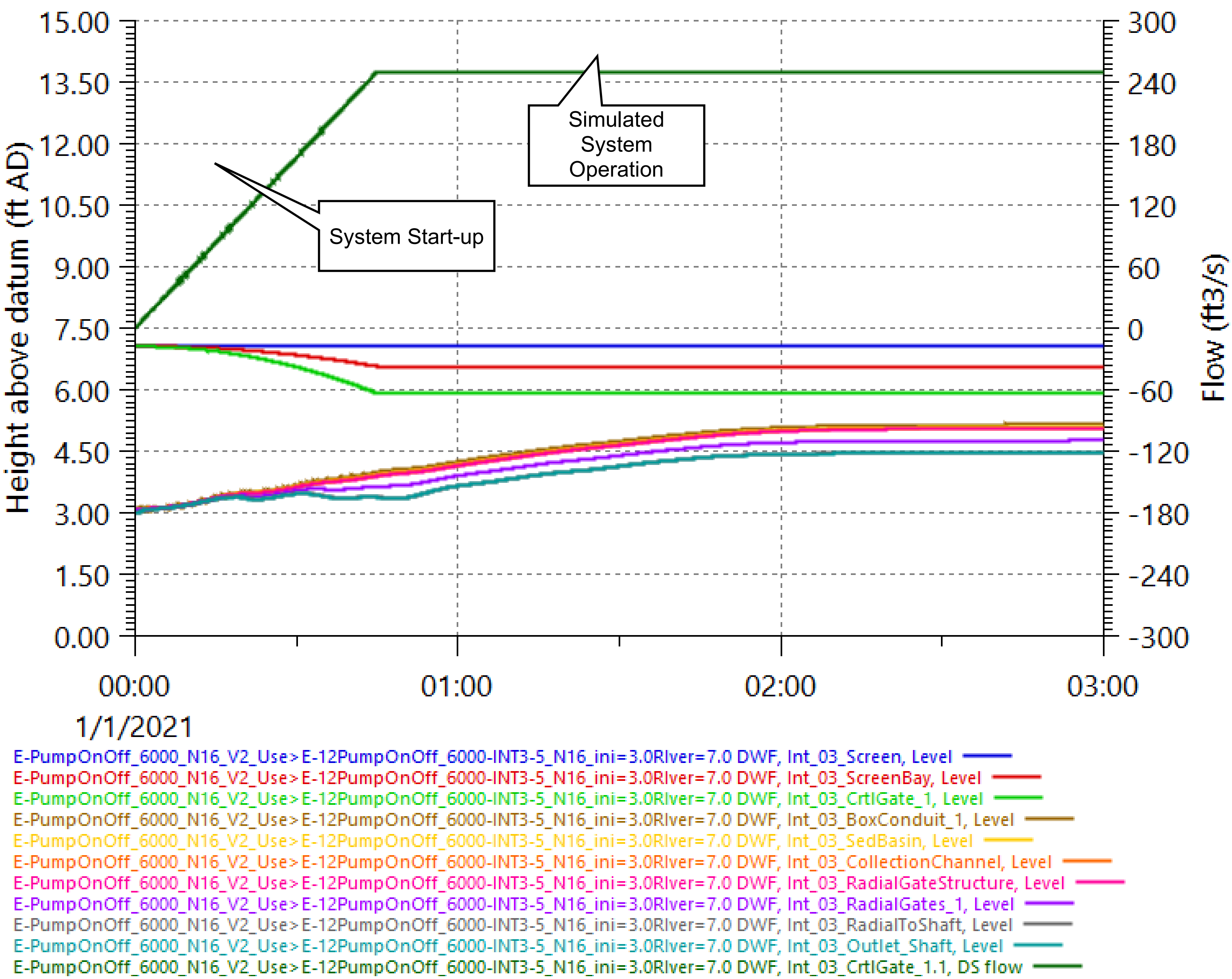

Figure 23. C-E-3 Flow and Water Levels during System Startup, Low River, Manning's n= 0.016

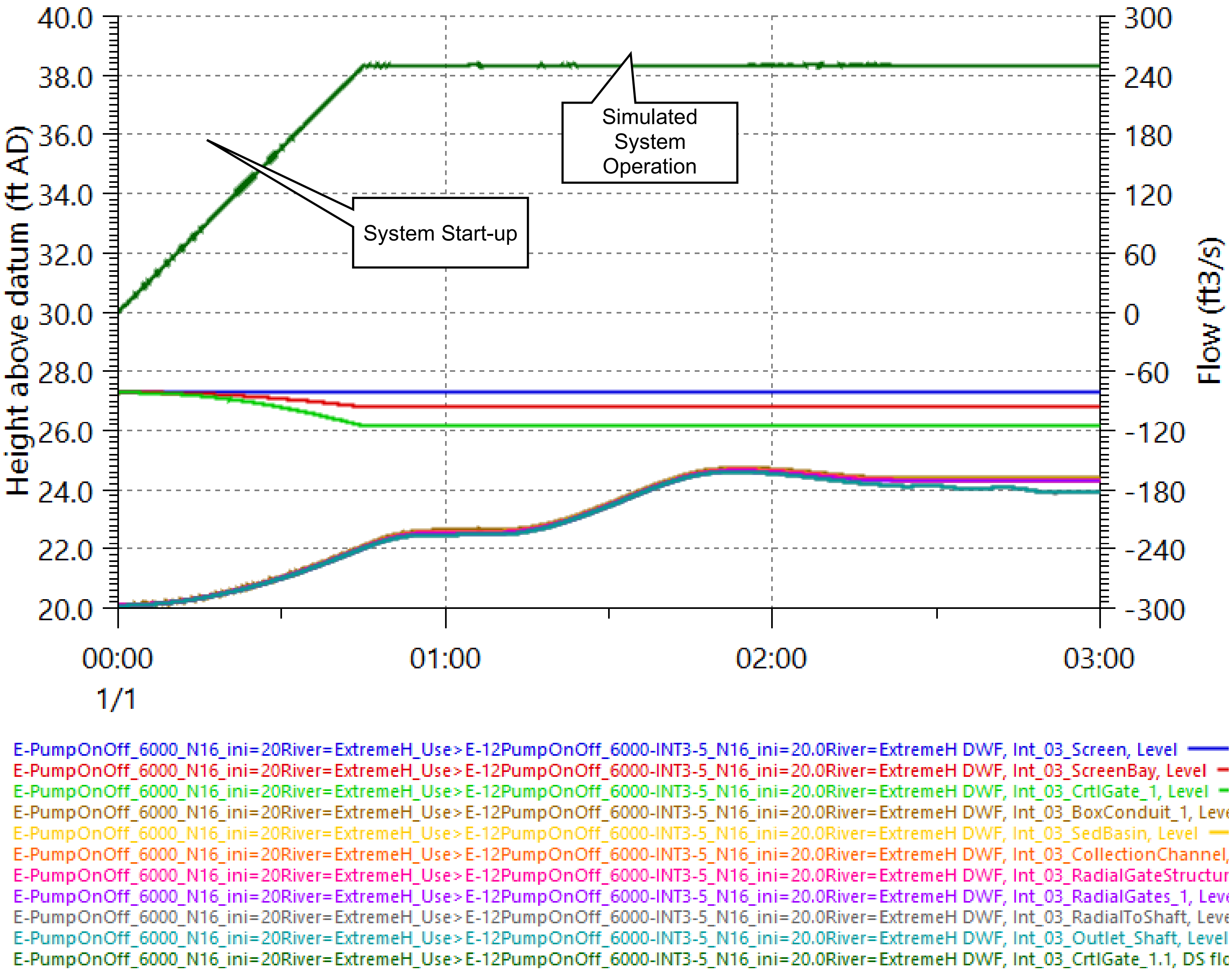

Figure 24. C-E-3 Flow and Water Levels during System Startup, High River, Manning's n= 0.016

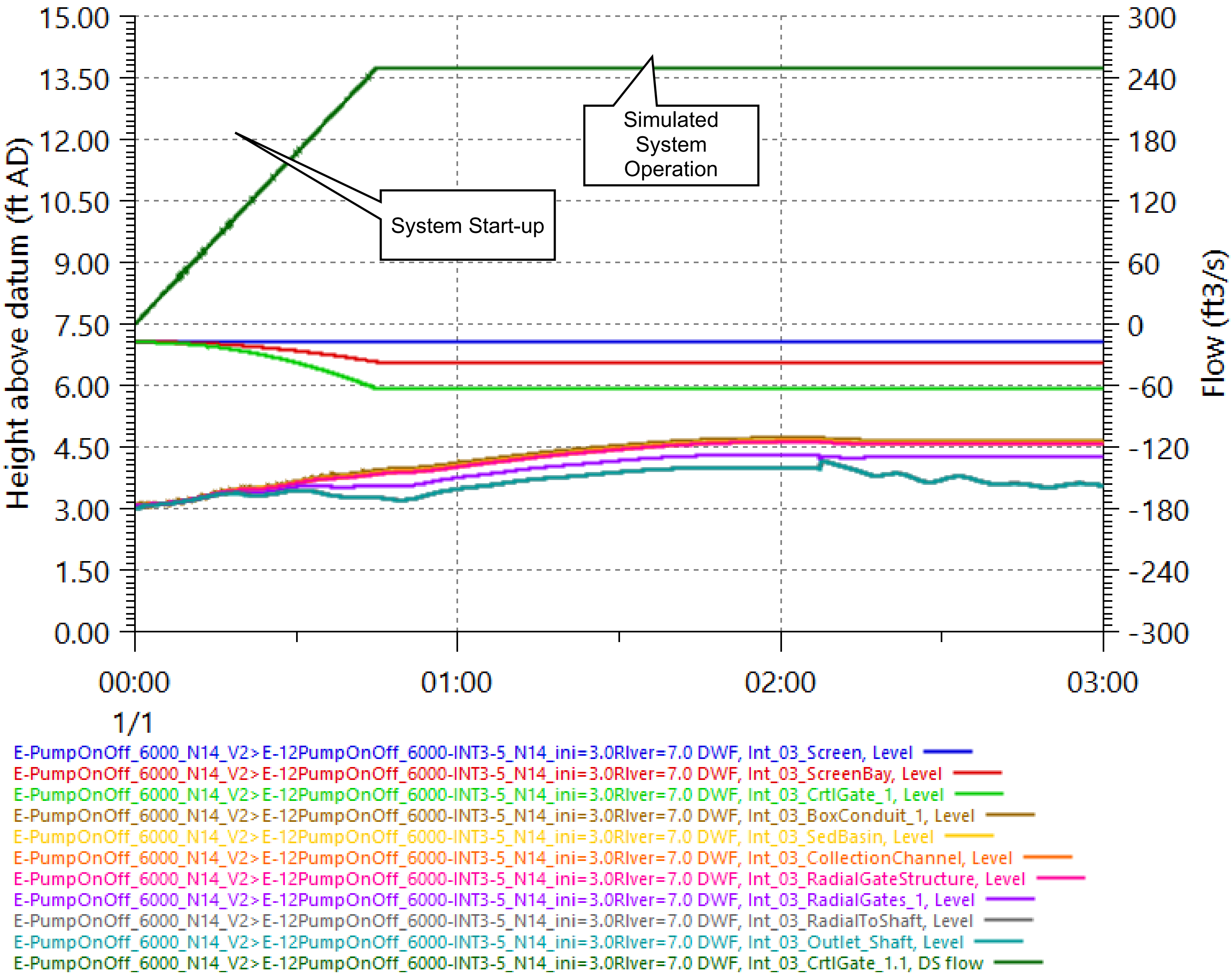

Figure 25. C-E-3 Flow and Water Levels during System Startup, Low River, Manning's n= 0.014

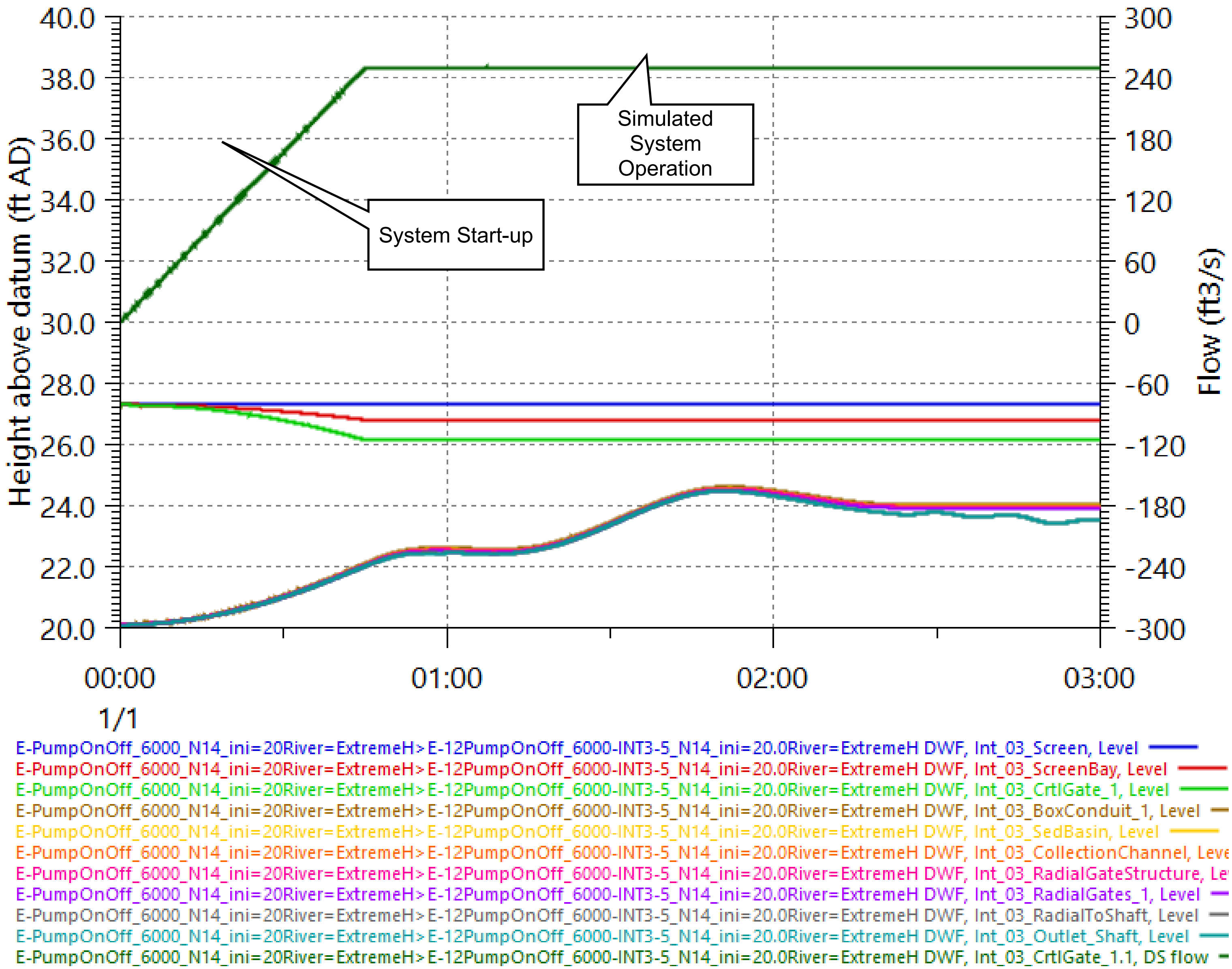

Figure 26. C-E-3 Flow and Water Levels during System Startup, High River, Manning's n= 0.014

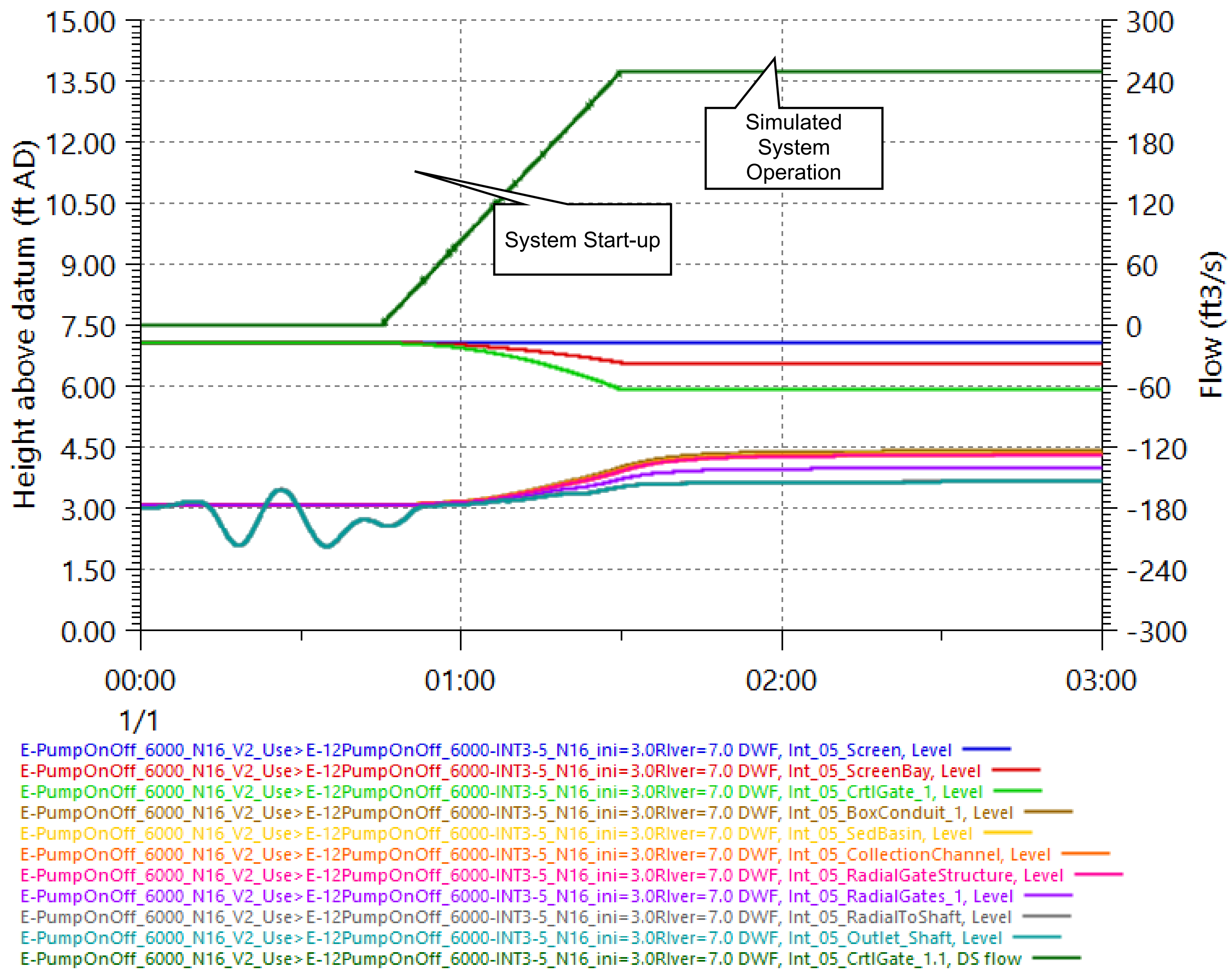

Figure 27. C-E-5 Flow and Water Levels during System Startup, Low River, Manning's n= 0.016

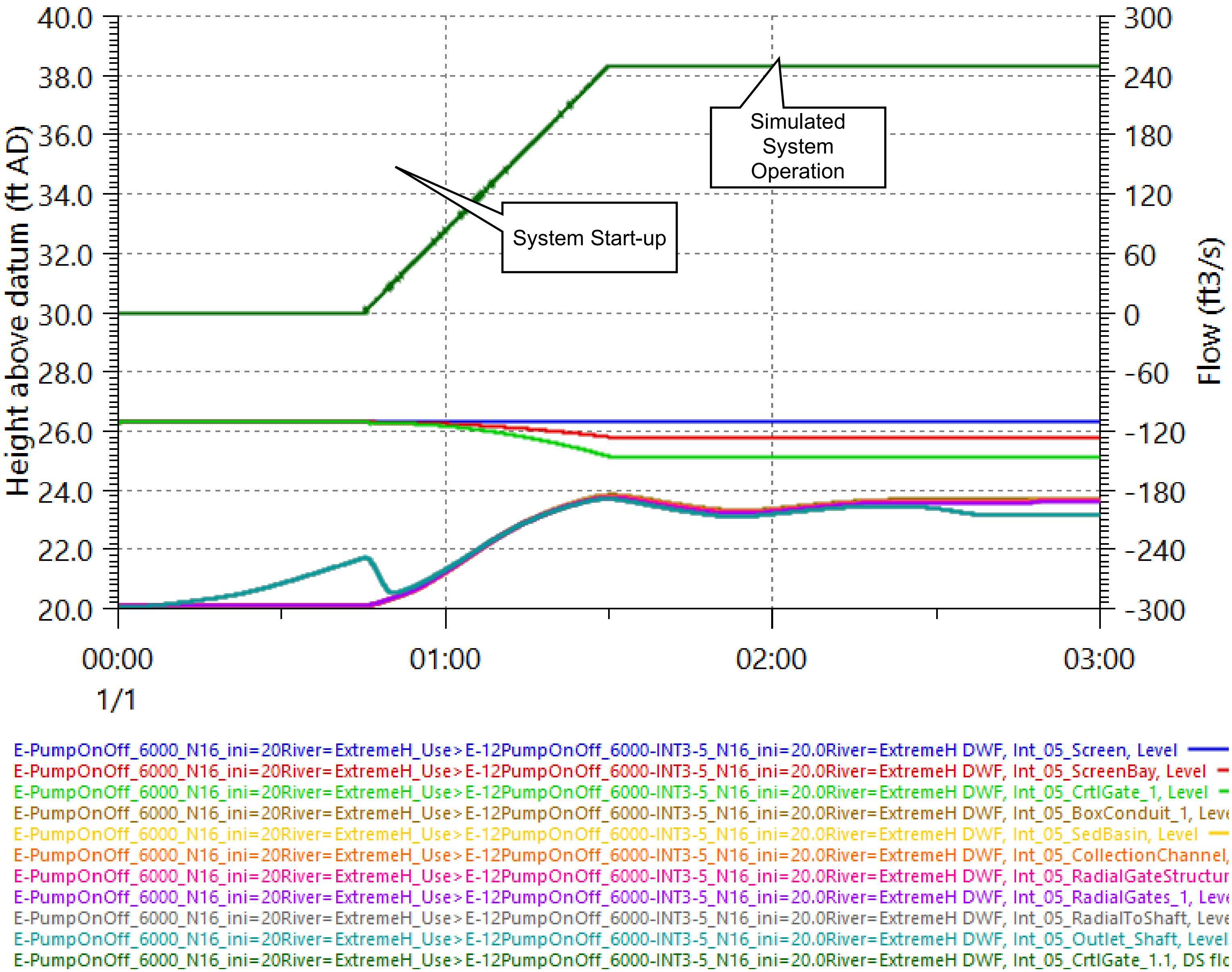

Figure 28. C-E-5 Flow and Water Levels during System Startup, High River, Manning's n= 0.016

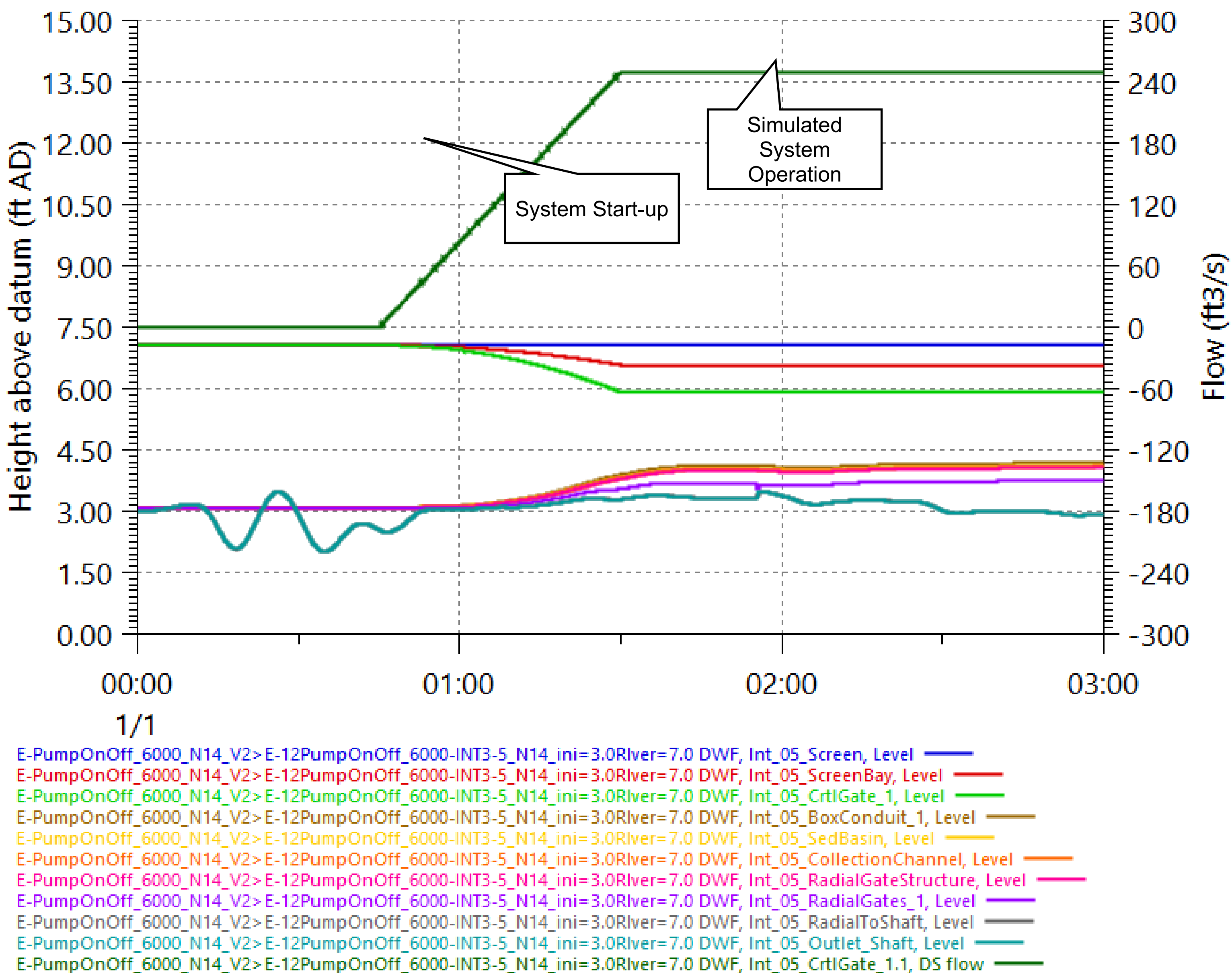

Figure 29. C-E-5 Flow and Water Levels during System Startup, Low River, Manning's n= 0.014

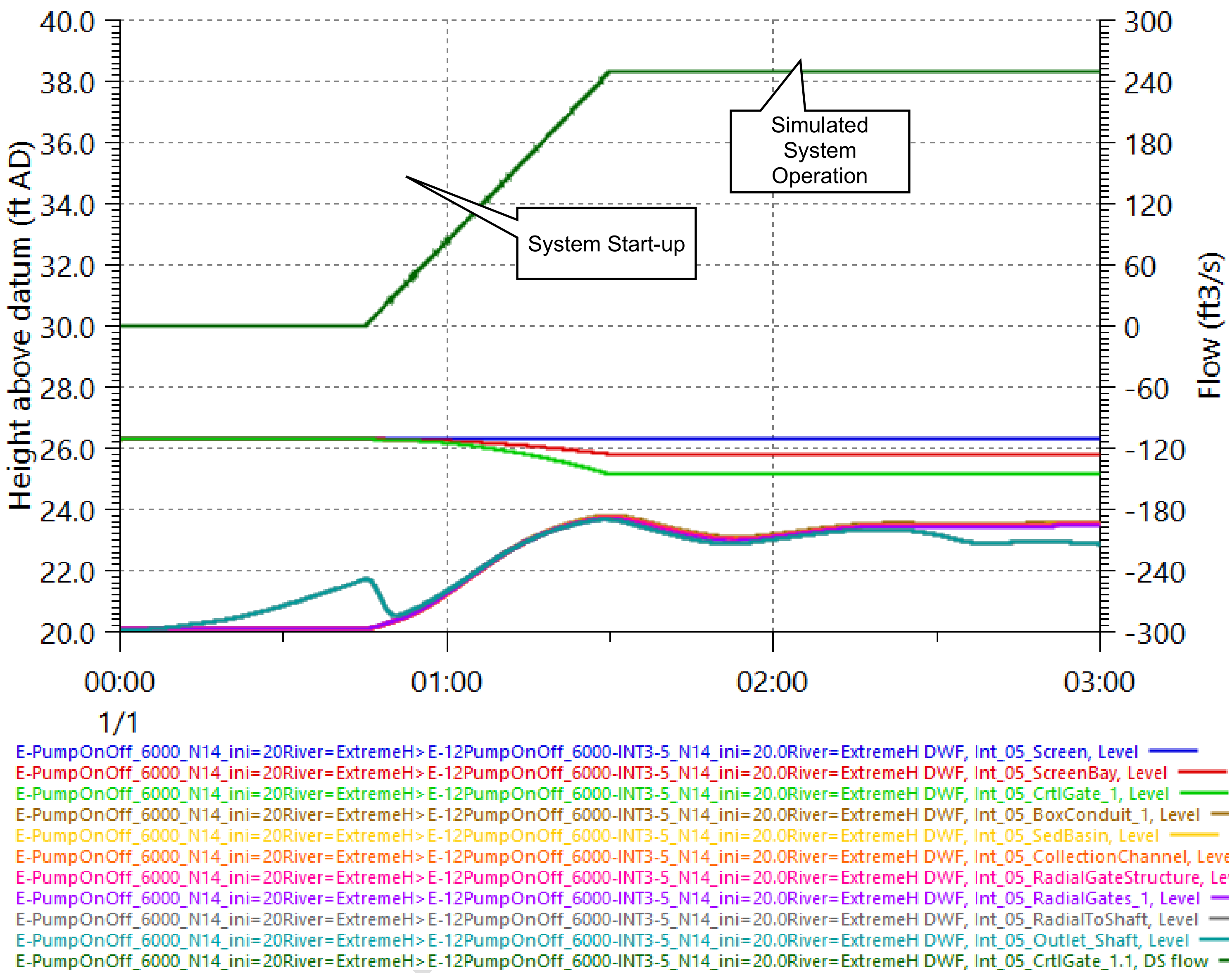

Figure 30. C-E-5 Flow and Water Levels during System Startup, High River, Manning's n= 0.014

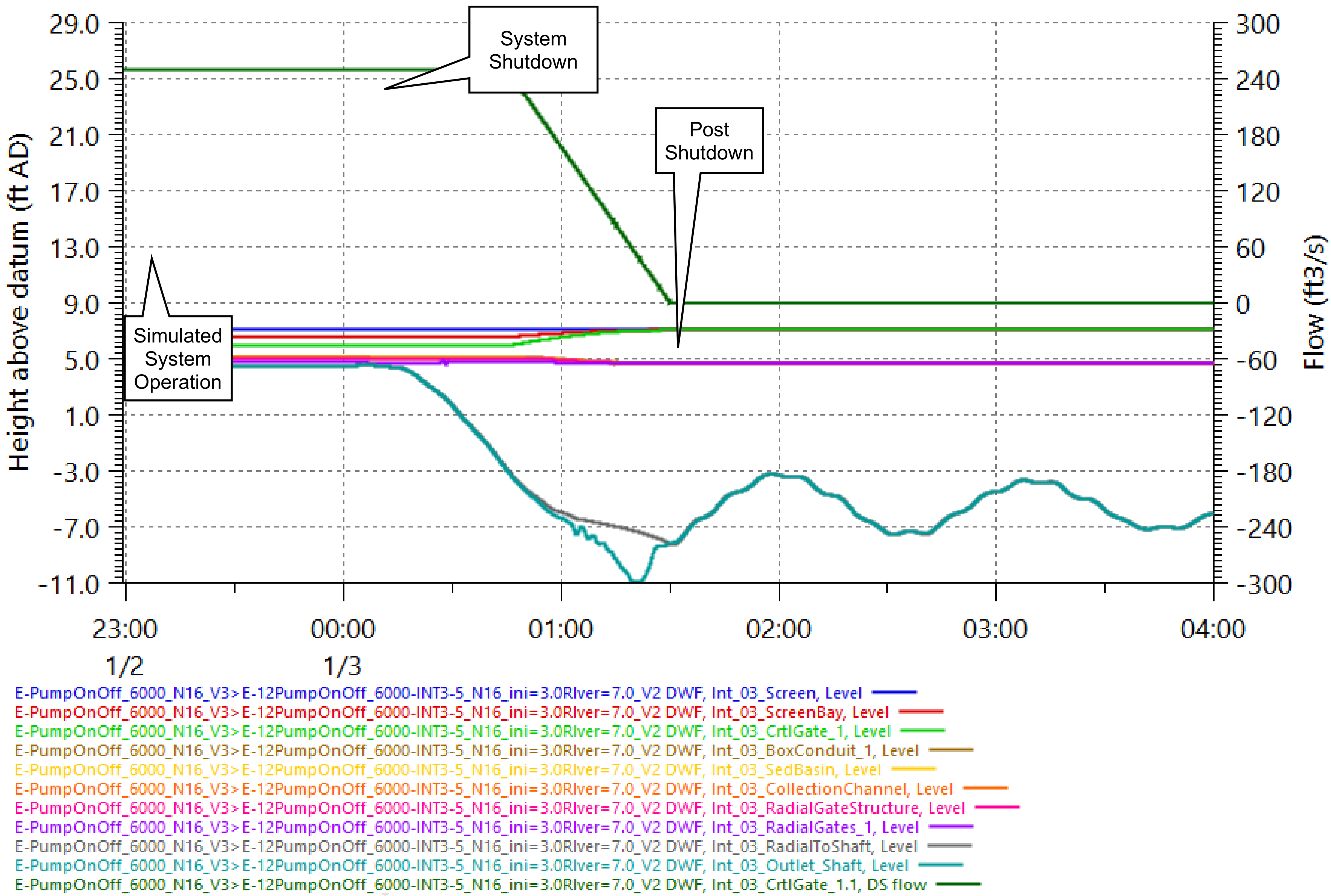

Figure 31. C-E-3 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.016

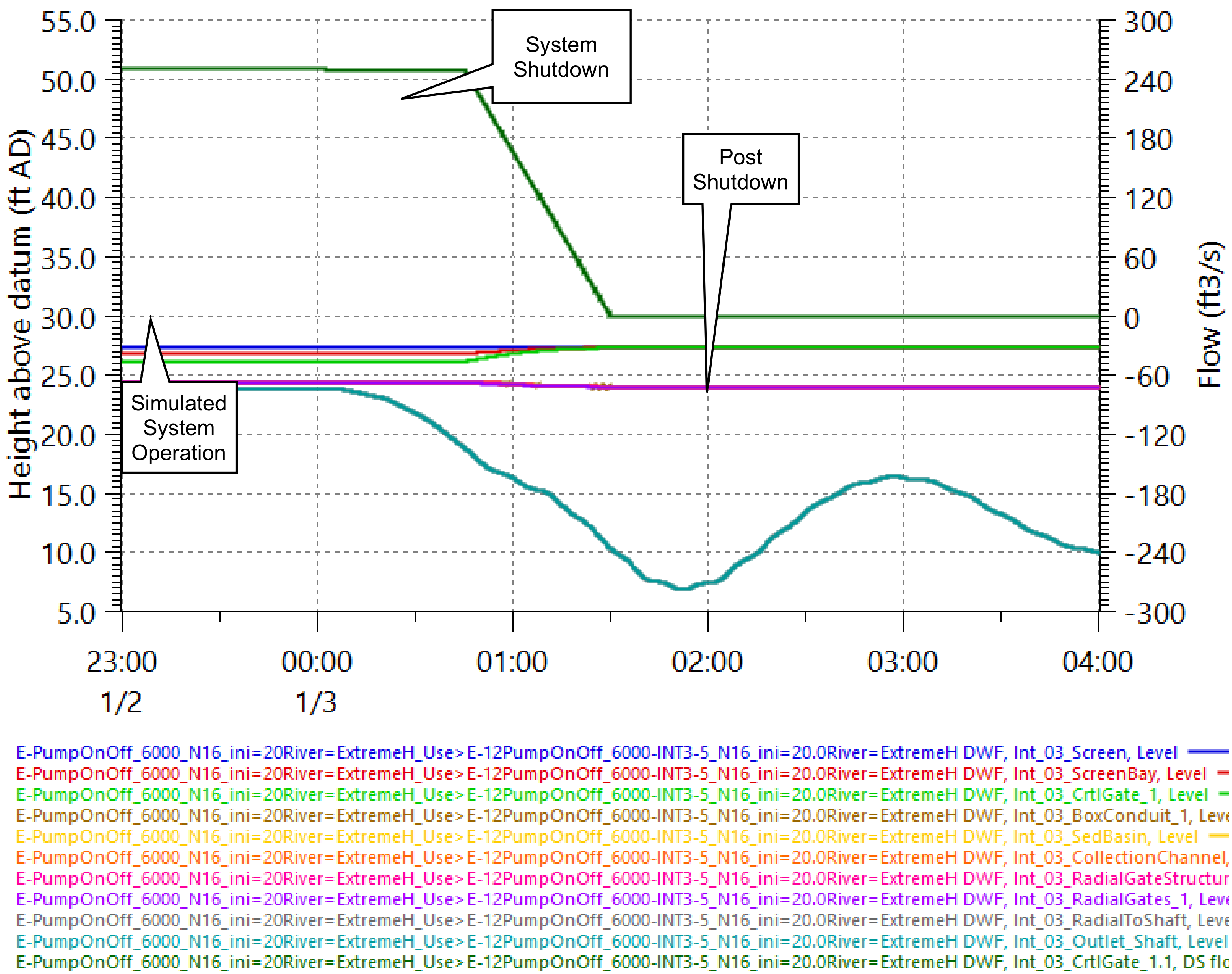

Figure 32. C-E-3 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.016

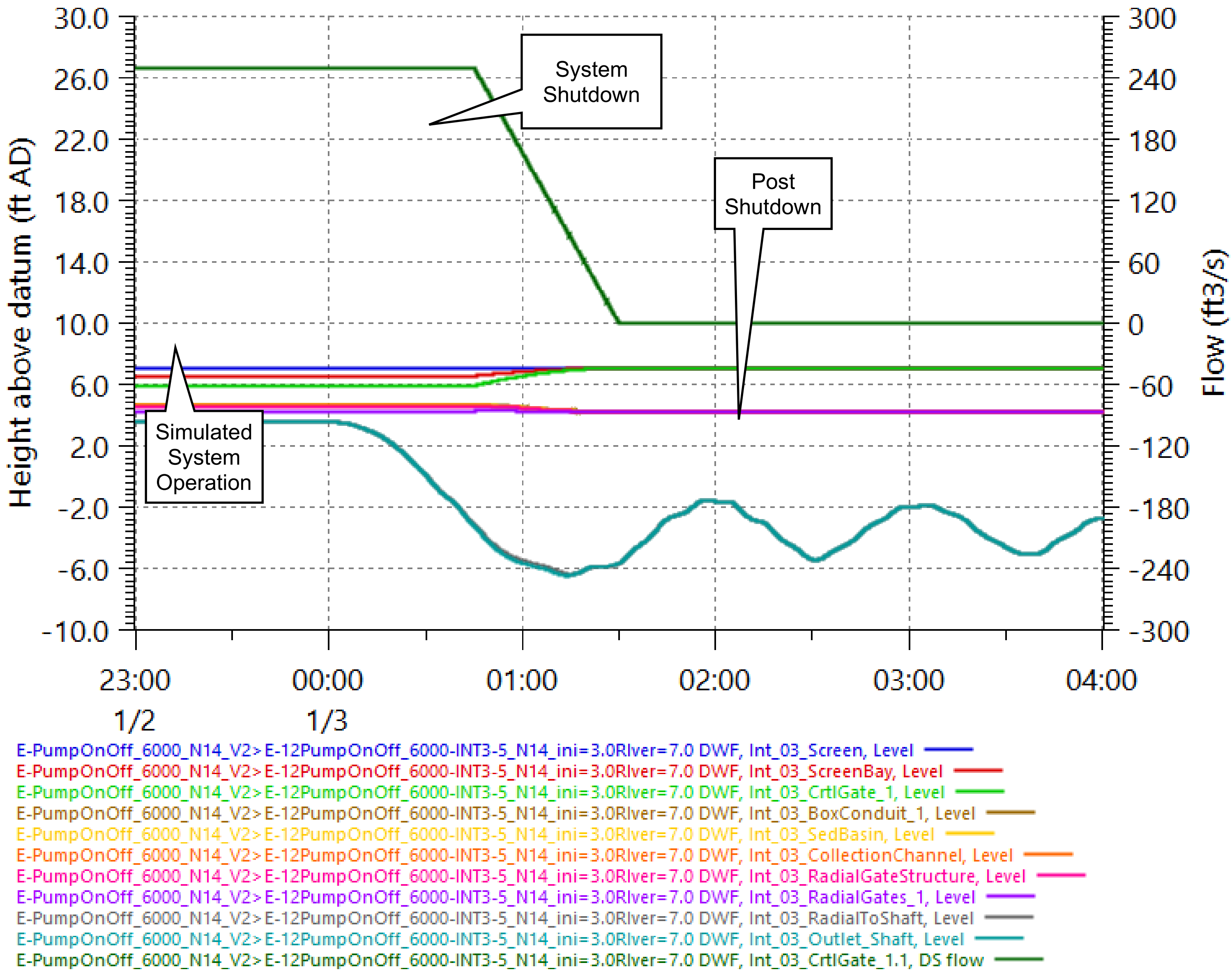

Figure 33. C-E-3 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.014

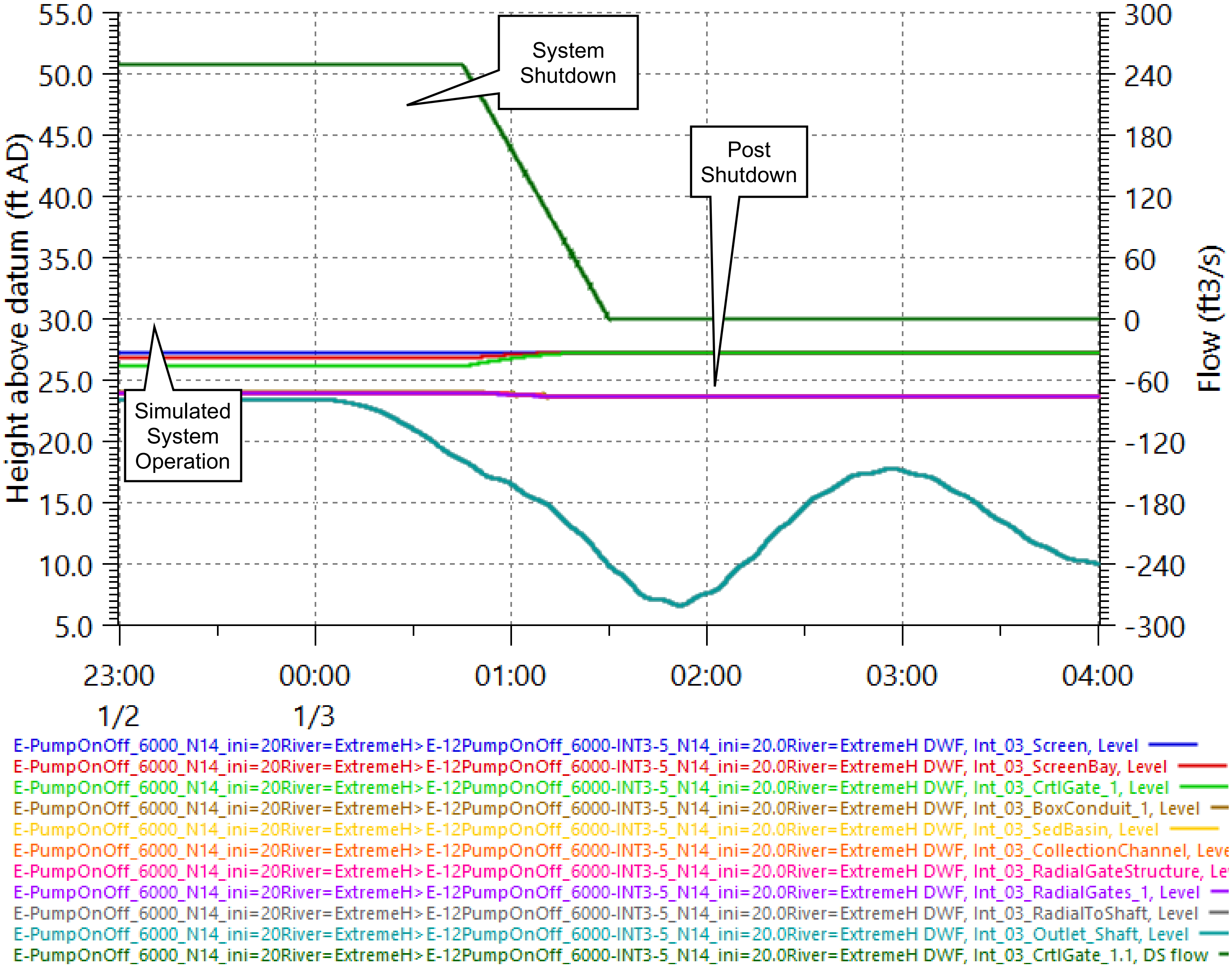

Figure 34. C-E-3 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.014

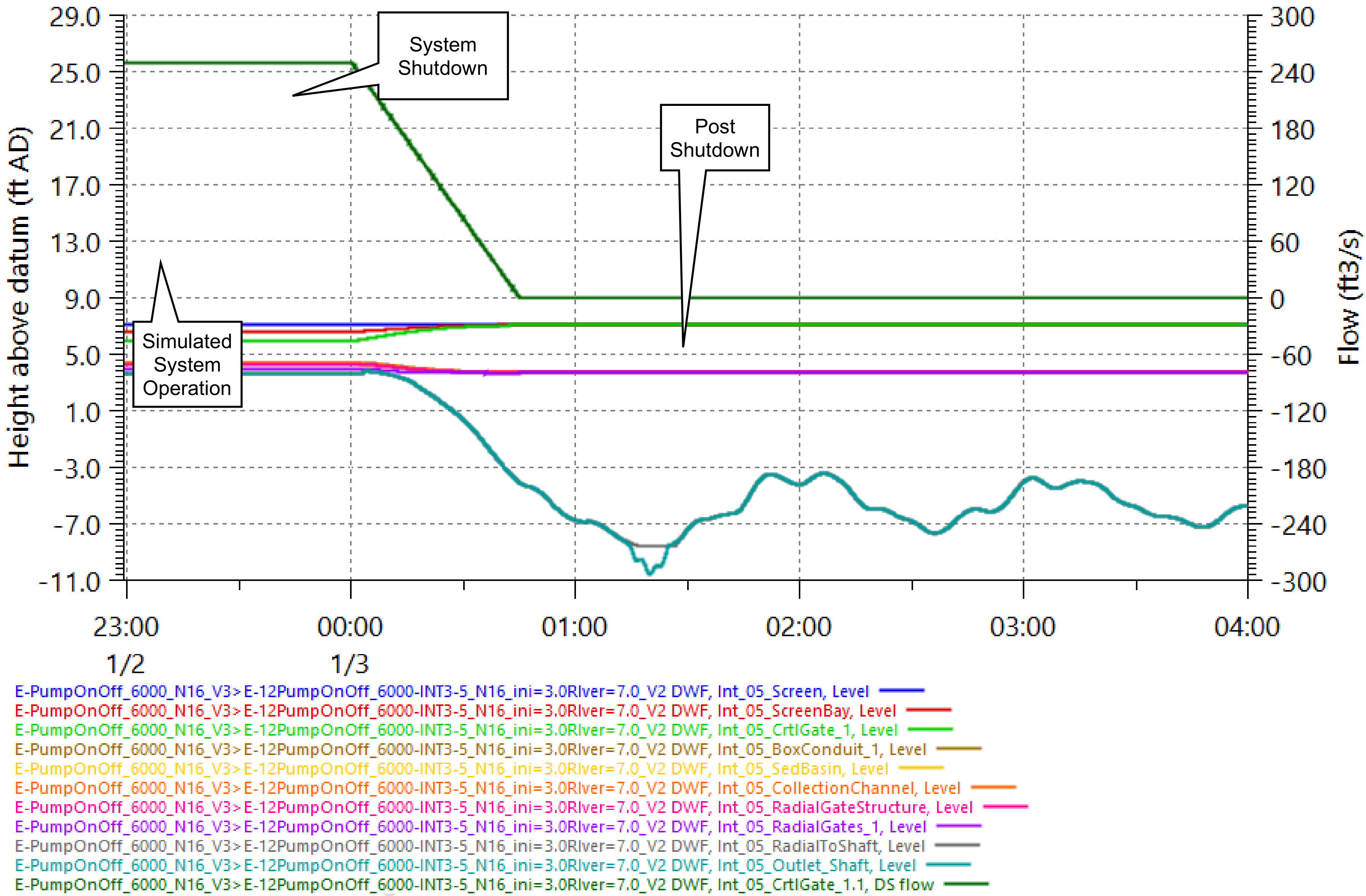

Figure 35. C-E-5 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.016

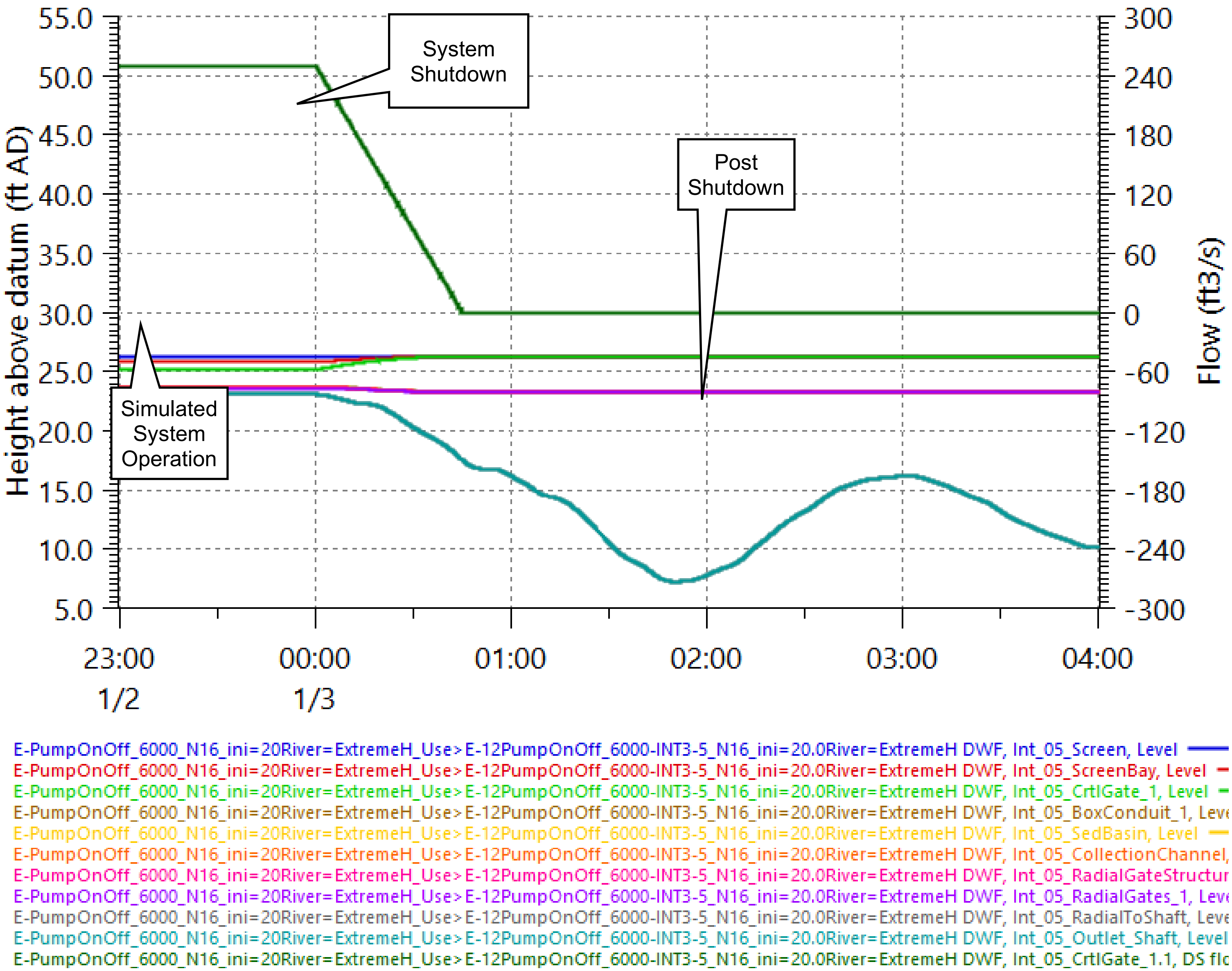

Figure 36. C-E-5 Flow and Water Levels during System Startup, High River, Manning’s n= 0.016

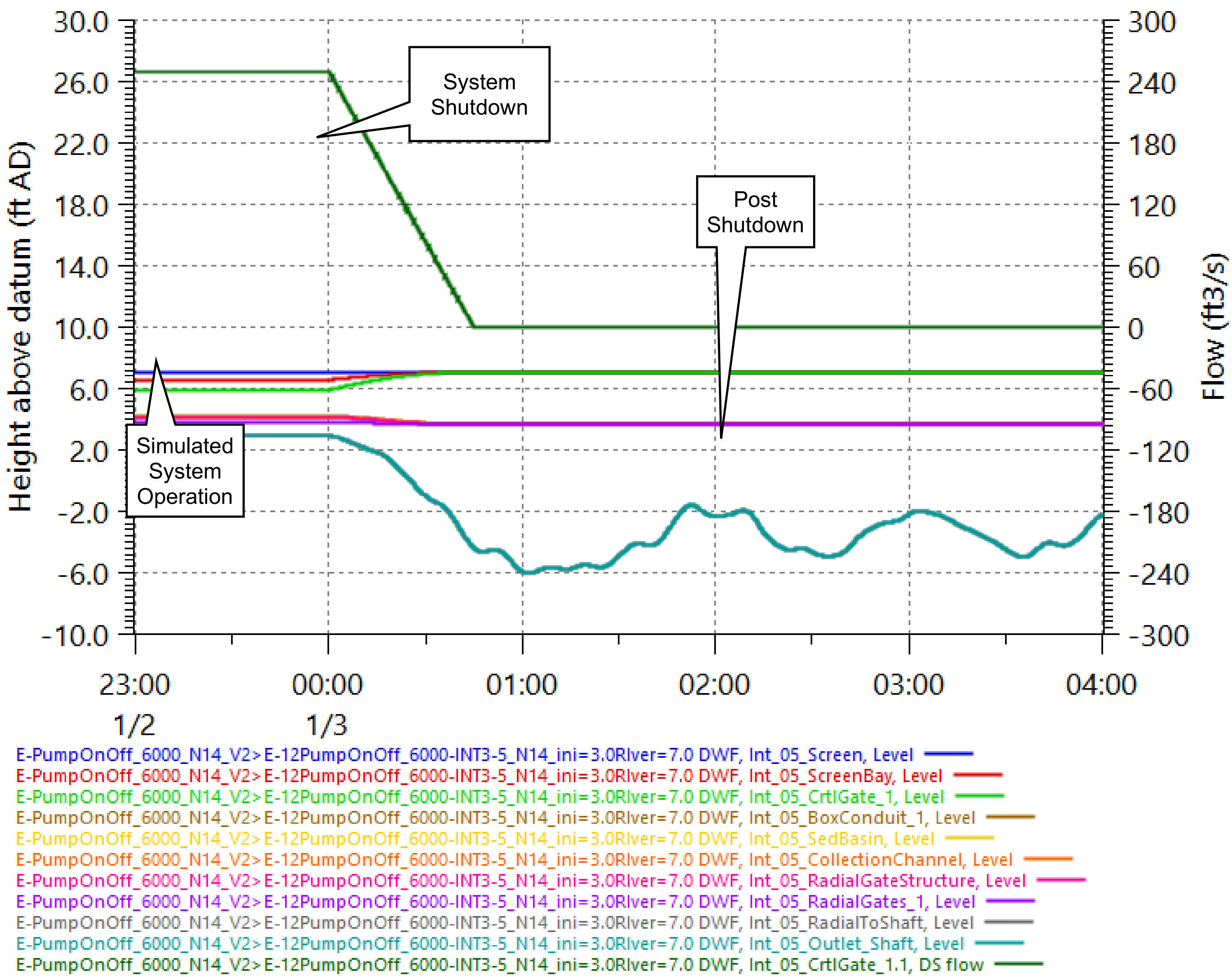

Figure 37. C-E-5 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.014

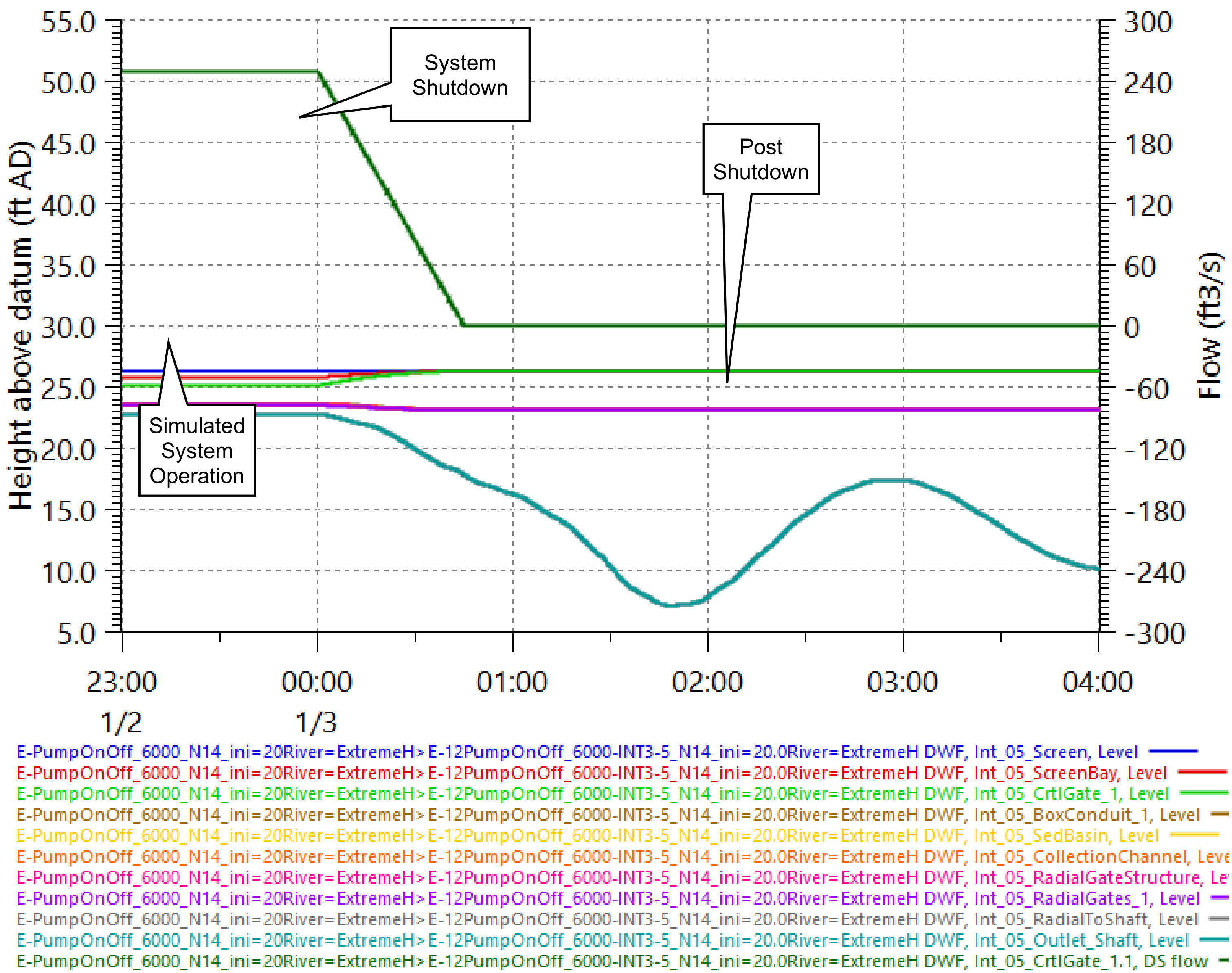

Figure 38. C-E-5 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.014

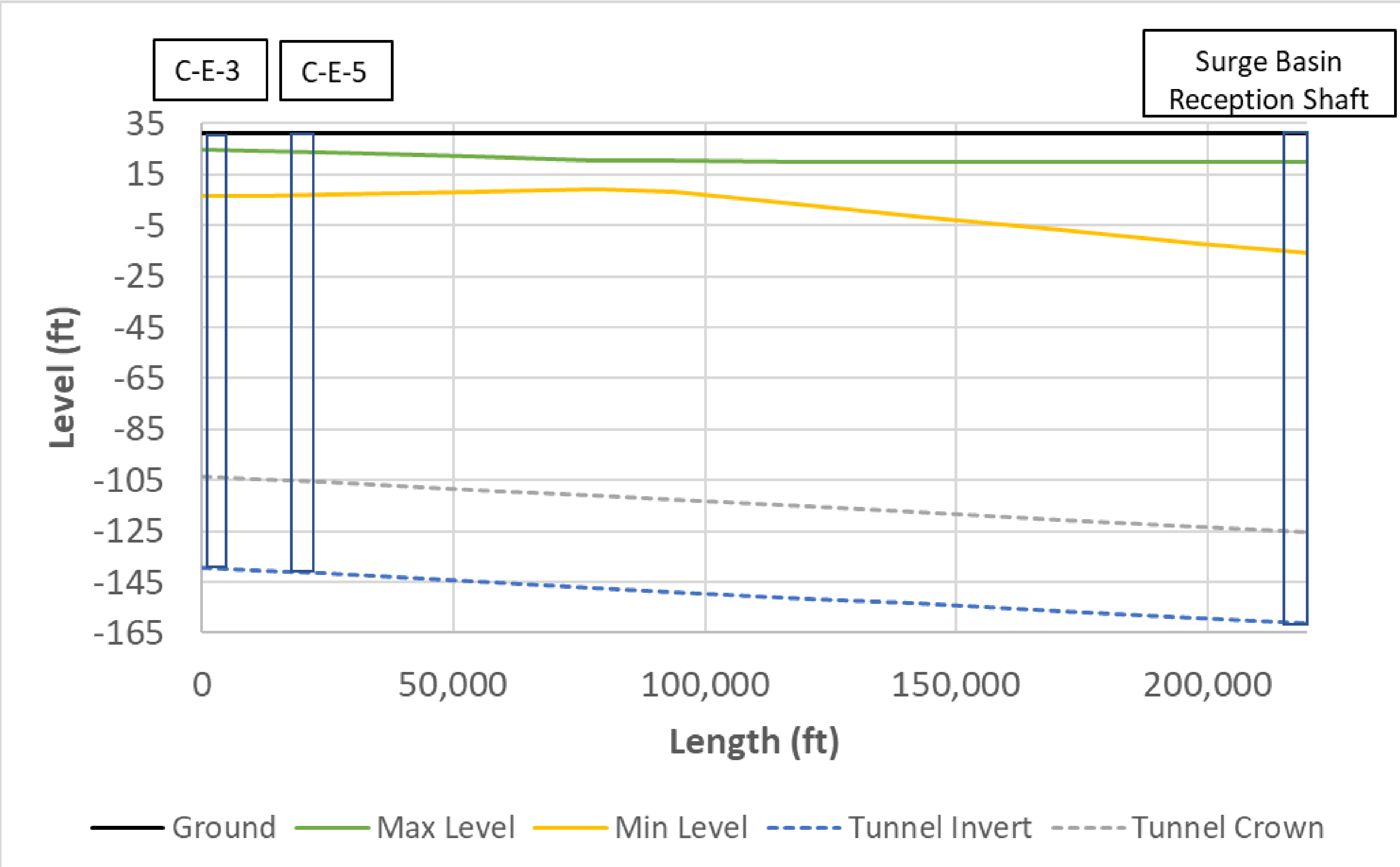

Figure 39. Minimum and Maximum WSEL Profile Startup and Shutdown Scenario, Bethany Reservoir Alternative, 6000 cfs, Manning's *n* 0.014, High River level

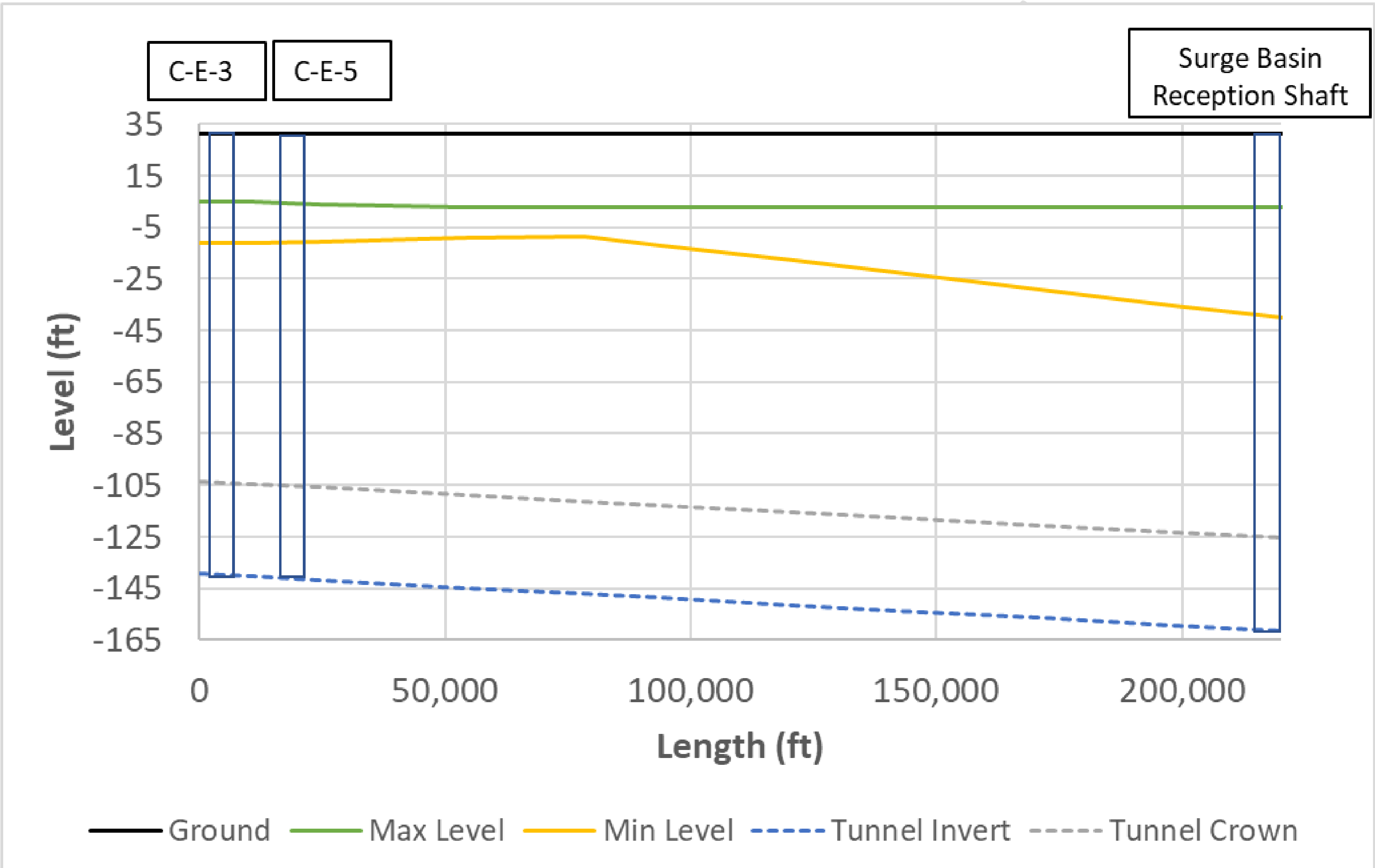

Figure 40. Minimum and Maximum WSEL Profile Startup and Shutdown Scenario, Bethany Reservoir Alternative, 6000 cfs, Manning's *n* 0.016, Low River

2.4.3.3 Summary

The following summarizes the startup and shutdown operations for the Bethany Reservoir Alternative with a project design capacity of 6,000 cfs:

- A steady-state diversion capacity of 6,000 cfs was established for both startup and shutdown of the project system without the need of an additional hydraulic equalization facility, such as an intermediate forebay.
- The system stabilized during the startup, shutdown, and flow transitions to the set-point diversion capacity of 6,000 cfs in part due to:
  - Shafts along the tunnel acted as equalization chambers
  - Pumps were operated with variable-frequency drives and were maintained within their POR to match startup and shutdown flow diversion rates associated with the intakes
- System startup and flow transitions to the set-point steady-state operation was 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 was achieved using the simulated rate of 1,000 cfs per 15 minutes.
- WSEL oscillations occurred within the shafts along the tunnel and the BRPP wet well following the system shutdown simulation. These oscillations are primarily due the tunnel shafts acting as equalization chambers along the tunnel alignment. These oscillations will dissipate once flow is reestablished within the conveyance system. Changes in the WSEL of the project's hydraulic facilities due to oscillations to the tunnel HGL are gradual and remain within the conceptual design limits of the project's tunnel, hydraulic structures, and connecting components.

3. Analysis and Evaluation

3.1 Steady-state Hydraulic Head Loss Analysis

A steady-state, hydraulic head loss analysis was performed between the intake outlet shafts to the BRPP wet well for the project design capacities of 3,000, 4,500, 6,000, and 7,500 cfs.

For the steady-state head loss analysis of each project design capacity, the outlet shafts for each intake were modeled with a finished inside diameter of 83 feet and the Surge Basin Reception Shaft was modeled with a finished inside diameter of 120 feet. The Twin Cities Double Launch Shaft was modeled as a double shaft with finished inside diameters of 115 feet (each shaft), and the Lower Roberts Launch and Reception Shaft was modeled as a single shaft with a finished inside diameter of 115 feet. It is recognized that for the Bethany Reservoir Alternative, a dual launch shaft would be constructed at Lower Roberts Island, but one of those shafts would be filled prior to operations. All other intermediate shafts were each modeled with a finished inside diameter of 70 feet. The flow scenarios evaluated are shown in Table 2.

Table 2. Steady-State Hydraulic Headloss Analysis – Project Flow Capacities Evaluated

Project Design Capacity Options (cfs)	Tunnel ID (feet)	Manning's n	River Level	Intake Order
3,000	26	0.016	Low	C-E 5
3,000	26	0.014	High	C-E 5

Table 2. Steady-State Hydraulic Headloss Analysis – Project Flow Capacities Evaluated

Project Design Capacity Options (cfs)	Tunnel ID (feet)	Manning's n	River Level	Intake Order
4,500	31	0.016	Low	C-E 3+5
4,500	31	0.014	High	C-E 3+5
6,000	36	0.016	Low	C-E 5+3
6,000	36	0.014	High	C-E 5+3
7,500	40	0.016	Low	C-E 5+3+2
7,500	40	0.014	High	C-E 5+3+2

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

Figures 41 through 44 plot the tunnel head loss results that were developed for each of the flow conditions. Tables 3 through 6 summarize the corresponding head loss to the BRPP wet well WSEL for each maximum design capacity and assigned Manning's *n*.

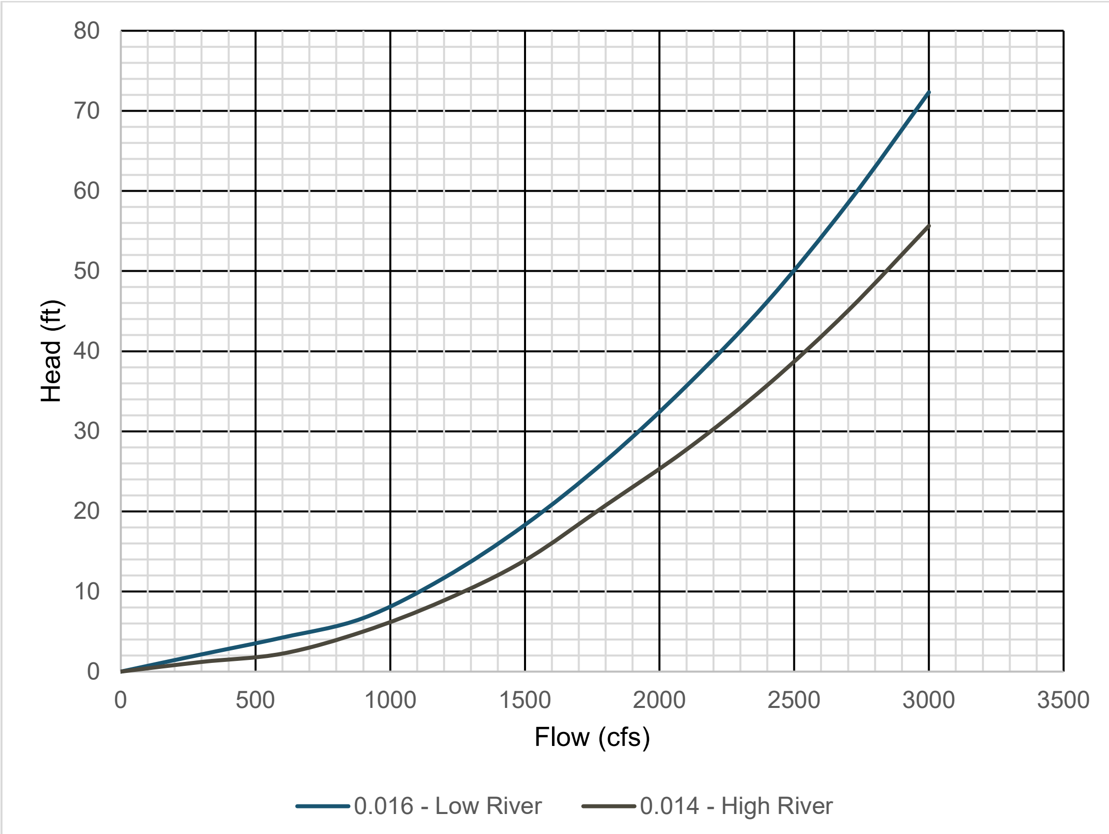

Figure 41. System Head Curves – 26-foot-inside-diameter Tunnel with a Project Design Capacity of 3,000 cfs

Table 3. Tunnel Head Loss and BRPP Wet Well Water Surface Elevation for a Project Design Capacity of 3,000 cfs

River Level	Manning's *n*	Head Loss from Intakes to BRPP Wet Well (feet)	Wet Well WSEL (feet)
Low River	0.016	72.3	-68.7
High River	0.014	55.6	-29.3

Referring to Figure 41 and Table 3 for the 26-foot-inside-diameter tunnel, at the project design capacity of 3,000 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 72.3 feet and result in a WSEL in the BRPP wet well of -68.7 feet. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develop a steady-state head loss of 55.6 feet and result in a WSEL in the BRPP wet well of -29.3 feet.

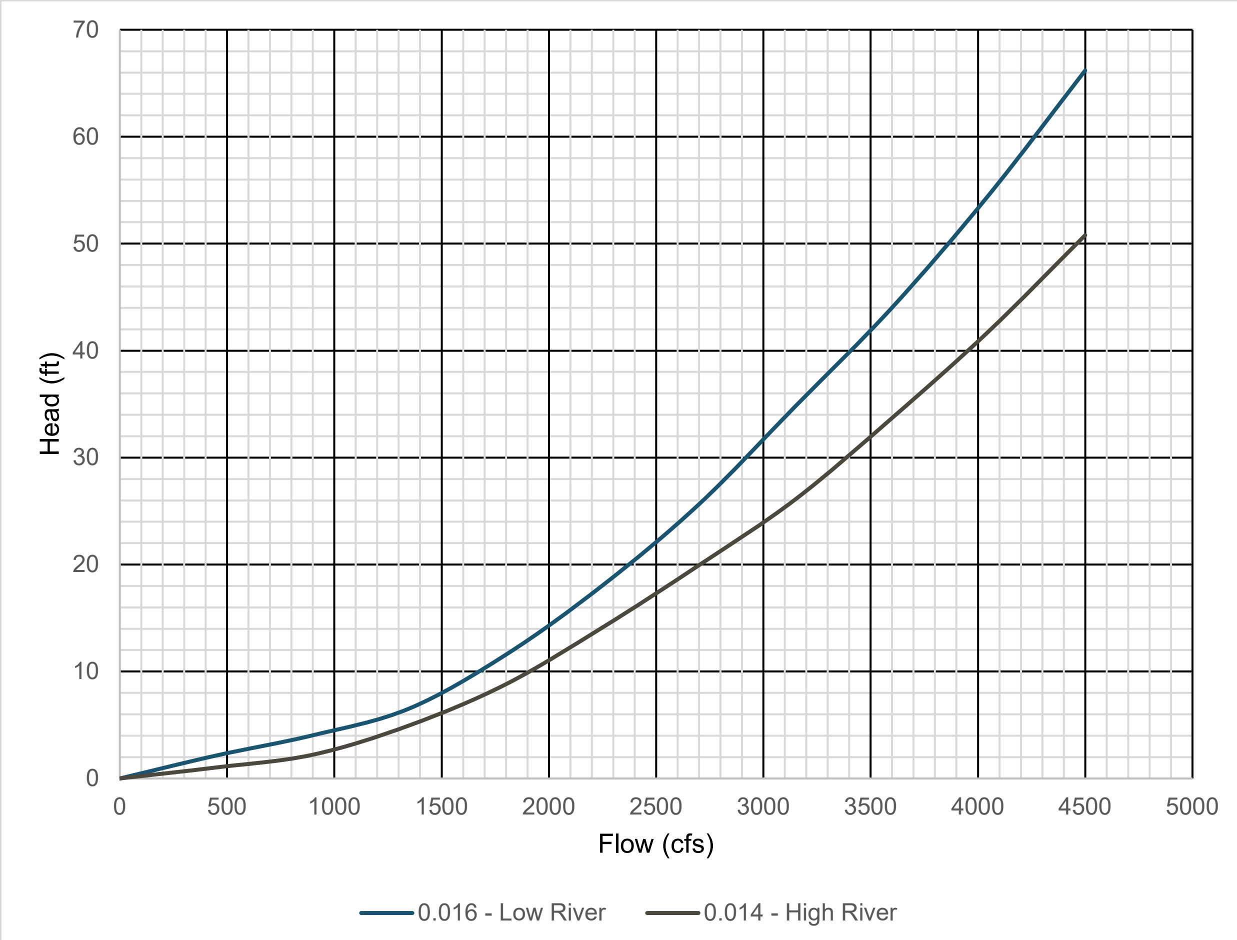

Figure 42. System Head Curves – 31-foot-inside-diameter Tunnel with a Project Design Capacity of 4,500 cfs

Table 4. Tunnel Head Loss and BRPP Wet Well Water Surface Elevation for a Project Design Capacity of 4,500 cfs

River Level	Manning's *n*	Head Loss from Intakes to BRPP Wet Well (feet)	Wet Well WSEL (feet)
Low River	0.016	66.2	-62.5
High River	0.014	50.8	-23.5

Referring to Figure 42 and Table 4 for the 31-feet-inside-diameter tunnel, at the project design capacity of 4,500 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 66.2 feet and result in a WSEL in the BRPP wet well of -62.5 feet. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 50.8 feet and results in a WSEL in the BRPP wet well of -23.5 feet.

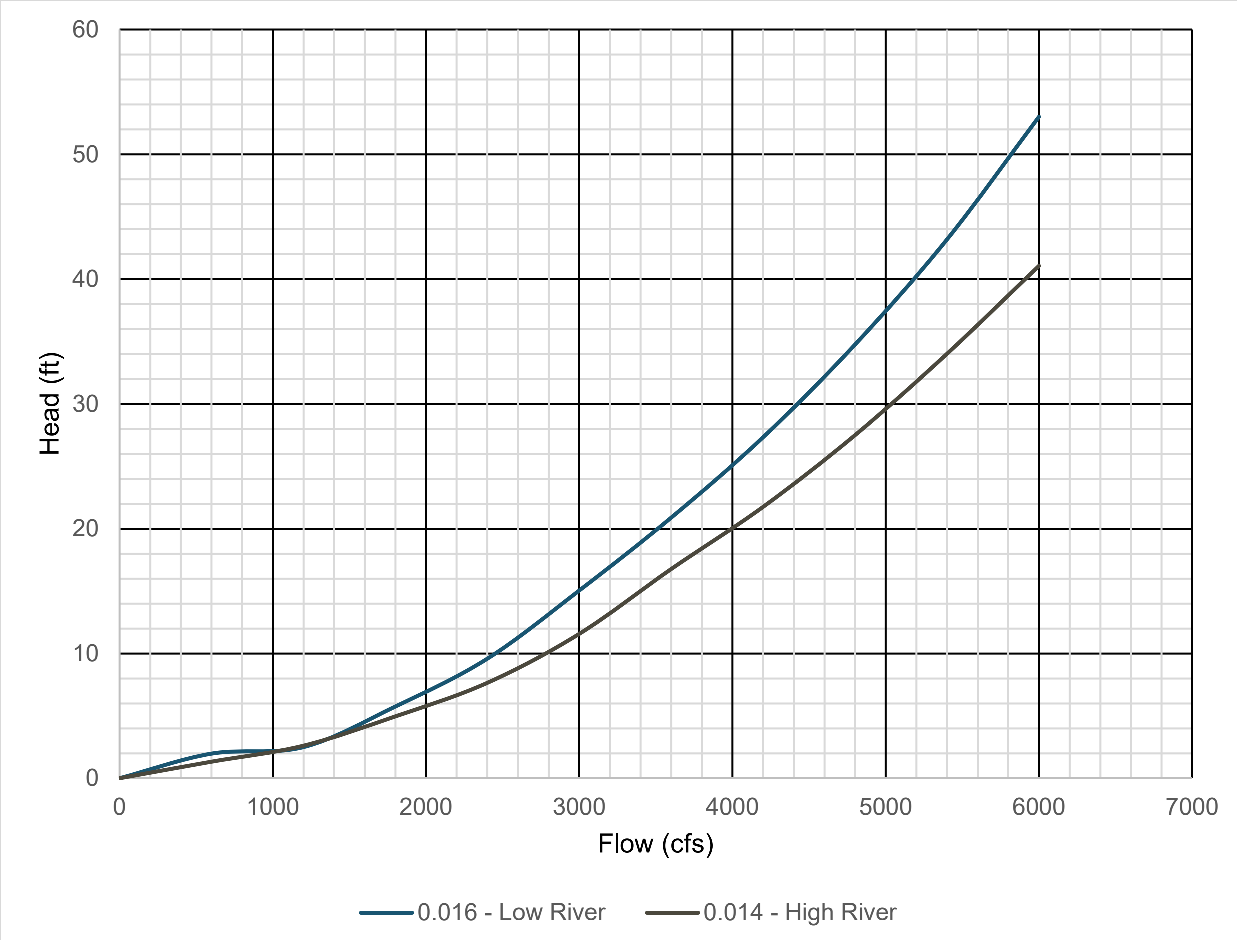

Figure 43. System Head Curves – 36-foot-inside-diameter Tunnel with a Project Design Capacity of 6,000 cfs

Table 5. Tunnel Head Loss and BRPP Wet Well Water Surface Elevation for a Project Design Capacity of 6,000 cfs

River Level	Manning's *n*	Head Loss from Intakes to BRPP Wet Well (feet)	Wet Well WSEL (feet)
Low River	0.016	53.0	-49.3
High River	0.014	41.1	-13.8

Referring to Figure 43 and Table 5 for the 36-feet-inside-diameter tunnel, at the project design capacity of 6,000 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 53 feet and result in a WSEL in the BRPP wet well of -49.3 feet. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 41.1 feet and results in a WSEL in the BRPP wet well of -13.8 feet.

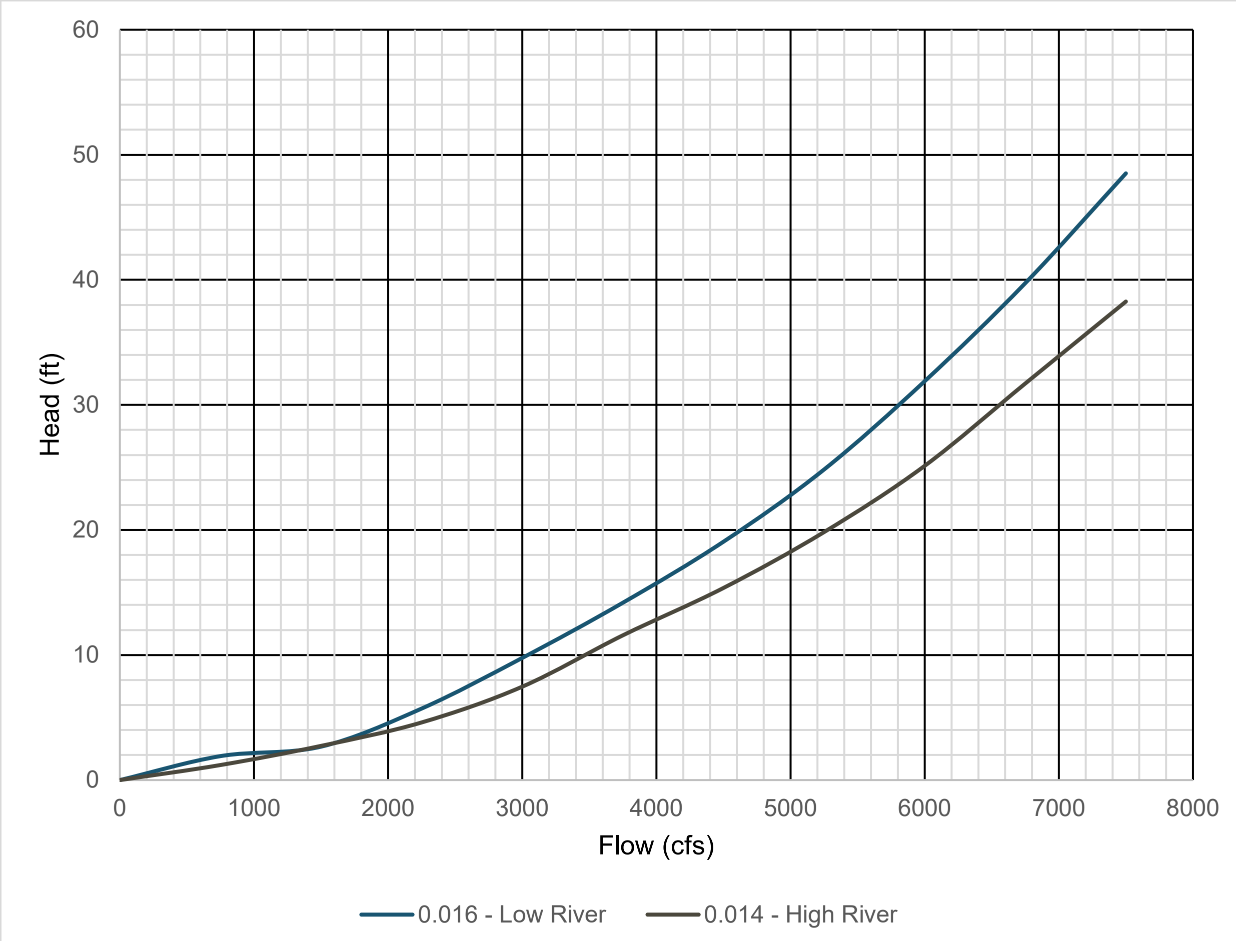

Figure 44. System Head Curves – 40-foot-inside-diameter Tunnel with a Project Design Capacity of 7,500 cfs

Table 6. Tunnel Head Loss and BRPP Wet Well Water Surface Elevation for a Project Design Capacity of 7,500 cfs

River Level	Manning's *n*	Head Loss from Intakes to BRPP Wet Well (feet)	Wet Well WSEL (feet)
Low River	0.016	48.5	-44.7
High River	0.014	38.3	-10.1

Referring to Figure 44 and Table 6, at the project design capacity of 7,500 cfs, the low river level and Manning's *n* of 0.016 develop a steady-state head loss of 48.5 feet and result in a WSEL in the BRPP wet well of -44.7 feet. At the same design flow capacity, the high river level and Manning's *n* of 0.014 develops a steady-state head loss of 38.3 feet and results in a WSEL in the BRPP wet well of -10.1 feet.

3.1.1 Steady-state Hydraulic Grade Line Development

A steady-state HGL was developed for the tunnel at a project design capacity of 6,000 cfs. The HGLs shown on Figure 45 depicts the steady-state condition in the tunnel for low river at Manning's *n* of 0.016, normal river with Manning's *n* of 0.014, and high river level at Manning's *n* of 0.014.

The Bethany Reservoir Aqueduct HGL was developed for the 6,000-cfs project design capacity and the maximum downstream Bethany Reservoir WSEL (245 feet), resulting in a maximum capacity of 1,500 cfs per aqueduct pipeline. The maximum steady-state HGL for the Bethany Reservoir Aqueduct is shown on Figure 46.

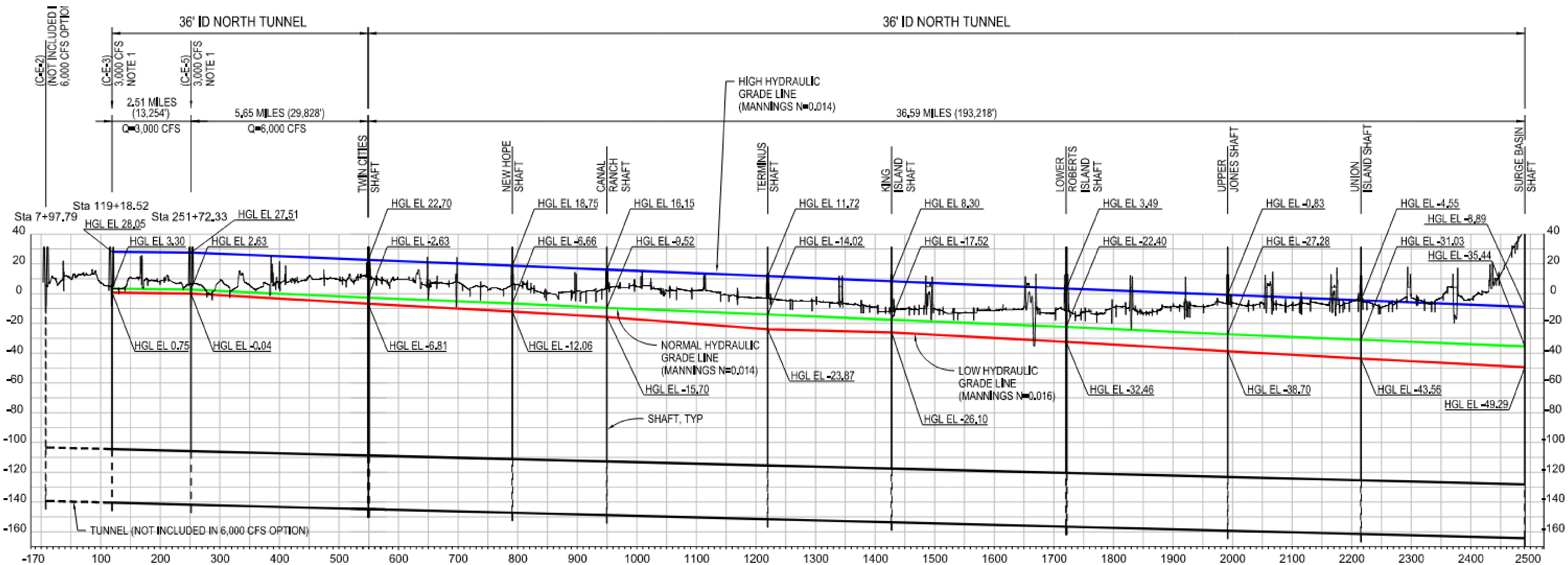

Figure 45. Steady-state HGL; 36-feet-diameter Tunnel; 6,000-cfs Project Design Capacity – Low, Normal, High River Levels

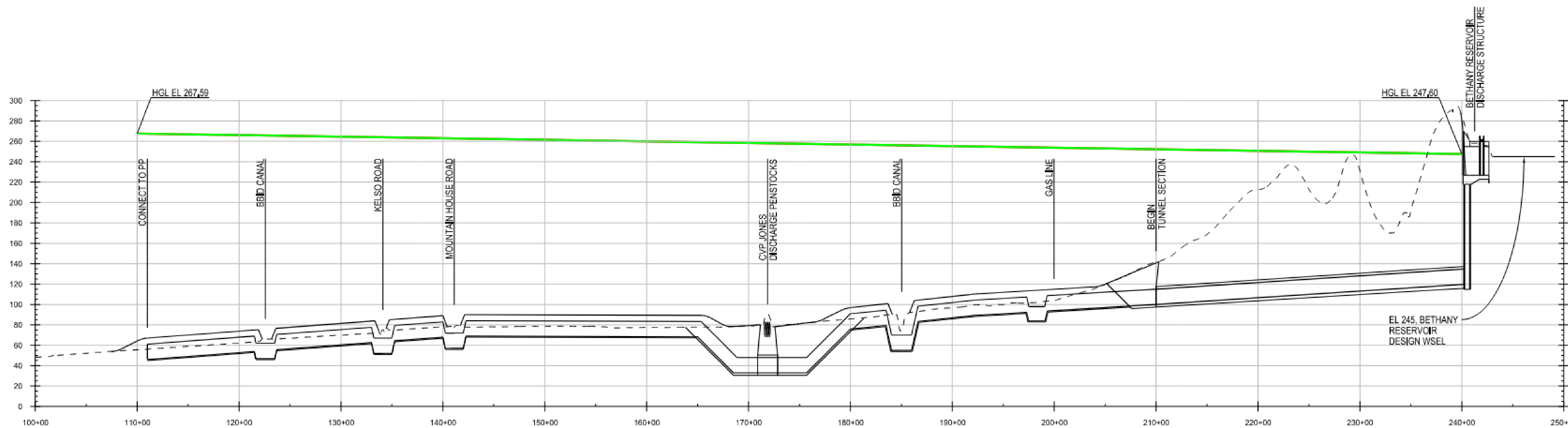

Figure 46. Maximum Steady-state HGL; Bethany Reservoir Aqueduct; 1,500 cfs

4. Surge Analyses

4.1 Model Description

In accordance with the methodology and criteria established for the project as described in the *Capacity Analysis for Preliminary Tunnel Diameter Selection TM* (DCA 2021d), hydraulic transient-surge analyses were performed between the Intake C-E-2 drop shaft and the tunnel Surge Basin structure, which is located within the BRPP site, as shown on the engineering concept drawings. The analyses were conducted to size the surge overflow basin and establish the maximum and minimum HGLs along the entire tunnel system resulting from transient-surge events and design flow conditions at maximum and minimum Sacramento River WSELs.

Hydraulic transient-surge analyses were also performed for BRPP's welded steel discharge pipelines (Bethany Reservoir Aqueduct) located between the BRPP and the Bethany Reservoir Discharge Structure and the Jones Outlet Structure at the approach channel to the Jones Pumping Plant, as shown on the engineering concept drawings. The analyses were conducted to establish the maximum and minimum HGLs along each aqueduct pipeline and to determine surge mitigation features that would maintain transient-surge pressures to within the internal pressure design limits of the aqueduct pipelines.

The maximum tunnel flow velocity of 6 feet per second was recommended based on the hydraulic criteria previously established. For the analyses, 36-foot and 40-foot tunnel diameters were used for the main tunnel corridor for project design capacities of 6,000 cfs and 7,500 cfs, respectively. The analyses were used to evaluate a simultaneous pump shutdown condition caused by power failure at the BRPP, which is the worst-case transient scenario. The vapor pressure was assumed to be -14.2 pounds per square inch (psi). The scenarios evaluated are shown in Table 7 for the two design flow capacities.

The tunnel design for project design 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 7. Hydraulic Transient-Surge Scenarios

Scenario	Project Design Capacity [cfs]	Tunnel Diameter [feet]	River Elevation [feet]	Intake Radial Gates Closing Time [minutes]
1	6,000	36	3.6	12
2	6,000	36	27.3	12
3	7,500	40	3.6	12
4	7,500	40	28.2	12

Transient-surge events in the tunnel were simulated by simultaneously stopping all pumps in operation at the BRPP, followed by the simultaneous closure of radial gates located at the sediment basin outlets of each intake in operation. Closure of the radial gates at the sediment basin outlet would prevent turbulence in the sedimentation gates and reverse flows into the Sacramento River from the intakes during the simulated transient-surge event.

Transient-surge events in the BRPP's discharge pipelines were simulated by simultaneously stopping all pumps and closing the pump control valves, located at the discharge of each pump, within 15 seconds. Each transient-surge event simulated a maximum flow of 1,500 cfs in each pipeline. For the project design capacity of 7,500 cfs, four 15-foot-diameter pipelines would convey up to 6,000 cfs (1,500 cfs per pipeline) to the Bethany Reservoir and one 15-foot-diameter pipeline would convey up to 1,500 cfs to the approach channel to the Jones Pumping Plant. For the project design capacity of 6,000 cfs, four 15-foot-diameter pipelines would convey up to 6,000 cfs to the Bethany Reservoir and no flow would be conveyed to the Jones Pumping Plant approach channel.

4.2 Transient-Surge Results

4.2.1 Tunnels

Results of the transient analyses for the main tunnel, presented on Figures 47 through 50, show the hydraulic transient maximum and minimum HGL elevations that occur throughout the transient-surge events along the tunnel corridor, using 83-foot-inside-diameter intake shafts, two 115-foot-inside-diameter launch shafts at Twin Cities (double shaft), one 115-foot-inside-diameter launch/reception shaft at the Lower Roberts (one cell of the double shaft was simulated to limit the steady-state water volume in the double shaft to reduce the tunnel overflow volume at the Surge Basin), 120-foot-inside-diameter overflow shaft at the BRPP Surge Basin, and 70-foot inside diameters for all other reception and maintenance shafts. The top of weir surrounding the outlet of the overflow shaft within the BRPP Surge Basin structure was set at elevation 18.00 feet, as shown on the concept drawings.

Hydraulic transient-surge mitigation features for the tunnel simulated in this analysis included the overflow shaft and connecting surge basin located at the BRPP. The surge basin structure would be located above the main tunnel and connect to the vertical reception shaft, as shown on the engineering concept drawings. The Surge Basin would be an open-top, rectangular, belowground level open basin-type structure and would be constructed with diaphragm walls and a reinforced concrete floor slab. The Surge Basin containment volume (between the top of the circular overflow weir and the Surge Basin floor) would be sized to accommodate water that would accumulate during a tunnel overflow condition, which would result from a hydraulic transient-surge event within the main tunnel. The top elevation of the diaphragm walls would vary to match the finished grade around the structure and have a top of floor slab elevation of 7.0 feet to match the top outlet elevation of the reception shaft.

The Surge Basin would include a circular weir wall surrounding the outlet of the vertical reception shaft. The circular weir wall would extend vertically from the top of the Surge Basin floor slab to a top-of-wall elevation of 18.0 feet. The weir wall would incorporate gated openings around its circumference that are normally closed and would allow operators to drain the overflow volume back into the tunnel after the transient event was over. During a hydraulic transient-surge event within the main tunnel, water from the tunnel would automatically flow over the circular weir wall and into the surge basin. Such a surge event would be the result from an electrical power failure at the BRPP site (or other emergency that would generate a transient-surge condition) and would overflow when the water surface elevation within the reception shaft exceeds 18.0 feet. The circular weir wall with its gated openings in the closed position would prevent water stored within the surge basin from reentering the tunnel. The gated openings would only be opened to drain the surge basin into the tunnel shaft and BRPP wet well conduit after the transient-surge event dissipates within the system.

During normal operation of the BRPP, the surge basin would be maintained empty, providing suitable storage capacity to accommodate overflow volumes associated with transient-surge events as described above. Following a tunnel transient-surge or wet weather event where the free-water surface of stored water within the basin is above a predetermined set-point elevation, the BRPP would not be permitted to operate until the basin has been emptied below this set-point elevation so sufficient storage is available within the basin for tunnel overflow volumes associated with transient-surge events. The Surge Basin facility would include permanent dewatering pumps, as shown on the engineering concept drawings, to automatically drain water contained within the basin structure captured during wet weather events. Pumped water from the Surge Basin would be discharged into the Bethany Reservoir Aqueduct and flow to the reservoir. Additional details regarding the operation and control of the Surge Basin would be developed during final design.

The envelope of the maximum and minimum HGLs are plotted across the tunnel alignment between the C-E-3 drop shaft- and the Surge Basin overflow shaft for the steady-state flow conditions of 6,000 cfs and C-E-2 to the Surge Basin overflow shaft for the steady-state flow condition of 7,500 cfs. For reference, elevation 32 feet (about equal to maximum freeboard level at Intake C-E-2) is shown by a green horizontal dashed line. The tunnel intake drop shafts and Surge Basin overflow shaft are notated in each graph. Intermediate shafts along the alignment are indicated by vertical lines and are not named. The tunnel crown elevation for each diameter evaluated is shown as a dashed blue line on each graph.

The radial gates in each intake (in operation) were simultaneously closed immediately following the simulated power failure at the BRPP. The gate closure rate was identical, and gates were closed at a linear rate of 12 minutes from their last operating position to fully closed.

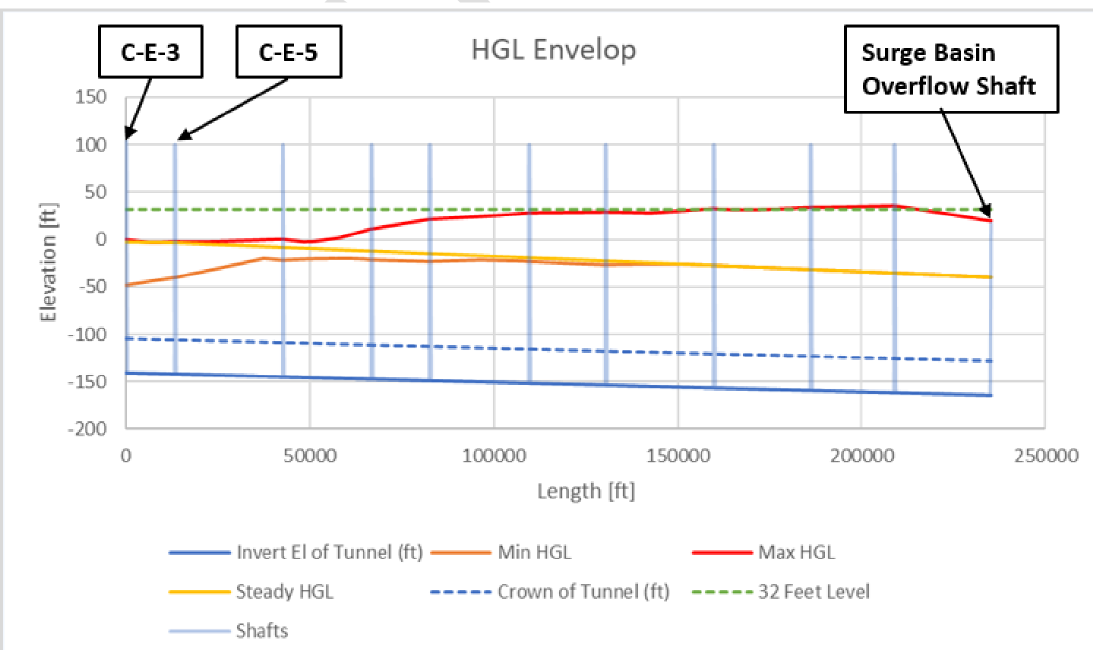

Figure 47. Scenario 1 – Bethany Reservoir Alternative Main Tunnel; Project Design Capacity 6,000 cfs; Low River Level; Minimum and Maximum Hydraulic Gradeline Profiles

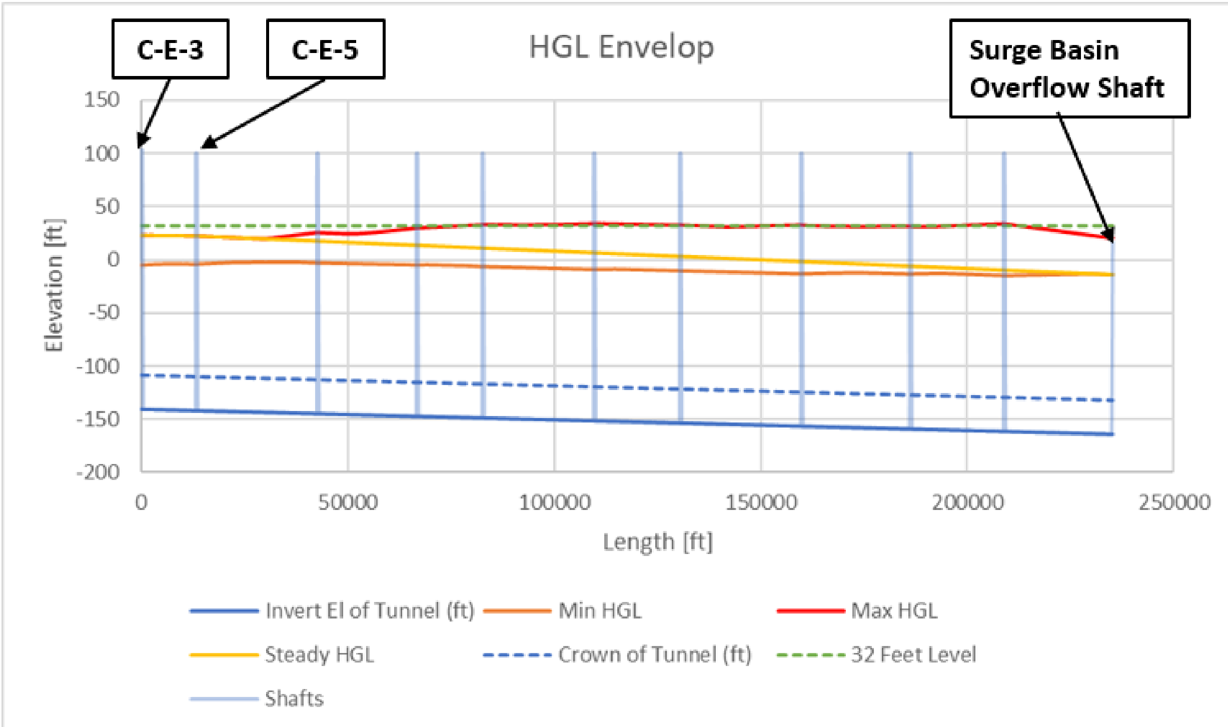

Figure 48. Scenario 2 – Bethany Reservoir Alternative Main Tunnel; Project Design Capacity 6,000 cfs; High River Level; Minimum and Maximum Hydraulic Gradeline Profiles

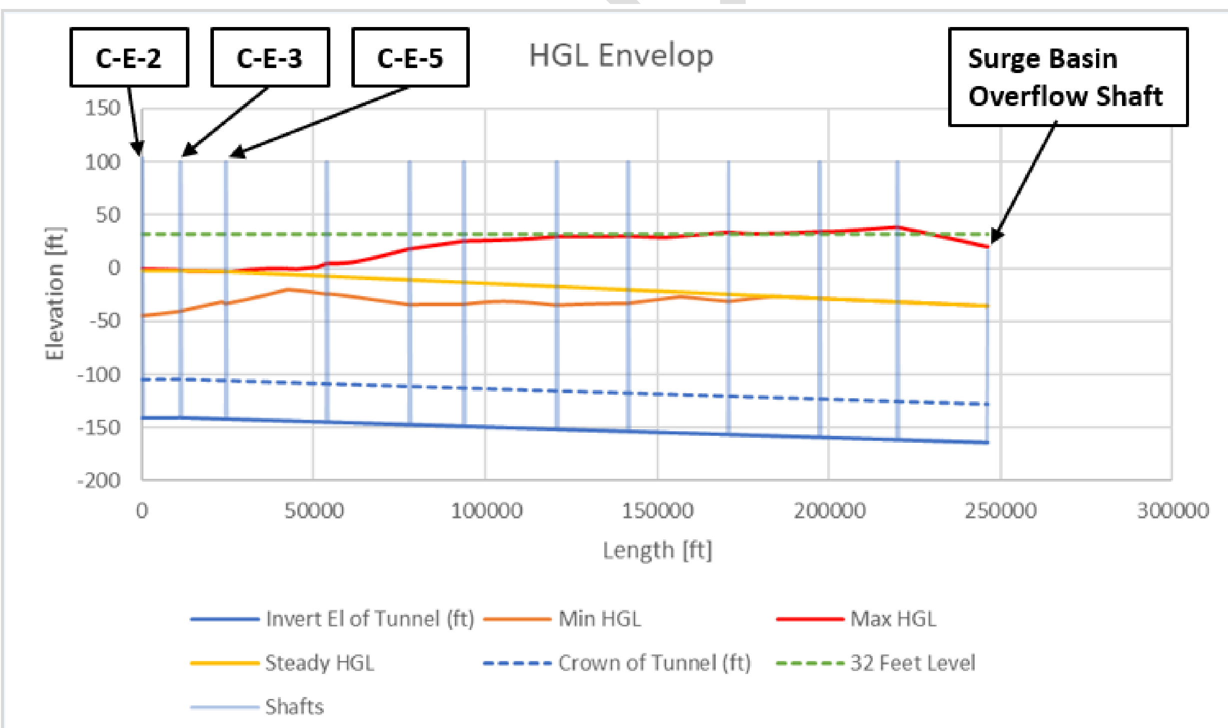

Figure 49. Scenario 3 – Bethany Reservoir Alternative Main Tunnel; Project Design Capacity 7,500 cfs; Low River Level; Minimum and Maximum Hydraulic Gradeline Profiles

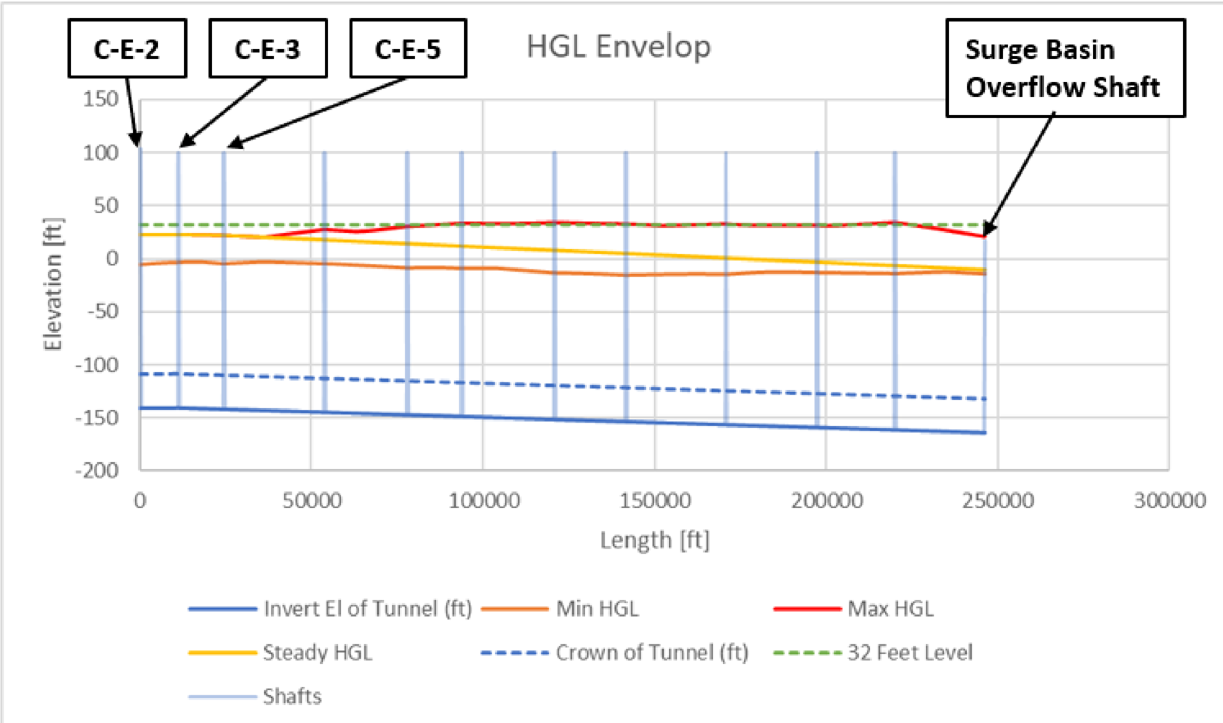

Figure 50. Scenario 4 – Bethany Reservoir Alternative Main Tunnel; Project Design Capacity 7,500 cfs; High River Level; Minimum and Maximum Hydraulic Gradeline Profiles

For the project design capacity of 6,000 cfs, the maximum tunnel overflow volume at the overflow shaft (located within the BRPP site complex) was estimated to be 4.80 million cubic feet. This maximum overflow volume occurs during a transient-surge event with the Sacramento River at the maximum WSEL (Scenario 2). For the project design capacity of 6,000 cfs, the conceptual design of the Surge Basin has been sized to contain a maximum tunnel overflow volume of 6.0 million cubic feet, which is 1.25 times the maximum calculated overflow volume of 4.80 million cubic feet.

For the project design capacity of 7,500 cfs, the maximum tunnel overflow volume at the overflow shaft was calculated as 6.90 million cubic feet. This maximum overflow volume occurs during a transient-surge event with the Sacramento River at the maximum WSEL (Scenario 4). For the project design capacity of 7,500 cfs, the conceptual design of the Surge Basin has been sized to contain a maximum tunnel overflow volume of 8.63 million cubic feet, which is 1.25 times the maximum calculated overflow volume of 6.90 million cubic feet.

The results of the transient-surge analysis indicate that no negative pressures are developed along the entire length of the tunnel alignment, and all pressures were within the conceptual design limits of the tunnel at either the maximum or minimum Sacramento River WSEL evaluated at each intake.

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 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 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 8 and 9 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 8 and 9 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 8. Bethany Reservoir Alternative Main Tunnel – 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)
C-E-3 Maintenance shaft	83.0	0.0	23.0	27.3	30.3
C-E-5 Maintenance Shaft	83.0	-2.5	22.3	27.3	30.3
Twin Cities Double Launch Shaft	115.0 (each shaft)	0.0	26.0	27.3	30.3
New Hope Tract Maintenance Shaft	70.0	11.75	30.0	27.3	33.0
Canal ranch Tract Maintenance Shaft	70.0	22.0	33.5	27.3	36.5
Terminous Tract Reception shaft	70.0	28.0	34.0	27.3	37.0
King Island Maintenance Shaft	70.0	29.5	33.0	27.3	36.0
Lower Roberts Island Launch Shaft	115.0	33.0	33.0	27.3	36.0
Upper Jones Tract Maintenance Shaft	70.0	34.0	32.0	27.3	37.0
Union Island Reception shaft	70.0	36.0	34.0	27.3	39.0
Surge Basin Reception Shaft	120.0	20.0	20.0	27.3	30.3

Table 9. Bethany Reservoir Alternative Main Tunnel – 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)
C-E-2 Reception Shaft	83.0	-1.0	23.0	28.2	31.2
C-E-3 Maintenance Shaft	83.0	-2.0	22.8	28.2	31.2
C-E-5 Maintenance Shaft	83.0	-3.5	22.0	28.2	31.2
Twin Cities Double Launch Shaft	115.0 (each shaft)	4.3	28.0	28.2	31.2
New Hope Tract Maintenance Shaft	70.0	18.0	31.0	28.2	34.0
Canal ranch Tract Maintenance Shaft	70.0	26.0	34.0	28.2	37.0
Terminous Tract Reception Shaft	70.0	30.0	34.5	28.2	37.5
King Island Maintenance Shaft	70.0	30.5	33.0	28.2	36.0
Lower Roberts Island Launch Shaft	115.0	34.0	33.0	28.2	37.0
Upper Jones Tract Maintenance Shaft	70.0	34.3	32.0	28.2	37.3
Union Island Reception Shaft	70.0	38.3	34.0	28.2	41.3
Surge Basin Reception Shaft	120.0	20.0	21.0	28.2	31.2

Table 10 summarizes the maximum and minimum HGL values that occur within the calculated transient-surge HGL envelop for each scenario. As can be seen in Table 10, the maximum HGLs occurs under Scenario 1 (minimum Sacramento River WSE for the project design capacity of 6,000 cfs) and Scenario 3 (minimum Sacramento River WSE for the Project design capacity of 7,500 cfs). The magnitude of the HGL peaks for scenarios 1 through 4 are very similar due to the fixed overflow elevation within the Surge Basin. However due to the convergence of transient-surge oscillations caused by the shaft locations and volumes which interact with the major transient-surge wave of the tunnel, the maximum HGL was developed with the minimum Sacramento River WSEL at the intakes as opposed to the maximum surge HGL that was developed with the maximum Sacramento River WSEL for the Central and Eastern Tunnel Corridor Alternative.

Table 10. Bethany Reservoir Alternative Main Tunnel – Minimum and Maximum HGL Values

Scenario	Minimum HGL [feet]	Maximum HGL [feet]
1	-48.1	35.7
2	-14.9	34.1
3	-44.6	38.7
4	-15.4	34.2

4.2.2 BRPP Discharge Aqueduct

Results for the transient analyses for the Bethany Reservoir Aqueduct pipelines are presented on Figure 51. Since each aqueduct pipeline operates in parallel with one another and diameters are identical and sized for a maximum flow capacity of 1,500 cfs, Figure 46 results are applicable for all four pipelines to Bethany Reservoir for a project design capacity of 6,000 or 7500 cfs. Results for the discharge pipeline to the Jones Pumping Plant approach channel are presented on Figure 52. Figures 51 and 52 show the simulated the envelop of the maximum and minimum HGL elevations during the transient-surge events along the aqueduct pipelines.

Hydraulic transient-surge mitigation features for the Bethany Reservoir Aqueduct simulated in the analyses include four identically sized one-way surge tanks and combination air and vacuum release valves located along each pipeline. Each Bethany Reservoir Aqueduct pipeline would be connected to a separate Surge Tank, as shown on the engineering concept drawings. Each one-way Surge Tank would be configured to empty its stored water into the connected pipeline when the HGL of the pipeline falls below the tank's free-water surface elevation. During this condition, check valves located at the outlet of each tank would open and allow stored water within the tank to flow into the connected pipeline. The stored volume and free-water surface elevation within each tank have been sized to maintain the internal pressures of the connected pipeline to within its conceptual design limits (between -7 psi on the low pressure side and 225 psi on the high pressure side [1.5 times pipeline working pressure rating of about 150 psi]). Each Surge Tank would be identically sized with a finished inside diameter of 75 feet, finished floor elevation of 45.00 feet, and tank side wall height of 20 feet, as shown on the concept drawings. The free-water surface elevation of the stored water volume that was simulated within each tank for this analysis was elevation 60 feet (15 feet above the tank's finished floor).

Hydraulic transient-surge mitigation features for the discharge pipeline to the Jones Pumping Plant approach channel as simulated in this analysis included combination air and vacuum release valves located along the pipeline alignment and a weir wall located within the pipeline's outlet structure. The top of weir within the outlet structure was set at elevation 30.00 feet, as shown on the engineering concept drawings. The weir wall would maintain full pipe flow within the Aqueduct for all flow capacities from 0 to 1,500 cfs. A one-way surge tank is not required for surge protection on this pipeline.

The minimum and maximum HGL envelopes On Figures 51 and 52 are plotted across the pipeline alignments between the BRPP discharge piping system and the Bethany Reservoir Discharge Structure and Jones Outlet structure, respectively, for the steady-state flow condition of 1,500 cfs per pipeline. Each pipeline's invert elevation and steady-state HGL is plotted along its alignment. Locations of the pipelines' points of connection to the BRPP and outlet structures are noted in each graph.

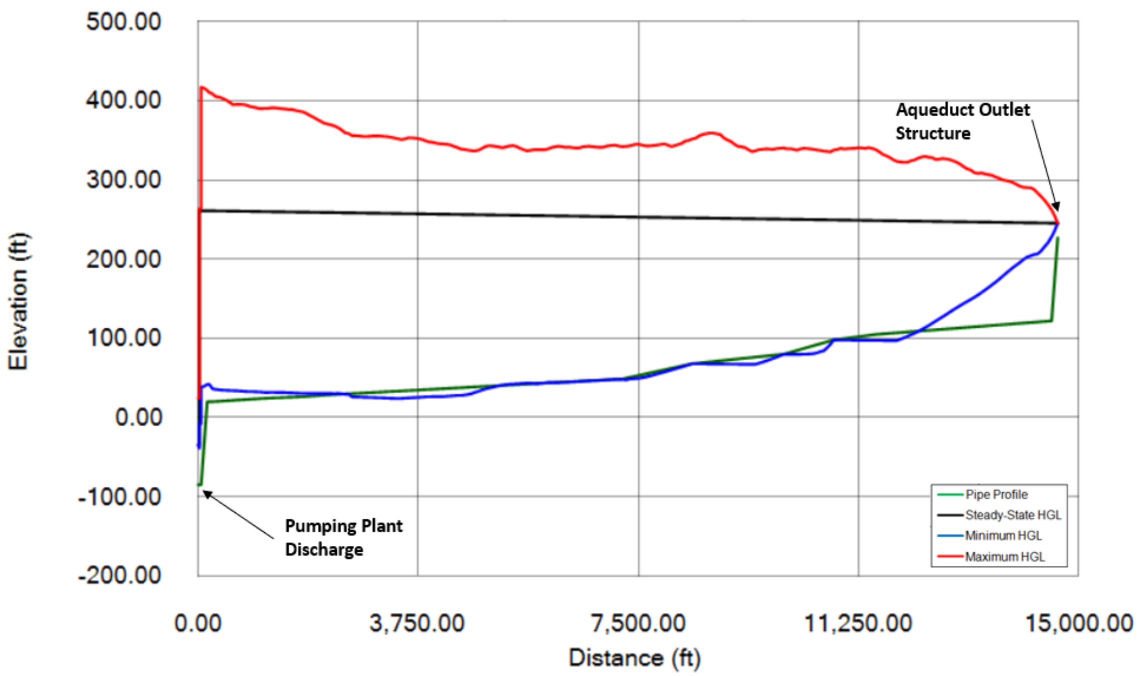

Figure 51. Bethany Reservoir Aqueduct; Design Capacity 1,500 cfs; Minimum and Maximum Hydraulic Gradeline Profiles

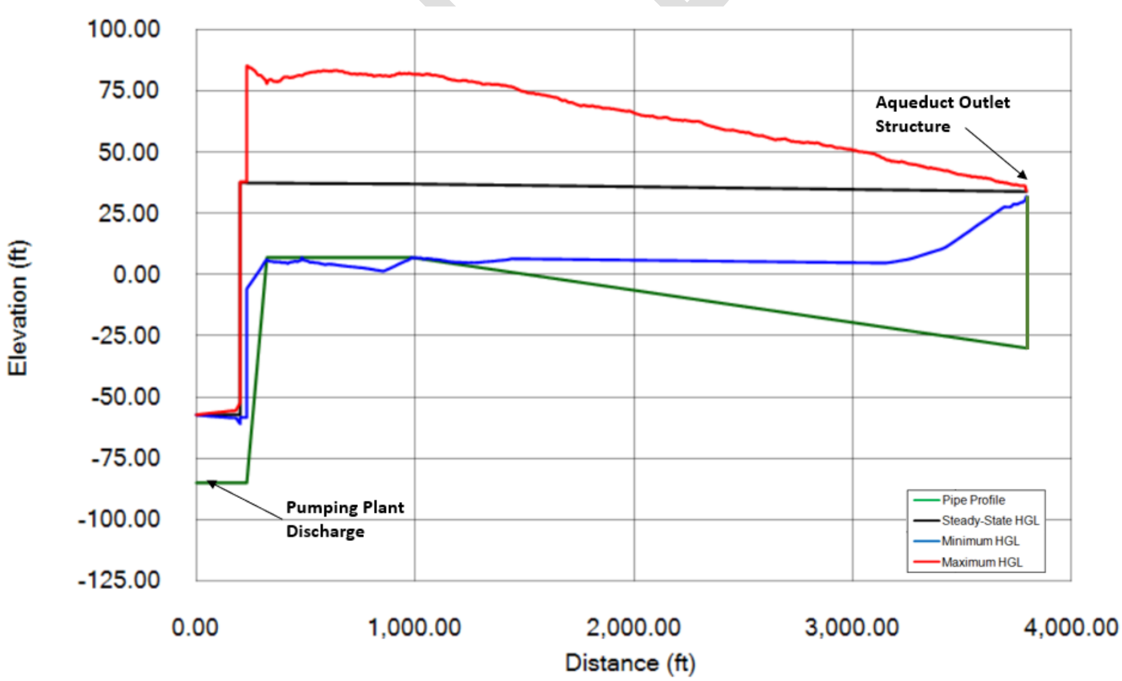

Figure 52. Jones Pumping Plant Approach Channel Aqueduct; Design Capacity 1,500 cfs; Minimum and Maximum Hydraulic Gradeline Profiles

The results for the Bethany Reservoir Aqueduct indicate negative internal pressures are developed at several locations along the alignment, as shown on Figure 51. The minimum internal pressure was -4.5 psi, which occurred at a location of 10,540 feet from the BRPP discharge point of connection. The maximum pressure of 220 psi occurs immediately downstream of the pump control valves within the BRPP. On the basis of these results and as shown on Figure 51, the HGL envelope developed during the transient-surge event is within the conceptual design limits of the aqueduct.

The results for the Jones Pumping Plant approach channel aqueduct indicate negative internal pressures were developed during the transient-surge event within the first 1,000 feet of the aqueduct, as shown on Figure 52. The minimum internal pressure was -2.4 psi which occurred at a location of 851 feet from the BRPP discharge point of connection. The maximum pressure of 75 psi occurs immediately downstream of the pump control valves within the BRPP. On the basis of these results and as shown on Figure 52, the HGL envelope developed during the transient-surge event is within the conceptual design limits of the aqueduct. See Table 11 for the maximum and minimum HGL values that occur within the HGL envelope developed for the Jones Pumping Plant approach channel aqueduct.

Table 11. Bethany Reservoir Alternative – Minimum and Maximum HGL Values

Aqueduct	Minimum HGL [feet]	Maximum HGL [feet]
Bethany Reservoir	35.5	417.3
Jones Pumping Plant Approach Channel	-60.9	85.3

5. Tunnel Dewatering

5.1 Tunnel Dewatering Assumptions

A tunnel dewatering analysis was conducted for the tunnel between the Sacramento River Intakes and the Surge Basin Reception Shaft and connecting BRPP wet well. To allow inspection, maintenance, or repair, the main tunnel and respective shafts would be designed to be dewatered. The 36-foot tunnel and project design capacity of 6,000 cfs option were used in the tunnel dewatering analysis.

Dewatering was simulated using two sequential pumping modes, as follows:

- Dewatering Using Permanent Pumps (within the BRPP):
  - Initial steady-state HGL within the tunnel and WSEL within the BRPP wet well of 26.3 feet (with all pumps off) was used as the initial condition for the dewatering analysis. The HGL of 26.3 feet is the maximum Sacramento River WSEL at C-E-5.
  - Each pump was sequentially started every 4-minutes until 12 pumps were in operation. With 12 pumps in operation, the combined pumped flow was 6,000 cfs (500 cfs per pump).
  - When the WSEL of -44.5 feet was reached within the BRPP wet well, the first pump was shutdown. Each consecutive pump shutdown occurred within 0.5 feet increments of falling wet well level. The last pump in operation was stopped at the wet well WSEL of -50.0 feet.
  - Transient waves that were generated within the tunnel following the shutdown of the permanent pumps were allowed to dissipate for a 12-hour period (with all pumps off). Due to the momentum

change of the pumped flow during the pump shutdown sequence and the established tunnel HGL gradient (higher HGL in the wet well than at the intake shafts during pump shutdown process), the average WSEL of -66.80 feet within the wet well was achieved at the end of the 12-hour period. After the 12-hour period, the WSEL in the wet well rose and fell 2-feet above and below the WSEL of -66.80 over 20-minute time intervals.

  - All pumps were controlled in the ICM model by RTC rules:
    - All pumps were operated within their maximum and minimum speed range. The speed range was defined between 50 percent and 100 percent of the manufacturer's maximum rated speed for each pump.
    - All pumps were operated within their allowable operating range (AOR) over their full operating speed range.

- Dewatering Using Submersible Vertical Turbine Pumps:
  - Submersible pumps would be temporarily (or permanently) installed in the Surge Basin reception shaft located just upstream of the BRPP wet well. The submersible pumps' discharge piping would be routed from the Surge Basin shaft structure to a point of connected to the BRPP discharge aqueduct to convey pumped flow directly into the Bethany Reservoir as shown on the engineering concept drawings. The submersible discharge piping from the submersible pumps would not be connected to the Jones Pumping Plant approach channel aqueduct.
  - Initial WSEL within the tunnel and the BRPP wet well was -66.80 feet which was established after the permanent pumps shut down. The submersible pumps were started after a 12-hour time delay following the shutdown of the permanent pumps to allow transient waves to dissipate within the tunnel.
  - Final WSEL within the tunnel: -164.18 feet (tunnel invert elevation at the point of connection to the Surge Basin Reception Shaft)
  - Pumped flow capacities associated with the submersible vertical turbine pumps were analyzed to establish recommended maximum dewatering flows corresponding to lower WSELs in the tunnel and Surge Basin reception shaft.

During dewatering, at shallow flow depths (lower water surface elevations) within the tunnel, flow velocities and depths may approach critical hydraulic conditions and become unstable resulting in the formation of hydraulic jumps. The formation of hydraulic jumps within the tunnel may result in too low of a net positive suction head available (NPSHa) condition at the pump causing a pump shut down due to insufficient flow and/or insufficient suction head entering the pump suction.

This hydraulic analysis was conducted to determine maximum recommended dewatering flow capacities corresponding to the WSELs within the tunnel to avoid the formation of hydraulic jumps within the tunnel, and to provide guidelines for the selection and operation of the submersible pumps for dewatering the tunnel.

The dewatering volume calculations do not include volumes at the intake facilities except the intake drop shafts. It is assumed that the radial gates upstream of the intake drop shafts would be closed during dewatering process. Figure 53 shows the project schematic with proposed submersible dewatering pump placement.

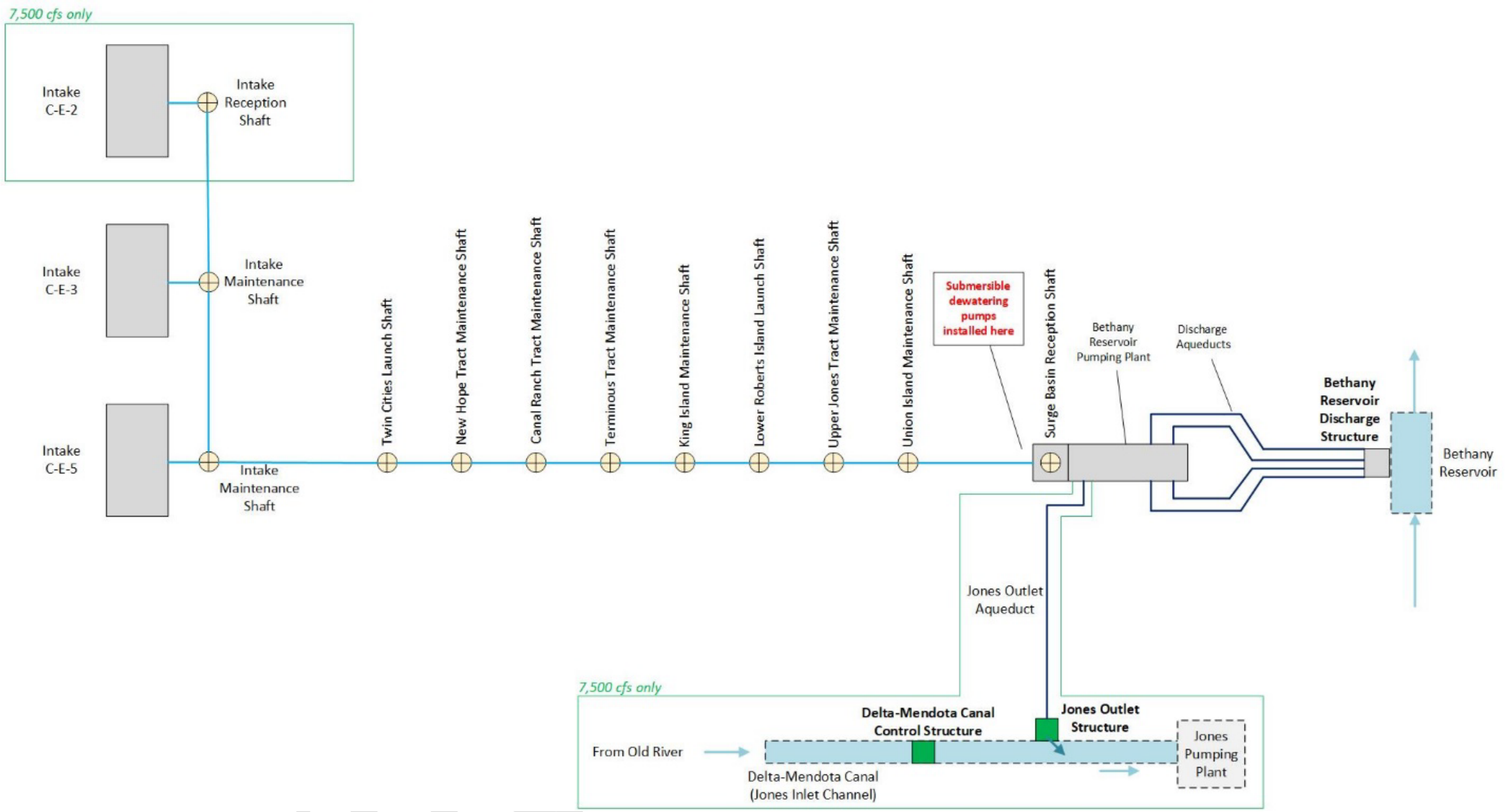

Figure 53. Project Schematic with Proposed Submersible Dewatering Pumps Location

5.2 Modelling Results

Figure 54 provides the results of tunnel dewatering flows for the Bethany Reservoir Alternative tunnel alignment with Manning's *n* coefficients of 0.016 and 0.014, whereby hydraulic jumps within the tunnel would not be formed (that is, flows are subcritical). As can be seen on Figure 48, dewatering flows (identified as Instantaneous Flow) are plotted along the x-axis and tunnel WSEL (referenced at the tunnel exit into the Surge Basin Reception Shaft) are plotted along the y-axis. Subcritical flow curves versus tunnel WSEL were plotted at Froude Numbers of 0.8, 0.85, 0.9 and 0.95. The Froude Number of 0.80 was selected for this analysis to provide a conservative estimate against the critical flows and critical depths associated with dewatering rates within the project's main tunnel.

To determine the maximum subcritical flow on Figure 54, select a WSEL within the Surge Basin Reception Shaft, find the intersection of the subcritical flow curve associated with the Froude Number of 0.80 at the selected Surge Basin Reception Shaft WSEL and determine the corresponding instantaneous flow. The instantaneous flow value provides the maximum tunnel dewatering flow rate that can be achieved without forming a hydraulic jump within the tunnel. As can be seen on Figure 54 for instantaneous WSELs in the Surge Basin Reception Shaft structure above -148.0 feet, permissible tunnel dewatering flows are above the maximum design flow capacity of 6,000 cfs for the project. As such, the use of the main pumps within the BRPP may operate unrestricted up to the maximum design flow capacity of 6,000 cfs and down to a WSEL within the wet well of -50.0 feet (limited by pump submergence). For WSELs within the Surge Basin Reception Shaft structure of less than -148.0 feet, dewatering flows must not exceed the instantaneous flows shown on Figure 54.

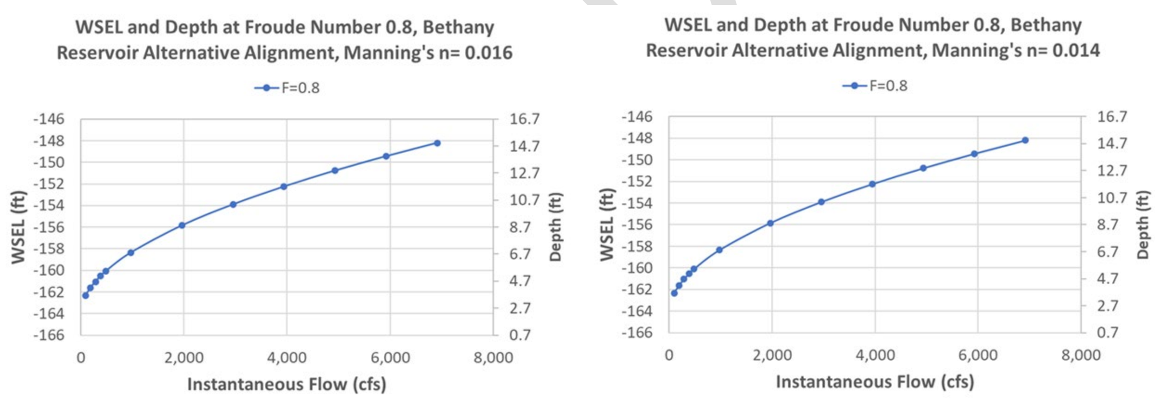

Figure 54. Instantaneous Flow and Level at Froude Numbers of 0.80, 0.85, 0.90, and 0.95 (Left: Manning's *n* = 0.016, Right: Manning's *n* = 0.014)

Tables 12 and 13 show the results of the analysis, which provide the maximum recommended dewatering flow capacities within the tunnel corresponding to the WSELs within the Surge Basin Reception Shaft at and below -148.0 that would not result in the formation of hydraulic jumps throughout the entire main tunnel alignment.

Table 12. Maximum Recommended Dewatering Flows Versus Water Levels for Manning's n = 0.016

Froude Number = 0.8		
Tunnel Flow (cfs)	WSEL in Surge Basin Reception Shaft (feet)	Depth above Tunnel Invert Elevation at Surge Basin Reception Shaft (feet)
7,000	-148.1	16.0
6,000	-149.4	14.7
5,000	-150.7	13.4
4,000	-152.2	11.9
3,000	-153.8	10.3
2,000	-155.8	8.3
1,000	-158.3	5.8
500	-160.0	4.1
400	-160.5	3.6
300	-161.0	3.1
200	-161.5	2.6
100	-162.2	1.9

Table 13. Maximum Recommended Dewatering Flows Versus Water Levels for Manning's n = 0.014

Froude Number = 0.8		
Tunnel Flow (cfs)	WSEL in Surge Basin Reception Shaft (feet)	Depth above Tunnel Invert Elevation at Surge Basin Reception Shaft (feet)
7,000	-148.2	16.0
6,000	-149.4	14.7
5,000	-150.8	13.4
4,000	-152.3	11.9
3,000	-153.9	10.3
2,000	-155.8	8.3
1,000	-158.3	5.8
500	-160.1	4.1
400	-160.5	3.6
300	-161.0	3.1
200	-161.6	2.6
100	-162.3	1.9

5.3 Dewatering Volume

The calculated total dewatering volume required to completely dewater the project's main tunnel and connecting shafts between the intakes to the Surge Basin reception shaft structure starting with an initial HGL for the tunnel system of 26.3 feet is 268,627,500 cubic feet.

5.4 Dewatering Pumps

For this evaluation, the permanent pumps would be initially operated to lower the BRPP wet well down to a WSEL of -50 feet. Submersible vertical turbine pumps were considered as the dewatering pumps for elevations below -50.0 feet. The dewatering pumps could be stored in the equipment storage building within the BRPP complex (per the manufacturer's instructions) when not in use (or left permanently installed and periodically exercised per the manufacturer's instructions). Each pump would be installed within the Surge Basin Reception Shaft and supported from the Surge Basin bridge structure. A common 60-inch-diameter welded steel discharge pipeline (permanently installed) would be routed to the BRPP structure. The pump discharge pipeline would be equipped with a flow meter, pressure control valves and an isolation valve that would be located within an intermediate floor within the BRPP structure. Pumps would operate with adjustable-frequency drives (AFDs) which are permanently installed in the BRPP structure, as shown on the engineering concept drawings.

5.4.1 Candidate Pump Manufacturer and Performance Requirements

The pump manufacturer Andritz was consulted for selections of candidate submersible vertical turbine pumps for tunnel dewatering. The pump considered in this analysis is among the largest Andritz offers for the range of flow and head conditions associated with dewatering the main tunnel. The candidate manufacturer's pump performance curve was evaluated based on the required envelope of system flow and head conditions as previously defined. The pump selection from Andritz was used to illustrate the performance requirements at various system head conditions.

Figure 55 shows the system static head conditions developed for high head and low head conditions as discussed here.

- The high head system head curve (SHC) is the maximum total dynamic head conditions encountered by each of the two dewatering pumps. This condition represents the maximum static head condition between the Surge Basin Reception Shaft and the Bethany Reservoir WSELs, respectively. For this SHC, the WSEL in the reception shaft was set at -164.18 feet which is the invert elevation of the tunnel and the maximum WSEL at the Bethany Reservoir was estimated at 245.00 feet.
- The low head SHC is the minimum total dynamic head conditions encountered by each of the two dewatering pumps. This condition represents the minimum static head condition between the Surge Basin reception shaft and the Bethany Reservoir WSELs, respectively. For this SHC, the WSEL in the reception shaft was set at -50 feet which is the WSEL when the dewatering pumps would be started. The WSEL in the Bethany Reservoir was set at 238.00 feet.

The system maximum and minimum static head conditions were plotted against the candidate pump performance curve, as shown on Figure 55.

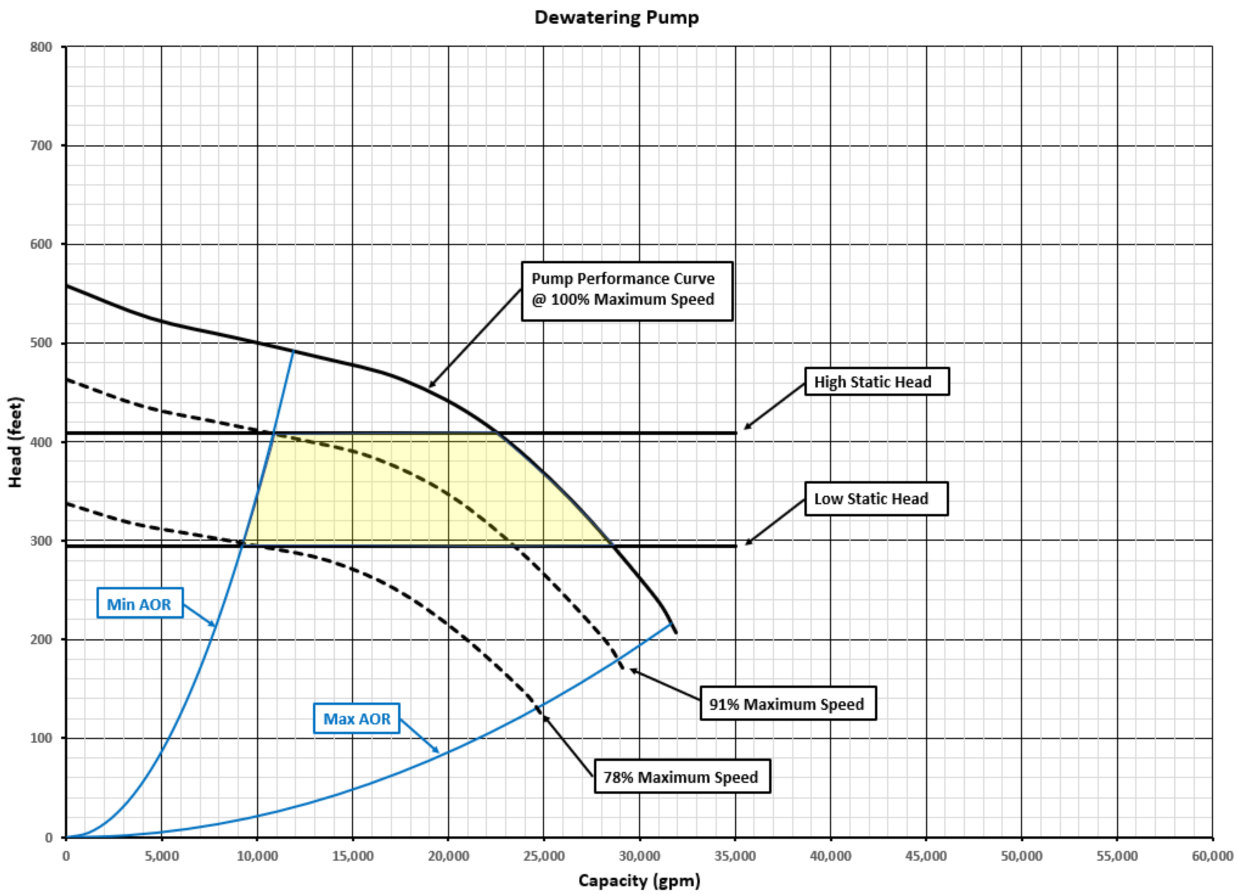

Figure 55. Candidate Pump Performance Curve Versus System Head Curves

The candidate pump selection is the Andritz, Model 6780.1/2 pump with a maximum rated operating speed of 1,190 rpm, 2,750 horsepower, with a rated capacity of 20,835 gallons per minute (gpm) (46.43 cfs) at 435 feet of total dynamic head. The minimum and maximum flows defining the pump's AOR are also plotted with blue curves, as shown on Figure 55.

As can be seen on Figure 55, the maximum flow achievable at the high static head condition is 22,500 gpm (50.1 cfs) at 409.0 feet. The maximum flow achievable at the low static head condition is 28,500 gpm (63.5 cfs) at 295.0 feet. Each of the maximum flow conditions of the pump are established at the pumps maximum rated speed.

As can be seen on Figure 55, the entire envelop of system head conditions are within the pump's AOR. For this pump, a flow control valve would not be required for throttling. However, space has been provided within the conceptual design of the BRPP to accommodate other pump selections that may require throttling based on the pump's required AOR. To reduce the pumped flow capacity based on the instantaneous flow restrictions shown on Figure 54, and Tables 10 and 11 the dewatering pumps will operate with variable frequency drives. Referring back to Figure 55, under low static head conditions, the minimum achievable flow within the pump's AOR would be 9,000 gpm (20.1 cfs) at an operating speed of 78 percent of the pump's maximum rated speed. Under the high static head conditions, the minimum achievable flow within the pump's AOR would be 11,000 gpm (24.5 cfs) at an operating speed of

91 percent of the pump's maximum rated speed. Each of the minimum achievable flow capacities is below the maximum instantaneous flow limits shown in Tables 10 and 11.

Based on this discussion, the two dewatering pumps (operating in parallel) can deliver up to a combined flow rate range of 45,000 gpm (100 cfs) to 57,000 gpm (127 cfs) between the lowest to highest system head conditions associated with the dewatering process. Each pump would be operated with a VFD. A flow meter would be used to control the operating speed of each pump based on the desired flow set-point. The pump selection would maintain the dewatering flow rate well within the maximum flow rates established in Tables 10 and 11 throughout the entire dewatering process.

6. Dewatering Duration

Twelve permanent pumps within the BRPP were operated for a total combined pumped flow capacity of 6,000 cfs. The first pump was started with an initial BRPP steady-state wet well WSEL of 26.3 feet. Each additional pump was started in four-minute intervals until all twelve pumps were in operation. Following each pump's startup, operating speeds were adjusted to maintain 500 cfs for each pump in operation. When the WSEL in the BRPP wet well achieved -44.5 feet, the first permanent pump was shutdown. Each consecutive pump shutdown occurred in 0.5 feet increments of falling wet well level. The last pump in operation was stopped at the wet well WSEL of -50.0 feet. The total duration required to lower the BRPP WSEL from 26.3 feet to -50.0 feet (after the first permanent pump was started and last the pump was stopped) was 2 hours. A 12-hour time delay following the shutdown of the permanent pumps was simulated to allow transient waves to dissipate within the tunnel following the permanent pump shutdown sequence.

After the 12-hour time delay following the shutdown of the permanent pumps, the tunnel HGL gradient subsided and the BRPP wet well WSEL converged to -66.8 feet. Both candidate dewatering pumps were then started and operated at their maximum speed from the starting WSEL range within the Surge Basin Reception Shaft of -66.8 feet to -162.68 feet (1.50 feet above the tunnel invert elevation in the Surge Basin Reception Shaft). Below the WSEL of -162.68 feet, only one pump could be operated at reduced speed with excessive cycling (starting and stopping) due to the formation of hydraulic jumps within the tunnel. Therefore, the installation of additional, smaller capacity submersible pumps (potentially installed in the shafts at the southern end of the tunnel) would be required to pump out the remaining water (below the HGL of -162.68 feet). The time duration to dewater the tunnel between HGLs of -66.8 feet to -162.68 feet with the two vertical turbine submersible pumps in operation was 28.5 days. The remaining water volume left in the tunnel below the HGL of -162.68 is 810,000 cubic feet. Assuming three additional submersible pumps (each pumping 3.3 cfs [1,500 gpm]), the time duration to pump the remaining water from the tunnel would be about 24 hours.

Based on the results of the model simulation, the total duration required to completely dewater the main tunnel starting with a steady-state HGL of 26.3 would be about 722 hours (30.1 days).

7. Conclusions

The operational study performed 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.

Based on the results of hydraulic transient-surge analyses for the Bethany Reservoir Alternative, the maximum and minimum HGL envelopes for the tunnel and pipelines were found to be within the conceptual design pressure limits for the boundary conditions evaluated at the Project's design flow capacities of 6,000 and 7,500 cfs options. The top of shaft structure elevations for shafts located between the Sacramento River intakes and the Surge Basin structure have been established in the concept design to provide a minimum 3-foot freeboard above the maximum HGL (at each shaft) calculated per the greater HGL elevation associated with the transient-surge analyses for the main tunnel or the Sacramento River 200-year flood with sea level rise WSEL at the intakes.

Dewatering of the tunnel for the Bethany Reservoir Alternative, at a Project design capacity of 6,000 cfs, can be achieved using the permanent pumps for initial BRPP wet well WSEL drawdown from 26.3 to -50 feet then using submersible pumps located at the Surge Basin Reception Shaft. A more detailed analysis will be performed during the future phase of the design to further evaluate tunnel dewatering for WSEs below 1.5 feet above the tunnel invert (at the Surge Basin Reception Shaft) which would include several portable, small capacity sump pumps.

8. References

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9. 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 Suibing Liu / EDM SME InfoWorks ICM Kevin Neilson / EDM SME Transient	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 Naimey, California Professional Engineering License M28450.

Note to Reader

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