Appendix A2. Hydraulic Analysis of Delta Conveyance Options (Final Draft)

1. Introduction and Purpose

This technical memorandum (TM) describes the preliminary hydraulic analysis performed for the Bethany Reservoir Alignment for the Delta Conveyance Project (Project). This analysis considers a single main tunnel sized to a 36-foot-inside-diameter and a maximum Project design flow capacity of 6,000 cfs. The size of the main tunnel for the Project was based on the results of a preliminary Project capacity and tunnel diameter sizing analysis conducted for the Central Corridor tunnel alignment that was previously considered for the Project. A copy of the Project capacity and tunnel diameter sizing analysis for the Central Corridor tunnel alignment is provided in Attachment 1.

The tunnel diameter sizing analysis for the Central Corridor tunnel alignment was completed prior to the development of the Eastern Corridor tunnel alignment option and the Bethany Reservoir Alignment. For the Central Corridor tunnel alignment, Intakes C-E-2 and C-E-3 would each be designed for the maximum Sacramento River diversion flow capacity of 3,000 cubic-feet-per-second (cfs) to achieve the Project design flow capacity of 6,000 cfs. The results of this analysis led to the decision to size the main tunnel diameter for the Central Corridor to 36-foot-inside-diameter for the single tunnel system for the Project design flow capacity of 6,000 cfs.

The analyses criterion, methodology and results described in Attachment 1 remain appropriate for selecting the tunnel size of 36-foot-inside-diameter for the Bethany Reservoir Alignment single tunnel system for the maximum design flow capacity of 6,000 cfs. Based on the selected tunnel diameter of 36-foot-inside-diameter for the Bethany Alignment, at the Project design flow capacity of 6,000 cfs, the operating free-water surface elevation range of the Bethany Reservoir Pumping Plant's below ground level wet well was considered feasible. Further the preliminary hydraulic and transient-surge analyses conducted for the Bethany Reservoir Alignment at the Project design flow capacity of 6,000 cfs described in this TM demonstrated that internal pressures within the tunnel were maintained within the conceptual-level design criteria for the single tunnel system. Therefore, a 36 foot-inside-diameter tunnel was selected for the Bethany Reservoir Alignment.

Results of this analysis were used to:

- Develop the hydraulic gradeline (HGL) envelopes for the 6,000 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 Alignment Project Schematic

1.1 Organization

This TM is organized as follows:

- Methodology
- Analysis and Evaluation
- Surge Analyses
- Tunnel Dewatering
- Dewatering Duration
- Conclusions
- References
- Attachment 1- Capacity Analysis for Preliminary Tunnel Diameter Selection

2. Methodology

The following describes the methodology for this evaluation:

- Selected the tunnel with finished inside diameter of 36-foot to be evaluated for the Project design flow capacity of 6,000 cfs.
- Conducted system end-to-end hydraulic head loss for the Bethany Reservoir Alignment tunnel alignment corridor using the candidate tunnel diameter and selected 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 for the design flow capacity of 6,000 cfs
- 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 the candidate tunnel diameter and developed the envelope of maximum and minimum HGLs for each transient-surge condition.

3. Criteria

The Concept Engineering Report (CER)) Appendix A1 *Hydraulic Analysis Criteria* outlines the preliminary criteria used for this analysis.

The following criteria were developed to guide the evaluation of achievable dewatering flows for the Bethany Reservoir alignment tunnel alignment.

- Standard Roughness Coefficients: Manning's "n" of 0.014 and 0.016 were utilized as the minimum and maximum standard roughness coefficients for the hydraulic analysis to evaluate the range of resultant dewatering flow conditions associated with both internal friction factors for the tunnel.
- Froude Number: 0.8 was selected to evaluate dewatering flows to avoid critical depths that form hydraulic jumps within sections of the tunnel.
- Governing Equations and Modeling Approach
  - The intakes, connecting conduits, sediment basins, tunnel and hydraulic control structures were modeled within the systemwide hydraulic model. The modeling approach and governing equations are defined in the Hydraulic Analysis Criteria (Draft) TM, dated May 29, 2020.

   - Rating curves for minor losses are pre-programmed into the Infoworks model. These have been reviewed and determined to be acceptable for application in the modeling effort.

- Tunnel and Shaft Diameters: The tunnel diameter is 36 feet and shaft IDs and locations are as shown on the concept drawings.
- Minimum variable operating speed for dewatering pumps was limited to 50 percent of the pump's maximum rated operating speed

3.1 Assumptions and Boundary Conditions

Appendix A1 *Hydraulic Analysis Criteria* 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 as follows:

- Locations of intakes, shafts, surge basin and pumping plant are shown on the Engineering Concept Drawings.
- Tunnel inside diameter connecting intakes C-E-3 and C-E-5 outlet shafts match the main tunnel.
- 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 Engineering Concept Drawings for each intake (C-E-3 and C-E-5):
   - Intake C-E-3: min WSEL of 3.72 feet; design WSEL of 4.59 feet; normal WSEL of 5.92 feet (50 percent annual exceedance); 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 (50 percent annual exceedance); and high WSEL of 26.3 feet
- Top of the overflow weir within the surge basin structure was set at elevation 18 feet.
- The WSEL of 245 feet was assumed for the Bethany Reservoir.

3.2 Tools

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

3.2.1 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. .

Following the BRPP is a pressurized Bethany Reservoir Aqueduct system that would be composed of four pipelines. Each pipeline would convey a maximum capacity of 1,500 cfs. The four parallel pressurized pipelines were modelled using the criteria for pressure pipe documented in CER Appendix A1 *Hydraulic Analysis Criteria*. 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.

3.2.2 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.

3.3 Real-Time Controls

3.3.1 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.

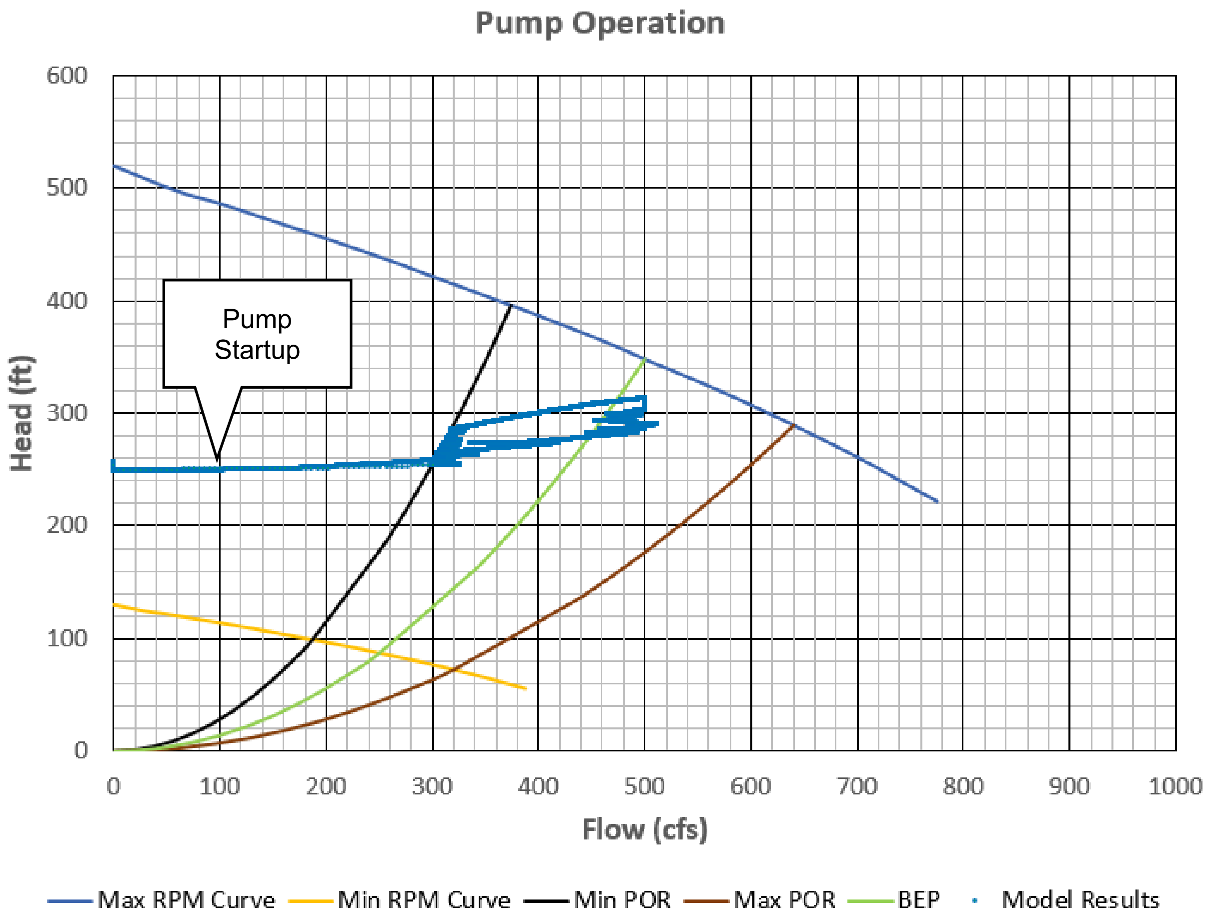

Figure 2. Example RTC Pump Operation Scatter Plot

3.3.2 RTC Rules

3.3.2.1 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.

3.3.2.2 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. 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.

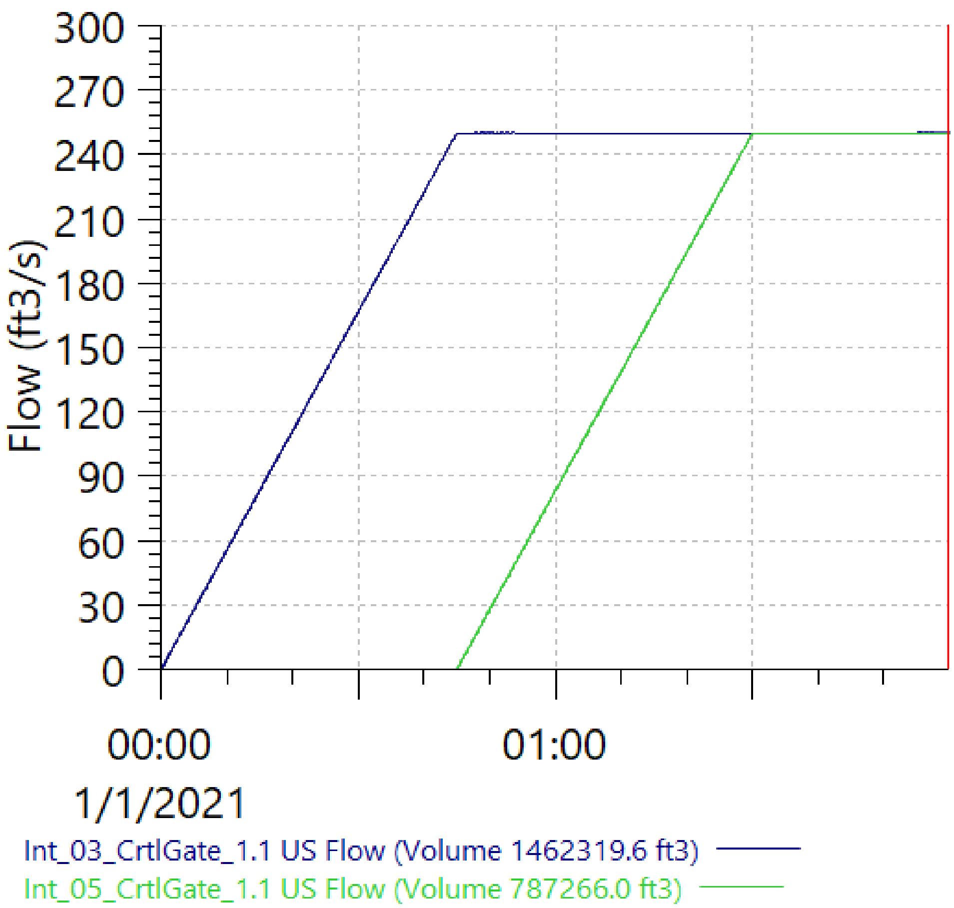

Figure 3. Intake Control Gate Startup Operation

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.

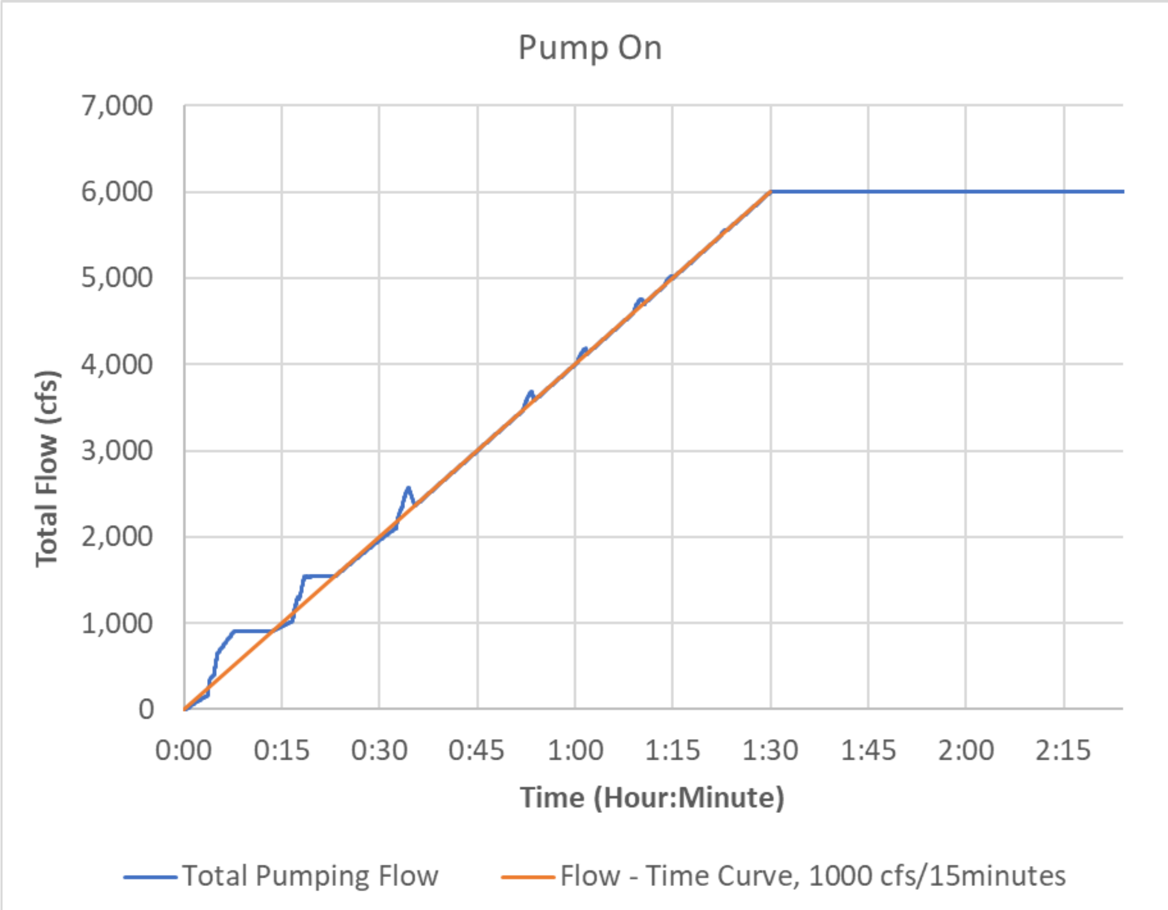

Figure 4. Real-time Control on Pump Startup Operation

3.3.2.3 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.

3.3.2.4 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.

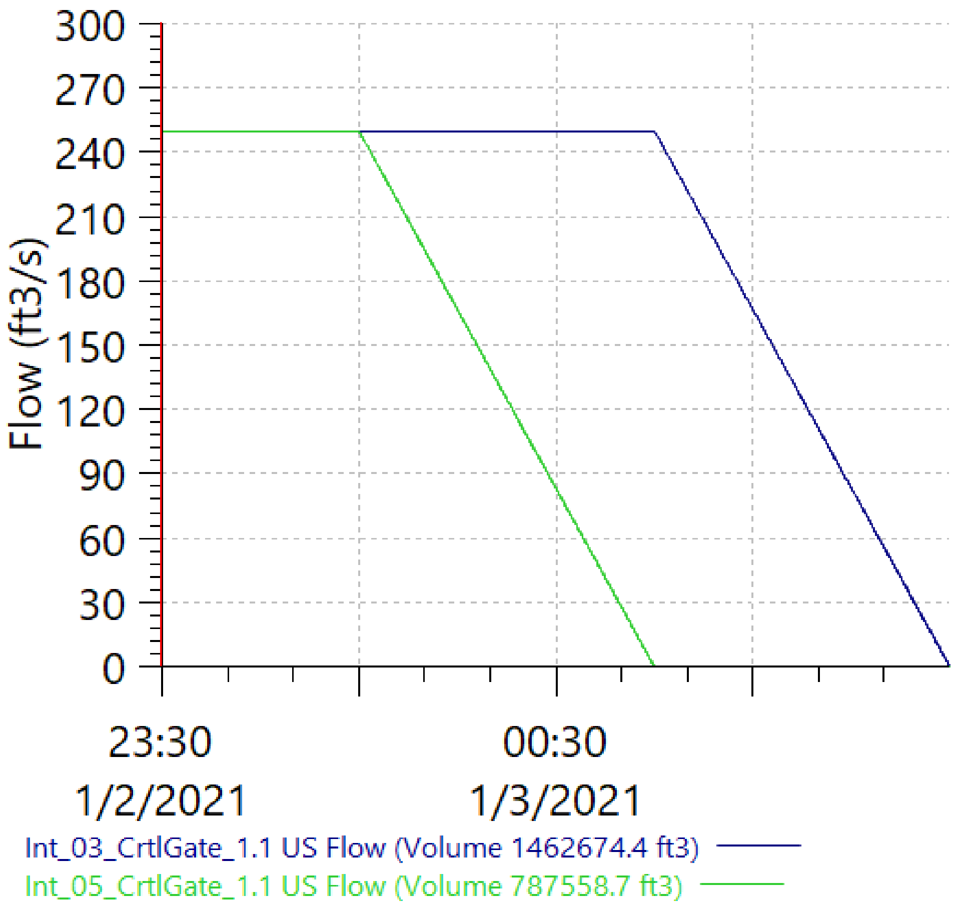

Figure 5. Intake Control Gate Shutdown Operation

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.

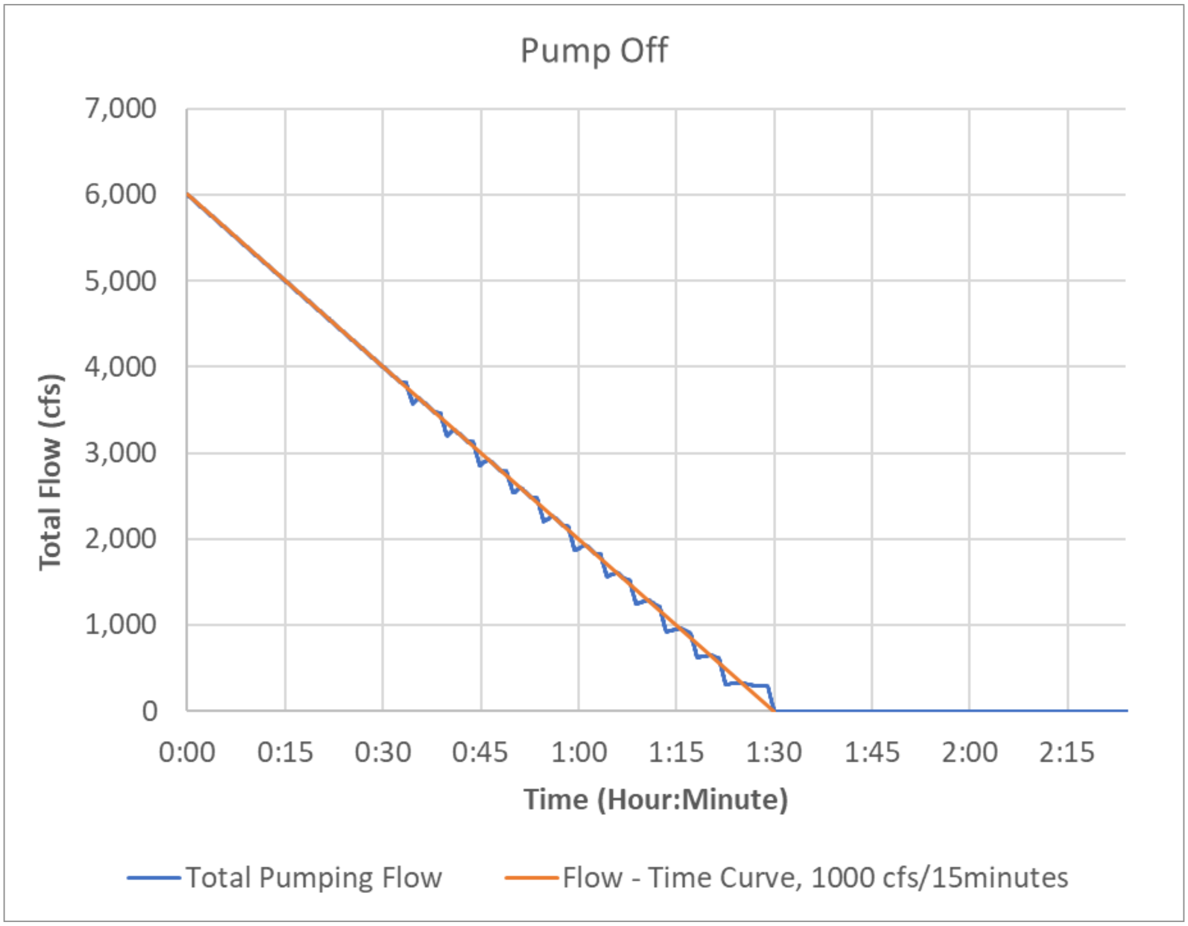

Figure 6. Real-time Control on Pump Shutdown Operation

3.3.3 Operational Cycle Study

3.3.3.1 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.

3.3.3.2 Model Simulation Results

The individual pump startup and the corresponding wet well water level variations are presented on Figures 7 through 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 through 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 through 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 through 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.

Figures 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 through 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 17 through 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 through 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.

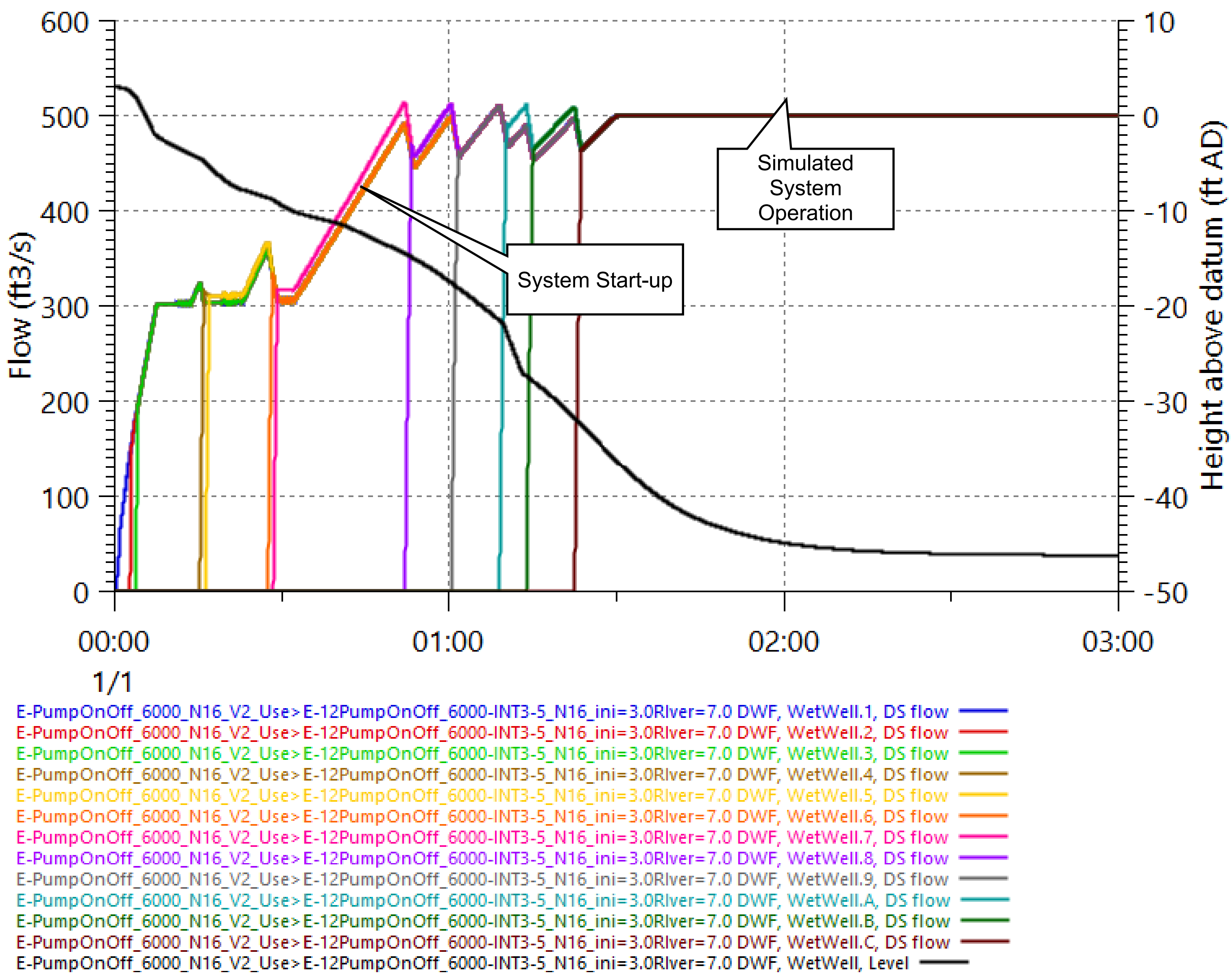

Figure 7. Pump Startup and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.016

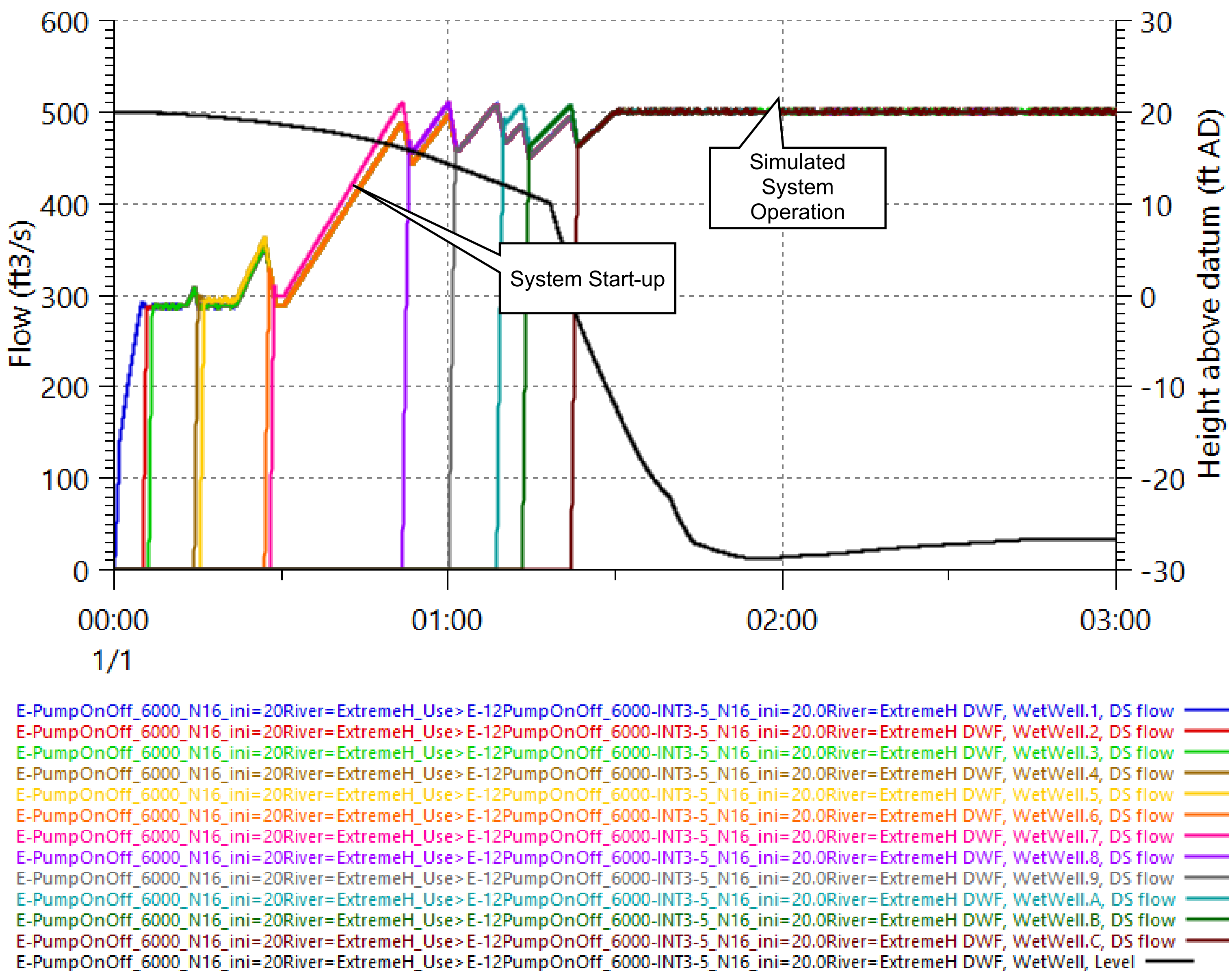

Figure 8. Pump Startup and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.016

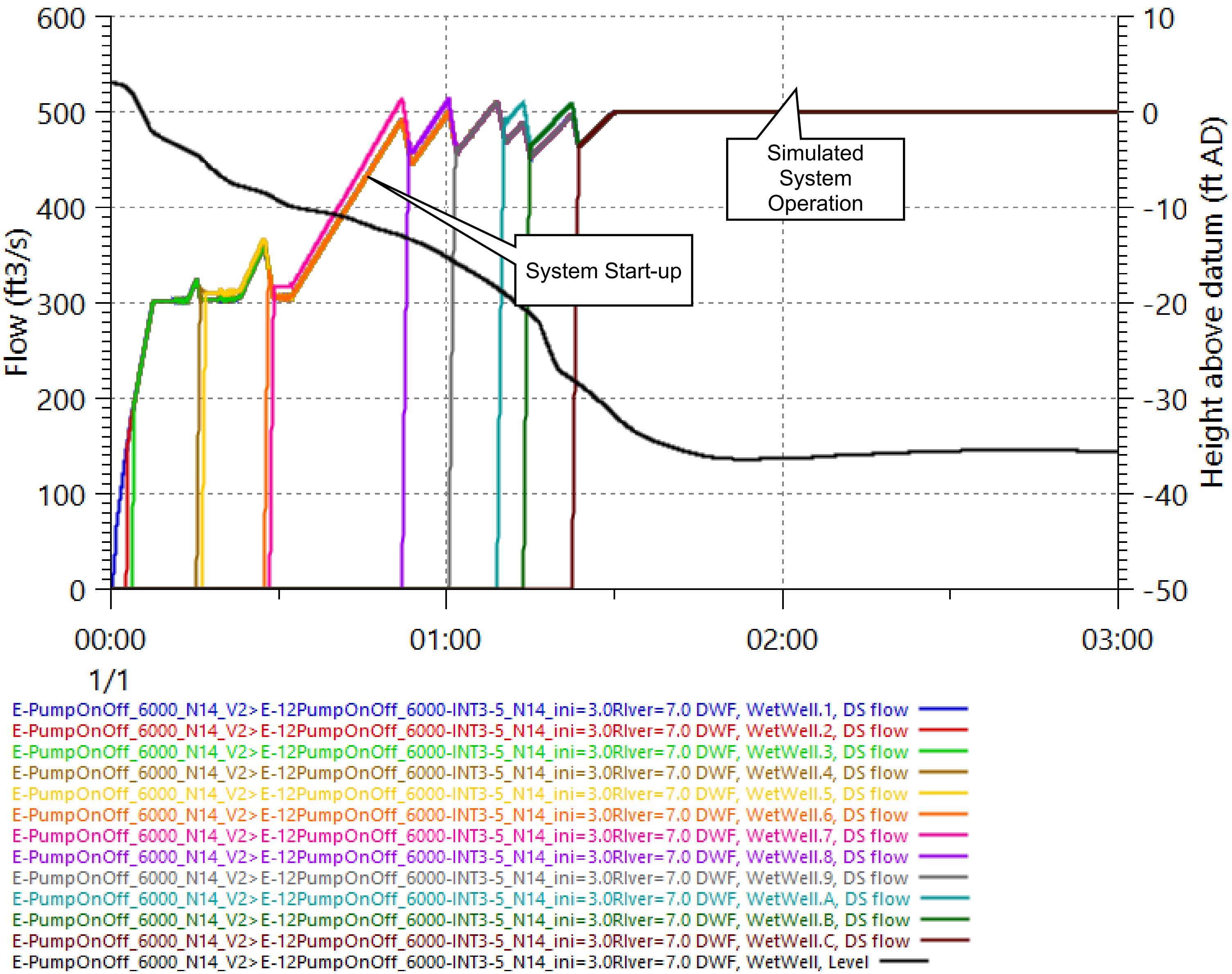

Figure 9. Pump Startup and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.014

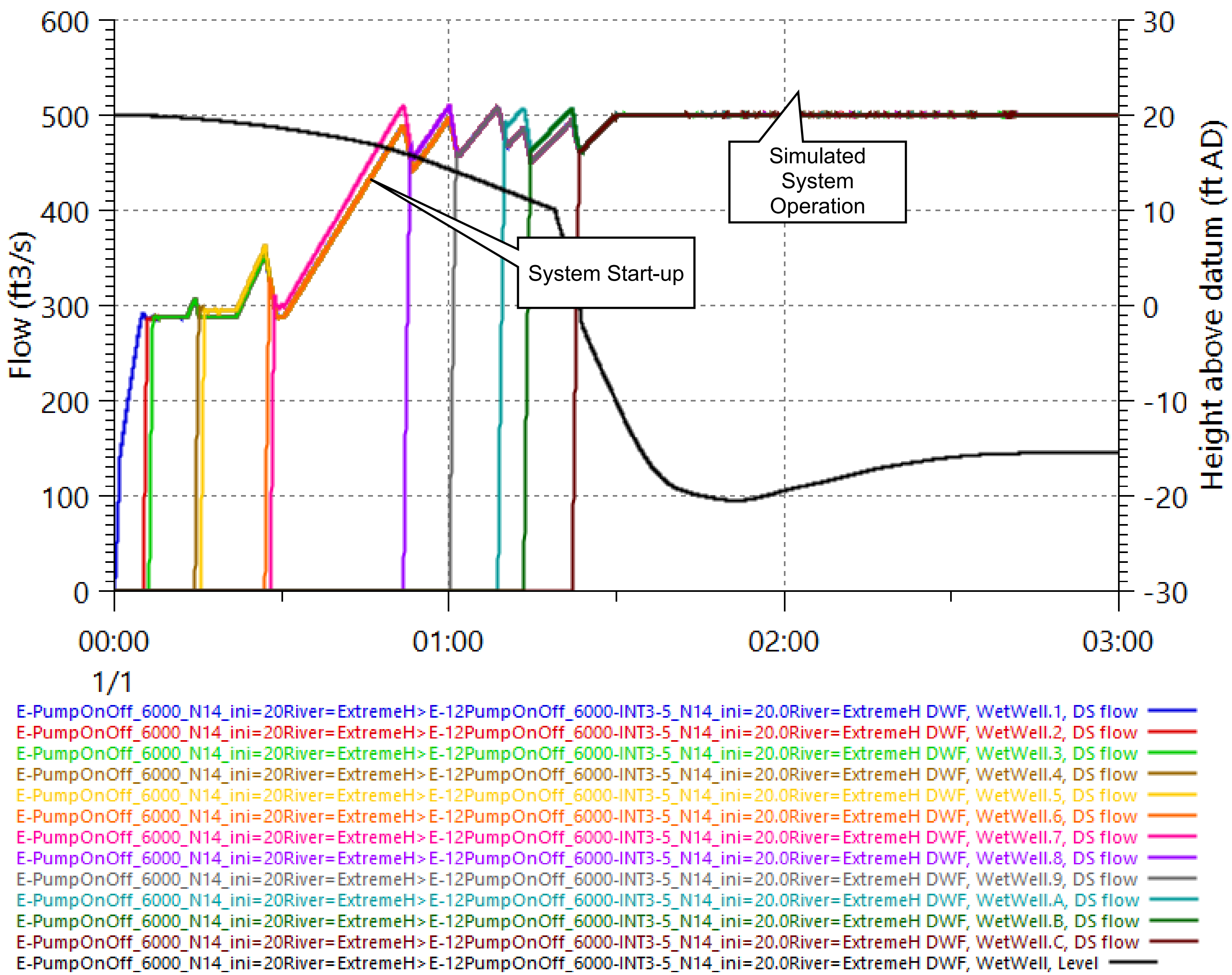

Figure 10. Pump Startup and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.014

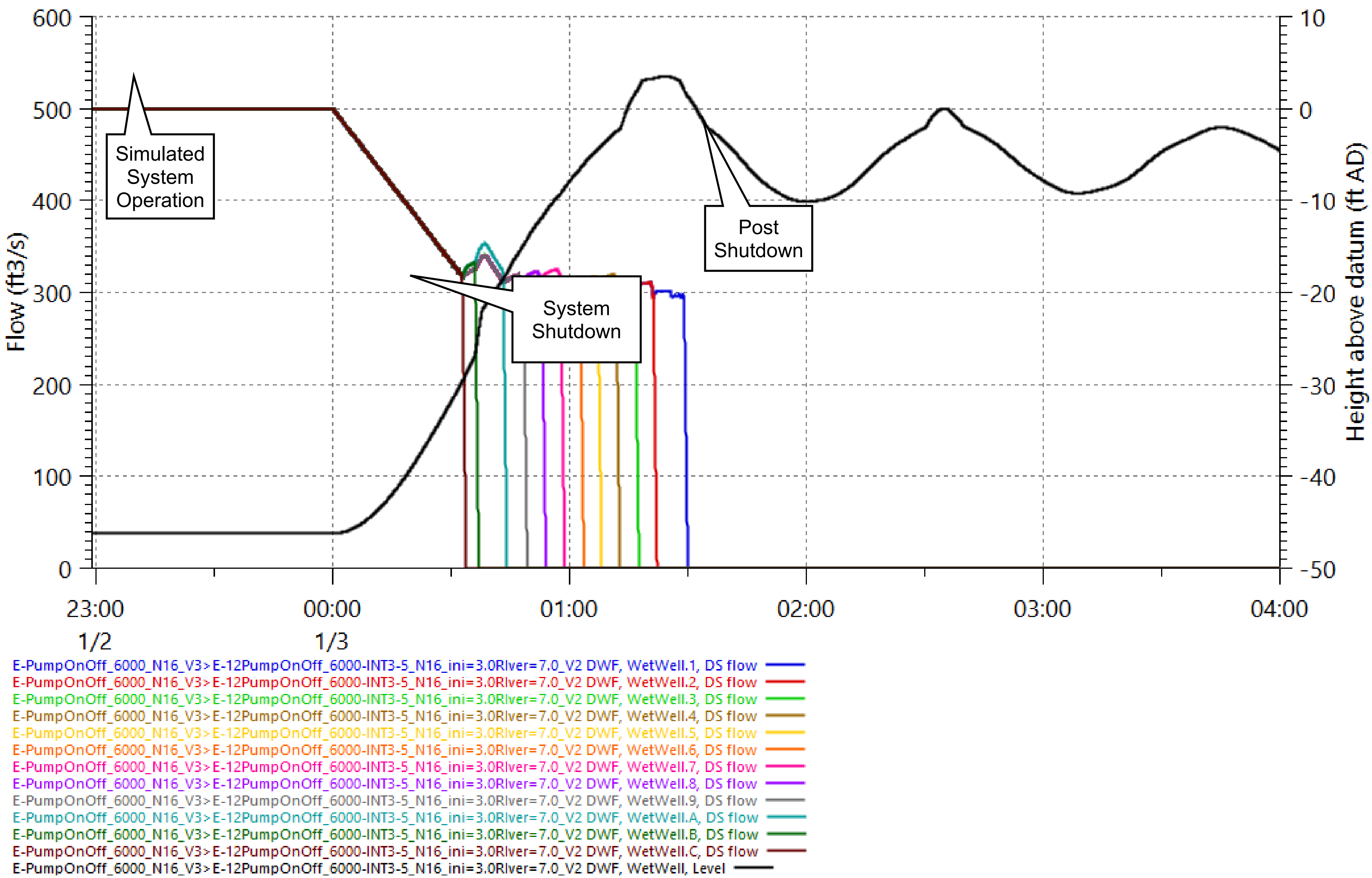

Figure 11. Pump Shut Down and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.016

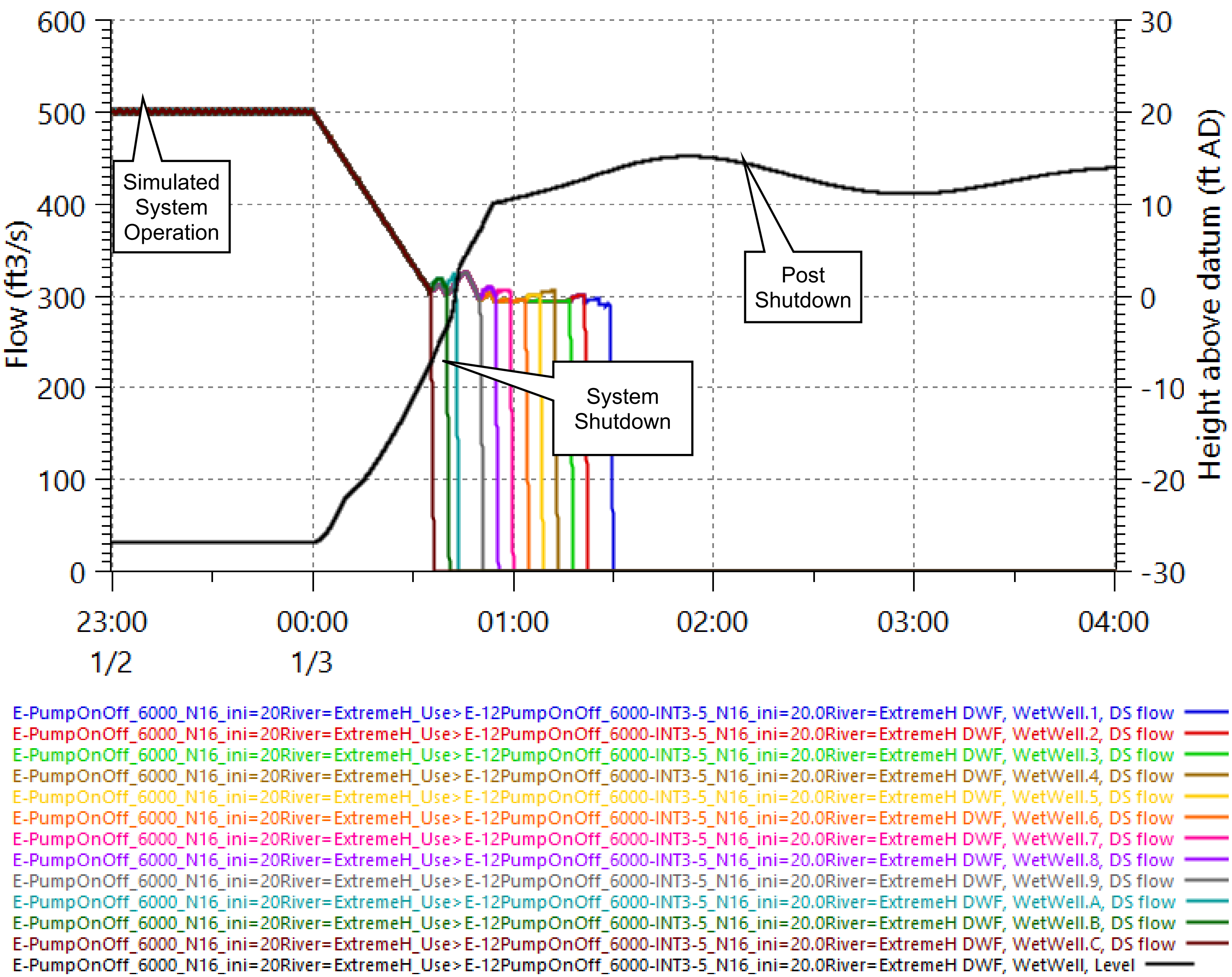

Figure 12. Pump Shut Down and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.016

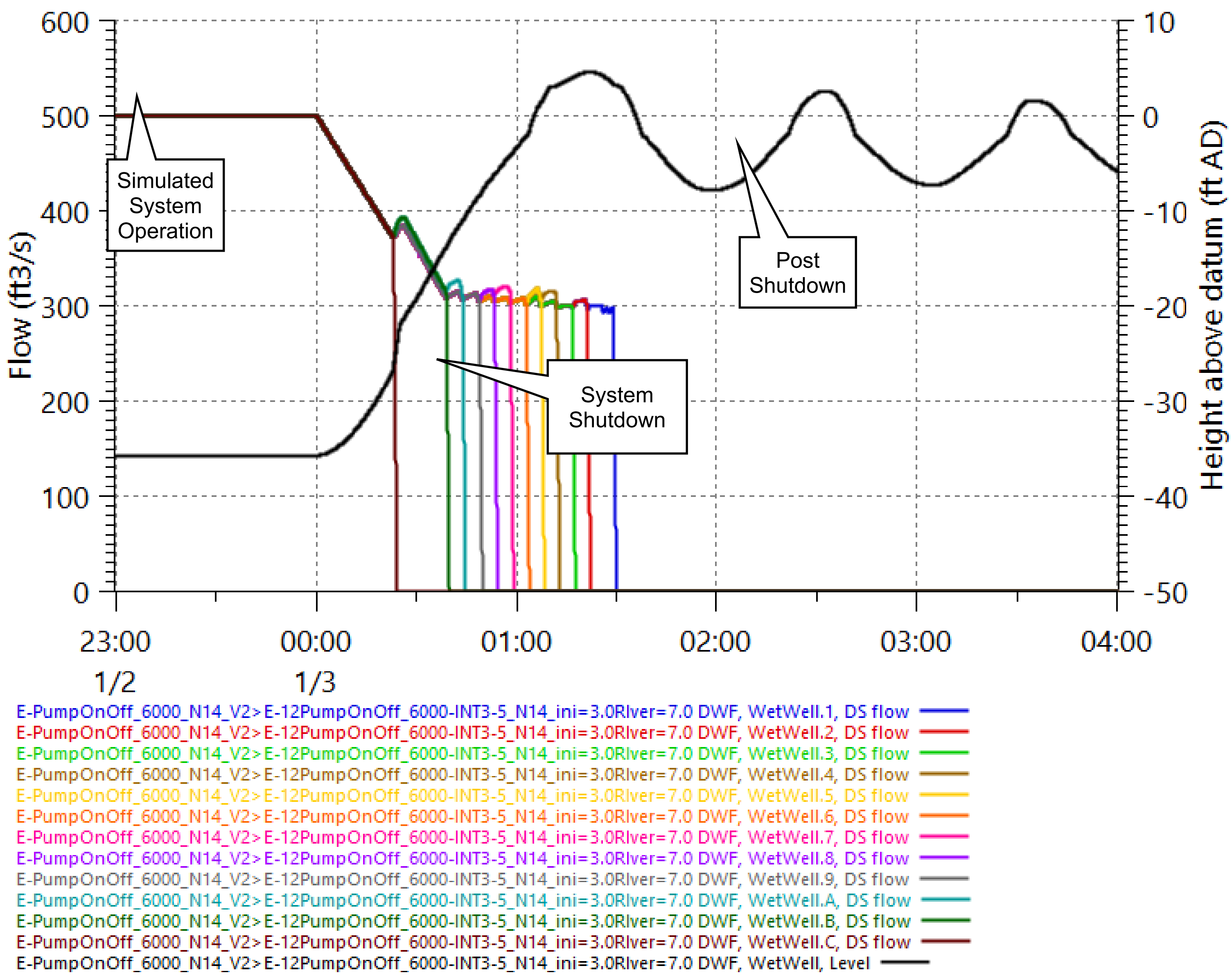

Figure 13. Pump Shut Down and Wet Well Water Level, 6000 cfs, Low River, Manning's n= 0.014

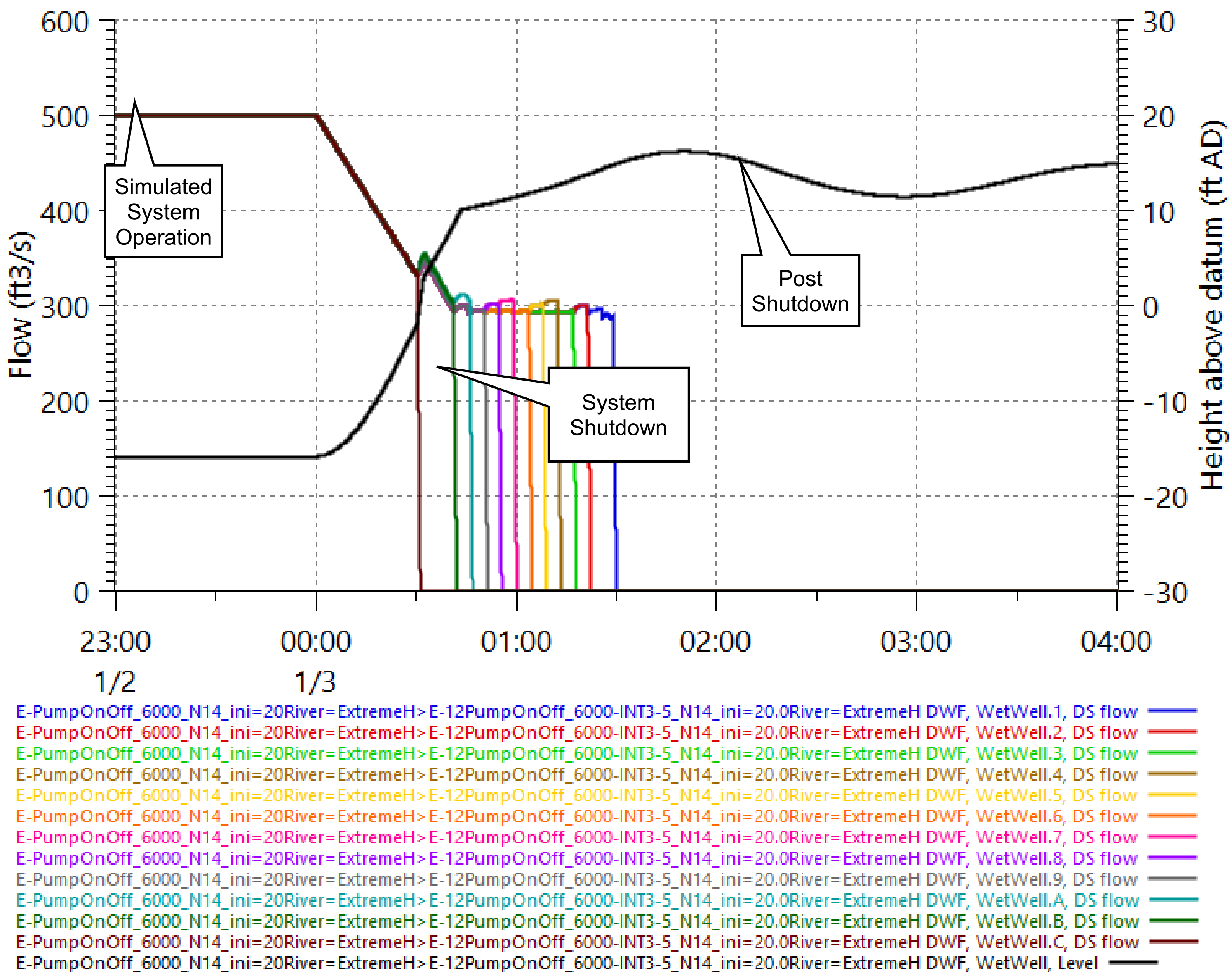

Figure 14. Pump Shut Down and Wet Well Water Level, 6000 cfs, High River, Manning's n= 0.014

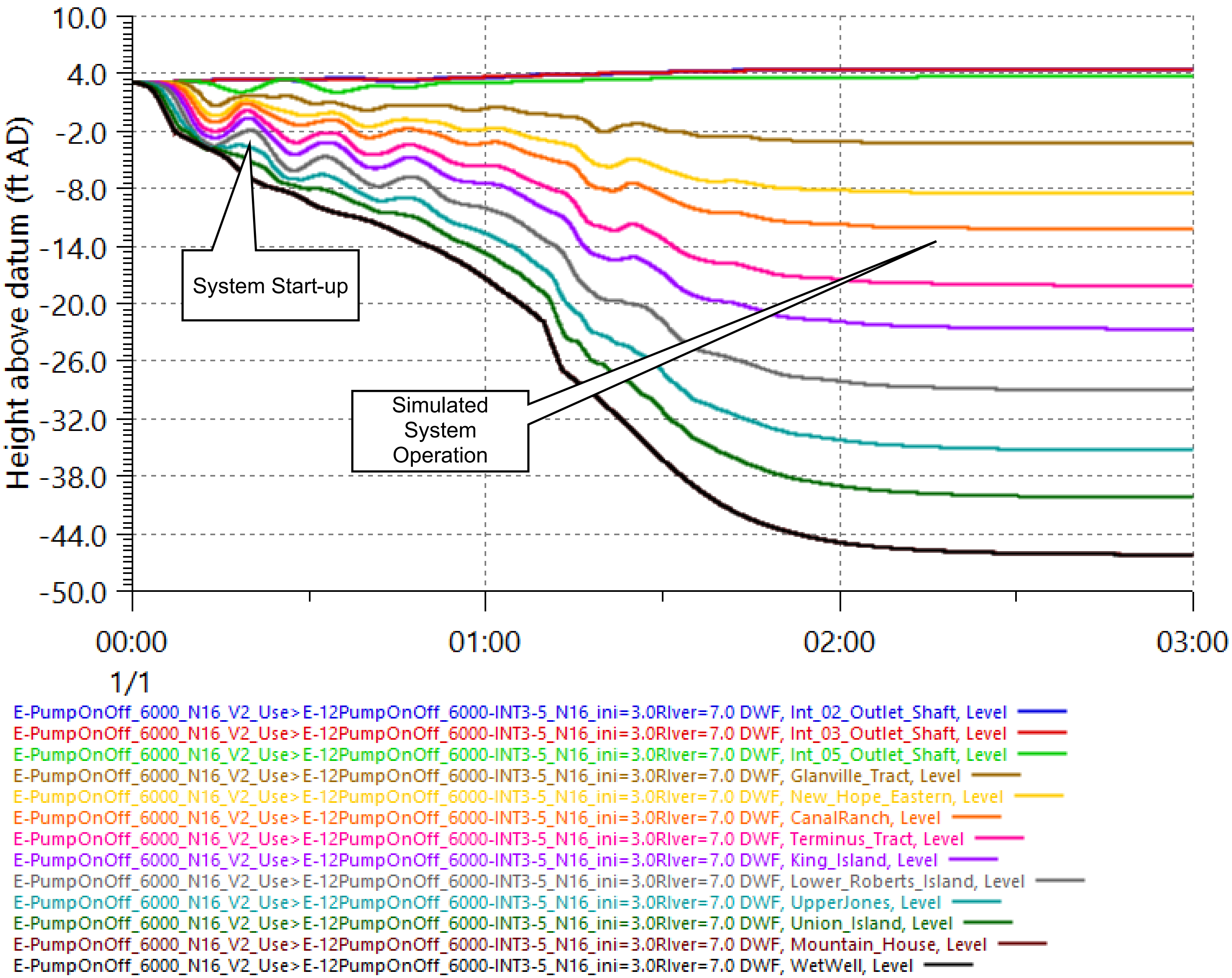

Figure 15. Tunnel Shaft Water Levels during System Startup, 6000 cfs, Low River, Manning's n= 0.016

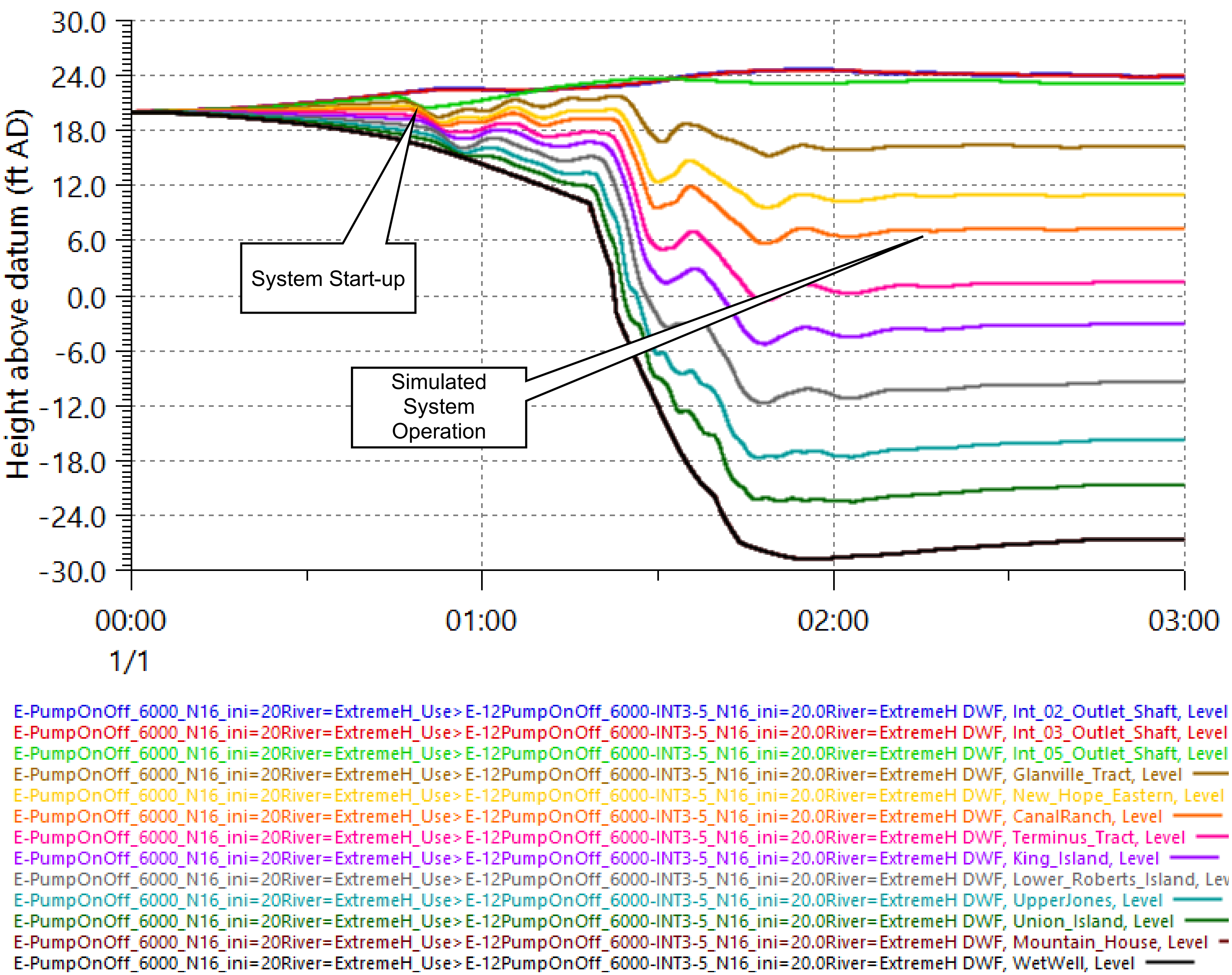

Figure 16. Tunnel Shaft Water Levels during System Startup, 6000 cfs, High River, Manning's n= 0.016

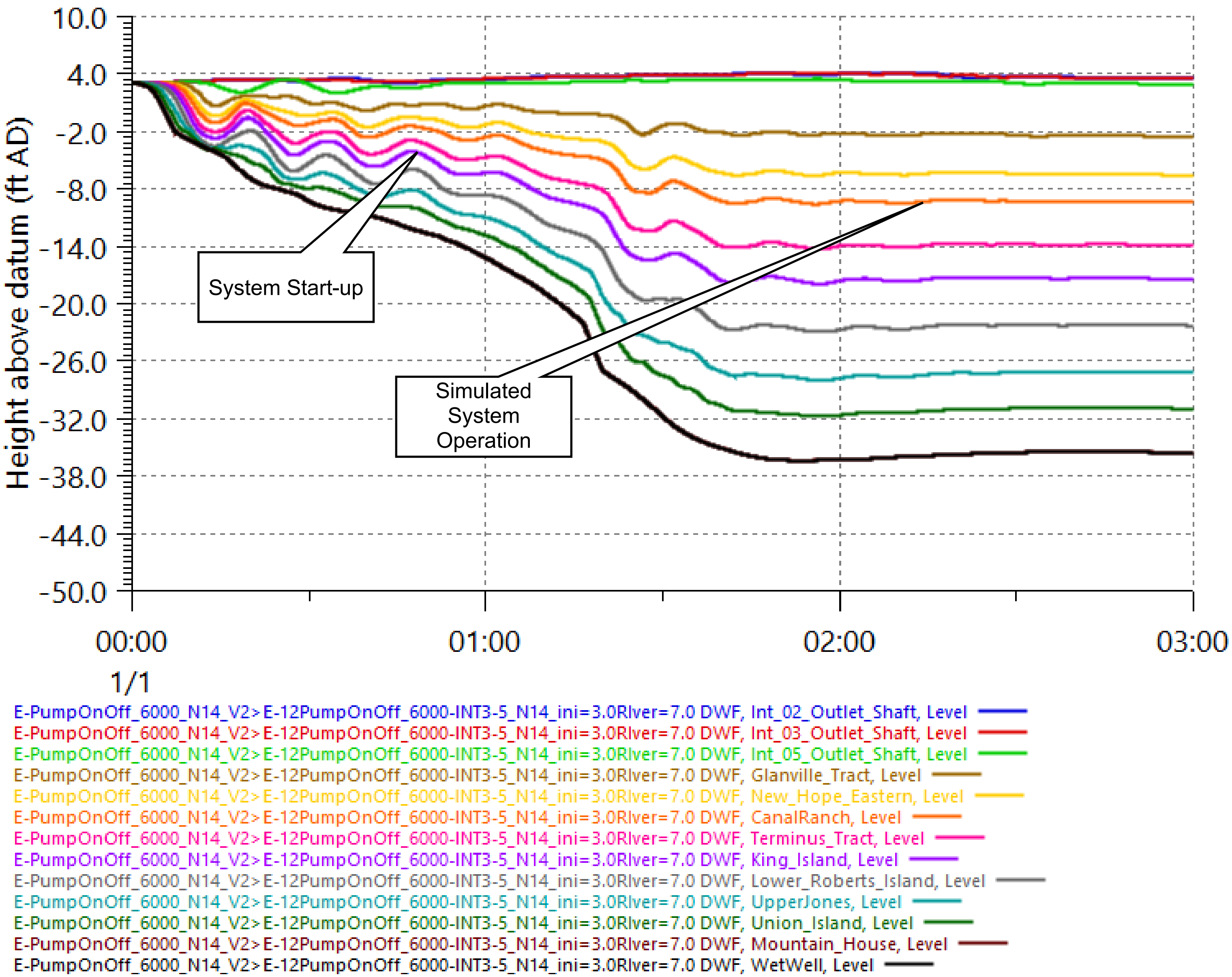

Figure 17. Tunnel Shaft Water Levels during System Startup, 6000 cfs, Low River, Manning's n= 0.014

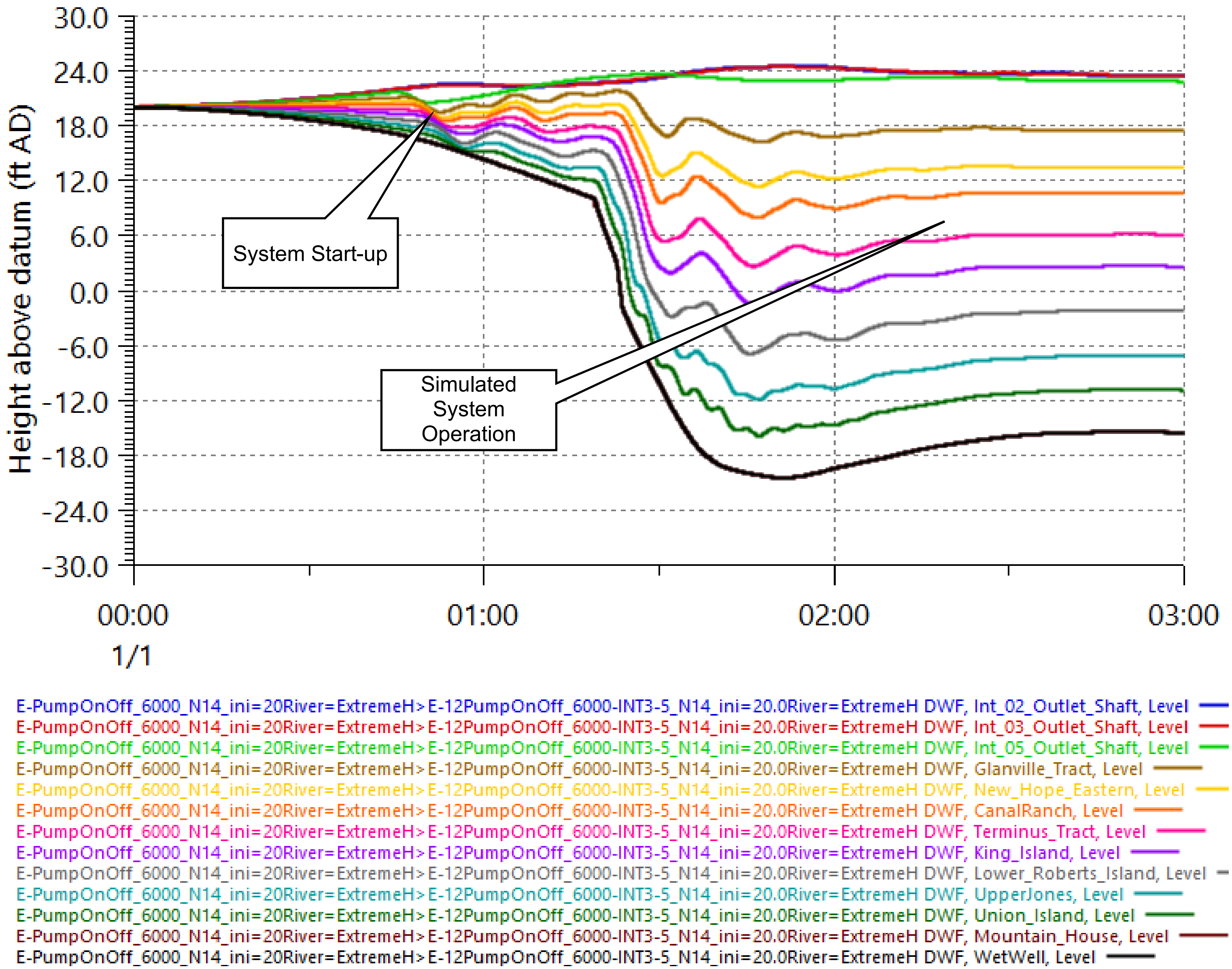

Figure 18. Tunnel Shaft Water Levels during System Startup, 6000 cfs, High River, Manning's n= 0.014

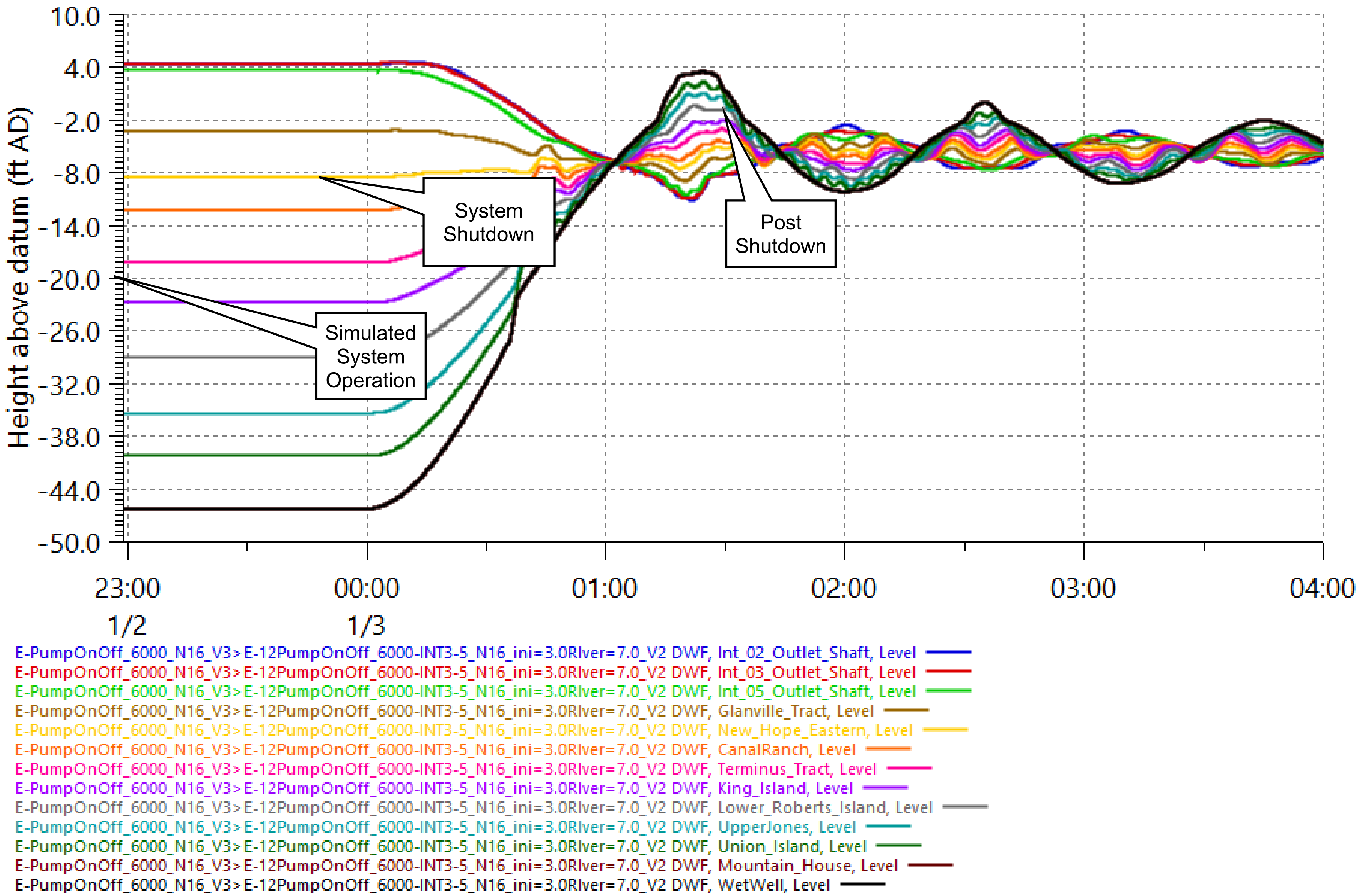

Figure 19. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, Low River, Manning's n= 0.016

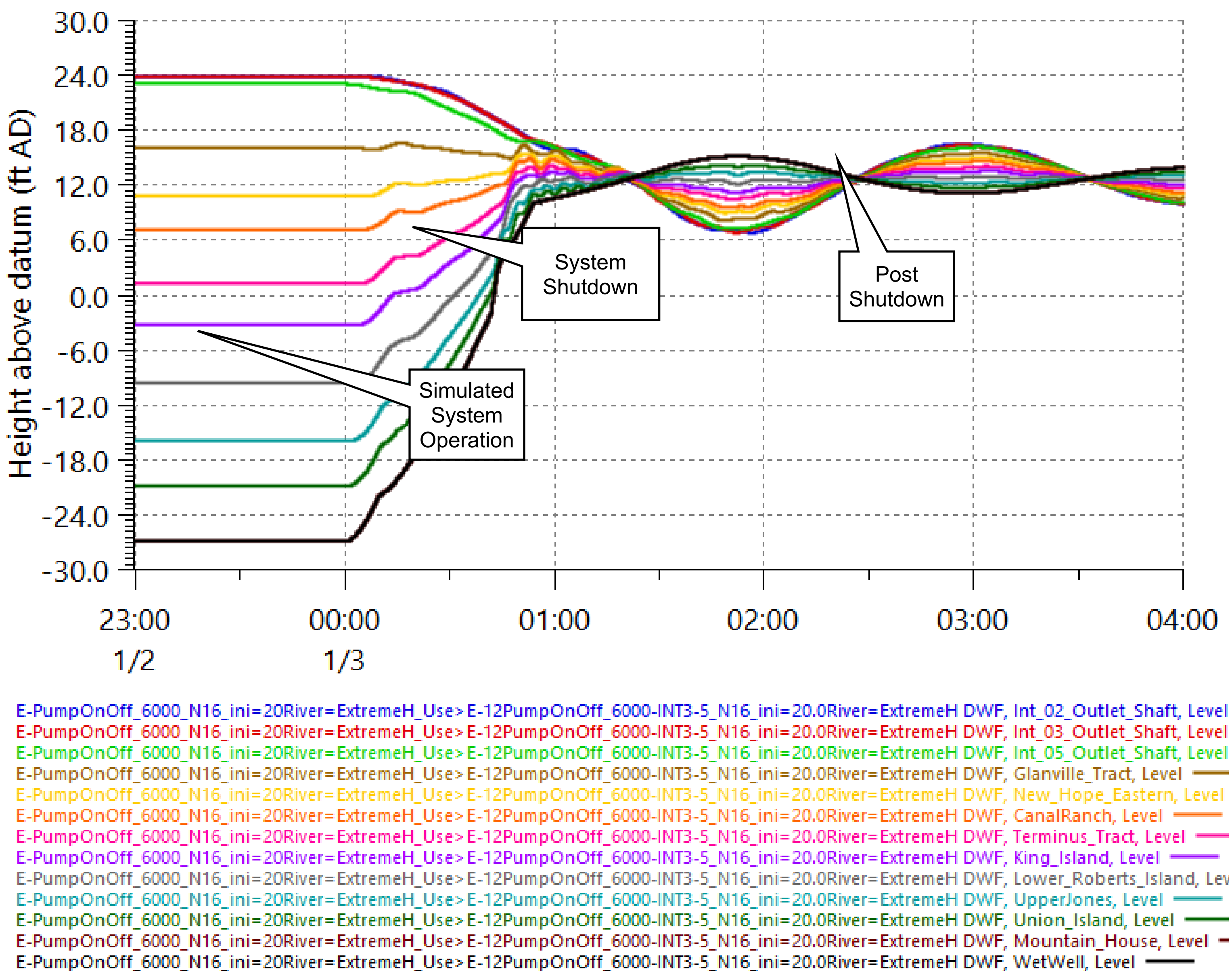

Figure 20. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, High River, Manning's n= 0.016

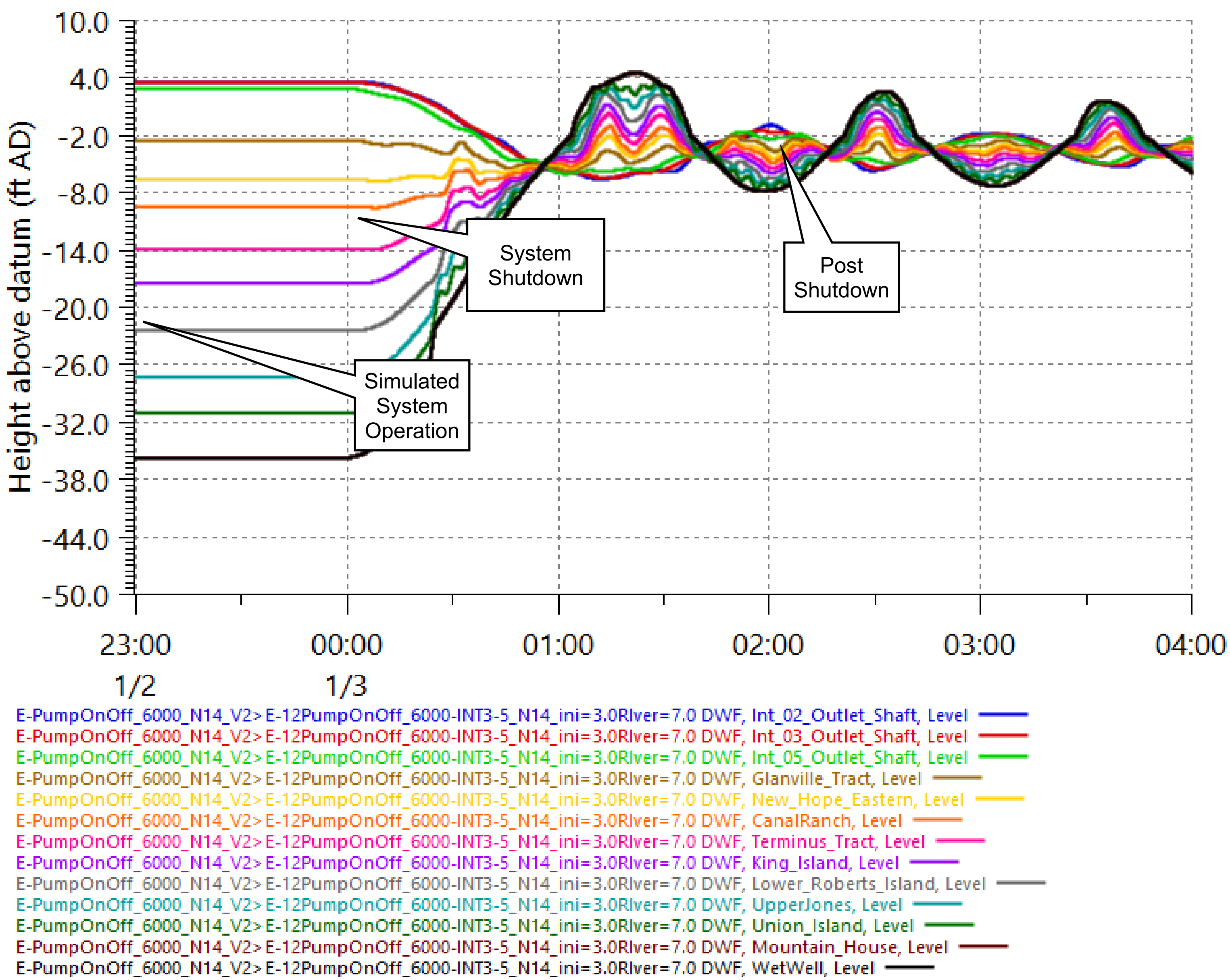

Figure 21. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, Low River, Manning's n= 0.014

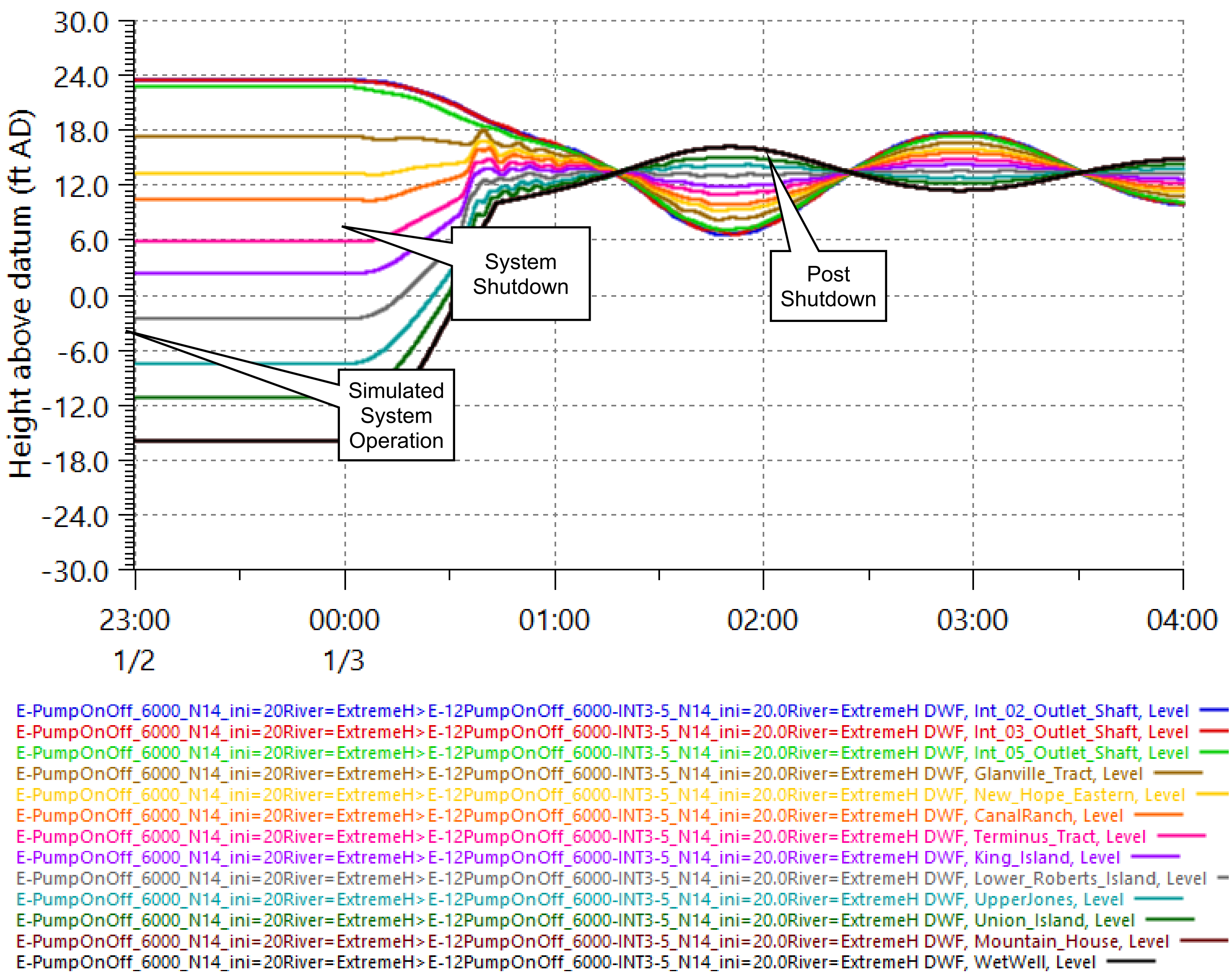

Figure 22. Tunnel Shaft Water Levels during System Shut Down, 6000 cfs, High River, Manning's n= 0.014

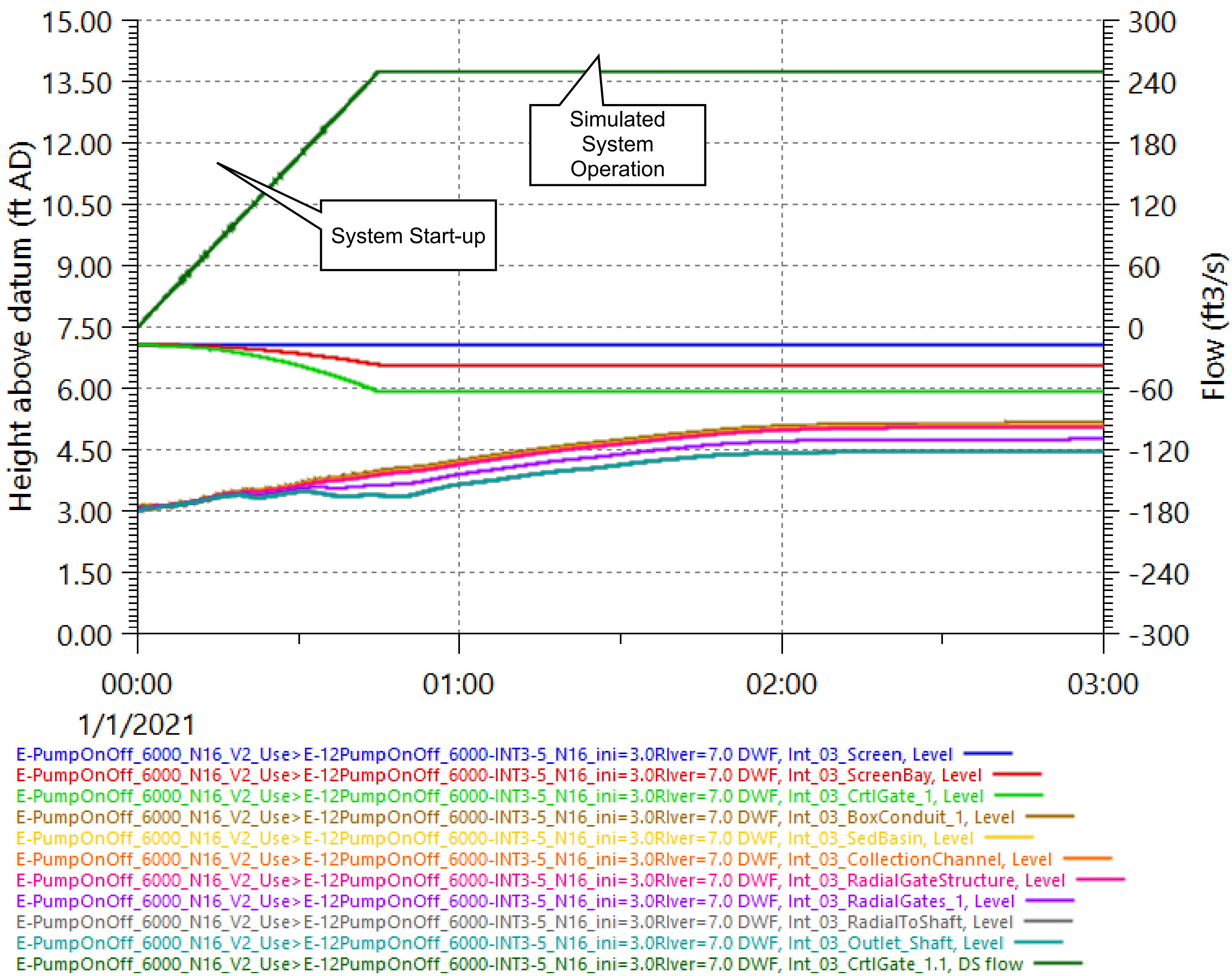

Figure 23. C-E-3 Flow and Water Levels during System Startup, Low River, Manning's n= 0.016

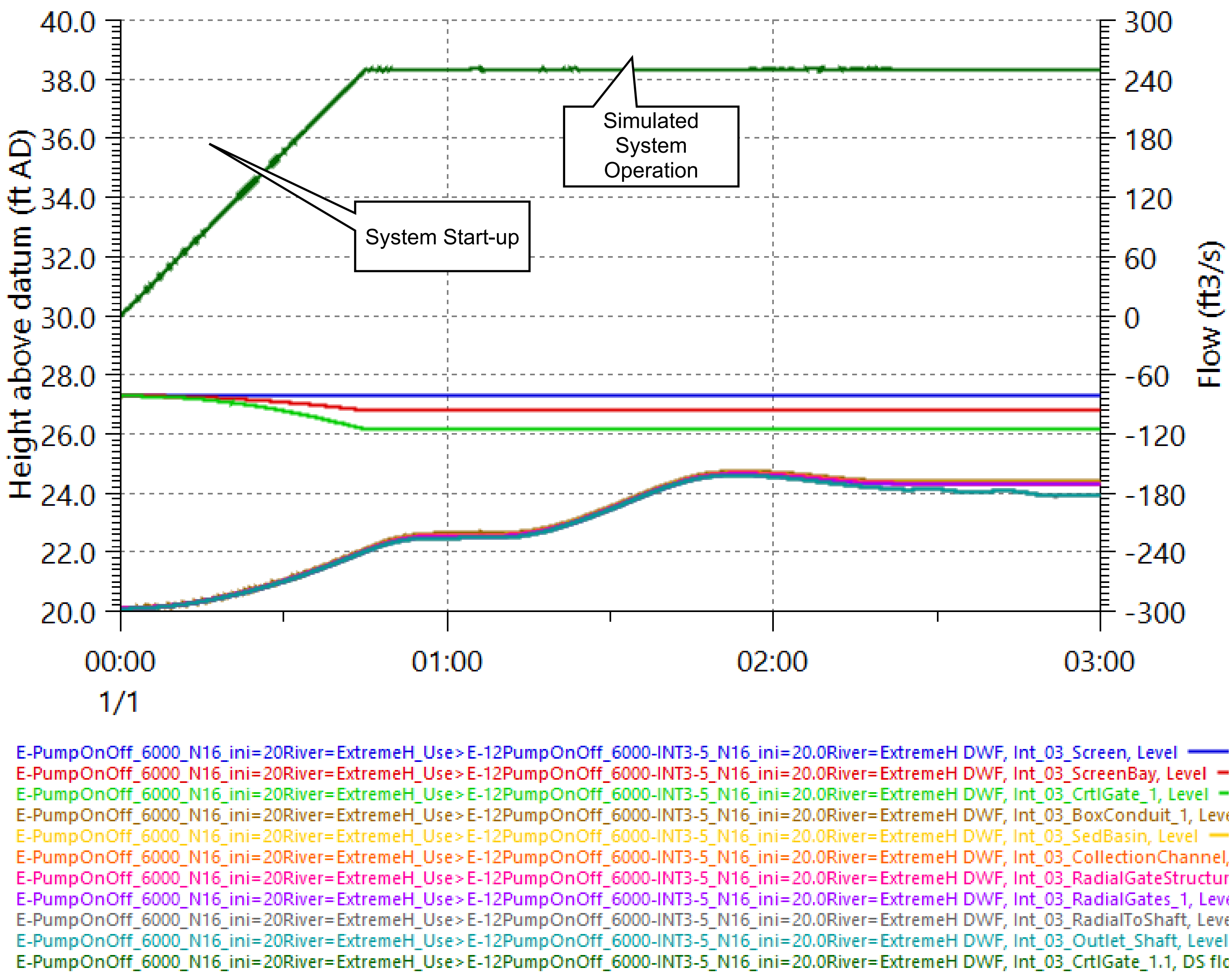

Figure 24. C-E-3 Flow and Water Levels during System Startup, High River, Manning's n= 0.016

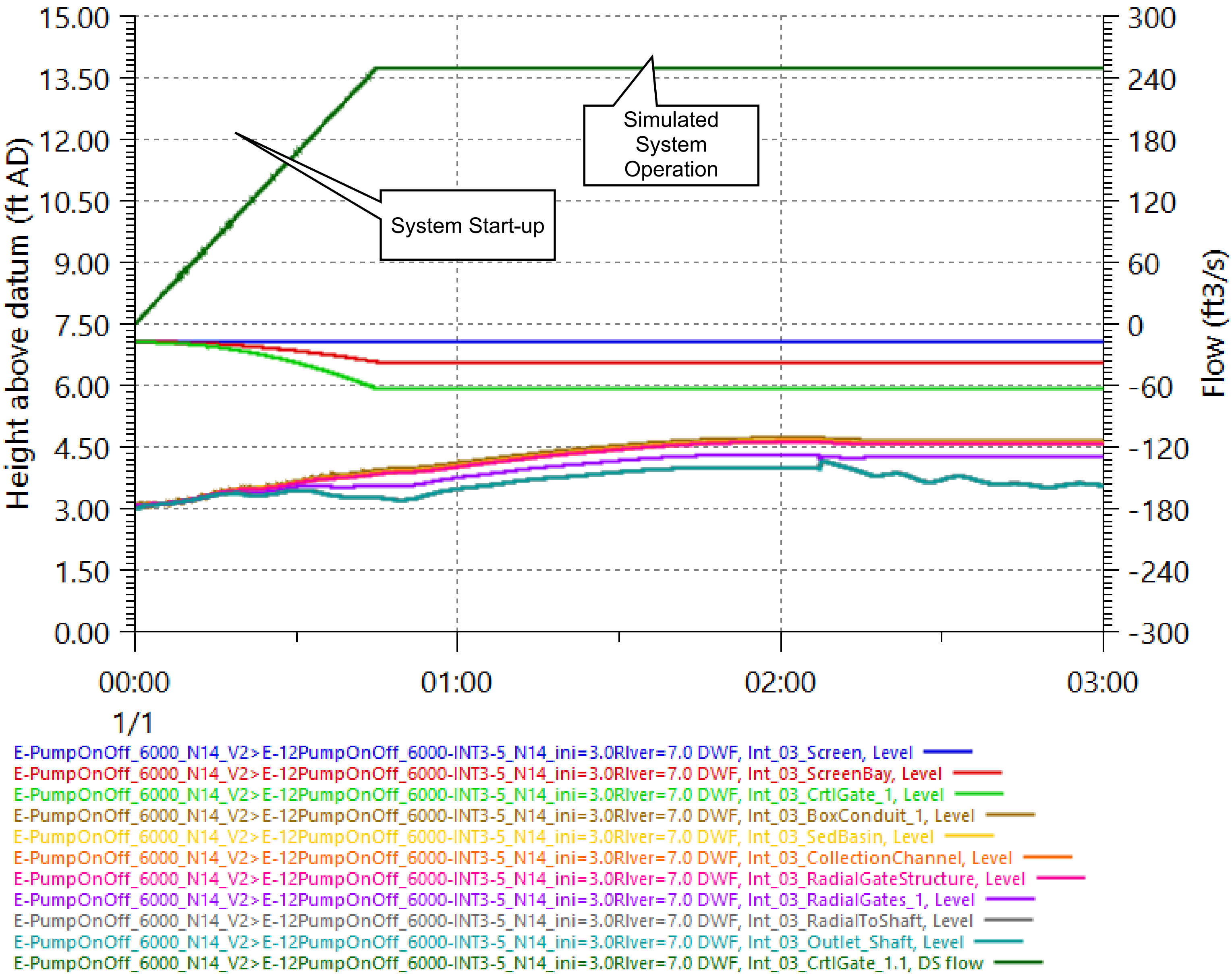

Figure 25. C-E-3 Flow and Water Levels during System Startup, Low River, Manning's n= 0.014

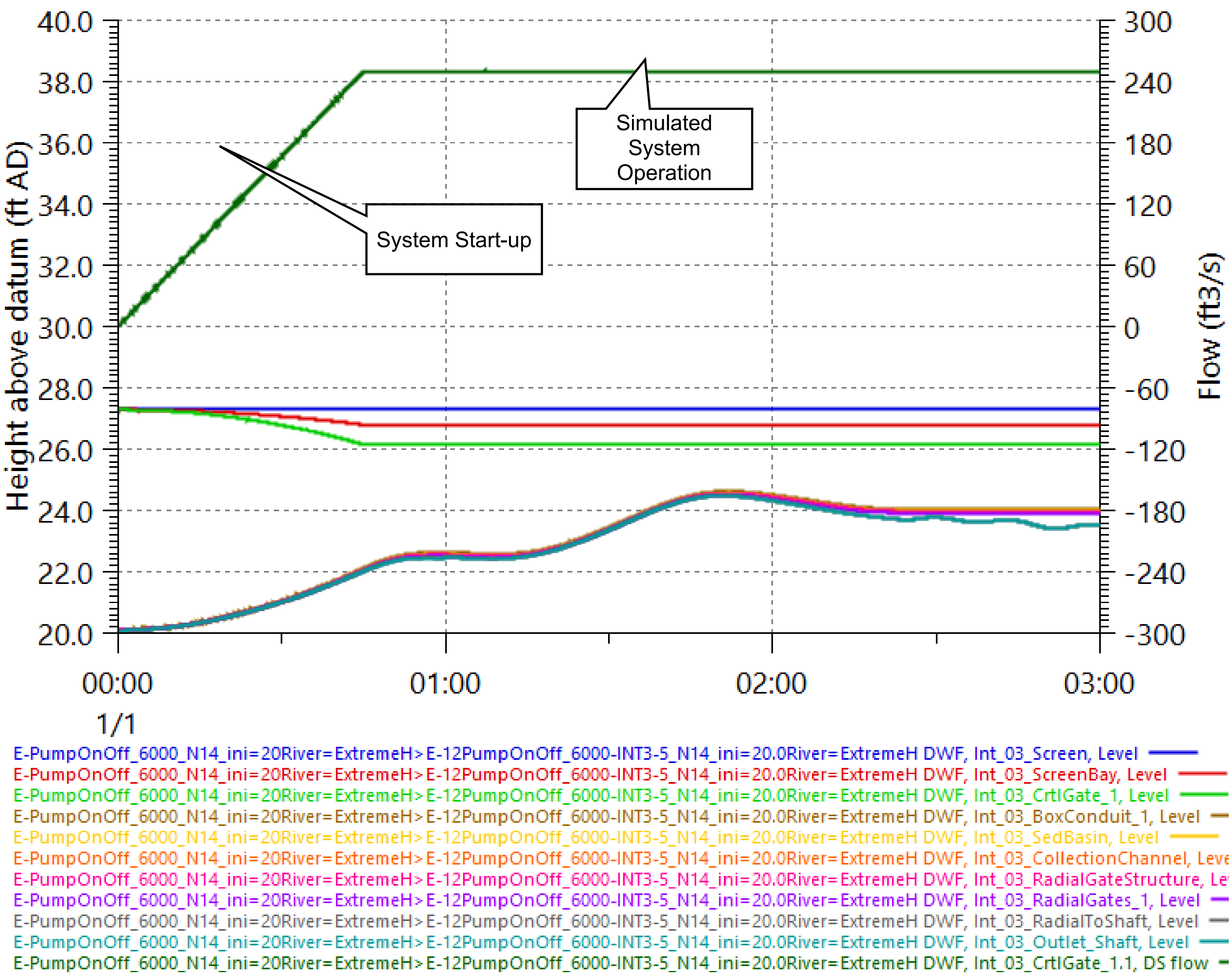

Figure 26. C-E-3 Flow and Water Levels during System Startup, High River, Manning's n= 0.014

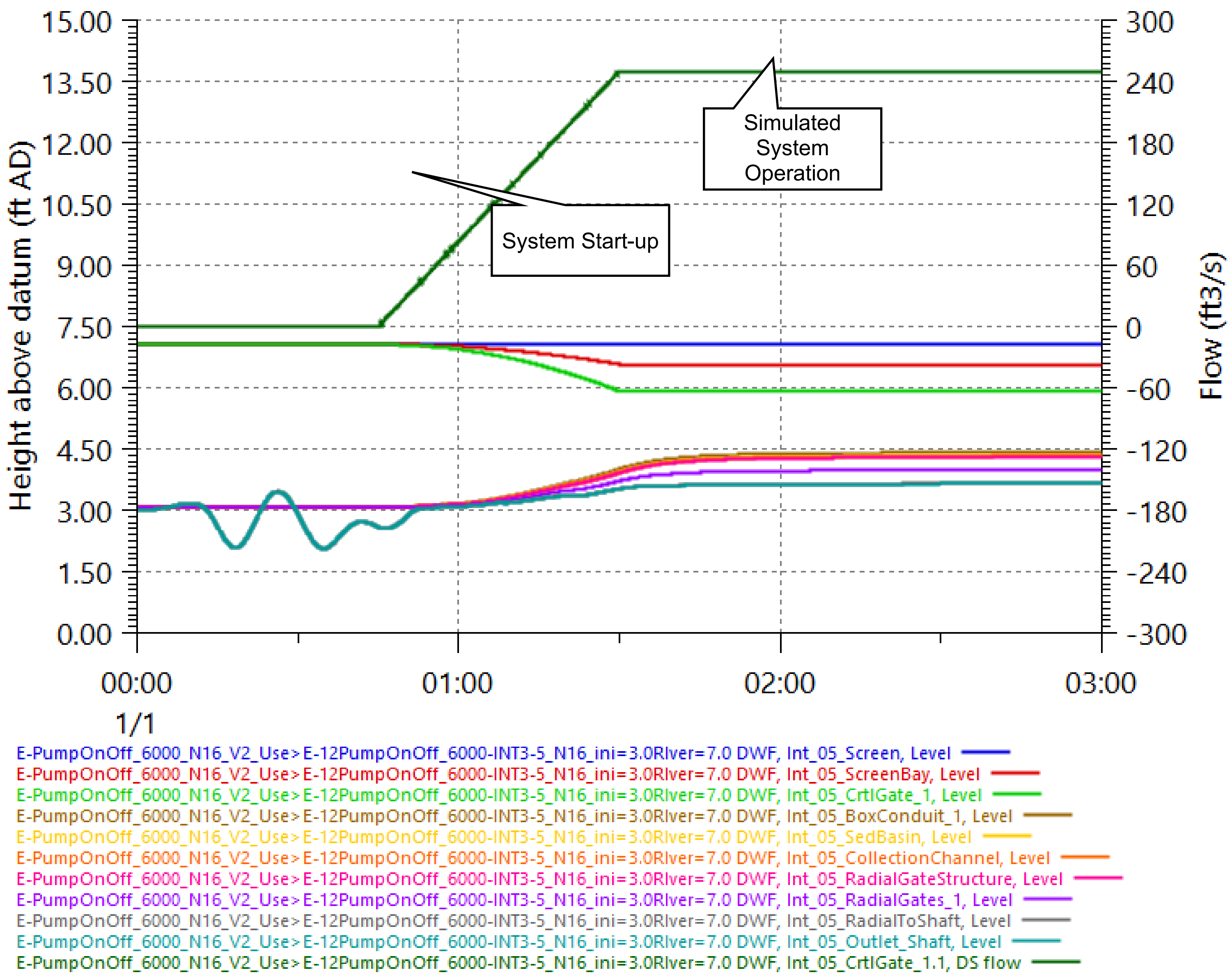

Figure 27. C-E-5 Flow and Water Levels during System Startup, Low River, Manning's n= 0.016

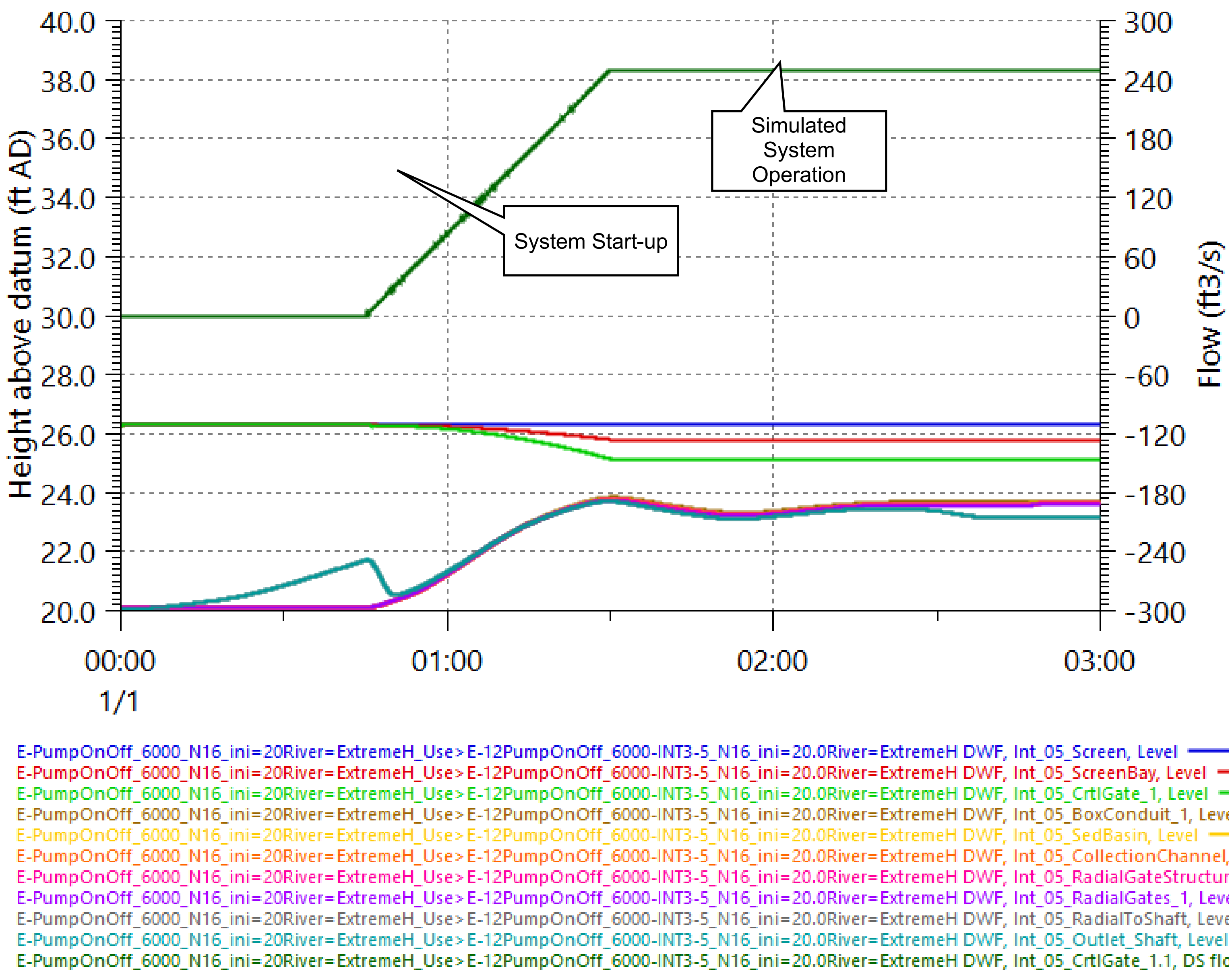

Figure 28. C-E-5 Flow and Water Levels during System Startup, High River, Manning's n= 0.016

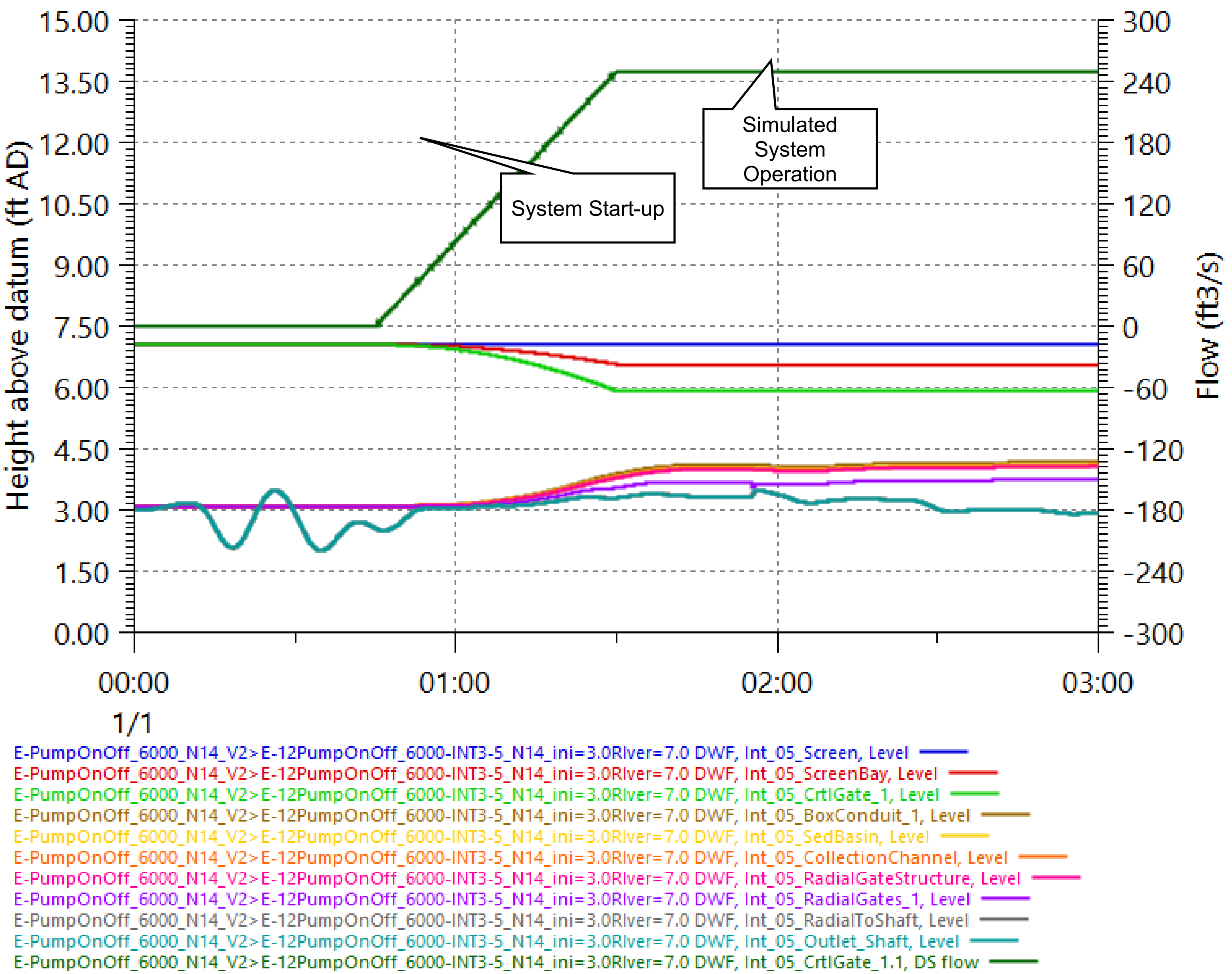

Figure 29. C-E-5 Flow and Water Levels during System Startup, Low River, Manning's n= 0.014

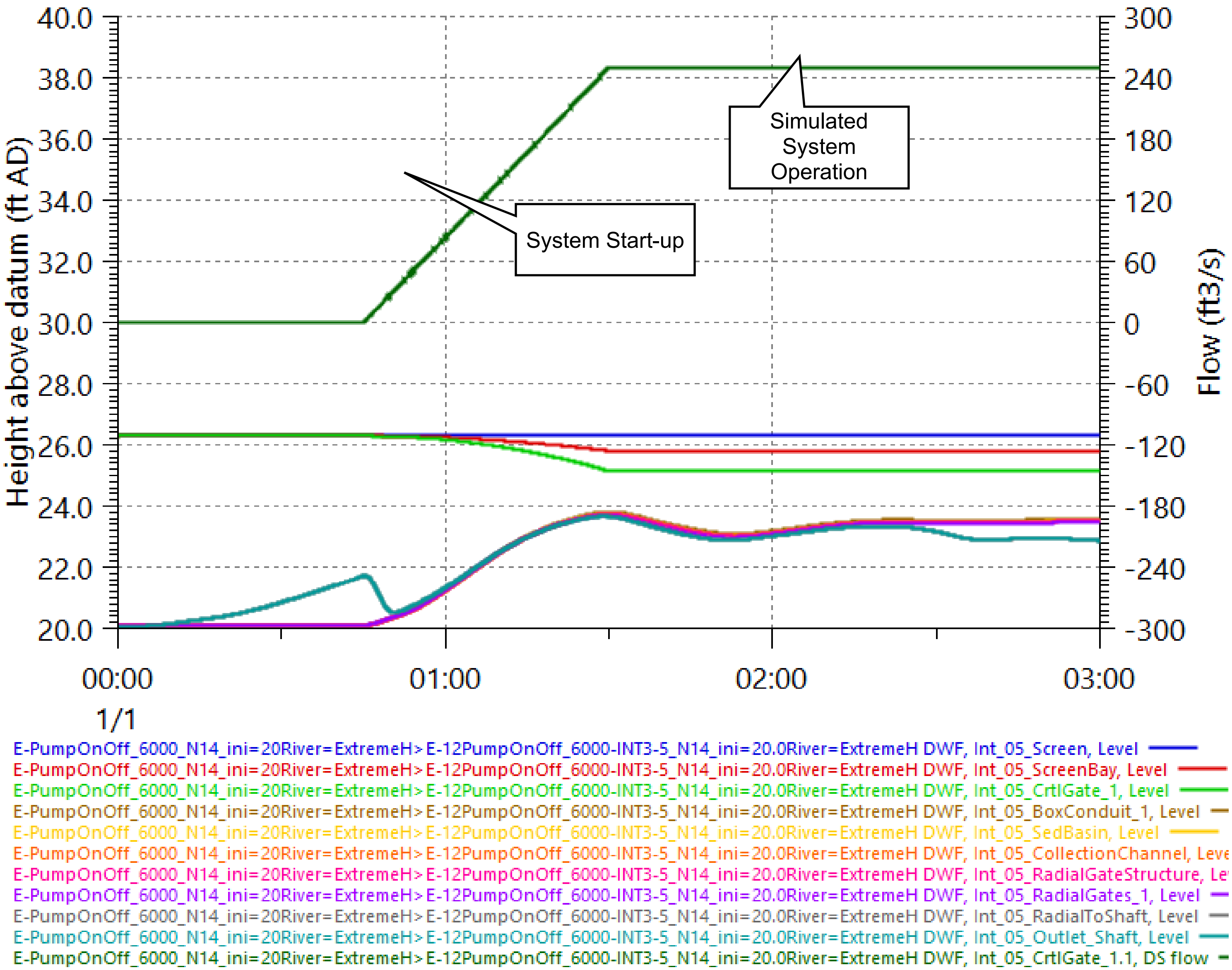

Figure 30. C-E-5 Flow and Water Levels during System Startup, High River, Manning's n= 0.014

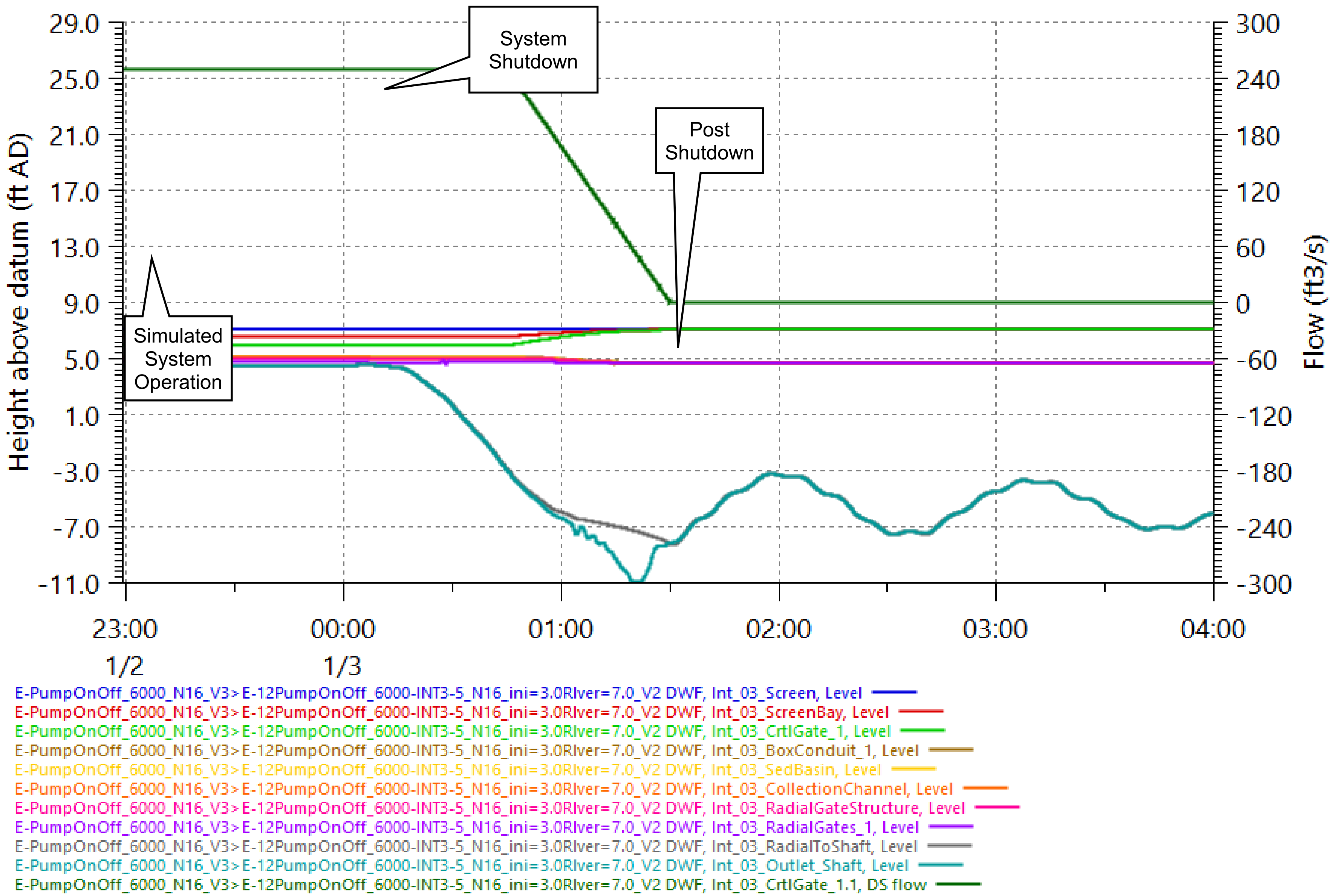

Figure 31. C-E-3 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.016

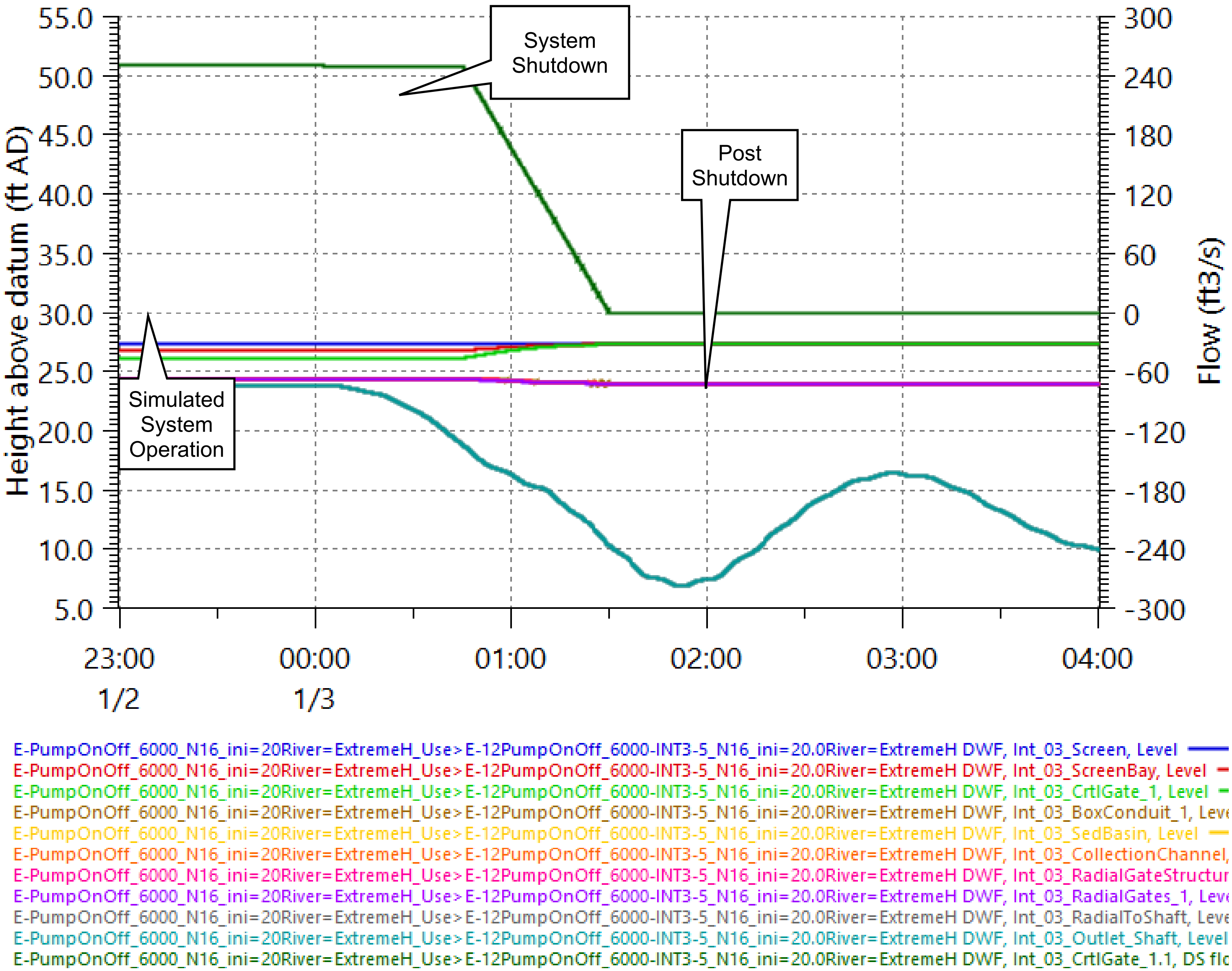

Figure 32. C-E-3 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.016

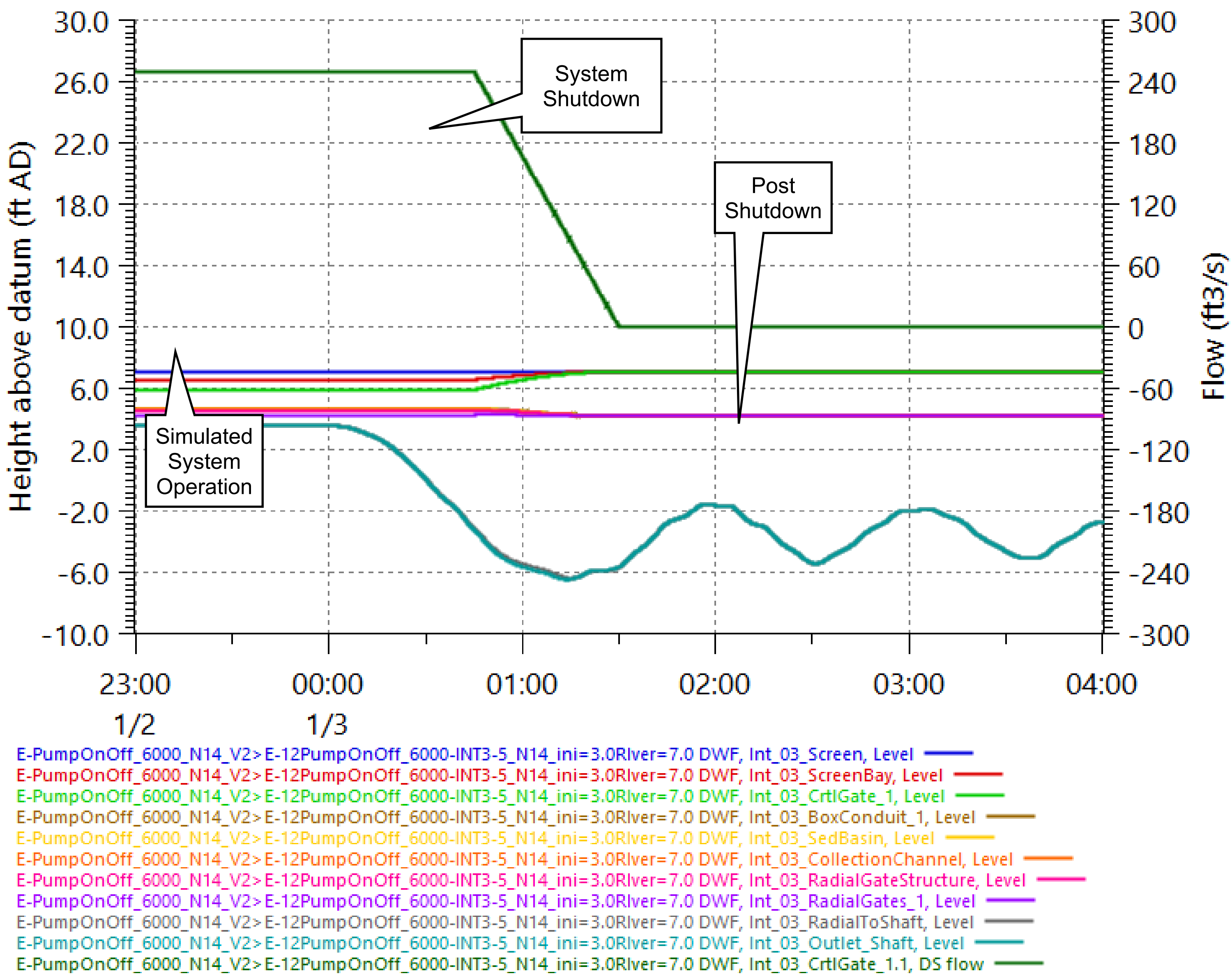

Figure 33. C-E-3 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.014

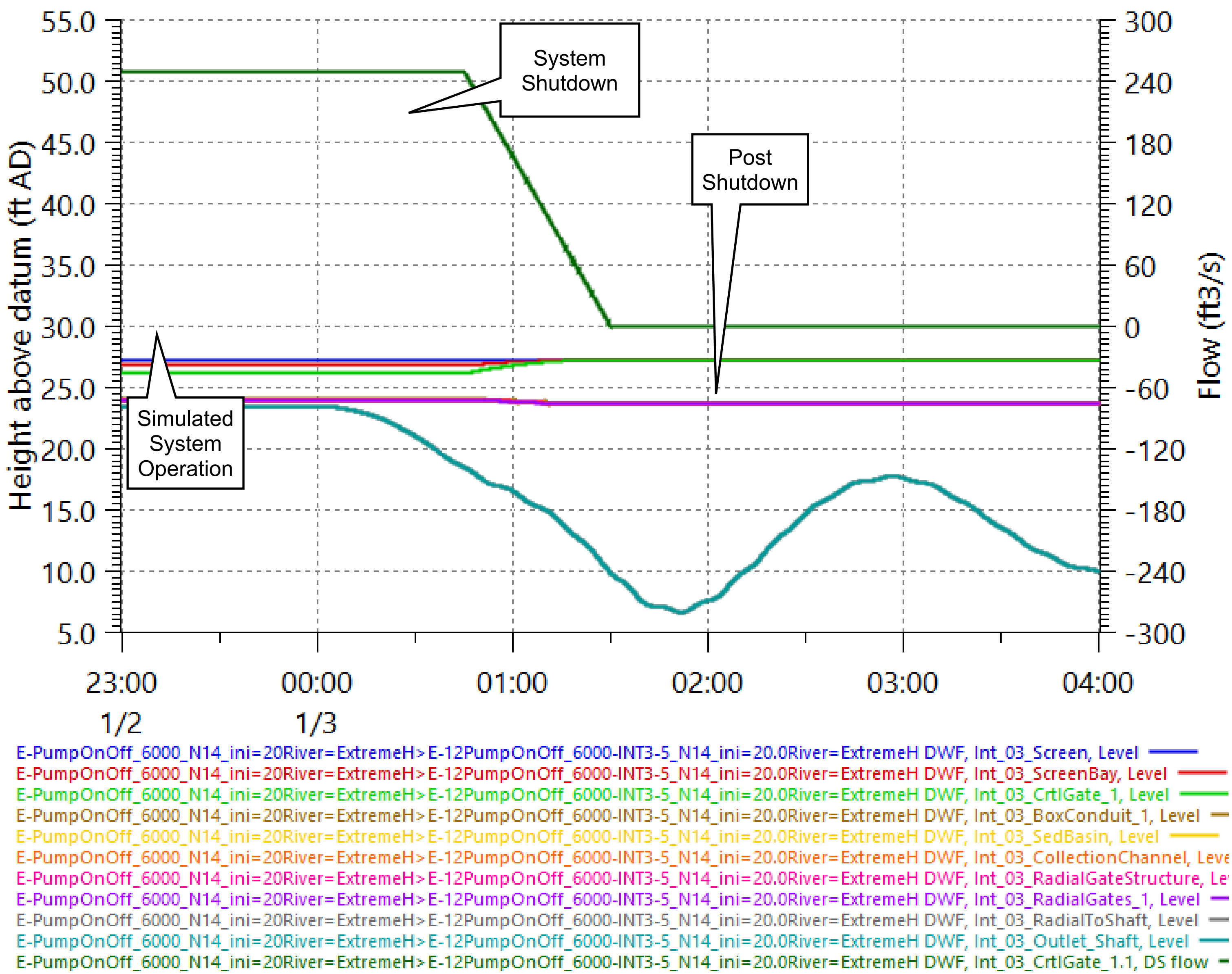

Figure 34. C-E-3 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.014

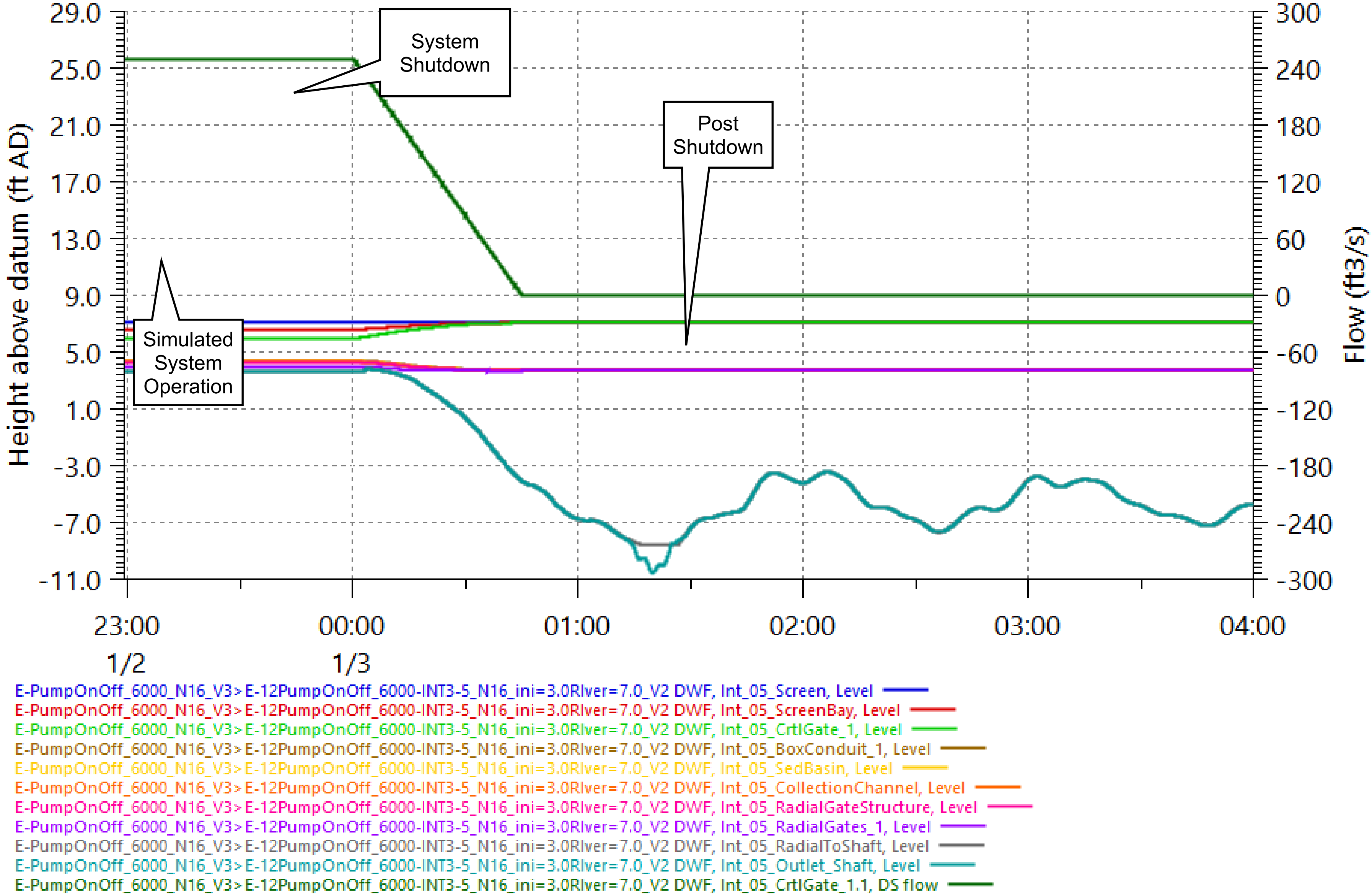

Figure 35. C-E-5 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.016

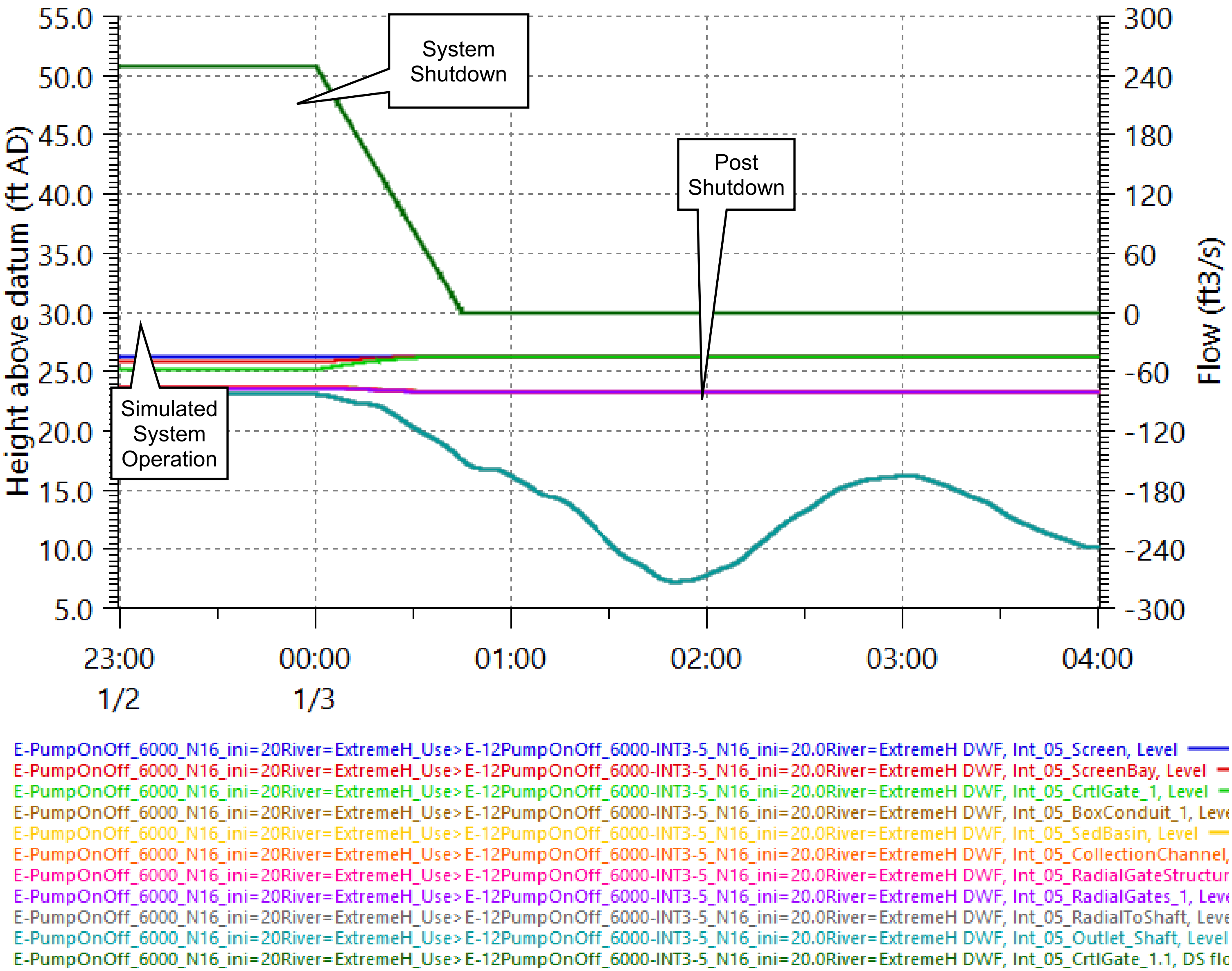

Figure 36. C-E-5 Flow and Water Levels during System Startup, High River, Manning's n= 0.016

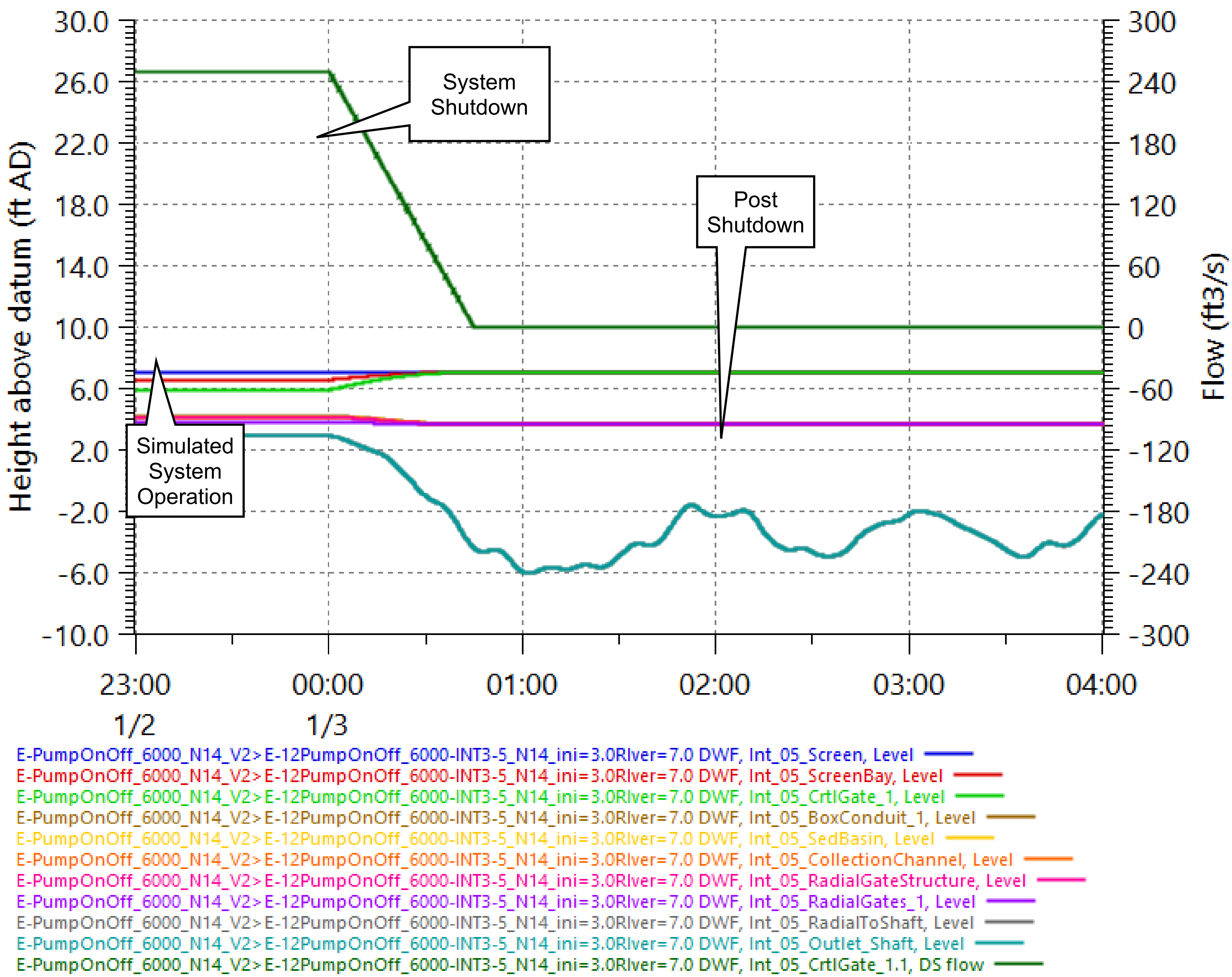

Figure 37. C-E-5 Flow and Water Levels during System Shut Down, Low River, Manning's n= 0.014

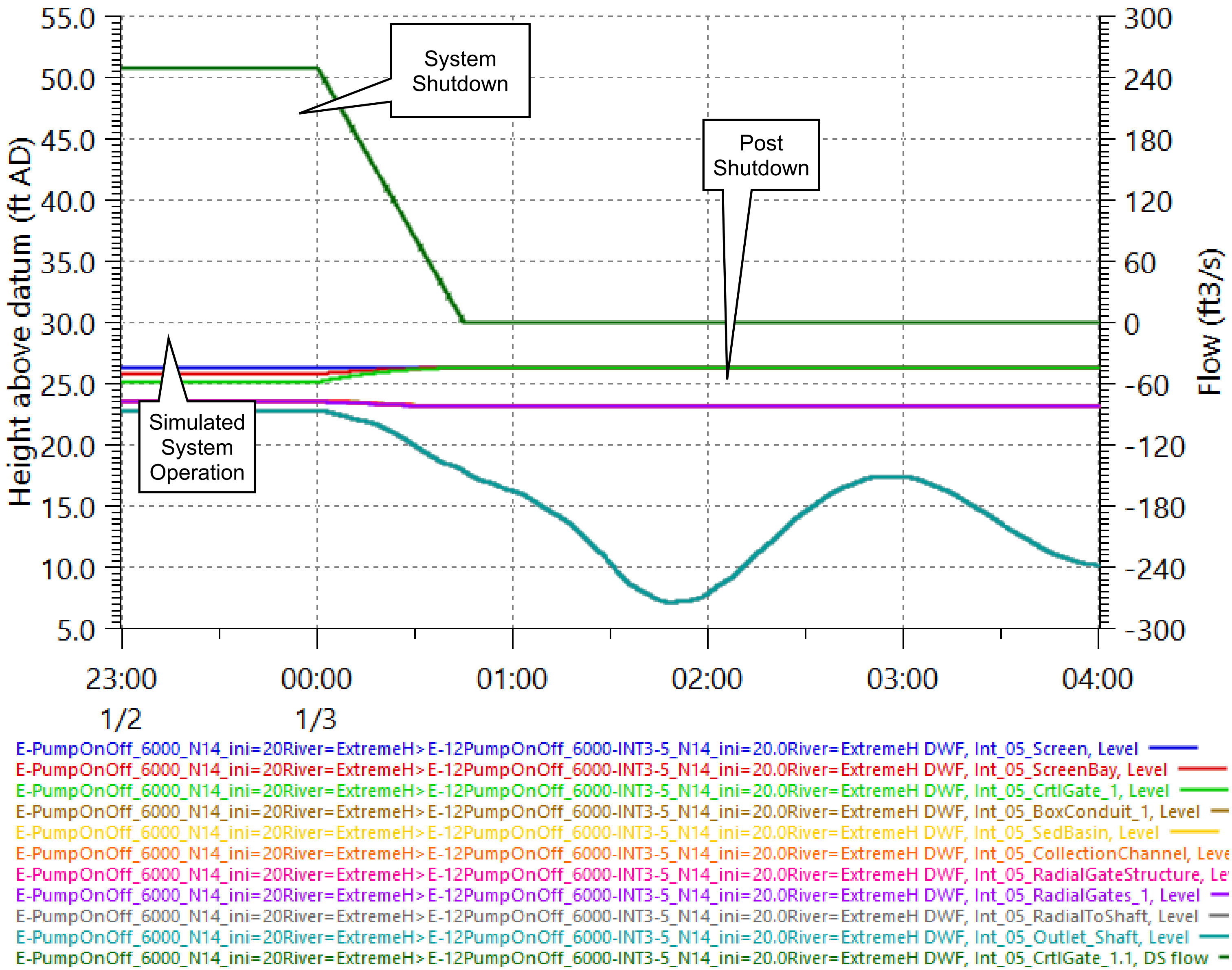

Figure 38. C-E-5 Flow and Water Levels during System Shut Down, High River, Manning's n= 0.014

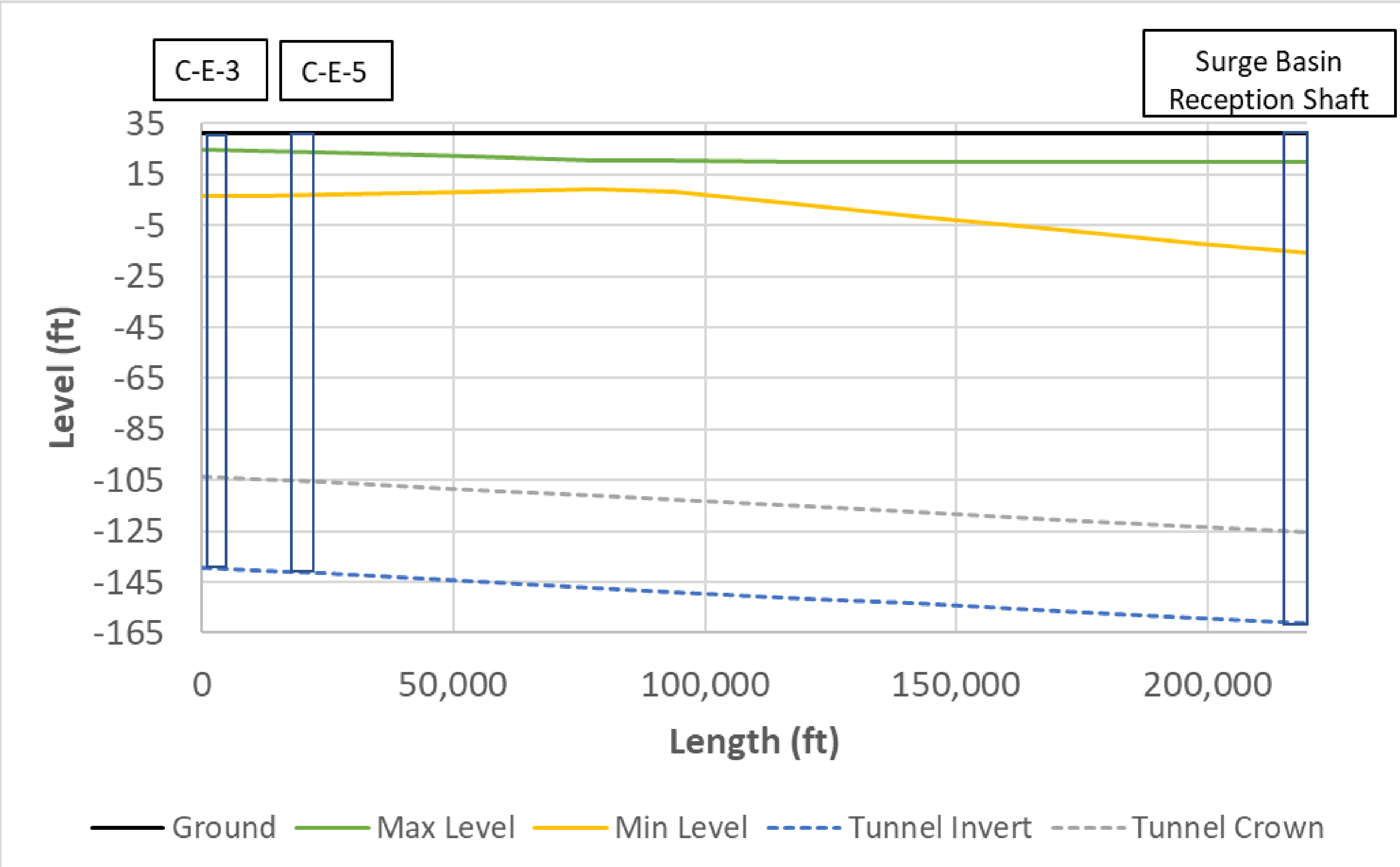

Figure 39. Minimum and Maximum WSEL Profile Startup and Shutdown Scenario, Bethany Reservoir Alignment, 6000 cfs, Manning's *n* 0.014, High River level

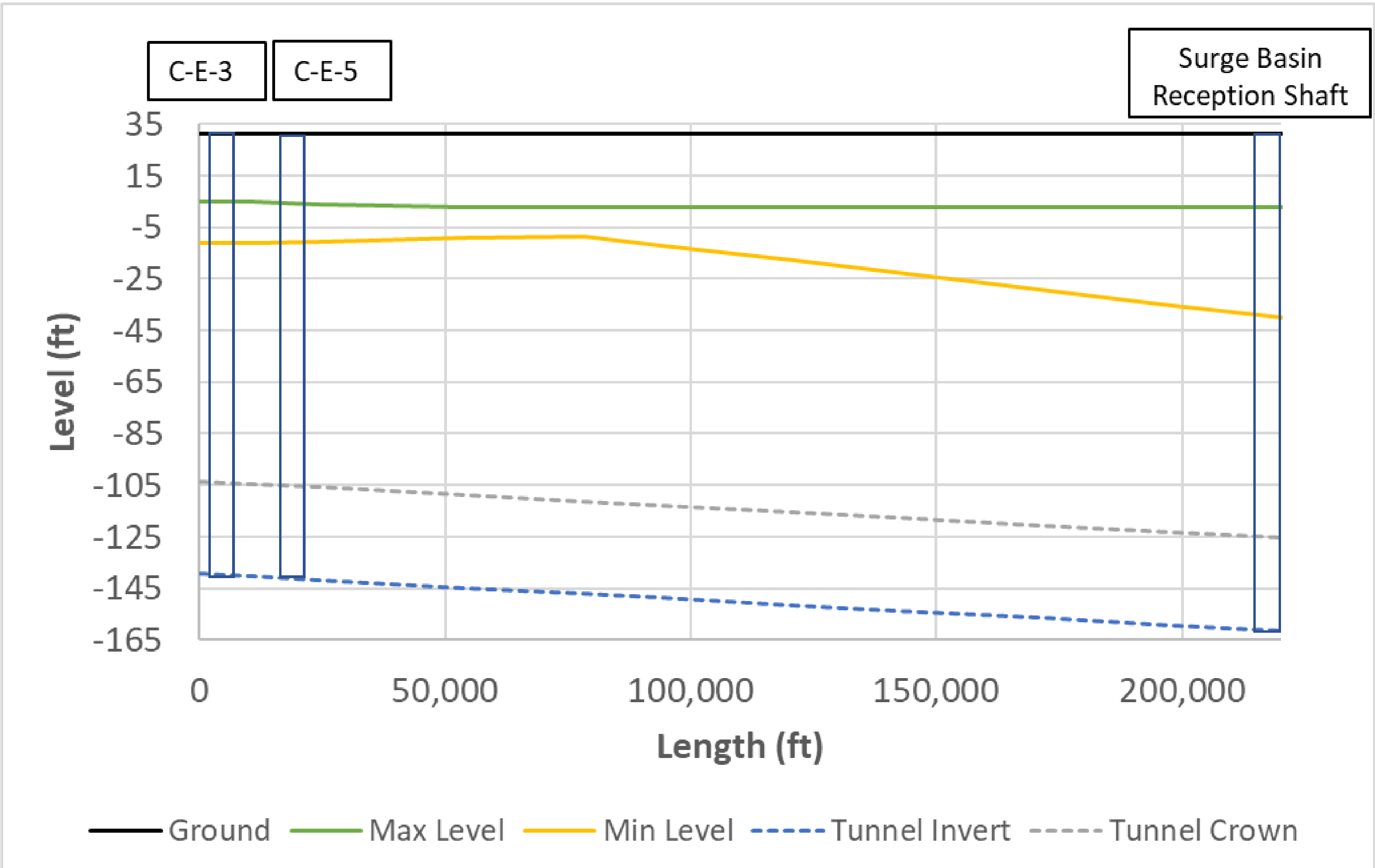

Figure 40. Minimum and Maximum WSEL Profile Startup and Shutdown Scenario, Bethany Reservoir Alignment, 6000 cfs, Manning's *n* 0.016, Low River

3.3.3.3 Summary

The following summarizes the startup and shutdown operations for the Bethany Reservoir Alignment with the 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.

4. Analysis and Evaluation

4.1 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 capacity of 6,000 cfs.

For the steady-state head loss analysis of the Project design capacity of 6,000 cfs, 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 Island Double Launch Shaft was modeled as a single shaft with a finished inside diameter of 115 feet. It is recognized that 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 1.

Table 1. Steady-State Hydraulic Headloss Analysis – Project Flow Capacities Evaluated

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

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.

Figure 41 plots the tunnel head loss results that were developed for the 6,000 cfs Project flow condition. Table 2 summarizes the corresponding head loss to the BRPP wet well WSEL for each assigned Manning's *n*.

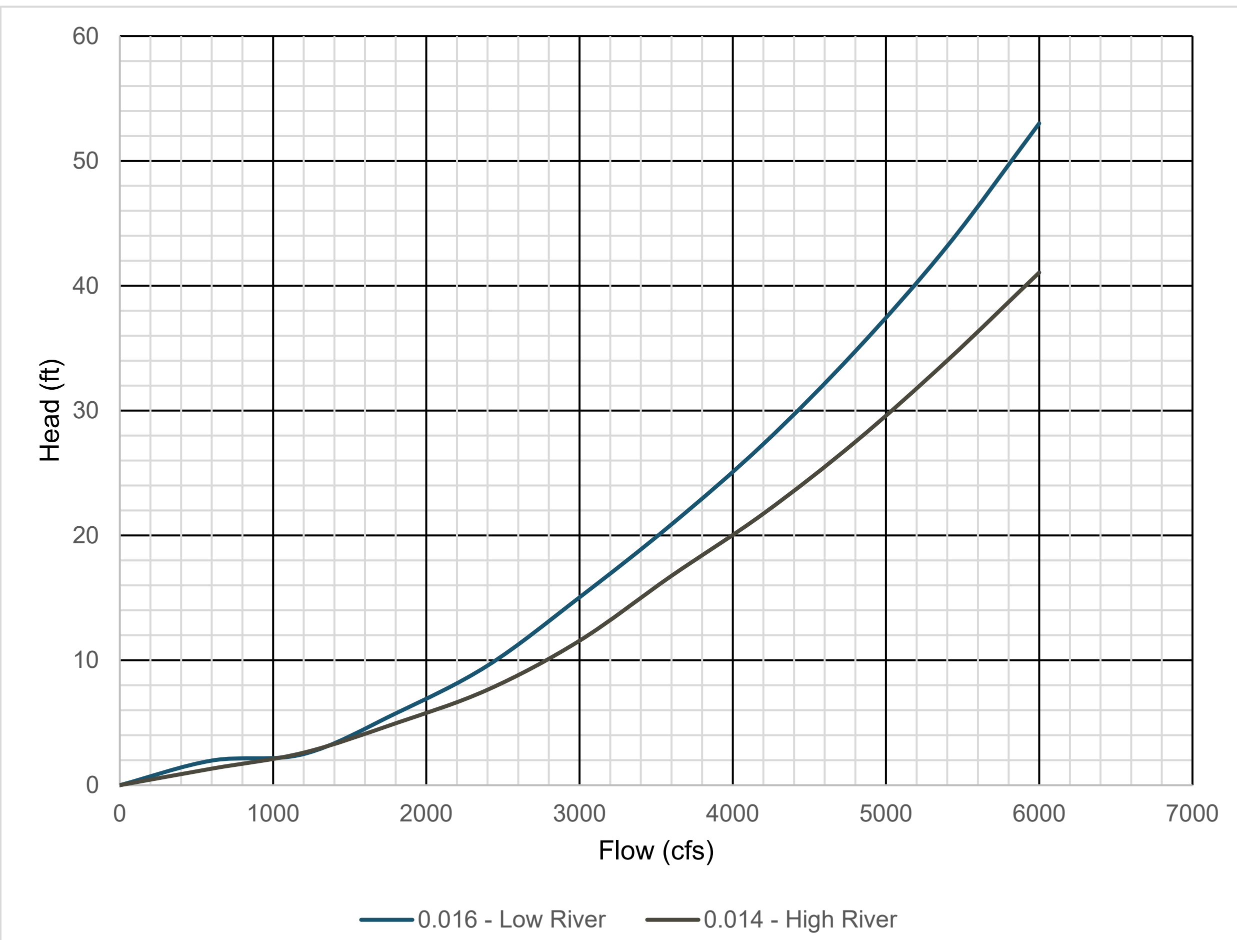

Figure 41. System Head Curves – 36-foot-inside-diameter Tunnel with a Project Design Capacity of 6,000 cfs

Table 2. 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 41 and Table 2 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.

4.1.1 Steady-state Hydraulic Grade Line Development

A steady-state HGL was developed for the tunnel for the Project design capacity of 6,000 cfs. The HGLs shown on Figure 42 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 43.

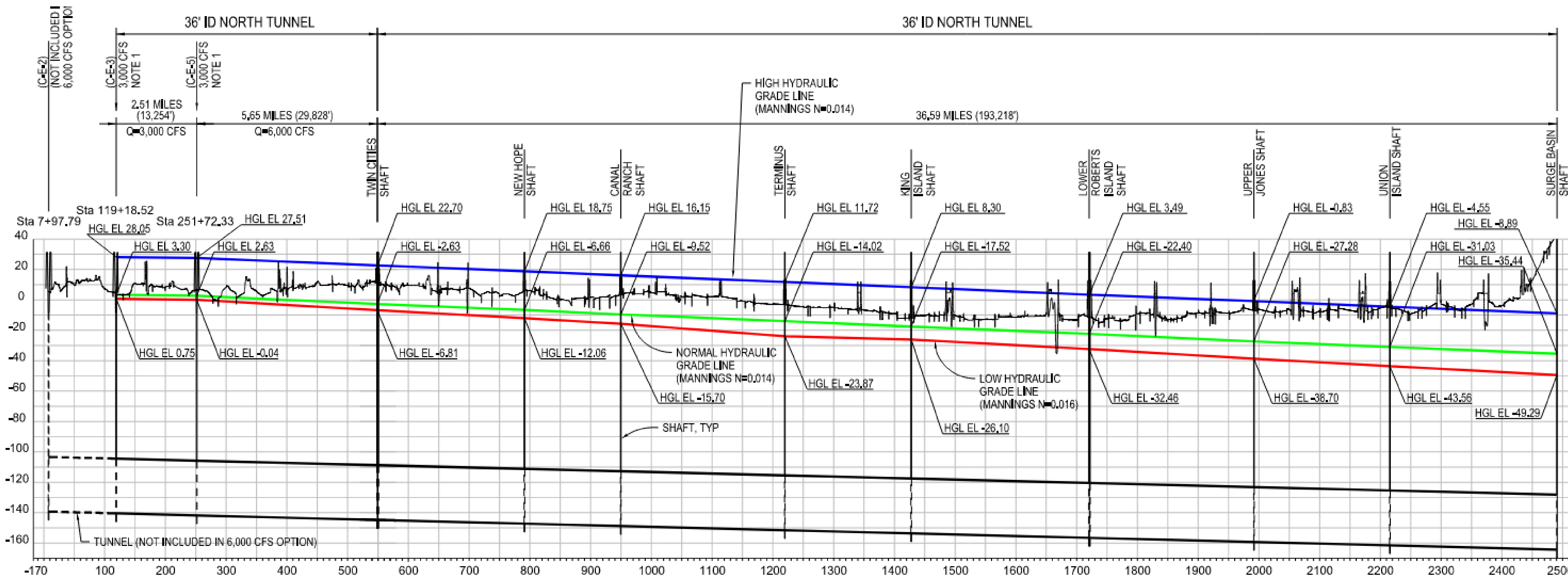

Figure 42. Steady-state HGL; 36-feet-diameter Tunnel; 6,000-cfs Project Design Capacity – Low, Normal, High River Levels

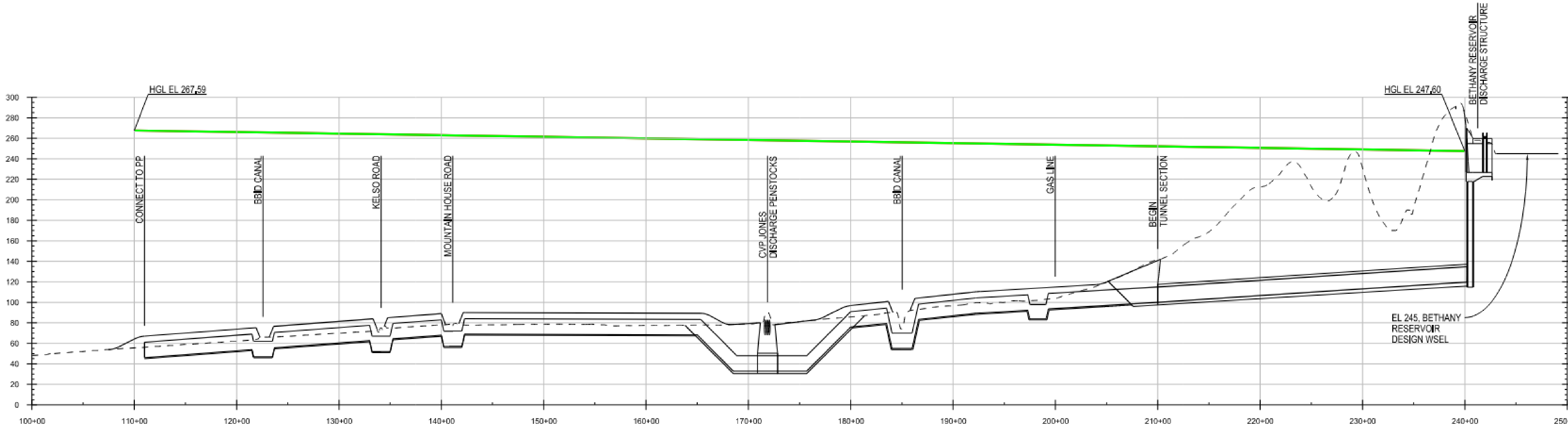

Figure 43. Maximum Steady-state HGL; Bethany Reservoir Aqueduct; 1,500 cfs

5. Surge Analyses

5.1 Model Description

Hydraulic transient-surge analyses were performed between the Intake C-E-3 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 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 a 36-foot tunnel diameter was used for the main tunnel corridor for the Project design capacities of 6,000 cfs. This analysis was 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 3.

The lower friction factor (Manning's *n* of 0.014) was used for this analysis to provide conservative transient-surge results.

Table 3. 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

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. Four 15-foot-diameter pipelines would convey up to 6,000 cfs to the Bethany Reservoir.

5.2 Transient-Surge Results

5.2.1 Tunnels

Results of the transient analyses for the main tunnel, presented on Figures 44 and 45, 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 shaft at the Lower Roberts Island (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 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.

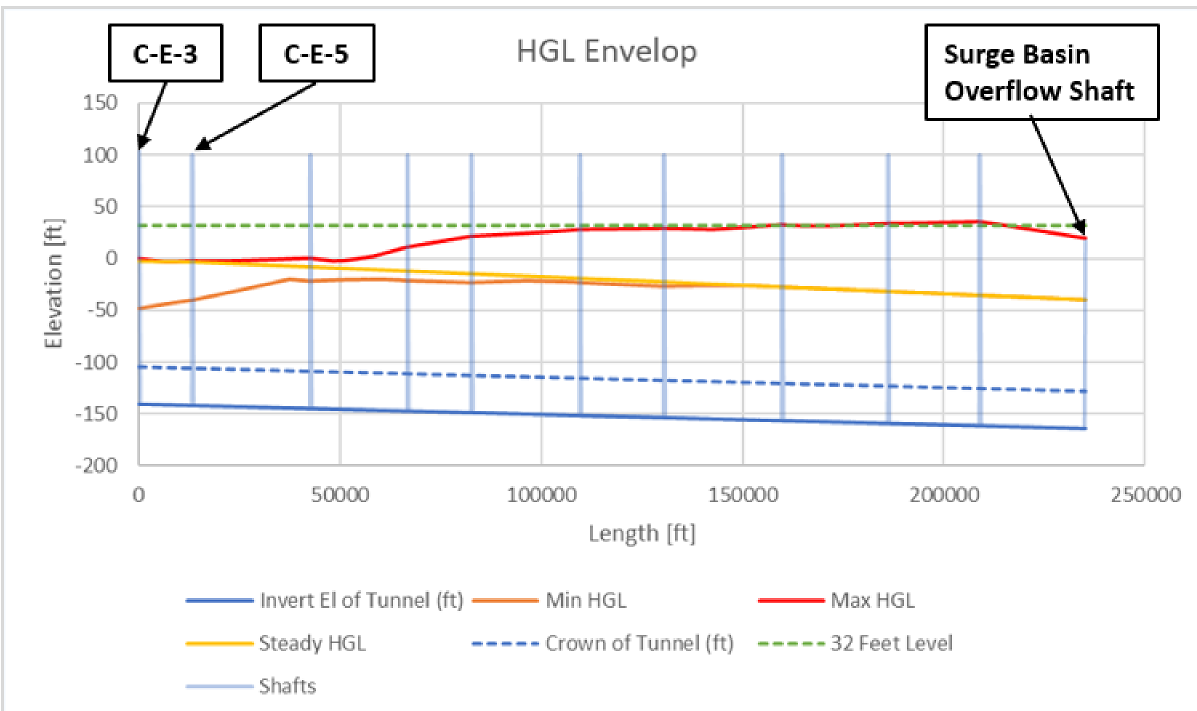

Figure 44. Scenario 1 – Bethany Reservoir Alignment Main Tunnel; Project Design Capacity 6,000 cfs; Low River Level; Minimum and Maximum Hydraulic Gradeline Profiles

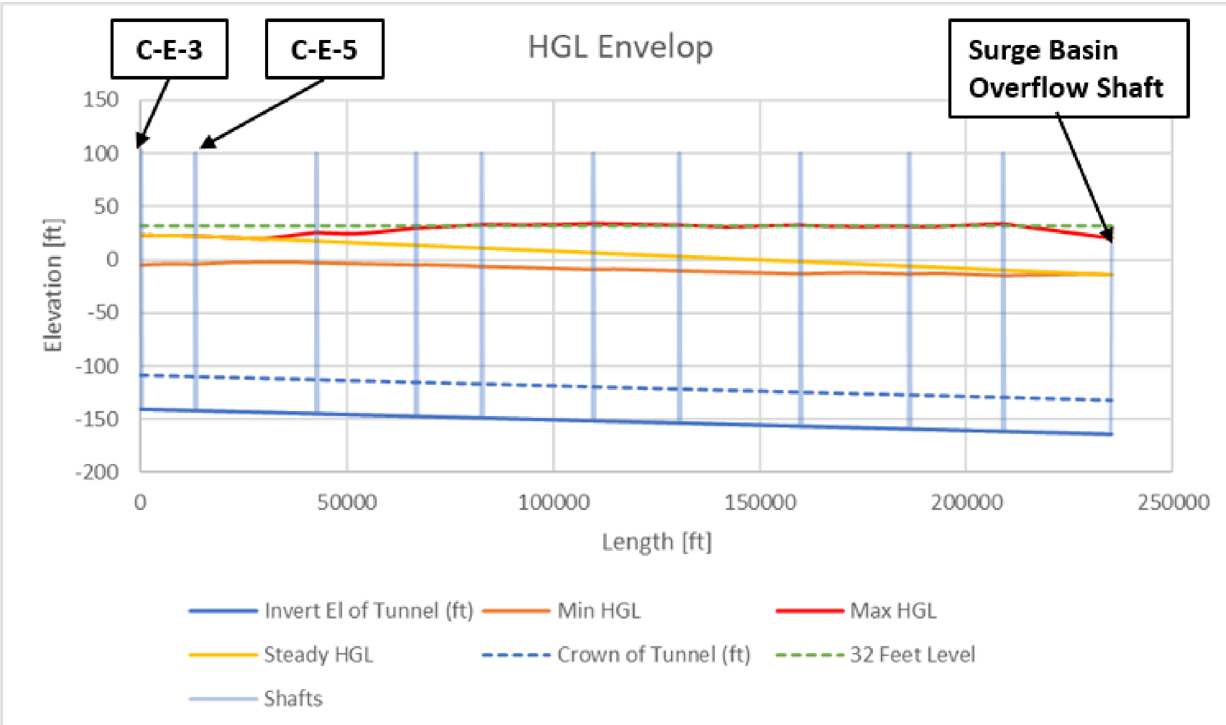

Figure 45. Scenario 2 – Bethany Reservoir Alignment Main Tunnel; Project Design Capacity 6,000 cfs; High River Level; Minimum and Maximum Hydraulic Gradeline Profiles

At 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.

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 capacity of 6,000 cfs were compared against the maximum Sacramento River WSEL associated with the 200-year flood with sea level rise at C-E-3. 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.

Table 4 summarizes the calculated maximum surge HGL at each tunnel shaft developed for Scenarios 1 and 2, 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 selected Project design capacity of 6,000 cfs. The top of shaft elevations shown in Table 5 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 capacity of 6,000 cfs and are shown in the engineering concept drawings.

Table 4. Bethany Reservoir Alignment 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 5 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 WSEL for the Project design capacity of 6,000 cfs). The magnitude of the HGL peaks for scenarios 1 and 2 are very similar due to the fixed overflow elevation within the Surge Basin.

Table 5. Bethany Reservoir Alignment Main Tunnel – Minimum and Maximum HGL Values

Scenario	Minimum HGL [feet]	Maximum HGL [feet]
1	-48.1	35.7
2	-14.9	34.1

5.2.2 Bethany Reservoir Aqueduct

Results for the transient analyses for the Bethany Reservoir Aqueduct pipelines are presented on Figure 46. 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 the Project design capacity of 6,000 cfs. Figure 46 shows the simulated envelop of the maximum and minimum HGL elevations during the transient-surge events along the 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).

The minimum and maximum HGL envelopes on Figure 46 are plotted across the pipeline alignments between the BRPP discharge piping system and the Bethany Reservoir Discharge Structure 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.

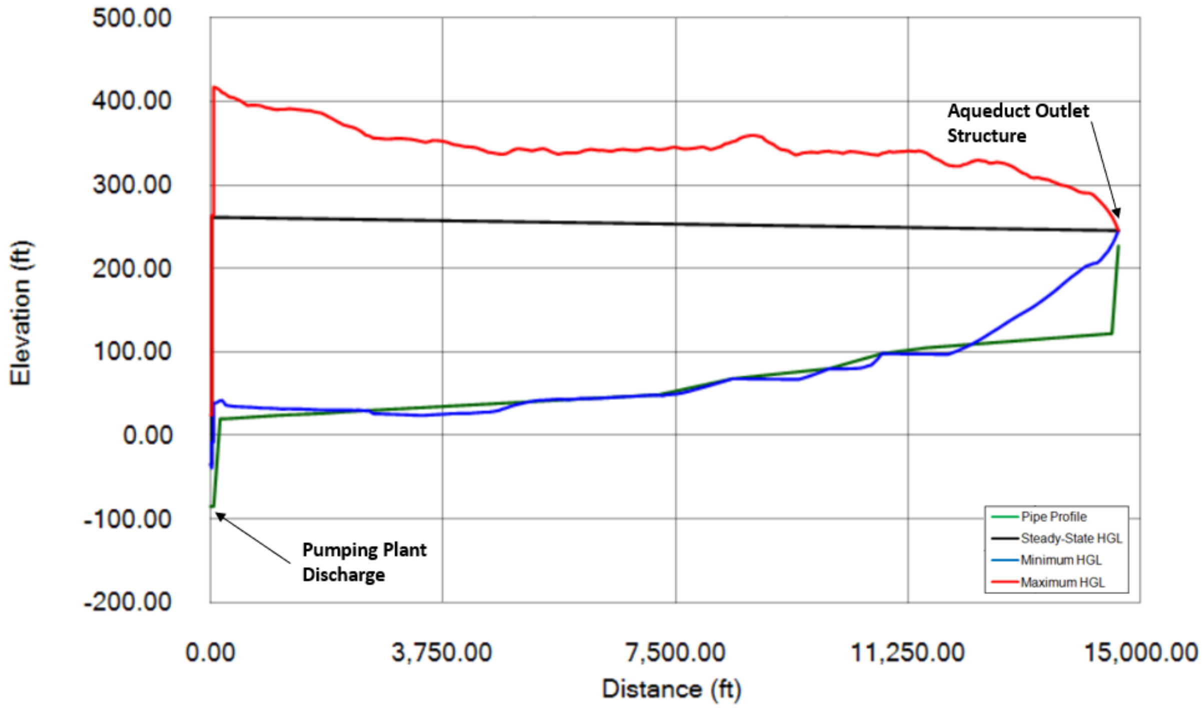

Figure 46. Bethany Reservoir Aqueduct; Design Capacity 1,500 cfs; Minimum and Maximum Hydraulic Gradeline Profiles

The results for the Bethany Reservoir Aqueduct indicate negative internal pressures are developed at several locations along the alignment, as shown on Figure 46. 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 46, the HGL envelope developed during the transient-surge event is within the conceptual design limits of the aqueduct.

6. Tunnel Dewatering

6.1 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 was 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 47 shows the Project schematic with proposed submersible dewatering pump placement.

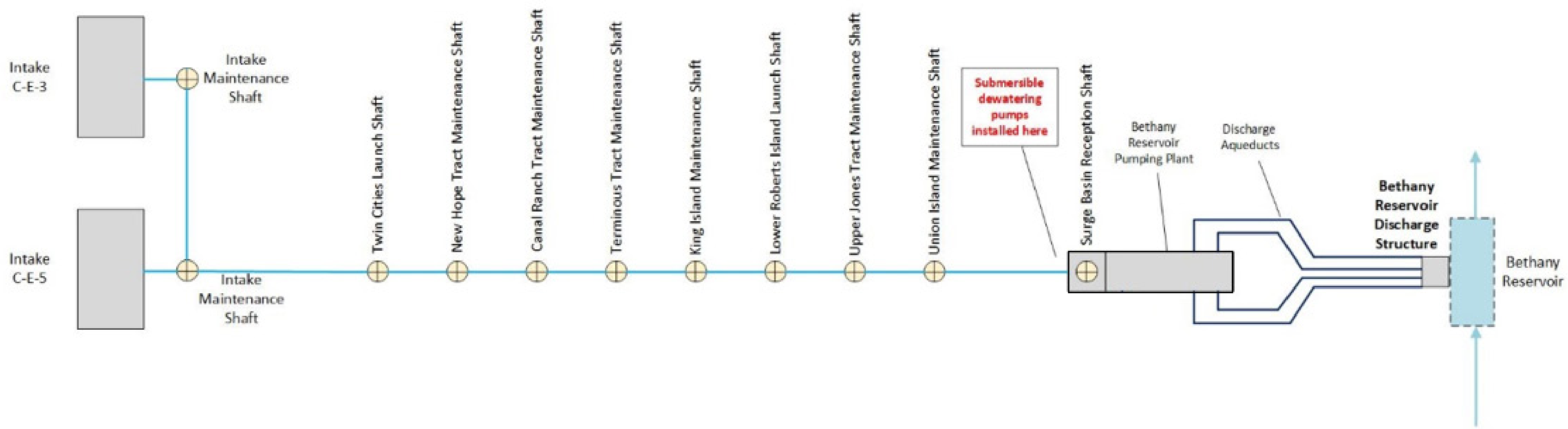

Figure 47. Project Schematic with Proposed Submersible Dewatering Pumps Location

6.2 Modelling Results

Figure 48 provides the results of tunnel dewatering flows for the Bethany Reservoir Alignment 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 48, 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 48 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 48.

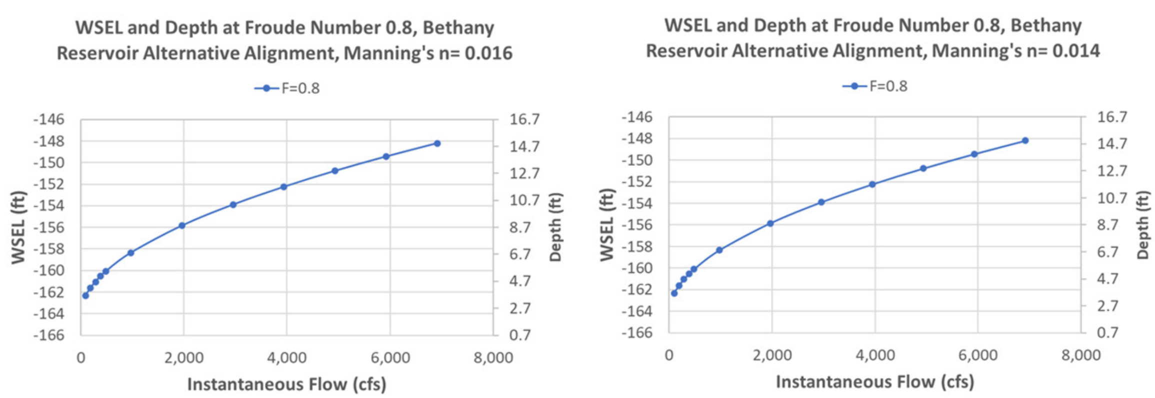

Figure 48. Instantaneous Flow and Level at Froude Numbers of 0.80, 0.85, 0.90, and 0.95 (Left: Manning's *n* = 0.016, Right: Manning's *n* = 0.014)

Tables 6 and 7 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 6. 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 7. 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

6.3 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.

6.4 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.

6.4.1 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 49 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 49.

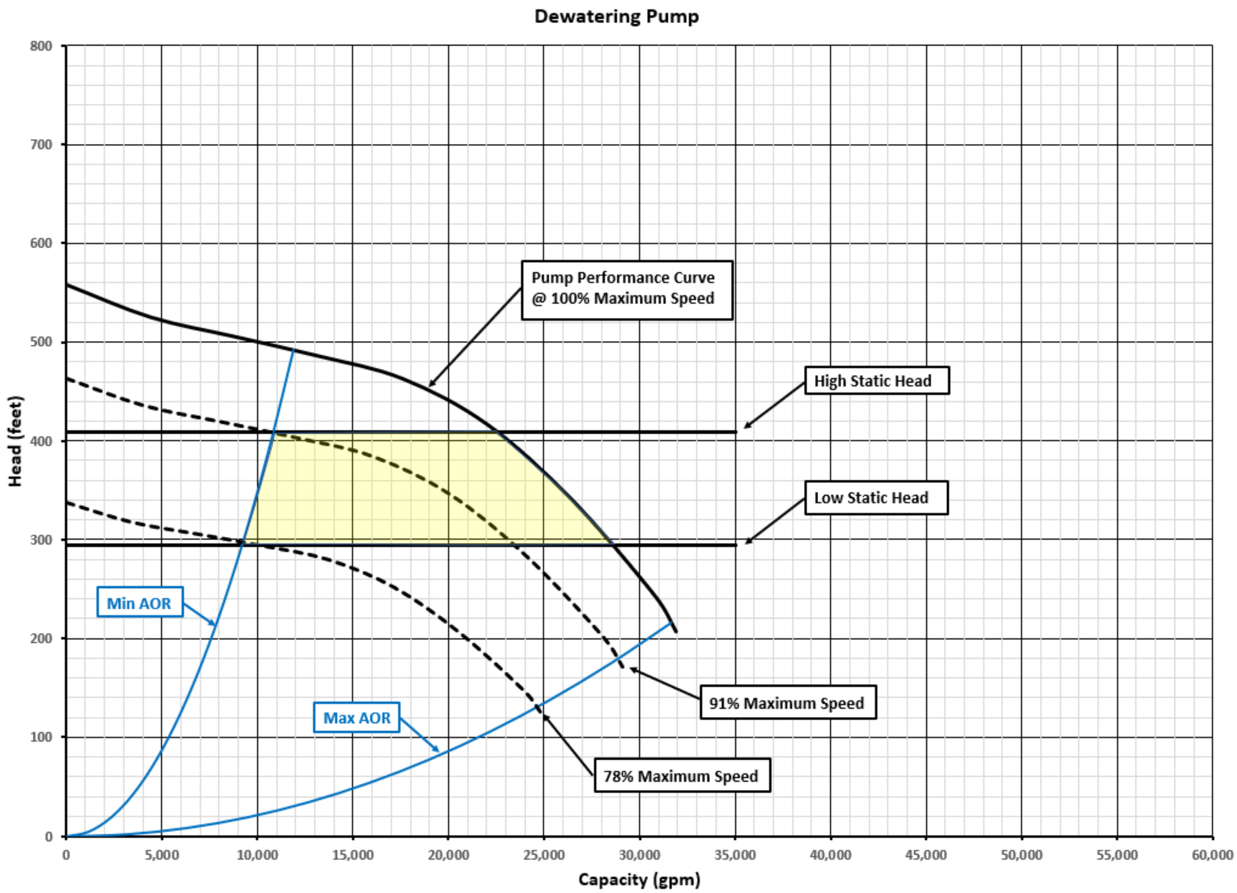

Figure 49. Candidate Pump Performance Curve Versus System Head Curves

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 49.

As can be seen on Figure 49, 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 49, 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 48, and Tables 6 and 7 the dewatering pumps will operate with variable frequency drives. Referring back to Figure 49, 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 6 and 7.

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 9 and 10 throughout the entire dewatering process.

6.5 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 Alignment, 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 capacity of 6,000 cfs. 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 Alignment, 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

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

Attachment 1
Capacity Analysis for Preliminary Tunnel Diameter Selection

Attachment 1. Capacity Analysis for Preliminary Tunnel Diameter Selection

This Attachment 1 to Appendix A2 describes a preliminary analysis performed for selecting the tunnel diameter for the conceptual-level design of the single tunnel system for the Delta Conveyance Project (Project). This Attachment only considered the Central Corridor tunnel alignment of the Project as this preliminary analysis was conducted prior to the development of the Eastern Corridor tunnel alignment option and the Bethany Reservoir Alignment. For the Central Corridor tunnel alignment in this Attachment, Intakes C-E-2 and C-E-3 would each be designed for the maximum Sacramento River diversion flow capacity of 3,000 cubic-feet-per-second (cfs) to achieve the Project design flow capacity of 6,000 cfs. The results of this analysis led to the decision to size the main tunnel diameter for the Central Corridor to 36-foot-inside-diameter for the single tunnel system.

The analyses criterion, methodology and results described herein remained appropriate for selecting the tunnel size of 36-foot-inside-diameter for the Project's Bethany Reservoir Alignment single tunnel system for the maximum design flow capacity of 6,000 cfs. Based on the selected tunnel diameter of 36-foot-inside-diameter for the Bethany Alignment, at the Project design flow capacity of 6,000 cfs, the operating free-water surface elevation range of the Bethany Reservoir Pumping Plant's below ground level wet well was considered feasible. Further hydraulic and transient-surge analyses conducted for the Bethany Reservoir Alignment at the Project design flow capacity of 6,000 cfs demonstrated that internal pressures within the tunnel developed during the transient-surge wave response were maintained within the conceptual-level design criteria for the single tunnel system. Therefore, a 36 foot-inside-diameter tunnel was selected for the Bethany Reservoir Alignment.

1.1 Purpose

The purpose of this Attachment is to perform a preliminary analysis for tunnel diameter selection for the Project. Under the proposed Project, the new north Delta facilities would be sized to convey up to 6,000 cfs of water from the Sacramento River to the State Water Project (SWP) facilities in the south Delta. This Attachment recommends the proposed maximum tunnel flow velocity criteria and corresponding minimum tunnel inside diameter (ID) for maximum design flow capacity of 6,000 cfs.

1.1.1 Background

Potential tunnel diameters were previously identified, considered, and evaluated in support of the WaterFix project. Those tunnels would connect the Sacramento River intakes (intakes) to an intermediate forebay (IF) and extend from the IF to the pumping plant. Note, the original twin tunnels of WaterFix from the IF to the pumping plant have been withdrawn in favor of a single tunnel and subsequent analysis to this Attachment has resulted in elimination of the IF.

This evaluation considered the maximum Project diversion flow of 6,000 cfs with up to two intakes. Tunnel flow velocities and corresponding hydraulic head loss and transient-surge conditions were examined within the Project diversion flow capacity to establish a range of technically feasible candidate tunnel diameters, including their effects on other Project feature configurations (that is, size and operating depths of hydraulic structures).

Hydraulic performance as well as preliminary construction and operating cost analyses were conducted for each candidate tunnel diameter considered and compared to a 44-foot-diameter tunnel. The 44-foot-diameter tunnel was an early and preliminary baseline for a single tunnel analysis based on the original WaterFix hydraulic design criteria.

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 Figure 1's dashed boundary. Evaluation of the southern tunnels connecting the Southern Forebay to the existing approach channels of the existing export Harvey O. Banks Pumping Plant (Banks) and potentially C.W. "Bill" Jones Pumping Plant (Jones) would be conducted as part of a separate hydraulic analysis of the South Delta Conveyance Facilities (SDCF) System.

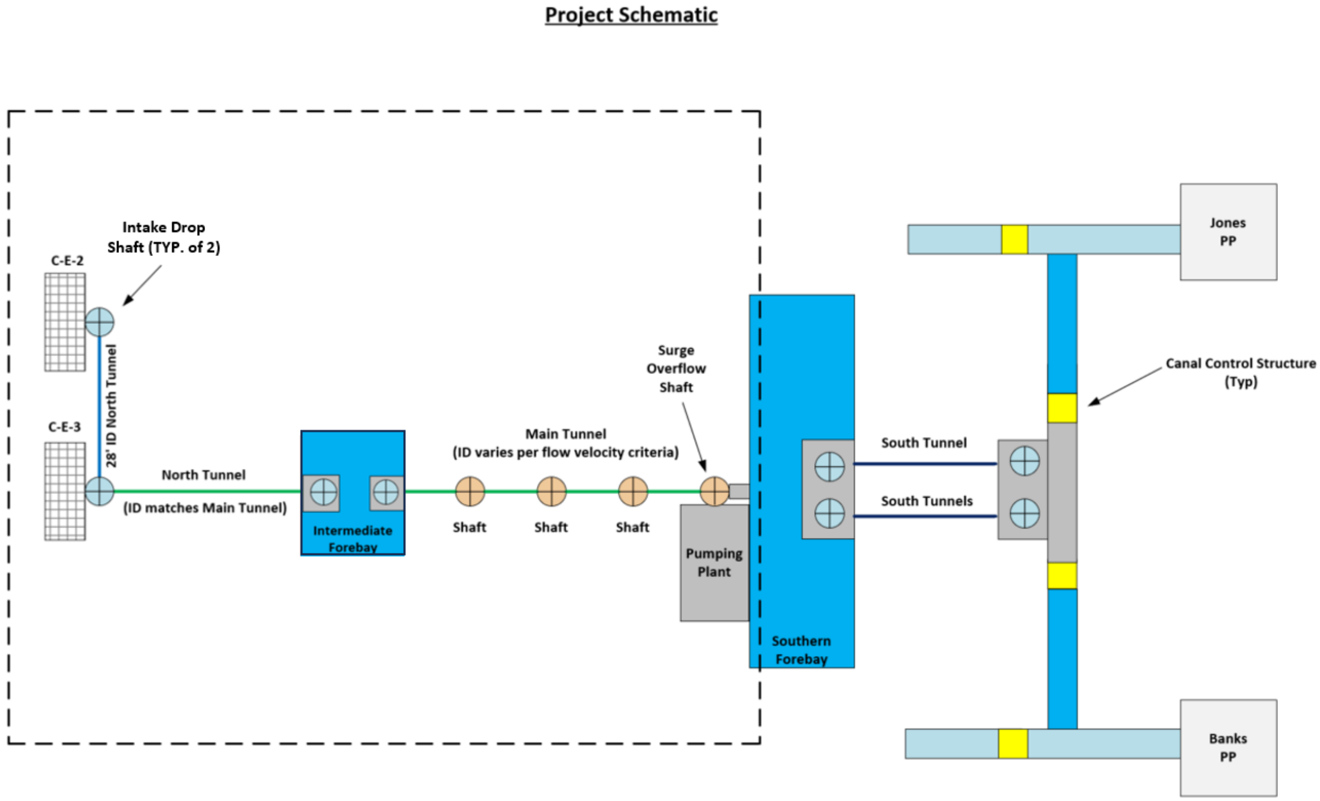

Figure 1. Delta Conveyance System Project Schematic

The following Project configuration was considered for this evaluation:

- Main tunnel alignment
- Two intake locations on the left bank of the Sacramento River, as follows:
  - Intake C-E-2, located downstream of Scribner's Bend between Clarksburg and Hood at approximate River Mile (RM) 41.1
  - Intake C-E-3, located upstream of Hood at approximate RM 39.4

For purposes of this early tunnel sizing exercise, an IF was included in the analysis. While subsequent hydraulic evaluations resulted in elmination of the IF, this design change had no impact on the results of this tunnel sizing analysis. The IF is located north of Twin Cities Road and east of Snodgrass Slough and includes:

- Tunnels connecting the intakes and IF per Figure 1
- A single tunnel connecting C-E-3 to the IF

- A single tunnel connecting the IF and the pumping plant wet well
- The pumping plant, which conveys water to the Sourthern Forebay located on the northwestern side of the existing Clifton Court Forebay

1.1.2 Summary of Results

Based on the results of the hydraulic and capacity analysis, and construction cost comparison as described in this Attachment, it is recommended that the maximum tunnel flow velocity be limited to 6 feet per second (fps) for the maximum design flow capacity of 6,000 cfs. This velocity criteria results in the recommended tunnel diameter of 36-foot finished inside diameter at the maximum design flow capacity of 6,000 cfs.

1.2 Methodology

Tunnel diameter options were evaluated using the following process:

1) Selected the range of candidate tunnel diameters to be evaluated for the Project that achieve the assigned flow velocity criteria range.
2) Conducted system end-to-end hydraulic head loss analysis using candidate tunnel diameters, and reviewed results against corresponding system operating water surface elevations (WSELs) within the tunnel, IF, and pumping plant wet well, and hydraulic horsepower (hp) required by the pumping plant. Compared these results with the characteristics of the baseline 44-foot tunnel diameter.
3) Conducted hydraulic transient-surge analyses and developed the envelope of maximum and minimum hydraulic grade lines (HGLs) for each transient-surge condition. Conducted analysis with three IF footprints of 800 by 1,500 feet (width by length), 800 by 1,000 feet, and 500 by 500 feet, and with no IF included in the Project.
4) Conducted a construction cost comparison of each tunnel diameter option considered against the 44-foot baseline tunnel diameter.
5) Conducted an annual power cost comparison for each tunnel diameter head loss compared against the head loss of the 44-foot baseline tunnel diameter. The present value cost associated with the annual power consumption was included with the construction cost comparison for each tunnel diameter evaluated.
6) Evaluated the candidate tunnel diameter options according to relative suitability of flow velocity, transient-surge response, impacts to operating depths of Project elements (IF and pumping plant wet well invert elevation requirements), and construction and operating costs.
7) Verify recommended velocities are suitable for tunnel lining system.

1.2.1 Data and Information Sources

Average monthly flows were taken from the DSM2 model results presented in Appendix 5A, Section C, of the *Final Bay Delta Conservation Plan/California WaterFix Environmental Impact Report/Environmental Impact Statement* (Final EIR/EIS) (DWR and Reclamation 2016).

1.2.2 Tunnel Sizing and Evaluation Criteria

Criteria were developed from experience with sizing tunnels and water transmission systems to guide the identification and evaluation of candidate tunnel sizes. Criteria included:

- Project Design Flow Range:
  - Hydraulic evaluation was conducted over the range of flows between 0 cfs and the maximum design flow capacity of 6,000 cfs.
- Tunnel Flow Velocity Range:
  - At full capacity, the minimum tunnel flow velocity was limited to 3.5 fps. The minimum tunnel flow velocity was considered to maintain a minimum scour velocity for cleaning sediment within the tunnel.
  - At full capacity, the maximum tunnel flow velocity was limited to 8.0 fps. The maximum flow velocity is consistent with the upper flow velocity range for typical water transmission mains and usually provides a balance between pipe size, power cost, and acceptable transient-surge conditions. This upper limit flow velocity is considered suitable for tunnels with segmental concrete lining systems.
- Candidate Tunnel Diameter Sizing:
  - The minimum candidate tunnel diameter for the maximum design flow capacity 6,000 cfs was calculated based on the maximum flow velocity of 8 fps.
  - The maximum candidate tunnel diameter for the maximum design flow capacity of 6,000 cfs was calculated based on the minimum flow velocity of 3.5 fps.
- Standard Roughness Coefficients:
  - The maximum and minimum interior equivalent roughness coefficients (Manning's N) of 0.016 and 0.014, respectively, have been assigned for the interior roughness of the segmentally lined tunnels and provide a sufficiently conservative analysis range suitable for the conceptual-level tunnel head loss analysis.
  - Manning's N of 0.016 was used as the standard roughness coefficient for the hydraulic head loss analysis to evaluate a more conservative case of resultant tunnel head loss due to the highest tunnel interior friction condition.
  - Manning's N of 0.014 was used as the standard roughness coefficient for the hydraulic transient-surge analysis to evaluate a more conservative case of resultant maximum and minimum surge pressures within the tunnel hydraulic grade line (HGL) due to lower tunnel interior friction condition.
- Steady-State Hydraulic Head Loss Analysis:
  - The source of diversion flows from the Sacramento River was simulated from intakes C-E-2 and C-E-3 for design flow capacities up to 6,000 cfs.
- Transient-Surge Analysis:
  - Hydraulic transient-surge analysis was conducted between the C-E-2 drop shaft to the pumping plant wet well overflow facility at the design flow capacity of 6,000 cfs.
  - The upstream flow conditions were controlled by simulating closure of radial gates at the entrance to the drop shaft at each intake structure. Downstream flow conditions were simulated by fixed weir openings within the pumping plant's overflow shaft.

- Simultaneous shutdown of the main raw water pumps in the pumping plant followed by the closure of sediment basin outlet gates at each intake in operation was simulated for each transient-surge analysis. Outlet gates remained open in their last setpoint position for 5 minutes following pump shutdown and were then simultaneously closed within 1 minute. Closure of the sediment basin outlet gates was simulated to prevent reverse flow into the Sacramento River from the intakes during the transient-surge event.
- Transient-surge analysis was conducted with IF footprints of: 800 by 1,500 feet (width by length); 800 by 1,000 feet; 500 by 500 feet; and with no IF included in the Project.
- The source of diversion flows from the Sacramento River for the steady-state head loss analysis was simulated from intakes C-E-2 and C-E-3 for the design flow capacity of 6,000 cfs.
- Hydraulic transient-surge analysis was conducted with both the minimum and maximum Sacramento River elevations at C-E-2 and C-E-3 at the design flow capacity of 6,000 cfs.

- Cost Analysis:
  - Comparative construction costs were derived from six separate estimates prepared for a single tunnel system, each at different diameters, which include launch and retrieval shafts and associated tunnel structures. Shaft diameters were adjusted to reflect the change in tunnel diameter. Direct cost estimate values were based on 2019 pricing and factored by a 1.76 multiplier to account for programmatic costs, such as risk, contingency, and other soft costs. Using separate estimates, a cost equation was established between the tunnel diameter, tunnel length, and cost to develop comparative costs for the tunnel diameters considered in this analysis.
  - Power cost comparisons for calculated head loss were conducted for each tunnel diameter for two Project operating conditions to establish the upper and lower range of calculated costs. The operating design flow conditions evaluated were:
    - (1) The maximum design rated capacity of 6,000 cfs assumed for 24 hours per day and 365 days per year for the highest cost
    - (2) Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final EIR/EIS (DWR and Reclamation 2016) for the lower cost.
  - The net present value (NPV) of the highest and lowest power costs were computed over a 100-year period with a discount rate of 3.0 percent at an electrical power cost of $0.07 per kilowatt-hour (kWh). A combined pump and motor efficiency of 80 percent was used.

1.2.3 Assumptions and Boundary Conditions

Basic assumptions and boundary conditions that apply to identifying and evaluating the Project include the following:

- Alignment of tunnels and locations of the intakes, shafts, IF, pumping plant, and Southern Forebay are as described in the California WaterFix Draft Supplemental EIR/EIS (DWR and Reclamation 2018)
- Tunnel ID between the C-E-2 drop shaft and the C-E-3 drop of 28 feet
- These design flow sequences of operation at intakes develop the maximum system head loss between the C-E-2 drop shaft and the pumping plant wet well over the full design flow range evaluated:
  - 0 to 2,250 cfs; C-E-2 only

  - 2,250 to 4,500 cfs; C-E-2 maintains the diversion capacity of 2,250 cfs, and C-E-3 begins diverting flows up to 2,250 cfs
  - 4,500 to 6,000 cfs; diversion capacities of C-E-2 and C-E-3 are increased at equal capacities up to the maximum diversion rate of 3,000 cfs each
- Sacramento River elevations (North American Vertical Datum of 1988 [NAVD88]) assumed for the hydraulic head loss and transient-surge analysis:
  - Minimum of 1.9 and maximum of 31.4 feet at C-E-2
  - Minimum of 1.6 and maximum of 30.4 feet at C-E-3
- Overflow weir crest at the pumping plant wet well inlet shaft was set at 19 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-ft2); 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-ft2; this value was calculated based on the candidate pump's BHP at the rated design point condition
- Wave speed for the main tunnel between C-E-3 and the pumping plant wet well was calculated as 1,589 fps using the Wave Speed Calculator function in HAMMER
- Wave speed for the 28-foot-diameter tunnel between C-E-2 and C-E-3 was calculated as 1,589 fps using the Wave Speed Calculator function in HAMMER

1.2.4 Tools

1.2.4.1 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. The model configuration consisted of the following components:

- Sacramento River intakes C-E-2 and C-E-3 including inlet structures, screens, control gates, sediment basins, and tunnel drop shafts
- IF, including separate tunnel inlet and tunnel outlet structures
- For IF removed, the forebay was removed and the two shafts were connected by a tunnel segment
- Pumping plant wet well and gravity flow and surge overflow shaft
- Tunnels connecting the intakes to the IF and the IF to the pumping plant wet well and gravity flow and surge overflow shaft

1.2.4.2 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.

1.3 Analysis and Evaluation

1.3.1 Steady-State Hydraulic Head Loss Analysis

In accordance with the methodology and criteria described, a steady-state, hydraulic head loss analysis was performed for a range of tunnel diameters between the C-E-3 drop shaft and the IF, and from the IF to the pumping plant wet well that resulted in flow velocities between 3.5 to 8.0 fps for the maximum design flow of 6,000 cfs.

Table 1 summarizes candidate tunnel diameters meeting the flow velocity criteria. Tunnel diameters are shown in 1-foot increments within the tunnel flow velocity range of 3.5 to 8.0 fps.

A steady-state hydraulic analysis was conducted for each tunnel diameter shown in Table 1. To determine the minimum resultant WSELs in the IF and the pumping plant wet well, tunnel head losses were separately calculated between the intakes to the IF, and from the IF to the Pumping plant wet well for each design flow range for each candidate tunnel diameter.

Table 1. Summary of Candidate Tunnel Diameters Relative to Design Flow and Flow Velocity Criteria, Design Flow Capacity = 6,000 cfs

Tunnel Diameter (feet)	Tunnel Flow Velocity (fps)	Tunnel Diameter (feet)	Tunnel Flow Velocity (fps)
47	3.5	38	5.3
46	3.6	37	5.6
45	3.8	36	6.0
44	3.9	35	6.2
43	4.1	34	6.6
42	4.3	33	7.0
41	4.5	32	7.5
40	4.8	31	7.9
39	5.0	Not Applicable	Not Applicable

The steady-state hydraulic analysis incorporated the highest friction factor (Manning's N of 0.016) and the lowest Sacramento River elevations of 1.9 and 1.6 at C-E-2 and C-E-3 respectively, for each candidate tunnel diameter to establish the lowest HGL (maximum head loss) throughout the Project (intakes to pumping plant wet well) and the resulting minimum operating WSELs at each hydraulic facility (IF and pumping plant wet well). This analysis included head losses through the fish screens at each intake. Fish screens were assumed to be in a clean condition.

Figure 2 plots the tunnel head loss results that were developed between C-E-2 and the IF over the full envelope of design flow conditions. On Figure 2 the maximum design flow capacity of 6,000 cfs is shown as a black vertical dashed line. The head loss curves are plotted from a minimum flow of 750 cfs up to the maximum design flow capacity. Tunnel diameter plots are generally shown in 2-foot increments. Head loss corresponding with tunnel diameters not shown can be obtained through visual inspection for any flow condition.

Head loss curves shown on Figure 2 have an inflection point at 2,250 cfs. This is due to flows entering the tunnel from C-E-2 only for all tunnel flows up to 2,250 cfs. For all tunnel flows greater than 2,250 cfs but less than 6000 cfs, flows from C-E-2 are combined with flows from C-E-3. As such, the maximum tunnel flow between the C-E-2 and C-E-3 drop shafts is limited to 3,000 cfs (maximum diversion flow from C-E-2), and the maximum tunnel flow between the C-E-3 drop shaft and the IF is limited to 6,000 cfs (maximum combined diversion flows from C-E-2 and C-E-3). Table 2 summarizes the head loss and corresponding IF WSEL for each tunnel diameter (at 1-foot increments) up to the maximum design flow capacity of 6,000 cfs.

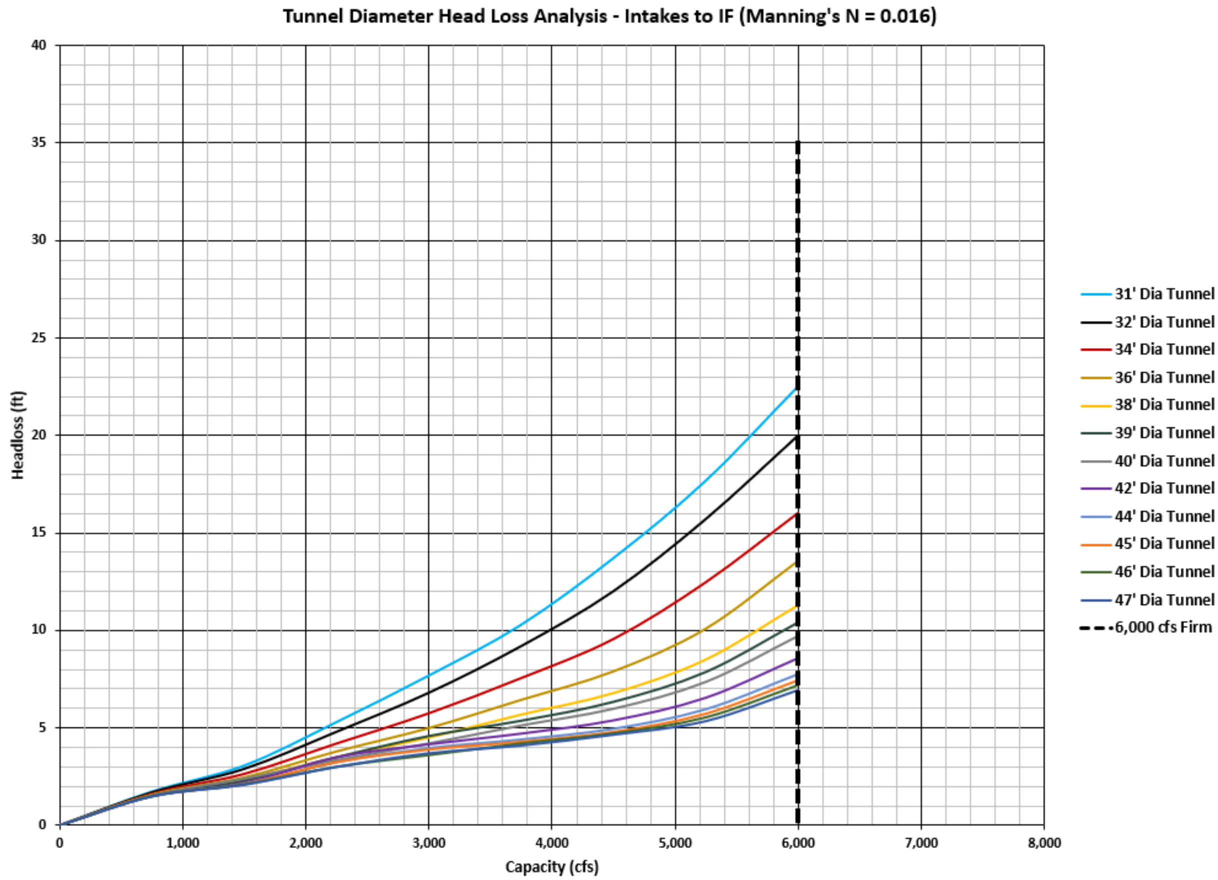

Figure 2. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 6,000 cfs

Table 2. Tunnel Head Loss and Intermediate Forebay Water Surface Elevation, Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
47	3.5	6.9	-5.0
46	3.6	7.2	-5.3
45	3.8	7.4	-5.5
44	3.9	7.8	-5.9
43	4.1	8.2	-6.3
42	4.3	8.6	-6.7

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from Intakes to IF (feet)	IF WSEL (feet)
41	4.5	9.1	-7.2
40	4.8	9.7	-7.8
39	5.0	10.4	-8.6
38	5.3	11.3	-9.4
37	5.6	12.3	-10.4
36	6.0	13.6	-11.7
35	6.2	15.2	-13.3
34	6.6	16.0	-14.1
33	7.0	18.4	-16.5
32	7.5	20.0	-18.1
31	7.9	22.5	-20.6

Referring to Figure 2 and Table 2, at the maximum design flow capacity of 6,000 cfs, the minimum tunnel diameter of 31 feet (7.9-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 22.5 feet and results in a WSEL in the IF of -20.6 feet. At the same design flow capacity, the maximum tunnel diameter of 47 feet (3.5-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 6.9 feet and results in a WSEL in the IF of -5.0 feet. The head loss comparison between the minimum and maximum tunnel diameters (31 and 47 feet) at the design flow capacity of 6,000 cfs is 22.5 versus 6.9 feet.

Figure 3 plots the tunnel head loss results developed between the IF and the pumping plant wet well over the full envelope of design flow conditions. The steady-state WSELs associated with the IF were established by the steady-state head loss analysis between the intakes and IF at the same design flow conditions and candidate tunnel diameter. The maximum design flow capacity of 6,000 cfs is shown in the graph as a vertical blue dashed line. For clarity, not all tunnel diameters summarized in Table 1 are shown on Figure 3. Head loss associated with tunnel diameters not shown can be obtained through visual inspection. A legend is provided on each figure that defines the color code for each tunnel diameter head loss curve shown.

Tables 3 summarizes the head loss and corresponding pumping plant wet well WSEL for each tunnel diameter (at 1-foot increments) up to the maximum design flow capacity of 6000 cfs.

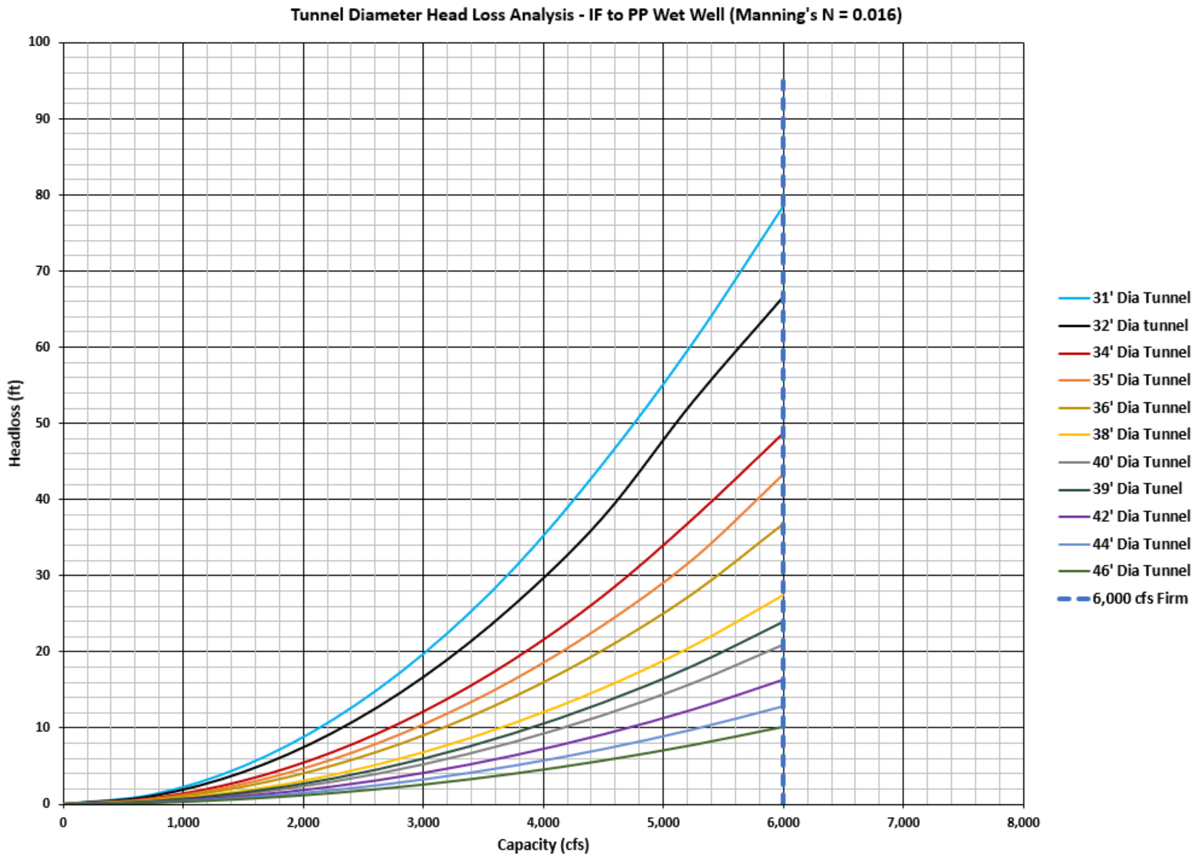

Figure 3. Flow Capacity versus Head Loss for Candidate Tunnel Diameters up to Maximum Design Flow 6,000 cfs

Table 3. Tunnel Head Loss and Pumping Plant Wet Well Water Surface Elevation at Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
47	3.5	9.2	-14.2
46	3.6	10.2	-15.5
45	3.8	11.4	-16.9
44	3.9	12.8	-18.7
43	4.1	14.4	-20.7
42	4.3	16.3	-23.0
41	4.5	18.5	-25.7
40	4.8	21.0	-28.8
39	5.0	24.0	-32.6
38	5.3	27.5	-36.9
37	5.6	31.7	-42.1
36	6.0	36.8	-48.5
35	6.2	43.3	-56.6

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss from IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
34	6.6	48.7	-62.8
33	7.0	57.8	-74.3
32	7.5	66.6	-84.7
31	7.9	78.5	-99.1

Referring to Figure 3 and Table 3, at the maximum design flow capacity of 6,000 cfs, the minimum tunnel diameter of 31 feet (7.9-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 78.5 feet and results in a WSEL in the pumping plant wet well of –99.1 feet. At the same design flow capacity, the maximum tunnel diameter of 47 feet (3.5-fps flow velocity at 6,000 cfs) develops a steady-state head loss of 9.2 feet and results in a WSEL in the pumping plant Wet Well of -14.2 feet. The head loss comparison between the minimum and maximum tunnel diameters (31 and 47 feet) at the design flow capacity of 6,000 cfs is 78.5 versus 9.2 feet.

1.3.2 Evaluate Tunnel Head Loss with Candidate Pump Selection

The results of the steady-state head loss analysis conducted for each tunnel diameter for the flow range between 600 to 6,000 cfs were compared against the achievable system head operating envelopes for pumping equipment sized for the capacity and total dynamic head (TDH) conditions for the Project. The purpose of this evaluation was to identify head loss constraints associated with achievable pump performance for further screening feasible tunnel diameter options. This analysis was conducted with hydraulic performance characteristics associated with single-stage, pull-out-style vertical pumps.

General pump capacity sizing charts from Flygt-Xylem (a current candidate pump manufacturer considered for the conceptual design development of the pumping plant), shown on Figure 4, indicate a maximum achievable capacity for single-stage, vertical pull-out-style pumps at rated TDH conditions of up to 90 feet is 400,000 gallons per minute (gpm) (890 cfs) per pump (as shown on the left chart on Figure 8). For TDH conditions between 90 and 350 feet, the maximum achievable capacity is limited to 250,000 gpm (557 cfs) per pump and requires multiple impeller stages for TDH conditions between 150 to 350 feet to achieve a 557 cfs (as shown in the gray regions on the right chart on Figure 4).

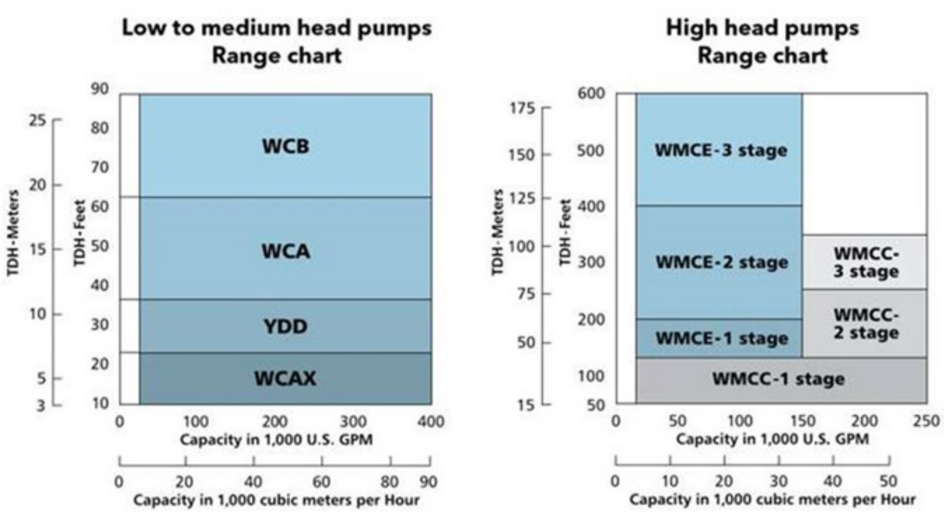

Figure 4. Low to High Head Range Chart for Flygt Large Vertical Column Pull-Out-Style Pumps

The high WSEL of the Project's Southern Forebay is currently set at 17.5 feet. As such, the minimum WSEL within the pumping plant wet well cannot be lower than -72.5 feet (17.5 feet minus 90 feet) to use the 890-cfs capacity pumps. For pumping plant wet well WSELs lower than -72.5 feet, the lower-capacity (557 cfs) pumps must be used. The lower-capacity pumps would require substantially larger pumping plant and wet well structures to achieve the required maximum design flow capacity as compared to using the higher-capacity pumps.

In addition to achieving the required maximum design flow capacity, the pumping plant must also be capable of operating at lower flow capacities at lower TDH conditions. The current design flow capacity range for the pumping plant is from 10 percent of the maximum design flow capacity to the maximum established design flow capacity and includes steady-state operation for all flows in between (that is, no flow gaps within the design flow range). To use a pump capable of achieving the TDH conditions at the required maximum and minimum TDH conditions, variable speed drives would be required.

For the purpose of this preliminary evaluation, the variable speed turndown of the candidate pumps would be limited to 50%. For example, for pumps with a maximum rated speed of 200 revolutions per minute (rpm), the minimum pump speed would be limited to 100 rpm (50 percent of 200 rpm). Using the pump industry standard Affinity Laws, the lowest TDH conditions for a pump with a maximum rated speed of 200 rpm that is operated at 100 rpm would be 0.25 times the head conditions at maximum speed. So that steady-state flows can be achieved at lower flow and head conditions associated with the Project (high Sacramento River elevations, low Southern Forebay WSELs, and lower tunnel head loss conditions) within the pump's recommended performance range, the rated TDH condition limit is established by the minimum achievable TDH at maximum speed (within the pump manufacturer's allowable performance range) multiplied by 0.25.

For the maximum design capacity of 6,000 cfs, the estimated design head condition at 600 cfs (10 percent of the maximum design flow capacities) is approximately 10 feet (static head plus system head loss). Static head conditions associated with the Project's design head conditions associated with this evaluation consider the Sacramento River elevation of 6.9 feet and the Southern Forebay WSEL of 14.8 feet (7.9 feet of static head). Therefore, the manufacturer's recommended minimum TDH operating condition at the pump's maximum speed condition cannot be more than 40 feet (10 feet divided by 0.25).

Figure 5 shows a pump performance curve for the Flygt pump model 180X120 WCF high-capacity pump operating at the rated maximum speed of 200 rpm. This preliminary pump selection was evaluated and found to be feasible for conceptual-level evaluation for the Project. As shown on Figure 5 the minimum recommended head condition (runout head condition) within the pump manufacturer's allowable operating region (AOR) is 30 feet. The minimum head condition within the preferred operating range (POR) is 50 feet at maximum speed. This minimum head condition results in pump operation just outside of the POR but well within the AOR, and stable pump operation would be expected.

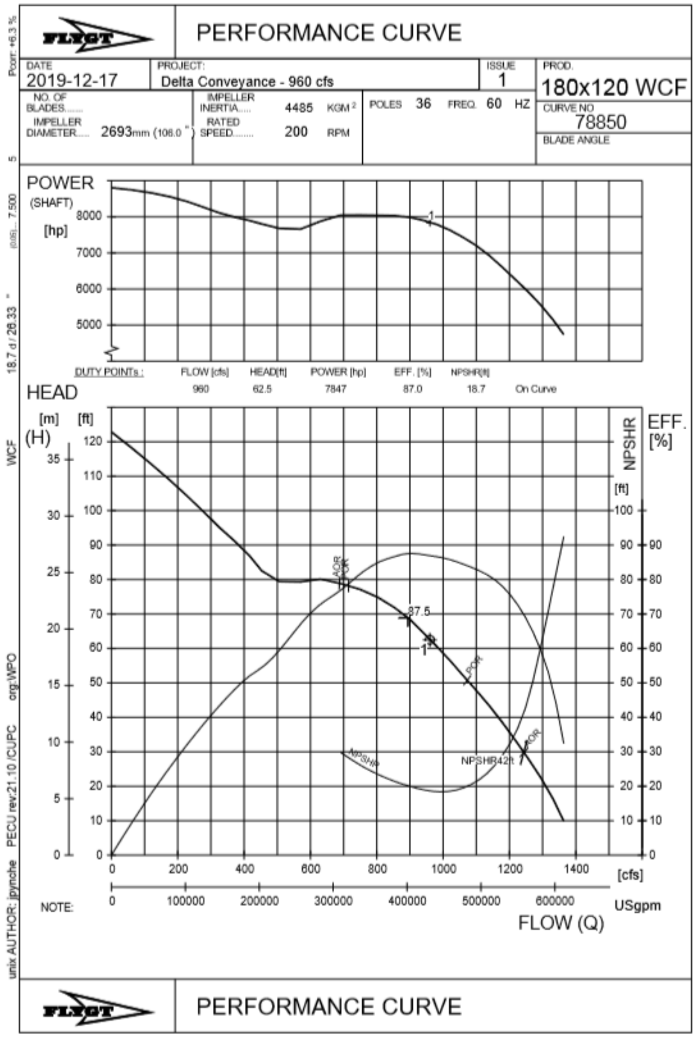

Figure 5. Flygt Pumps' Characteristic Pump Performance Curve, Model 180x120 WCF

The maximum head condition within the pumps' POR is around 79 feet. Note, in this pump characteristic performance curve, the manufacturer has identified the POR and AOR to be essentially the same performance condition. At this level of preliminary analysis, steady-state operation of at least 5 feet below the maximum POR and AOR head of 79 feet is recommended to provide a suitable margin below the AOR head because operation exceeding the AOR head condition is where the manufacturer considers significant pump recirculation to begin. When the margin is applied to the maximum POR and AOR TDH, the maximum operating head condition is limited to 74 feet (79 feet minus 5 feet) at maximum speed.

Based on the Southern Forebay's calculated maximum WSEL of 17.5 feet, and to use the same pump for the entire envelope of total design head conditions, the WSEL in the pumping plant wet well must not be below -56.5 feet (17.5 feet minus 74 feet) for the maximum design flow and TDH conditions associated. Therefore, to use the higher-capacity pumps, head loss associated with each tunnel diameter for each maximum design flow capacity cannot not exceed 58.4 feet based on the minimum Sacramento River elevation of 1.9 feet, the maximum WSEL of 17.5 feet in the Southern Forebay, and the tunnel head loss calculated with a Manning's N of 0.016.

1.3.2.1 Evaluate High-Capacity Candidate Pumps at Maximum Design Flow Condition 6,000 cfs

Table 4 shows the head loss summary between C-E-2 and the pumping plant wet well for each tunnel diameter option at the maximum design flow capacity of 6,000 cfs. The steady-state WSELs at the IF and the pumping plant wet well are also shown for each tunnel diameter option. WSELs at the IF and wet well are based on the minimum Sacramento River elevation of 1.9 feet at C-E-2.

Table 4. Head Loss and Water Surface Elevation Summary, Maximum Design Flow Capacity 6,000 cfs

Tunnel Diameter (feet)	Flow Velocity (fps)	Head Loss Intakes to IF (feet)	IF WSEL (feet)	Head Loss IF to Pumping Plant Wet Well (feet)	Pumping Plant Wet Well WSEL (feet)
47	3.5	6.9	-5.0	9.2	-14.2
46	3.6	7.2	-5.3	10.2	-15.5
45	3.8	7.4	-5.5	11.4	-16.9
44	3.9	7.8	-5.9	12.8	-18.7
43	4.1	8.2	-6.3	14.4	-20.7
42	4.3	8.6	-6.7	16.3	-23.0
41	4.5	9.1	-7.2	18.5	-25.7
40	4.8	9.7	-7.8	21.0	-28.8
39	5.0	10.4	-8.6	24.0	-32.6
38	5.3	11.3	-9.4	27.5	-36.9
37	5.6	12.3	-10.4	31.7	-42.1
36	6.0	13.6	-11.7	36.8	-48.5
35	6.2	15.2	-13.3	43.6	-56.9
34	6.6	16.0	-14.1	48.7	-62.8
33	7.0	18.4	-16.5	57.8	-74.3
32	7.5	20.0	-18.1	66.6	-84.7
31	7.9	22.5	-20.6	78.5	-99.1

As can be seen in Table 4, the 35-foot-diameter tunnel results in a head loss of 58.8 feet (15.2 feet plus 43.3 feet) and a minimum pumping plant wet well WSEL of -56.9 feet at the maximum design flow capacity of 6,000 cfs. Therefore, tunnel diameters 35 feet and smaller are eliminated from further consideration because their head loss exceeds 58.4 feet and results in WSELs in the pumping plant wet well below -56.5 feet at the maximum design flow condition of 6,000 cfs.

1.3.2.2 Tunnel Sizing Summary

Based on the steady-state hydraulic analysis and the high-capacity candidate pump performance envelope criteria, it recommended to maintain the minimum finished inside tunnel diameter of 36 feet for the Project maximum design flow capacity of 6,000 cfs, which results in a flow velocity of 6.0 fps. Other corridors under consideration would require additional tunnel length and additional head loss. At 6 fps design velocity, it is expected the tunnel diameters would be suitable for all corridors.

1.3.3 Hydraulic Transient-Surge Analysis

In accordance with the methodology and criteria described, a hydraulic transient-surge analysis was performed for the Delta Conveyance system 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 alignment resulting from transient-surge events at selected tunnel diameters and design flow conditions.

The maximum tunnel flow velocity of 6 fps was recommended based on the results of the steady-state hydraulic head loss evaluation. For this analysis, tunnel diameters between the C-E-3 drop shaft and the pumping plant wet well were selected based on the tunnel steady-state flow velocities of 6 and 7 fps at the maximum design flow rate of 6,000 cfs, respectively, to evaluate the single tunnel system at the recommended maximum velocity of 6 fps, and to evaluate transient-surge results sensitivity with higher flow velocities at the design flow conditions of 6,000 cfs. The tunnel diameters and corresponding flow velocities are shown in Table 5.

The tunnel diameters between C-E-2 and C-E-3 was maintained at 28 feet. The lower friction factor (Manning's N of 0.014) was used for this analysis to provide conservative transient-surge results.

Table 5. Selected Design Flow Capacities and Candidate Tunnel Diameters for Transient-Surge Analysis, Design Flow Capacity = 6,000 cfs

Tunnel Diameter (feet)	Tunnel Flow Velocity (fps)
36	6
33	7

Each candidate diameter was evaluated with three IF volumes corresponding with IF facility footprints of 1,500 by 800, 1,000 by 800, and 500 by 500 feet, and with no IF included in the Project. The minimum working WSEL for each IF footprint was set at the steady-state, free WSEL determined in the head loss analysis for each tunnel diameter identified in Table 5 at the design flow condition of 6,000 cfs. In each condition, initial steady-state conditions were established prior to simulating the transient-surge event. Each candidate tunnel diameter was evaluated with both the maximum and minimum Sacramento River elevations at each intake and with the invert elevation of the overflow weir at the pumping plant overflow shaft set at 19 feet.

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 within 6 minutes at each intake in operation. Outlet gates remained open in their last steady-state setpoint condition (prior to pump shutdown) for 5 minutes following pump shutdown and were then simultaneously closed within 1 minute. Closure of the intake control gates prevented reverse flow into the Sacramento River from the intakes during the transient-surge event.

Figures 6 through 9 show the hydraulic transient maximum and minimum HGL elevations that occur throughout the transient-surge events along the tunnel alignment using the four IF configurations described. The maximum and minimum HGL elevations are shown as the envelope of the maximum and minimum HGLs and are plotted across the tunnel alignment from the C-E-2 drop shaft to the overflow shaft at the pumping plant. The tunnel and drop shafts at the C-E-2, C-E-3, IF, and pumping plant overflow are depicted and noted in each graph. The tunnel crown elevation for each diameter evaluated is shown as a dashed blue line on each graph. Other shafts, including maintenance shafts along the tunnel alignment, are not shown but were included in the transient-surge model.

The horizontal axis of each graph depicts the tunnel alignment length and general location (in linear feet) of the drop shafts, the IF inlet and outlet drop shafts (near 50,000 feet along the alignment), and the overflow shaft located at the end of the tunnel alignment. The vertical axis defines the invert and crown elevations of the tunnel along the alignment, and the invert and top elevations of the shafts, IF inlet and outlet shafts, and the overflow shafts. The vertical axis also defines the calculated steady-state, and maximum and minimum transient-surge HGLs that are plotted along the alignment.

On Figures 6 through 9, the tunnel diameter between C-E-2 and C-E-3 drop shafts was maintained at 28 feet. The tunnel diameter between the C-E-3 drop shaft and the pumping plant overflow shaft was varied based on Table 11 for each tunnel flow velocity and design flow capacity condition. A reference HGL of 32 feet (depicted as a dashed line across the entire tunnel alignment) is shown to assist in visually inspecting the maximum transient-surge HGL across the alignment for all conditions evaluated.

The invert elevations of the C-E-2 and C-E-3 drop shafts at their tunnel connections are -122.63 feet and -131.04 feet, respectively. For the tunnel to maintain full pressurized flow throughout each section of the alignment, the minimum HGL elevation along the alignment must be higher than the interior crown of the tunnel (invert elevation of the tunnel plus the tunnel finished ID). For example, the crown of the tunnel at the C-E-2 drop shaft is -94.63 feet (invert elevation of -122.63 feet plus 28 feet). The crown of the tunnel at the C-E-3 drop shaft is the invert elevation of -131.04 plus the candidate tunnel diameter being evaluated.

For all IF volume configurations, including when the IF was removed from the tunnel, surge overflow was only permitted at the pumping plant overflow shaft for all transient-surge conditions evaluated. Top-of-shaft elevations for all tunnel shafts along the tunnel alignment (including the C-E-2 and C-E-3 drop shafts) were raised above their respective high-WSELs to contain the tunnel volume so that the maximum resultant maximum HGL conditions along the tunnel alignment could be evaluated.

1.3.3.1 Condition 1: 36-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 6 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 36 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 6 fps at the design flow condition of 6,000 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained above -94.63 feet in the C-E-2 drop shaft and -95.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is just below -50 feet for all IF configurations evaluated. For the three IF volumes, the minimum HGL was generally equivalent to steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. For the configuration with the IF removed, the minimum HGL was generally equivalent to the steady-state conditions within the last 50,000 feet of the tunnel. No negative pressures were determined to be within the tunnel sections throughout the entire alignment for all IF configurations.

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The highest resultant maximum HGL occurred with the IF footprint volumes of 800 feet by 1,500 feet (width by length), which was slightly above 32 feet. The maximum HGL elevation at C-E-2 drop shaft for the IF footprint configuration of 800 feet by 1,000 feet was 32 feet. For the IF configuration of 500 feet by 500 feet and with the IF removed, the maximum HGL at the C-E-2 drop shaft was below 32 feet. For the configuration with the IF removed, the maximum HGL remained generally linear, below 32 feet along the entire tunnel alignment.

1.3.3.2 Condition 2: 36-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 6.0 fps

Figure 7 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 36 feet between the C-E-2 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 6,000 cfs and flow velocity of 6 fps. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -95.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and the configuration with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint volumes (approximately -25 feet). For the tunnel configuration with the IF removed, the WSEL in the C-E-2 drop shaft falls just below -50 feet and is generally the same as that determined for Condition 1. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 2 versus Condition 1), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 2 than the overflow volume that occurred in Condition 1 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The differences in these overflow volumes result in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 1 and 2. No negative pressures were determined to be within the tunnel sections throughout the entire alignment.

The resultant maximum HGL elevation for each of the three IF volumes occur at the C-E-2 drop shaft. The maximum HGL elevations for the two IF footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet were well above 50 feet. The maximum HGL elevation for the IF footprint of 500 feet by 500 feet was slightly above 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed from the tunnel, the maximum HGL occurred downstream of the shaft, where the IF was previously located, and did not exceed the steady-state HGL at the C-E-2 drop shaft. With the IF removed, the maximum HGL profile was generally uniform between the IF and the pumping plant overflow shaft. Based on the maximum HGL elevations described for this transient-surge condition, an overflow condition would occur at the C-E-2 and C-E-3 facilities for all IF footprint volume configurations. An overflow condition at the C-E-2 or the C-E-3 facilities would not occur for the tunnel configuration with the IF removed.

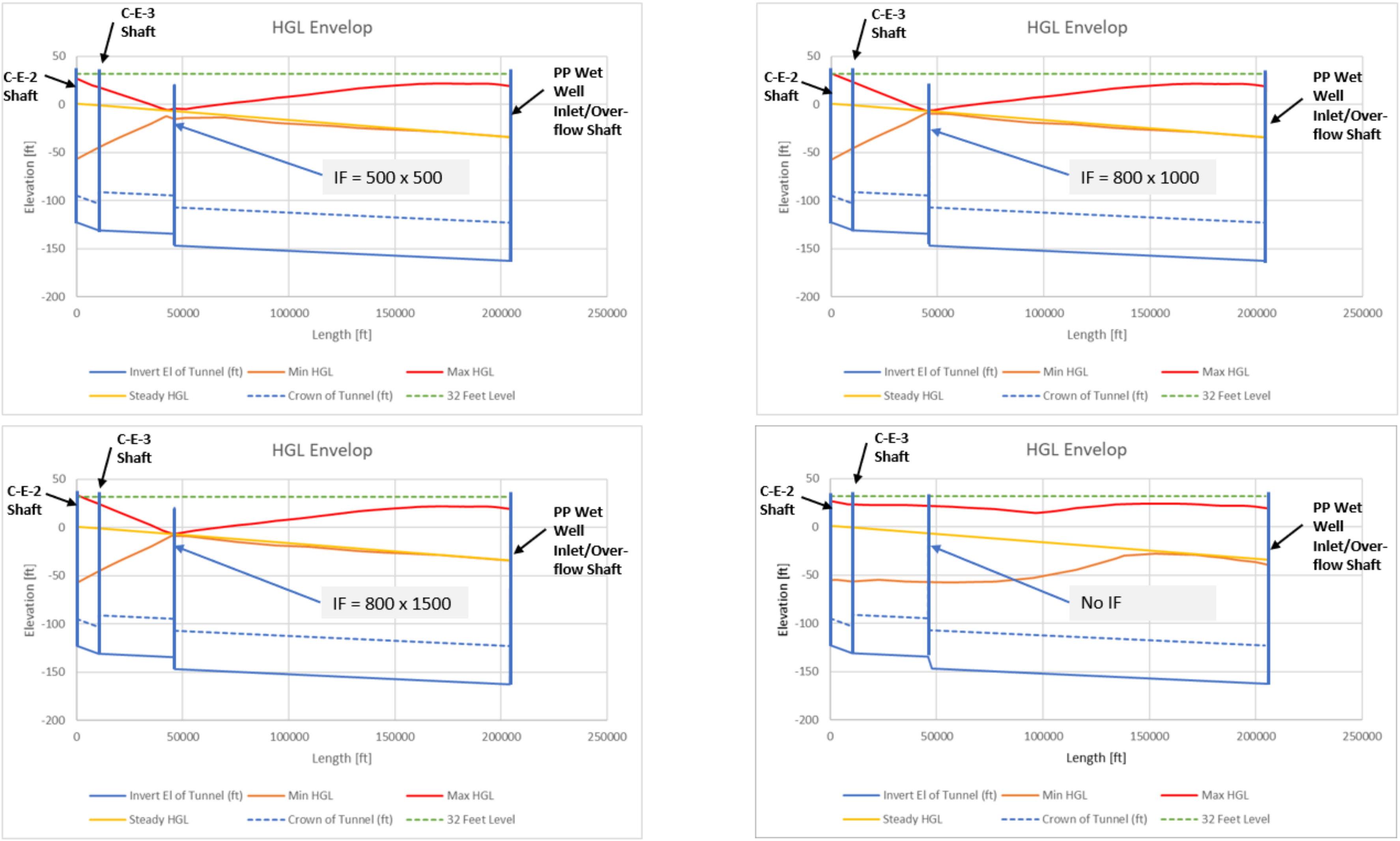

Figure 6. Condition 1, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 6 fps at 6,000 cfs design flow capacity using various IF sizes

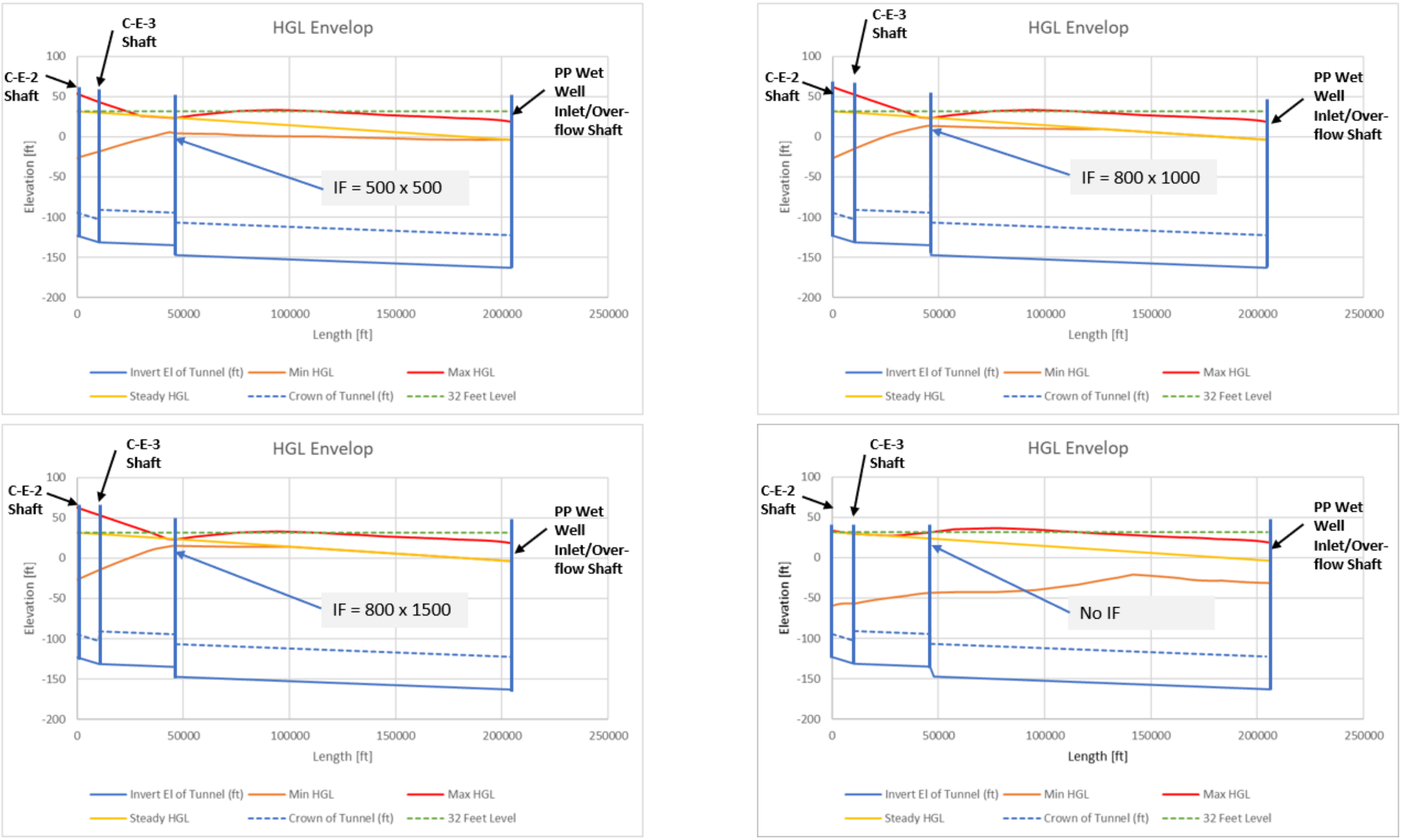

Figure 7. Condition 2, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 6 fps at 6,000 cfs design flow capacity using various IF sizes

1.3.3.3 Condition 3: 33-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Minimum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 8 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 33 feet between the C-E-3 drop shaft and the pumping plant overflow shaft. Prior to the transient-surge condition, the tunnel flow velocity was 7 fps at the design flow condition of 6,000 cfs. To evaluate the lowest resultant minimum transient-surge HGL elevations along the tunnel, the Sacramento River was set to its minimum elevations at C-E-2 and C-E-3.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -98.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft is approximately -70 feet for all IF configurations evaluated and is similar to the Condition 1 results. For the three IF volumes, the minimum HGL was generally equivalent to steady-state conditions for the tunnel section between the IF and pumping plant overflow shaft. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. Based on these results increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing the results of Condition 3 to Condition 1.

The resultant maximum HGL elevation for all IF configurations occurred at the C-E-2 drop shaft. The highest resultant maximum HGL occurred with the IF footprint configuration of 800 feet by 1,500 feet, which was 32 feet. The maximum HGL elevations associated with the IF footprint configurations of 800 feet by 1,000 feet and with the IF removed were slightly below the maximum level determined for the 800-foot by 1,500-foot footprint configuration. For the configuration with the IF removed, the maximum HGL remained generally linear along the entire alignment. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing the results of Condition 3 to Condition 1.

1.3.3.4 Condition 4: 33-foot Tunnel Diameter, Tunnel Flow = 6,000 cfs, Maximum Sacramento River Elevation at C-E-2, Tunnel Flow Velocity = 7.0 fps

Figure 9 shows the results of the maximum and minimum transient-surge HGL elevations with a tunnel diameter of 33 feet between the C-E-3 drop shaft and the pumping plant overflow shaft with the maximum Sacramento River elevations at C-E-2 and C-E-3, and an initial steady-state flow of 6,000 cfs and corresponding flow velocity of 7 fps. This analysis was conducted to evaluate the highest resultant transient-surge HGL elevations along the tunnel at this design flow condition.

For the three IF sizes simulated and with the IF removed, the minimum transient-surge HGL elevations are maintained well above -94.63 feet in the C-E-2 drop shaft and -98.04 feet in the C-E-3 drop shaft, and above the crown of the tunnel along the entire alignment.

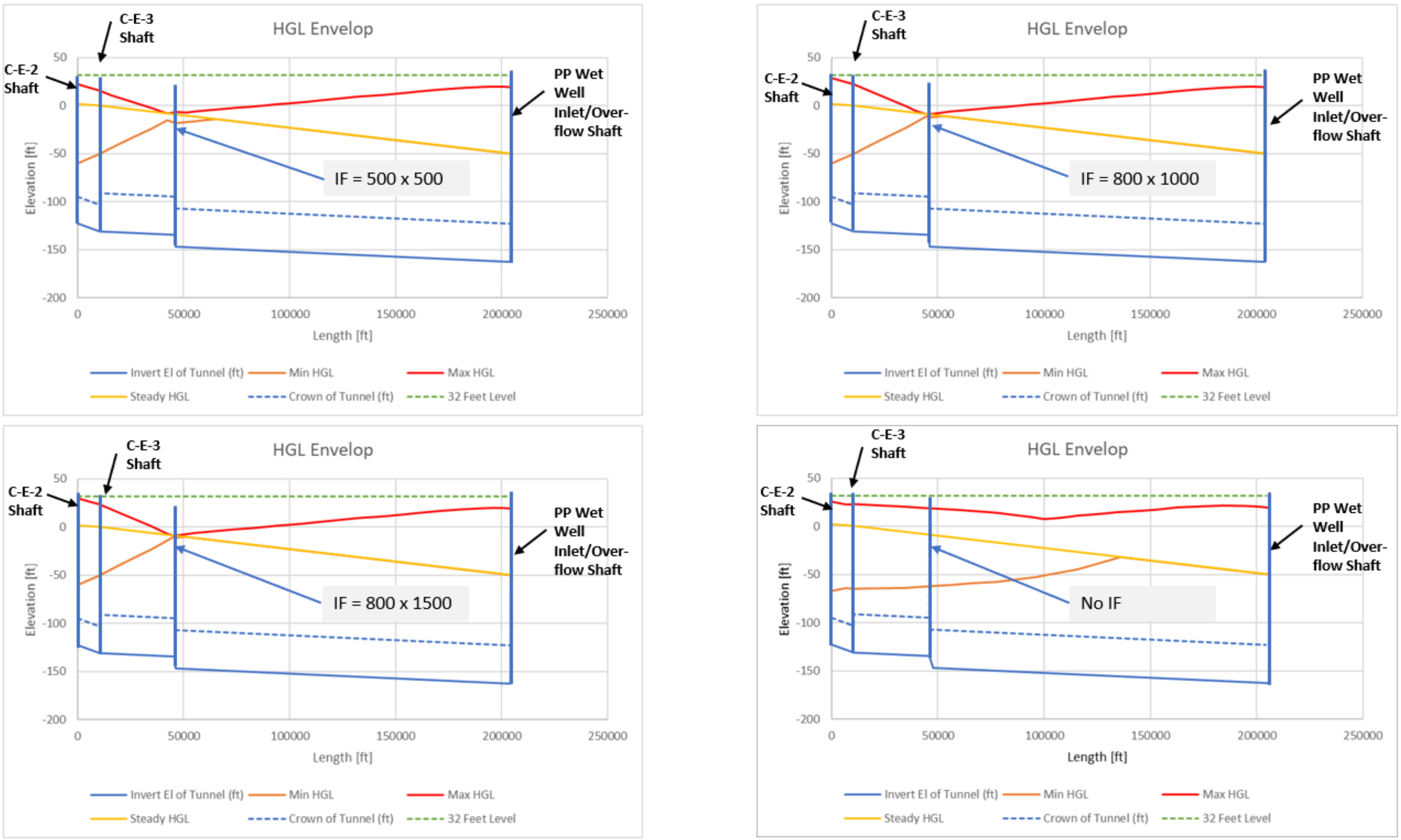

Figure 8. Condition 3, Maximum and Minimum Hydraulic Grade Lines

Note: Graphs show minimum Sacramento River elevation, tunnel flow velocity of 7 fps at 6,000 cfs design flow capacity using various IF sizes

The minimum HGL condition along the alignment for each of the three IF volumes and with the IF removed occurs at the C-E-2 drop shaft. The minimum WSEL within the C-E-2 drop shaft was generally the same for the three IF footprint configurations (approximately -25 feet) and similar to the results in Condition 2. For the tunnel configuration with the IF removed, the WSEL in the C-E-2 drop shaft falls below -50 feet and is about the same as Condition 3. This is primarily due to the higher Sacramento River elevations at C-E-2 and C-E-3 (under Condition 4 versus Condition 3), which caused a larger overflow volume at the pumping plant overflow shaft in Condition 4 than the overflow volume in Condition 3 (with the lower Sacramento River elevations at C-E-2 and C-E-3). The differences in these overflow volumes result in similar minimum WSELs at the C-E-2 drop shaft for both Conditions 3 and 4. No negative pressures were determined to be within the tunnel sections throughout the entire alignment. As such, increasing the flow velocity from 6 to 7 fps had little effect to the minimum HGL when comparing Condition 2 and Condition 4.

The resultant maximum HGL elevations for all IF volumes occur at the C-E-2 drop shaft. The maximum HGL for the IF footprints of 800 feet by 1,500 feet and 800 feet by 1,000 feet was well above 50 feet. The maximum HGL for the IF footprint of 500 feet by 500 feet was slightly below 50 feet. For all IF footprint volumes, the HGL was slightly above 32 feet immediately downstream of the IF location and remained at or just below 32 feet (and well above the steady-state HGL) for the remaining tunnel section between the IF and the pumping plant overflow shaft. With the IF removed, the maximum HGL occurred downstream of the shaft, where the IF was previously located, and did not exceed the steady-state HGL at the C-E-2 drop shaft. For all IF configurations, the maximum HGL profile was generally uniform between the IF and the pumping plant overflow shaft.

Based on the results described for this transient-surge condition, overflow would occur at the C-E-2 and C-E-3 facilities for all IF footprint volumes. Overflow at the C-E-2 or C-E-3 facilities would not occur for the tunnel configuration with the IF removed. Based on these results, increasing the flow velocity from 6 to 7 fps had little effect to the maximum HGL when comparing Condition 2 and Condition 4.

1.3.3.5 Hydraulic Transient-Surge Analysis Summary

Based on the results of the hydraulic transient-surge analysis, the maximum and minimum HGL envelope was found to be within the capability of a segmental concrete tunnel lining.

It was determined that with tunnel configurations that incorporated the three IF footprint volumes of 500 feet by 500 feet, 800 feet by 1,000 feet, and 800 feet by 1,500 feet, an overflow condition would occur at the C-E-2 and C-E-3 drop shafts, along with significantly higher maximum HGL conditions in the northern sections of the tunnel. It was further determined that the IF was not required to mitigate transient-surge conditions within the tunnel.

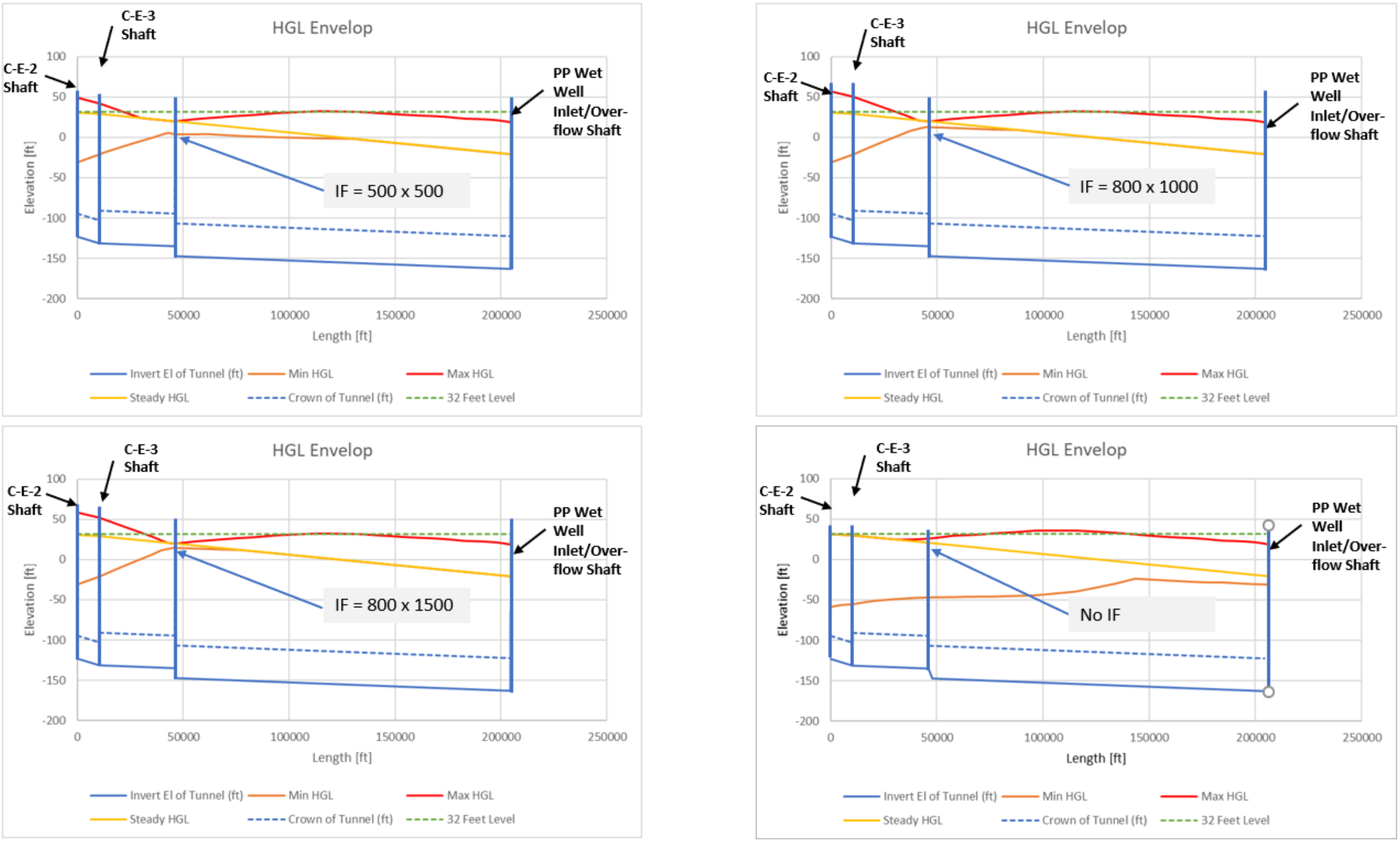

Figure 9. Condition 4, Maximum and Minimum Hydraulic Grade Lines

Notes: Graphs show maximum Sacramento River elevation, tunnel flow velocity of 7 fps at 6,000 cfs design flow capacity using various IF sizes

1.3.4 Screened Candidate Tunnel Diameter Cost Model Analysis

In accordance with the methodology and criteria described, a combined construction and operating cost evaluation was conducted comparing the 44-foot-diameter baseline tunnel against the candidate tunnel diameters sized for the maximum design flow capacity of 6,000 cfs.

To establish the operating cost range, head loss was conducted for the baseline tunnel diameter of 44 feet and for each candidate tunnel diameter under two operating scenarios. These scenarios were:

1) Operating the Project at the specific design flow capacity of 6,000 cfs for 24 hours per day and 365 days per year over a 100-year period to establish the upper highest operating cost value
2) Operating the Project based on the monthly average flows per the DSM2 model runs presented in Appendix 5A, Section C, of the Final EIR/EIS (DWR and Reclamation 2016) over a 100-year period to establish a lower candidate operating cost value

These scenario assumptions result in a sufficiently conservative operating cost analysis range suitable for the conceptual-level analysis. The NPVs of the power costs were calculated for the head loss differences between the 44-foot baseline tunnel diameter and the candidate tunnel diameters at the design flow condition of 6,000 cfs, and the monthly average flows per the DSM2 model runs over a 100-year period.

To establish the cost comparison range (power plus construction cost), the estimated cost of construction and the range of the NPV of the power costs associated with tunnel head loss for each candidate tunnel diameter (including the baseline diameter of 44 feet) were combined. The combined cost range of the baseline case tunnel diameter of 44 feet was then subtracted from the combined cost range of each candidate tunnel diameter to establish the cost difference.

Table 6 summarizes the cost difference comparison results that were developed for each candidate tunnel diameter sized for the maximum design flow capacity. Table 6 lists the candidate diameter, tunnel flow velocity, calculated head loss, estimated construction cost and corresponding power cost associated with the head loss comparison to the base case tunnel diameter (44-foot). Table 6 also lists the design flow capacity and velocity criteria, programmatic cost adjustment, calculated head loss and head loss comparison to the base case tunnel diameter, the NPV cost range comparison and cost difference range for each tunnel diameter considered against the baseline tunnel diameter of 44 feet.

Table 6 summarizes the cost comparison results for candidate tunnel sizes selected for the design flow capacity of 6,000 cfs. The 44-foot-diameter tunnel is within the tunnel velocity design criteria and is shown in Table 4 with zero construction and operating power costs, as it is the baseline tunnel diameter.

For example, the 36-foot-diameter tunnel (which has a flow velocity of 5.9 fps at 6,000 cfs) has a construction cost savings of $1,467,500,000 when compared to the construction cost of the 44-foot-diameter tunnel. The 36-foot-diameter tunnel (at 6,000 cfs) has a calculated head loss of 50.4 feet, which is 29.8 feet higher than the calculated head loss associated with the 44-foot-diameter tunnel. This results in a power cost NPV increase of $366,500,000. At the monthly average flows per the DSM2 model, the increase in operating cost NPVs between the 36-foot- and the 44-foot-diameter tunnel is $37,300,000 (lowest operating cost difference). Subtracting the upper and lower NPVs for the power costs from the construction cost, the cost savings for the 36-foot- versus the 44-foot-diameter tunnel ranges between $1,101,000,000 and $1,430,200,000.

Table 6. Tunnel Construction and Operating Cost Comparisons, Design Flow Capacity 6,000 cfs

Delta Conveyance Tunnel Diameter Evaluation Cost Difference Comparison

Design Flow Capacity	6,000	cfs
Baseline Case Tunnel Dia	44	ft (Note 1)
Minimum Flow Velocity Criteria	3.5	ft/s (Note 2)
Maximum Flow Velocity Criteria	8.0	ft/s (Note 3)

NPV Headloss Power Cost Delta 100 yr period @ 3.0% Discount and $.07 per kW/hr

Programatic Cost Adjustment 1.76

Candidate Tunnel Diameter (feet)	Tunnel Flow Velocity (ft/s) @ 6,000 cfs	Total System-wide Headloss (feet) @ 6,000 cfs	(Note 4) Tunel Construction Cost Savings Compared to Baseline Million ($)	Headloss Diff Compared to Baseline (feet)	NPV Max Headloss Power Cost Diff @ 6,000 cfs 24 hrs/day per year Compared to Baseline Million ($)	(Note 5) NPV Headloss Power Cost Diff @ Monthly Avg Flows per DSM2 Model Compared to Baseline Million ($)	Cost Diff Range Total Construction and Headloss Power Cost Diff @ 6,000 cfs 24 hrs/day per year Compared to Baseline Million ($)	Cost Diff Range Total Construction and Headloss Power Cost Diff @ Monthly Avg per SDM2 Model Results Compared to Baseline Million ($)
46	3.6	17.4	366.8	(3.2)	(39.4)	(7.3)	**327.42**	**359.48**
45	3.8	18.9	183.4	(1.7)	(20.9)	(3.7)	**162.5**	**179.7**
44	3.9	20.6	0.0	0.0	0.0	0.0	**0.0**	**0.0**
43	4.1	22.6	(183.4)	2.0	24.6	2.7	**(158.8)**	**(180.7)**
42	4.3	24.9	(366.8)	4.3	52.9	4.2	**(313.9)**	**(362.6)**
41	4.5	27.6	(550.4)	7.0	86.1	8.6	**(464.3)**	**(541.8)**
40	4.8	30.7	(733.7)	10.1	124.2	14.1	**(609.5)**	**(719.6)**
39	5.0	34.4	(917.1)	13.8	169.7	15.8	**(747.4)**	**(901.3)**
38	5.3	38.8	(1,100.5)	18.2	223.8	21.4	**(876.7)**	**(1,079.1)**
37	5.6	44.0	(1,284.1)	23.4	287.8	30.1	**(996.3)**	**(1,254.0)**
36	5.9	50.4	(1,467.5)	29.8	366.5	37.3	**(1,101.0)**	**(1,430.2)**
35	6.2	58.8	(1,650.9)	38.2	466.1	47.2	**(1,184.8)**	**(1,603.7)**
34	6.6	64.7	(1,834.3)	44.1	542.4	51.0	**(1,291.9)**	**(1,783.3)**
33	7.0	76.1	(2,017.7)	55.6	682.6	75.3	**(1,335.1)**	**(1,942.4)**
32	7.5	86.6	(2,201.2)	66.1	811.7	90.3	**(1,389.5)**	**(2,110.9)**
31	7.9	101.0	(2,384.6)	80.4	988.8	108.4	**(1,395.8)**	**(2,276.2)**

Note 1: Construction cost derived from estimate for 44 foot dia tunnel, central alignment, effect of diameter change extrapolated form partial estimates for 28, 36 and 40 foot diameter tunnels

Note 2: Tunnel flow velocity criteria of 3.5 ft/s established for the 6,000 cfs Firm-Design Flow Capacity per 2018 CER (dual tunnel concept)

Note 3: Tunnel flow velocity criteria of 8.0 ft/s established as the maximum flow velocity for this evaluation

Note 4: Tunnel construction cost delta comparison of the Candidate Tunnel Diameter against the Baseline Tunnel Diameter (44 feet)

Note 5: Average monthly flows from the DSM2 model runs presented in Appendix 5A, Section C, of the 2016 Final Envorinmental Impact Report/Environmental Impact Statement

A construction cost comparison was conducted for the pumping plant facilities associated with a 44-foot- and a 36-foot-diameter tunnel for the design flow capacity of 6,000 cfs. This analysis was conducted to determine whether the construction cost difference associated with a deeper wet well due to the smaller-diameter tunnel developed more head loss and lower wet well operating depths as compared to a 44-foot-diameter tunnel would exceed the construction cost savings for the 36-foot-diameter tunnel, as described. This cost comparison included the programmatic cost adjustment of 1.76.

The results of this analysis indicate that the construction cost increase between the pumping plant facilities associated with the 31-foot- versus the 44-foot-diameter tunnel system is $119,000,000 and is not enough to offset the construction cost savings associated with the 36-foot-diameter tunnel when compared to the 44-foot-diameter tunnel. As such, the results summarized in Table 6 favor constructing a 36-foot-diameter tunnel as compared to the baseline tunnel diameter of 44 feet.

1.4 Conclusions and Recommendations

Based on the results of the hydraulic and capacity analysis of the tunnel diameters options evaluation as described in this Attachment for the Project design flow capacity of 6,000 cfs, the following is recommended for further development as part of the Project's conceptual design:

- The maximum flow velocity criteria should be limited to 6.0 fps.
- The recommended minimum finished ID of the tunnel sections between C-E-3 and the pumping plant wet well is a 36 feet for the design flow capacity of 6,000 cfs.
- The head loss associated with the minimum recommended tunnel diameters is 36.8 feet at the maximum design flow capacity of 6,000 cfs with a 36-foot-inside-diameter tunnel.
- The IF does not provide significant surge mitigation at the maximum recommended tunnel flow velocity of 6 fps or slower. However, the IF facility may be beneficial for operating the system and further hydraulic modeling is recommended to determine if the IF should remain.

1.5 References

California Department of Water Resources (DWR) and Bureau of Reclamation (Reclamation). 2016. *Bay Delta Conservation Plan/California WaterFix Final Environmental Impact Report/Environmental Impact Statement*.

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