APPENDIX E1
IRON GATE HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

TABLE OF CONTENTS

1.0 INTRODUCTION

This appendix includes a summary of data, design methodology, and other information used in the civil, hydrotechnical, geotechnical, and mechanical design of the dam removal operations and structure evaluations at the Iron Gate Hydropower Facility.

Appendix E2 provides a summary of the computational fluid dynamic (CFD) modeling completed to support the design of the tunnel modifications. Drawdown modeling results are included as Appendix G.

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

Material properties have been assessed for existing structures, in-situ soils, and construction materials that will be used for the project. Foundation conditions are discussed in the Geotechnical Data Report (VA103-640/1-2). Technical Specification 31 05 00 Materials for Earthwork provides material specifications for the construction materials that will be used for the project. Gradation curves for the construction materials are provided on Drawings G0050 and G0051.

2.2 DIVERSION TUNNEL EXISTING CONDITIONS

2.2.1 GATE SHAFT AND TUNNEL UPSTREAM REACH

The existing low-level outlet control at Iron Gate Dam consists of a hydraulically actuated, gravity-close, reinforced concrete bulkhead gate. It is installed at the bottom of a 160 ft high shaft and is comprised of two sections of concrete bulkhead, the lower of which has not been moved since original construction. The gate slot and concrete bulkheads close a waterway opening that is horseshoe-shaped and is 15 ft - 6 in wide by 16 ft - 9 in high. A concrete collar and 9 ft diameter blind flange were installed downstream of the control gate during a 2007 construction program, to allow isolation and underwater inspection of the control gate.

Underwater inspection and survey has been completed in the diversion tunnel gate shaft and upstream reach using a Remotely Operated Vehicle (ROV). The following two key observations were presented in the data of the underwater inspection (ASI, 2020):

1. It was found that the gate guides are heavily corroded and, in some portions, slightly misaligned, and that debris are found on top of the gate itself. To facilitate the operation of the gate during drawdown, the gate guides must be cleaned, and debris must be removed from the gate shaft in order to establish a clear travel path for the gate.
2. The tunnel's upstream reach appears in decent condition. Survey of the tunnel's upstream reach has shown minimal sedimentation and no visible anomalies that would point to signs of collapse or damage.

The complete underwater survey package, which includes the underwater inspection video and the final inspection report are provided in Appendix P of the Existing Conditions Assessment Report.

2.2.2 TUNNEL DOWNSTREAM REACH

The downstream reach from the gate to the outlet portal currently features a concrete-lined segment for approximately 90 ft immediately downstream of the gate that transitions to a concrete invert slab for approximately 120 ft of unknown strength and reinforcement. The rest of the tunnel downstream of the gate (approximately 500 ft long) is unlined, except for a 25 ft long concrete-lined segment at the outlet. The existing gate structure offers ventilation in the form of two embedded 8-inch pipes directly downstream of the gate that daylight back into the gate shaft above the water surface.

A detailed survey of the Iron Gate diversion tunnel downstream of the blind flange and 9 ft orifice was conducted by the Yurok Tribe in November 2020 to confirm tunnel dimensions and geometry being used for hydraulic design. Two forms of survey data were collected:

- LiDAR data was collected for portions of the tunnel above the water surface.
- Total station survey was completed to capture the bathymetry, or tunnel invert geometry, below the water surface.

The results of the survey are summarized in a memo titled "Iron Gate Low-Level Outlet Survey Data Acquisition and Processing" (Yurok Tribe, December 11, 2020), which is provided as Attachment 1 to this Appendix.

2.3 TUNNEL MODIFICATIONS

2.3.1 GENERAL

The reservoir drawdown will be facilitated by the operation of the existing diversion tunnel. This section presents the design details pertaining to tunnel modifications. The tunnel modifications occur in two phases - those required to operate the diversion tunnel safely and effectively during reservoir drawdown, known as the Pre-Drawdown Works, and those required for final gate shaft and tunnel closure.

Diversion Tunnel modifications are shown in the Drawing C4100 series outlined in Table 2.1.

Table 2.1 Iron Gate Facility Drawings – Tunnel Modifications

Drawing Number	Drawing Title
C4120	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Best Fit Liner Option
C4121	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Best Fit Liner Option - Profile, Typical Section and Detail
C4122	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Best Fit Liner Option – Typical Sections and Details - (Sheet 1 of 2)
C4123	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Best Fit Liner Option – Typical Sections and Details - (Sheet 2 of 2)
C4124	Iron Gate Facility - Diversion Tunnel Venting – Best Fit Liner Option - Plan and Profile
C4125	Iron Gate Facility - Diversion Tunnel Venting – Best Fit Liner Option - Section and Details
C4130	Iron Gate Facility - Diversion Tunnel – Best Fit Liner Option – Sections (Sheet 1 of 4)
C4131	Iron Gate Facility - Diversion Tunnel – Best Fit Liner Option – Sections (Sheet 2 of 4)
C4132	Iron Gate Facility - Diversion Tunnel – Best Fit Liner Option – Sections (Sheet 3 of 4)
C4133	Iron Gate Facility - Diversion Tunnel – Best Fit Liner Option – Sections (Sheet 4 of 4)
C4170	Iron Gate Facility - Gate Shaft - Closure Plan
C4175	Iron Gate Facility - Tunnel Intake - Closure Plan
C4176	Iron Gate Facility - Tunnel Outlet - Closure Plan
C4190	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Baffled Option
C4191	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Baffled Option - Profile, Typical Section and Detail
C4192	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Baffled Option – Typical Sections
C4193	Iron Gate Facility - Diversion Tunnel Pre-Drawdown Works – Baffled Option – Baffle Details
C4194	Iron Gate Facility - Diversion Tunnel Venting – Baffled Option - Plan and Profile
C4195	Iron Gate Facility - Diversion Tunnel Venting – Baffled Option - Section and Details

2.3.2 PRE-DRAWDOWN WORKS

In operating the existing gate during drawdown at the maximum reservoir water level, the diversion tunnel's downstream reach will be subjected to partially filled conduit flow with a hydraulic jump that fills the tunnel. The reach, upstream of the existing gate, is expected to experience full and pressurized conduit flow during drawdown. The gate location is the point of reference used in defining the diversion tunnel's upstream and downstream reaches as presented in the following sections.

The tunnel modifications for the downstream reach are focused on addressing the differing flow regimes that are expected during drawdown operations, especially at the maximum operational reservoir levels.

The downstream tunnel survey was only available after the Draft 100% design was presented in November 2020 and as such the CFD analysis presented herein provides support for two options for tunnel modifications going forward:

1. The Best Fit Liner Option – modeling to validate the Draft 100% design concept that utilizes 150 ft of new concrete invert and sidewall liner to protect the tunnel from the high velocity flows exiting the existing reinforced modified horseshoe liner.
2. Baffled Option – captures the natural geometric roughness of the bedrock tunnel and adds two new steel wrapped baffles to initiate energy dissipation earlier in the alignment to prevent high velocity flows from developing in the unlined tunnel once the hydraulic jump stabilizes.

Both options are considered viable, and the designs are complete. The baffled option is value alternative made possible by the detailed downstream tunnel survey. CFD simulations of the Iron Gate diversion tunnel discharge included investigation of the effects of tailwater on the hydraulic conditions in the tunnel. The simulation included vent piping on the downstream tunnel crown and tailwater at elevation 2,182.3 ft, corresponding to the discharge condition of the maximum reservoir water level of 2,331.3 ft shown on Table 1 – Drawdown Monthly Inflows and Steady-State Water Surface Levels of Drawing C4055. The CFD simulation results show that discharge through the diversion tunnel, for both modification options, at the maximum reservoir level of 2,331.3 ft is 3,485 ft^3/s.

2.3.2.1 BEST FIT LINER OPTION – HYDRAULIC BEHAVIOR MAXIMUM HEAD

In the simulation, tailwater level is maintained at elevation 2,182.3 ft approximately 100 ft downstream of the tunnel outlet structure, corresponding to the length of the outlet channel. Figure 2.1 shows a plan and elevation view of the diversion tunnel from the CFD model and displays the water surface level and flow velocity contours, after full gate opening. The geometric expansion out of the orifice at full gate opening causes a hydraulic jump to be formed in the tunnel. This hydraulic jump reaches the full tunnel height across a length of approximately 100 ft before being swept out of the tunnel.

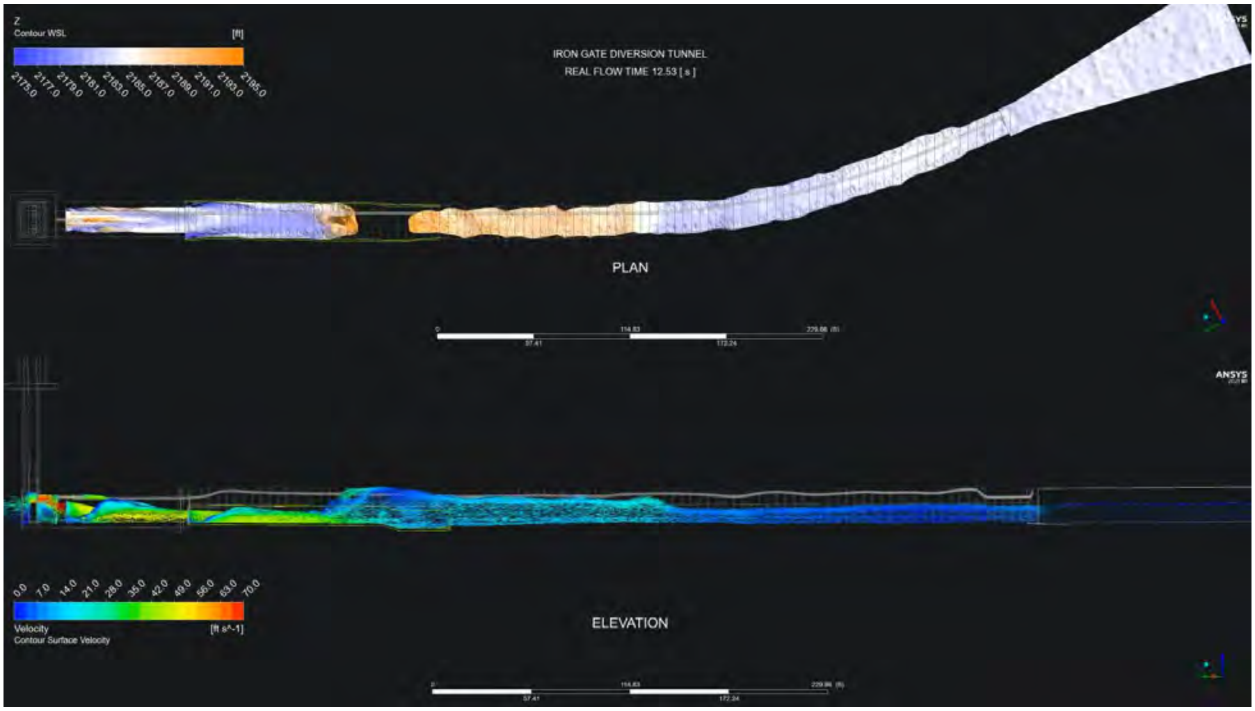


Figure 2.1 Iron Gate Diversion Tunnel CFD – Best Fit Liner Option - Water Surface Profile and Flow Velocity Contours – Initial Stages after Full Gate Opening

Figure 2.2 shows the hydraulic conditions in the diversion tunnel at the full gate opening discharge condition with the maximum reservoir level of 2,331.3 ft. The following conditions were observed in the CFD simulation:

- The plan view shows a ceiling air pocket along the length of the tunnel downstream of the 9 ft diameter flange opening as marked by the water-surface level contours along the length of the tunnel.
- The elevation view shows a hydraulic jump downstream of the existing concrete-lined tunnel. The downstream air vent pipe is simulated to be impacted by the hydraulic jump and by the flow moving downstream of it. Localized peak flow velocities greater than 30 ft/s contact the downstream air vent pipe at two locations:
    - o Hydraulic jump location in the tunnel at the downstream end of the best-fit liner.
    - o Flow constriction at the tunnel outlet structure.
- With constant tailwater, a hydraulic jump forms downstream of the tunnel in the outlet channel.

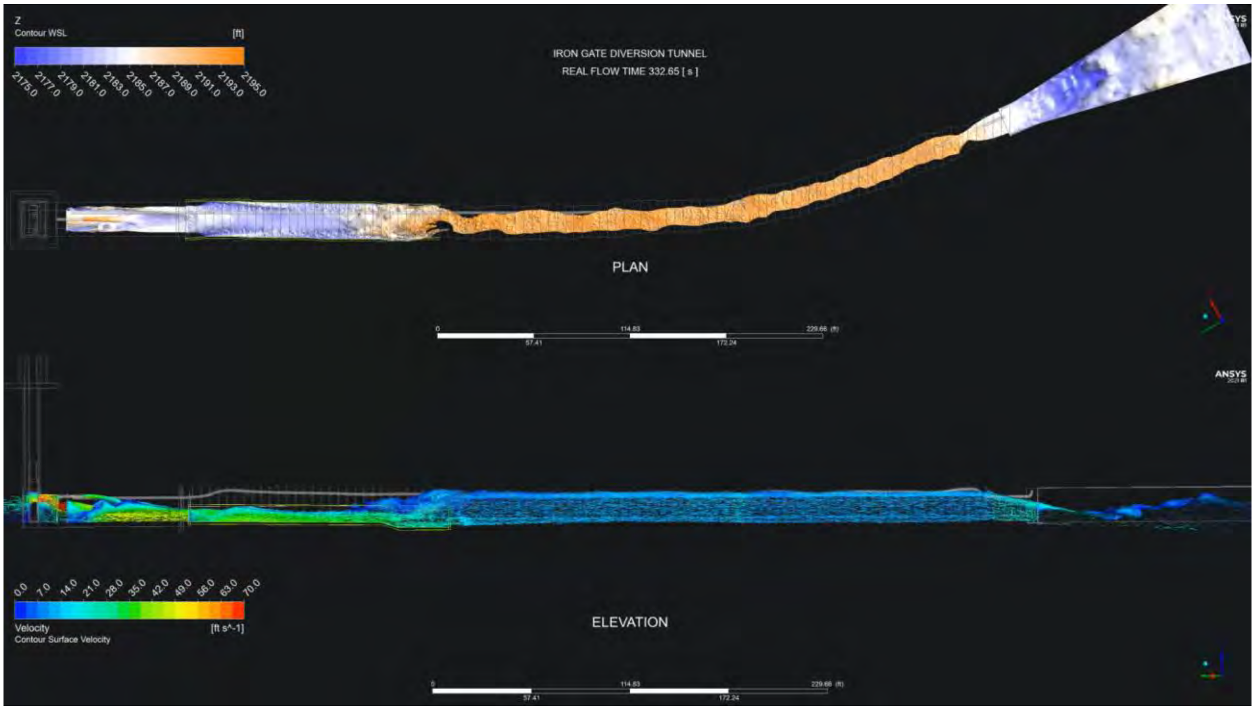


Figure 2.2 Iron Gate Diversion Tunnel CFD – Best Fit Liner Option – Water Surface Profile and Flow Velocity Contours –at Maximum Reservoir Level 2,331.3 ft

Proposed tunnel modifications to facilitate the hydraulic conditions observed in the CFD simulations and analysis above are discussed in the following section.

2.3.2.2 BEST FIT LINER OPTION - CONCRETE LINER DESIGN

A reinforced concrete liner for the tunnel side walls and invert will extend approximately 150 ft downstream of the existing 9 ft diameter flange and concrete-lined segment of the tunnel (i.e. downstream of the grout curtain) as a protective measure against scour due to high velocity flows.

The current structural design of the concrete liner under hydrostatic loading is based on the grout curtain being functional and performing effectively, limiting external hydrostatic pressure. The observations made during the visual inspections of the tunnel support this conclusion. The ground water drains downstream of the grout curtain through the jointed tunnel rock face. The liner design includes drain holes placed at regular intervals at the base of the side walls. Frequent drain holes, every 8 ft, will provide redundancy to drainage in the event of individual holes experiencing debris blockage.

Considering the high flow velocities that are expected during drawdown, especially at high reservoir water surface levels, embedded PVC pipes centered at the location of the drain holes will be installed. This will allow adverse negative pressures due to the high flow velocities to be directed through the PVC pipes thereby preventing jacking pressures from developing behind the liner while at the same time allowing the drain holes to relieve groundwater pressure. Figure 2.3 shows the general arrangement and function of the drain holes; details are shown on Drawing C4122.

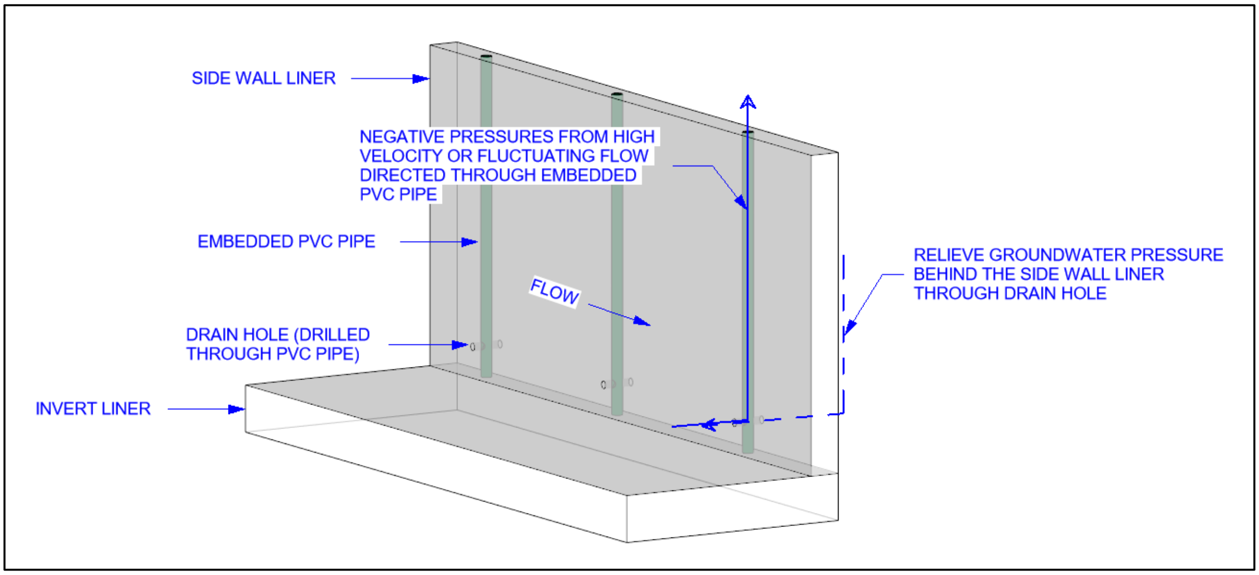


Figure 2.3 Side Wall Liner – Drain Hole and Embedded PVC Pipe Arrangement

The hydrodynamic conditions are the primary consideration in designing the liner to mitigate the risk of damage and erosion to unlined tunnel walls to achieve a reliable hydraulic cross section and maintain the integrity of the bedrock. The following table summarizes the loads considered in the design.

Table 2.2 Concrete Liner Design – Summary of Loads

Description	Estimated Input Value	Treatment
Hydrostatic – Groundwater	20 ft of head	Drainage holes in liner walls
Hydrodynamic – Interface/bed shear stress due to high velocity	Flow velocities of 20 to 70 ft/sec	Side wall includes rock dowels and liner reinforcement
Hydrodynamic – Uplift due to pressures from high velocity flow and joint irregularities	Negligible[1]	Slab joint details to have strict tolerances on spacing and vertical offset
Hydrodynamic – impact and thrust at locations of high turbulence	Varies	Side wall includes rock dowels and liner reinforcement

2.3.2.3 BAFFLED OPTION – HYDRAULIC BEHAVIOR AT MAXIMUM HEAD

The tunnel is currently partially lined with unknown reinforcement characteristics or completely unlined for approximately 475 ft downstream of the reinforced horseshoe liner downstream of the gate. Without any new concrete invert or sidewall liner, CFD analysis of the existing tunnel conditions show that the hydraulic jump that forms in the tunnel stabilizes just downstream of the existing concrete-lined tunnel. This is notably further upstream than the location of the hydraulic jump when the new concrete liner is constructed which eliminates the natural geometric roughness of the tunnel.

It was observed in the CFD simulations, once the detailed survey was incorporated, that the tunnel has adequate energy-dissipating capacity to induce a hydraulic jump, thus subjecting majority of the unlined portion of the tunnel to low velocity flows. Given the proximity of the hydraulic jump to the existing heavily reinforced lined section of the tunnel, it is proposed in the baffled option that the new concrete liner be eliminated by pulling the jump further upstream with the addition of baffles inside the existing reinforced concrete liner so that the rough partially lined portion of the tunnel is no longer exposed to high energy flow for an extended period of time.

Figure 2.4 presents the plan and elevation view of the tunnel with the baffles added, showing the hydraulic jump having travelled further upstream and stabilized inside the existing reinforced concrete liner.

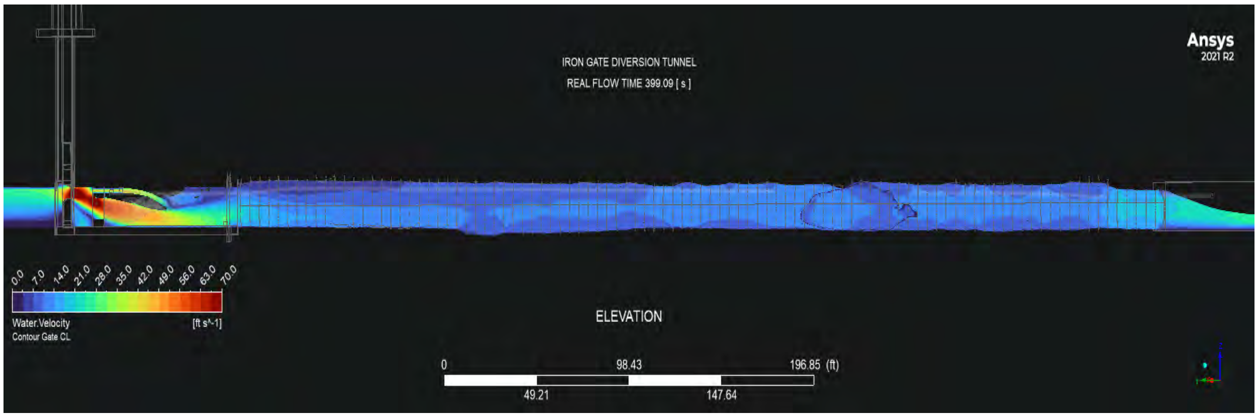


Figure 2.4 Iron Gate Diversion Tunnel CFD – Baffled Option – Water Surface Profile - Maximum Reservoir Level 2,331.3 ft

As shown above, with the combination of the unlined geometry and the use of baffles, the unlined portion of the tunnel downstream of the existing liner and grout curtain sees maximum flow velocities of 10 to 15 ft/s once the hydraulic jump has stabilized. These lower velocities facilitate the elimination of the new concrete liner if upstream baffles are constructed in lieu. Figure 2.5 shows the location of the existing liner, the proposed baffle location and the beginning of the unlined portion of the tunnel.

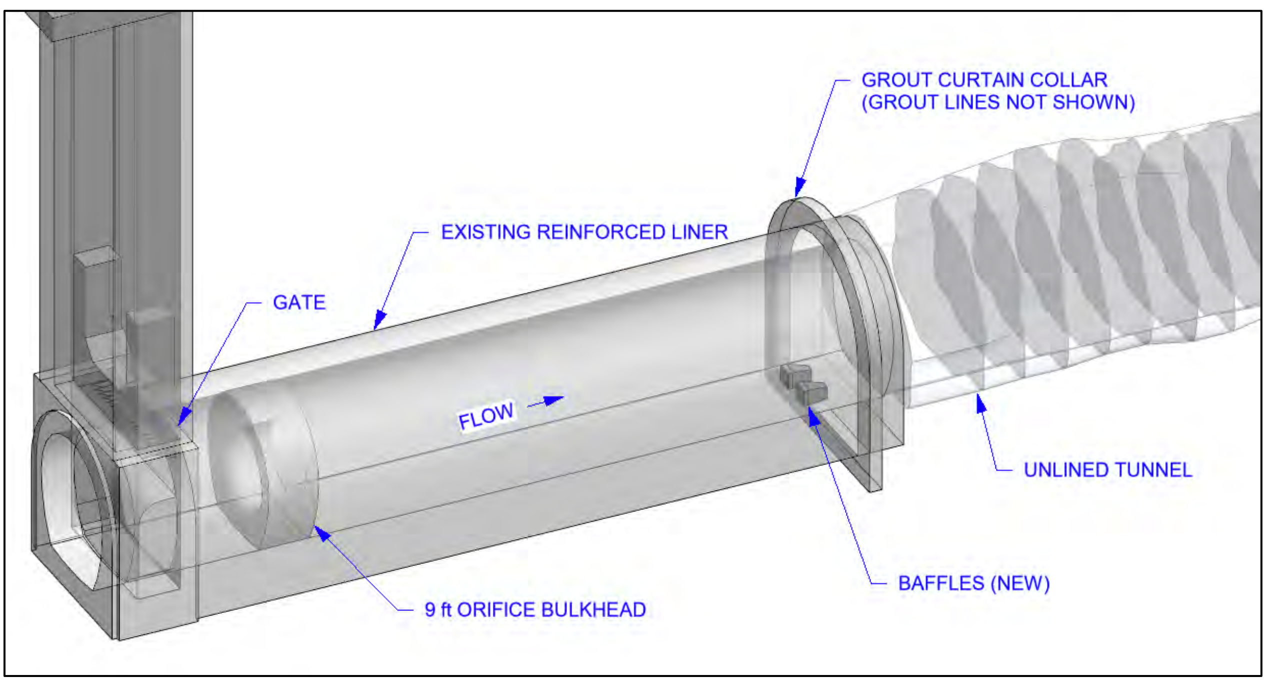


Figure 2.5 Modelled Baffle Location

2.3.2.4 BAFFLED OPTION – BAFFLE DESIGN

The baffles are designed to withstand the maximum hydrodynamic force associated with the fast-moving flow of 70 ft/sec coming from the Ø9 ft orifice. The baffle design incorporates the following features:

1. Cast-in-place reinforced concrete block with a 2'x2' frontal area, 3' long that tapers along its length. This tapered shape is based on the studies by USACE and USBR (2009); it serves to mitigate against cavitation-induced damage. The baffle block shall be installed in a 2" deep notch cut into the existing concrete-lined tunnel invert to offer improved bearing and stability.
2. Steel plate facing at the sides of the baffle for protection against concrete spalling.
3. Post-tensioned threaded dowels anchored into rock to resist uplift, stagnation pressures and improve stability.

The final design of the baffled option is presented in C4190 drawing series. The details of the proposed baffles are shown on the attached Drawing C4192.

2.3.3 PRE-DRAWDOWN WORKS - VENTILATION DESIGN

2.3.3.1 GENERAL

Ventilation design for the downstream reach is based on analysis of the hydraulic characteristics and air demand estimated using CFD modeling. The applicable software is ANSYS® Fluent v20. The main findings that support the ventilation design are summarized as follows:

- The existing gate structure ventilation, two embedded 8-inch pipes directly downstream of the gate that daylight back into the gate shaft above the water surface, is expected to be insufficient and at risk of being swamped by the high discharge flows required to achieve drawdown.
- At high flows, the flow regime in the tunnel downstream reach consists of open channel supercritical flow that transitions into a hydraulic jump – the conjugate depth of the supercritical flow in the tunnel limits the available opening at the outlet which may be counted upon to provide means of venting thereby rendering airflow from the outlet unreliable. Figure 2.4 shows the flow conditions inside the downstream reach at the maximum operating reservoir level before the hydraulic jump has stabilized.

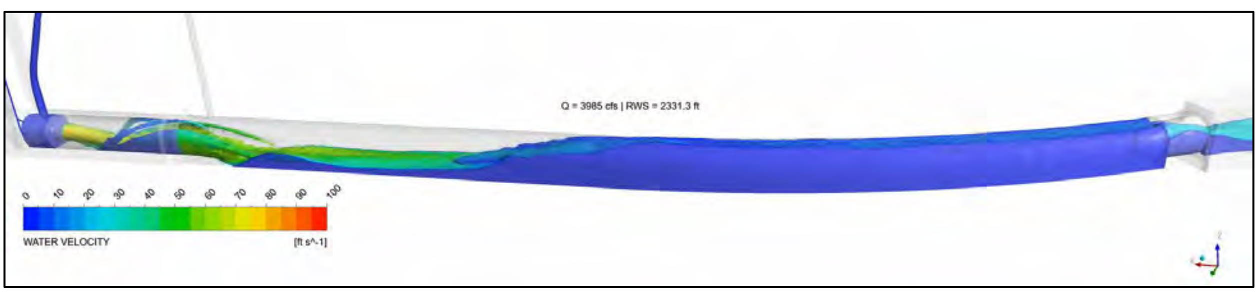


Figure 2.6 Hydraulic Jump Forming Inside Tunnel

- There are two zones inside the tunnel downstream reach that require venting to establish proper air flow and maintain stable hydraulic discharge behavior under varying gate opening and upstream water surface conditions, shown in Figure 2.5.

- o Zone 1: Vent between the existing gate and the 9 ft diameter orifice, approximately 20 ft downstream of the gate.
- o Zone 2: Vent downstream of the 9 ft diameter orifice.

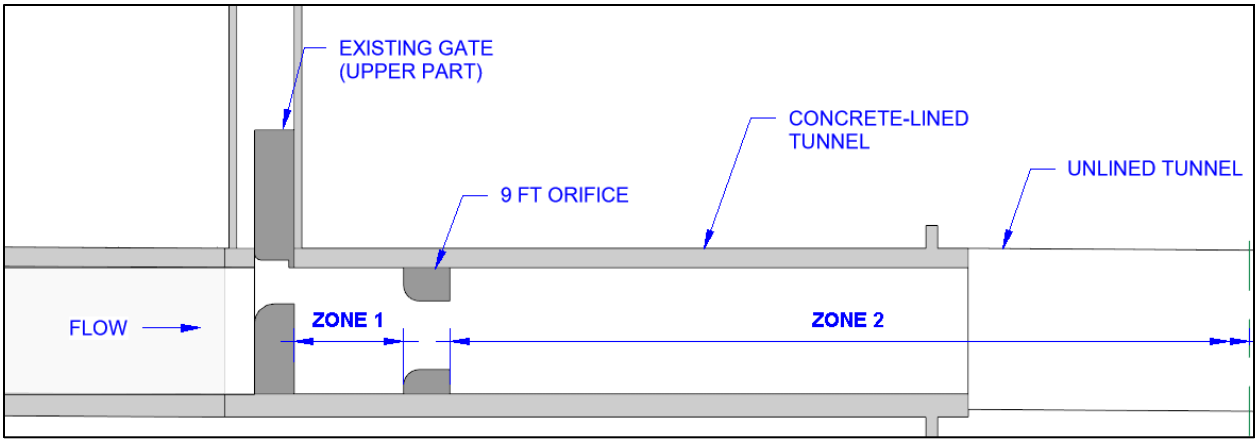


Figure 2.7 Areas Inside Tunnel Targeted by the Venting – Tunnel Elevation View

Two options for providing the additional venting were considered:

- By way of vertical vent holes drilled through the embankment.
- By suspending vent pipes from the crown of the tunnel.

CFD analyses have shown that both options provide an efficient means of ventilation. However, the option of suspending the vent pipes from the crown was selected as the optimum solution and was carried forward in the design.

Zone 1 is vented by a 2 ft diameter opening drilled at the upper right (looking downstream) quadrant of the 9 ft orifice concrete collar.

Zone 2 is vented by a 2 ft diameter solid wall HDPE pipe suspended from the tunnel crown, labeled in the design drawings as the downstream vent pipe. The downstream vent pipe is located at the upper left (looking downstream) quadrant of the tunnel centerline and extends from the downstream face of the concrete orifice all the way to the tunnel outlet portal.

The civil design of the downstream vent pipe and its anchoring into bedrock accounts for exposure to hydrodynamic loads including the following:

- Floatation in areas where the tunnel will flow full during high outflows
- Drag/friction due to flow velocity where the tunnel will flow full
- External lateral thrust loads where the tunnel and ventilation round a sweeping bend

Details of the proposed vent design are shown on Drawings C4120 to C4125 and additional design information is provided in the sections below.

2.3.3.2 VENT PIPE ALIGNMENT

The following describes the selected vent pipe alignment after completing CFD modelling to optimize the design:

1. The downstream vent pipe is aligned to be located on the ceiling at the left (looking downstream) quadrant of the tunnel centerline. The tunnel LiDAR survey indicates less tunnel wall irregularity and rock protrusion as compared to the right side which makes this alignment conducive for installing the downstream vent pipe. The geometric alignment of the vent pipe features a straight and curved segment in the horizontal plane. Vertical clearance between the vent pipe and the tunnel crown is targeted to not exceed 2 ft.
2. 45° pipe elbows for the downstream vent pipe are specified at locations of transition in tunnel geometry to bring the vent pipe as close to the tunnel crown as possible:
    a. Ceiling transition from the existing concrete-lined tunnel to the unlined tunnel.
    b. Tunnel crown overbreak section just upstream of the tunnel outlet concrete structure.
    c. Ceiling transition from the unlined tunnel to the outlet structure.
3. 90° pipe elbow at the tunnel and downstream vent pipe outlet to extend the vent pipe above elevation 2,192.5 ft.

2.3.3.3 ANALYSIS AND DESIGN OF DOWNSTREAM VENT PIPE SUPPORTS

Evaluation of the vent pipe supports was completed to ensure that flow-induced vibration of the vent pipe or the vent pipe vertical anchors does not result in structural damage. Calculations were made to determine the forcing frequency from crossflow-induced vibrations on the vertical anchors and longitudinal flow-induced vibrations on the empty vent pipe with the structural natural frequency of the downstream vent pipe configuration. A factor of safety is required to provide a suitable margin of safety between the structural natural frequency and the forcing frequency that could cause structural damage from dynamic stresses induced by vibration. The following analyses were performed to support the vibration assessment:

1. Vibration analysis of the flow around anchors based on:
    a. Structural natural frequency analysis of the vent pipe anchors
    b. Forcing frequency based on vortex shedding by Strouhal number method
2. Vibration analysis of the flow around the vent pipe based on:
    a. Structural natural frequency analysis of the vent pipe (flow moving transverse to the vent pipe axis)
    b. Forcing frequency based on vortex shedding by the Strouhal number method

Structural natural frequency is calculated on a system that consists of the structural mass where the driving force is applied. The first mode of frequency is assumed to produce the highest intensity vibrations in the absence of damping. The structural mass elements are described in Table 2.3:

Table 2.3 Structural Mass System

Structural Mass	Driving Force	Single Mass	Degree of Constraint
System 1 – Anchor rods (1/2-inch diameter)	Flow around anchor rods	Uniformly distributed, modulus of elasticity, moment of inertia and mass	Fixed end at rock interface, simple support on the vent pipe clamp connection
System 2 – Vent pipe	Flow around vent pipe (transverse to vent pipe axis)	Resultant mass of vent pipe in a span between two hanger locations	Two anchor rods per pipe hanger, fixed end at the rock interface and unrestrained on the clamp connection

The forcing frequency is calculated based on flow-induced vibration on the anchor rod or the vent pipe. The frequency calculations were performed following the method by USBR (1981) developed for the design of trashracks in hydraulic structures. The development of resonant frequency will not occur or will be minimized when the forcing frequency and structural natural frequencies differ by a factor of 2.5. This method of assessing flow-induced vibrations was selected given that the hydraulic conditions on trashracks is anticipated to be similar to the flow around the vent pipe and its vertical anchors.

Table 2.4 shows the calculated forcing and structural natural frequencies. The forcing frequency will be sufficiently low with structural natural frequencies exceeded by more than 2.5 times forcing frequency.

Table 2.4 Downstream Vent Pipe – Vibration Analysis Results

	Forcing Frequency [1] (Hz)	Structural Natural Frequency (Hz)	Factor
System 1 – Flow around anchor rods	38.7	102.8	2.6
System 2 – Flow around vent pipe (transverse to vent axis)	2.0	6.9	3.4

NOTES:

1. FORCING FREQUENCY IS CALCULATED USING A MEAN FLOW VELOCITY OF 20 FT/S AS OBSERVED IN THE CFD SIMULATION.

Structural analysis and design of the vent pipe anchor rods was conducted according to AISC 360-16 Load and Resistance Factor Design (LRFD) with the following load cases and design resistances:

1. Steady state load cases:
   a. Drag force around pipe
   b. Drag force around anchor rod
   c. Buoyant force at vent pipe
2. Transient load cases:
   a. Drag force of hydraulic jump impacting the vent pipe
   b. Air pressure force at the inlet or outlet of the pipe
3. Design resistance of the anchor rods (#8 ASTM A615 Grade 75 Dywidag Threadbar)
   a. Compression
   b. Flexure
   c. Shear

The load cases that govern the design of the downstream vent pipe supports and the respective resistances that result from the updated anchor support design are shown in Table 2.5.

Table 2.5 Downstream Vent Pipe Support – LRFD Loading Condition

Load Case	Factored Applied Force or Moment per Anchor Rod [1]	Factored Anchor Rod Resistance [2]
Buoyant force at vent pipe	3,529 lbf (compressive force on anchor rod)	4,278 lbf
Drag force around vent pipe and around anchor rods	1,463 lbf-ft (moment on anchor rod)	1,766 lbf-ft
Air pressure at the inlet of the vent pipe	582 lbf-ft (moment on anchor rods)	1,766 lbf-ft
Drag force of hydraulic jump impacting the vent pipe	3,143 lbf (compressive force on anchor rod)	4,278 lbf

NOTES:

1. LOAD FACTORS OF 1.5 ARE APPLIED TO HYDRODYNAMIC FORCE EFFECTS TO ACCOUNT FOR VARIATIONS OF THE FORCE DURING FREQUENCY AND MODES OF VIBRATION.
2. MATERIAL RESISTANCE FACTOR OF THE ANCHOR RODS ARE BASED ON AISC 360-16 (LRFD).

As a result of the structural analysis, the downstream vent pipe hanger configuration was updated as described below:

1. Target clearance from tunnel crown to the vent pipe is limited to 2 ft.
2. Areas of high hydrodynamic loading are specified with more frequent vent pipe hanger spacing, including:
   a. From the vent pipe inlet to the downstream extent of the existing concrete-lined tunnel; close hanger spacing is required in this region due to high air pressure loading and the occurrence of the initial wave formation at the time of gate opening.
   b. In the vicinity of the sustained hydraulic jump in the tunnel.
   c. From the downstream end of the bend in the vent pipe to the upstream extent of the concrete-lined tunnel outlet structure.
3. Other sections where flowing water is acting in the longitudinal direction of the vent pipe, the spacing of the hangers are as follows:
   a. 12 ft typical spacing in the straight segment of the vent pipe.
   b. 8 ft typical spacing in the bent segment of the vent pipe.

2.4 CHANNEL PROTECTION

2.4.1 DRAWDOWN

At high flows, energy dissipation is achieved inside the tunnel due to the formation of the hydraulic jump. As such, velocities in the tailrace area of the existing powerhouse are considered within a safe range to eliminate the need to riprap the powerhouse prior to final grading.

Upstream of the tunnel intake, local areas of erosion or scour are not a concern due to the presence of riprap on the upstream dam face and the bedrock abutment.

2.4.2 POST DRAWDOWN CHANNEL FINAL GRADING

The details for the channel's final grading, including disposal site locations, are shown on Drawings C4210 to C4212 and are outlined below.

Table 2.6 Iron Gate Facility Drawings – Channel Final Grading

Drawing Number	Drawing Title
C4210	Iron Gate Facility - Embankment Removal - Grading General Arrangement Plan
C4211	Iron Gate Facility - Embankment Removal - Grading General Arrangement Sections
C4212	Iron Gate Facility - Embankment Removal - Grading Channel Profile, Section and Detail

A significant amount of the final river channel at the Iron Gate dam site is expected to be excavated to bedrock. Erosion protection is not required on bedrock.

Stable cut slopes are shown on the drawings listed in the table above.

2.4.2.1 EROSION PROTECTION DESIGN

Erosion protection is designed for regions of the final Iron Gate Channel where channel and embankment excavation may not be to bedrock or where the fill slopes at the spillway and powerhouse disposal areas are expected to contact the banks of the river. Erosion protection for the final river channel at Iron Gate is designed for the post-dam removal 1% flood event with 3 ft of freeboard. Final channel characteristics and geometry were used to develop a HEC-RAS 2D model, which produced a velocity profile and water surface elevation. The hydraulics of the final channel were modelled to determine the required erosion protection.

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

The depth-area-capacity relationships for the Iron Gate reservoir are based on the 2018 bathymetric survey (NAVD88 datum) and are shown on Drawing C4050. The reservoir capacity at elevations relevant to the Iron Gate facility are summarized in Table 3.1.

Table 3.1 Reservoir Storage Capacity for Various Elevations

Key Elevation Description	Elevation (ft)	Capacity (acre-ft)
Spillway Crest	2,331.3	54,714
Normal Maximum Operating Level	2,331.3	54,714
Normal Minimum Operating Level	2,327.3	50,414
Power Intake Invert	2,295.0	26,713
Extended Cofferdam Crest	2,231.2	3,844
Historic Cofferdam Crest	2,212.0	1,185
Breach Plug Initial Crest	2,202.0	465

3.2 OUTLET STRUCTURE RATING CURVES

Discharges during drawdown will be made through the modified diversion tunnel using the existing outlet control gate, the existing power intake and turbine/bypass, and spillway releases. Details of the development of the discharge rating capacities using CFD modeling are presented in Appendix E2. The Iron Gate discharge rating curves are presented on Drawing C4050.

The CFD model developed for the diversion channel and existing outlet control gate indicates that the maximum capacity is approximately 4,000 cfs (Appendix E2).

3.3 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN

The reservoir drawdown will be completed utilizing the spillway, power intake and turbine/bypass, and the existing outlet control gate in the diversion tunnel. The drawdown model (detailed in Appendix G) is developed to assess the drawdown sequencing in terms of reservoir water surface levels under a range of hydrologic conditions.

Reservoir water surface levels are simulated in the drawdown model (Appendix G) for the full record of inflows available for the 2019 Biological Opinion (2019 BiOp) dataset. The 2019 BiOp flows are available for 36 years, from October 1980 through September 2016. The results of the drawdown model are summarized in three ways:

- Individual year simulations are provided in the attached Iron Gate Simulated Drawdown Figures 1 through 36 (Attachment 2). These plots indicate the following:
    - o Reservoir water surface levels.
    - o Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway, power intake and bypass, and flows through the diversion tunnel).
- Maximum daily reservoir water surface level daily non-exceedance percentiles (percentiles) of reservoir water surface levels from all model runs are shown on Figure 3.1, and on Drawing C4050. This figure represents the results from all 36 model simulations as non-exceedance percentiles to summarize the distribution of the results on any given day of the simulations. These results do not represent a single simulation, but are based on all model simulations.
- Ensemble figures with each line representing a single model simulation for a different year, (also referred to as spaghetti figures) are shown on Figure 3.2. This figure overlaps the simulated reservoir water surface levels on a common x-axis that spans January 1 to September 30. Each line represents a single model simulation.

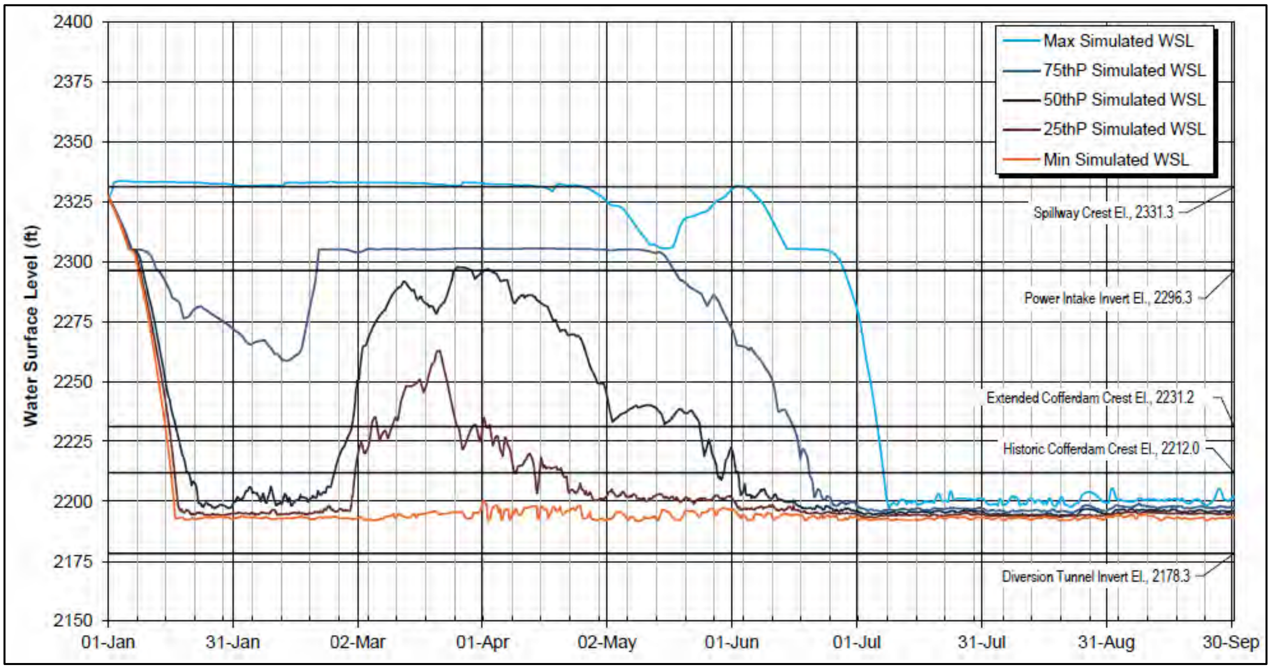


Figure 3.1 Iron Gate Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

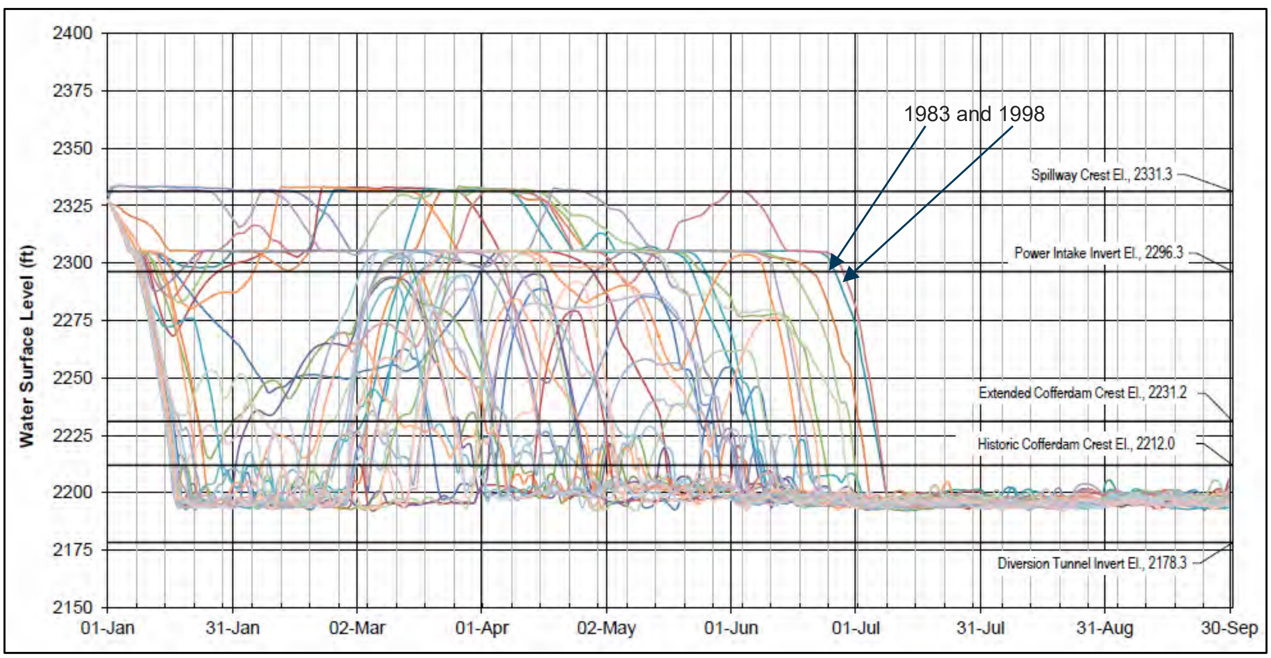


Figure 3.2 Iron Gate Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

The simulated water surface levels on Figure 3.1 show that the reservoir water levels drop below the crest of the historic cofferdam in mid-June for the 75th percentile, while the remaining model simulations achieve a lowered reservoir water level in early July. There are three model years (1983, 1984 and 1998) indicated on Figure 3.2 that show elevated reservoir water surface levels past July 1. In these years, the reservoir water surface levels drop below the crest of the historic cofferdam by July 8.

Figure 3.2 shows that many of the model simulations achieve reservoir drawdown in January, however, the reservoir refills in the higher flow months of February through May. There can be large fluctuations in the reservoir water surface levels from March through June. Spillway flows are observed after January for 31% of the simulations, and for power intake and bypass valve for an additional 36% of the simulations. The reservoir water surface level does not rise above the power intake invert in the remaining 33% of the simulations.

The reservoir water surface level can rapidly rise in March, April and May resulting from large inflow events. Examples of this are seen in simulation years 1981, 1989, and 1993, where the reservoir water surface level was below the historic cofferdam crest in January but then rapidly rose in response to the high inflows. These inflows may be a function of required flushing flows from the upstream irrigation project as described in USBR (2018), or, are influenced by the flows from unregulated tributaries entering the Iron Gate reservoir, such as Jenny Creek.

Figure 3.3 shows reservoir drawdown distributions for various relevant facility components or dam removal sequences. These distributions represent cumulative percentages of model simulations indicating the dates when the reservoir water surface level is lower and sustained below a certain elevation. The actual date when the water surface elevation will be sustained in the drawdown year can be different than shown on Figure 3.3. depending on the hydrological conditions and the drawdown sequencing applied. The water levels shown on Figure 3.3 are based on average daily conditions for the 36 drawdown model simulations. Low probability flood flows (e.g., the 5% or 1% probable flood flows) may not have occurred within this period, and may not be reflected in these drawdown distributions. Occurrence of such events may shift the distributions to a later date. The following observations are made based on Figure 3.3:

- Elevation 2,331.3 ft - represents the spillway crest. Approximately 70% of the drawdown simulations have reservoir water levels sustained below the spillway crest by January 2, 80% of the simulations by April 1, and 100% of the simulations by June 5. To meet the 1% criteria, the embankment crest will not be brought below 2,331.3 ft until after June 16 of the drawdown year.
- Elevation 2,308.6 ft – represents the start of Sequence 3 of the dam removal. Approximately 70% of the simulations have reservoir water levels sustained below this elevation by January 6, 80% of the simulations by April 22, 90% by May 2, and 100% by June 13. To meet the 1% criteria, the embankment crest will not be brought below 2,308.6 ft until after June 16 of the drawdown year.
- Elevation 2,308.1 ft – represents the start of Sequence 4 of the dam removal. The timing is essentially the same as for Sequence 3 shown above. To meet the 1% criteria, the embankment crest will not be brought below 2,308.1 ft until after July 1 of the drawdown year.
- Elevation 2,296.3 ft – represents the power intake invert. Approximately 33% of the simulations have reservoir water levels sustained below this elevation by January 14, 50% of the simulations by April 7, 80% by May 26, and 100% by June 28. To meet the 1% criteria, the embankment crest will not be brought below 2,296.3 ft until after July 1 of the drawdown year.

- Elevation 2,231.2 ft – represents the crest of the Extended Cofferdam, which protects against the 1% probable flood between July 16 and the end of August, and against the 5% probable flood until September 15 with minimum 3 ft freeboard. Approximately 40% of the simulations have reservoir water levels sustained below this elevation by May 3, 70% by June 9, and 100% by July 6. The final stages of dam removal can start after July 6 with the completion of works by September 15. To meet the 1% criteria for the lowest flow months, the embankment crest will not be brought below 2,231.2 ft until after July 16 of the drawdown year.
- Elevation 2,212 ft – represents the crest of the Historic Cofferdam. Approximately 20% of the simulations have reservoir water levels sustained below this elevation by May 3, 40% of the simulations by June 1, and 100% by July 8. The preparatory works for the final dam breach including excavation and riprapping of the final breach channel can be initiated after this date followed by the period with the lowest reservoir levels.
- Elevation 2,202 ft – represents the crest of the final breach Plug. Approximately 50% of the simulations have reservoir water levels sustained below this elevation by June 13, 80% of the simulations by July 3, and 100% by September 28. Once all preparatory works are complete for the final breach and reservoir release, the final downcutting to the breach plug can be initiated. After August 16 and following the preparation of the final breach channel and breach plug, the reservoir will be breached through the plug crest at El. 2,202 ft at a water surface at or below 2,201 ft.

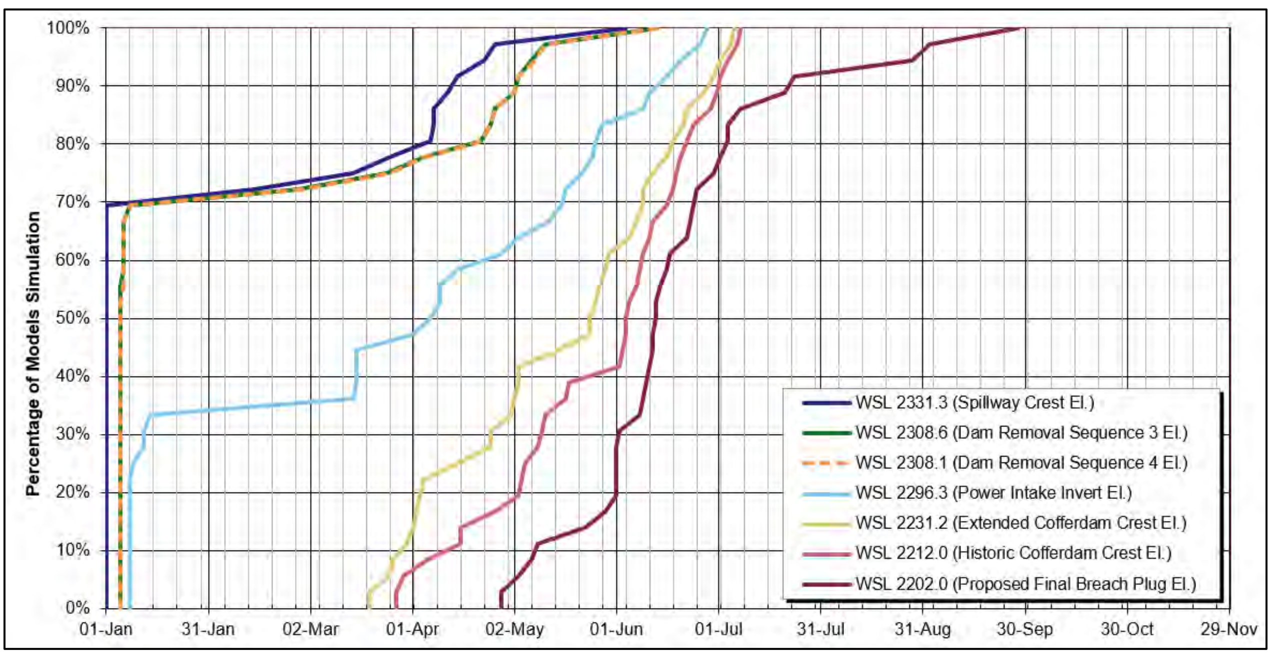


Figure 3.3 Iron Gate Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below the Crest of the Historic Cofferdam

3.3.1.1 POST-DRAWDOWN RIVER DIVERSION

River diversion is achieved when all the inflows are passed through the diversion tunnel with negligible attenuation in the post-drawdown period up to the end of September (i.e., the outflows are roughly equal to the inflows). The drawdown model and Figure 3.1 and Figure 3.2 indicate that the post-drawdown water surface levels will range between 2,192 ft and 2,206 ft for average daily conditions evaluated in the drawdown model. These levels do not account for the low probability flood flows (i.e., the 1% and 5% probable flood flows), which would cause increases to these levels and reservoir ponding.

3.4 STEADY-STATE WATER SURFACE ELEVATIONS

3.4.1 PRE-DRAWDOWN TAILWATER LEVELS

The tailwater levels at the Iron Gate facility prior to opening the diversion tunnel are reported on Drawing C4050. A hydrodynamic model is used to calculate the water surface levels downstream of the Iron Gate dam for pre-drawdown conditions to determine the tailwater levels near the outlet of the diversion tunnel. The model uses HEC-RAS 2D with a Manning's n of 0.04 and considers a range of flows to evaluate the water surface levels at the low point in the access road to the downstream tunnel portal shown on Drawing C4500. The location of the tailwater level is shown on Figure 3.4.

Figure 3.4 Iron Gate Dam Pre-Drawdown Tailwater Rating Curve Location

The resulting stage-discharge relationship is shown on Drawing C4050 and Figure 3.5. A sensitivity analysis of the model uses Manning's n roughness of 0.03 and 0.06 to account for potential variability in channel roughness. The results of the sensitivity analysis are included on Figure 3.5.

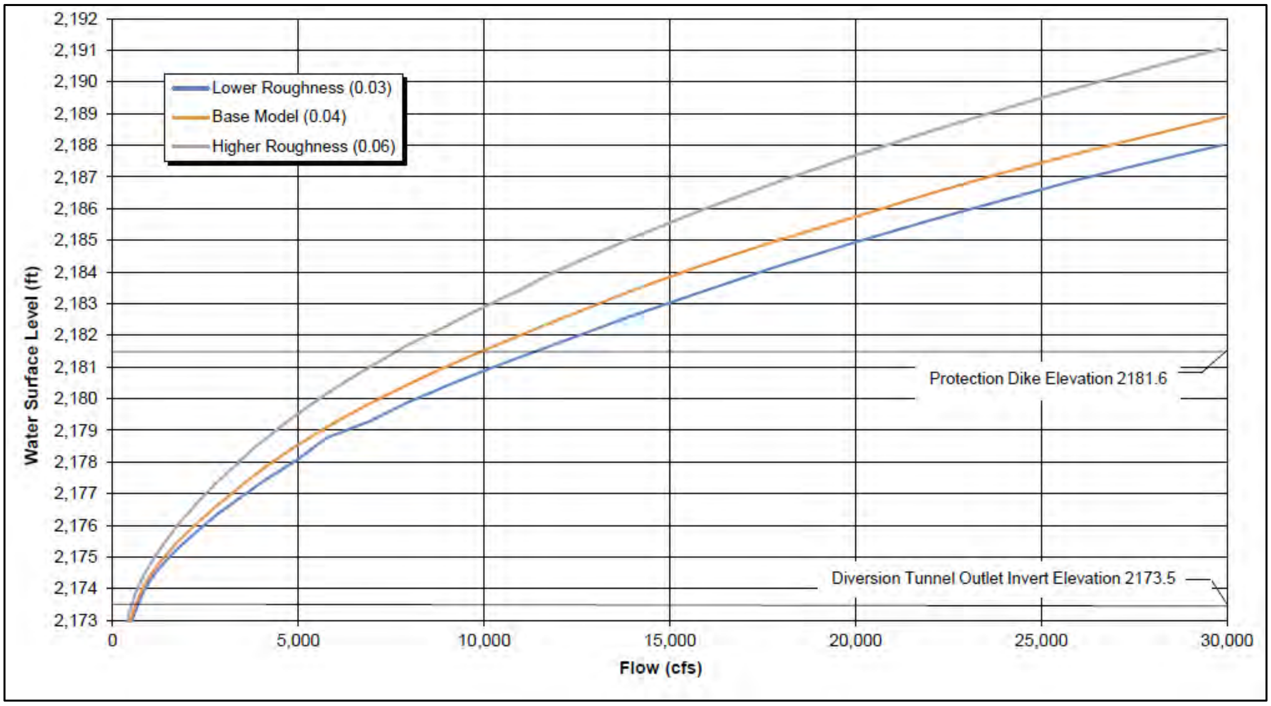


Figure 3.5 Iron Gate Dam Pre-Drawdown Tailwater Level Rating Curve Sensitivity Results

The low point in the access road shown on Drawing C4500 is at elevation 2,182.0 ft. This elevation relates to a steady-state flow of approximately 5,500 cfs assuming 3 ft of freeboard below the road crest using the base case hydrodynamic model.

3.4.2 EMBANKMENT AND RESERVOIR LEVELS

Flood water surface levels at the embankment are shown on Drawing C4055 for steady-state inflows. The statistical flood flows (high water) are based on peak instantaneous flows outlined in Appendix A6, while the daily average flows are average flows over a 24-hour period. The flood flows assume that the J.C. Boyle, Copco No. 1 and Iron Gate reservoirs provide flow attenuation from January through June 15 of the drawdown year. Once river diversion has been achieved at these facilities, the flood flows will no longer be attenuated, therefore, no flow attenuation is assumed for the flood flows between June 16 and December of the drawdown year, as discussed in Appendix A6. The levels are calculated using the discharge rating curves developed for the outlet structures as shown on Drawing C4055.

3.4.3 POST-DRAWDOWN TAILWATER LEVELS

The tailwater levels at the Iron Gate facility at the outlet of the diversion tunnel are reported for the post-drawdown period on Drawing C4055. A hydrodynamic model was used to calculate the water surface elevations downstream of the Iron Gate dam for post-drawdown conditions to determine the tailwater levels at the toe protection berm for the final dam breach as shown on Drawing C4207. The model uses HEC-RAS 2D with a Manning's n of 0.04. The model considers a range of flows to evaluate the water surface levels downstream of the diversion tunnel outlet as shown on Figure 3.6.

Figure 3.6 Iron Gate Dam Post-Drawdown Tailwater Rating Curve Location

The resulting stage-discharge relationship is shown on Drawing C4050 and Figure 3.7. Sensitivity of the model uses Manning's n roughness of 0.03 and 0.06 to account for potential variability in channel roughness. The results of the sensitivity analysis are also included on Figure 3.7.

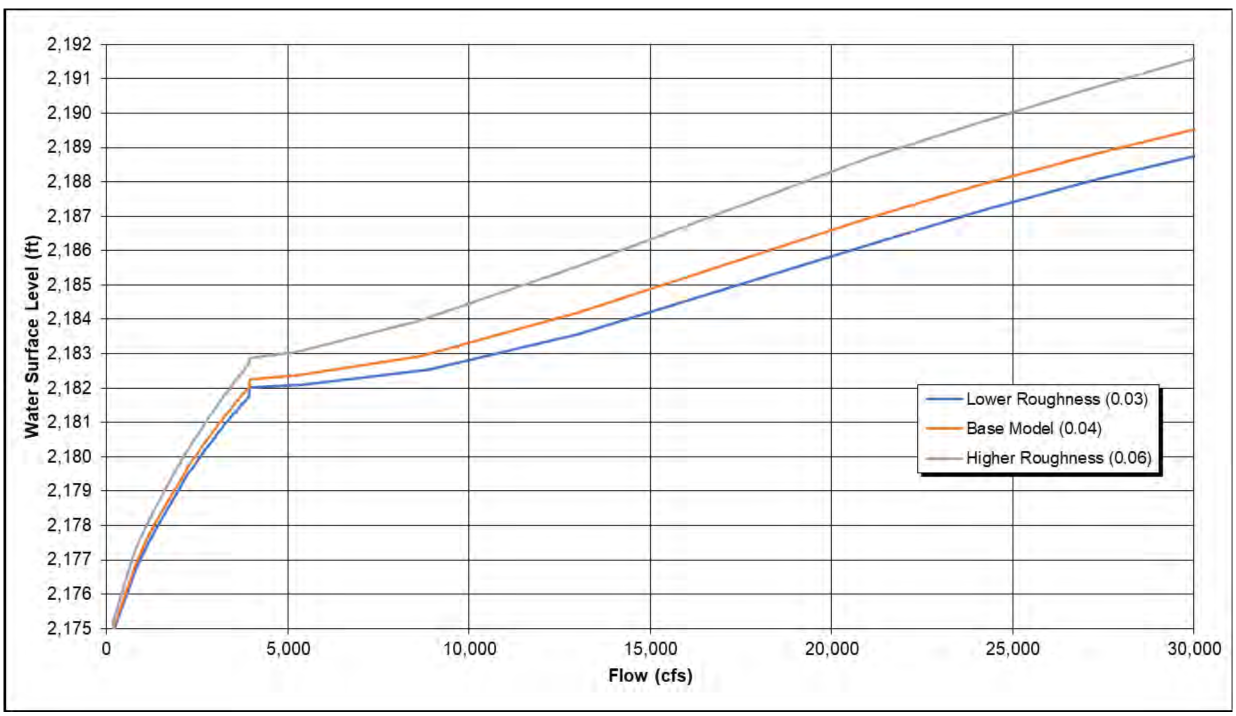


Figure 3.7 Iron Gate Dam Post-Drawdown Tailwater Level Rating Curve Sensitivity Results

3.5 FINAL COFFERDAM BREACH

3.5.1 DAM BREACH OVERVIEW

During the final stages of dam removal, the dam crest will be progressively lowered in the final stage to maintain 3 ft freeboard above the 1% probable flood reservoir level in the period from July 16 to August 31. The flows in this period are typically low and peak floods are primarily driven by controllable releases at the upstream irrigation project. The available storage in the Upper Klamath Lake in this period allows for flood storage, which can control river flow downstream and the inflows to Iron Gate reservoir.

The dam crest will be lowered to the target elevation of the Extended Cofferdam, which is 3 ft above the 1% probable flood reservoir level for August. The Extended Cofferdam is formed from the dam embankment and includes the historic cofferdam, as shown on Drawing C4255 and Figure 3.8. The Extended Cofferdam crest is below the 1% probable flood level for the period of September 1-15, but maintains an 8.9 ft freeboard above the 5% probable flood level for this period, and provides a reduced embankment volume for in-river removal following the initiation of the final breach.

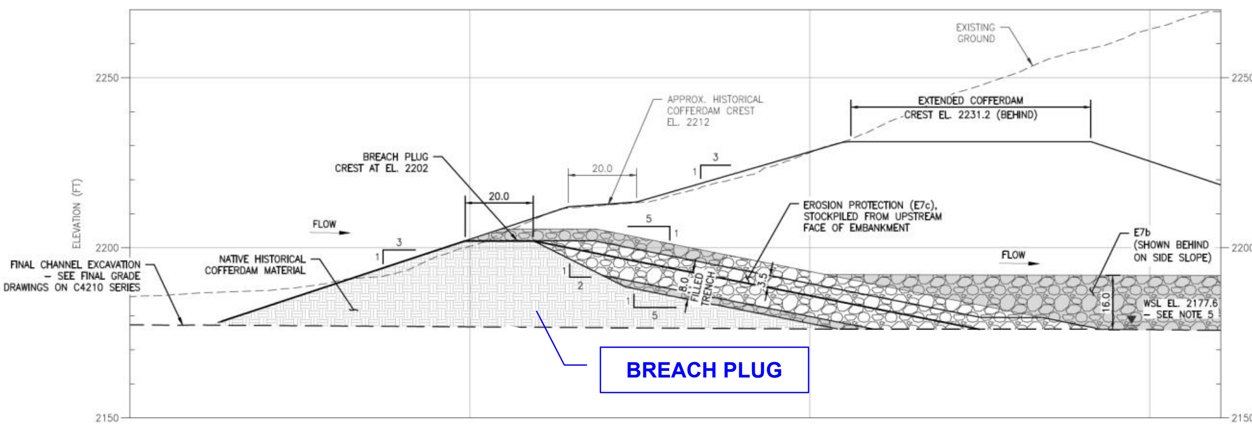


Figure 3.8 Extended Cofferdam Configuration Prior to Final Dam Breach

To prepare for the controlled breach, a trapezoidal channel will be excavated through the Extended Cofferdam adjacent to the right bank, starting at the downstream end of the dam and terminating at the Historical Cofferdam at the location of the breach Plug, as shown on Drawing C4250. The trapezoidal channel is designed with a base width of 20 ft and side slopes graded at 2H:1V or to suit the existing right bank bedrock slope. The breach Plug will be formed by stabilizing the existing Historic Cofferdam embankment with riprap to reduce the risk of breach flows eroding the embankment materials rapidly and resulting in uncontrolled breach conditions.

The controlled breach is initiated by excavating a notch through the breach Plug to allow the reservoir to discharge into the Extended Cofferdam trapezoidal channel. The dimensions of the notch through the breach Plug govern the discharge of the dam breach.

The following analyses are used to support the final dam breach design:

- Dam breach analysis – conducted to determine the breach parameters that would result in peak flows within the targeted maximum flows of approximately 6,000 cfs.

- Riprap design for the breach Plug and in the Extended Cofferdam channel – designed to protect the Extended Cofferdam trapezoidal channel from erosion at the estimated peak breach flows.

Final dam breach design drawings are found on Drawings C4250 and C4255, as shown in Table 3.2.

Table 3.2 Iron Gate Facility Drawings – Final Dam Breach

Drawing Number	Drawing Title
C4250	Iron Gate Facility - Embankment Removal - Final Breach Plan
C4255	Iron Gate Facility - Embankment Removal - Final Breach - Breach Plug Details

3.5.2 DAM BREACH ANALYSIS

3.5.2.1 GENERAL

The dam breach analysis is used to understand the possible range of peak discharges given the uncertainties related to the selection of breach parameters. The assessment attempts to establish the most sensitive and limiting breach parameters required for safe final breaching and reservoir release that would result in discharges that are within the target peak flow 6,000 cfs at USGS Gaging Station No. 11516530, Klamath River below Iron Gate Dam. The results of this assessment are used in the design of the final breach channel, and to inform the recommendation on the preferred reservoir levels and timing for the final release of the Iron Gate reservoir through breaching the Plug.

HEC-RAS hydrodynamic modeling (USACE, 2019) facilitates the evaluation of different dam breach scenarios with varying inflows and reservoir levels in consideration of various climatic and operational uncertainties during the dam removal processes. The analysis includes breach scenarios through the Plug during the final stages of dam decommissioning to assess the peak breach outflows for different breach widths, breach formation times, and starting reservoir water surface levels. In addition, a hypothetical overtopping breach of the Extended Cofferdam caused by a high storm event is considered to understand the level of risk downstream of the Iron Gate Dam. Sensitivity analysis allows for the assessment of the uncertainties related to the selection of dam breach parameters, which is a standard approach in dam breach analysis.

The HEC-RAS dam breach model utilises the HEC-RAS drawdown model (Appendix G). The Iron Gate dam breach model extends upstream of the dam for the full length of the reservoir and approximately 4,000 ft downstream from the breach Plug, as shown on Figure 3.9. The model captures dynamic reservoir routing during the breach, but does not extend sufficiently far downstream to fully evaluate the downstream inundation extents or impacts in terms of the flood wave propagation and attenuation. In this dam breach analysis the potential impacts to the downstream Klamath River reaches are evaluated qualitatively by comparing the peak breach outflow conditions at the dam to the known monthly flow conditions since the construction of the dam.

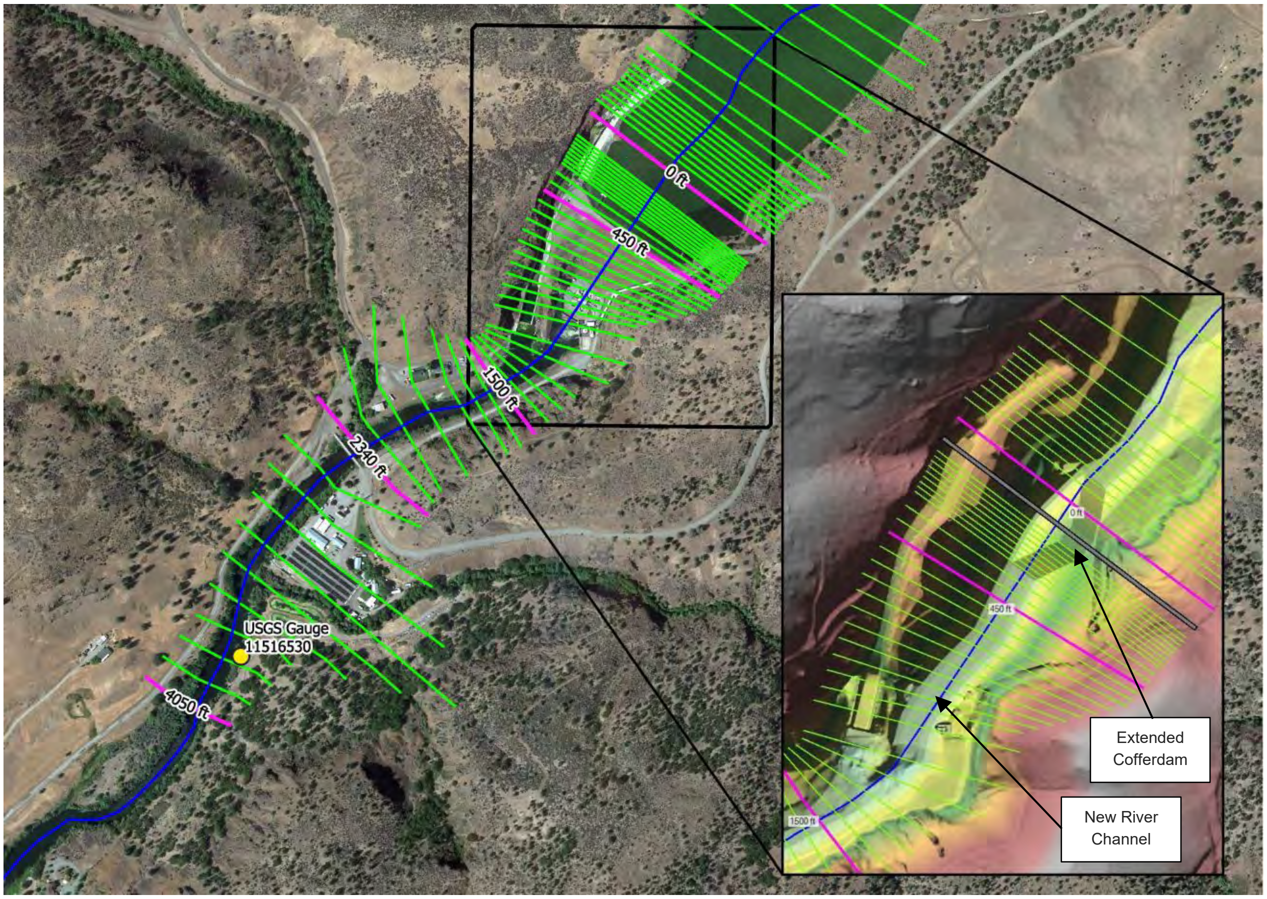


Figure 3.9 HEC-RAS Model Setup and Cross Sections for Dam Breach Analysis

3.5.2.2 DAM BREACH PARAMETERS

Breach parameters used in the analysis include: breach width, breach formation time, breach side slopes, breach bottom elevation, the overtopping weir coefficient, and the type of breach formation progression (linear or sinusoidal). Possible ranges for the breach parameters are based on the FERC Guidelines (FERC, 2015), and the terrain geometry at the Historic Cofferdam location. Sensitivity analysis is used to evaluate the breach parameters for various scenarios using the McBreach add-on to HEC-RAS, which utilizes probabilistic hydraulic modeling through Monte Carlo simulations.

The cross section through the breach Plug used in the dam breach analysis is shown on Figure 3.10. The reservoir volume at the crest of the Extended Cofferdam and at the crest of the breach Plug is equivalent to 3,844 acre-ft and 465 acre-ft, respectively, as shown in Table 3.1.

The ranges used for the breach parameters presented in Table 3.3 are based on FERC Guidelines (2015).

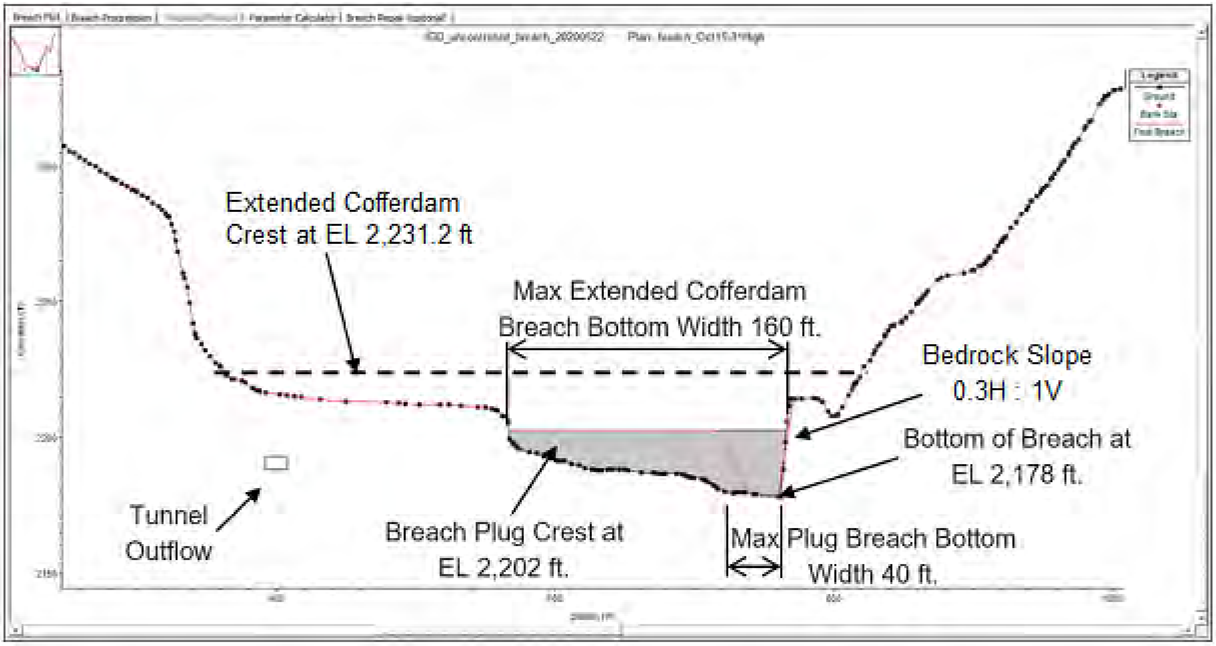


Figure 3.10 Cross-Section Used in HEC-RAS for Dam Breach Analysis

Table 3.3 Input Breach Parameters for Probabilistic Monte Carlo Simulations in McBreach

Scenario	Breach Parameter (Unit)	Method[1]	Distribution[2]	Min.	Max.
All scenarios	Final Bottom Elevation (ft.)[3]	Deterministic	-	2,178	
	Right Side Slope (xH:1V)[4]	Deterministic	-	0.30	
	Left Side Slope (xH:1V)	Probabilistic	Uniform	0.25	1.00
	Formation Time (hr)	Probabilistic	Uniform	0.10	1.00
	Weir Coefficient[5]	Probabilistic	Uniform	2.60	3.30
	Progression	Deterministic	-	Sinusoidal	
Breach Plug	Final Bottom Width (ft.)	Probabilistic	Uniform	1.00	40.0
Extended Cofferdam		Probabilistic	Uniform	17.5	160.0

NOTES:
1. METHOD IS INDICATED AS "DETERMINISTIC" FOR THE PARAMETER VALUES DRIVEN BY THE TERRAIN GEOMETRY, OR "PROBABILISTIC" FOR THE CONDITIONS WHERE A RANGE OF VALUES IS EVALUATED.
2. "UNIFORM" DISTRIBUTION INDICATES THAT THE SELECTION OF THE PARAMETER VALUE WITHIN A GIVEN RANGE IS RANDOM.
3. BOTTOM OF BREACH ELEVATION IS ASSUMED AT BEDROCK EL. 2,178 ft.
4. THE RIGHT SIDE BREACH SLOPE IS LIMITED TO THE SLOPE OF THE BEDROCK AT THIS LOCATION.
5. HEC-RAS MANUAL (USACE, 2019) RECOMMENDS THAT THE WEIR COEFFICIENT USED IN OVERTOPPING BREACH ASSESSMENTS OF EMBANKMENT DAMS IS IN THE RANGE OF 2.6 TO 3.0, HENCE THIS RANGE IS CONSERVATIVE.

The results of the sensitivity analysis on breach parameters are used to inform the critical design parameters for the breach Plug, as follows:

- The peak discharge is very sensitive to the reservoir water surface level and reservoir volume at the time of the breach. Breaching at reservoir levels lower than 2,200 ft would result in peak breach discharges that would not cause overbank flooding providing the breach parameters are controlled as indicated below.
- The peak discharge is very sensitive to the breach bottom width. Limiting the breach bottom width to approximately 20 ft or less would result in peak breach discharges that would not cause overbank flooding. This could be achieved by appropriate riprapping of the breach channel sides, that would prevent it from excessive widening.
- The peak discharge is moderately sensitive to the breach formation time at low reservoir levels. Having a breach formation time longer than approximately 0.3 hours would result in peak breach discharges that would not cause overbank flooding. This could be achieved by riprapping the downstream face of the breach plug with riprap size that would be marginally mobile, such that rapid downcutting does not occur, but progressive erosion is still possible.
- The breach peak discharge is not very sensitive to the other breach parameters shown in Table 3.3.

These findings are utilized to develop the breach Plug design shown on Drawing C4250.

Sensitivity analysis on breach parameters for the Extended Cofferdam indicate that even the smallest and slowest breach would cause very high peak discharges and potentially extensive downstream impacts due to overbank flooding. Due to a relatively high volume of water stored in the Iron Gate reservoir, and much higher hydraulic head in the breach, breach peak outflows range from approximately 65,000 cfs to over 130,000 cfs. This analysis confirms that the potential overtopping of the Extended Cofferdam must be prevented. This will be accomplished by completing the breach prior to September 16 when the flood risk is within the accepted criteria, or alternatively by coordination with upstream water management agencies to manage flood risk after September 16. Furthermore, the final breach must be implemented when inflows and pond level are forecast to be at a sufficiently low level to achieve the peak outflow criteria.

3.5.2.3 BREACH OUTFLOW HYDROGRAPHS

Additional dam breach HEC-RAS modeling is conducted for various conditions relevant to breaching of the Plug. This analysis uses pre-defined breach parameters based on the results of the probabilistic analysis to model the breach outflow hydrographs for various combinations of breach parameters and flow conditions that could be expected during the final breach of the Plug. The modeling includes routing the steady-state inflows through the reservoir, and routing the reservoir outflows through the tunnel and through the developing breach in the Plug, and then further downstream through the new channel constructed through the dam footprint, as shown on Figure 3.9.

The HEC-RAS outflow hydrographs for one of the model scenarios is shown on Figure 3.11. It illustrates that the inflow of 1,090 cfs (equivalent to the mean September 1-15 flow) is passed through the tunnel at the start of the model. Once the breach starts developing, the flows through the breach increase, while the flows through the tunnel decrease with decreasing reservoir levels. The total flows downstream of the Iron Gate facility include both the tunnel flows and the breach flows, as shown with the third hydrograph on Figure 3.11. Once the breach is complete, all inflows are passed through the breach opening and steady-state conditions are established through a newly formed river channel.

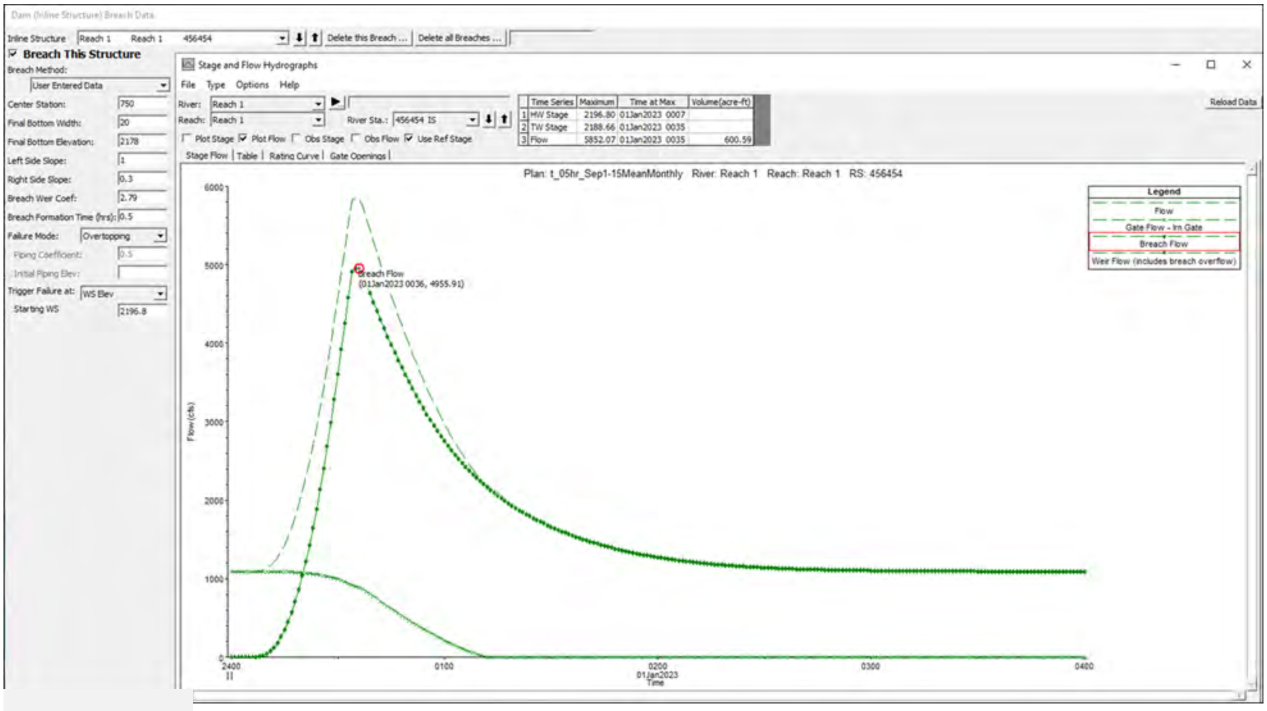


Figure 3.11 Example HEC-RAS Output for the Breach, the Tunnel and Total Outflows

Figure 3.12 shows the total downstream outflow (tunnel and breach) for three possible scenarios of breaching of the Plug through a 20 ft wide bottom breach (B = 20 ft) during the mean September 1-15 hydrologic conditions. The reservoir inflow is 1,090 cfs, while the reservoir water surface level is at 2,196.8 ft at the beginning of the breach. All breach parameters for the three scenarios are equal except for the time to breach. The three scenarios compare the breach outflows in case the Plug riprap erodes quickly in 0.1 hr (T_f = 0.1 hr), moderately fast in 0.5 hr (T_f = 0.5 hr), or at a slower rate of 1 hr (T_f = 1 hr).

The scenario with the fast breach formation time of 0.1 hr results in a peak discharge of approximately 6,600 cfs (approximately 5,600 cfs through the breach and 1,000 cfs through the tunnel). The moderately fast breach with a formation time of 0.5 hr results in a total peak discharge of approximately 5,900 cfs (approximately 5,000 cfs through the breach and 900 cfs through the tunnel). The slower breach with a formation time of 1 hr results in a total peak discharge of approximately 4,900 cfs (approximately 4,200 cfs through the breach and 700 cfs through the tunnel). These flows represent the total flow at the breach location. Additional attenuation of the breach outflows occurs through the new river channel constructed through the existing dam footprint, as discussed in Section 3.5.2.4.

Riprapping the Plug downstream face with a marginally mobile riprap size will aid in controlling and slowing down the breach formation time and aid in achieving flows below the maximum target flow of 6,000 cfs. The modeling indicates that even in case the breach developed very fast in 0.1 hr, the peak outflow right at the breach location would be 10% over the maximum target flow of 6,000 cfs; however, it would attenuate quickly to below the target flows.

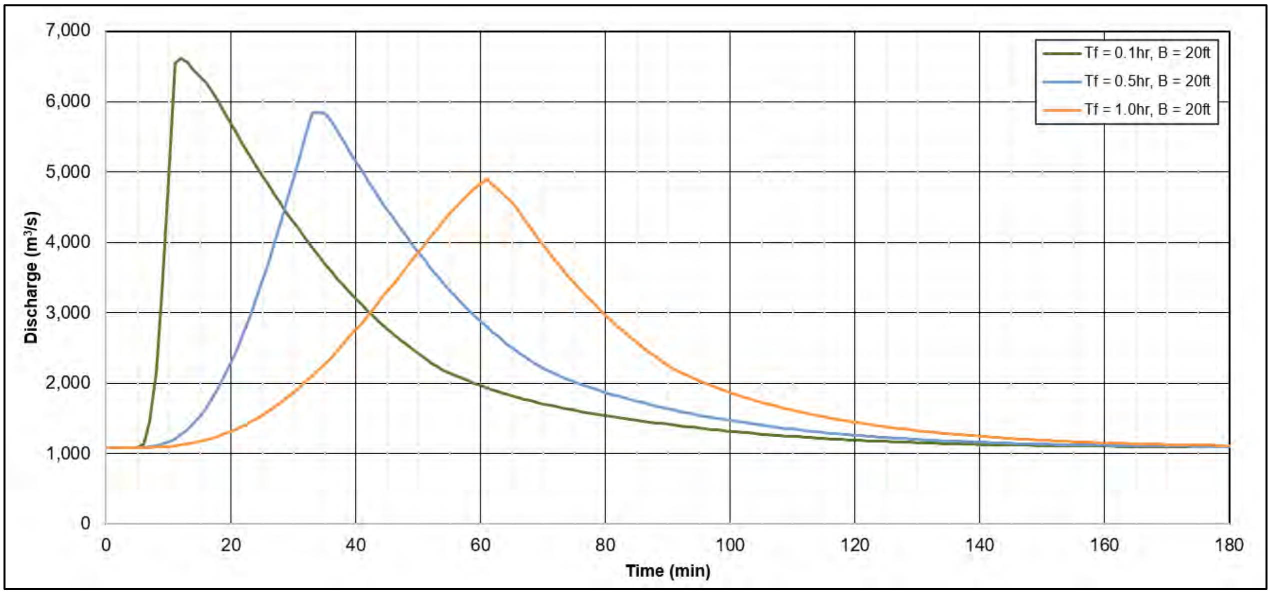


Figure 3.12 Total Outflow Hydrographs (Breach Plug and Tunnel) for Different Breach Formation Times

3.5.2.4 DOWNSTREAM ATTENUATION

The HEC-RAS model used in this breach analysis extends for about 4,000 ft downstream of the breach Plug. Table 3.4 summarizes the peak flow attenuation predicted at cross sections shown on Figure 3.10 for the three breach hydrographs from Figure 3.12. Lakeview Bridge across the Klamath River is located 2,340 ft downstream of the breach Plug location, while the USGS gauge 11516530 is located about 3,500 ft downstream. The flow attenuates to below the maximum target flow of 6,000 cfs by the time the flood wave reaches Lakeview Bridge even in the case of a very fast breach of 0.1 hr.

Table 3.4 Downstream Peak Flow Attenuation

	Attenuated Flow at Downstream Location (cfs)		
Distance Downstream of Breach (ft)	T_f = 1 hr B = 20 ft	T_f = 0.2 hr B = 20 ft	T_f = 0.1 hr B = 20 ft
0	4,890	5,850	6,620
450	4,870	5,830	6,560
1,500	4,750	5,590	6,060
2,340	4,680	5,420	5,790
4,050	4,570	5,220	5,490

NOTES:
1. DISTANCE 0 INDICATES THE LOCATION IS AT THE BREACH PLUG.
2. THE ATTENUATED FLOW MAGNITUDE IS EQUIVALENT TO THE TOTAL OUTFLOW (BREACH PLUG AND TUNNEL).

Inundation maps have been prepared in the pre-dam removal 2D hydrodynamic modeling study (Yurok Tribe / USBR, 2020) for a steady peak flood magnitude of 10,980 cfs (USBR, 2012). These maps indicate that such flows may cause very limited overbank flooding downstream of the Iron Gate Dam, but are not predicted to cause major flooding or damage to the downstream properties. The three breach scenarios shown in Figure 3.12 and Table 3.4 have peak discharges that attenuate to approximately one half of the flood modelled in the Yurok Tribe / USBR 2020 study within the first 4,000 ft (less than 1 mi) downstream of the breach location. These peak breach flows are not expected to cause overbank flooding within this reach of the Klamath River or farther downstream.

3.5.3 RIPRAP DESIGN

As discharge proceeds through the breach opening and afterwards when the breach is complete, the Extended Cofferdam trapezoidal channel will be protected by a layer of riprap material to prevent the toe of the cofferdam from eroding.

Riprap design is separated into three zones shown in Figure 3.13 (Drawing C4250) and Table 3.5, as follows:

- Zone 1: Downstream face of the breach plug –Riprap for Zone 1 is sized based on the Shield's parameter approach as presented in USBR PAP-0809 Riprap Design for Overtopped Embankments (1998) and in USBR PAP-0790 Simplified Design Guidelines for Riprap Subjected to Overtopping Flow (2010) – the goal is to mobilize the riprap material immediately downstream of the breach cut while protecting the rest of the breach plug's downstream face. This allows the breach to gradually progress while mitigating against the risk of an abrupt breach expansion.
- Zone 2: Riprap-filled trench –if the breach cut widens and reaches the trench line, this feature is expected to unload riprap onto the side slope of the breach cut thereby protecting the slope from further erosion and acting to prevent the breach from widening beyond the trench line. Riprap for Zone 2 is sized such that the mean flow velocity is less than the critical velocity for riprap mobilization based on FHWA Evaluating Scour at Bridges (2012).
- Zone 3: Downstream of the breach plug, at the trapezoidal channel – is sized to protect the channel from erosion during peak breach outflows. Riprap for Zone 3 is assessed similar to Zone 1 but with the goal of not mobilizing the riprap material in the trapezoidal channel.

The breach plug riprap design summary is shown in Table 3.5.

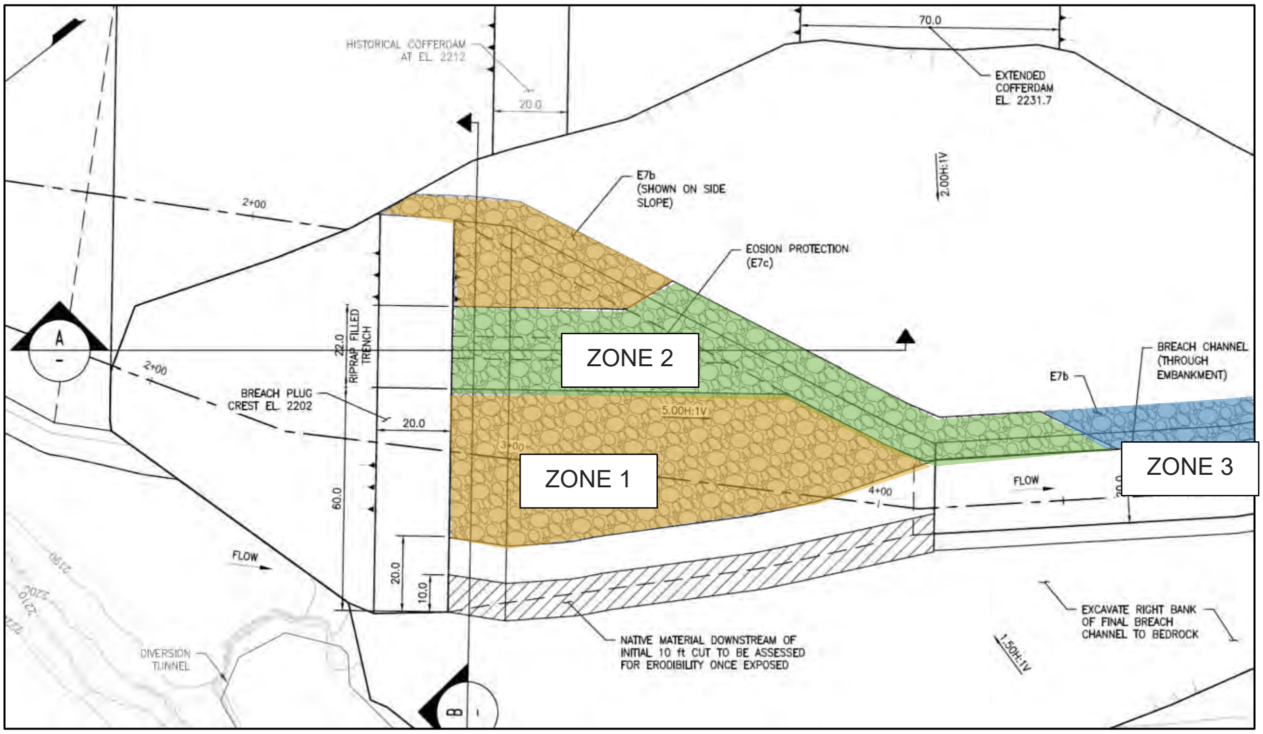


Figure 3.13 Breach Plug and Extended Cofferdam Channel – Riprap Zones – Plan

Table 3.5 Riprap Design Summary

	Riprap Size, D_{50}	Riprap Material	Channel Slope (%)	Flow (ft^3/s)	Notes
Zone 1	20 in	E7b	20% Max at start of breach. Will vary and decrease as breach flows down cut through the plug to pass more flow.	132	• Flow corresponds to the estimated flow through the initial notch (4 ft deep x 10 ft wide) that initiates the breach.
Zone 2	36 in	E7c		6,600	• Flow corresponds to the estimated peak breach flow (see Section 3.5.2) over.
Zone 3	20 in	E7b	0.5% Max.	6,600	• Flow corresponds to the estimated peak breach flow (see Section 3.5.2) as applied to the trapezoidal channel.

3.6 FINAL GRADING HYDRAULIC CONDITIONS

Volitional fish passage channel characteristics and geometry presented on Drawings C4210, C4211, and C4212 are used to develop a hydrodynamic model to determine the discharge-stage relationship post-dam removal. The model uses HEC-RAS 2D with a Manning's n of 0.04.

The resulting stage-discharge relationship is shown on Figure 3.14 at the location of the current dam centerline. Sensitivity of the model uses a Manning's n of 0.03 and 0.06 to account for potential variability in roughness elements added to the channel to provide localized habitat elements. The results of the sensitivity analysis are included on Figure 3.14.

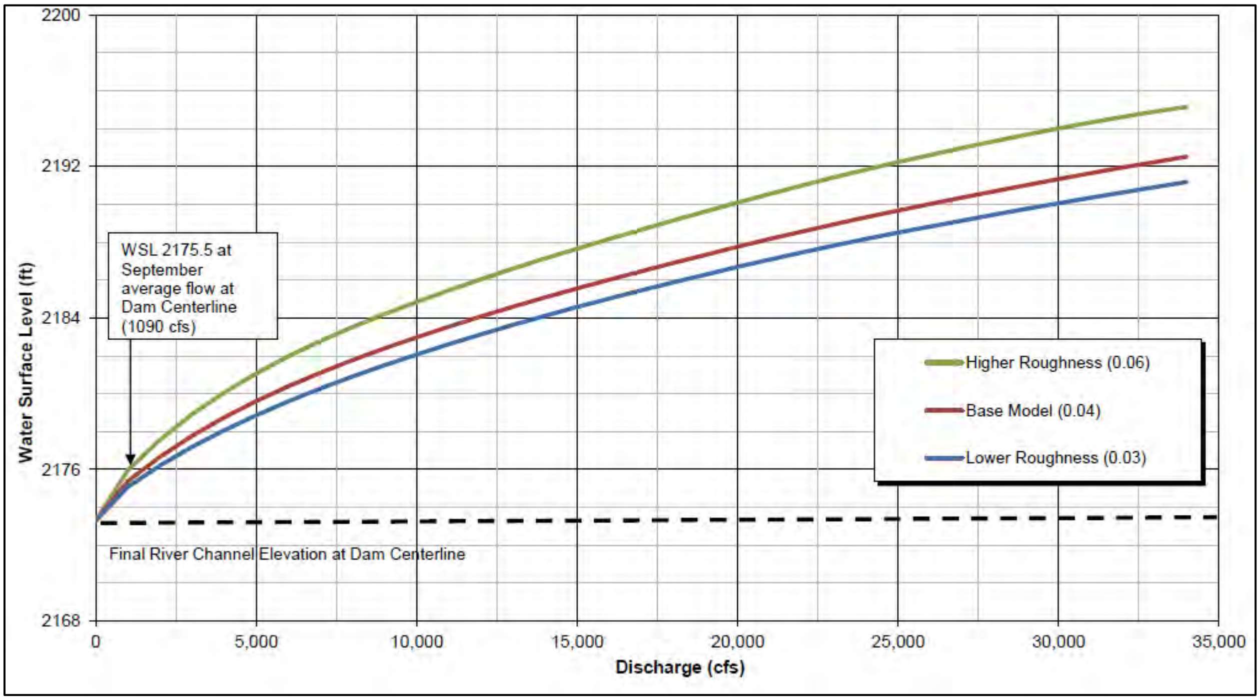


Figure 3.14 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

Dam removal construction activities in the vicinity of the final river channel are scheduled to continue into the fall. Steady-state water surface levels for probable floods and mean monthly flows for specified periods in September through November, are provided for reference in Table 3.6 using the base model Manning's n value of 0.04.

In addition, steady-state water surface levels for the final river channel for the annual probable floods, the mean annual flow, and the annual 25% and 75% flow durations are provided in Table 3.7 using the base model Manning's n value of 0.04.

Table 3.6 Final River Channel Monthly Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge (cfs)					
		Time Period					
		Sep 1 – 15	Sep 16 – 30	Oct 1 – 15	Oct 16 – 31	Nov 1 -15	Nov 16 - 30
Statistical High Water (Flood Conditions)	5% Probable Flood	2,000	2,200	6,000	6,500	7,800	8,800
	20% Probable Flood	1,700	1,700	4,500	4,800	5,400	6,100
	50% Probable Flood	1,400	1,400	3,800	3,800	4,100	4,400
Mean Monthly Flow for Time Period		1,090	1,090	1,120	1,210	1,300	1,310
Flow Condition		Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline					
Statistical High Water (Flood Conditions)	5% Probable Flood	2,176.7	2,176.9	2,180.4	2,180.8	2,181.6	2,182.3
	20% Probable Flood	2,176.3	2,176.3	2,179.2	2,179.4	2,179.9	2,180.5
	50% Probable Flood	2,175.9	2,175.9	2,178.5	2,178.5	2,178.8	2,179.1
Mean Monthly Flow for Time Period		2,175.5	2,175.5	2,175.5	2,175.7	2,175.8	2,175.8

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,173.3 ft.

Table 3.7 Final River Channel Annual Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge with Attenuation from Upstream Facilities (cfs)	Discharge with No Attenuation from Upstream Facilities (cfs)
Statistical High Water (Flood Conditions)	1% Probable Flood	31,200	33,600
	5% Probable Flood	19,300	25,400
	20% Probable Flood	10,900	16,200
	50% Probable Flood	7,500	11,700
Annual Flow Duration 25% of Time Equaled or Exceeded		1,880	1,880
Mean Annual Flow		1,820	1,820
Annual Flow Duration 75% of Time Equaled or Exceeded		1,000	1,000
Flow Condition		**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline**	**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline**
Statistical High Water (Flood Conditions)	1% Probable Flood	2,191.7	2,192.4
	5% Probable Flood	2,187.4	2,189.8
	20% Probable Flood	2,183.5	2,186.1
	50% Probable Flood	2,181.4	2,183.9
Annual Flow Duration 25% of Time Equaled or Exceeded		2,176.5	2,176.5
Mean Annual Flow		2,176.4	2,176.4
Annual Flow Duration 75% of Time Equaled or Exceeded		2,175.4	2,175.4

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,173.3 ft.

4.0 GEOTECHNICAL

4.1 DIVERSION TUNNEL

The Norwegian Method is used to assess the adequacy of the existing rock cover above the tunnel crown and adjacent to the tunnel walls for sustaining the maximum hydrostatic loading imposed at the beginning of reservoir drawdown.

The maximum hydrostatic loading is calculated from the Maximum Water Level and the tunnel crown. The rock cover comprises only bedrock. A factor of safety of 1.5 is applied to the calculation of the minimum rock cover since the diversion tunnel is critical to achieving reservoir drawdown within the short timespan required by the project schedule. A sensitivity check has also been completed with a factor of safety of unity. The required rock cover in both cases exceeds the existing bedrock cover above the tunnel crown, at approximately 65 ft upstream of the outlet portal. As a result, the design is based on the concept to use the existing gate for drawdown over an extended period. The design reduces the risk of hydro-fracture stemming from pressurized flow should the tunnel lining become compromised.

4.2 DAM EMBANKMENT STABILITY DURING DRAWDOWN

Stability of the dam embankment during reservoir drawdown was assessed by Limit Equilibrium Analysis (LEA) for transient pore pressure distributions produced by a generalized drawdown curve, which was defined based on results from the drawdown simulations (1981 through 2016) for a fully open gate. GeoStudio (GEO-SLOPE, 2020) was used to complete the seepage analysis and LEA (Spencer and Morgenstern-Price or GLE method of slices). The acceptance criterion is defined by a Factor of Safety (FOS) of 1.3, based on the more conservative recommendation of the USACE (2003).

The drawdown simulations indicated variable drawdown rates and multiple drawdown-refill cycles that could involve sizeable changes in water level elevation over a short duration. As a result, a generalized curve was defined to drawdown at the fastest simulated rate for the largest total head difference and to provide ther greatest potential for re-saturation once the reservoir refilled. The full curve is shown in the inset of Figure 4.1 and the corresponding stability analyses, at 1-day timesteps for the transient seepage analysis, indicated the lowest FOS occurred during the initial reservoir drawdown. Consequently, a higher resolution drawdown curve was developed for the first 31 days of the full drawdown curve with 1-hour timesteps for the transient seepage analysis. The higher-resolution curve is shown on Figure 4.1.

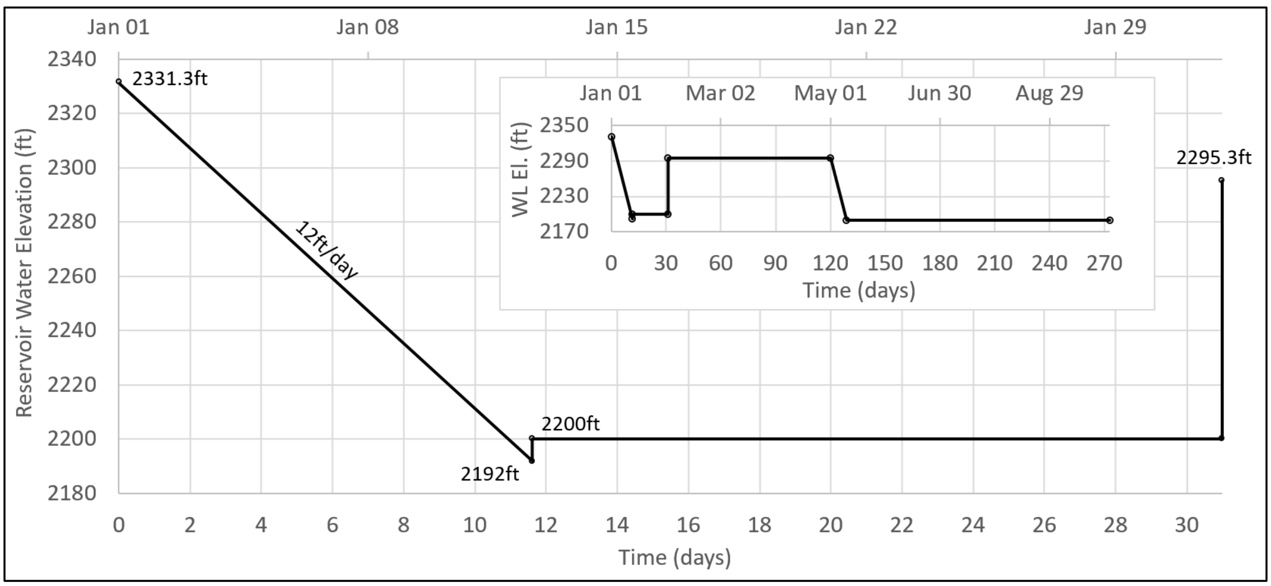


Figure 4.1 Generalized Drawdown Curve For Stability Analyses

Material properties used in the analyses are shown in Table 4.1 and include both base case and sensitivity values. The properties are adopted from the STIDs and an evaluation of historical data or revised to more conservative values. A sensitivity check was completed on a second model by making changes to material properties according to those shown in Table 4.1.

Table 4.1 Material Properties for Drawdown Stability Assessment

Material	Horizontal Hydraulic Conductivity (ft/s)	Vertical:Horizontal Hydraulic Conductivity Ratio	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core	1e-6	1 (0.1)	130	22 (17)	0
Shell	8e-3	1 (0.5)	135 (123)	35 (30)	0
Filter	1e-2	1	135	35	0
Riprap	1e-2	1	135	35	0
Random Fill	8e-3	0.5	135	35	0
Bedrock	1e-12	1	impenetrable		

NOTES:

1. SENSITIVITY CHECKS COMPLETED WITH VALUES IN PARENTHESES.

The analysis model geometry is shown on Figure 4.2. Three scales of slip are considered in the LEA. The first was a full-height slip, extending from the dam crest to the upstream toe. The second was a smaller slip, involving the lower slope or from the bench at elevation 2,280 ft to the upstream toe. The third was a smaller slip that involved the upper slope, extending from the dam crest to the bench at elevation 2,280 ft.

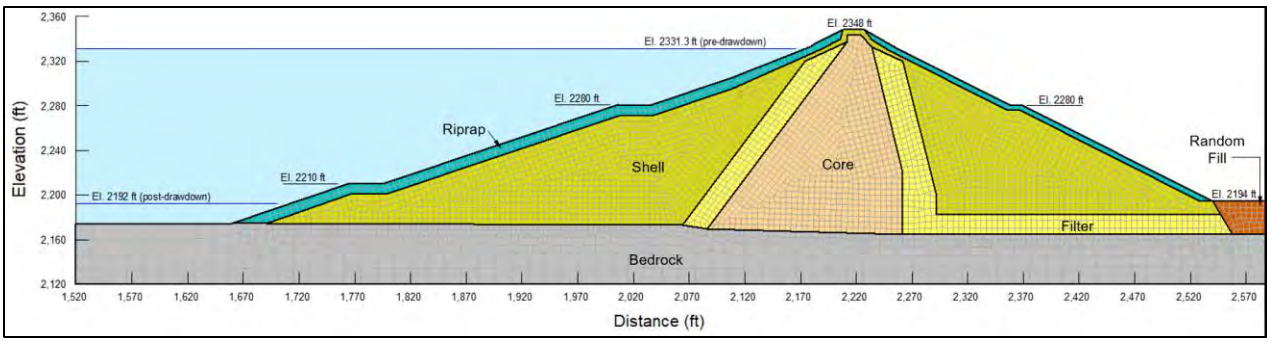
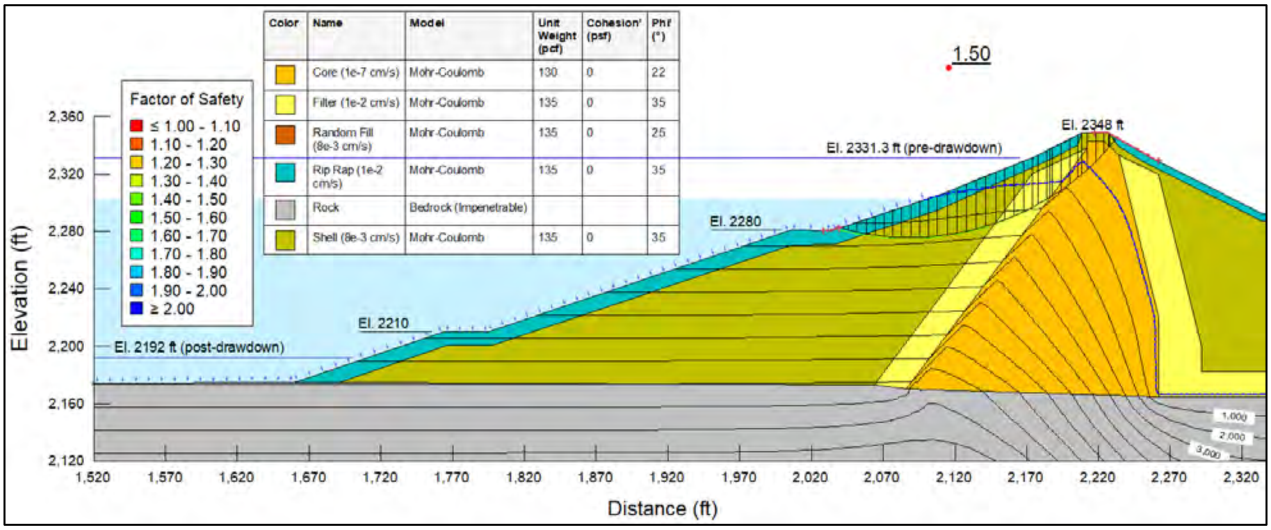


Figure 4.2 Drawdown Analysis Model Geometry

The stability results indicated the lowest FOS for the three scales of slips is 1.5 for base case properties and 1.3 for the sensitivity check. These results indicate the dam embankment is expected to be stable during the defined drawdown curve.

The upper slope slip governs stability for both the base case properties and the sensitivity checks. The critical slips are shown on Figure 4.3 and Figure 4.4, along with pore pressure contours in psf. The timesteps associated with the critical slips are 2.5 and 2.7 days after drawdown commences, respectively for base case properties and for the sensitivity check.

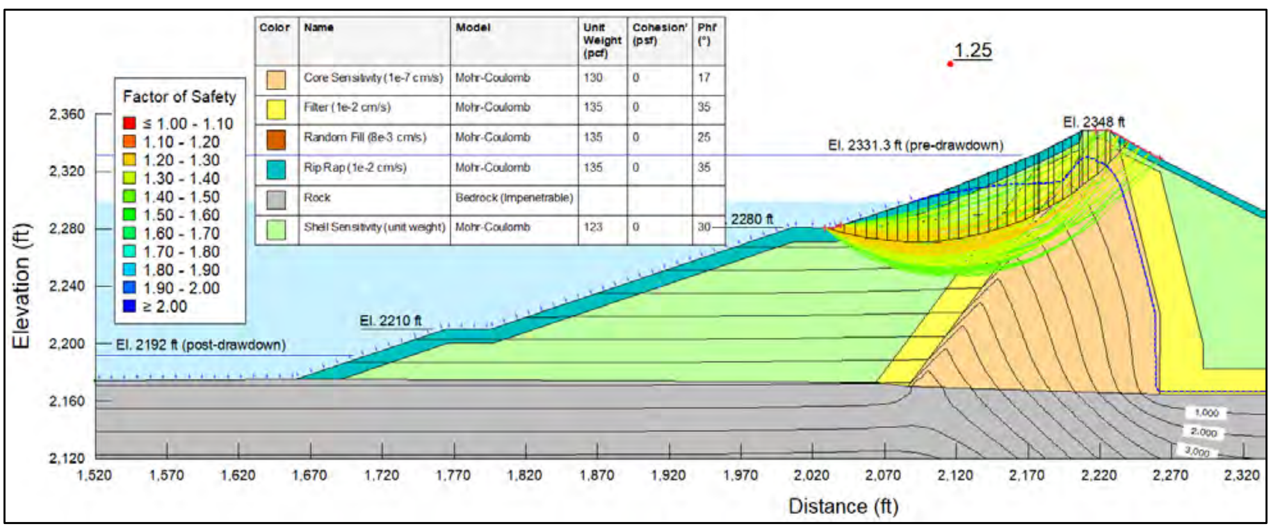
Stability results are summarised in Table 4.2 for the three scales of slips. Results for the GLE and Spencer methods are similar; the lower FOS is reported.



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

Figure 4.3 Drawdown Stability Results for Upper Slope Slip with Base Case Properties



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

Figure 4.4 Sensitivity Check Drawdown Stability Results for Upper Slope Slip

Table 4.2 Factor of Safety Results for Drawdown Stability

Slip Scale	Base Case Properties	Sensitivity Check
Full Height	1.70	1.34
Lower Slope	1.61	1.30
Upper Slope	1.50	1.25

The sensitivity check indicates stability of the upper slope could decrease to marginally achieve the target FOS for a conservative scenario that combines the lowest strength parameters and adverse permeability conditions for the core and shell materials. However, the sensitivity FOS is above 1.1, which is the lower value for the acceptance criteria of an operating hydropower facility recommended by the USACE.

4.3 STABILITY DURING DAM REMOVAL

The removal of the embankment dam at Iron Gate will be staged such that stability will equal or exceed the current condition set out in the STID. This will be achieved by:

- Dam removal staging by horizontal lifts, preferably from the right abutment to the left abutment
- Upstream and downstream slopes maintained at equal to or shallower than existing
- Crest width buttressing the impervious core zone material maintained at equal to or wider than the current dam crest

All materials removed from the dam site have been accounted for in the following disposal sites:

- Existing spillway
- Final grading at the powerhouse site
- Upland Disposal Site

A mass balance of the material being removed and the material in the disposal sites is complete. The approximate embankment removal volumes by sequence are shown in Table 4.3. The approximate fill volumes in each disposal site are presented in Table 4.4. The removal sequence is governed by the applicable flood probability and the need to maintain a 3 ft freeboard for the associated flood level.

Table 4.3 Embankment Cut Volumes by Sequence Number

Sequence Description	Drawing Number	Flood Probability (Monthly or Semi-Monthly)	Start Date	End Date	Minimum Crest Elevation (ft)	Approximate Sequence Cut Volume (CY)	Spoil Location
1	C4203	1%	1-May	31-May	2,336.3	55,000	Spillway End Dump Ramps, Spillway Toe, Powerhouse Toe Protection Berm
2	C4204	1%	1-Jun	15-Jun	2,335.5	1,000	Spillway Fill
3	C4205	1%	16-Jun	30-Jun	2,308.6	71,000	Spillway Fill
4	C4206	1%	1-Jul	15-Jul	2,297.5	89,000	Spillway Fill, Upland Disposal Site
5	C4207	1%	16-Jul	31-Jul	2,243	341,000	Upland Disposal Site
6	C4208	5%	1-Aug	31-Aug	2,231.2	96,000	Upland Disposal Site
7	C4209	5%	Sequence 6 Completion	31-Aug	2,231.2	183,000	Upland Disposal Site
Breach Channel	C4250	Not Applicable	Sequence 7 Completion	15-Sep	2,202.0	45,000	Upland Disposal Site, Powerhouse Fill
Final Grade	C4210	Not Applicable	Breach Completed	1-Oct	Not Applicable	62,000	Upland Disposal Site, Powerhouse Fill
			Total Embankment Removal Volume (CY)			**943,000**	

Table 4.4 Embankment Removal Mass Balance

Estimated Spillway Fill (C4220 & C4221)	249,200 yd^3
Estimated Powerhouse Site Fill (C4401 to C4402)	30,790 yd^3
Upland Disposal Site (C4230 & C4231)	662,710 yd^3
Total Estimated Embankment Cut	**942,700 yd^3**

Additional spoil capacity is available in the upland disposal site. As currently modelled, it offers a total storage capacity of 1,200,000 yd^3. This allows for flexibility and optimization by the Project Company.

4.4 EXCAVATION SLOPES

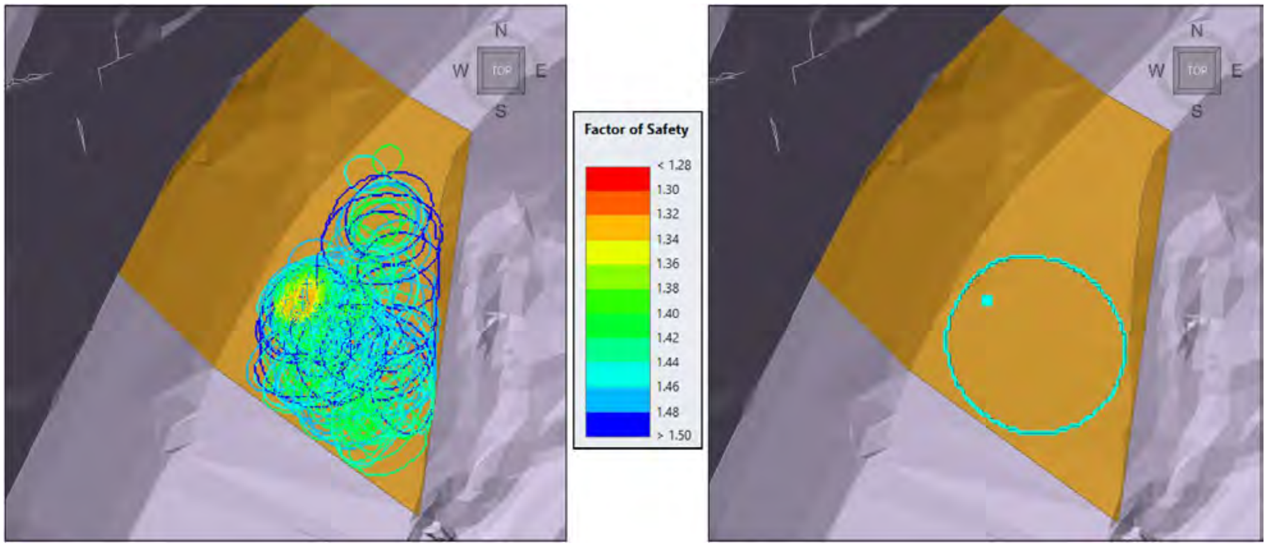
The dam embankment will be removed in stages and the river channel will be restored by excavating the existing dam embankment to form temporary and permanent slopes. Excavated slopes are designed to be stable with suitable drainage. Areas where excavated slopes within the dam footprint cannot be considered stable in the long term due to the valley geometry will be excavated to bedrock.

Stability analyses of a simplified model are used to evaluate the final grade of the rockfill shell that will remain post dam removal. LEA are completed in three dimensions using Slide3 (Rocscience, 2020) and the Spencer and GLE methods of slices. Two loading conditions are considered. The static long-term and yield acceleration (ky) determination for approximating seismic displacement. The acceptance criteria require a FOS of 1.5 for static long-term stability and FOS of 1.0 for yield acceleration determination without strength reduction. In addition, STID-8 (PacifiCorp, 2015a) indicated displacements of 2 ft are acceptable according to a FERC guideline for the operating dam. Seismic displacements are approximated by two semi-empirical methods, developed by Makdisi and Seed (1978) and Bray and Travasarou (2007). The design seismic loading was defined by STID-5 (PacifiCorp, 2015b), for a Maximum Credible Earthquake (MCE) of 0.25 g, and an assumed magnitude 7.5 earthquake.

The model geometry was simplified since the bedrock contact is unknown by assuming the shell material extended to an infinite depth within lateral extents roughly delineated based on existing topography and the design slope. Dry conditions are assumed for the analyses. The unit weight and shear strength of the shell material are the same as those used in the STID analyses: 135 pcf and 35°.

The yield accelerations estimated for half-height and full-height slips are 0.15 g to 0.16 g. Less than 2 ft are estimated for the corresponding seismic displacement.

For the static case, predicted full-height slips marginally achieve the target FOS. Smaller-scale, localised, and relatively shallow slips are possible but FOS greater than 1.3 are predicted. Search results are shown on Figure 4.5 for smaller-scale and full-height slips.



NOTES:
1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.
2. TWO UNITS CONSIDERED IN MODEL SIMPLIFICATION

Figure 4.5 Static Stability Results for Left Bank Dam Final Grade

4.5 DISPOSAL SITES

Disposal sites are designed and developed with stable permanent slopes and suitable drainage requirements. The material placed in the disposal sites will be track-walked and graded with a bulldozer. Three disposal areas are proposed as the primary locations for material disposal: spillway, powerhouse, and an upland site.

Stability analyses of the disposal sites have been completed using the LEA and the Spencer and GLE methods of slices. The LEA considered two loading conditions: static long-term and yield acceleration (ky) determination for estimating seismic displacement. A FOS of 1.5 was targeted for static long-term stability. Yield acceleration was determined for a FOS of 1.0 and material strength was not reduced. STID-8 (PacifiCorp, 2015a) indicated displacements of 2 ft are acceptable according to a FERC guideline for the operating dam, which has been applied to the stability of the disposal sites. Seismic displacements have been approximated by two semi-empirical methods, developed by Makdisi and Seed (1978) and Bray and Travasarou (2007). The design seismic loading was defined by STID-5 (PacifiCorp, 2015b), for a MCE of 0.25 g, and an assumed magnitude 7.5 earthquake.

The analysis model geometry for the disposal sites are based on the design drawings.

Material properties, as shown in Table 4.5, are based on design gradation limits and available information collected during dam construction. Although bulking of the dam fill is expected during stripping and disposal fill placement, higher values of unit weight for the dam rockfill and core materials lower the FOS. As a result, the unit weights of the dam materials wasted in the disposal sites are maintained at their dam construction (i.e., compacted) values. The strength of the dam core material was also assumed at its compacted value. The lower-bound Leps (1970) shear-normal function was assumed for the strength of the dam rockfill and the average Leps (1970) shear-normal function was assumed for the strength of the E9/E9b cap. Dry conditions are assumed for the analyses.

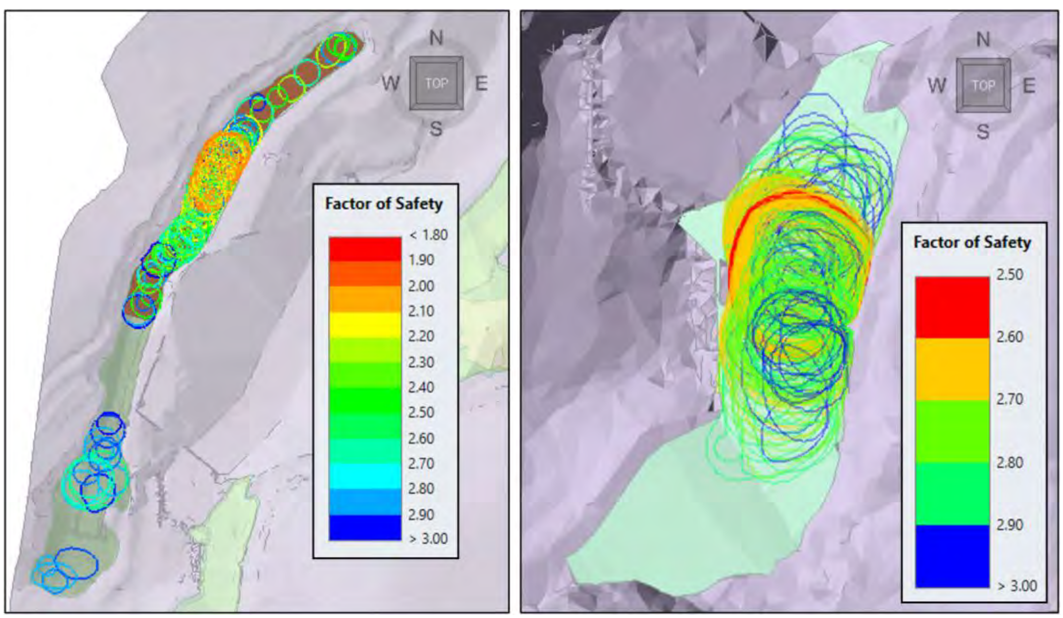
Table 4.5 Disposal Site Material Properties

Material	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core/Disposal Clay	130	22	0
Shell/Disposal Rockfill	130	Leps (1970) Lower-bound Shear-Normal Function	
E9/E9b Disposal Cap	135	Leps (1970) Average Shear-Normal Function	
Bedrock	130	Impenetrable	

The analysis results for the static case of the upland disposal site indicate target FOS is achieved. The pseudo-static analysis indicate displacements are likely to occur during the design seismic event. The slip is estimated to displace less than 2 ft.

The LEA for the spillway disposal site indicates the static FOS target is achieved with the lowest FOS slips coincident with the dam core fill placement (upstream end of the spillway). The results of the static slip search are shown on Figure 4.6. The pseudo-static screening suggests the target FOS is marginally achieved. Nonetheless, the minimum yield acceleration produced and the associated seismic displacement estimate is less than 2 ft.

Slope stability analyses for the powerhouse disposal site indicate the static FOS target is achieved. The pseudo-static analysis with strength and MCE reductions resulted in a FOS greater than 1.0, which suggests seismic displacements are not expected to be a concern.



NOTES:

1. SPILLWAY DISPOSAL SITE SHOWN IN THE LEFT IMAGE AND POWERHOUSE DISPOSAL SITE IN THE RIGHT IMAGE.
2. BROWN ENTITY (LEFT IMAGE) REPRESENTS POSSIBLE EXTENT OF DAM CORE FILL PLACEMENT.

Figure 4.6 Static Stability Result for Spillway and Powerhouse Disposal Sites

4.5.1 FOUNDATION PREPARATION

Foundation preparation of staging and disposal sites will consist of stripping, removing, and disposing of organics, soft materials, or silts.

5.0 MECHANICAL

The content of this section is summarized from the original KP memorandum "Iron Gate Diversion Tunnel Gate Memorandum" (DV-20-1543).

5.1 GENERAL

The primary mechanical scope required for drawdown at the Iron Gate facility is comprised of two components:

- Inspection, upgrades, testing and recommissioning of existing diversion tunnel control gate hoisting equipment and systems.
- Review and mechanical modifications to maintain functionality of the three fish collection ponds during the pre-drawdown works.

The sections below summarize the findings of a reconnaissance visit to the Iron Gate diversion tunnel gatehouse on June 30, 2020 and the key findings of the analysis performed to identify the gate and hoist mechanism design operating parameters.

5.2 SITE VISIT OBJECTIVE AND SYSTEM INSPECTION

5.2.1 OVERVIEW

The reconnaissance site visit to the diversion tunnel gate house was performed with the following people:

- Robert Roach – PacifiCorp
- Stuart Flett – Knight Piésold
- Alexander Manos – Knight Piésold
- ASI Marine L.P. – (ASI) Personnel

The objective of this site visit was to:

- Reach a conclusion relative to the gate's design hoisting capabilities and potential limitation based on its existing condition.
- Inspect the existing hydraulic gate hoist mechanism and associated accessible auxiliary systems.
- The following systems were accessible and were inspected:
    - Gate hydraulic power unit (HPU).
    - Gate hoist mechanism hydraulic cylinders (x2).
    - Gate hoist hydraulic accumulators.
    - Gate House Electrical Systems.
- Communicate with PacifiCorp personnel to receive additional data relative to the system operation, diversion tunnel flows and potential limitations to the system operation.
- Better understand the additional information that might be needed and the additional analysis that might be necessary to completely understand system limitations. This reconnaissance site visit allowed KP to

better understand the unknown and better identify the additional considerations for further inspection and analysis that could prove beneficial, prior to use of the gate and hydraulic mechanism during drawdown.

Considering that the actuation and testing of such a system is the prime way of checking the condition of the system components and in lack of adequate system testing and data, the visual inspection was the only means of completing this inspection and getting a better understanding of the system operation and its potential limitations. KP also studied the available provided documentation by PacifiCorp, along with additional data received by the Gate hoist mechanism manufacturer. The conclusion of the system hoisting capabilities and potential limitation are included in the final section of this Memo. The findings of KP's inspection and preliminary due diligence are presented in the following sections.

5.2.2 SYSTEM INSPECTION

5.2.2.1 HYDRAULIC POWER UNIT

The performed visual inspection showed that the hydraulic unit was in an overall exterior good condition. No visible signs of oil leaks were observed. Accessible hydraulic hoses were in good condition, and there was no indication of motor or solenoid overheating or burned cables.

5.2.2.2 HYDRAULIC CYLINDER AND HOIST RODS

The visual inspection showed that the hydraulic cylinders and rods were overall in good condition. However, Figure 5.1 indicates that there is slight cylinder rod offset when operating the two hydraulic cylinders which mandates a specific sequence of pin dogging.

Figure 5.1 Hoist Stem Pin Location

Mark-ups on the hoist rods were identified which potentially indicate past flow events which could be useful to confirm gate operation at un-balanced conditions with significant gate opening, see Figure 5.2. These flow events remain to be confirmed along with other items that need to be confirmed by PacifiCorp. Should these be accurate, these flow events indicate flows between 1,850 cfs to 2,200 cfs. These flows correspond to gate openings between 24 inch (42%) up to 36 inch (63%) based on the updated gate rating curve, developed by Black and Veatch and also found in the Gate house gate control panel.

Figure 5.2 Historical Flow Rate Recordings on Hoist Stems

5.2.2.3 HYDRAULIC ACCUMULATORS

The system battery of four 20-gallon hydraulic accumulators was inspected and the following were identified:

- All accumulators are in place, but not all seem operational.
- Though all accumulator isolation valves seemed fully open at the time of inspection and though there was no lock out tack out (LOTO) labels positioned on accumulator No3, there is an obvious leak in accumulator No3 (See Figure 5.3) which indicates a potential need to isolate this accumulator when the system is operating.

Figure 5.3 Accumulator No. 3 Leak Location

The extent of the leakage is unknown, since the system is not pressurized but it is anticipated that it will pose a risk to maintain the accumulator connected to the battery of remaining accumulators. Since the operation of the accumulator battery is to provide an emergency stroke when the main operating system is not operational, either because of and electrical failure or a mechanical failure of the pumps, it should be anticipated that the provided emergency stroke of the gate hoist mechanism will be limited due to having only 75% of the accumulator volume available. Also, the lifting capability, system reaction and the transient dampening effect provided by the battery of accumulator, during the raise, lower and stop actions of the gate actuation will also worsen, with an unknown effect to the system operation.

5.2.2.4 HPU, HYDRAULIC CYLINDER, AND GATE HOIST INSTRUMENTATION AND CONTROLS

The site investigation allowed KP personnel to better identify the extent of instrumentation and controls implemented for the HPU and gate operation:

- The hydraulic cylinder and gate hoist (support beams) mechanism incorporate instrumentation such as limit switches and position indicators. Further research in available drawings (See I&C Drawings AA-88078, AA-88081, and AA-880823 included in Appendix K) allowed KP to reach the following conclusion.

- The limit switches are primarily related with the open/close function of the gate. The limits switches allow the system to identify the fully open and fully close position of the hydraulic cylinders.
- The identified limit switches on site (called LS-1 and LS-2 in the system drawings and described as equalizer limit switches) are used to identify potential excessive offset between the two hydraulic cylinder operations. These limit switches allow system protection during the actuation of the hydraulic cylinders, when an excessive offset above or equal to 1 inch is recorded. This limit switch allows an HPU function (by energizing HPU solenoids) which allow hydraulic cylinder individual separate operation when an offset is identified during the cylinder opening and/or closing function.
- The position indicator identified on site potentially allows gate regulation in intermediate positions.

- The HPU instrumentation seemed to be limited on pressure gauges and pressure switches mounted on the hydraulic piping of the HPU system. It is anticipated that other instrumentation is also available but not possible to inspect and identify considering the “closed box” arrangement of the typical HPU systems, when compared with other piping systems.
- Digital instrumentation recording of system pressure and limits, hydraulic cylinder position limits were not obvious and visible and could potentially be non-existent or not hooked up on a central PLC or the system DCS. The latter (connection the plant DCS system) remains to be confirmed since a site visit to the plant DCS system was not feasible.
- However, it would be recommended that all instrumentation that is not digital with a visual indication and recording needs to be converted to digital instrumentation and all signals need to be hooked up to a PLC or DCS system for monitoring of gate operation. This will allow for a better control of the gate and its operating parameters during the drawdown period.

5.2.2.5 DIVERSION GATEHOUSE ELECTRICAL SYSTEMS

The visual inspection did not identify any items of major concern for the gatehouse electrical system, however the following observations were made:

- The HPU pumps are all mounted on one motor, which might be an issue during future extended operation of the unit. The motor is a continuous duty running motor that is coupled with both HPU pumps. Each pump serves a different operating purpose. The smaller of the two pumps cannot be necessarily considered as standby of the main larger pump which drives the gate hoist mechanism and hydraulic cylinders. The fact that the system does not have a typical HPU arrangement with a stand-by pump/motor available might present limitation in future operations and needs to be addressed.
- The system electrical circuit (see Figure 5.4) shows the existence of a system pressure switch (location unknown) that allows and cuts the pump power supply under a high-pressure condition. The location of this pressure switch in the hydraulic circuit could not be identified with ease or certainty. This would need to be identified to better understand the system operation and the available protections that effect system operation.

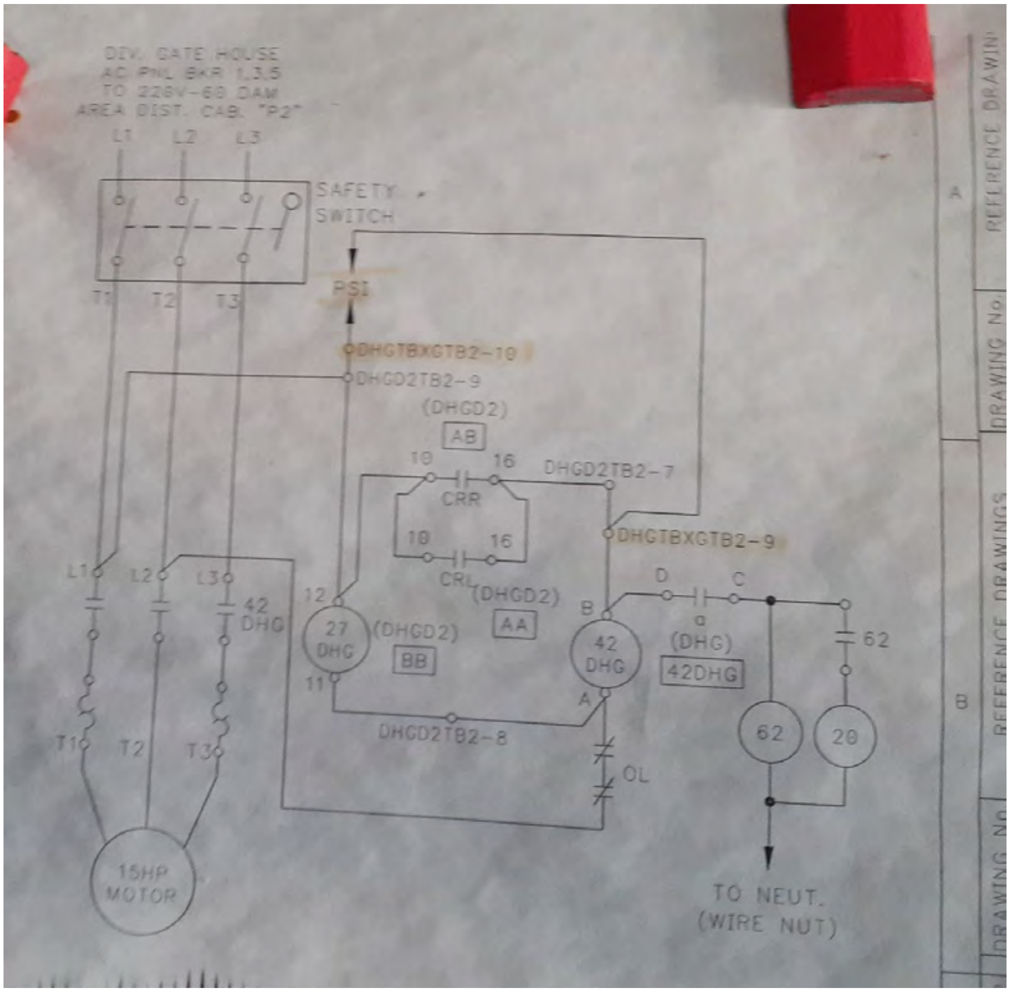


Figure 5.4 Electrical Schematic

KP's current understanding of the system specifications and limitations, in lack of existing one- and three-line diagrams and wiring diagrams, are as follows:

- The system electrical supply seems to include a main 200A low voltage 240/120 V AC power supply that connects with a UPS system. The UPS system provides an alternative source of DC power supply for the system DC instrumentation and could potentially be used during a power outage incident to maintain I&C DC power supply.
- The UPS system batteries could not be identified and visually inspected. The UPS system would not provide electrical supply redundancy for the AC HPU motor and other AC loads.
- It would be considered mandatory that the UPS system voltage and battery condition and their capacity is measured and evaluated for future use.
- An alternative source of AC supply, such as a diesel generator should be considered for the future extended operations during drawdown.

In lack of AC supply and pumping capacity the use of the hydraulic hand pump can be considered to charge the accumulator battery and actuate the hydraulic cylinders. This should be used only in an emergency and in lack of AC electrical supply but can be an option.

5.2.3 COMMUNICATION WITH PACIFICORP ON SYSTEM OPERATION

According to information provided by the PacifiCorp personnel (Robert Roach - Senior Environmental Analyst) the maximum gate opening recorded on the rating curve found in the gate house, at a gate opening between 23.5 inch and 24.9 inch is not necessarily related with the Gate and Gate hosting mechanism limitations nor with limitations of flow running through the diversion tunnel.

- The maximum gate opening recorded on the rating curve found in the gatehouse, at a gate opening between 23.5 inch and 24.9 inch is most possibly related with the unit maximum flow which is about 1,700 cfs to 1,800 cfs and needs to be bypassed during a powerhouse shutdown. As confirmed with the available drawings. The diversion gate HPU system is interlocked with the power conduit intake gate.
- From an environmental aspect, it was conveyed that the minimum tailrace environmental flow, at all times, needs to be maintained between a minimum of 700 cfs up to a typical 1,300 cfs.
- The above system requirements have limited the testing of the diversion tunnel from achieving maximum gate opening in unbalanced conditions.
- As such the above flows are the anticipated unbalanced head flows that the gate and the tunnel has typically been subjected to.
- A full gate opening at current static head would exceed the above flow and regulatory and environmental requirements.

Separately from the information received from the PacifiCorp personnel, by analysing the available gate rating curves, it seems that the operating limit assigned on the gate between a 23.5 inch and 24.9 inch of gate opening might also be related to the maximum recorded tunnel flows which are about 2,000 cfs at maximum static head.

5.2.4 OPEN ITEMS AND NEXT STEPS

With regards to the site inspection and assessments of equipment conditions, the following conclusions are made:

- It is not possible to know the limitations of the existing gate and hoist mechanism actuation since gate actuation to its full opening has not been recorded in recent years.
- System component replacement and system setpoint modification (pressure setpoints) might have been performed, which could affect the anticipated system operation behavior when compared with the original design component specifications. The original design specifications were provided by the HPU manufacturer and these could be further checked with the current system components.
- O&M information relative to the performed and recommended Gate and HPU/Hydraulic Cylinder operations (such as frequent system actuation, as per manufacturer's recommendations) and system maintenance (Electrical Supply/UPS system inspections and checks, I&C Checks, Oil conditioning and replacement) is limited. The significance, however, of this item to the performance of the system is high and such further information would be necessary.

- Considering that the system is comprised of a variety of components most of which are not accessible due to their submergence, the assessments made as a result of the site visit are subject to change based on additional data received by PacifiCorp or from future necessary system inspections. Future inspection should be especially focused on the non-accessible, submerged system components (gate rods, rod supports and connection couplings, pin condition, gate stem connection condition, gate body and specifically concrete condition, gate sealing and seating surface condition, gate roller and gate rolling/sliding surface condition), additional sealing performed on the submerged gate sealing surfaces during past gate operation attempts and other equivalent non accessible gate components that can affect the system operation and hinder smooth gate operation.
- The system operation overall would need to be reviewed with PacifiCorp operator's independent of the results of the reconnaissance site visit and the preliminary system analysis. A training session for the use of the system prior to operations would be deemed necessary if not mandatory for safe system operations. The option of consulting the original manufacturer's (MacMillin's) personnel or a third party certified HPU company to provide further insight to system operation and its limitations would also be advised.

5.3 OPERATIONAL ANALYSIS OF GATE AND HOIST MECHANISM

5.3.1 PAST INSPECTION DATA

KP performed a review of available data inspection reports and other documents that allowed for determination of past operations of the hydraulic gate hoist with the associated gate openings and flows.

Several tests were performed under balanced and unbalanced head conditions and they are summarized in Table 5.1 that includes the gate opening with comments on the results of the test.

Table 5.1 Diversion Tunnel Gate Testing and Inspection Summary

Date	Opening Height	Head Conditions	Comments
12/13/2007	15 inch (x2) 27 inch	Balanced Head	The Gate was tested twice being lifted 15 inch each time to evaluate the return position of the gate. The gate was opened on the third and final time to 27 inch to inspect the sealing surface between the upper and lower portions of the gate.
03/16/2010	2.25 inch	Unbalanced Head	The gate was limited to a 2.25 inch opening due to the limit switches preventing further travel. A flow of 50 cfs was recorded passing through the diversion tunnel during the one minute of flow.
02/08/2019	48 inch	Balanced Head	The gate was manually opened using the HPU to the height of 48 inch, but a pressure burst disc had ruptured during the closure of the gate. This test was repeated after the rupture disc was replaced and the system repaired. The final test was done from the Iron Gate Powerhouse Control Room where the SCADA system was used to open the gate to the 48 inch height. The gate was then manually closed due to the operating conditions that restricted the SCADA System from closing the gate.

5.3.2 COMMISSIONING DATA

The original design of the tunnel included only the split diversion gate which is currently operated from the diversion tunnel gatehouse. In 2009, an orifice structure was installed directly downstream of the gate which affects the gate rating curve and associated flows that are developed through the tunnel.

The original gate rating curve as developed during the original system commissioning in 1964, during the filling of the Iron Gate reservoir is presented in Figure 5.5.

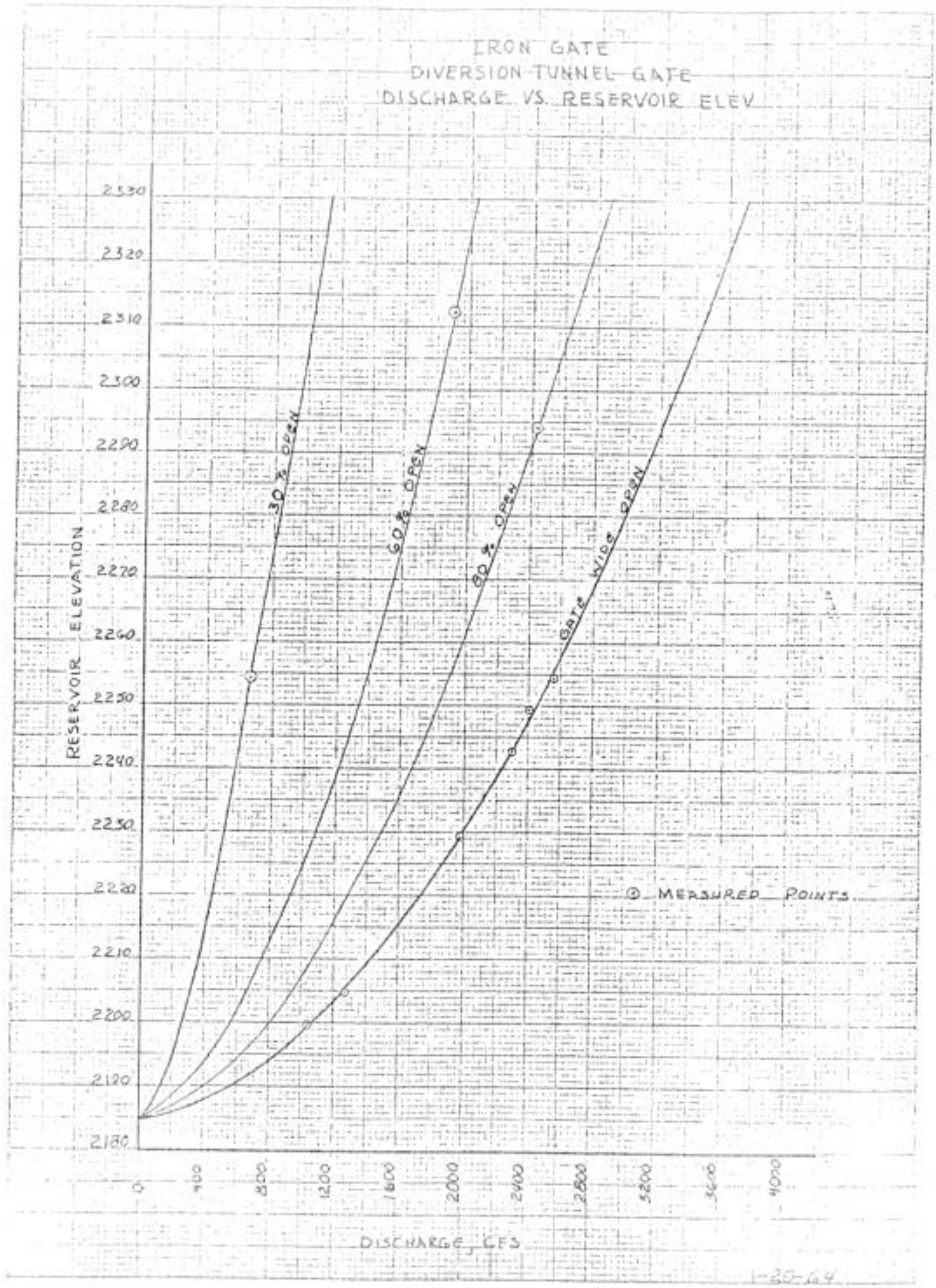


Figure 5.5 Original Iron Gate Rating Curve

The above curve presents the rating curves developed by extrapolation of the nine (9) measured points (gate openings and flow), as measured in the field during this commissioning phase of the project in 1964. Table 5.2 summarizes the measured values of flow and gate openings, prior to the installation of the orifice downstream of the gate in 2009.

Table 5.2 Iron Gate Openings and Measured Flow (Prior to 2009)

Gate Opening			0 "	3 "	6 "	9 "	12 "	15 "	17 "	21 "	24 "	27 "	30 "	34 "	42 "	46 "	54 "	57 "
			0%	5%	11%	16%	21%	26%	30%	37%	42%	47%	53%	60%	74%	80%	95%	100%
Pool Level: 2328.0	100% Nominal Head	140 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Pool Level: 2312.0	89% Nominal Head	124 ftWG	X	X	X	X	X	X	X	X	X	X	X	2000 cfs	X	X	X	X
Pool Level: 2294.0	76% Nominal Head	106 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	2510 cfs	X	X
Pool Level: 2254.0	47% Nominal Head	66 ftWG	X	X	X	X	X	X	700 cfs	X	X	X	X	X	X	X	X	2590 cfs
Pool Level: 2249.0	43% Nominal Head	61 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	2400 cfs
Pool Level: 2242.5	39% Nominal Head	54 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	2300 cfs
Pool Level: 2229.0	29% Nominal Head	41 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	1995 cfs
Pool Level: 2204.0	11% Nominal Head	16 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	1250 cfs
Pool Level: 2195.0	5% Nominal Head	7 ftWG	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	1050 cfs

By evaluating the measured data provided in the system original curve it is apparent that:

- **The gate has been opened at 100%** of its full opening, equal to about 57 inch (4 ft 9 inch) at a recorded pool level of about 2,254 ft pool level, a static head of 61 ft above the upper gate CL, between 40% to 45% of the current nominal head, establishing maximum flow of 2,550 cfs.
- **The maximum static head and pool elevation that the diversion gate has been subjected to, is about 124 ft**, and this is at pool elevation of 2,312 ft. This equals to about **90% of the current nominal head**. The flow established with a gate opening of 60% was about 2,000 cfs.
- **Higher flows than 2,590 cfs under a static head have not been recorded in the tunnel**.

This original rating curve was adjusted after 2009 by Black and Veach after the installation of the orifice, accounting for the increased head losses. The adjusted rating curve is presented in Figure 5.6 as developed by Black and Veach. This can also be found in the diversion tunnel gatehouse, on the gate operating panel. Figure 5.7 provides the current rating curve for the diversion tunnel.

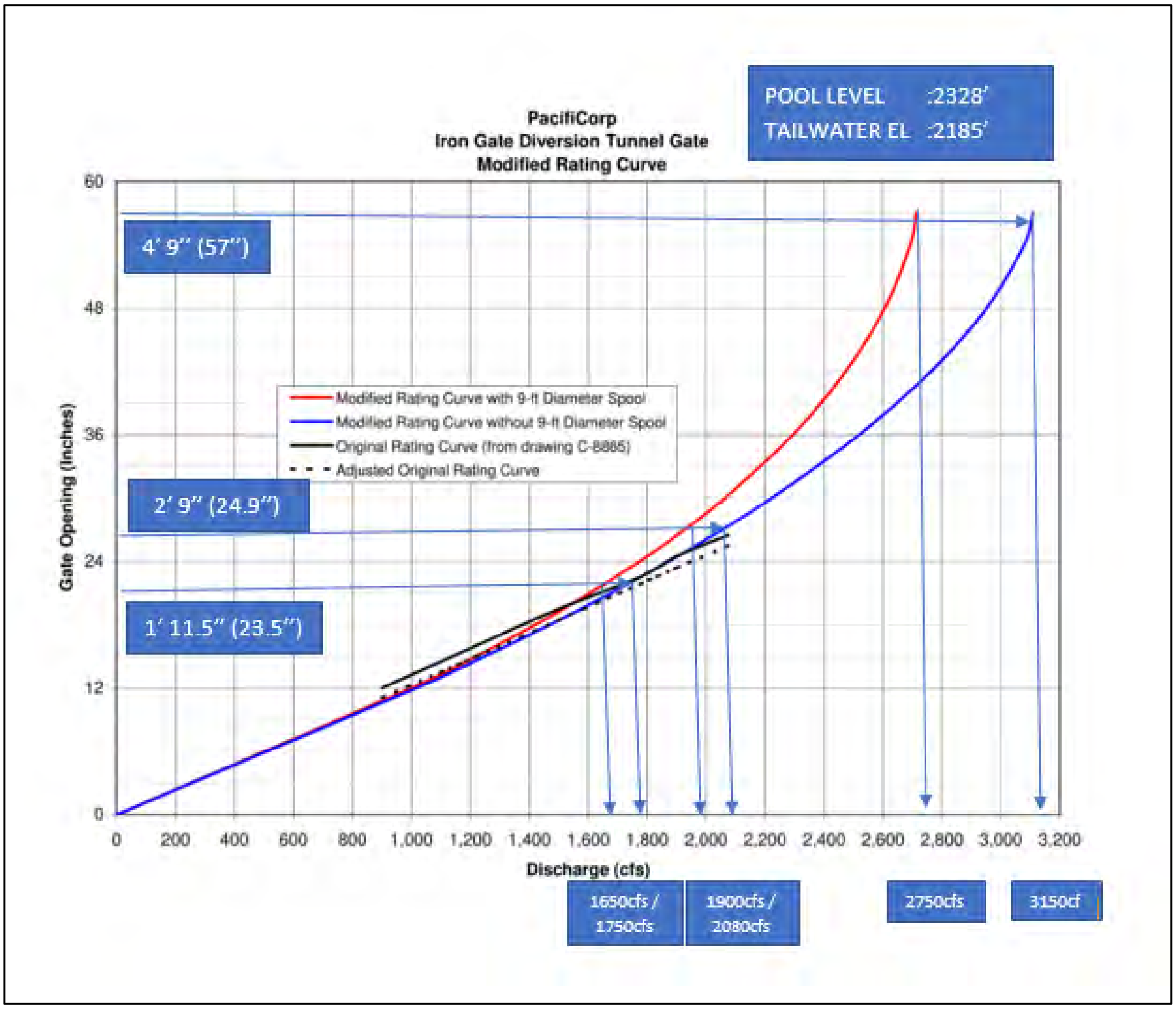
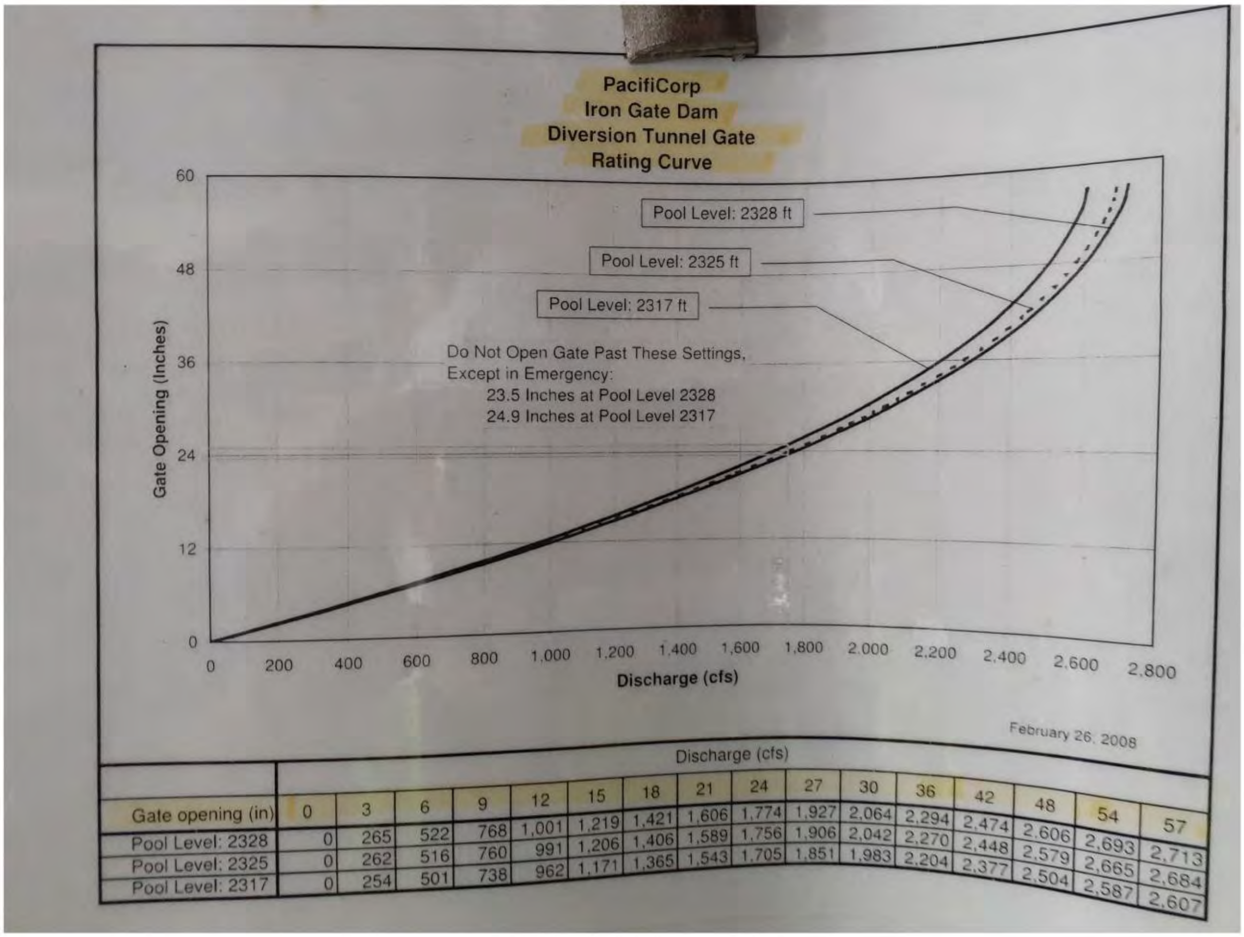


Figure 5.6 **Modified Operational Rating Curve After Installation of Blind Flange**



Gate opening (in)	0	3	6	9	12	15	18	21	24	27	30	36	42	48	54	57
Pool Level: 2328	0	265	522	768	1,001	1,219	1,421	1,606	1,774	1,927	2,064	2,294	2,474	2,606	2,693	2,713
Pool Level: 2325	0	262	516	760	991	1,206	1,406	1,589	1,756	1,906	2,042	2,270	2,448	2,579	2,665	2,684
Pool Level: 2317	0	254	501	738	962	1,171	1,365	1,543	1,705	1,851	1,983	2,204	2,377	2,504	2,587	2,607

Figure 5.7 PacifiCorp Operational Tunnel Rating Curve

Attachments:

1 – Memo: “Iron Gate Low-Level Outlet Survey Data Acquisition and Processing” (Yurok Tribe, December 11, 2020).

2 – Iron Gate Dam Facility Simulated Drawdown

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YUROK TRIBE

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EXTERNAL TECHNICAL MEMO

Date: December 11th, 2020

To: Erik M. Esparza, PE
Design-Build Coordination Manager
Kiewit Engineering Group, Inc.
200 Columbia House Blvd.
Vancouver, WA 98661

Isaac Bukoski
Engineer/Scientist
Knight Piésold and Co.
1999 Broadway, Suite 900
Denver, CO 80202

From: Cort Pryor
Survey Manager
Yurok Tribe Fisheries Department – Design & Construction Division

RE: Iron Gate Low-Level Outlet Survey Data Acquisition and Processing

Introduction

The Kiewit Engineering Group and Knight Piésold and Company tasked the Yurok Tribe Fisheries Team (Yurok Tribe Design Construction Division and Points West Surveying) with conducting a survey of the downstream portion of the Iron Gate Dam Low-Level Outlet Structure for the purposes of documenting existing conditions and to provide data for future design modification.

This technical memo documents data acquisition and processing activities related to this task.

Acquisition Details

The survey of the downstream portion of the Low-Level Outlet Structure was conducted between November 17th and November 20th of 2020. The following organizations and representatives were onsite for the survey activities:

- Denny Campbell – PacifiCorp Representative
- Anthony Middleton – Kiewit Representative/Estimator
- Sam Bush – Knight Piésold Representative/Manager Technician
- Mike White – Smokin' Fire Productions Representative/Technical Rescue Specialist
- Cort Pryor – Yurok Tribe Representative/Survey Manager and Technical Lead
- Michael Pulley – Points West Surveying Representative/Licensed Land Surveyor

Details of daily activities performed by the Yurok Tribe Fisheries Team is as follows:

November 17th

- Participated in confined space training at the Holiday Inn Express, Yreka CA. Training led by Mike White of Smokin' Fire Productions,
- Mobilized to project site and establish GNSS base receiver at GMA-205,
- Set temporary spikes CP 100 and CP 101 using RTK ties to GMA-205. Recovered mag nail, CP 102, in painted air target; RTK tie to GMA-205,
- Recovered NGS HPGN monuments MX1299 and AF8315. Performed GNSS static ties to GMA-205, and
- Returned to project site and performed a second set of RTK ties to CP 100, CP 101, and CP 102.

November 18th

- Site safety briefing and lock out tag out of low-level outlet,
- Performed total stations observations of CP 101 and CP 102 from CP 100 to verify distances and angles,
- Establish control point CP 100 at outlet structure portal entrance,
- Establish additional survey control within outlet structure and performed recon of the structure, and
- initiated bathymetric survey of the outlet structure.

November 19th

- Site safety briefing,
- Continue and completed bathymetric survey of outlet structure,
- Initiated and completed above water scan of outlet structure, and
- Released lock out tag out of low-level outlet.

November 20th

- Perform additional set of distance and angle observation on CP 101 and CP 102 from CP100. No access to CP 200 or additional control points within the outlet structure allowed.

Table 1. summarizes daily survey activities and Figure 1. provides an overview of the survey area.

Table 1: Data Acquisition Dates and Daily Work Activity

Date	Activity	Comments
November 17, 2020	Confined Space Training Establish Survey Control at Project Site	Training Provided by Smoke Fire Productions Static Ties to NGS Control Points RTK ties to Temporary Control Points
November 18, 2020	Establish Survey Control in Outlet Structure Begin Bathymetric Survey	Total Station Survey
November 19, 2020	Complete Bathymetric Survey Initiate and Complete Above Water Survey	Total Station Survey Mobile Ground-Based LiDAR Survey
November 20, 2020	Finalize Site Survey Control Demobilization	Total Station Survey (No Outlet Structure Entry)

Figure 1: Iron Gate Dam Low-Level Outlet Location Map

Geodetic Control

The horizontal datum is based on NAD83 HARN (1991.35) and the vertical datum is NAVD88, Geoid 09. Data are projected in California State Plane Zone 1 and units are US Survey Feet (USft). Three existing monuments (GMA-205, MX1299, and AF8315) and seven temporary control points were utilized during the survey effort. The basis of the survey were National Geodetic Survey (NGS) Monuments MX1299 and AF8315. Northing, Easting, and Elevation values were held at these two monuments and static ties were performed to GMA-205. GMA-205 is a previously established control point used during the Iron Gate Reservoir bathymetric and LiDAR survey effort. All temporary control points were established from GMA-205 using multiple RTK observations and or total station observations. Survey equipment consisted of Trimble R-10 Model 2 GNSS receivers, a Trimble S-5 robotic total station, and a Leica TCRP 1203 robotic total station.

Photo 1: Trimble S-5 on CP 100 Reviewing Survey Control

Low-Level Outlet Reconnaissance

A reconnaissance of the low-level outlet structure was conducted on November 18th when survey control was being established within the structure. The initial scope of work indicated that single-beam sonar would be used to survey the bathymetric portion of the tunnel however during the reconnaissance it was determined that a conventional total station survey of the bathymetric portion of the tunnel would be most appropriate. The decision was based on the following factors:

- Significant amounts of aerated water were present in the upper third of the structure,
- Suspended material in the water column caused by moving through the tunnel,
- Detail of the concrete structure below the flange could only be obtained through a manual survey, and
- Time constraints related to data acquisition and subsequent processing.

Conventional Total Station Survey

Bathymetric data were collected using wet wading techniques and a Trimble S-5 robotic total station (Photo 2). The survey focused on defining the concrete structure immediately below the flange as well as defining the general tunnel shape with adequate detail to support future engineering design and modification work. The tunnel surface consists of irregular angular rock in areas where not overlain by concrete. Survey data was collected as high up on the tunnel wall as possible but was limited by the vertical and irregular shape of the tunnel walls. Limited data was collected on the outlet weir structure as this feature had been surveyed in a previous survey effort and much of the weir would be captured in the above water survey.

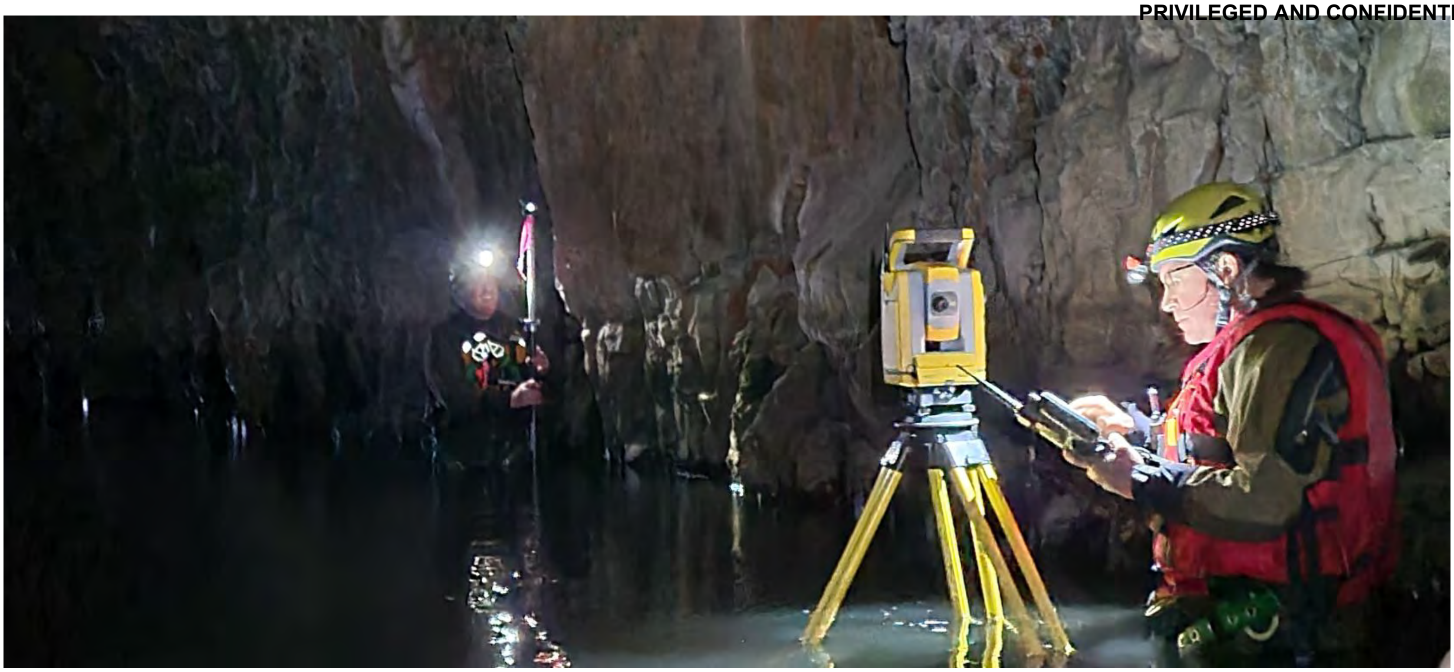

Photo 2: Bathymetric Data Collection Using Trimble S-5 Robotic Total Station

Mobile Ground-Based LiDAR Survey

A mobile ground-based LiDAR scan was conducted to characterize the dry portion of the outlet structure. The survey was performed using a GeoSLAM Zeb Horizon mobile LiDAR scanner. Unlike tradition ground-based scanning which is conducted from fixed stations and requires registration targets, the Zeb Horizon utilizes a three-dimensional Simultaneous Localization and Mapping (SLAM) algorithm which allows the scanner to be transited through the survey area. Three separate and overlapping loops were performed:

- Shallow Rock Fall (station 07+60.00) to Upstream Flange (station 04+98.17)
- Shallow Rock Fall (station 07+60.00) to Outlet Weir (station 10+60.00)
- Outlet Weir (station 10+60.00) to Downstream Riffle (station 10+98.5)

The scanner was transited through the tunnel using both wading and boat-based techniques. While transiting the outlet structure several smaller closed loops and occupations of known control were made to ensure proper registration of the dataset.

DATA PROCESSING

Geodetic Control

Data collected during the control survey was processed in Trimble Business Center Advanced version 5.32. Processing included evaluating data for erroneous instrument and rod heights as well as checking prism constants, evaluating multiple sets of angles and processing static GNSS baselines. Coordinates for survey control utilized or established during the survey effort are shown in Table 2.

Table 2: Survey Control Coordinates Utilized during the IG Low Level Outlet Structure Survey

ID	Northing (USft)	Easting (USft)	Elevation (USft)	Comment
MX1299	2596377.55	6400821.98	2548.28	Found HPGN CA 02 02 Survey Disk Held
AF8315	2531271.60	6402499.01	2662.08	Found HPGN D CA 02 UG Survey Disk Held
GMA-205	2587906.10	6443500.22	2464.33	Found Rebar w Cap Good Cond Established New Coords
CP_100	2587642.26	6442676.98	2199.73	Set Spike
CP_101	2587578.20	6442574.50	2199.27	Set Spike
CP_102	2587966.81	6442734.73	2192.78	Found Magnail
CP_200	2588063.07	6442665.91	2178.90	Found Bolt 1
CP_300	2588074.20	6442665.79	2181.00	Found Bolt 2
CP_686	2588286.98	6442727.03	2180.84	Found Lower Rod Thread
CP_250	2588342.84	6442768.82	2179.23	Set Spike

Conventional Surveys

Conventional topographic surveys were processed in Trimble Business Center Advanced version 5.32. Processing included: verifying values for geodetic control, verifying and modifying rod heights, verifying and modifying point codes, and sorting the data to various layers. A Triangulated Irregular Network (TIN) was constructed utilizing the points and feature breaklines and reviewed for consistency with field observations (Figure 2).

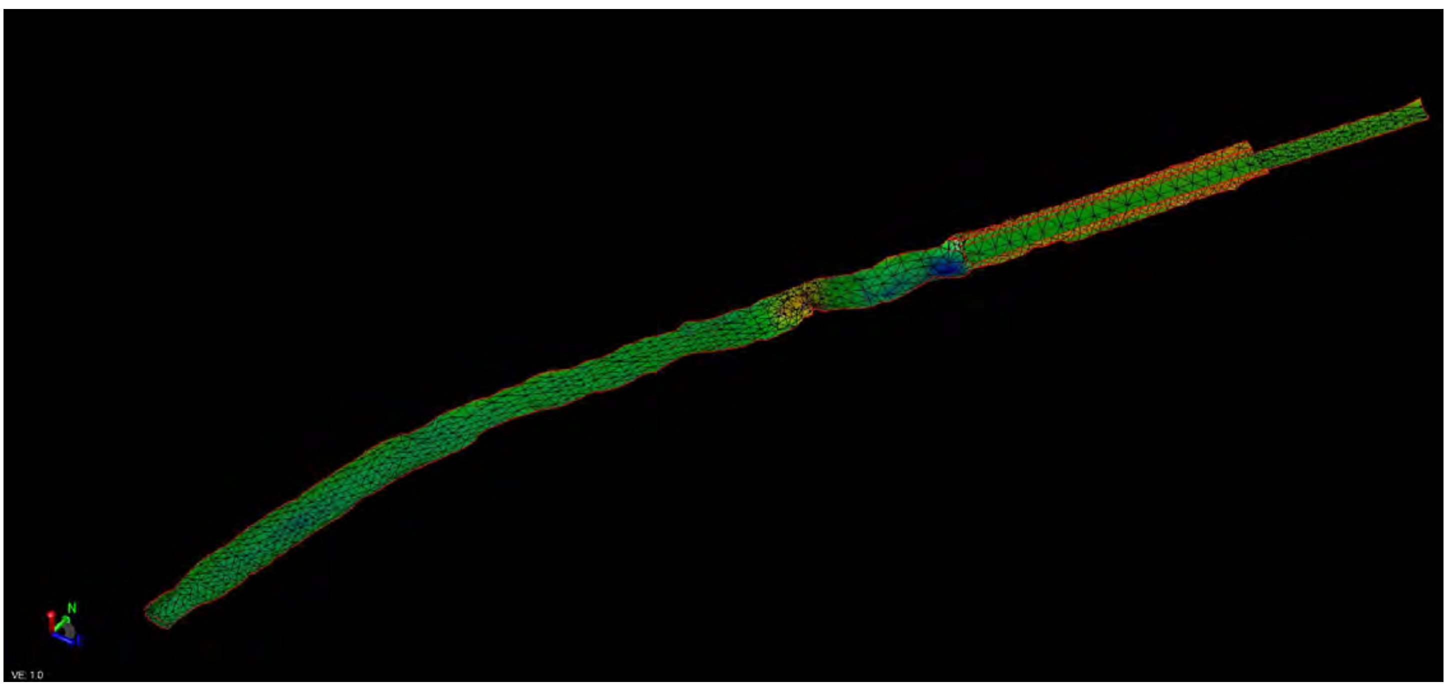

Figure 2: Isometric View of the TIN Surface Created from the Bathymetric Survey Points

Mobile Ground-Based LiDAR Survey

Initial processing of the ground-based LiDAR scan data occurred in GeoSLAM Hub version 6.1. Processing in Hub is an iterative process where the local and global SLAM algorithm is applied and refined during each iteration. After adequate values for convergence, rigidity, range and various other parameters are developed the stitched datasets are registered to site control and exported for further processing. Processing of the ground-based LiDAR scan data continued in CloudCompare version 2.12. Processing in CloudCompare includes:

- Verifying and refining registration of individual scans using pairs of equivalent points and Iterative Closest Point (ICP) registration algorithm,
- Refining registration of scans to survey control using pairs of equivalent points, and
- Cleaning of the point cloud using manual, automated, and statistical techniques.

All scans registered to within 0.10 ft of utilized survey control/reference locations in the Northing, Easting, and Elevation. An example of the cleaned ground-based LiDAR point cloud is shown in Figure 3

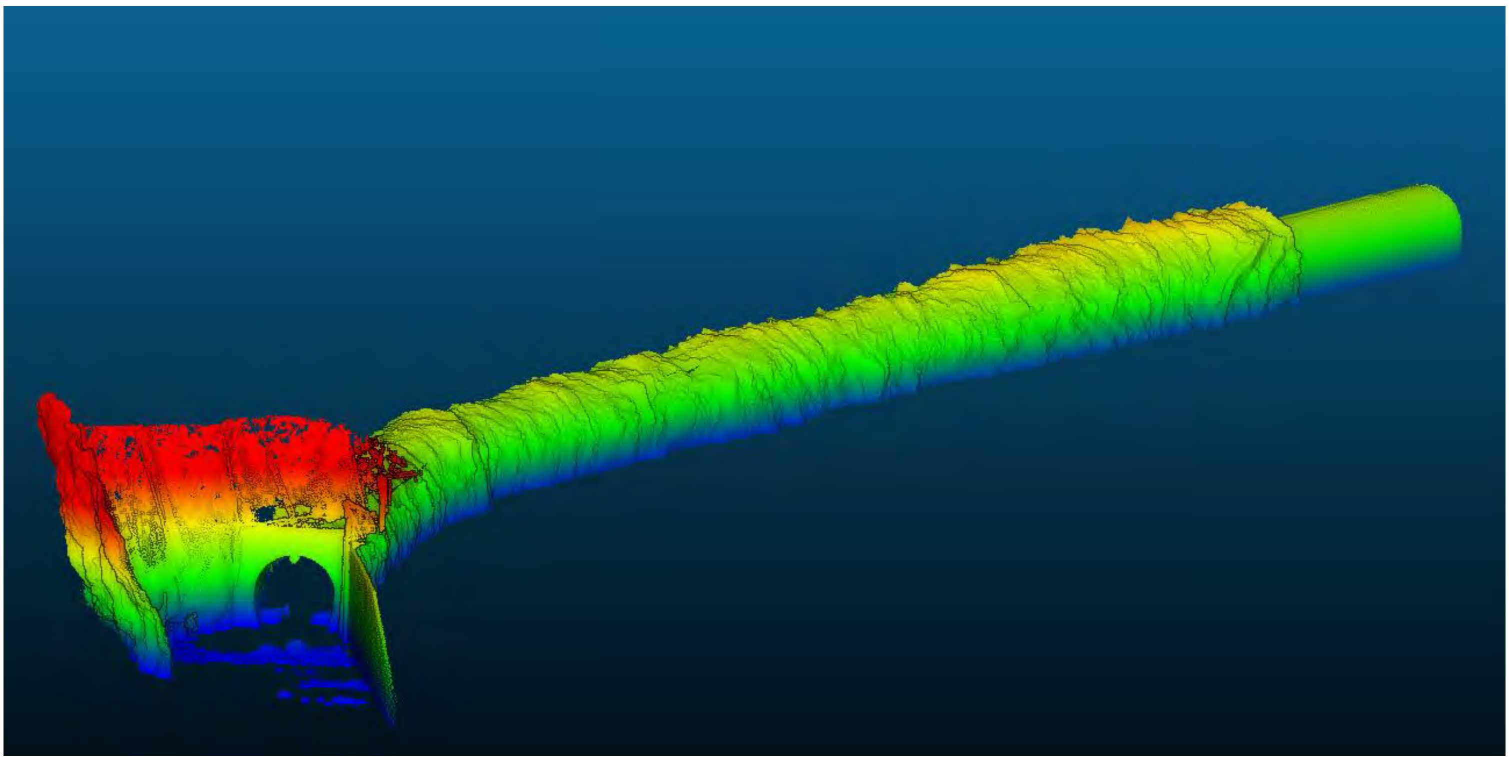

Figure 3: Isometric View of the Low-Level Outlet Tunnel Structure

Data Integration and Product Development

Data integration and product development occurred in both Trimble Business Center Advanced version 5.32 and CloudCompare version 2.12. The following describes the workflow:

Trimble Business Center Advanced

- Finalize bathymetric TIN surface,
- Import final ground-based LiDAR point cloud and verify conventional survey data and point cloud alignment,
- Modify provided horizontal alignment below station 04+90.00 to match survey data,
- Development cross section alignments along horizontal alignment at 10.00 ft intervals and at structural transitions,
- Extract bathymetric cross sections and centerline profile from TIN surface,
- Export cross section alignments to CloudCompare (skip to next section),
- Import extracted tunnel sections from CloudCompare and clean,

- Merge bathymetric sections and tunnel sections using a straight line to connect the appropriate end points, and export requested products/deliverables.

CloudCompare

- Import horizontal and cross section alignments,
- Extract tunnel sections along cross section alignments, and
- Export extracted tunnel sections to Trimble Business Center Advanced (back to previous section).

An example of the extracted tunnel sections is shown in Figure 4.

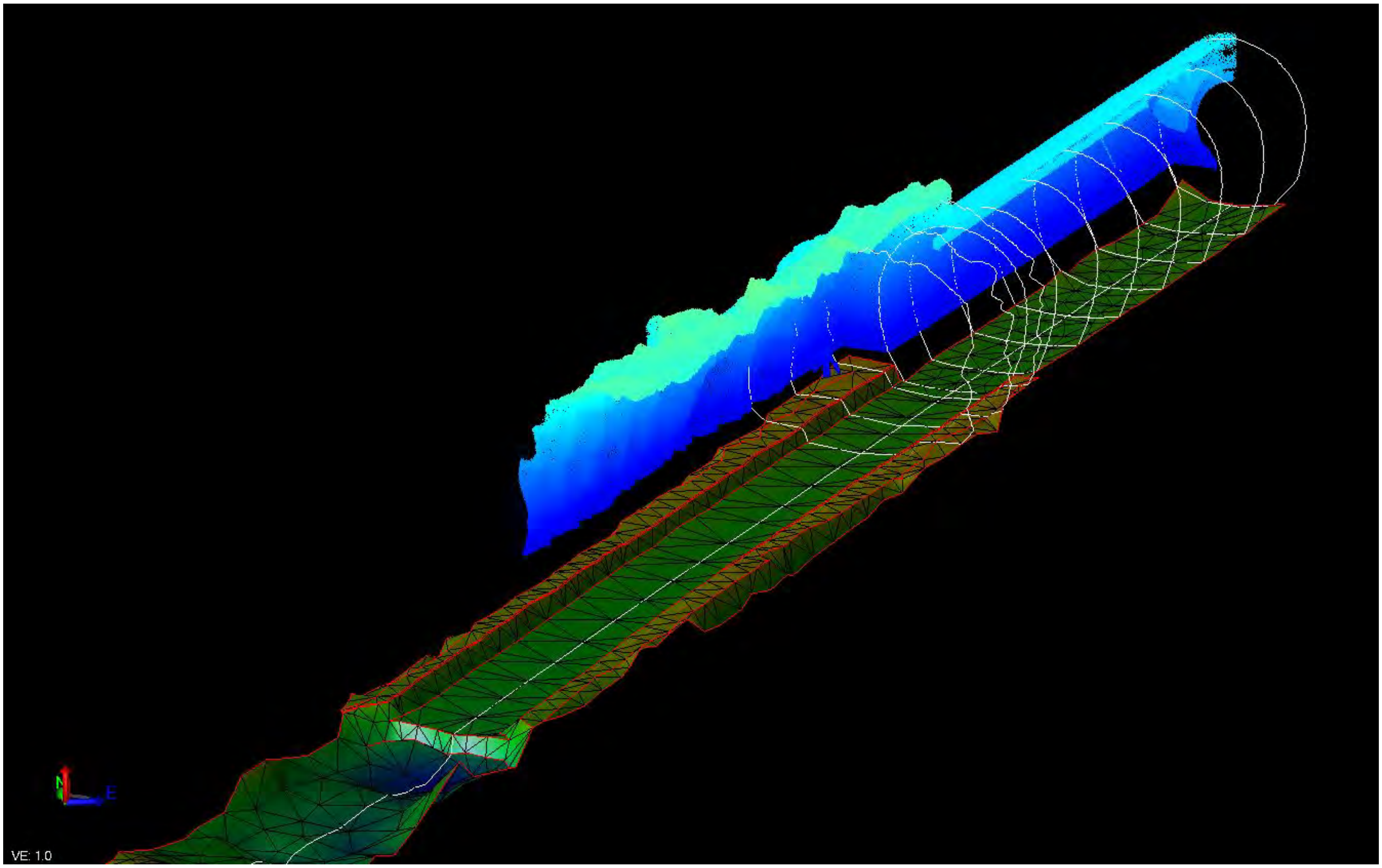

Figure 4: Isometric View of Bathymetric TIN, Outlet Structure Point Cloud and Extracted Features

Deliverables

All deliverables have been provided electronically and were uploaded to the Knight Piésold Serv-U File Share portal.

The following initial set of deliverables were provided on December 16th, 2020:

- IG_LowLevelOutlet_Scan_Pts – Full resolution kinematic ground-based LiDAR point cloud (laz format),
- IG_LowLevelOutlet_Scan_Pts_SubSample_0pt10ft – Sub-sampled (0.10 USft) kinematic ground-based LiDAR point cloud (laz format),
- IG_LowLevelOutlet_Bathymetric_Surface – sub-aqueous total station-based TIN surface (Land XML format),
- IG_LowLevelOutlet_Alignment_Modified – Modified Horizontal alignment (Land XML format),
- IG_LowLevelOutlet_CrossSection_Alignments – Planimetric cross-section locations along alignment at 10-ft intervals. Additional sections where necessary (dxf format)
- IG_LowLevelOutlet_Extracted_Scan_CrossSections – 3-D cross-sections of the dry tunnel area extracted from the sub-sampled LiDAR point cloud at the cross-section alignment locations (dxf format),
- IG_LowLevelOutlet_Extracted_Bathymetric_CrossSections – 3-D cross-section of the bathymetric portion of the tunnel extracted from the bathymetric surface at the cross-section alignment locations (dxf format), and
- IG_LowLevelOutlet_Extracted_Alignment_Profile – 3-D profile of the bathymetric portion of the tunnel extracted from the bathymetric surface along the modified alignment (dxf format).

An additional set of deliverables requested by Knight Piésold was uploaded on December 28, 2020:

- IG_LowLevelOutlet_Scan_Pts_Bathy_Pts_Combined– Full resolution kinematic ground-based LiDAR and total station bathymetric point cloud (laz format),
- IG_LowLevelOutlet_Scan_Pts_SubSample_0pt10ft_Bathy_Pts_Combined– Sub-sampled (0.10 USft) kinematic ground-based LiDAR and full resolution total station bathymetric point cloud (laz format),
- IG_LowLevelOutlet_Extracted_Scan_Bathy_CrossSections_Combined– 3-D cross-sections of the dry and wet areas of the tunnel extracted from the sub-sampled LiDAR point cloud and the full resolution bathymetric surface at the cross-section alignment locations (dxf format),
- IG_LowLevelOutlet_Bathy_Pts– Full resolution total station bathymetric survey points (dxf format),
- IG_LowLevelOutlet_Infrastructure_Pts – Survey points collected at outlet weir used as checks on scan (dxf format), and
- IG_LowLevelOutlet_SurveyControl_Pts– Survey control utilized and established during the survey effort (dxf format).

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

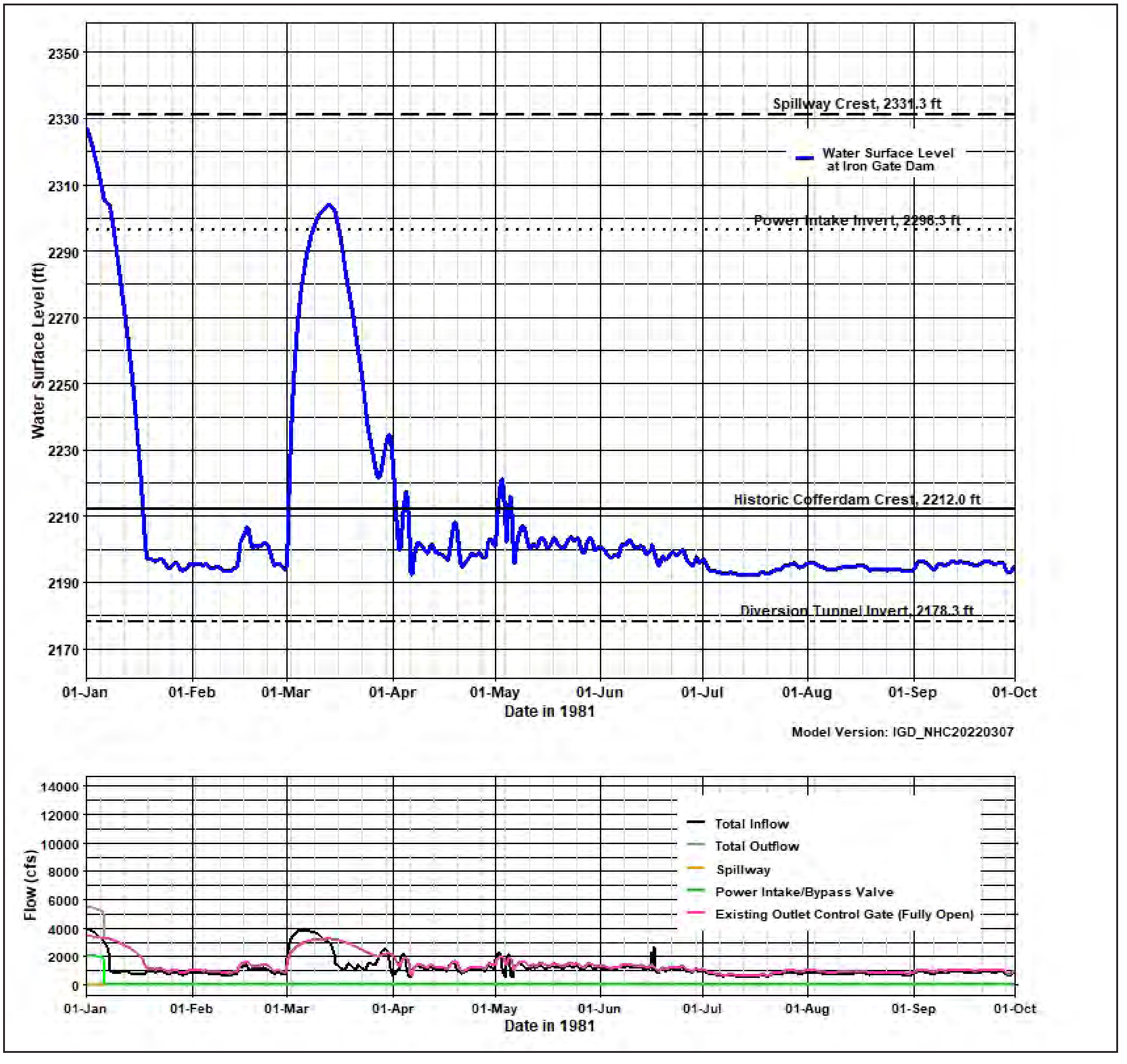

Figure 1 - Iron Gate Dam Facility Simulated Drawdown - Year 1981

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

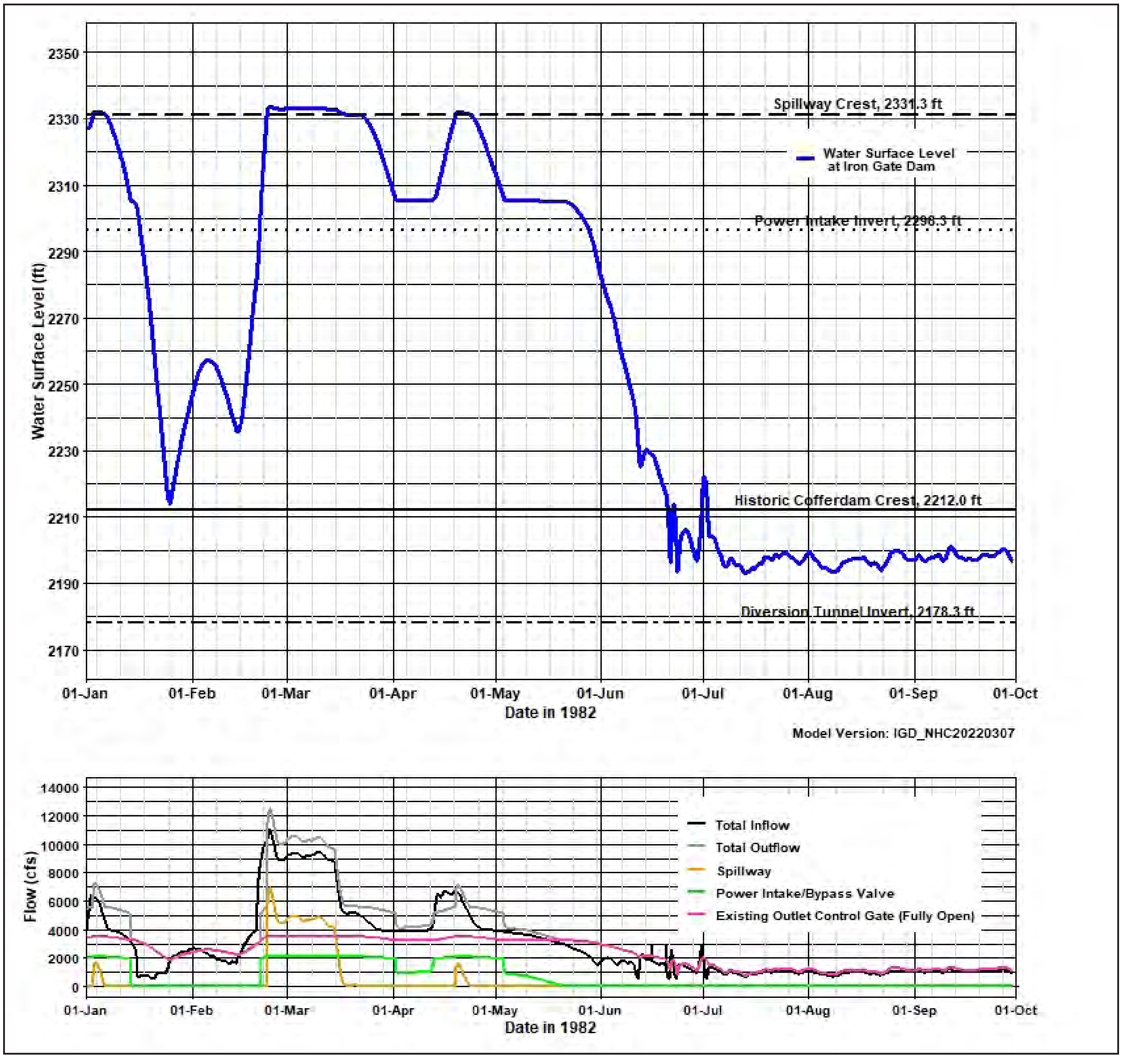

Figure 2 - Iron Gate Dam Facility Simulated Drawdown - Year 1982

April 22, 2022

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

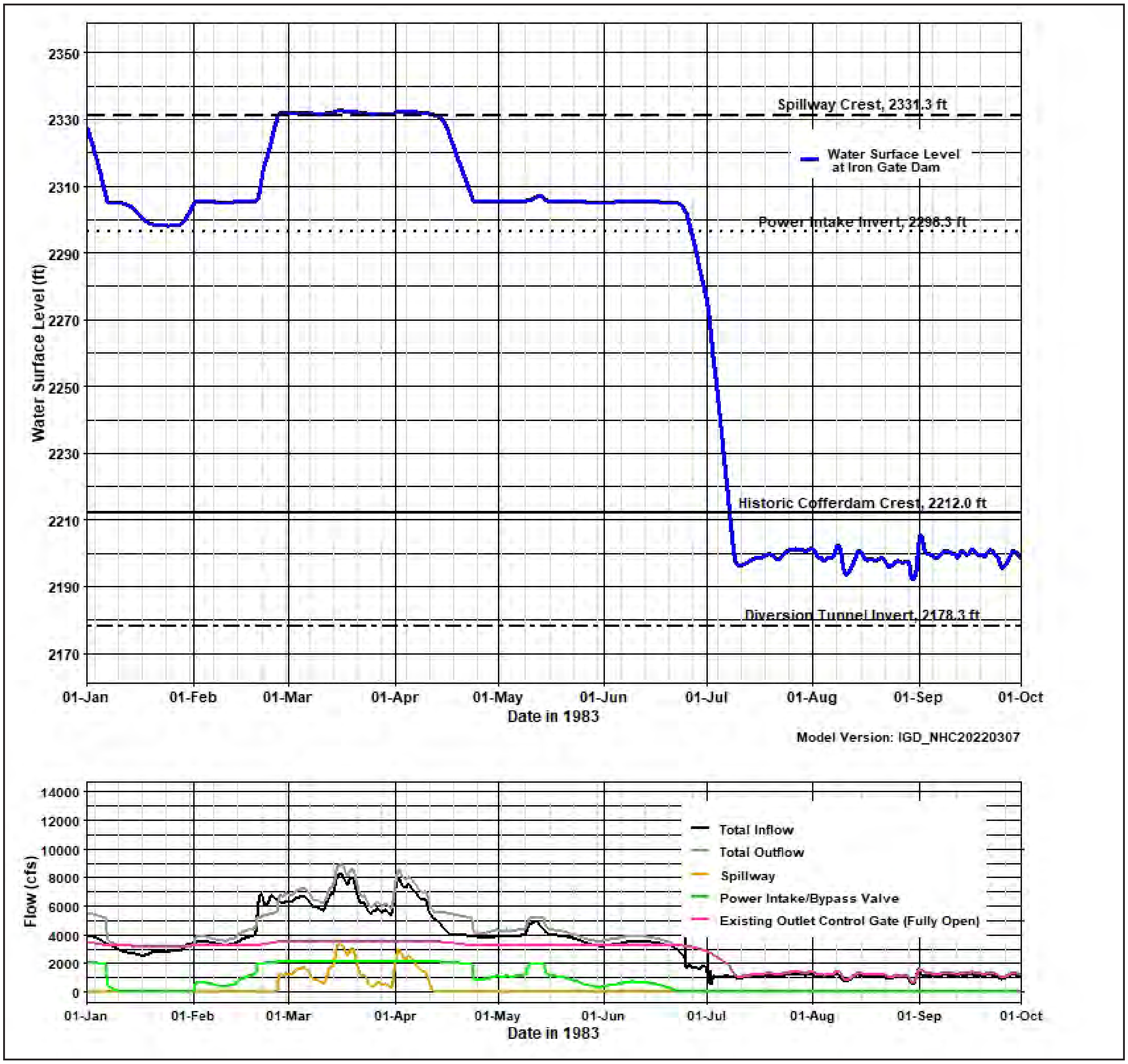

Figure 3 - Iron Gate Dam Facility Simulated Drawdown - Year 1983

April 22, 2022

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CONSULTING

IRON GATE DAM FACILITY SIMULATED DRAWDOWN

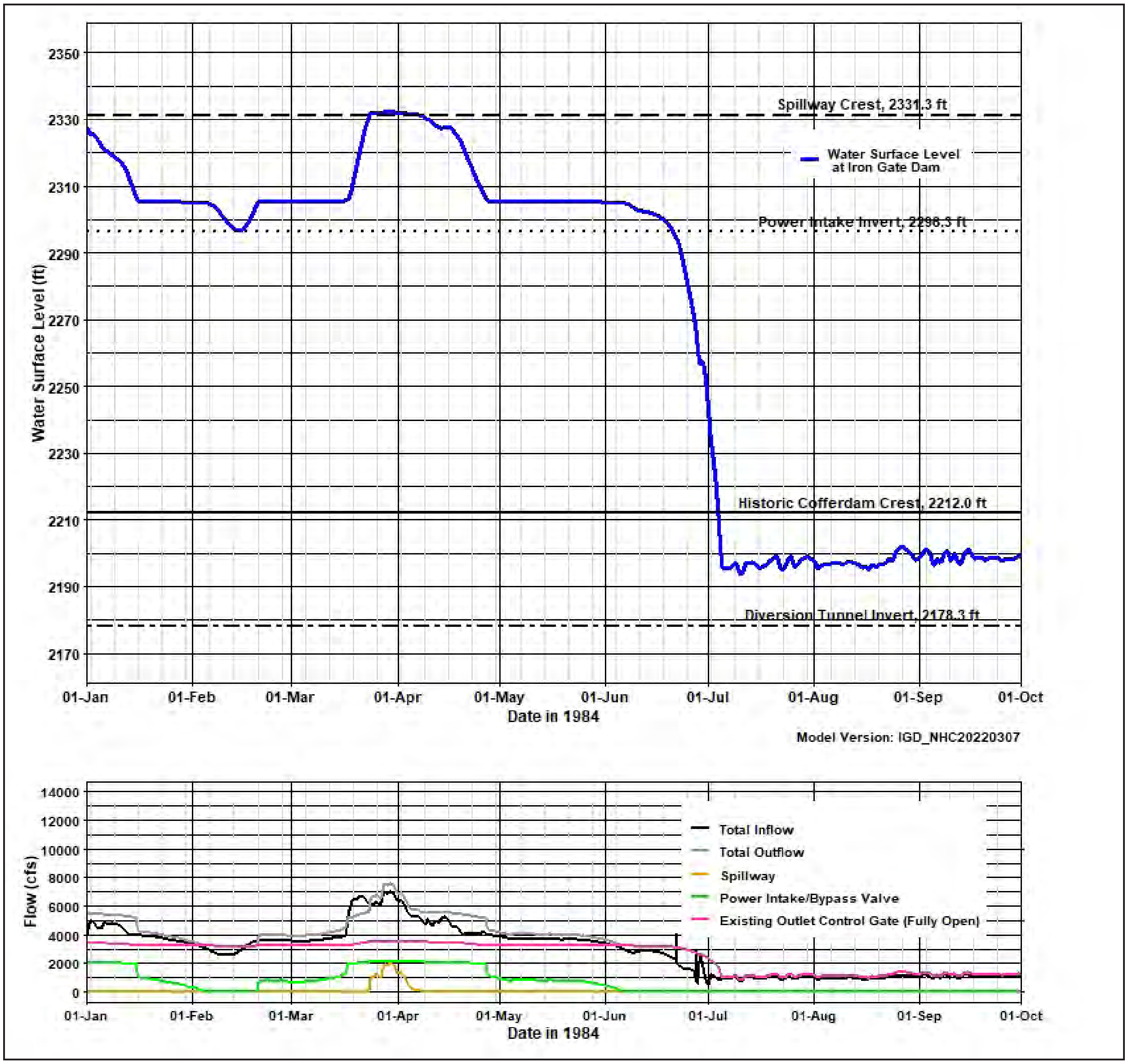

Figure 4 - Iron Gate Dam Facility Simulated Drawdown - Year 1984

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

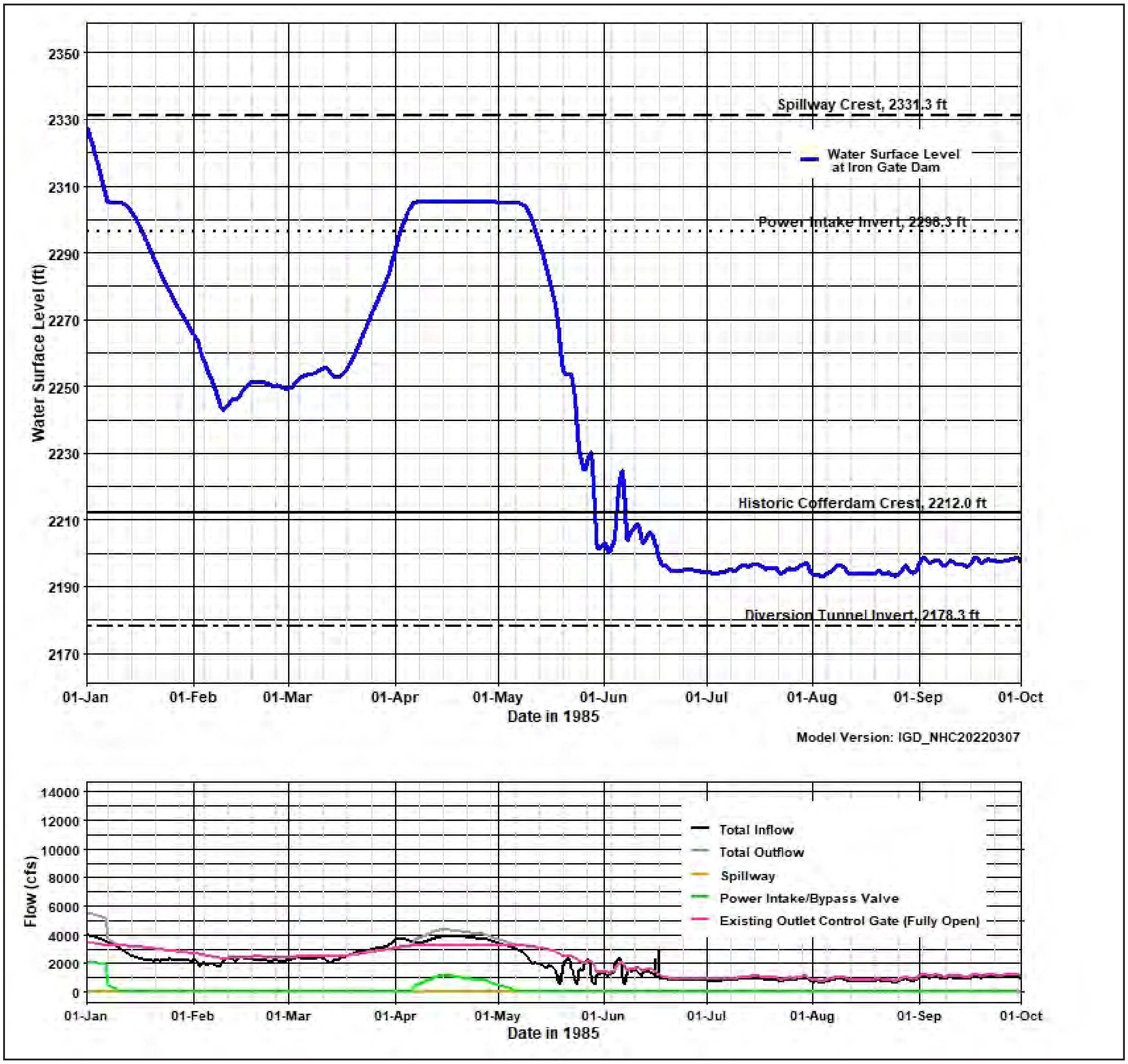

Figure 5 - Iron Gate Dam Facility Simulated Drawdown - Year 1985

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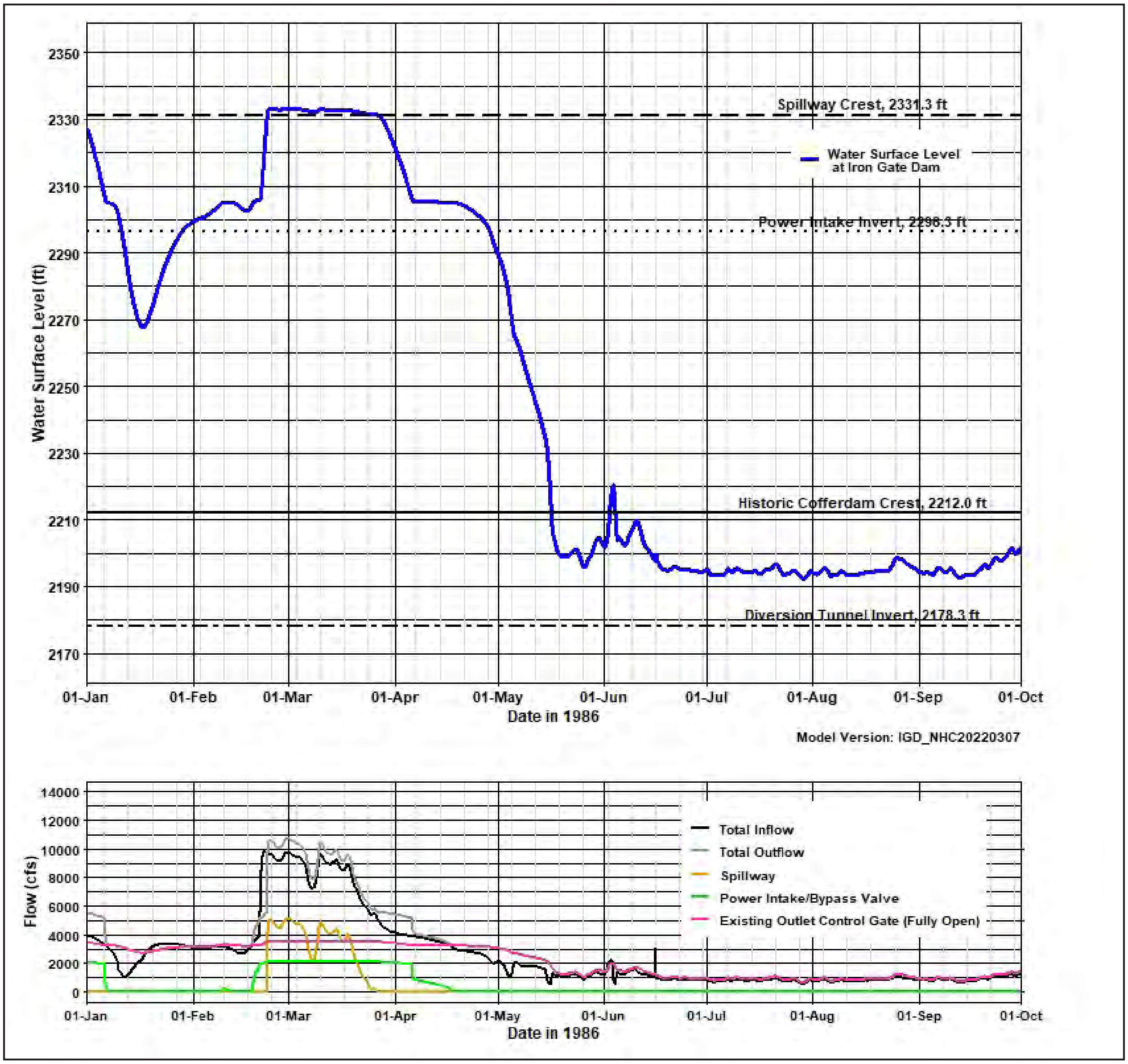

Figure 6 - Iron Gate Dam Facility Simulated Drawdown - Year 1986

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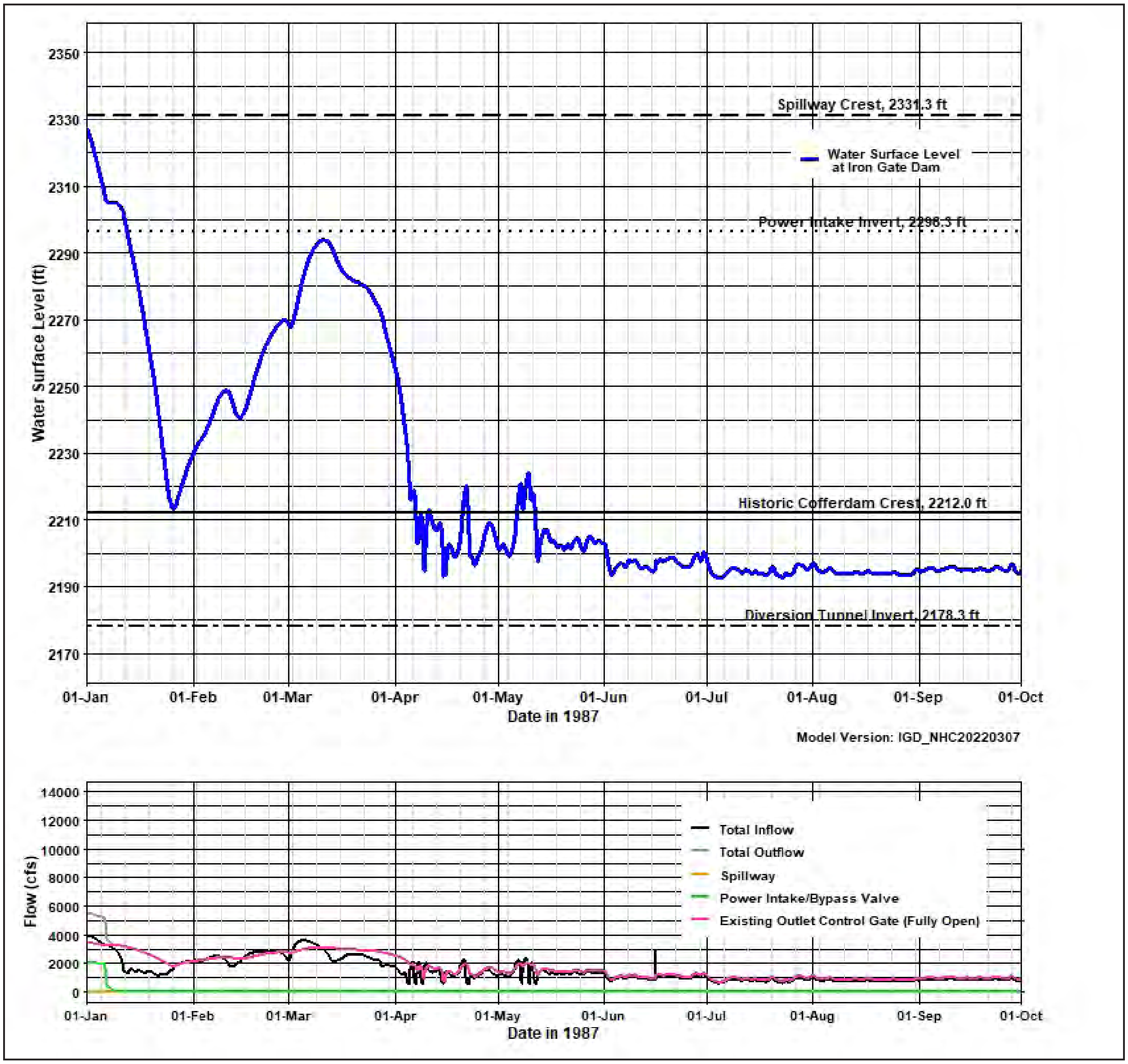

Figure 7 - Iron Gate Dam Facility Simulated Drawdown - Year 1987

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SIMULATED DRAWDOWN

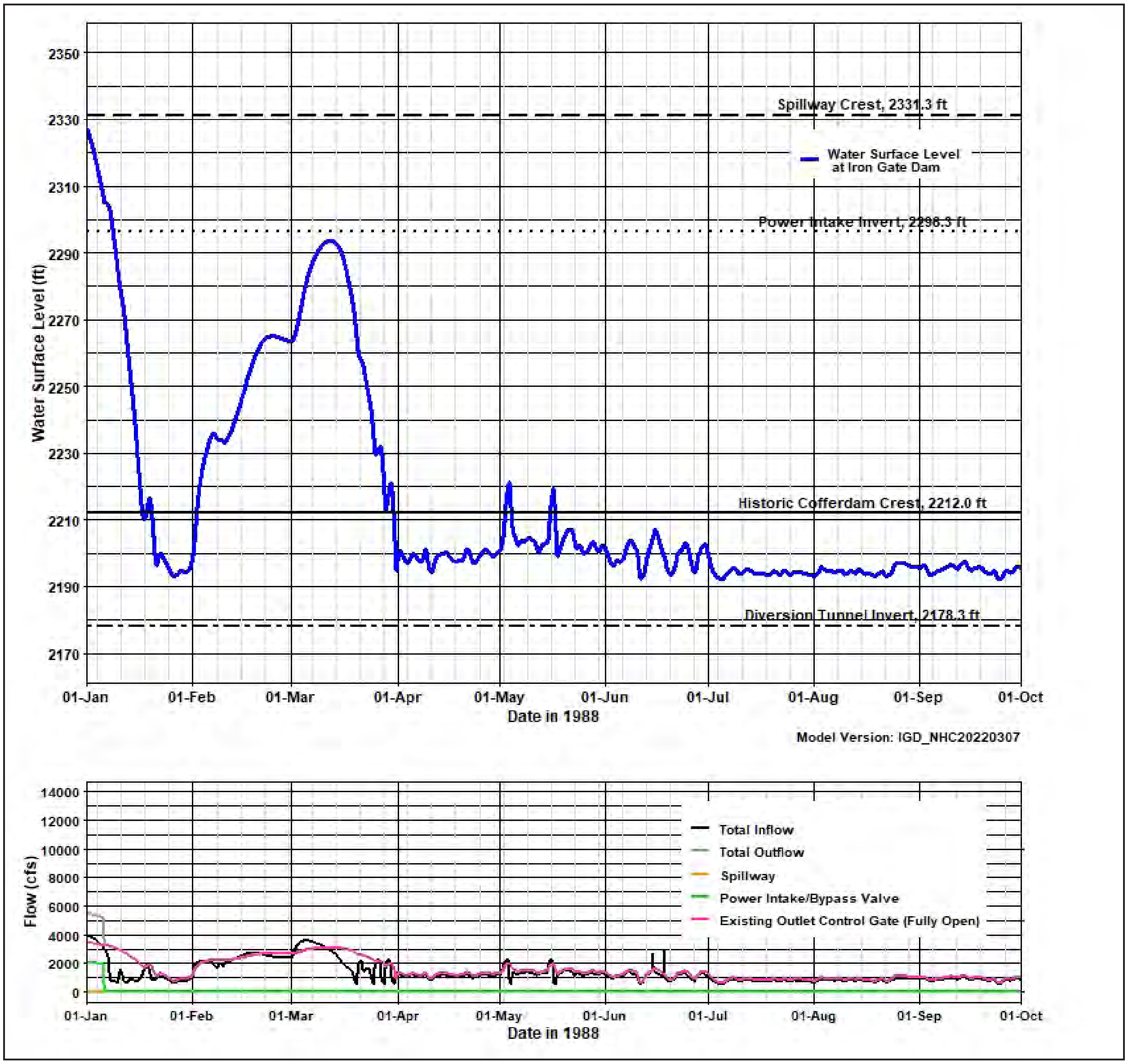

Figure 8 - Iron Gate Dam Facility Simulated Drawdown - Year 1988

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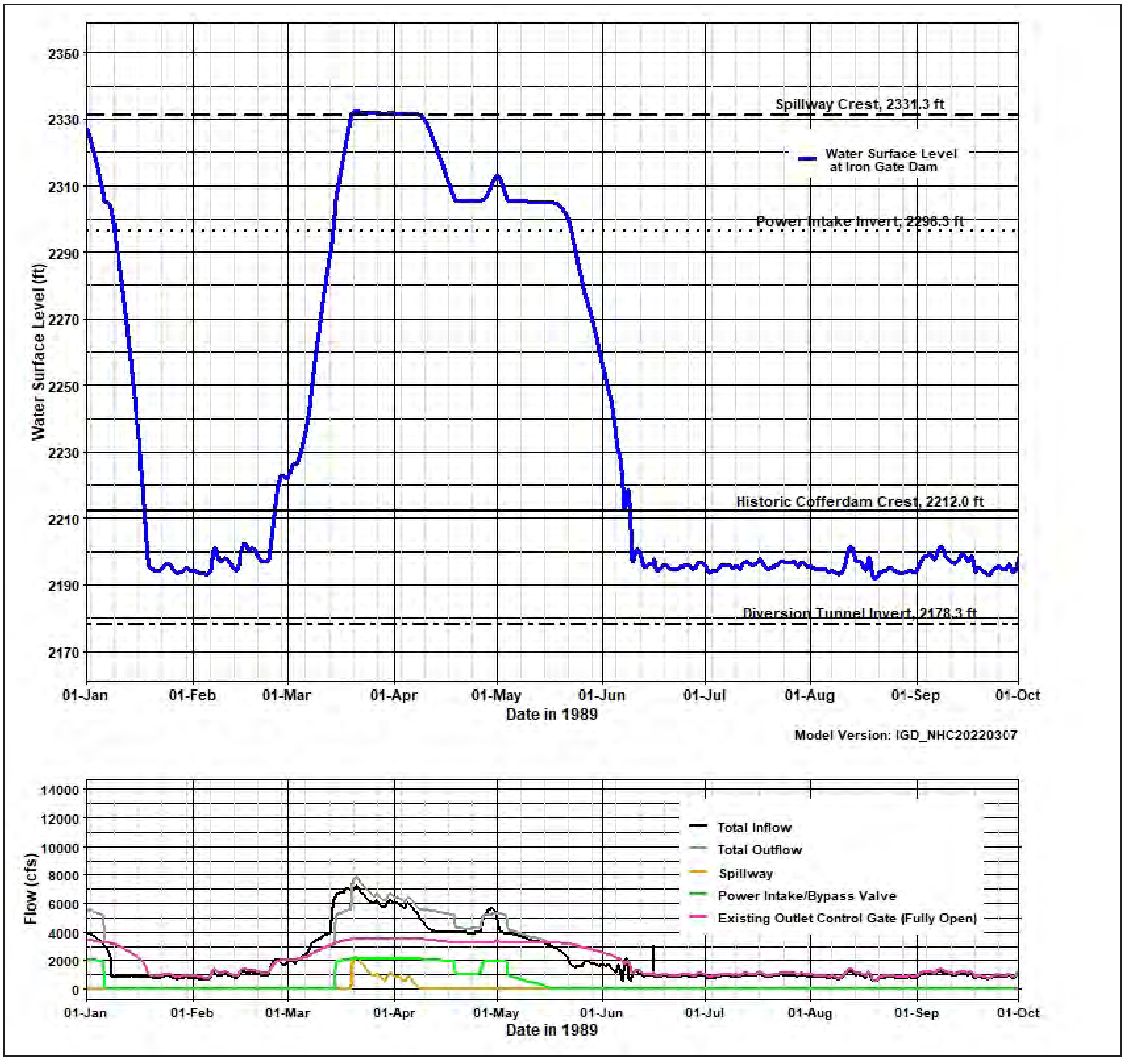

Figure 9 - Iron Gate Dam Facility Simulated Drawdown - Year 1989

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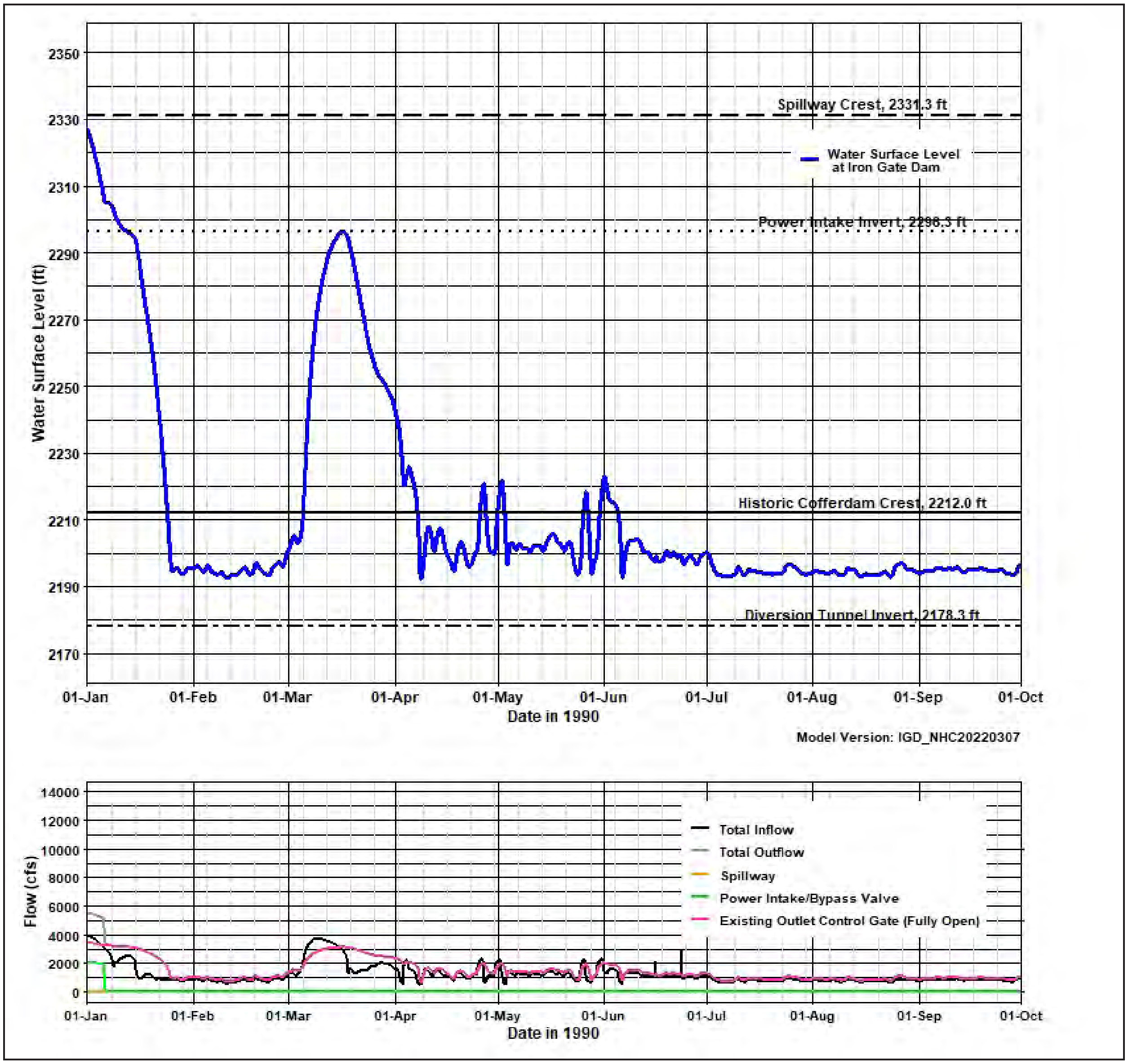

Figure 10 - Iron Gate Dam Facility Simulated Drawdown - Year 1990

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SIMULATED DRAWDOWN

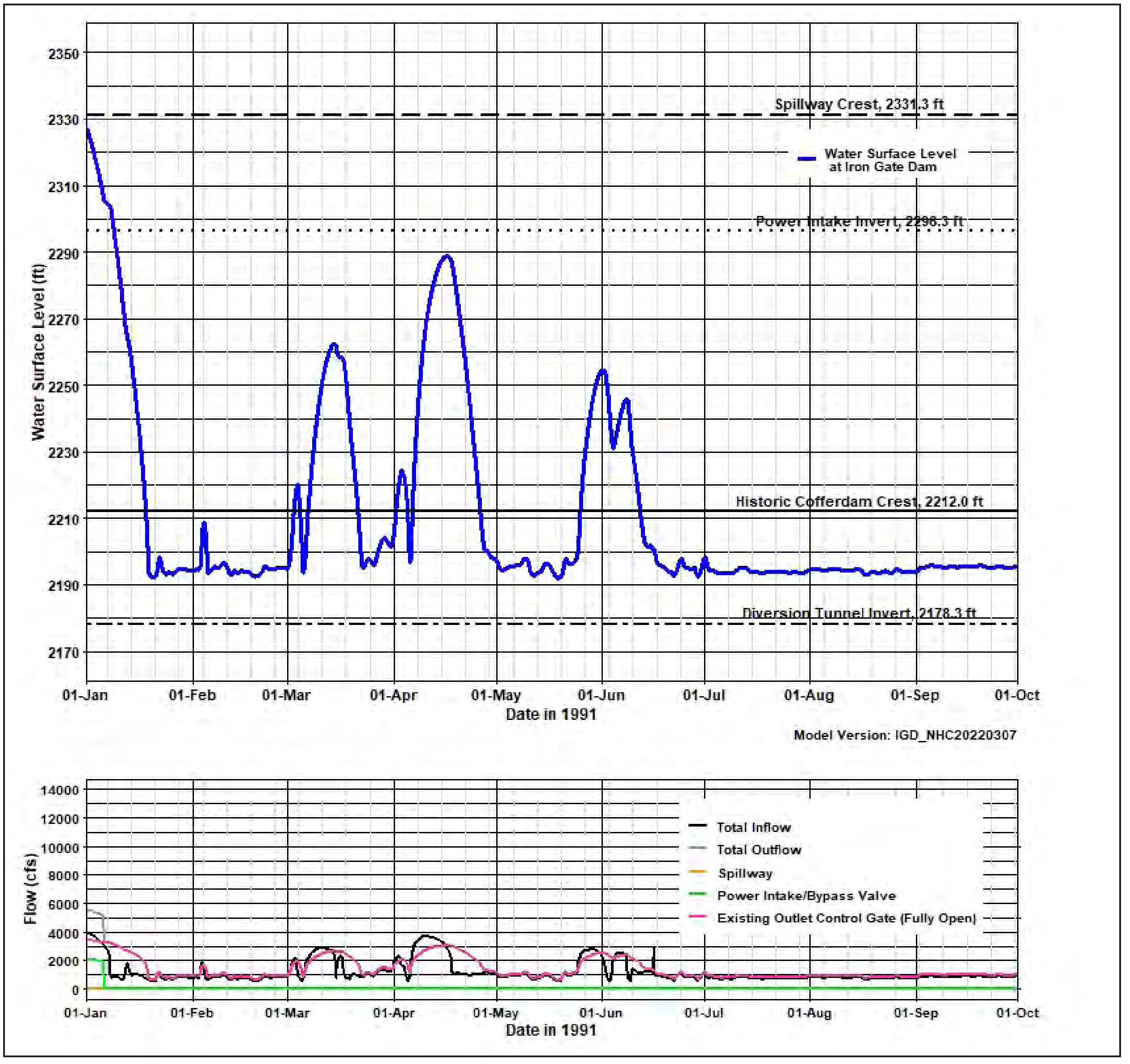

Figure 11 - Iron Gate Dam Facility Simulated Drawdown - Year 1991

April 22, 2022

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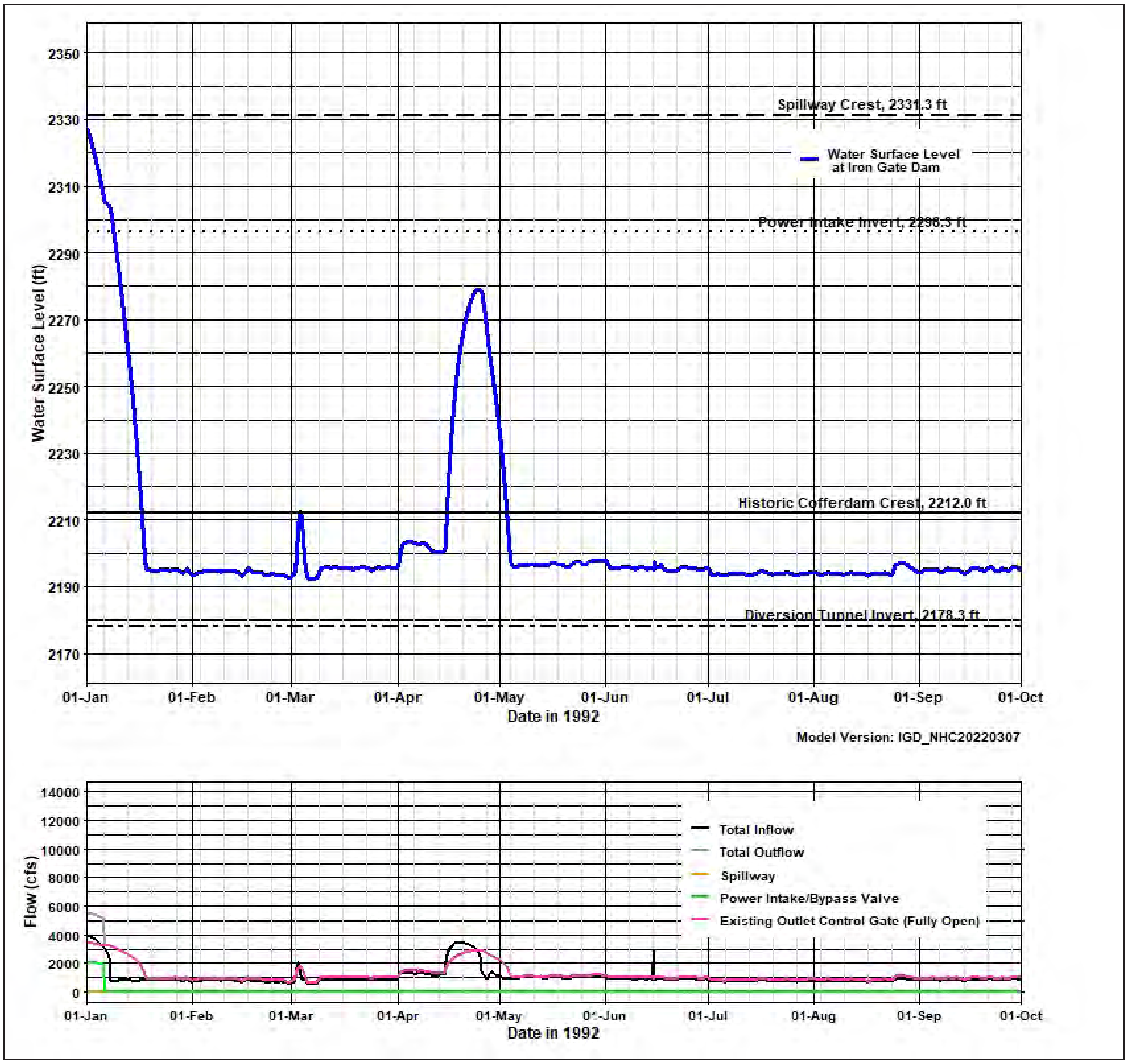

Figure 12 - Iron Gate Dam Facility Simulated Drawdown - Year 1992

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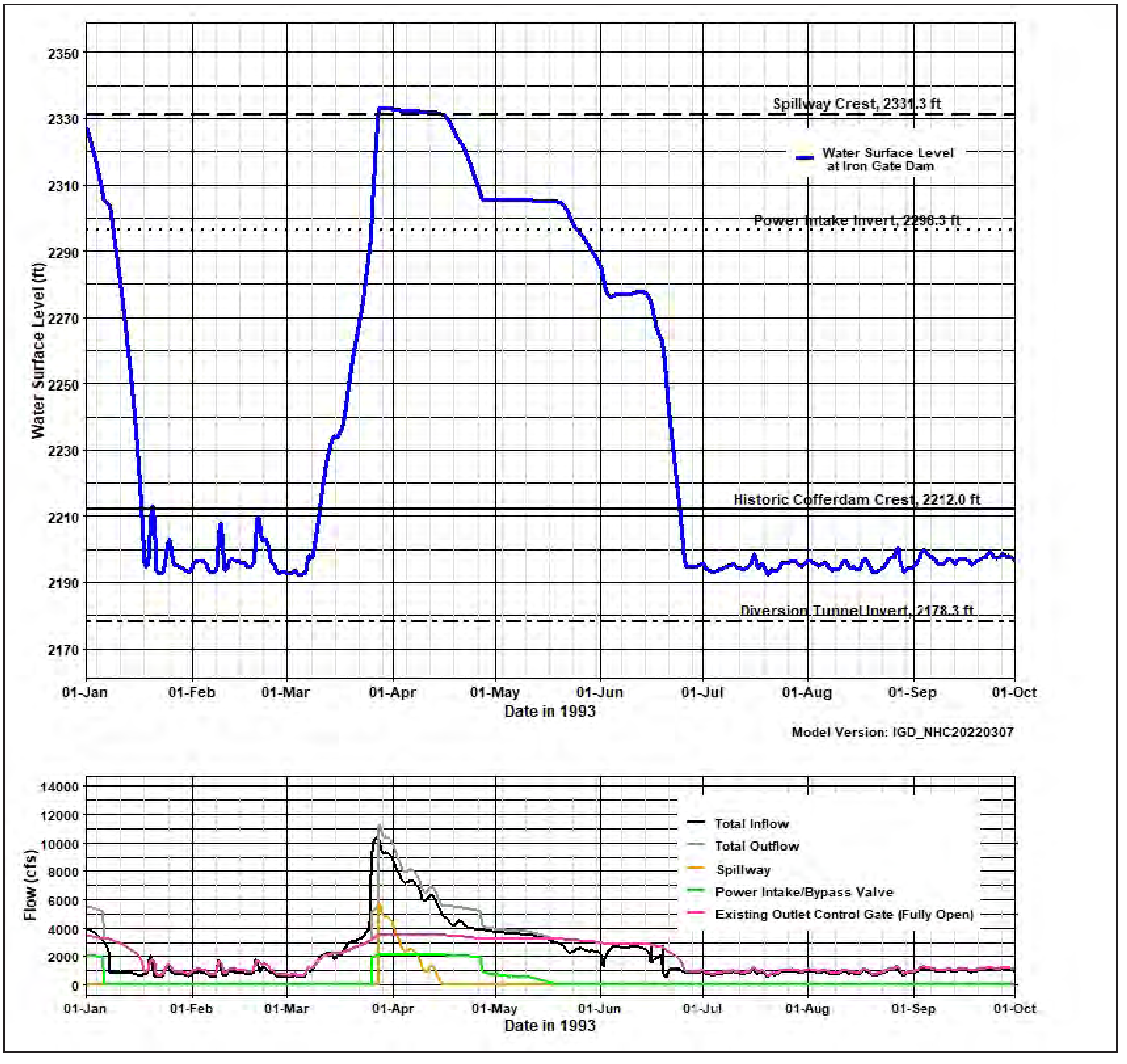

Figure 13 - Iron Gate Dam Facility Simulated Drawdown - Year 1993

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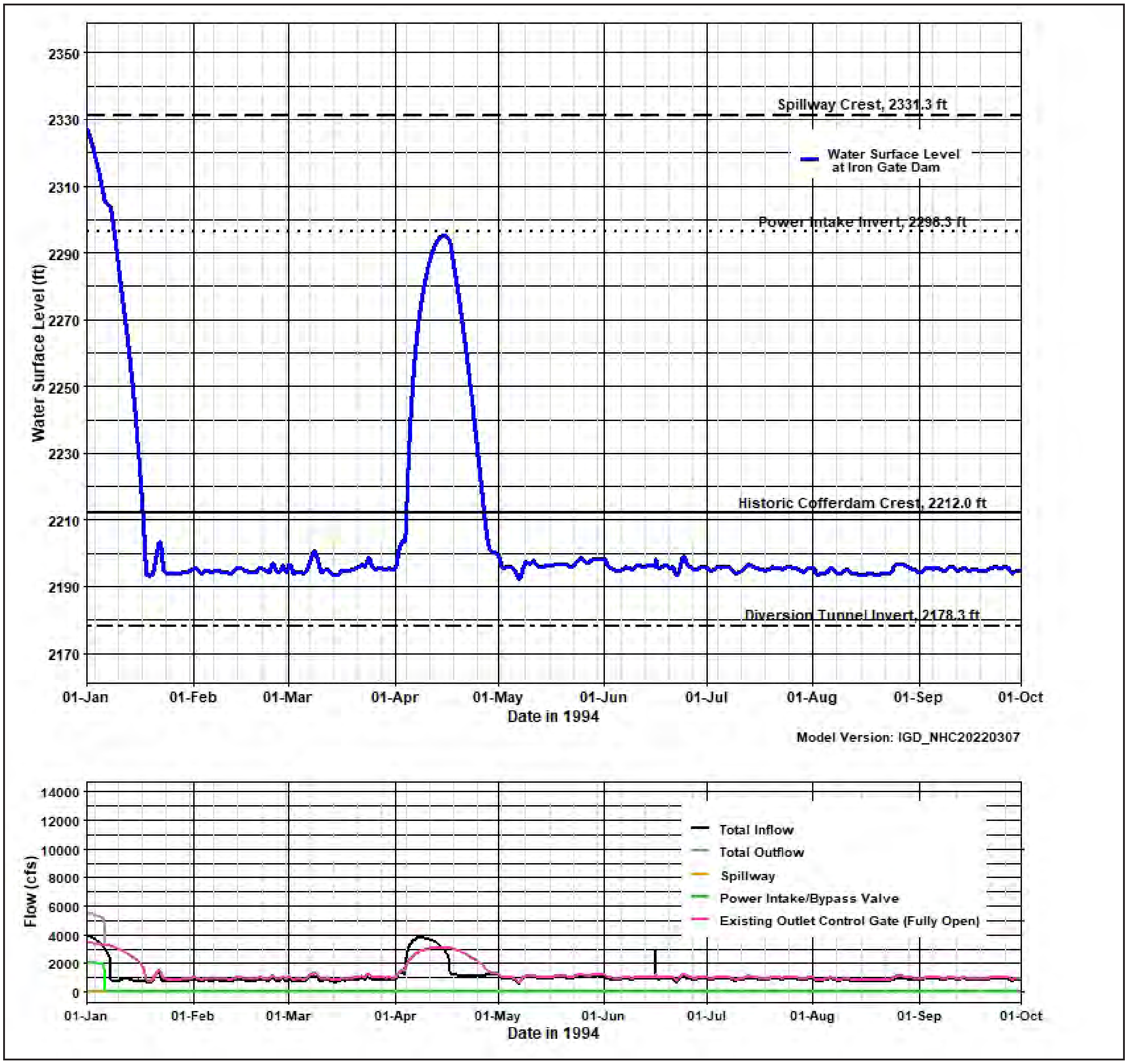

Figure 14 - Iron Gate Dam Facility Simulated Drawdown - Year 1994

April 22, 2022

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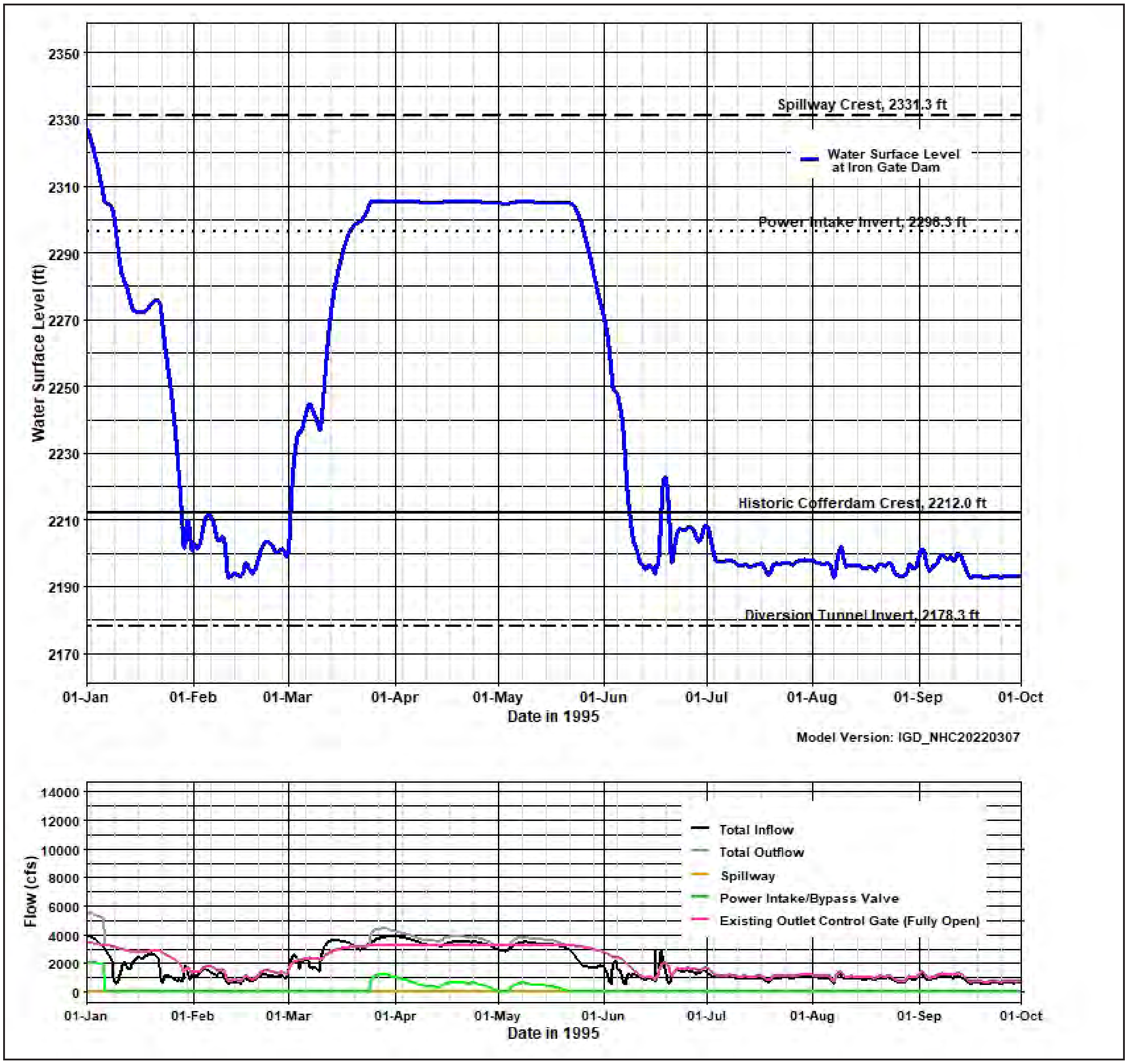

Figure 15 - Iron Gate Dam Facility Simulated Drawdown - Year 1995

April 22, 2022

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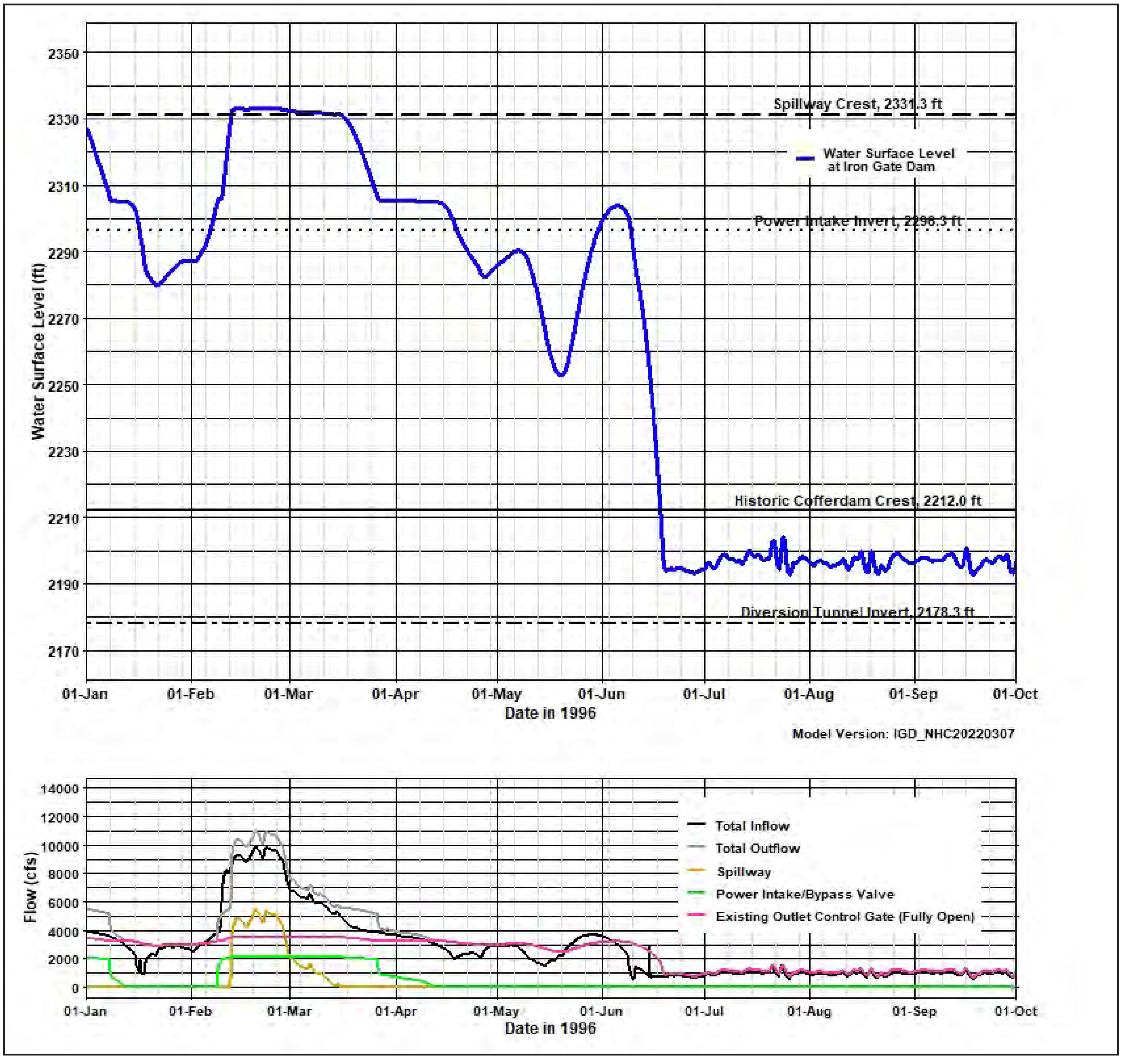

Figure 16 - Iron Gate Dam Facility Simulated Drawdown - Year 1996

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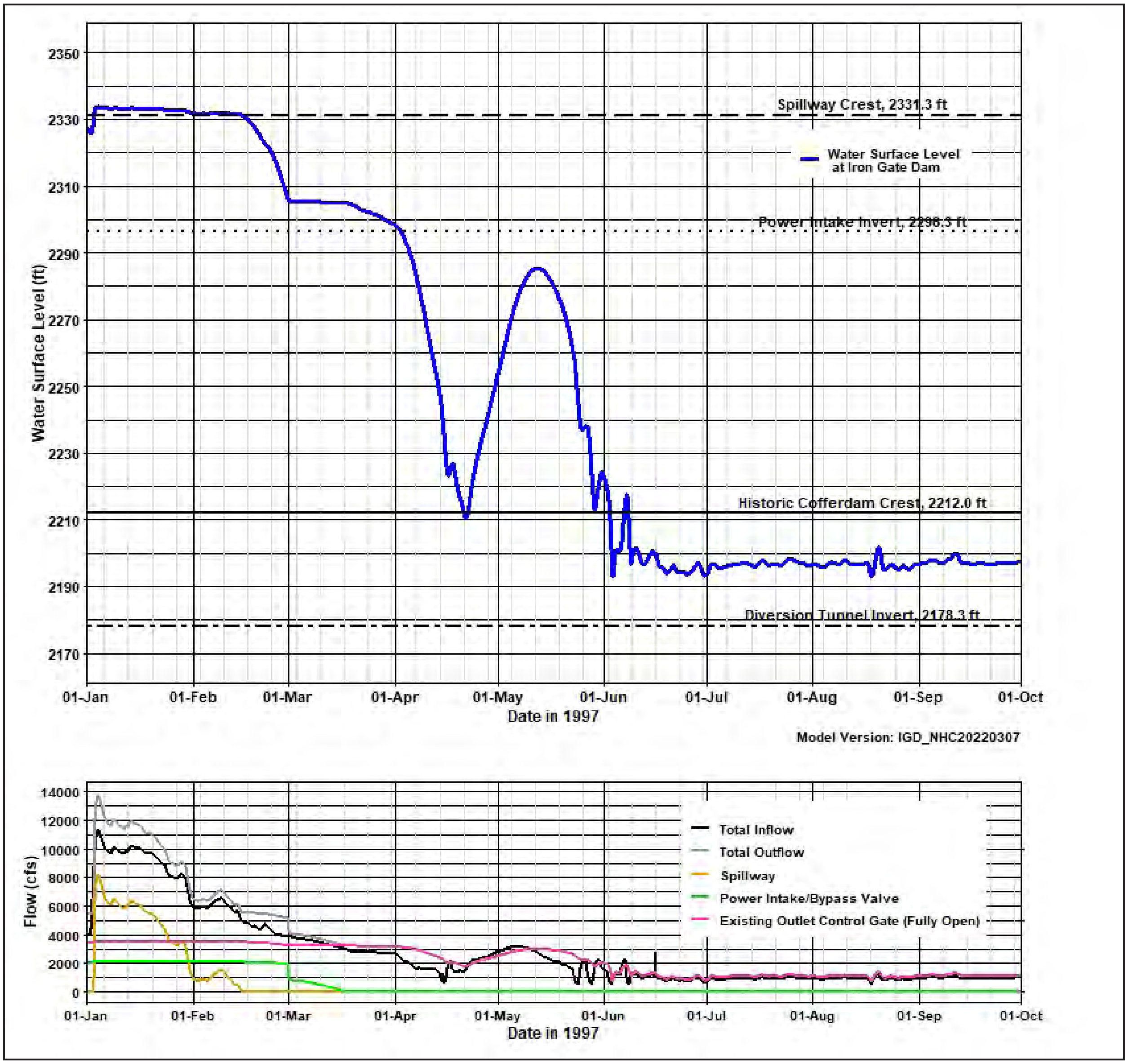

Figure 17 - Iron Gate Dam Facility Simulated Drawdown - Year 1997

April 22, 2022

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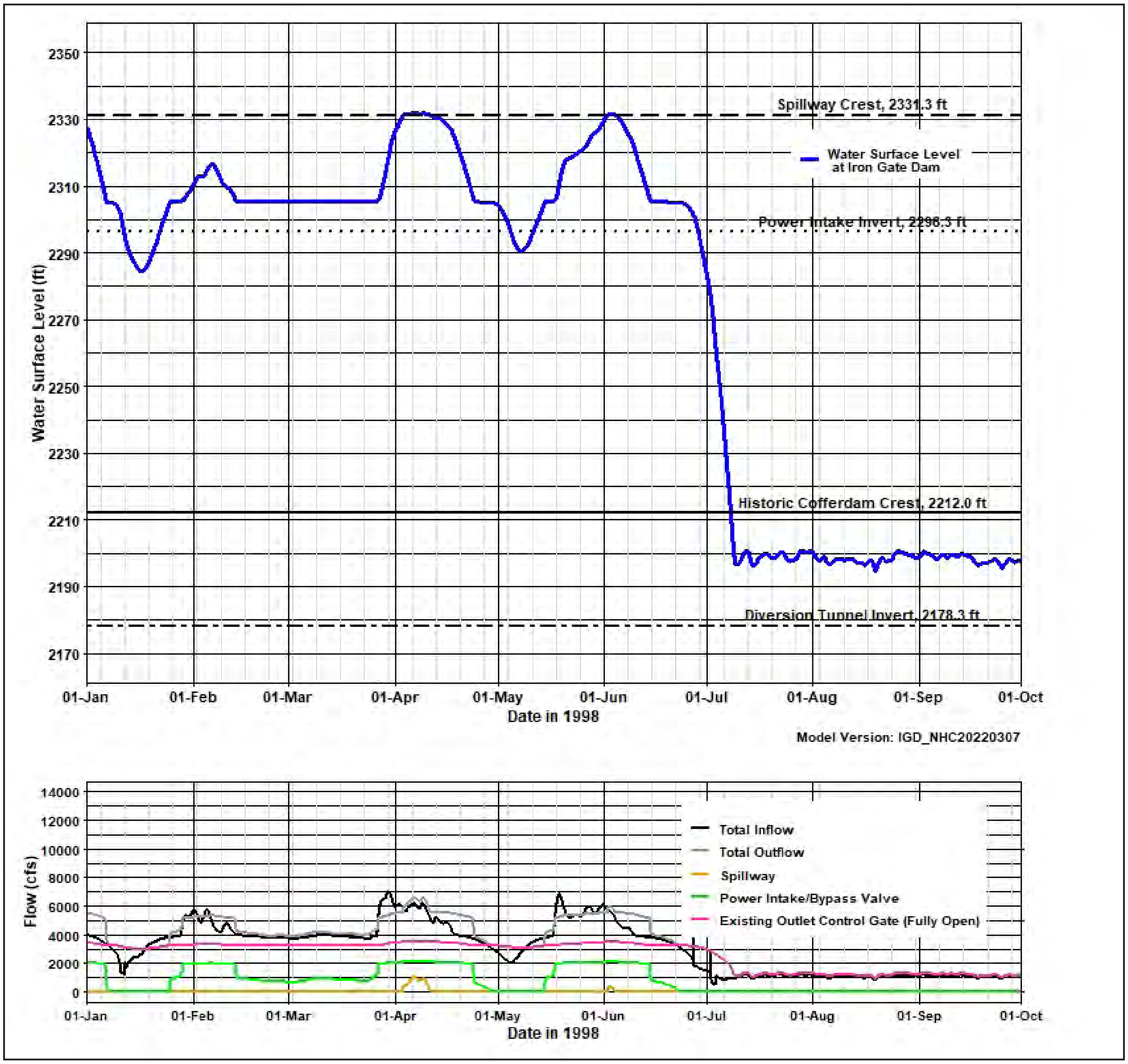

Figure 18 - Iron Gate Dam Facility Simulated Drawdown - Year 1998

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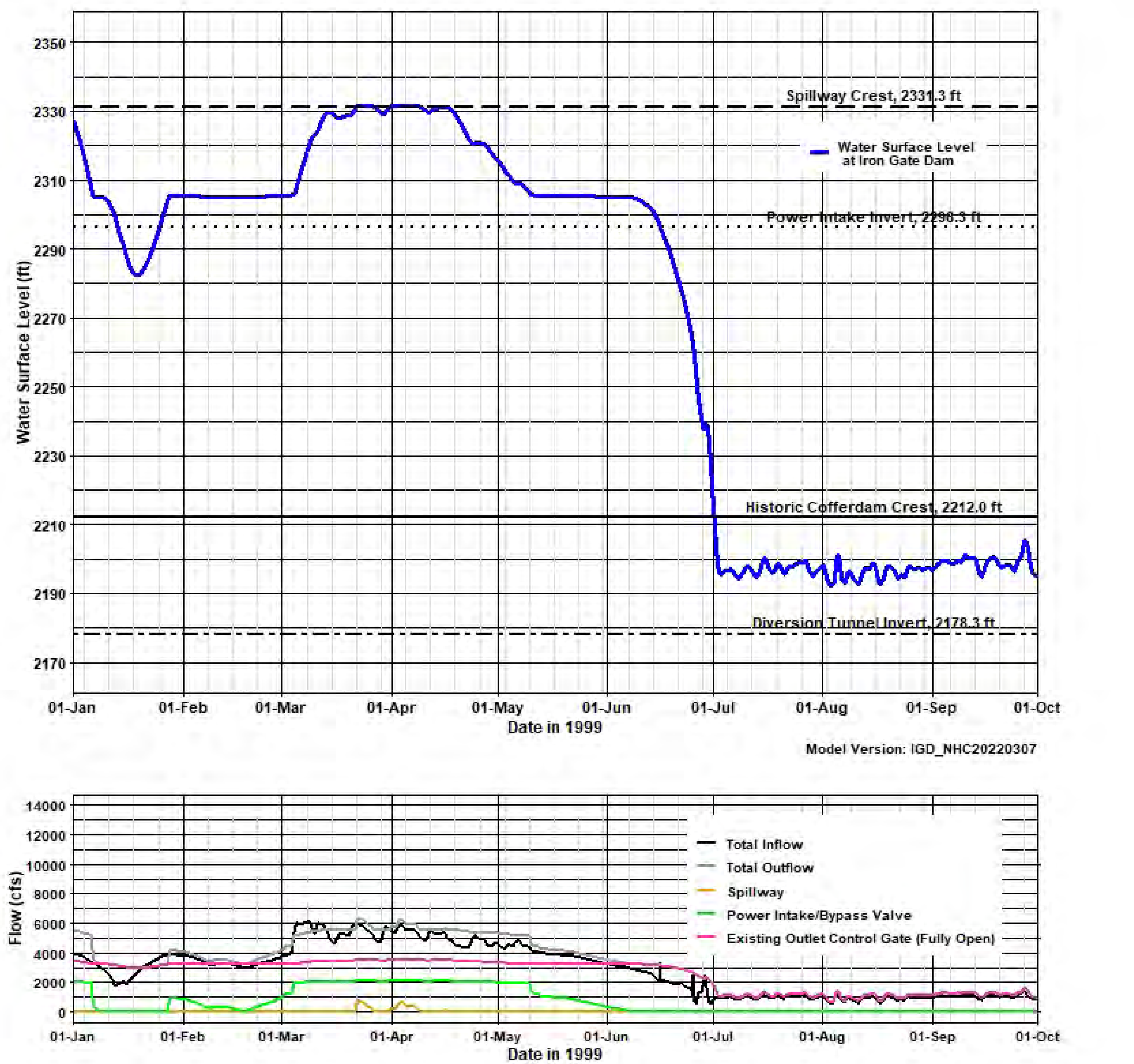

Figure 19 - Iron Gate Dam Facility Simulated Drawdown - Year 1999

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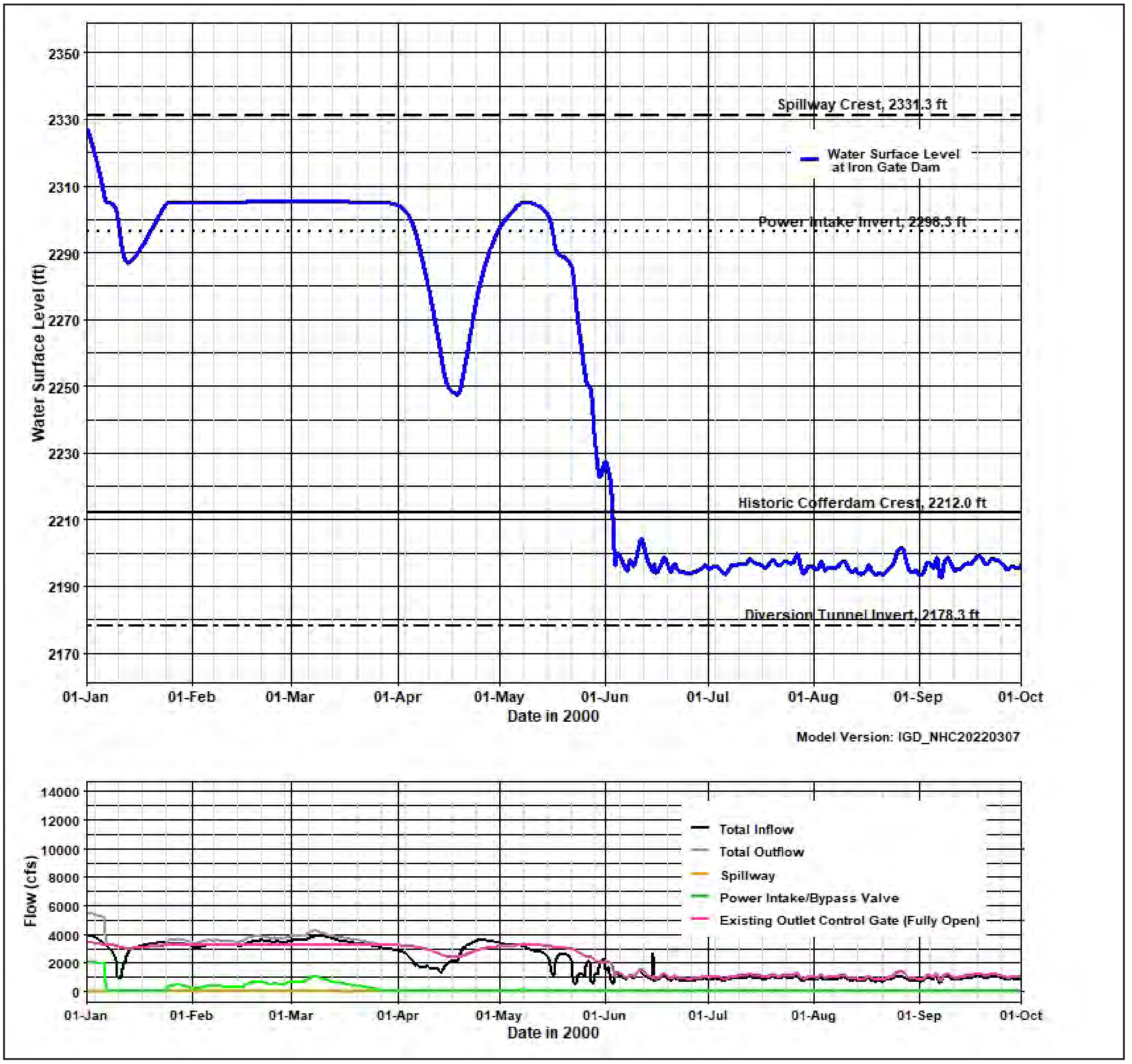

Figure 20 - Iron Gate Dam Facility Simulated Drawdown - Year 2000

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

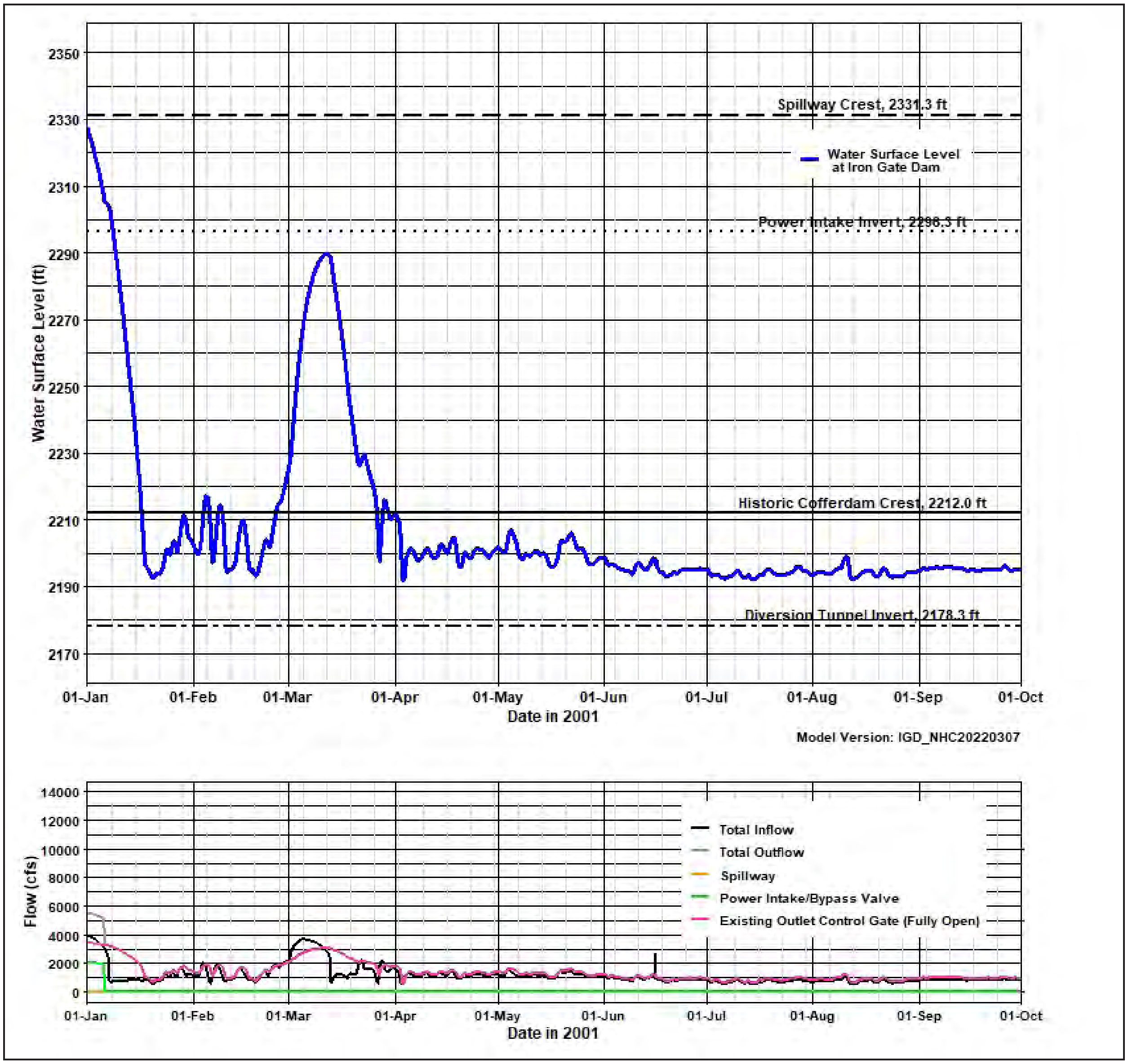

Figure 21 - Iron Gate Dam Facility Simulated Drawdown - Year 2001

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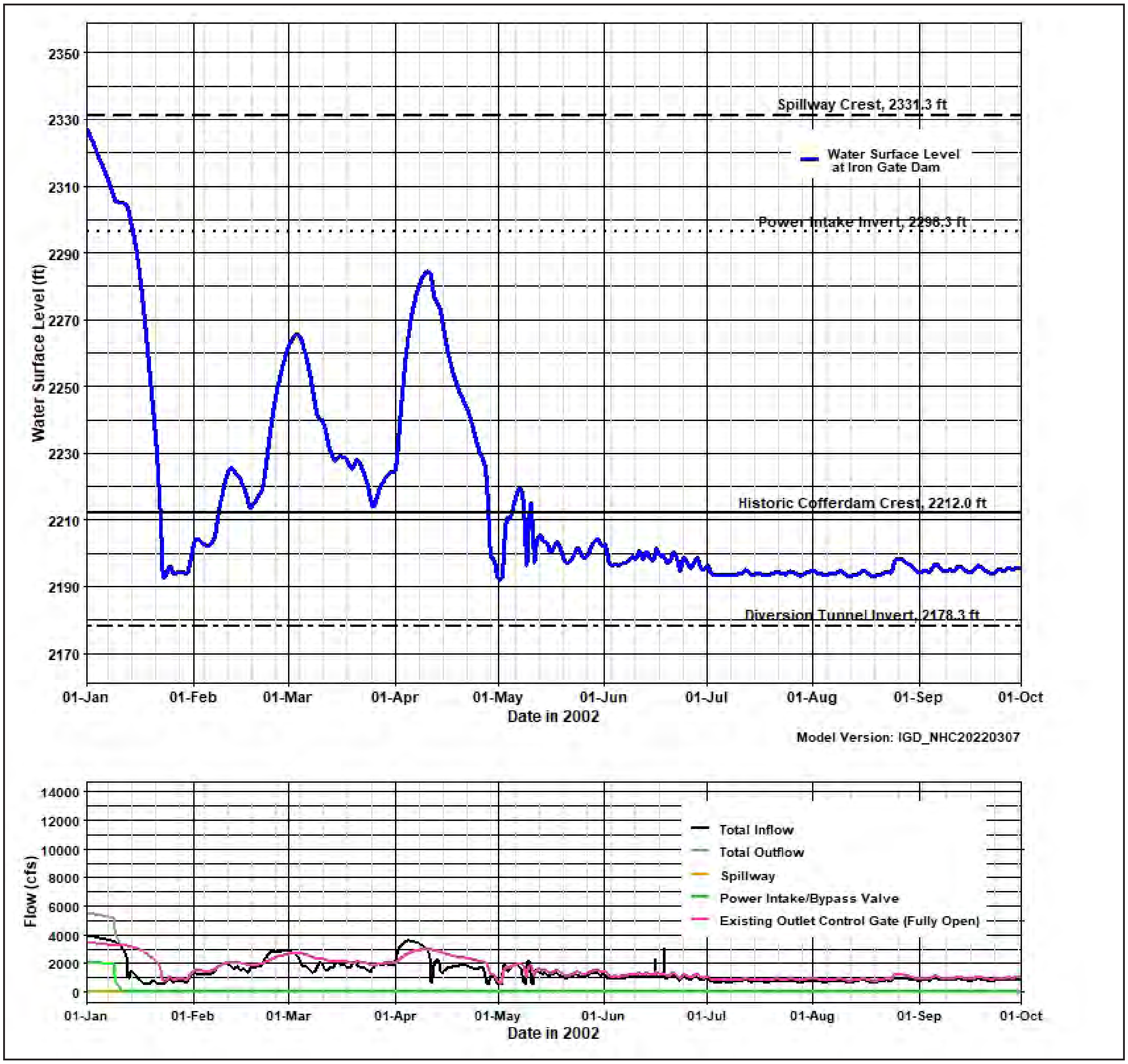

Figure 22 - Iron Gate Dam Facility Simulated Drawdown - Year 2002

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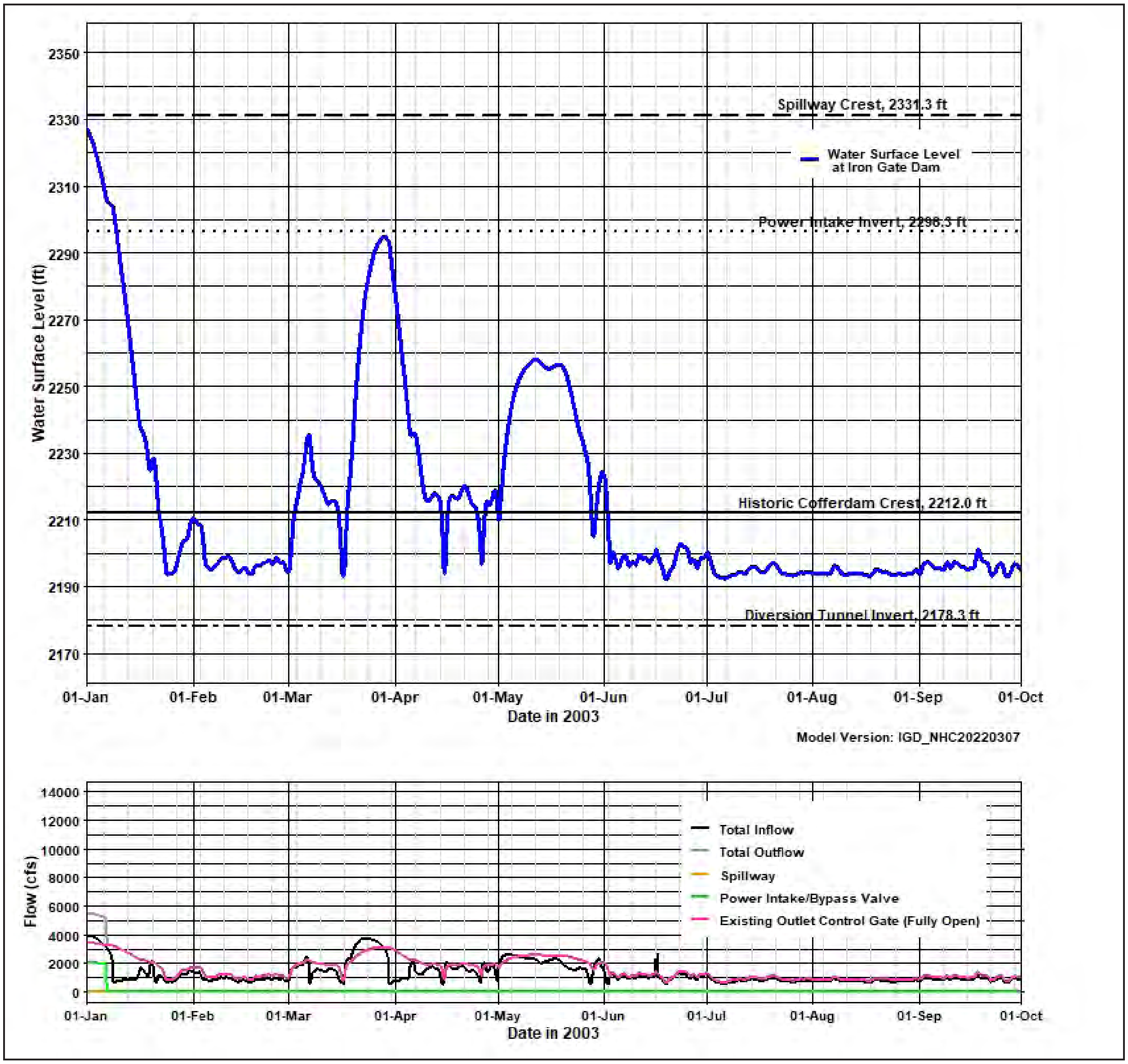

Figure 23 - Iron Gate Dam Facility Simulated Drawdown - Year 2003

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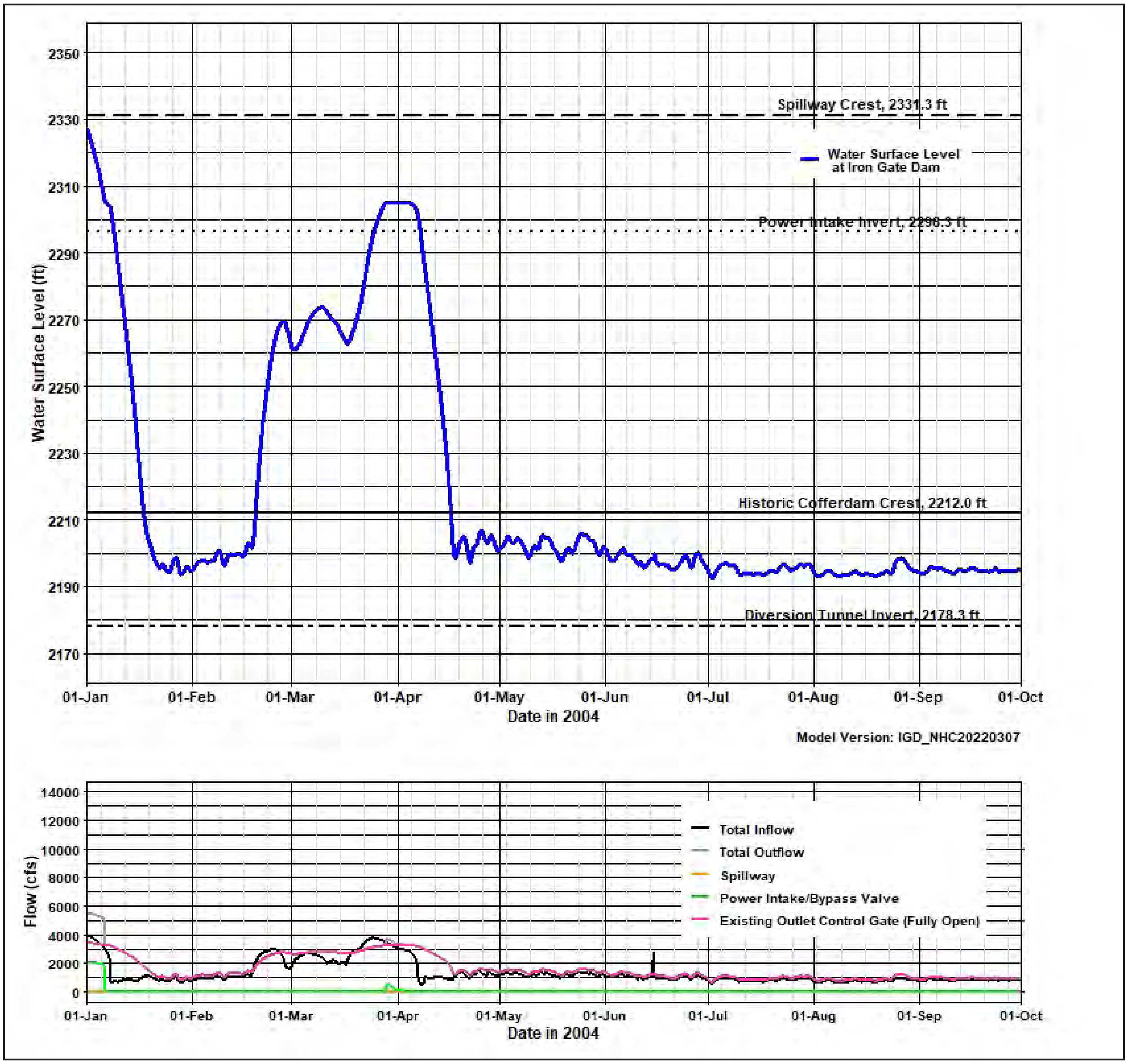

Figure 24 - Iron Gate Dam Facility Simulated Drawdown - Year 2004

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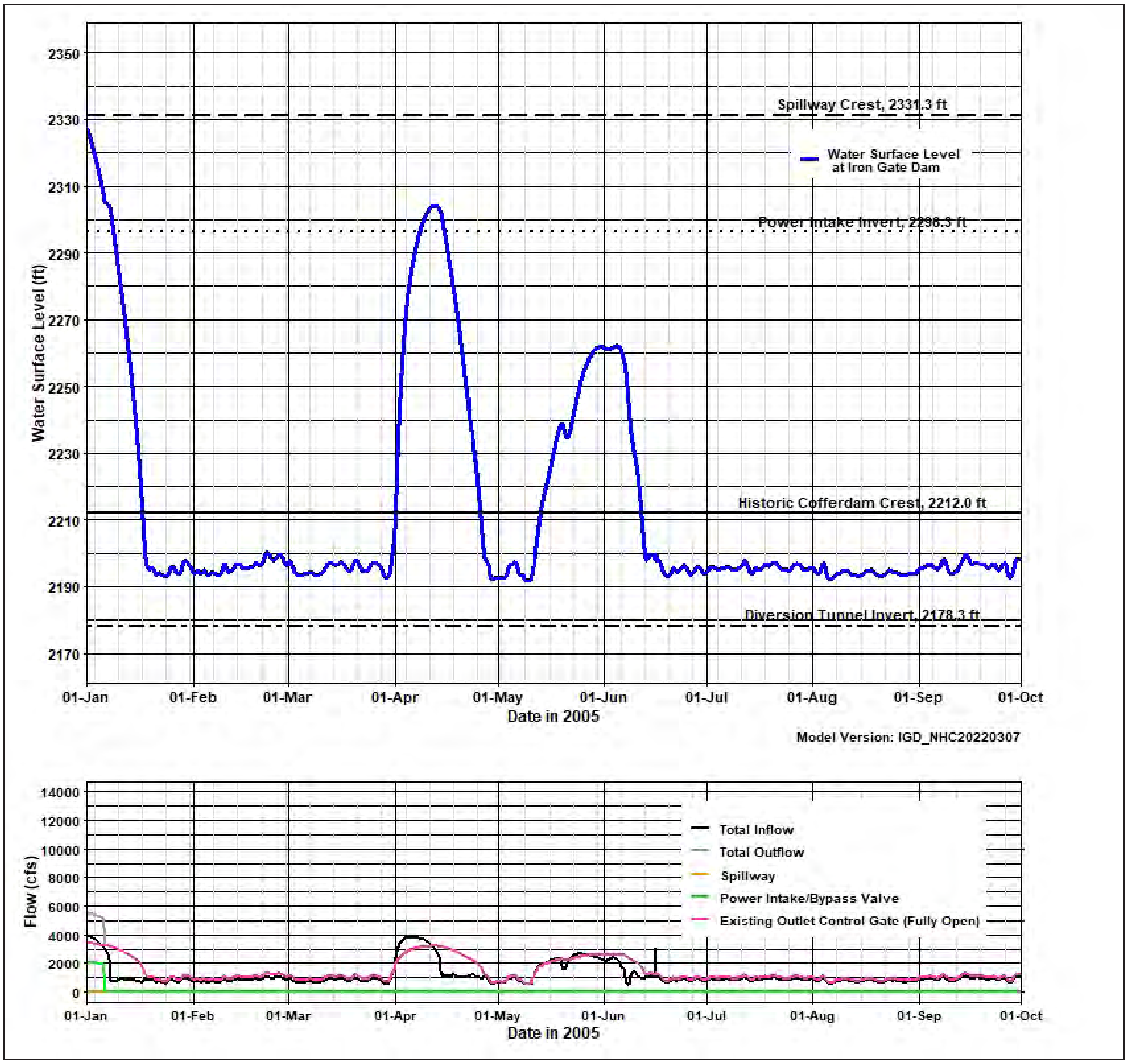

Figure 25 - Iron Gate Dam Facility Simulated Drawdown - Year 2005

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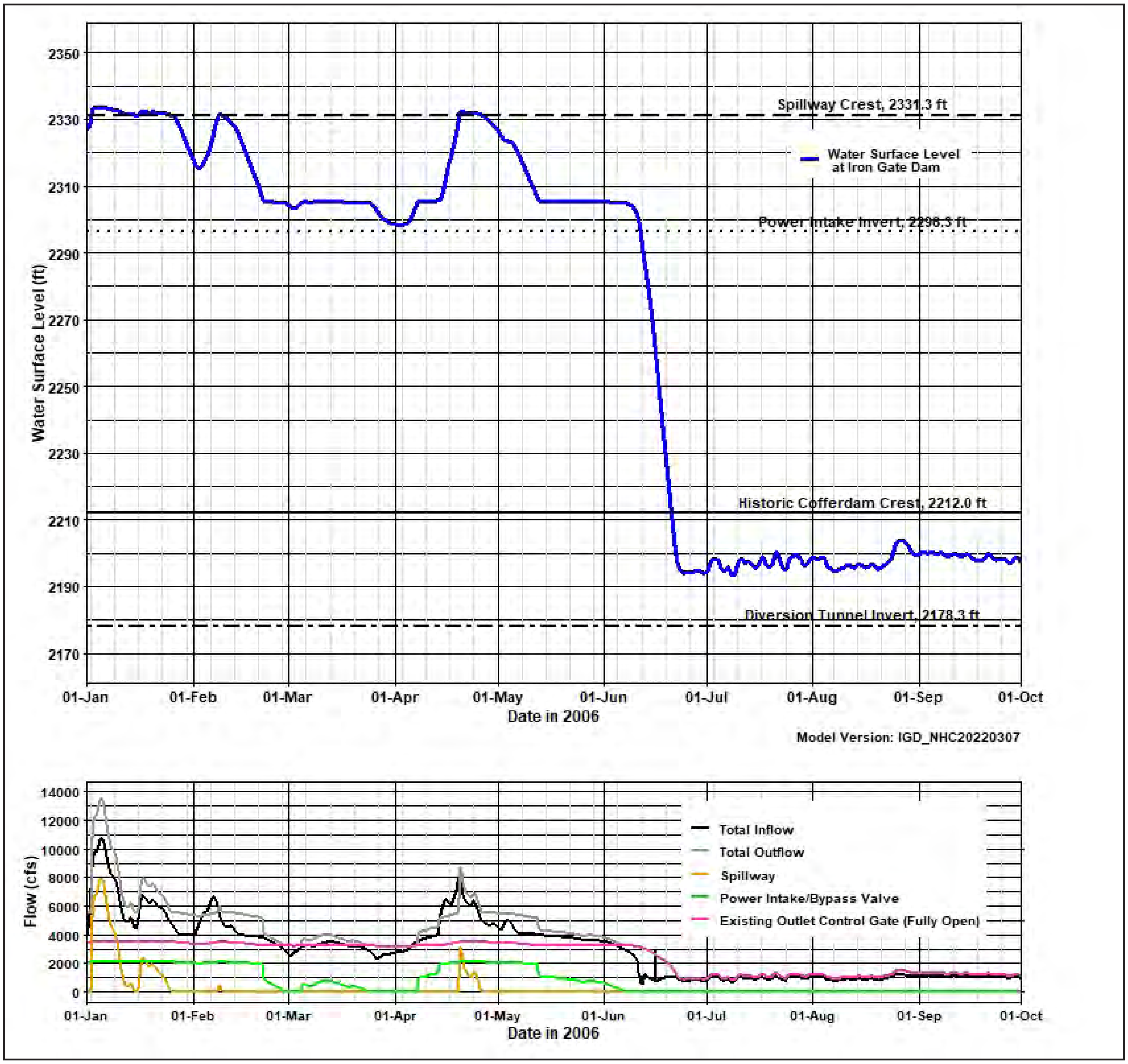

Figure 26 - Iron Gate Dam Facility Simulated Drawdown - Year 2006

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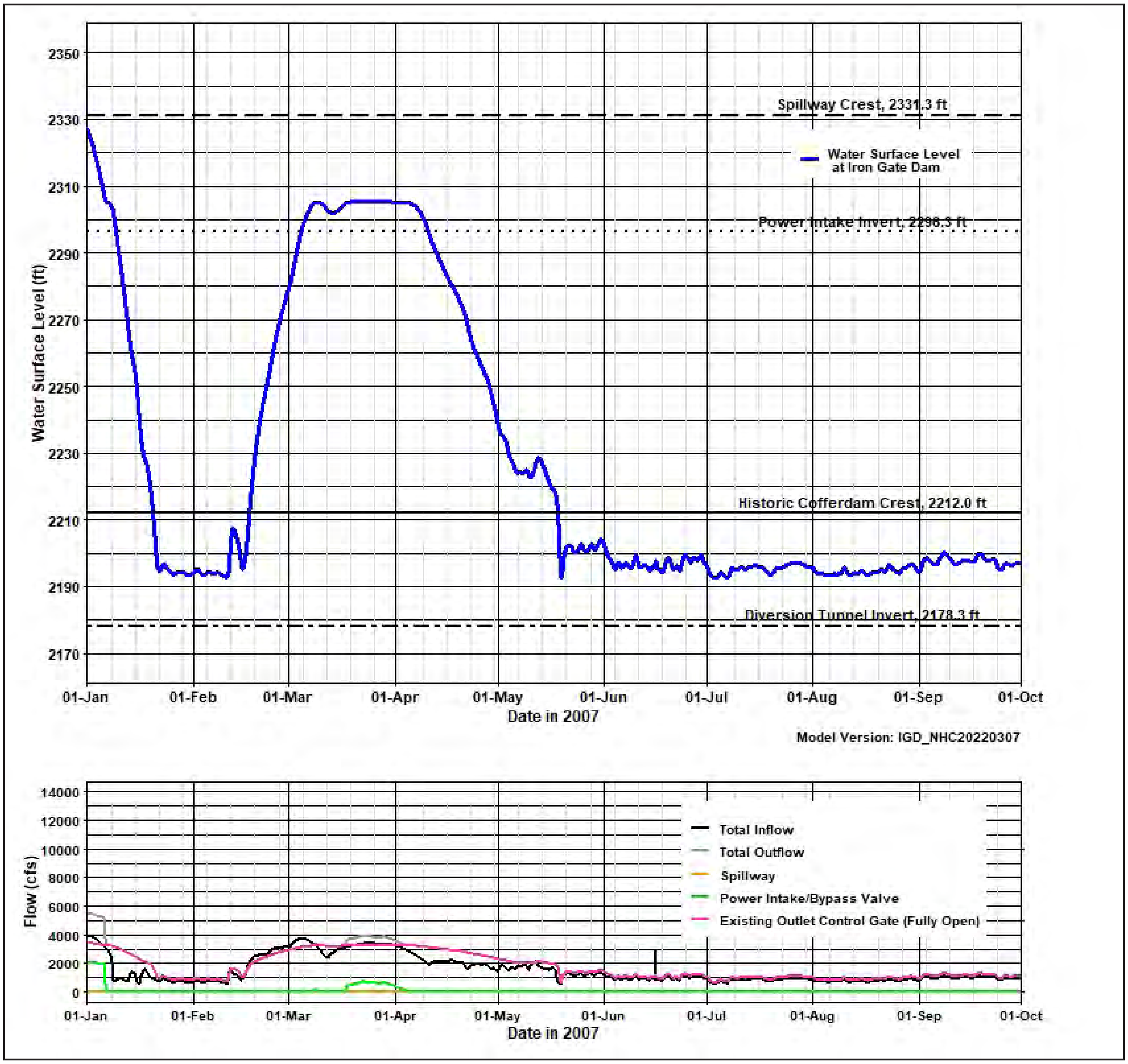

Figure 27 - Iron Gate Dam Facility Simulated Drawdown - Year 2007

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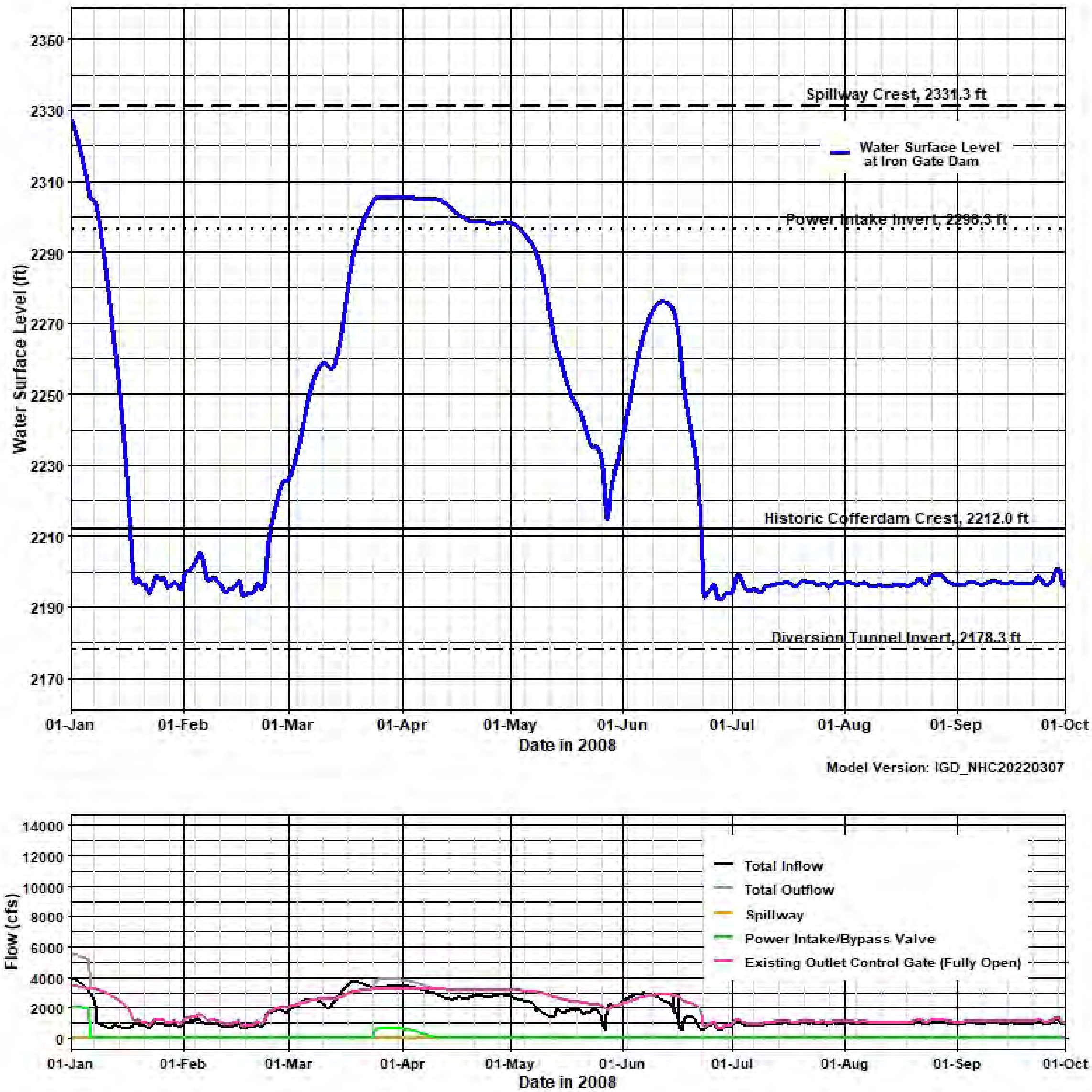

Figure 28 - Iron Gate Dam Facility Simulated Drawdown - Year 2008

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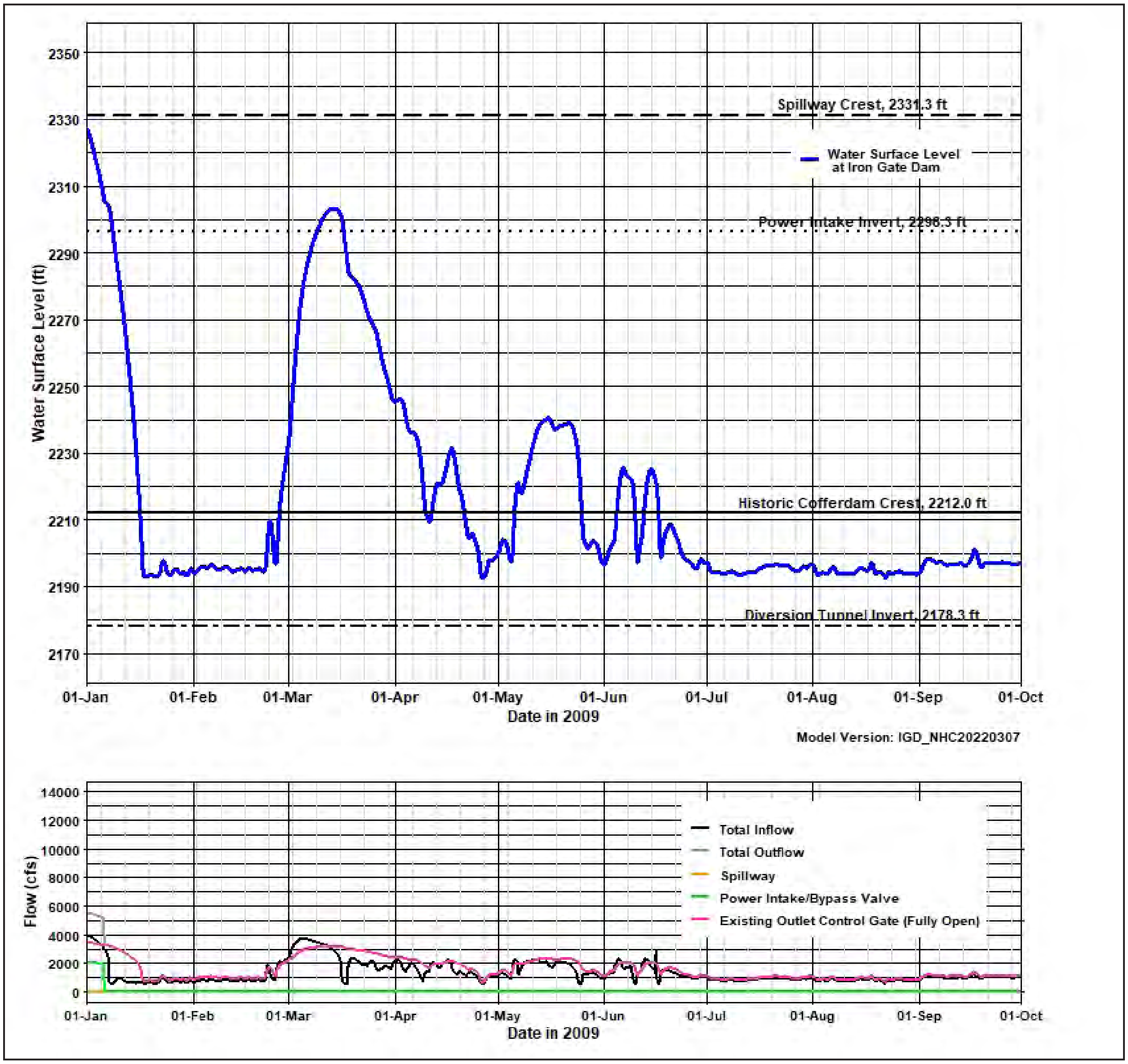

Figure 29 - Iron Gate Dam Facility Simulated Drawdown - Year 2009

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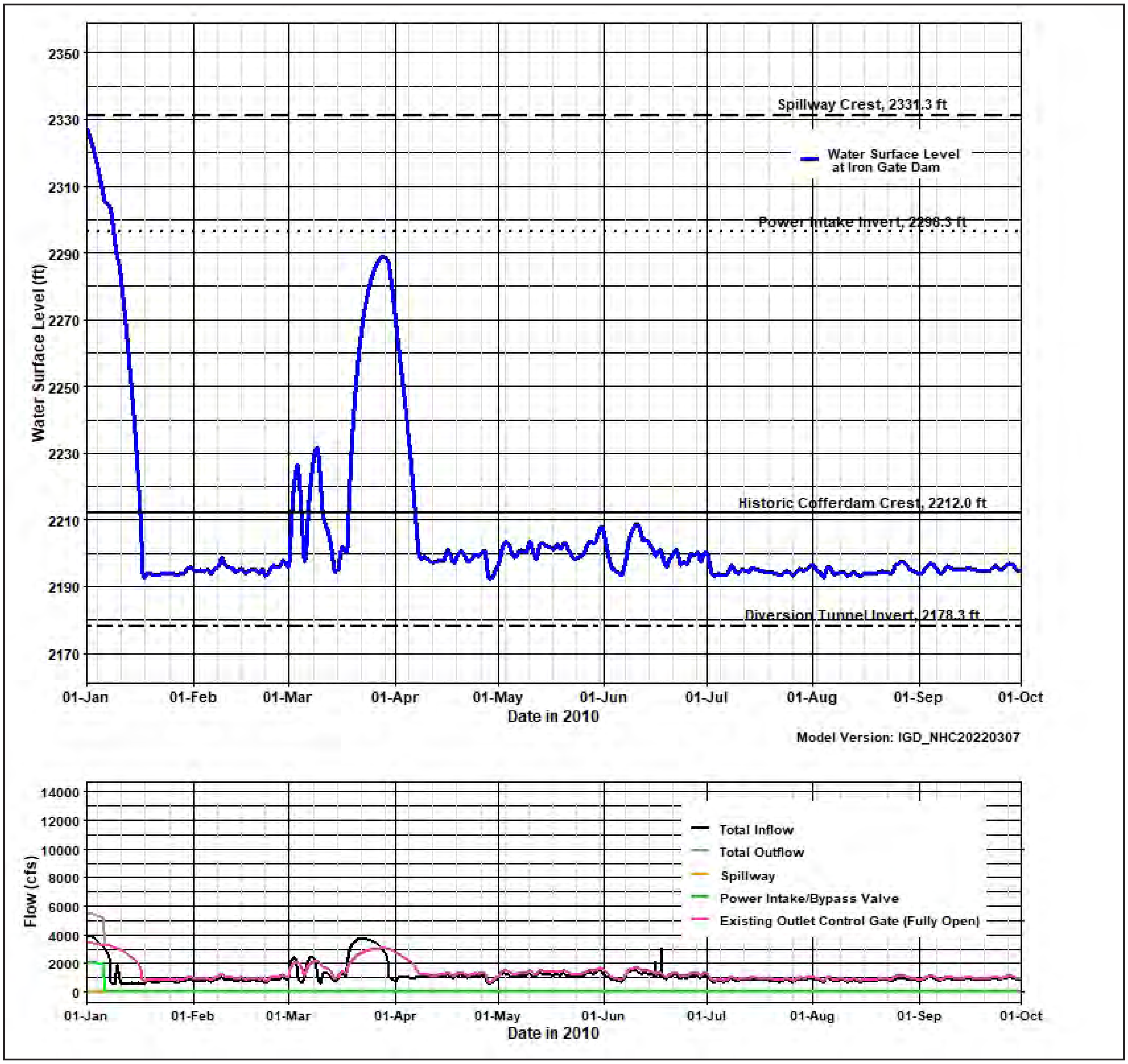

Figure 30 - Iron Gate Dam Facility Simulated Drawdown - Year 2010

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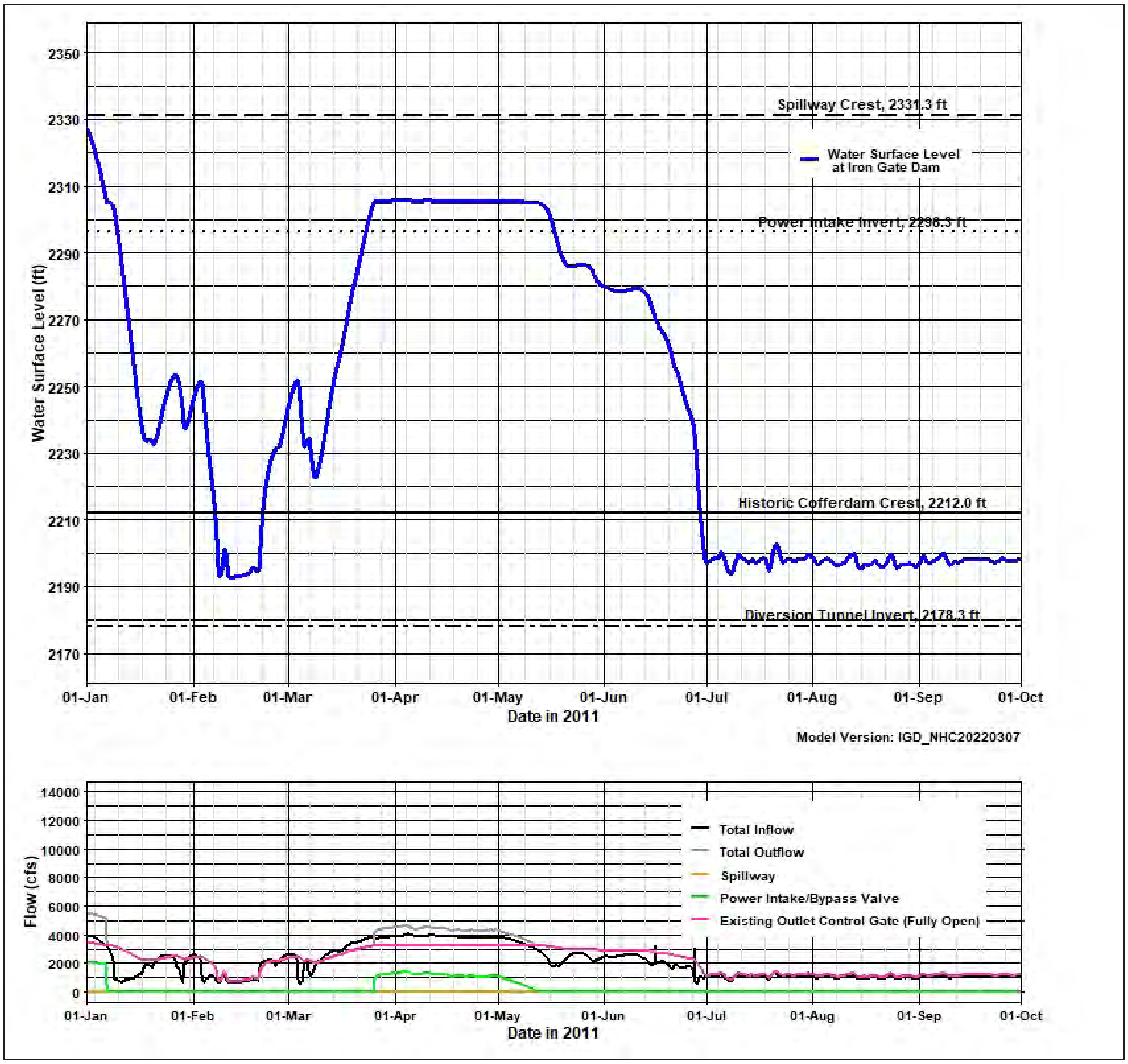

Figure 31 - Iron Gate Dam Facility Simulated Drawdown - Year 2011

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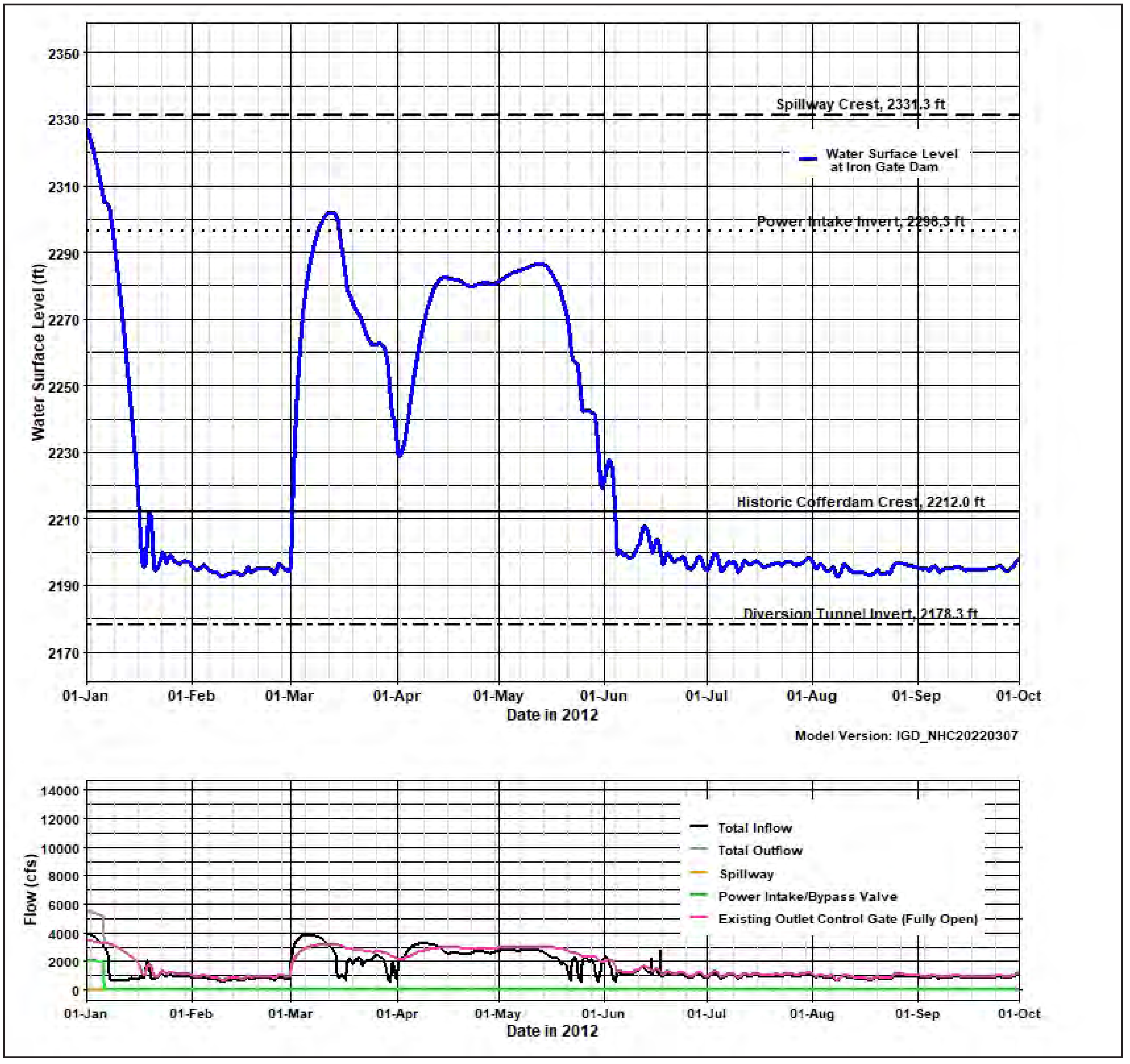

Figure 32 - Iron Gate Dam Facility Simulated Drawdown - Year 2012

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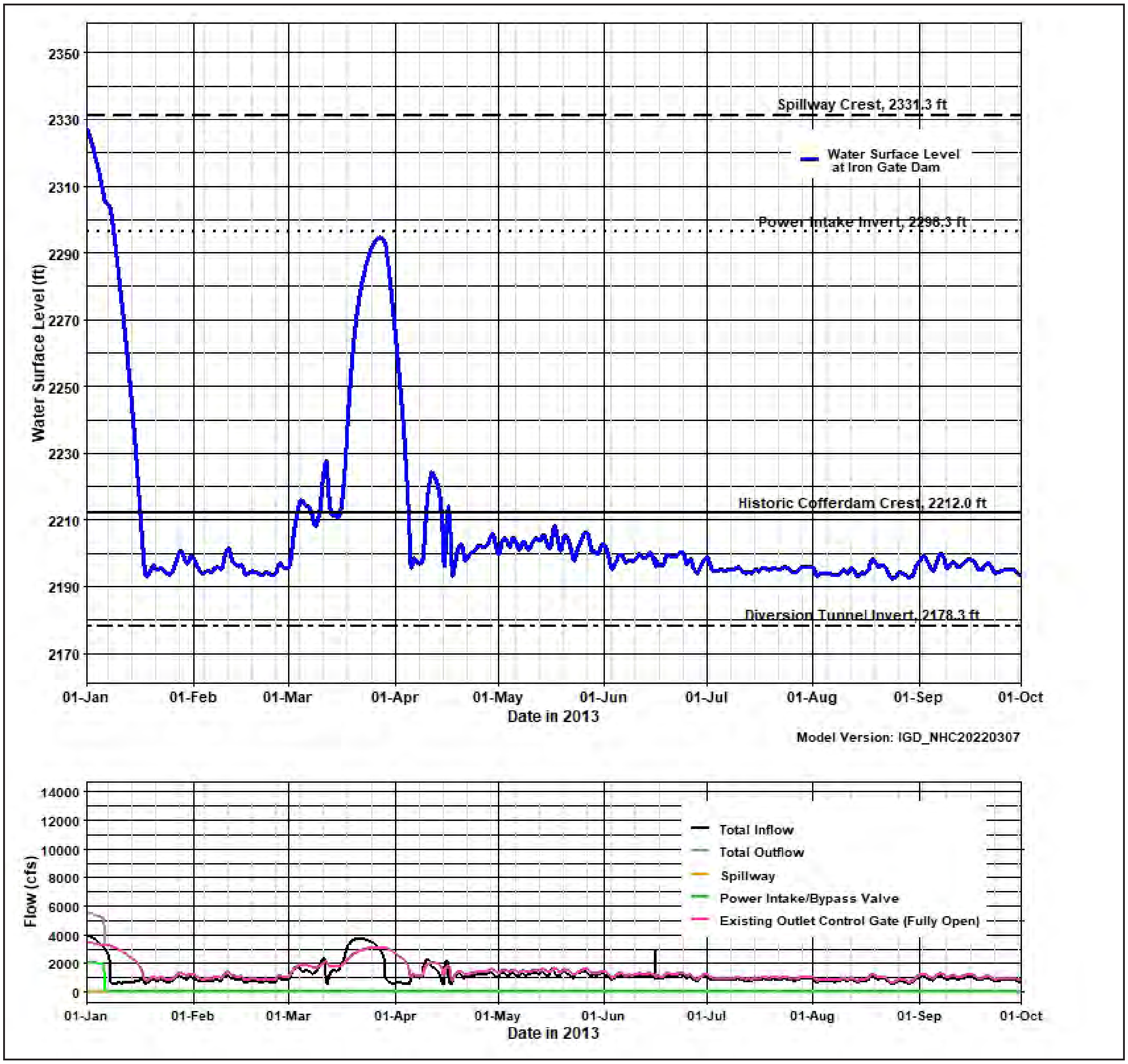

Figure 33 - Iron Gate Dam Facility Simulated Drawdown - Year 2013

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SIMULATED DRAWDOWN

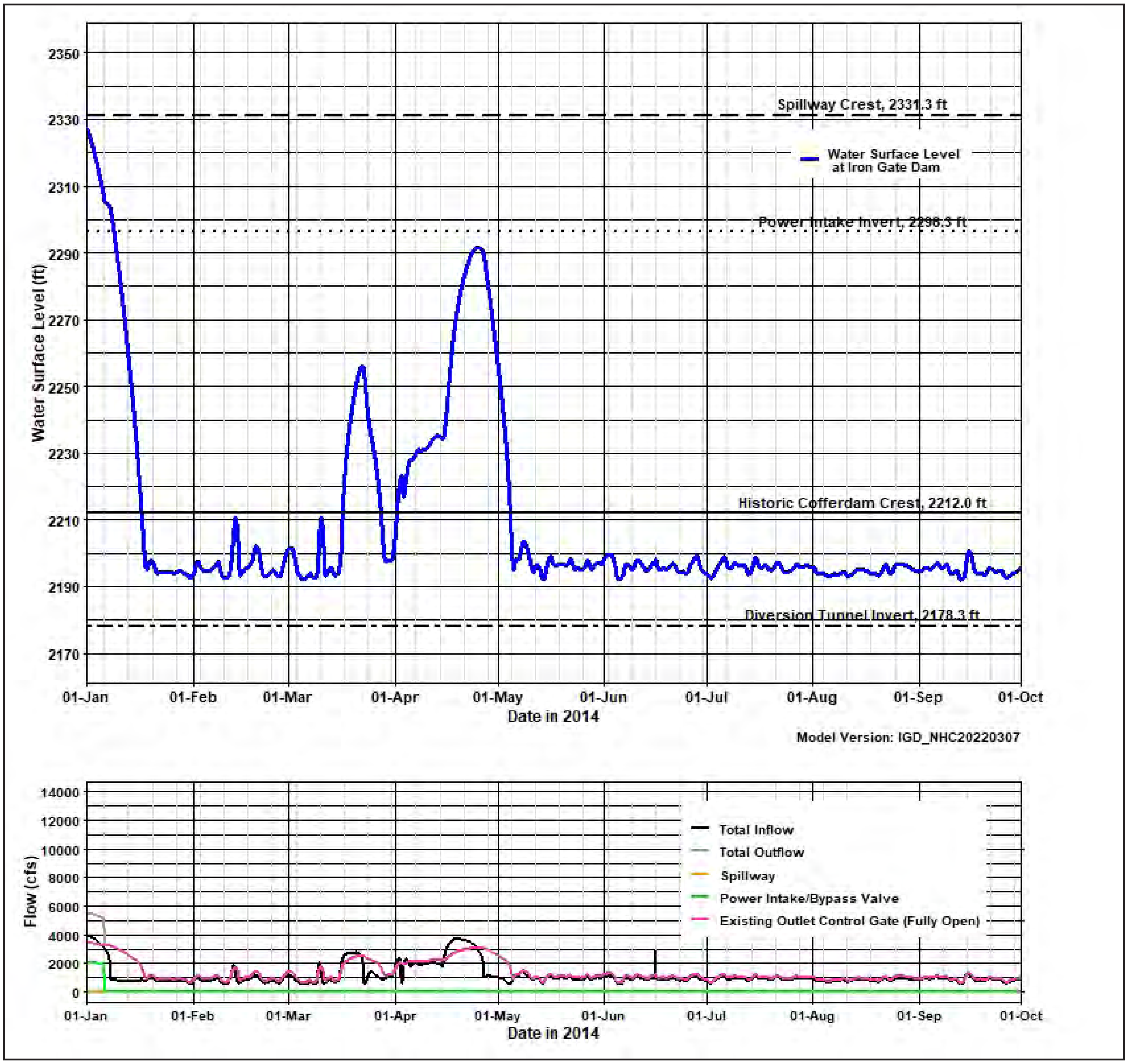

Figure 34 - Iron Gate Dam Facility Simulated Drawdown - Year 2014

April 22, 2022

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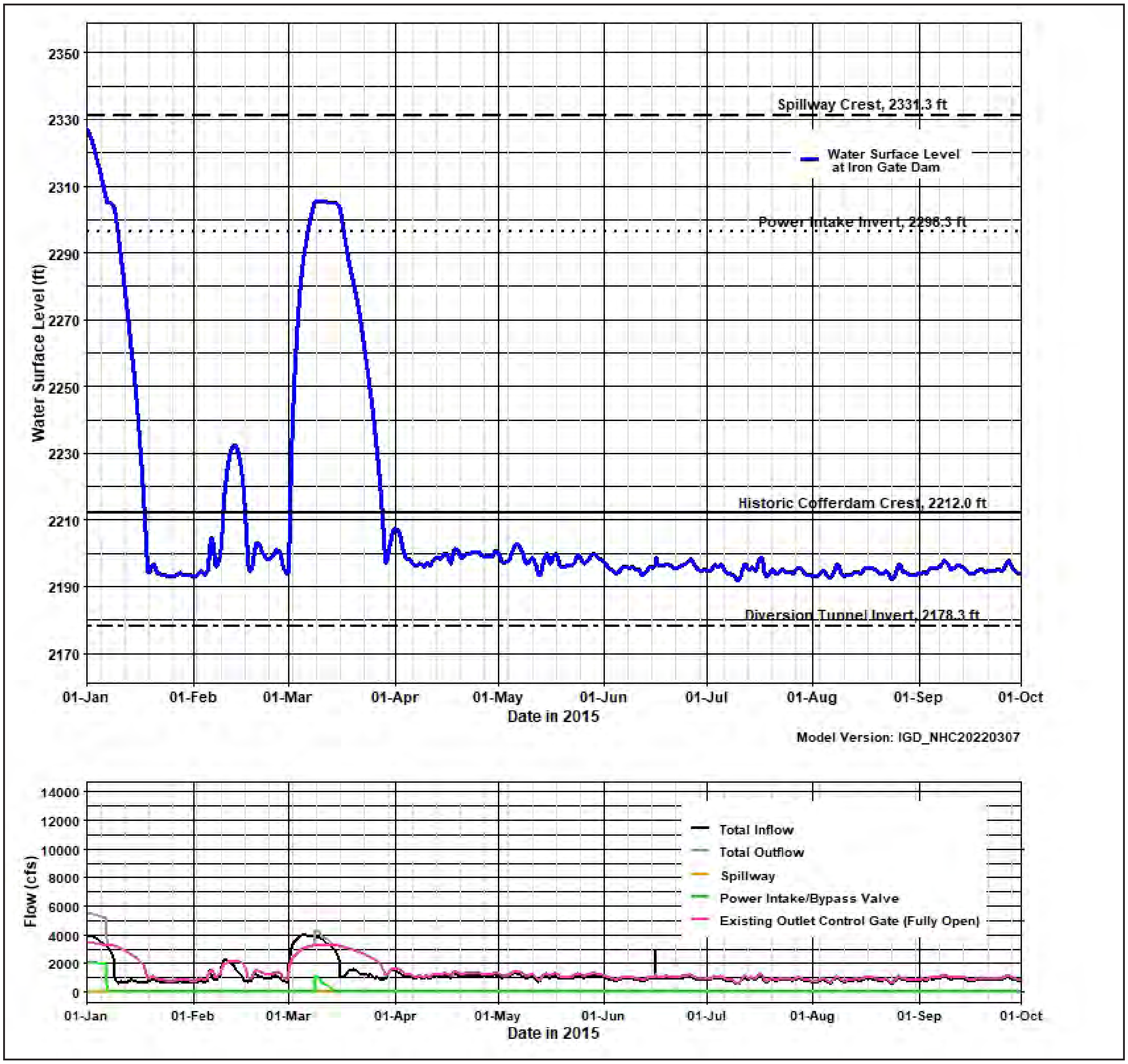

Figure 35 - Iron Gate Dam Facility Simulated Drawdown - Year 2015

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

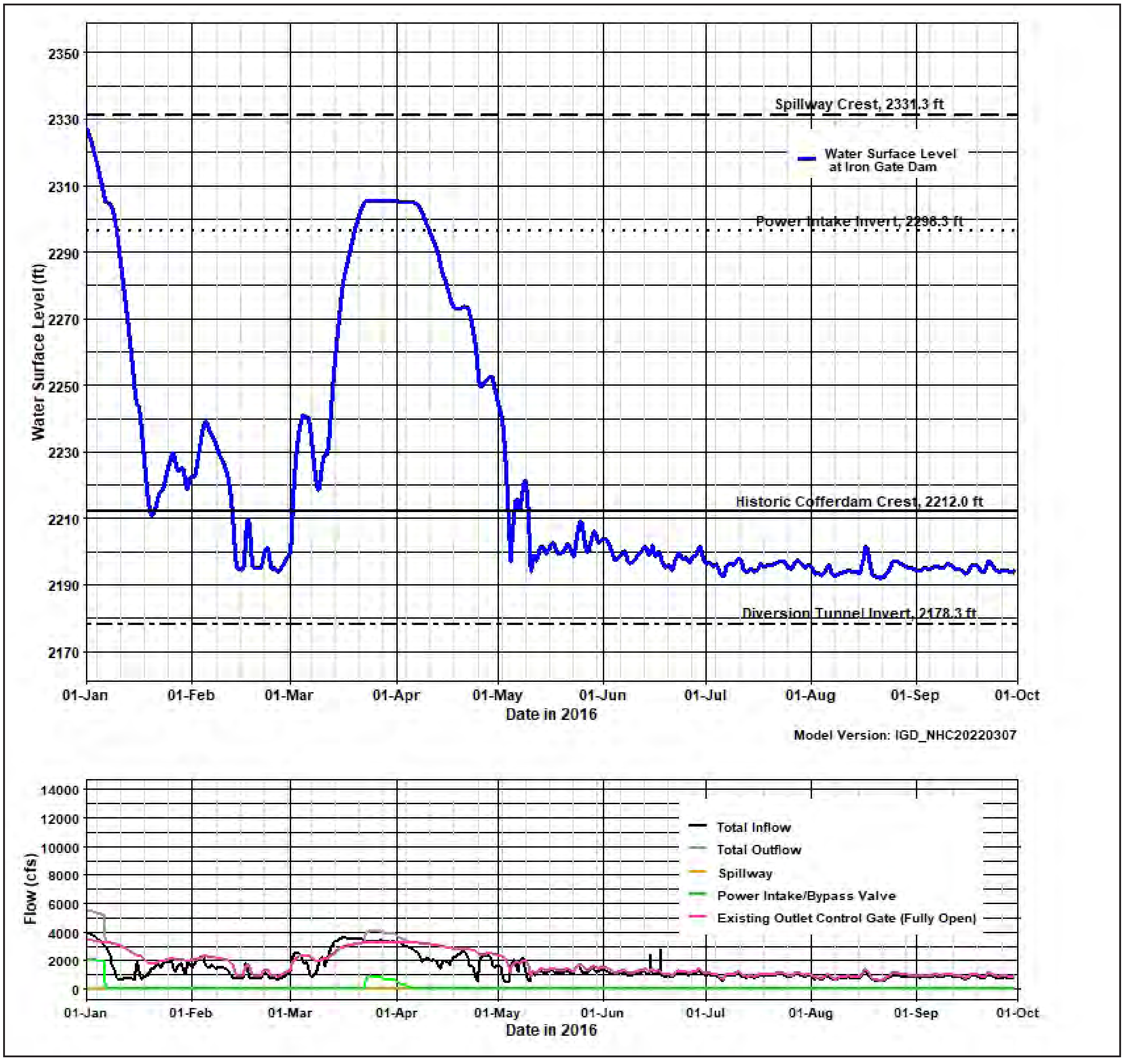

Figure 36 - Iron Gate Dam Facility Simulated Drawdown - Year 2016

Management System Certified by:

MEMORANDUM

Date:	April 29, 2021	**File No.:**	VA103-00640/01-A.01
		Cont. No.:	VA21-00482
To:	Mr. Nick Drury (Kiewit)		
Copy To:	Erik Esparza (Kiewit), Norm Bishop (KP)		
From:	K. Wechselberger		
Re:	**Iron Gate Tunnel Modeling – Updated with 2021 Survey Results (Draft, Rev D)**		

1.0 INTRODUCTION

The Iron Gate Tunnel CFD modeling completed and submitted as part of the Draft 100% Design Deliverables was based on a geometry and information obtained from historical construction drawings and visually observed conditions during preliminary tunnel investigations completed in 2019. The 2019 preliminary visual inspections identified variability from the interior dimensions in the historical construction drawings and confirmed the need for a detailed tunnel survey. Detailed survey of the tunnel downstream of the blind flange and 9 ft orifice was conducted and processed by the Yurok Tribe at the end of 2020 and made available to Knight Piésold in January 2021.

Initial validation of the Iron Gate tunnel design concept was subsequently completed based on updated CFD modeling that incorporated the recent Yurok survey results and a best-fit concrete liner consistent with the 100% design concept. This memorandum presents the CFD analysis results and observations of the tunnel's hydraulic behavior for the following flow conditions with and without the addition of tunnel concrete liner:

- Existing diversion tunnel low level outlet upper gate section fully opened to a maximum height of 57".
- Varying reservoir water surface levels between:
  - The spillway crest at El 2,331.3 ft. This represents a starting condition for reservoir drawdown.
  - Water surface level at El. 2202 ft. This represents the height of the final breach plug.

2.0 SURVEY RESULTS

The downstream tunnel survey was completed by the Yurok Tribe between November 17 and November 20, 2020. Details of the work are summarized in a technical memo titled, "Iron Gate Low-Level Outlet Survey Data Acquisition and Processing" (Yurok Tribe, December 11, 2020).

The downstream tunnel contains gate leakage and seepage water which is contained by a stoplog weir at the outlet of the tunnel. Removal of the stop logs was not permitted during the Yurok survey and the tunnel was not dewatered. The water depth varied but was chest deep in some tunnel locations. Two forms of survey data were collected:

- LiDAR data was collected for portions of the tunnel above the water surface.
- Total station survey was completed to capture the bathymetry, or tunnel invert geometry, below the water surface.

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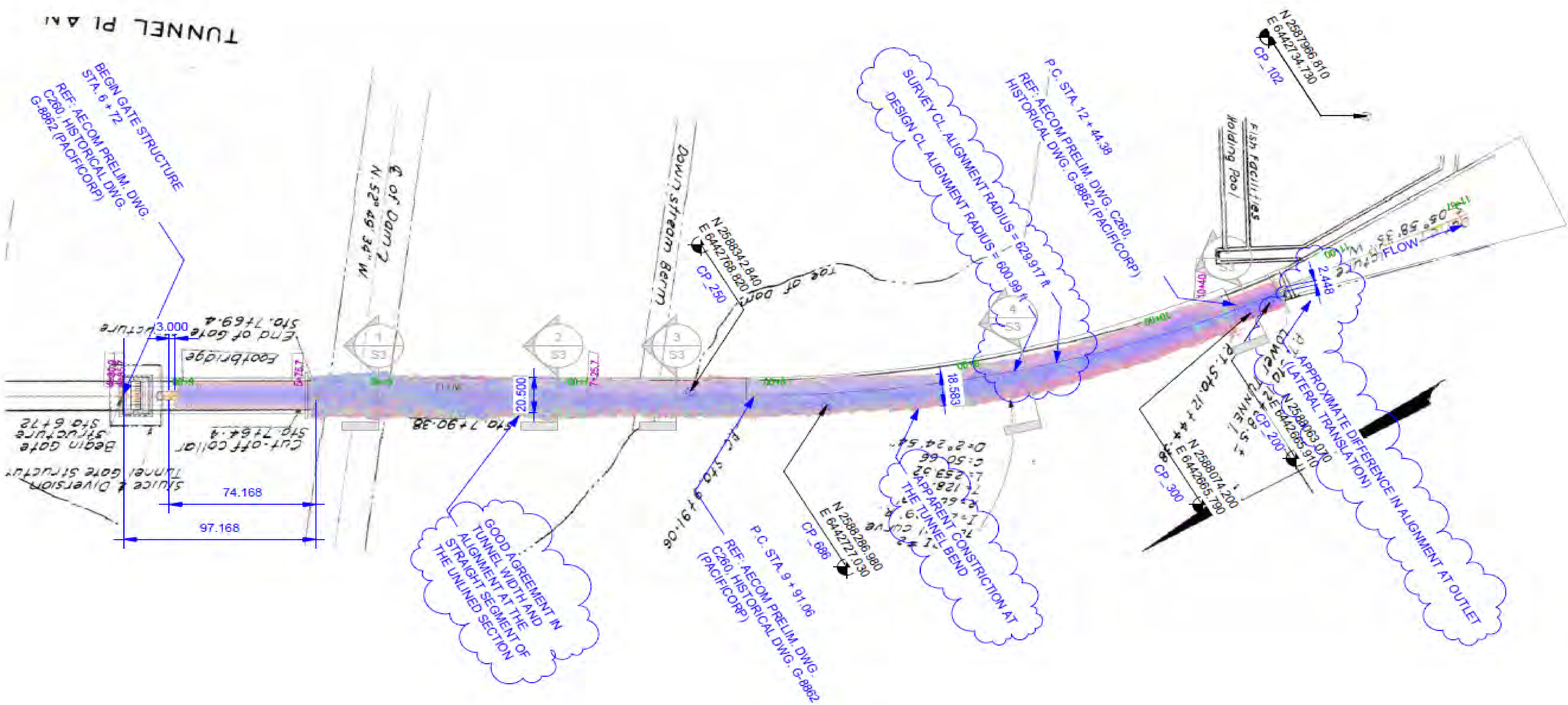
2.1 COMPARISON WITH IDEALIZED GEOMETRY USED IN 100% DESIGN DELIVERABLES

Good agreement was observed in the existing reinforced concrete liner that extends from the gate structure to approximately 100 ft downstream of the gate structure. Upon leaving the existing concrete reinforced liner and entering the unlined portion of the tunnel, differences between the tunnel cross section geometry and invert elevations warranted further investigation into the resulting hydraulic behavior, with updated CFD modeling. Two Figures that follow demonstrate the observed variability:

- Figure 2.1 shows the tunnel alignment in plan with the point cloud data overlayed on the historical tunnel data, with key features annotated.
- Figure 2.2 shows five sections along the tunnel alignment compared with the 100% design geometry overlayed for demonstration.

Key observations included:

- Increased surface rock roughness and irregularity in the unlined portion of the tunnel compared to the historic drawing tunnel design geometry.
- The majority of the tunnel invert was found to be higher than shown on the historic drawings. This resulted in a reduced average invert slope from what was historically reported as 0.64% to an actual average of 0.34% from the 9 ft orifice to the outlet portal.
- The expansion and contraction when exiting and re-entering the existing reinforced concrete liner was found to be less pronounced than shown on the historic drawings and than modelled in the 100% design. The zones of expansion and contraction are hydraulically significant geometric features that aid in energy dissipation and initiation of the hydraulic jump in the tunnel.
- A low point in the tunnel invert is observed in the tunnel profile between 250 ft and 300 ft downstream of the diversion tunnel gate. This is shown in Figure 2.1, cross section 2 above. This is the only portion of the tunnel where the Yurok surveyed tunnel invert is below the geometry used in the 100% design.
- A lateral constriction in tunnel cross-section is seen in the bend where the tunnel section width narrows from approximately 20.5 ft to 18.5 ft. This is shown in Figure 2.2.

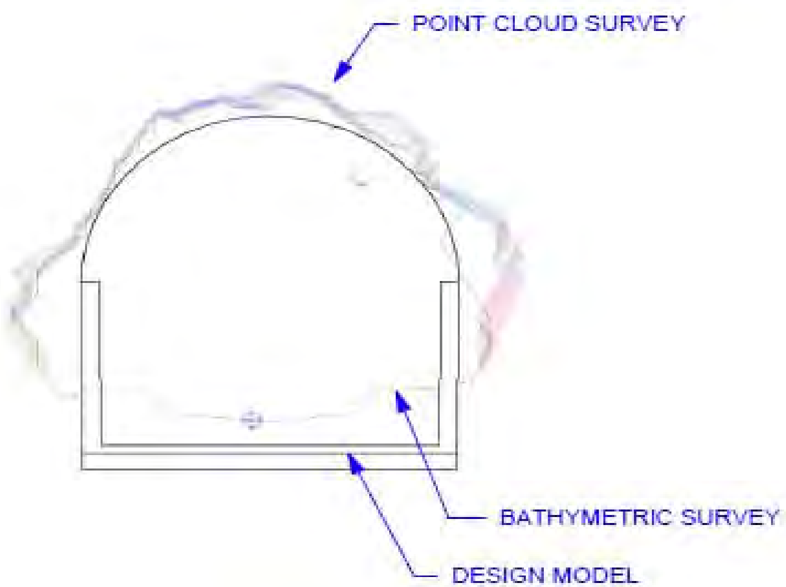


NOTES:

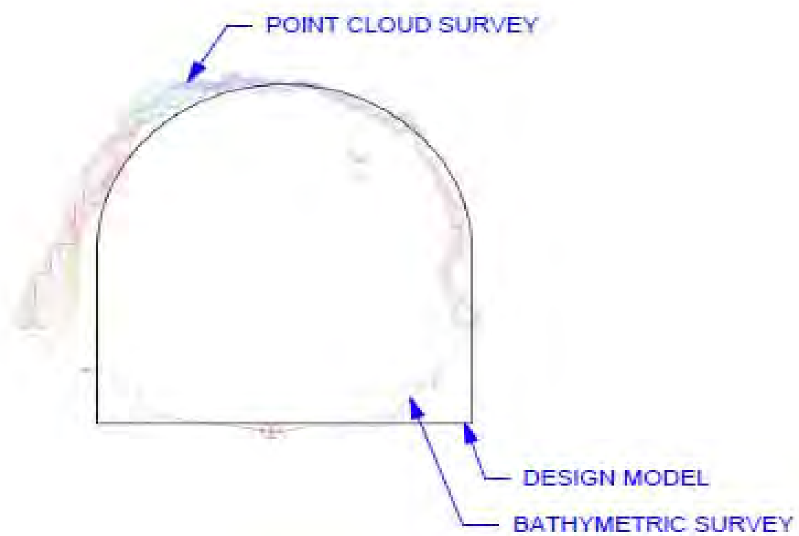
1. DIMESIONS AND ELEVATIONS IN FEET (ft).
2. STATIONING CONSISTENT HISTORICAL DRAWINGS FOR EASE OF COMPARISON.

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	29APR'21	ISSUED WITH MEMO	CPC	KTW

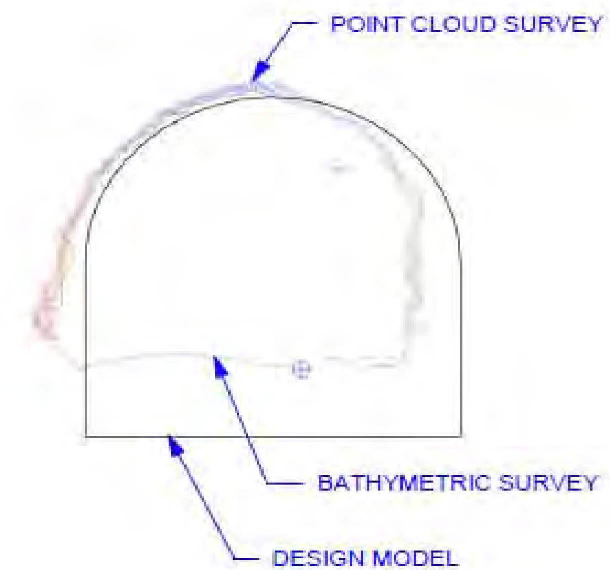
Kiewit Infrastructure West Co. (California)	
Klamath River Renewal Project	
IRON GATE DIVERSION TUNNEL DOWNSTREAM TUNNEL SURVEY COORDINATION PLAN	
Knight Piésold CONSULTING	P/A NO. VA103-00640/1; REF. NO. VA21-00482
FIGURE 2.1	REV 0



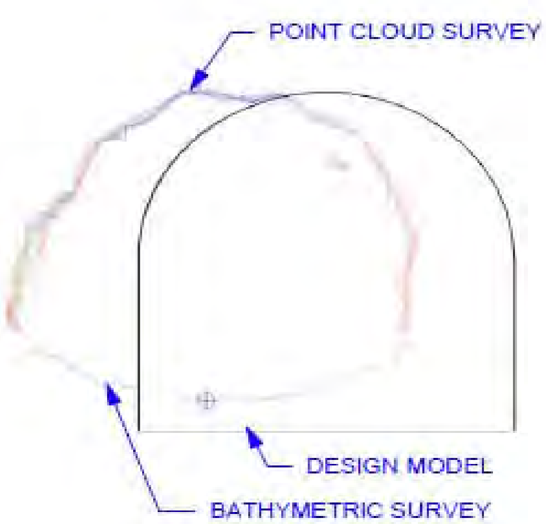
SECTION 1



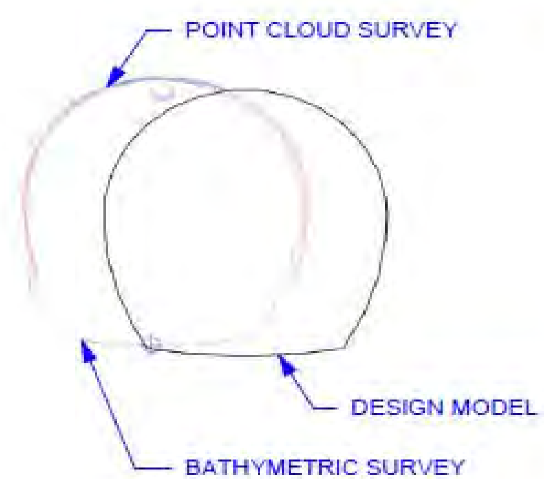
SECTION 2



SECTION 3



SECTION 4



SECTION 5

NOTES:

1. DIMESIONS AND ELEVATIONS IN FEET (ft).
2. STATIONING CONSISTENT HISTORICAL DRAWINGS FOR EASE OF COMPARISON.
3. SURVEYED TUNNEL CROSS SECTIONS COMPARED WITH IDEALIZED 100% DESIGN GEOMETRY

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	29APR'21	ISSUED WITH MEMO	CPC	KTW

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
DOWNSTREAM TUNNEL SURVEY COORDINATION SECTIONS

Knight Piésold CONSULTING

P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 2.2

REV 0

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3.0 UPDATED MODEL GEOMETRY

3.1 SURVEY DATA MANIPULATION AND ROUGHNESS PARAMETERS

Characteristic of the Yurok survey methods utilized, the total station survey bathymetric data and the LiDAR data yielded very different point density. Manipulation and filtering of the point cloud data generated by the LiDAR was required to construct a tunnel surface geometry that could be meaningfully used and meshed by ANSYS, the CFD analysis software. This was completed by creating intersection lines around the circumference of a circle divided into 24 segments and connecting the neighboring cross sections at 5 ft spacing. This is demonstrated in Figure 3.1 below.

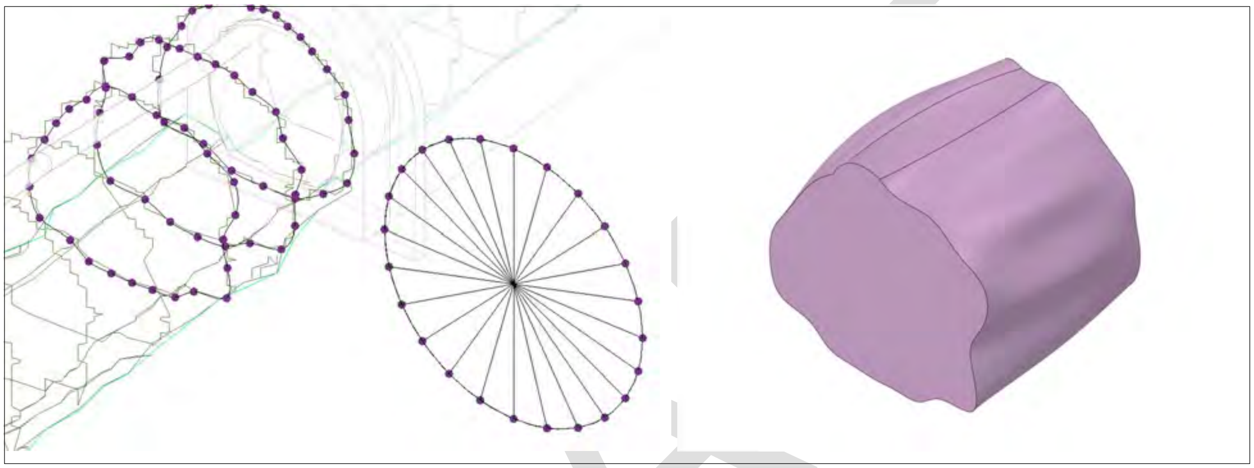

Figure 3.1 Construction of Model Geometry – Partial Segment of Tunnel

This approach effectively captured the macro roughness of the unlined bedrock tunnel geometry to a minimum mesh size of 0.25 ft. No other micro roughness was added to the surface in the model parameters. The following additional manual modifications were made to the tunnel Yurok survey geometry:

- A rock pile observed during the 2019 tunnel site investigation was captured in the invert geometry. This will be removed by the Contractor and was therefore removed in the model. This rock pile is located just downstream of the low point in the tunnel invert. That area is modelled as consistent with the invert level downstream of the rockpile.
- The model includes removal of the existing small diameter ventilation pipe along the crown and addition of the larger ventilation requirements consistent with the 100% design deliverables.

3.2 INCORPORATION OF LINER GEOMETRY

The 100% design deliverables showed a new reinforced concrete side wall and invert liner constructed for a length 150 ft downstream of the existing reinforced concrete liner in the portion of the tunnel where the velocities were expected to remain higher than acceptable for an unlined tunnel during higher reservoir levels (see Section 1 in Figure 2.1 above). This new reinforced concrete liner section is upstream of where initially the hydraulic jump is expected to fill the tunnel and bring average velocities in line with those acceptable for an unlined tunnel. This memo considers the results of two geometry cases as follows:

1. The as-surveyed tunnel invert geometry is not modified or raised to include a constructed liner in this unlined portion of the tunnel. The CFD model uses the as-surveyed invert elevations with cross sections established as described above in Section 3.1.

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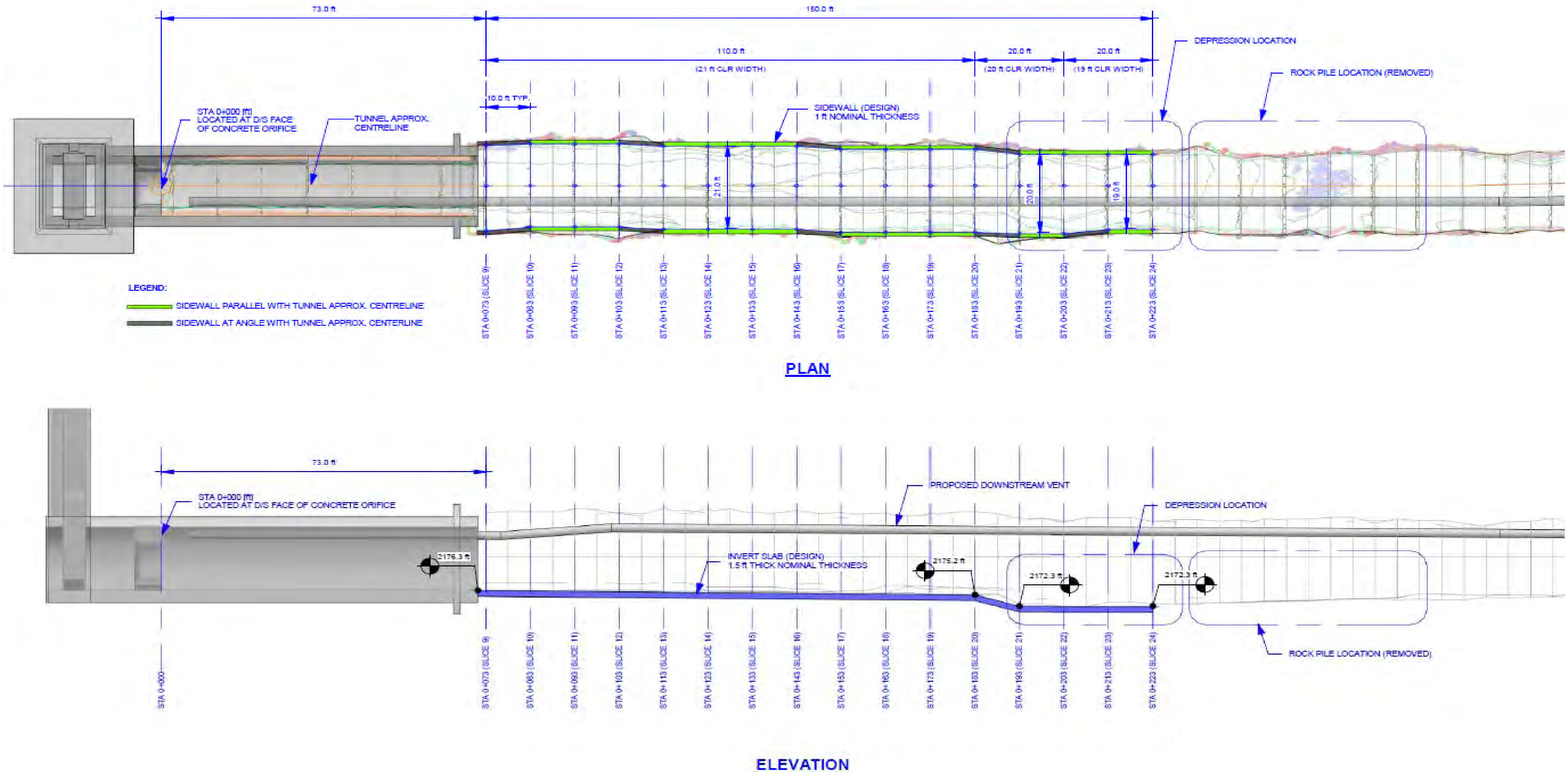
2. A "best-fit" concrete liner similar in concept to the 100% design, and reviewed by the Contractor for constructability, was incorporated from the end of the existing reinforced concrete liner to end of the low-point described above. This is approximately 150 ft in total length of concrete liner and considers the need to maintain the invert geometry. This best fit liner is characterised by:
   - A minimum cross section hydraulic width of 21 ft, widening where practical to minimize the concrete thickness and mimic some of the existing tunnel variability in tunnel width. The excavated tunnel width surveyed in the region varies from approximately 21 ft to 26 ft.
   - Tunnel invert excavated to establish concrete liner that matches the surveyed tunnel invert low point, with the rock side haunches along the tunnel invert removed to create a uniform hydraulic level invert width and connectivity with the side walls.

Figures 3.2 through 3.4 show the incorporated best-fit liner described in Geometry Case 2.



NOTES:

1. DIMESIONS AND ELEVATIONS IN FEET (ft).
2. STATIONING IS ZEROED AT THE DOWNSTREAM FACE OF THE CONCRETE ORIFICE FOR THE PURPOSES OF COORDINATION WITH REFERENCE DISTANCES USED IN THE CFD MODEL.

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0	29APR'21	ISSUED WITH MEMO	CPC	KTW

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
DOWNSTREAM TUNNEL SURVEY COORDINATION
BEST FIT LINER PLAN AND PROFILE

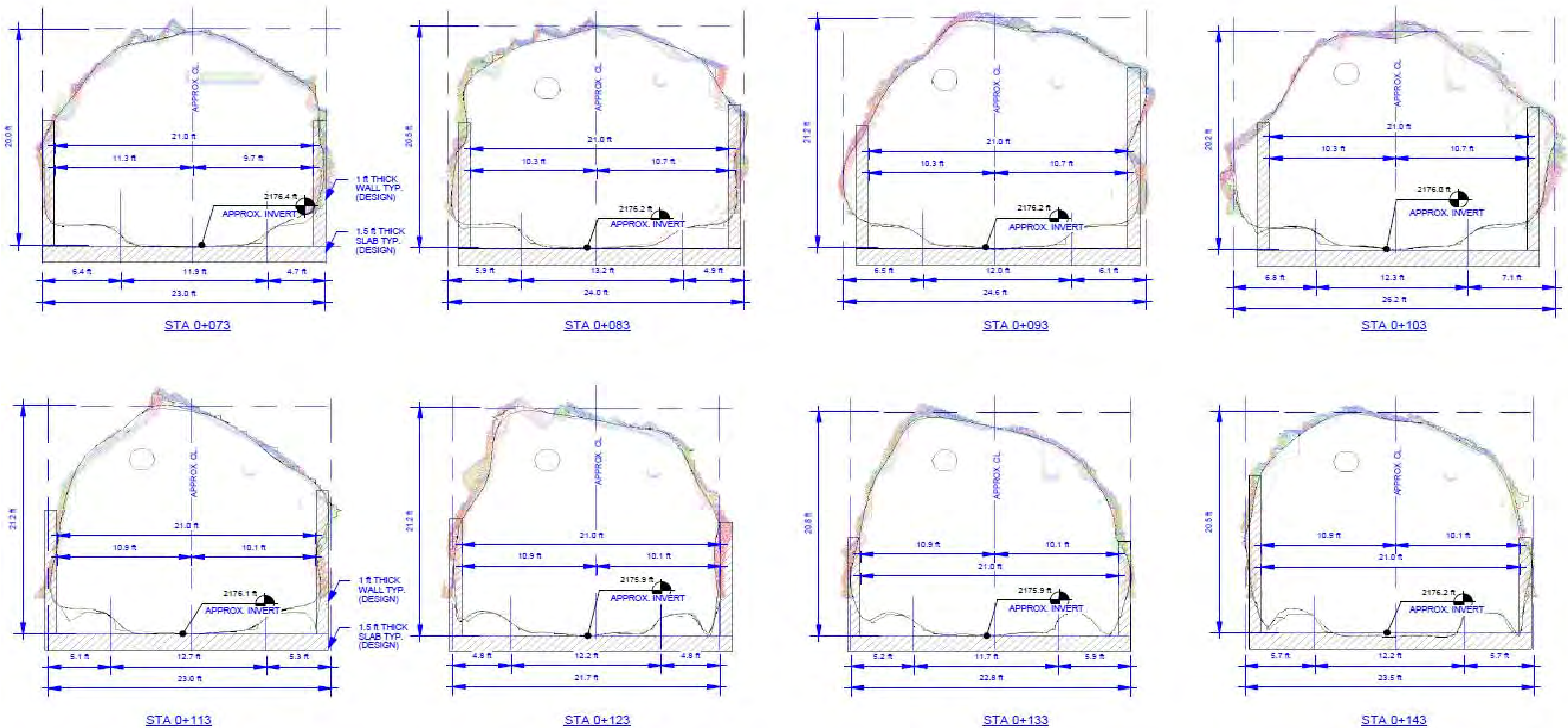
Knight Piésold CONSULTING

P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 3.2

REV 0



NOTES:

1. DIMESIONS AND ELEVATIONS IN FEET (ft).
2. STATIONING IS ZEROED AT THE DOWNSTREAM FACE OF THE CONCRETE ORIFICE FOR THE PURPOSES OF COORDINATION WITH REFERENCE DISTANCES USED IN THE CFD MODEL.

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Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
DOWNSTREAM TUNNEL SURVEY COORDINATION
BEST FIT LINER SECTIONS (1 OF 2)

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P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 3.3

REV 0

NOTES:

1. DIMESIONS AND ELEVATIONS IN FEET (ft).
2. STATIONING IS ZEROED AT THE DOWNSTREAM FACE OF THE CONCRETE ORIFICE FOR THE PURPOSES OF COORDINATION WITH REFERENCE DISTANCES USED IN THE CFD MODEL.

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IRON GATE DIVERSION TUNNEL
DOWNSTREAM TUNNEL SURVEY COORDINATION
BEST FIT LINER SECTIONS (2 OF 2)

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P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 3.4

REV 0

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4.0 RESULTS AND OBSERVATIONS

4.1 VEOCITY REDUCTION AND ENERGY DISSIPATION

The water surface elevation at the spillway crest represents the highest head and flow to be experienced during the Iron Gate reservoir drawdown.. One of the primary goals of this modelling was to conceptually validate the 100% design concept that utilized a reinforced concrete liner to protect the tunnel integrity in the zone of sustained high velocities and turbulence combined with the tunnel contraction at the existing reinforced concrete outlet portal to initiate a hydraulic jump and dissipate energy.

The variation in tunnel geometry as shown in the recent survey has reduced the contraction at the tunnel portal, but as observed in Figure 4.1, the desired hydraulic energy dissipation is achieved. Figure 4.2 presents the near atmospheric air pressures throughout resulting from the proposed air venting. Both geometry cases considered as part of this modelling produced reasonably consistent hydraulic behavior to the 100% design concept as it relates to the desired energy dissipation.

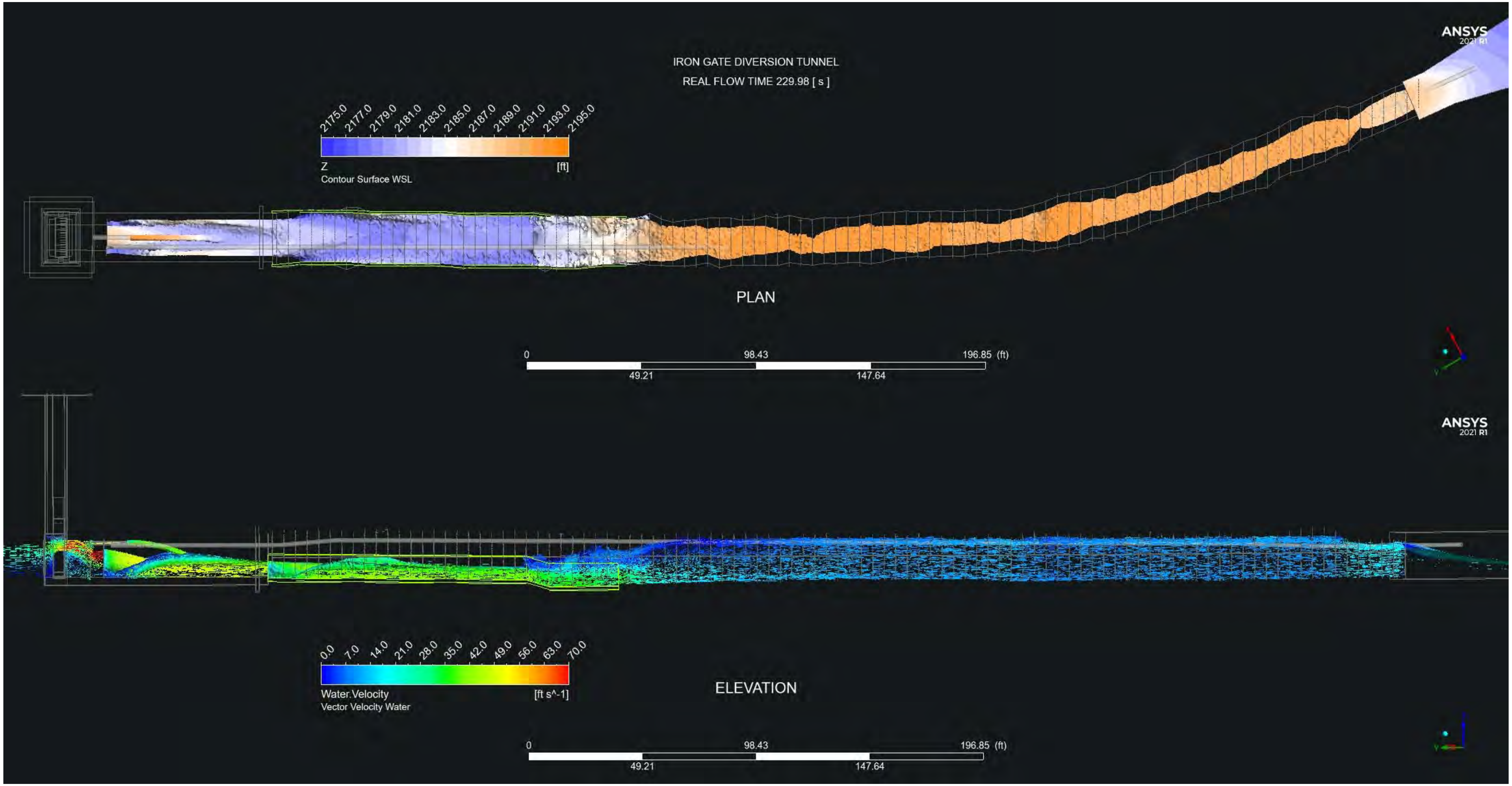
The following table summarizes the water velocities observed in the tunnel when the reservoir is discharging through the tunnel at reservoir WSL El. 2331.3.

Table 4.1 Water Velocity Range by Tunnel Region

Location inside Tunnel	Observed Water Velocity Range (ft/s)
Existing reinforced horseshoe concrete liner, approximately 100 ft downstream of gate structure	40 to 50
150 ft downstream of new reinforced concrete liner, downstream of existing horseshoe concrete liner.	25 to 45
Unlined tunnel between invert low point and existing reinforced outlet liner/structure	5 to 15
Existing reinforced outlet liner/structure	15 to 20
Outlet velocity	10 to 15

Other observations demonstrated in Figures 4.1 and 4.2 include:

- No adverse air pressures (positive or negative) develop inside and throughout the tunnel, suggesting adequate ventilation is provided with the proposed 100% design crown ventilation.
- Anchorage design of the ventilation proposed in the 100% design is considered adequate. There is no increase in the hydraulic drag and impact forces from those considered in the 100% design. The variability in the elevation of the tunnel crown was anticipated and accounted for in the installation notes specified on 100% Design Drg. C4125. The selection of a full circumference vent pipe clamp support provides vertical and horizontal support to the vent pipe. The minimum embedment depths into competent rock shall be maintained and the total length of steel rod adjusted accordingly. Each rock bolt will be individually tested during installation to verify its embedment integrity.
- There is effective dissipation of any hydraulic roller and potential high-pressure zone upstream of the blind flange.



NOTES:

1. RWS = RESERVOIR WATER SURFACE (ELEVATION IN FT).
2. WSL = WATER SURFACE LEVEL (ELEVATION IN FT).
3. CFD MODEL RUN IN ANSYS FLUENT VERSION 2021 R1.

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Kiewit Infrastructure West Co. (California)

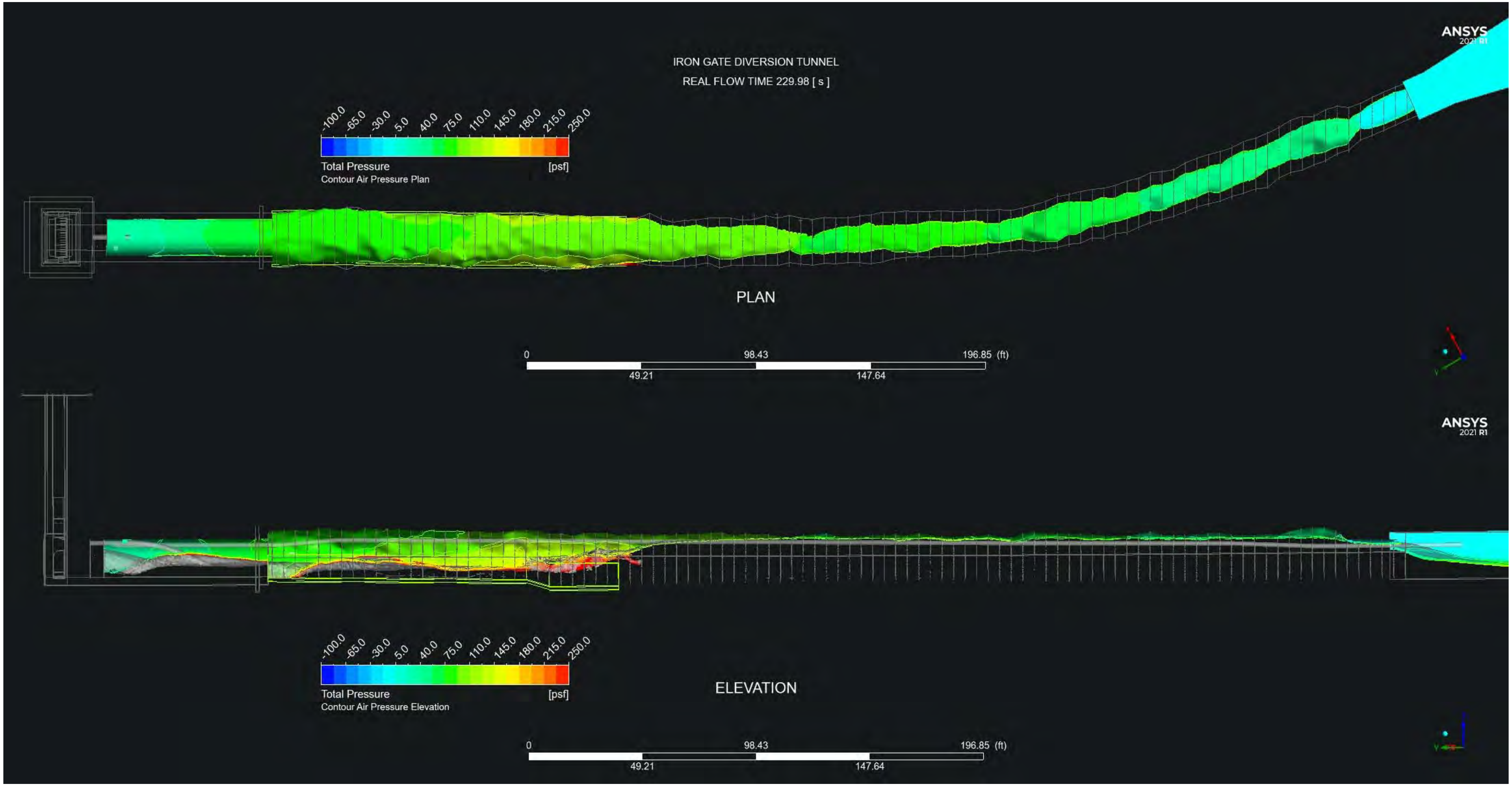
Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
BEST FIT LINED TUNNEL | RWS 2331.3 ft
CFD RESULTS - FLOW WSL AND VELOCITY

Knight Piésold CONSULTING

P/A NO. VA103-00640/1 | REF. NO. VA21-00482

FIGURE 4.1 | REV 0



NOTES:

1. RWS = RESERVOIR WATER SURFACE (ELEVATION IN FT).
2. WSL = WATER SURFACE LEVEL (ELEVATION IN FT).
3. CFD MODEL RUN IN ANSYS FLUENT VERSION 2021 R1.

0	29APR'21	ISSUED WITH MEMO	CPC	KTW
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Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
BEST FIT LINED TUNNEL | RWS 2331.3 ft
CFD RESULTS - AIR PRESSURE

Knight Piésold CONSULTING

P/A NO.	REF. NO.
VA103-00640/1	VA21-00482

FIGURE 4.2 REV 0

4.2 UPDATED RATING CURVE

The rating curve estimated in the 100% design as compared with the recommended rating curve resulting from the analysis presented herein is presented in Figure 4.3 below.

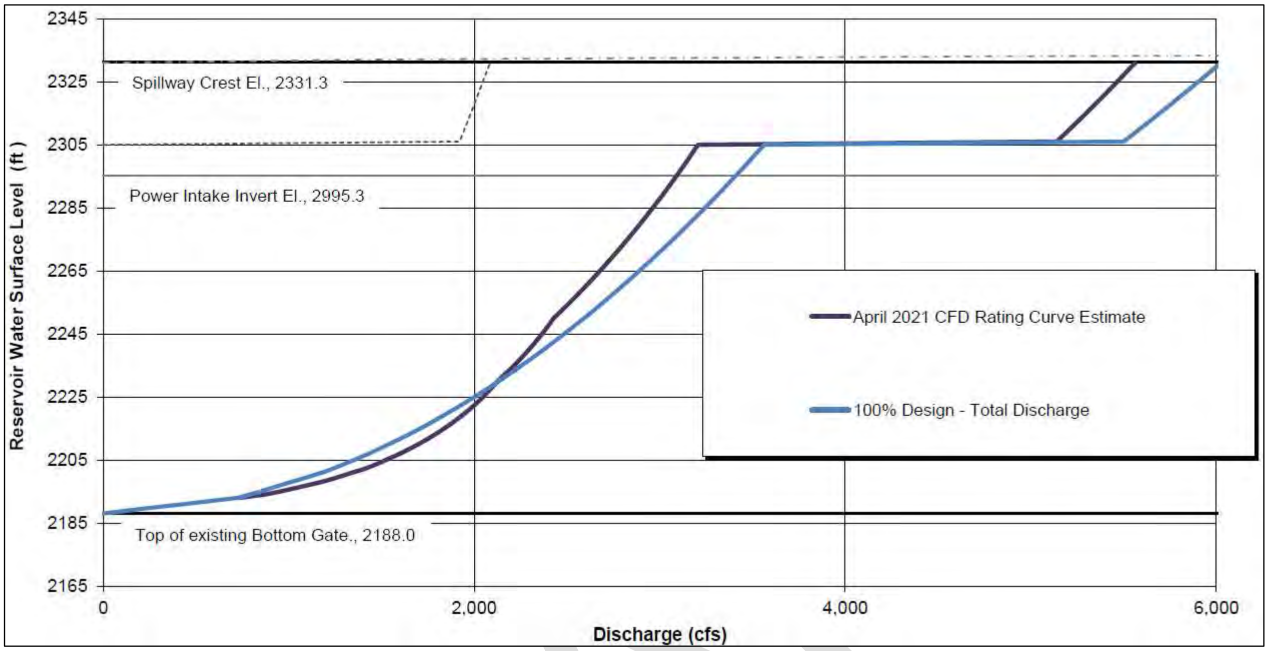

Figure 4.3 Updated Rating Curve

The tunnel discharge rating curve is the primary tool to evaluate the 1% and 5% semi-monthly water surface levels being used to plan the dam embankment removal milestones. The results show:

- a 5 to 10% reduction in flow capacity at the highest water surface level than what was reported in the 100% design.
- An increase in capacity at the lower modelled elevations between 2250 and 2202 ft.

In Table 4.2 values from the above curve titled, "April 2021 CFD Rating Curve Estimate" have been used to update and compare the relevant semi-monthly steady state water surface levels previously reported on 100% Design Drg C4055.

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TABLE 4.2

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IRON GATE DAM - PRE-DRAWDOWN CONSTRUCTION
FLOOD FLOWS AND STEADY-STATE WATER SURFACE LEVELS DURING DRAWDOWN

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Flow Condition[1]		June 1 - 15				June 16 -30				July 1-15				July 16 - 31			
		Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)
Statistical High Water (Flood Conditions)	1% Probable Flood	8700	2332.4	2332.5	0.1	4400	2305.4	2305.6	0.2	3,400	2294.5	2305.1	10.6	2,100	2228.7	2228.2	-0.5
	5% Probable Flood	4800	2305.6	2305.8	0.2	3000	2270.9	2287.8	16.9	2,100	2228.7	2228.2	-0.5	1,700	2214.7	2210.0	-4.7
	20% Probable Flood	3000	2270.9	2287.8	16.9	2000	2224.9	2222.3	-2.6	1,400	2206.1	2202.0	-4.1	1,400	2206.1	2202.0	-4.1
	50% Probable Flood	2000	2224.9	2222.3	-2.6	1500	2208.8	2204.3	-4.5	1,200	2201.4	2198.4	-3.0	1,300	2203.7	2200.2	-3.5
Monthly Flow Duration 25% of Time Equaled or Exceeded		1950	2223.1	2219.8	-3.3	1300	2203.7	2200.2	-3.5	1,140	2200.2	2197.5	-2.7	1,120	2199.9	2197.2	-2.7
Mean Monthly Flow		1720	2215.3	2210.7	-4.6	1280	2203.2	2199.8	-3.4	1,050	2198.6	2196.3	-2.3	1,050	2198.6	2196.3	-2.3
Monthly Flow Duration 75% of Time Equaled or Exceeded		1150	2200.4	2197.6	-2.8	1030	2198.3	2196.0	-2.3	910	2196.1	2194.5	-1.6	930	2196.5	2194.7	-1.8

Flow Condition		August				Sept 1 - 15				Sept 16 - 30				Oct 1 - 15			
		Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)	Flow (cfs)	Reservoir Surface Level (RSL)[2] 100% Design Rev C	Reservoir Surface Level (RSL)[2] April 2021 Rating Curve[3]	Change in RSL (ft)
Statistical High Water (Flood Conditions)	1% Probable Flood	2100	2228.7	2228.2	-0.5	2300	2236.8	2240.3	3.5	2700	2255.2	2266.4	11.2	7400	2332.0	2332.1	0.1
	5% Probable Flood	1700	2214.7	2210.0	-4.7	2000	2224.9	2222.3	-2.6	2200	2232.7	2234.1	1.4	6200	2331.1	2331.4	0.3
	20% Probable Flood	1400	2206.1	2202.0	-4.1	1700	2214.7	2210.0	-4.7	1700	2214.7	2210.0	-4.7	6200	2331.1	2331.4	0.3
	50% Probable Flood	1300	2203.7	2200.2	-3.5	1400	2206.1	2202.0	-4.1	1400	2206.1	2202.0	-4.1	5400	2305.9	2321.0	15.1
Monthly Flow Duration 25% of Time Equaled or Exceeded		1110	2199.7	2197.1	-2.6	1170	2200.8	2197.9	-2.9	1170	2200.8	2197.9	-2.9	1170	2200.8	2197.9	-2.9
Mean Monthly Flow		1040	2198.4	2196.1	-2.3	1090	2199.3	2196.8	-2.5	1090	2199.3	2196.8	-2.5	1120	2199.9	2197.2	-2.7
Monthly Flow Duration 75% of Time Equaled or Exceeded		900	2196.0	2194.3	-1.7	1000	2197.7	2195.6	-2.1	1010	2197.9	2195.7	-2.2	1010	2197.9	2195.7	-2.2

M:\1\03\00640\01\A\Data\Task 1400 - 100% Design\04 - Iron Gate\WFXX - Tunnel Survey and Model Coordination\Rating Curve Sensitivity\[4_IronGate_Summary_Hydraulics - April 2021 CFD Update KTW (Memo Rev D).xlsm]C4050_Tab1_IGD_2021

NOTES:

1. FLOWS ARE UNCHANGED FROM FROM THOSE PRESENTED IN THE 100% DESIGN Rev C
2. ALL RESERVOIR SURFACE LEVELS ARE STEADY STATE WITH THE GATE FULLY OPEN
3. ESTIMATED RATING CURVE IS BASED ON THREE DATA POINTS FROM APRIL 2021 CFD MODELLING (Cont. No. VA21-00482, April 8, 2021) THAT INCORPORATES THE NOVEMBER 2020 DOWNSTREM TUNNEL SURVEY AND A BEST-FIT REINFORCED CONCRETE LINER, CONSISTENT IN CONCEPT WITH THE 100% DESIGN.
4. WHERE RESREVOIR SURFACE LEVELS EXCEED THE EXCAVATED DAM CREST AS STAGED AND REMOVED THROUGHOUT THE DRAWDOWN YEAR, THE EMBANKMENT WILL BE OVERTOPPED TRIGGERING AN UNPLANNED BREACH.

D	30APR'21	ISSUED WITH MEMO VA21-00482	KTW	NB
REV	DATE	DESCRIPTION	PREP'D	RVW'D

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The following effects on embankment removal scheduling and risk as observed from the values presented in Table 4.2 need to be considered:

- The loss of diversion tunnel flow capacity at the highest elevation of WSL 2331.3 ft has been shown to have limited effect on the embankment removal schedule, as the drawdown in the upper portion of the reservoir above WSL El. 2300 ft can be assisted by the power intake as water can be passed through the penstock and powerhouse bypass valve.
- The loss of capacity at the mid-range reservoir WSL between 2250 ft and 2300 ft affects the removal schedule of the upper portion of the embankment prior to July 16 when based solely on the 1% criteria. Consideration for adopting a 5% probable flood risk beginning July 1 combined with the use upstream facilities to mitigate flows exceeding 5% probable flood flows should be adopted if the embankment removal schedule presented as part of the 100% design is to be maintained.
- The increase in diversion tunnel flow capacity at the lower reservoir WSL of 2202 ft, may reduce the magnitude of the peak breach outflows by lowering reservoir water levels at which the final dam breach is initiated. The increase in diversion tunnel flow at lower reservoir levels reduces the risk of an unplanned Breach Plug overtopping as the Contractor prepares for the final breach event.

5.0 CONCLUSION AND RECOMMENDATIONS

Based on the CFD model results discussed above, the overall high energy hydraulic behavior with the diversion tunnel gate full open to 57" and the reservoir at full pond (spillway crest = WSL 2,331.3 ft), is consistent with the 100% design concept.

The precise location and method of energy dissipation and hydraulic jump formation differs from the CFD computations developed based on historic geometry presented in the 100% design, but the cumulative effect of the following features from the Yurok surveyed geometry achieved the desired and similar effect:

- Geometry irregularity and associated macro roughness.
- Slightly shallower average invert slope along the alignment.
- Gradual expansion when transitioning out of the existing reinforced concrete liner, followed by sudden vertical expansion at invert low point.
- Lateral constriction around the bend.
- Continuous gradual vertical constriction approaching the existing reinforced liner at the outlet.
- Best-fit liner consistent with the intent and concept presented in the 100% design.

Adequate velocity reduction in the unlined portion and at the outlet of the tunnel is achieved. Adequacy of provided air venting is demonstrated. The 100% design recommendations for new reinforced concrete liner, restated for reference as follows, remain valid:

- Minimum reinforced concrete side wall liner thickness of 1 ft.
- Minimum invert reinforced concrete slab thickness of 1.5 ft, with the requirement for rock dowels/anchors based on rock quality, excavated invert geometry and achievable joint tolerances.
- Minimum sidewall reinforced concrete liner height of 10 ft.

Recommended design and construction planning moving forward is summarized as follows:

- Design drawings should be updated to reflect known tunnel geometric variability, including liner dimensions and vent pipe profile along crown, to replace notes that are currently trying to capture how to address anticipated variability.

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- Based on the results presented herein, the Contractor can consider potential effects on the embankment removal schedule to achieve the dam demolition schedule.
- During construction engineering, optimization of the constructed liner dimensions as needed, with consideration of balancing rock excavation with reinforced concrete volumes and ease/duration of construction may be considered. This should be competed in cooperation with the Contractor and if notable geometric changes are desired the diversion tunnel hydraulics would need to be confirmed with further CFD modelling. Maintaining the design cross-sectional areas and tunnel bottom invert elevations are important to ensure the reliability of the CFD modelling. The final lining solution must be configured to be consistent with this study and have a minimum hydraulic geometric impact.
- During the pre-drawdown construction period and upon dewatering of the downstream diversion tunnel, an inspection of the portion of the downstream tunnel rock surfaces which were under water during the Yurok survey work is necessary. During this pre-drawdown period the 9ft diameter bulkhead will be installed and all leakage flow is cut-off and the downstream tunnel dewatered, will be the appropriate time for the downstream tunnel inspection.
- During the pre-drawdown period and after the 9 ft diameter bulkhead is installed, and during the diver inspection and cleaning of the diversion tunnel gate and its guides with the gate fully open, dimensions should be taken by the divers to confirm the gate opening width and height. Also, any possible gate opening narrowing or obstruction should be noted from what has been assumed in the hydraulic modeling.

Please not hesitate to contact the undersigned should any further clarification on the above presented results be required.

Yours truly,
Knight Piésold

Prepared: Carlo Capucao

Prepared: Katrina Wechselberger

Reviewed: Norm Bishop

Approval that this document adheres to the Knight Piésold Quality System: CN

MEMORANDUM

Date:	May 25, 2022	**File No.:**	VA103-00640/01-A.01
		Cont. No.:	VA22-00905
To:	Mr. Nick Drury (Kiewit)		
Copy To:	Craig Nistor, Norm Bishop, Dr. Hank Falvey		
From:	Katrina Wechselberger, Carlo Capucao		
Re:	**Iron Gate Diversion Tunnel – Pre-drawdown Modifications Optimization**		

1.0 INTRODUCTION

A value optimization to the design concept of the Iron Gate Diversion Tunnel pre-drawdown modifications from the Draft 100% Design is presented in this memorandum. Further consideration of CFD modelling representing the unlined, existing condition of the tunnel has precipitated the design optimization presented herein. This was made possible by the LiDAR and bathymetric survey completed following the Draft 100% Design (Yurok Tribe, 2021).

The proposed design optimization eliminates the need for extended new reinforced concrete protection of the unlined rock tunnel, downstream of the existing reinforced concrete liner by effectively incorporating the existing geometric roughness of the tunnel and including additional means of energy dissipation inside the existing reinforced concrete liner. The proposed optimization eliminates high velocity flows in the unlined and partially lined portions of tunnel following the formation of the hydraulic jump further upstream than previously observed. The optimized hydraulic performance is achieved through the following:

1. Installation of two baffles at the end of the existing concrete-lined tunnel downstream of the 9 ft orifice – The baffles initiate energy dissipation by causing flow to be interrupted downstream from the gate and orifice prior to exiting the existing liner.
2. Elimination of the new reinforced concrete invert and side wall reinforced concrete liner downstream of the existing concrete liner and grout curtain - This maintains the existing geometric roughness offered by the unlined tunnel, further contributing to the energy dissipation that pushes the hydraulic jump upstream. The Historical drawings and the 2021 bathymetric survey (Yurok Tribe, 2021) indicate the presence of an existing invert liner which extends approximately 120 ft downstream of the gate structure. The existing invert liner will be inspected and retained if found to be in sound condition or excavated to sound bedrock.

The combined effect of baffles and elimination of the new reinforced concrete invert and side wall liner, causes the hydraulic jump to move further upstream than previously observed during the previous Draft 100% Design. The new hydraulic jump location is shown to stabilize within the existing, reinforced concrete-lined tunnel, maintaining high energy and turbulent flows inside the portion of the tunnel which is already heavily reinforced according to historical drawings. Following the formation of the newhydraulic jump, the partially lined or unlined portion of the tunnel will only be exposed to lower velocity flows for the duration of reservoir drawdown.

2.0 HYDRAULIC INPUT PARAMETERS

The focus of this memorandum is the hydraulic performance during the most extreme conditions associated with reservoir drawdown, and the subsequent design of appropriate tunnel modifications to safely convey the water during those conditions. The value optimization presented herein does not impact the hydraulic capacity, as outflows are shown to be consistent with the previous Draft 100% Design.

Transient CFD models were developed and analyzed using the software ANSYS Fluent Version 2021 R2 to validate the hydraulic performance of the tunnel and support the optimizations to the pre-drawdown modifications of the diversion tunnel. The CFD analysis features the following input parameters to demonstrate the hydraulic behavior inside the tunnel during the highest energy flow scenario:

- Flow through the tunnel corresponds to the maximum reservoir level during drawdown at EL. 2331.3 ft.
- Gate is opened fully, 57 in opening.
- Two tailwater conditions were considered:
  - No tailwater present when the gate is opened, in the event environmental flow by-pass through the powerhouse is unavailable.
  - Tailwater present in the tunnel downstream of the gate at EL. 2177.4 ft corresponding to the minimum environmental downstream Klamath river flow of 1000 cfs (see Drawing C4055).

3.0 TUNNEL ARRANGEMENT & GEOMETRY CONSTRUCTION

Modelling presented herein incorporate the following existing geometric diversion tunnel features:

1. For 25 ft upstream of the gate and 90 ft downstream of the gate, the tunnel is historically lined with reinforced concrete, with a minimum thickness of 2 ft and interior dimensions consistent with a modified horseshoe geometry. 3D geometry is modeled based on historical design drawings.
2. 57” high x 14.2 ft wide semi-circular gate opening.
3. Concrete bulkhead with Ø9 ft orifice situated 11.3 ft from the gate.
4. Existing invert liner downstream of the fully concrete-lined tunnel, 120.6 ft long, incorporated as the surveyed geometry (Yurok Tribe, 2021). This is represented within the surveyed geometry and has not been manually added to tunnel geometry.
5. As surveyed bedrock geometry inside the tunnel (Yurok Tribe 2021), with the rock pile downstream of the existing invert liner removed, resulting in a -4% average final grade.
6. Downstream vent pipe, Ø2 ft inside diameter.
7. Additional Ø2 ft drilled opening through the Ø9 ft orifice bulkhead, located in the tunnel centerline, placed at the highest elevation possible with the opening center at EL. 2191.55 ft.
8. The 2 ft thick, reinforced concrete liner with a modified horseshoe geometry exists for the final 25 ft of the diversion tunnel adding a hydraulic construction to the outlet.

Figure 2.1 below shows the approximate extent of the historical 15’-6” modified horseshoe reinforced liner. Note the flow direction in this figure from right to left, as it was oriented in the historical drawing. This is the opposite from the traditional convention showing flow from left to right as is used in the CFD results figures.

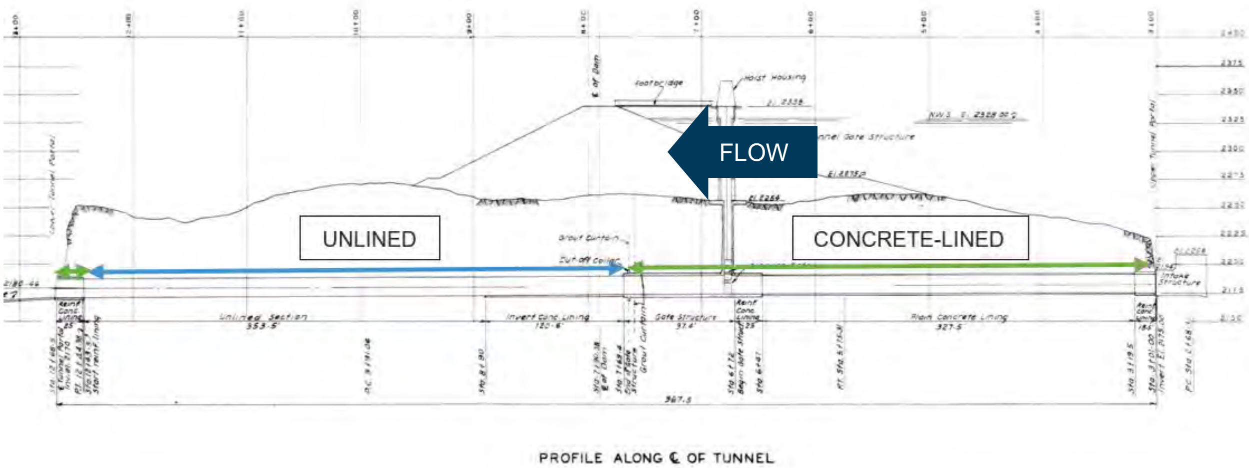


Figure 3.1 Historical Tunnel Profile

3.1 SURVEYED GEOMETRY CONSTRUCTION

The tunnel is partially lined with unknown reinforcement details or unlined for approximately 475 ft downstream of the reinforced horseshoe concrete liner downstream of the gate. Detailed survey of the tunnel downstream of the blind flange and 9 ft orifice was conducted and processed by the Yurok Tribe at the end of 2020 and made available to Knight Piésold in early 2021. Characteristic of the survey methods utilized, the total station survey bathymetric data and the LiDAR data yielded very different point density. Manipulation and filtering of the point cloud data generated by the LiDAR was required to construct a tunnel surface geometry that could be meaningfully used and meshed by ANSYS, the CFD analysis software.

The goal was to effectively represent geometric roughness offered by the unlined tunnel, without exaggerating its effect and allowing the CFD model to run smoothly, producing a meaningful result. This was achieved by creating intersection lines around the circumference of a circle divided into 24 segments and connecting the neighboring cross sections at 5 ft spacing. This is demonstrated in Figure 2.1 below.

The portion of the tunnel that is partially lined along its invert does not have any liner manually added to the CFD model geometry. This liner is reflected in the survey data and meshed in with the bedrock geometry.

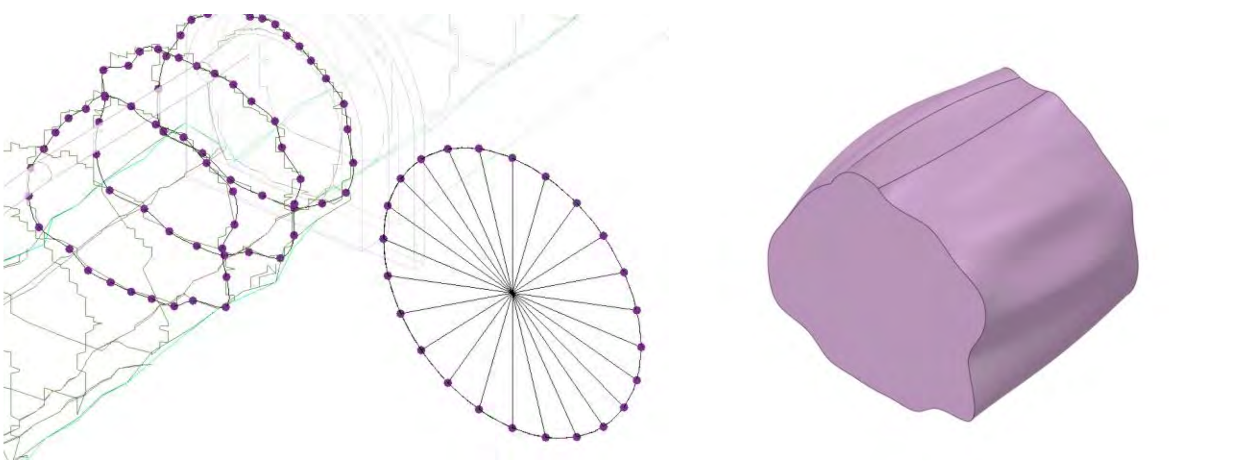


Figure 3.2 Unlined Tunnel – Construction of Model Geometry – Partial Segment of Tunnel

3.2 ADDITION OF BAFFLES

Additional means of CFD model energy dissipation are introduced in the form of two steel exterior lined, post tensioned, concrete baffles installed just upstream of the grout curtain near the outlet of the existing reinforced modified horseshoe liner. The upstream face of each baffle interrupting the high velocity flow is 2 feet by 2 feet. Each baffle is 3 feet long, to provide stability.

The CFD modelling has been completed for the following arrangements:

1. Existing condition – modifications include updated ventilation and removal of rock pile downstream of the partial invert liner.
2. Existing condition with 2 baffles.

4.0 RESULTS AND PROPOSED OPTIMIZAION

4.1 EXISTING CONDITION HYDRAULIC PERFORMANCE

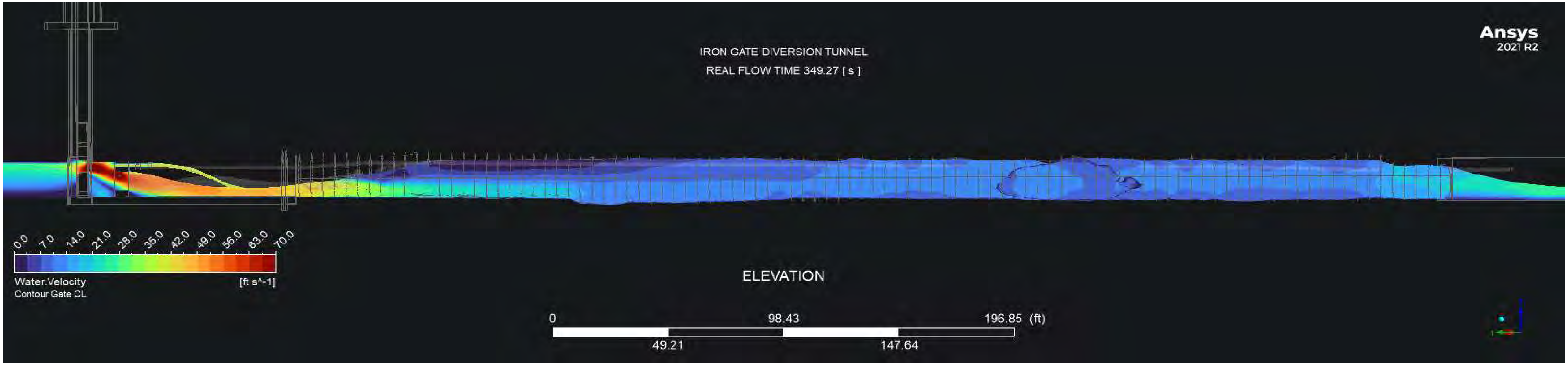
As part of the work to support the Draft 100% Design, CFD analysis, as presented in KP Memoranda VA20-01002 (2020) and VA21-00482 (2021), show that the flow regime in the tunnel downstream of the Ø9 ft orifice features a segment of supercritical, high-energy flow that transitions into subcritical, low velocity flow that fills the tunnel. The transition in flow is marked by a hydraulic jump which acts as the primary energy dissipator in the flow. In this flow regime, the gate opening and Ø9 ft orifice are the points of control of the total discharge through the tunnel.

This CFD analysis led to the following design decisions, outlined in their respective KP Memoranda:

- Air demand in the tunnel, especially upstream the hydraulic jump, is to be supplied by providing a Ø2 ft ID downstream vent pipe which extends from 3 ft downstream the Ø9 ft orifice bulkhead to ambient conditions at the outlet of the tunnel (KP Memorandum VA20-01002, 2020).
- Ø2 ft opening through the Ø9 ft orifice bulkhead shall be provided to alleviate negative pressures in the area between the gate and the Ø9 ft bulkhead at low flows corresponding to low reservoir levels (KP Memorandum VA20-01002, 2020).
- Rock pile located approximately 255 ft downstream of the gate to be removed to allow for flow conveyance (KP Memorandum VA21-00482, 2021).

Without any new concrete invert or sidewall liner, CFD analysis of the existing tunnel conditions and the previously discussed design features, show that the hydraulic jump that forms in the tunnel stabilizes just downstream of the existing concrete-lined tunnel. This is notably further upstream than the location of the hydraulic jump when the new concrete liner is constructed, that eliminated the natural geometric roughness of the tunnel.

Figure 3.1 presents the plan and elevation view of the tunnel when the hydraulic jump has stabilized just downstream of the existing concrete liner.



HYDRAULIC DATA:

AIR FLOW IN VENT:	**42.9 ft3/s**	
AIR FLOW MEAN VELOCITY IN VENT:	**11.4 ft/s**	
MIN AIR RPESSURE UPSTREAM OF JUMP:	**-117.1 psf**	**-0.06 atm**
MAX AIR PRESSURE IN AIR GAP IN TUNNEL	**889.4 psf**	**0.42 atm**

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	25MAY'22	ISSUED WITH MEMO	CPC	KTW

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Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
CFD RESULTS
FLOW VELOCITY AND HYDRAULIC DATA

Knight Piésold CONSULTING

P/A NO. VA103-00640/1 | REF. NO. VA22-00905

FIGURE 4.1 | REV 0

4.2 EXISTING CONDITION WITH BAFFLES

As observed in the CFD simulations of the existing conditions, the tunnel has adequate energy-dissipating capacity to induce a hydraulic jump, thus subjecting majority of the unlined portion of the tunnel to lower velocity flows. Given the proximity of the hydraulic jump to the existing heavily reinforced horseshoe lined section of the tunnel, it is proposed that the new concrete liner be eliminated by pulling the jump further upstream with the use of baffles, so that the rough, partially lined portion of the tunnel is no longer exposed to high energy flow for an extended period of time.

This is achieved by introducing two 2'x2'x3' (WxHxL) baffles into the high velocity flow providing additional means of energy dissipation earlier in the tunnel alignment. CFD simulation with the baffles show that the hydraulic jump moves upstream and stabilizes within the horseshoe concrete-lined portion of the tunnel.

Figure 4.2 shows the modelled geometry of the baffles and their location.

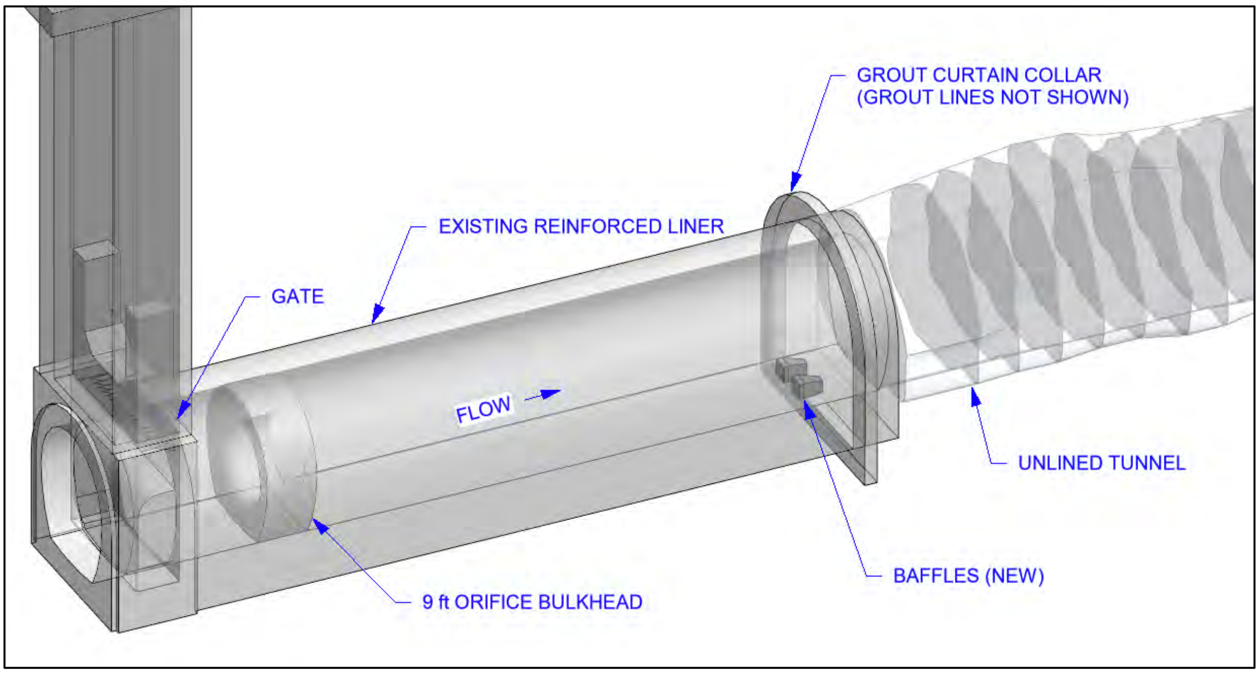
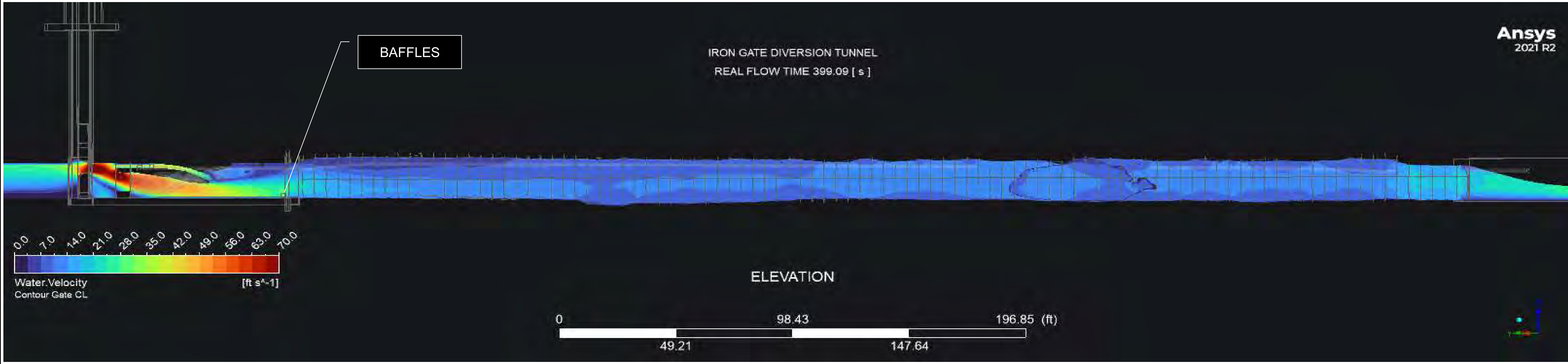


Figure 4.2 Modelled Baffle Geometry

Figure 4.3, below presents the plan and elevation view of the tunnel with the baffles added, showing the hydraulic jump travelling inside the existing liner. Water velocities in the unlined portion of the tunnel do not exceed 15 ft/sec and are less than 10 ft/sec in most places.



HYDRAULIC DATA:

AIR FLOW IN VENT:	**8 ft3/s**	
AIR FLOW MEAN VELOCITY IN VENT:	**3 ft/s**	
MIN AIR RPESSURE UPSTREAM OF JUMP:	**-248.8 psf**	**-0.12 atm**
MAX AIR PRESSURE IN AIR GAP IN TUNNEL	**823.1 psf**	**0.39 atm**

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	25MAY'22	ISSUED WITH MEMO	CPC	KTW

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

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FIGURE 4.3

REV 0

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4.3 CONCLUSION AND BAFFLE DESIGN DETAILS

The installation of the baffles offers reduced construction risk, in both cost and schedule, associated with the tunnel modifications and should be considered as a value alternative to the previously proposed construction of new reinforced concrete invert and sidewall liner shown on the C4120 series.

The baffles are designed to withstand the maximum hydrodynamic force associated with the fast-moving flow coming from the Ø9 ft orifice. The baffle design incorporates the following features:

1. Cast-in-place reinforced concrete block with a 2'x2' frontal area, 3' long that tapers along its length. This tapered shape is based on the studies by USACE and USBR (2009); it serves to mitigate against cavitation-induced damage. The baffle block shall be installed in a 2" deep notch cut into the existing concrete-lined tunnel invert to offer improved bearing and stability.
2. Steel plate facing at the sides of the baffle for protection against concrete spalling.
3. Post-tensioned threaded dowels anchored into rock to resist uplift, stagnation pressures and improve stability.

The final design of the baffled option is presented in C4190 drawing series. The details of the proposed baffles are shown on the attached C4193. Other tunnel modifications remain unchanged, including the new vent pipe hung from the crown of the tunnel and the drilled 2 ft hole through the concrete bulkhead to establish ventilation upstream of the 9 ft orifice. Spacing of the vent pipe supports has been adjusted to be concentrated nearer the existing liner to protect the pipe as the hydraulic jump forms further upstream. This is shown on drawing C4194 and C4195.

Yours truly,
Knight Piésold

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Reviewed: Norman Bishop

Approval that this document adheres to the Knight Piésold Quality System: CN

Attachments:

C4193 Baffle Details

References

Cox, C. and Frizell, K., 2009. Studies Evaluate Cavitation Potential of Baffle Blocks within a Stilling Basin on a Novel Stepped Spillway Design. U.S Army Corps of Engineers and U.S. Bureau of Reclamation.

Knight Piésold Ltd. (KP), 2020. Memorandum to: Dr. H.T. Falvey. Re: Iron Gate Diversion Tunnel Air Vent Requirements. May 5. Vancouver, BC. Ref No. VA20-01002 (VA103-640/1).

Knight Piésold Ltd. (KP), 2021. Memorandum to: Mr. Nick Drury (Kiewit). Re: Iron Gate Tunnel Modelling – Updated with 2021 Survey Results. May 6. Vancouver, BC. Ref No. VA21-00482 (VA103-640/1).

/eml

ATTACHMENT A

100% FINAL Design Report_Appendix E2_May 28

Page 26 of 26

REDACTED: Page 26 of 100% FINAL Design Report_Appendix E2_May 28 consists in its entirety of information about the location, character, or ownership of historic resources that, if disclosed, may cause a significant invasion of privacy; cause a risk of harm to the historic resource; or impede the use of a traditional religious site by practitioners. These pages are labeled as “Privileged” in accordance with 18 C.F.R. § 388.112, 18 C.F.R. § 388.107 and 36 C.F.R. § 800.11(c).