
KLAMATH RIVER RENEWAL PROJECT – OPTIMIZATION OF THE IRON GATE DAM HISTORIC DIVERSION TUNNEL USING CFD ANALYSIS TO SUPPORT RESERVOIR DRAWDOWN

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ABSTRACT

The Iron Gate Dam was the most downstream of four hydroelectric facilities on the Klamath River that were decommissioned in 2023 and 2024. The 58,000 acre-ft (71.5 km²-m) reservoir was retained by a 173-foot (52.7-metre) high embankment dam with an ungated overflow spillway. A near complete drawdown of the reservoir through the historic diversion channel was required prior to embankment removal.

Design of pre-drawdown modifications in the diversion tunnel had the following primary objectives:

- Achieve adequate discharge capacity to facilitate the reservoir drawdown during the limited winter and spring months of the drawdown year, and
- Optimize the hydraulic performance and energy dissipating properties of the tunnel to safely operate under full reservoir head and maximum flows

Accurate representation of the irregular unlined rock geometry in lieu of typical cross sections with a generic roughness was fundamental to understanding the inherent in situ energy dissipating qualities of the tunnel and facilitated the decision to eliminate expensive design features such as a stilling basin, new full diameter gate and extensive new liner system. Additional design concepts aided by the CFD analysis included air ventilation, gate downpull forces and the discharge rating curve for a complex and variable tunnel geometry.

RÉSUMÉ

Le barrage Iron Gate, le plus en aval des quatre barrages et installations hydroélectriques sur la rivière Klamath, a été déclassé en 2024. Il retenait un réservoir de 58 000 acres-pieds grâce à un barrage en remblais de 173 pieds de haut avec un déversoir à débordement sans vanne. Avant de retirer le remblai, un rabattement presque complet du réservoir via le tunnel de déviation historique était indispensable.

Les modifications à effectuer avant la vidange avaient deux objectifs principaux :

- Atteindre une capacité de débit suffisante pour sécuriser le rabattement du réservoir durant les mois d'hiver et de printemps limités, et
- Optimiser la performance hydraulique ainsi que les propriétés de dissipation d'énergie du contrôle de déviation et du tunnel pour opérer en toute sécurité sous une charge maximale et des débits qu'il n'avait pas été conçu pour supporter.

Une représentation précise de la géométrie rocheuse irrégulière, en remplacement des sections génériques, était cruciale pour identifier les qualités intrinsèques de dissipation d'énergie du tunnel. Cette compréhension a conduit à écarter certains éléments de conception coûteux, tels qu'un bassin d'amortissement, une nouvelle vanne de diamètre complet et un revêtement étendu. L'analyse a également permis d'évaluer des concepts supplémentaires, y compris l'amélioration de la ventilation, l'évaluation des forces de rappel de la vanne, et le développement de courbes de niveau-débit adaptées à la géométrie variable et complexe du tunnel.

1 INTRODUCTION

The Iron Gate Dam (IGD) and hydroelectric facility, commissioned in 1962, was the most downstream of four hydroelectric facilities in the Klamath River Renewal Project (Nistor et al. 2025) that were decommissioned in 2023 and 2024. The site is approximately 10 miles (16 km) east of the Interstate 5 Highway near the town of Hornbrook, California. The major components of the Iron Gate facility included:

- A reservoir of 58,000 acre-ft (71.5 km²-m) capacity at a full reservoir supply water surface elevation of 2,331 ft (710.5 m)
- A 173-ft (52.7-metre) high earth embankment dam.
- A 725-ft (221-m) long ungated, free overflow spillway
- An historic diversion tunnel, available for use as an emergency low level outlet and environmental flow bypass through the upper concrete sluice gate and further controlled by a flanged 9-ft (2.7-m) orifice.
- A separate power intake structure and surface mounted steel penstock with intake invert elevation of approximately 2296 ft (699.8 m) and a by-pass valve that bifurcates 15 ft (4.6 m) upstream of the scroll case.
- A single unit powerhouse, located downstream of the dam
- Switchyard, substation, and transmission lines

The crest of the ungated spillway at IGD was near the crest of the dam and the power intake structure was less than 50 feet (15.24 m) below the spillway crest, approximately one third of the height between the reservoir bottom and the spillway. This left the historic diversion tunnel as the primary means of evacuating the reservoir. Once the dam crest was lowered below the spillway and power intake inlet elevations during deconstruction, the diversion tunnel would serve as the sole conduit for passing river flows. Inability of the tunnel to serve this purpose partway through dam deconstruction would result in reservoir level rise and overtopping of the dam embankment. Dam breach studies indicated that peak outflows much greater in magnitude than natural flood events would result from an unplanned dam overtopping event at any stage during dam deconstruction. Modification and optimization of the historic diversion tunnel to perform this critical role in decommissioning was therefore a primary focus of the design effort.

A common challenge when working at existing facilities and, particularly, when considering decommissioning activities is how to evaluate and be confident in using facility components for purposes not originally considered or intended. The Iron Gate tunnel serves as a prime example of this challenge. Adding to this uncertainty is a desire to minimize the complexity of the construction effort as performance requirements are only applicable for the short period of time during the decommissioning. This paper retraces the design development of the tunnel modifications to make it fit for purpose and highlights where Computational Fluid Dynamics (CFD) Analysis provided valuable support in that process.

1.1 *Original Tunnel Geometry and Operations*

The IGD historic diversion tunnel was approximately 970 ft (295.7 m) long, with a two-part concrete lift gate located 380 ft (115.8 m) downstream of the inlet. The dimensions and lining type vary along the alignment. Based on historical drawings, and as shown in Figure 1.1 below, it was characterized by the following description:

- The tunnel was fully submerged upstream of the gate, and was understood to be lined, partially with plain concrete and partially with reinforced concrete.
- For 25 ft (7.6 m) upstream of the existing gate and 90 ft (27.4 m) downstream of the gate, the tunnel is lined with reinforced concrete, with a minimum thickness of 2 ft (0.7 m) and interior dimensions consistent with a 15.5-ft (4.7 m) wide modified horseshoe geometry.

- A grout curtain and concrete cut off collar are shown in the drawings to be approximately 80 ft (24.4 m) downstream of the gate.
- The tunnel is unlined for 475 ft (144.8 m) downstream of the cut-off collar with dimensions varying between 19 ft (5.8 m) and 22.5 ft (6.9 m) in height and width.
- 120 ft (36.6 m) of unreinforced concrete-lined invert extended downstream of the cut-off collar.
- The 2 ft (0.7 m) thick, reinforced concrete liner with a modified horseshoe geometry is reinstated for the final 25 ft (7.6 m) of the diversion tunnel adding a hydraulic constriction to the outlet.

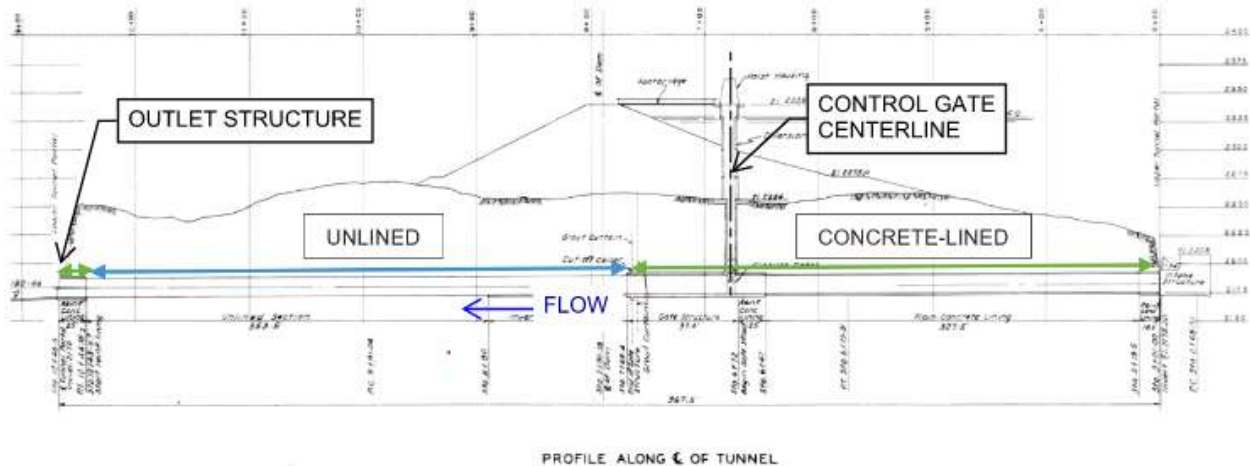


Figure 1 Iron Gate Diversion Tunnel – Profile View along Tunnel Centerline (Historical Drawing)

The control gate, annotated in the figure above just upstream of the dam centreline, consisted of a two-part concrete lift gate, shown in the Figure 1.2 below. The bottom portion of the gate was installed after the tunnel had served its primary purpose of river diversion for construction and was never intended to be removed. The top portion of the gate, comprising approximately 30% of the cross-section, was still operational, but was only operated and tested for partial gate openings.

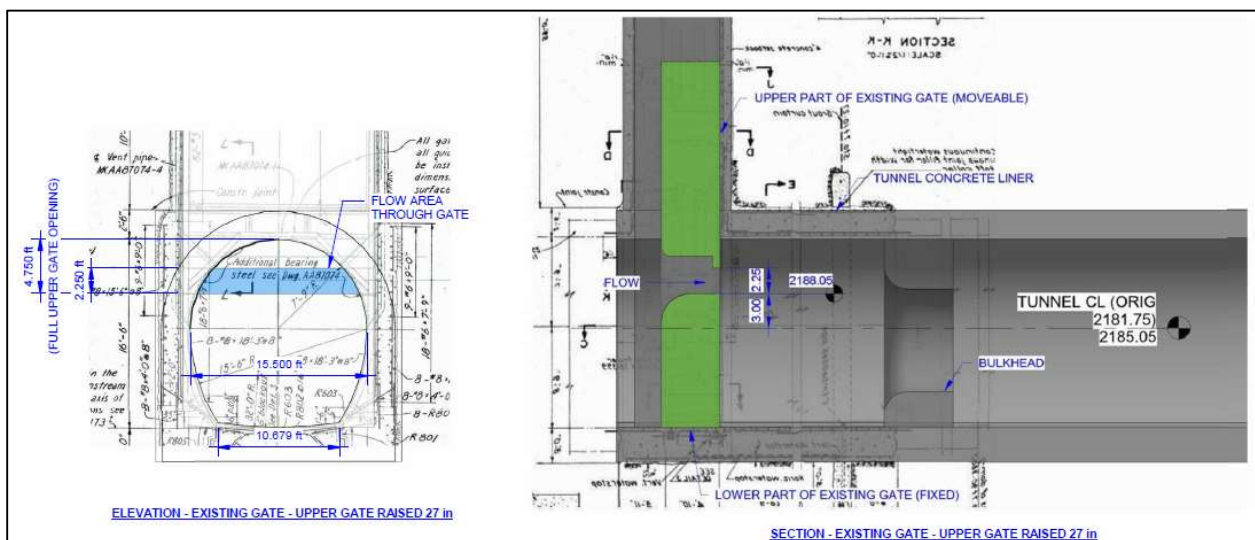


Figure 2 Iron Gate Diversion Tunnel – Control Gate

The Project design and decision-making activities revolved around modifying the existing historical diversion tunnel to operate under circumstances which they have not been originally designed for. Specifically, the historic diversion tunnel was commissioned, primarily, to divert the Klamath River flow

during the construction of the Iron Gate dam under low head conditions. During the life of the facility, it also served as an emergency low-level outlet for the environmental flow bypass when the spillway was inactive (because water surface levels were below the crest), and the generating equipment were offline. It was rarely used other than intermittent testing and not intended to pass much more than approximately 1000 ft³/s (28 m³/s) for a short period of time. It was not designed for decommissioning activities that would require it to serve as a low-level outlet that provided uninterrupted discharge with the gate open to capacity for months at a time.

2 DESIGN DEVELOPMENT

The diversion tunnel was determined to be the only practical means to evacuate the reservoir without risking a breach and uncontrolled erosion failure of the embankment dam but using it for drawdown would require assessment of the conditions inside the tunnel as well as the design of modifications to allow for the safe discharge of water for an extended period time to maintain target drawdown rates.

The following sections provide insight into the process of developing the final arrangement that supported the IGD reservoir drawdown in preparation of the final breach that could only occur at near empty water levels or risk downstream flooding (KP, 2022). The final breach activities are discussed further in Adria et al. (2025).

2.1 30% Design

The first concept evaluated during preliminary design for the historic diversion tunnel included installing a new control gate at the outlet of the tunnel and removing the existing concrete lift gate at the centreline of the dam. While this concept had the benefit of making the full cross section of the tunnel available for discharge, maximizing the drawdown rate, it posed a significant risk of seepage and fracturing of the surrounding rock mass as the unlined portion of the tunnel did not provide adequate rock cover to withstand internal pressures from the reservoir without a liner system. By moving the gate from upstream of the dam centreline to the tunnel outlet at the toe of the dam, it required the complete tunnel length be subject to the full head of the reservoir.

A modification to the initial concept was made by proposing a steel pipe tunnel liner that ran the length of the unlined portion of the tunnel, approximately 500 ft (152.4 m). This modification mitigated the risk of pressurizing the tunnel, but it still would have resulted in very high energy flows leaving the outlet of the tunnel. Initial CFD modelling completed at the time estimated velocities that exceeded 70 ft/s (21.3 m/s). Figure 2.1 below shows the preliminary velocity results with the gate at the tunnel outlet. This condition risked undermining the toe of the embankment dam or eroding the access roads opposite the tunnel outlet without a costly energy dissipation structure located immediately downstream of the tunnel outlet.

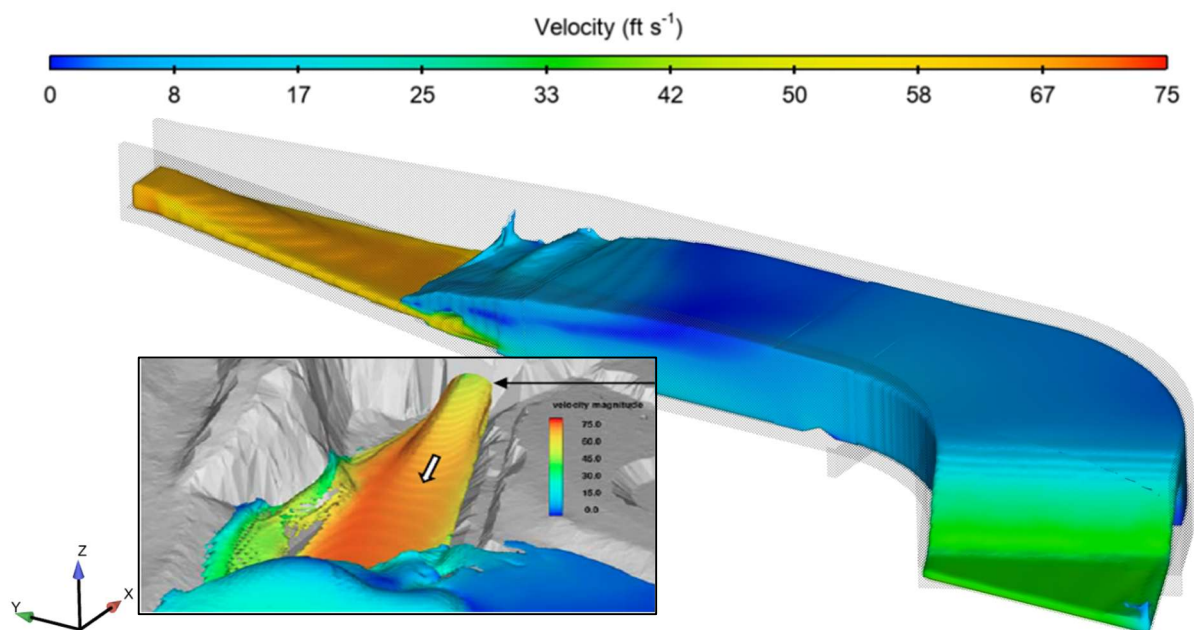


Figure 3 Control Gate at Tunnel Outlet – 3D Isometric View – Energy Dissipation Structure Concept (CFD Model in Flow 3D)

The scope of work associated with constructing a steel liner, removing the internal existing gate structure, and constructing a new gate and energy dissipation structure or stilling basin at the toe of the dam was cost prohibitive and also unfeasible from a schedule perspective due to the short period of time that pre-drawdown modifications needed to be completed in between the fall fish window and subsequent winter drawdown period.

2.2 60% Design

The next phase of design focused on maintaining main discharge control point at the existing gate location inside the tunnel near the centreline of the dam. This allowed the downstream, unlined portion of the tunnel to be kept at low pressure and also provided 475 ft (144.8 m) of tunnel downstream of the discharge control point to initiate energy dissipation, with the goal of seeing the flow velocity drop to manageable levels by the time it reached the tunnel outlet, thereby eliminating the need for additional energy dissipation at the toe of the dam.

At the time, it was believed that the full tunnel cross-section was required to meet discharge requirements to achieve the necessary rate of drawdown. For this reason, this option required thorough consideration of the feasibility of removing the submerged existing concrete gate sections and installing into the existing gate shaft a new gate that could be opened to the full cross-section of the tunnel. It was considered feasible but also risky and required additional information to be collected about the condition of the existing gate and shaft.

2.2.1 Evolving Drawdown Criteria and Dam Removal Sequencing Supporting the Use of the Existing Gate Opening

One of the performance criteria controlling the discharge requirements of the tunnel was a requirement that drawdown of the reservoirs had to be reasonably complete in the first three months of the drawdown year. This was understood to be related to attempting to flush as much sediment from the reservoirs as possible

during this first flushing event. This criterion evolved to allow an extended drawdown period that still saw a substantial flushing event in the first three months but provided the schedule flexibility to allow some partial refilling during the spring freshet season with final drawdown and breach being scheduled for late summer when the flows were lowest.

This change allowed the design team to consider whether the existing upper half of the concrete lift gate and the 9-ft (2.7-m) orifice could provide the necessary discharge capacities to achieve drawdown. A preliminary arrangement and discharge rating curve was developed using CFD Analysis to support the detailed drawdown modelling required to adopt the extended drawdown period. The drawdown modelling that supported this decision is discussed further in Bennett et al. (2025).

2.3 90% Design

CFD modeling of the existing tunnel arrangement, based on historical drawings, showed that the 9-ft (2.7-m) orifice, provided an initial point of flow constriction and subsequent expansion initiating the energy dissipation beyond standard friction losses. The centrelines of the existing gate and the existing orifice were offset within the tunnel cross-section leading to highly turbulent and recirculating flow. This region of the tunnel filled up with recirculating flow before exiting the orifice and proceeding into the rest of the tunnel. For all hydraulic configurations, upon initial gate opening, the flow remained supercritical, exiting the existing orifice and traveling along the tunnel invert until the flow reached the outlet. The outlet offered another point of flow constriction, initiating a hydraulic jump. Eventually, the hydraulic jump traveled upstream and stabilized at a point inside the tunnel. The flow downstream of the jump then transitioned into subcritical flow that filled the tunnel.

Equally as important to establishing stable flow hydraulics in the downstream portion of the tunnel, the modelling continued to demonstrate that the discharge capacity of this arrangement met the discharge targets for the drawdown operation. At this point, it was decided that the design effort and construction methodology would remain focussed on utilizing as much of the existing tunnel facility as possible for the reservoir drawdown and providing the necessary modifications in the tunnel to discharge the flows safely.

The optimizations had eliminated the need for a steel liner system and substantially reduced the need for an energy dissipation structure or stilling basin. Importantly, with the extended drawdown period, no new gate would be required at the tunnel outlet or at the dam centreline. The next phase of design could focus on the following main design and assessment activities:

1. Assessment of the hydraulic behavior inside the tunnel including flow velocities and air flow requirements during different reservoir levels and opening heights.
2. Assessment of the existing gate and lifting equipment to ensure the capacity of the hoist was adequate to reliably lift the upper portion of the gate clear of the tunnel crown, approximately 3 ft (1 m) higher than it had been operated since original commissioning.
3. Hydraulic behavior near the outlet of the tunnel to ensure the embankment toe and powerhouse access would not be compromised by the outflows.

2.3.1 Concrete Lined Invert for Assumed Tunnel Geometry.

Detailed three-dimensional survey of the downstream portion of the tunnel geometry was not available. Due to leakage at the existing control gate, the tunnel invert was fully submerged and there was no lighting inside the tunnel. The historic drawings showed a typical cross section of the tunnel, but no formal as-built data existed for the tunnel. Some spot checks of the tunnel dimensions had been performed to establish a baseline minimum cross-section.

The hydraulic analysis at this point in the design assumed a constant minimum cross-section based on this data. A typical roughness associated with fractured rock was assigned to the tunnel surface, but the macro geometry of the tunnel was essentially smooth. This was known to not be an accurate representation of the tunnel geometry but was considered conservative from the perspective of estimating critically high velocities.

As shown in Figure 2.2 below, CFD modeling of this arrangement indicated that the hydraulic jump in the tunnel stabilized downstream of the existing concrete-lined portion of the tunnel.

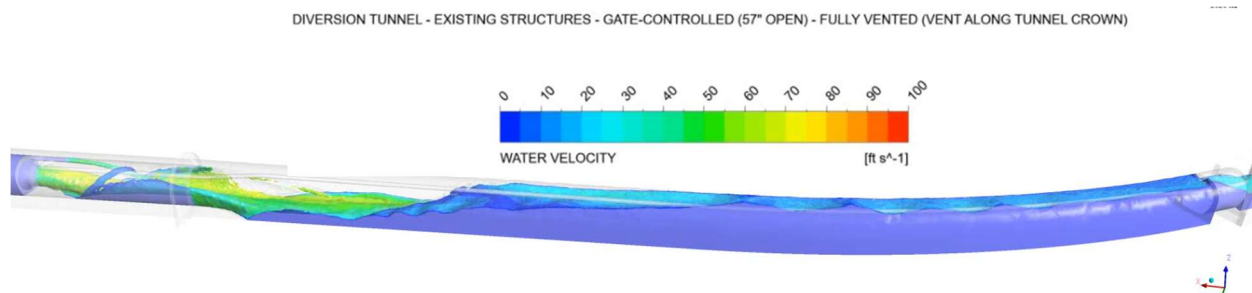


Figure 4 CFD Simulation – 90% Design – Tunnel Geometry based on Historical Drawings – 3D Isometric View

The fractured surrounding rock mass was considered vulnerable to scour and erosion where subject to high velocity flows and the turbulence of the hydraulic jump. Guidance from USACE EM 1110-2-1601 Hydraulic Design of Flood Control Channels (1994) suggested that the maximum permissible channel velocity over poor rock material is 10 ft/s (3 m/s). For this reason, it was decided that any portion of the tunnel subject to high velocity flows upstream of the jump needed to have a reinforced concrete liner installed to protect the tunnel from scour and cave-ins. The 90% design concept carried concrete lining at the invert and up the sides to approximately the spring line of the existing unlined portion of the tunnel. The height of the side wall liner was based on the depth of the high-velocity flow upstream of the jump. Figure 2.3 below shows the tunnel concrete lining concept.

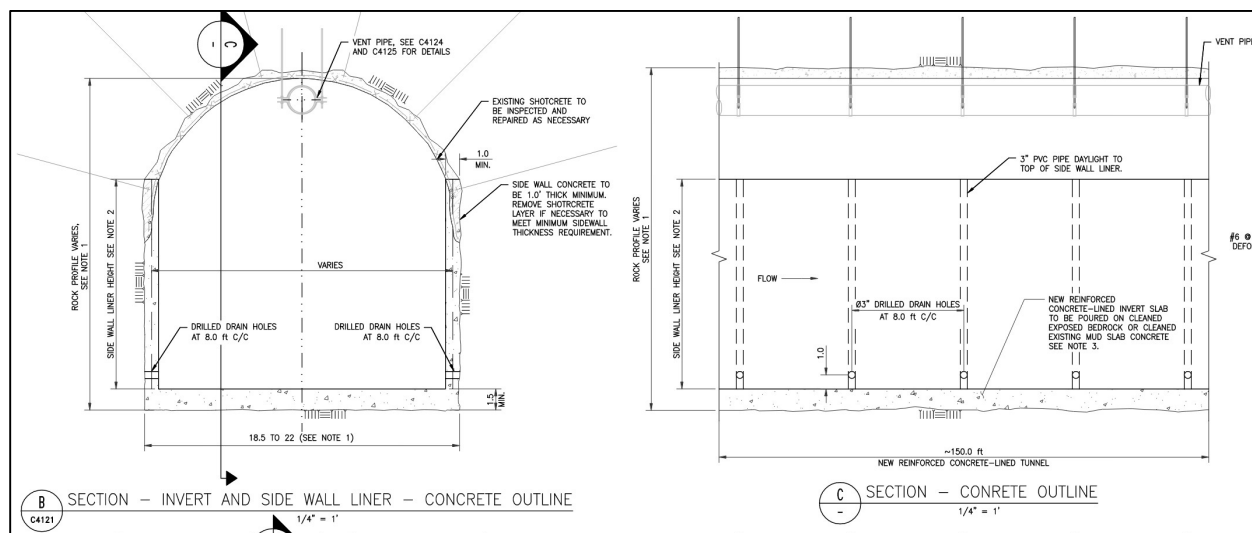


Figure 5 Tunnel Concrete-lining Concept – Section Views

Unfortunately, once the model was updated to include the concrete lined geometry and roughness for the new liner, friction losses were reduced, pulling the hydraulic jump back downstream and extending the length of required liner. The liner was required to protect the tunnel walls but was counter to the goal of reducing velocities inside the tunnel. Regardless, it facilitated the elimination of a stilling basin at the tunnel outlet and was more schedule and cost effective than a steel liner.

2.4 Air Flow Requirements

The existing gate assembly was vented through the gate shaft via embedded conduits in the concrete gate guides. These conduits provided effective ventilation for the existing arrangement with the smaller discharge requirements. Once the gate was opened fully and for an extended period, it was shown that the conduit openings were flooded as the region between the gate and orifice filled up with water. For the drawdown operations, two zones were identified inside the tunnel where new ventilation was recommended to establish proper air flow and maintain stable hydraulic discharge behavior under varying gate openings and upstream water surface conditions. These are described as follows and shown in Figure 2.4 below.

- Zone 1: Vent upstream of the orifice, downstream of the existing gate – air ventilation that targets the region between the existing gate and existing orifice to vent the hydraulic roller that occurs between the gate and orifice while this region fills with water and to satisfy the air demand at initial gate opening. Zone 1 was vented by a 2-ft (0.6-m) diameter opening drilled at the upper right (looking downstream) quadrant of the 9-ft (2.7-m) orifice concrete collar.
- Zone 2: Vent downstream of the 9-ft (2.7-m) diameter orifice – to vent the closed conduit hydraulic jump and promote conditions for open channel flow in the tunnel. Zone 2 was vented by a 2-ft (0.6-m) 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 CFD modeling utilized a two-fluid Mixture model in ANSYS, allowing the hydraulic behavior of the tunnel to provide air velocity magnitudes and vectors to demonstrate when the ventilation was active and in what conditions they were flowing with water.

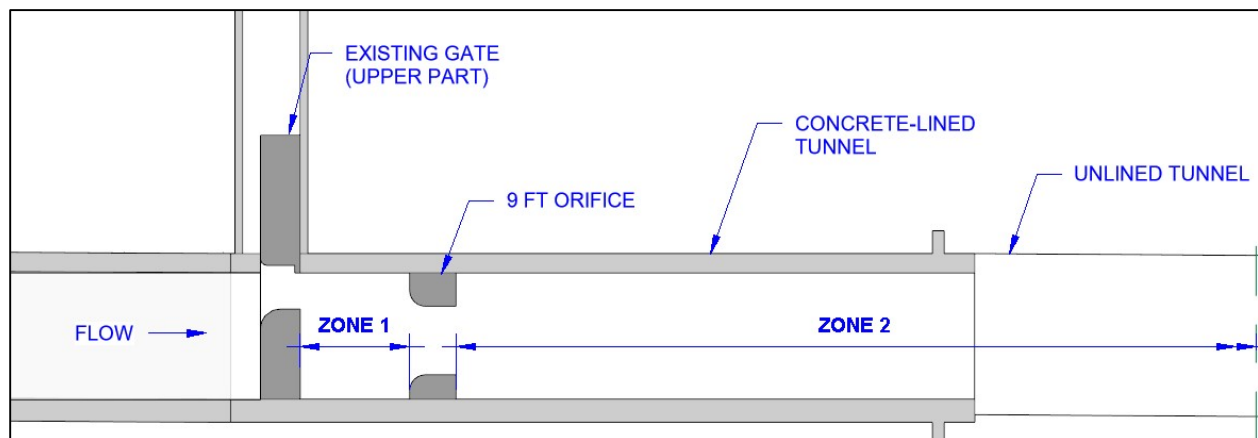


Figure 6 Air Ventilation Zones – Section View

2.5 Gate Assessment

A detailed condition and risk assessment on the existing lifting equipment was performed to determine its adequacy to operate the gate under the pressure of a full reservoir.

2.5.1 Lifting/Pulldown Force

The existing gate hoist had a rated capacity of 350,000 lbs (158,760 kg) with the possibility of increasing the capacity to 400,000 lbs (181,440 kg) with reduced safety factors and various possible mechanical upgrades. A CFD analysis was performed to evaluate whether increase in down pull forces upon gate opening would exceed the existing hoist's lift capacity, as shown in Figure 2.5. To simulate the operation of the gate and the resulting downpull forces, the CFD analysis involved the use of dynamic mesh adaptation. This feature allowed the mesh for the upper gate to be raised at the specified lifting rate for the gate as verified for partial opening on site, while the mesh through the gate opening and immediate surrounding dynamically conformed to the moving gate mesh.

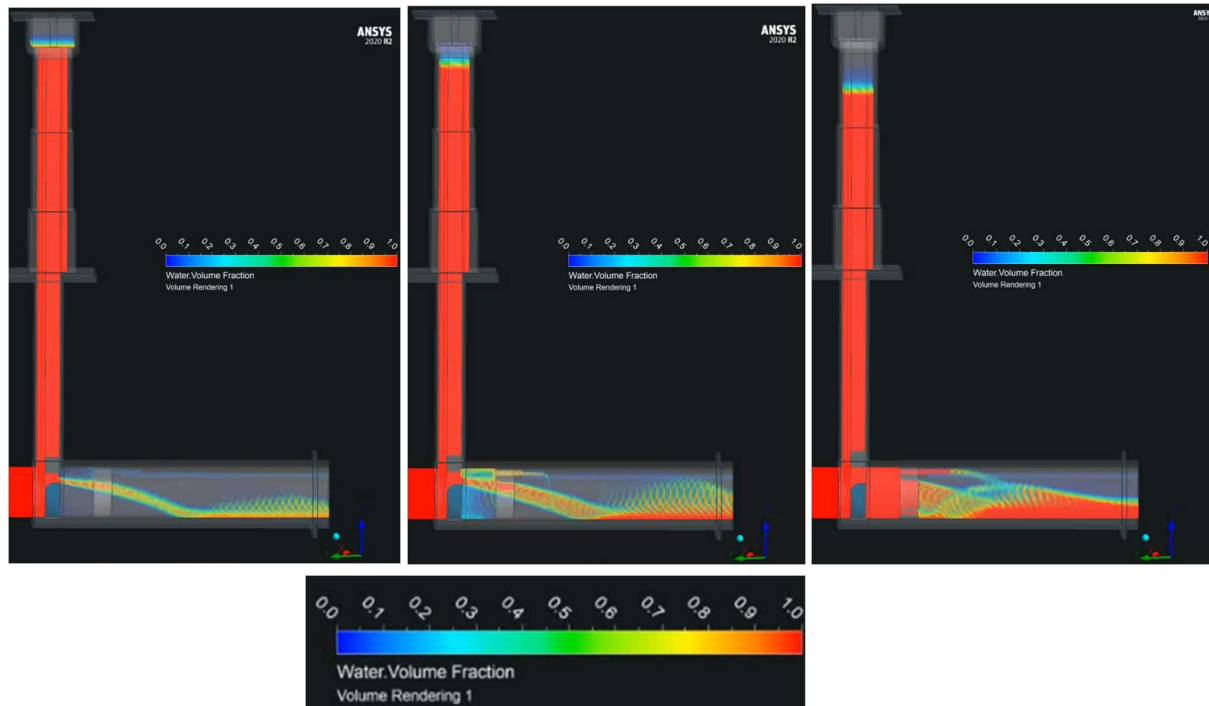


Figure 7 Gate Opening – CFD Simulation – Section Views – Time elapsed after initial gate raising = 60 s (Left); 120 s (Center); 180 s (Right)

It was found that the net downpull forces in the gate at the maximum reservoir level did not exceed the hoist's rated lift capacity. The maximum lifting force during gate opening occurred between about 10% and 50% of gate opening. This was result was consistent with the submerged condition downstream of the gate that developed in the latter half of the lifting cycle due to the bulkhead for the 9-ft (2.7-m) orifice. This formed part the hoist operation recommendations during drawdown as it supported keeping the gate open fully to maintain the downstream submerged condition. The goal was to avoid cycling the gate and lifting equipment through the highest loaded part of the lift and a variable air demand.

3 FINAL DESIGN

Throughout the design process of the pre-drawdown modifications in the diversion tunnel, the overarching design objectives were:

- Achieve adequate discharge capacity to facilitate the reservoir drawdown during the limited winter and spring months of the drawdown year, and
- Optimize the hydraulic performance and energy dissipating properties of the tunnel to safely operate under full reservoir head and maximum flows while minimizing the pre-drawdown construction scope.

The primary design input that was not available leading into the 90% design phase was a detailed tunnel survey downstream of the gate. Preliminary visual inspections of the tunnel downstream of the control gate identified variability from the interior dimensions in the historical construction drawings and confirmed the need for a detailed tunnel survey. The 90% design was intended to be feasible regardless of the results, affecting primarily some concrete volumes as the variable geometry was fitted for a concrete liner.

3.1 Introduction of Geometric Roughness for Friction Loss Optimization

Detailed survey of the tunnel downstream of the blind flange and 9-ft (2.7-m) orifice was conducted prior to final construction design using a LiDAR scan of the tunnel above the water surface, as well as a bathymetric total station survey for the invert inside and at the exit channel of the tunnel. Good agreement was observed between the tunnel survey data and historical drawings for the existing reinforced concrete liner that extends from the gate structure to approximately 100 ft (30.5 m) downstream of the control gate structure.

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 Fluent, 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.

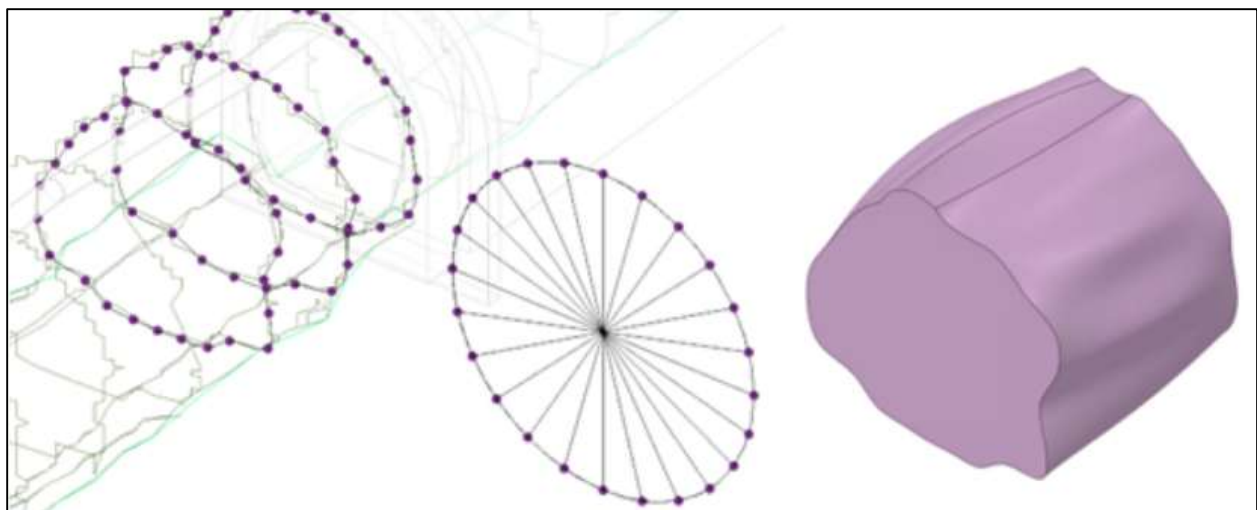


Figure 8 Construction of Model Geometry – Partial Segment of Tunnel

This approach effectively captured the macro roughness of the unlined bedrock tunnel geometry to a minimum mesh size of 0.25 ft (0.08 m). No other micro roughness was added to the surface in the model parameters. Subsequent CFD models utilized this updated tunnel geometry. The result is shown in Figure 3.2 below.

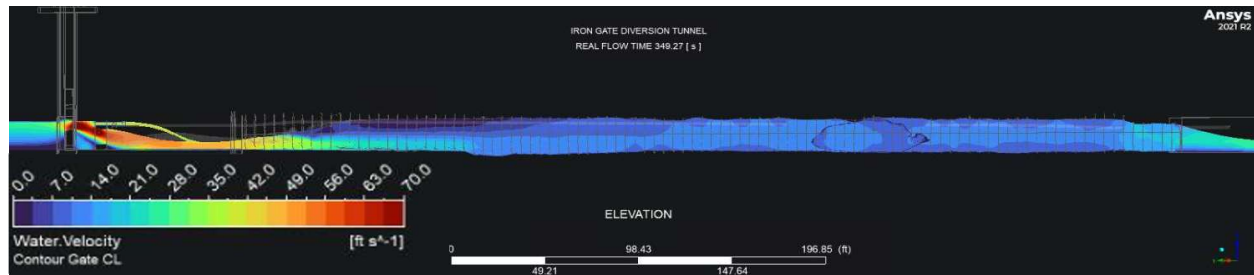


Figure 9 CFD Simulation – Tunnel Geometry based on Survey Data – Section View

Comparing this result with that of the typical cross-section combined with a concrete liner, the location of hydraulic jump traveled upstream, nearly inside the existing reinforced concrete liner. The geometric roughness of the survey was able to represent multiple locations of constriction and expansion within the unlined rock walls of the tunnel. These features may have been present during original construction or developed over time with intermittent use of the low-level outlet. This was a very positive result and suggested it may be possible to eliminate the concrete liner if the short period of time it took stabilize the hydraulic jump near the existing reinforced concrete liner was considered acceptable.

3.2 *Additional Energy Dissipation to Eliminate Concrete Liner*

As observed in the CFD simulations of the existing conditions, the tunnel has adequate energy-dissipating capacity to subject the 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 was proposed that the new concrete liner be eliminated. To support this decision and pull the jump further upstream and fully inside the existing reinforced concrete liner, the use of baffles inside the lined portion of tunnel was proposed, as shown in Figure 3.3.

Reinforced concrete baffles, 2 ft x 2 ft x 3 ft (0.6 m x 0.6 m x 0.9 m) WxHxL, were anchored through the existing concrete lining to the bedrock below and introduced into the high velocity flow providing additional means of initiating the energy dissipation earlier in the tunnel alignment. CFD simulation with the baffles show that the hydraulic jump moves upstream just enough to stabilize within the existing horseshoe concrete-lined portion of the tunnel.

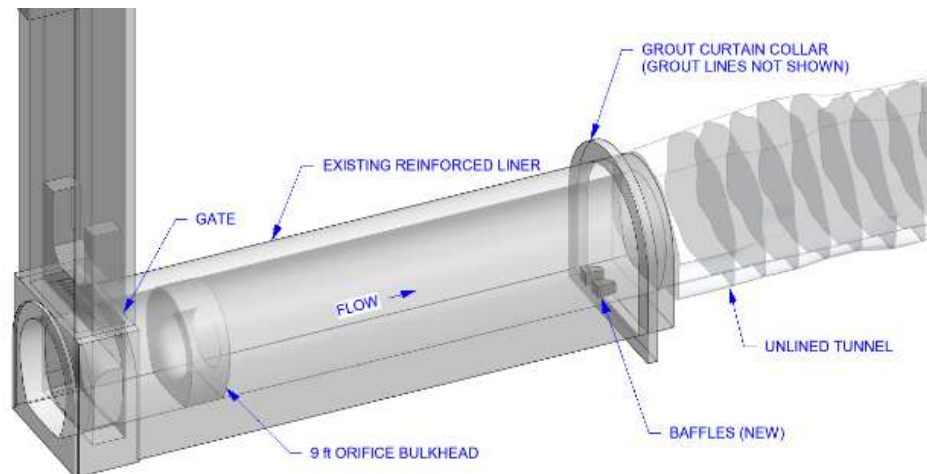


Figure 10 Tunnel Modification – Use of Baffles

With the combination of the unlined tunnel 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 (3.0 to 4.5 m/s) once the hydraulic jump has stabilized, as shown in Figure 3.4 below. This process was estimated to occur within 7 minutes of the full gate opening. These lower velocities facilitated the elimination of the new concrete liner.

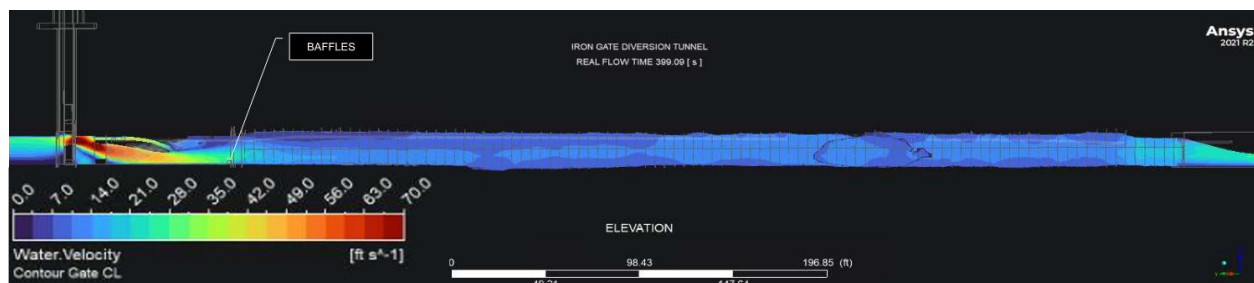


Figure 11 CFD Simulation – Introduction of Baffles – Section View

For added outlet constriction and to maintain tailwater levels directly following initial gate opening, a riprap side slope was added at the tunnel outlet. This was shown to improve performance and limit reacceleration due to rapid flow expansion when it exited the tunnel. Some unraveling of the side slope was intended and the volume was chosen to ensure blockage of the tunnel would not occur and impact the overall discharge rates from the tunnel.

3.3 Design for Discharge Capacity

3.3.1 Discharge-Reservoir Level Rating Curve

Considering the development of the hydraulic features within the tunnel since the discharge rating curve had been produced for input into the drawdown modelling, updated model had to be applied to the projected rating to curve. The majority of the modeling focused on the maximum head condition and the period of after gate opening until the hydraulic jump had stabilized within the tunnel creating stable hydraulic conditions. The discharge rating curve required these updated tunnel modifications to be applied at low head conditions. The low head conditions were less critical to the integrity and stability of the tunnel and tailrace area but were critical to achieving a low enough water level to safely initiate the final breach (Adria

et al., 2025) and support the overall construction schedule. Figure 3.5 below shows the variation in the rating curve prediction when changing from an idealized geometry to the surveyed geometry.

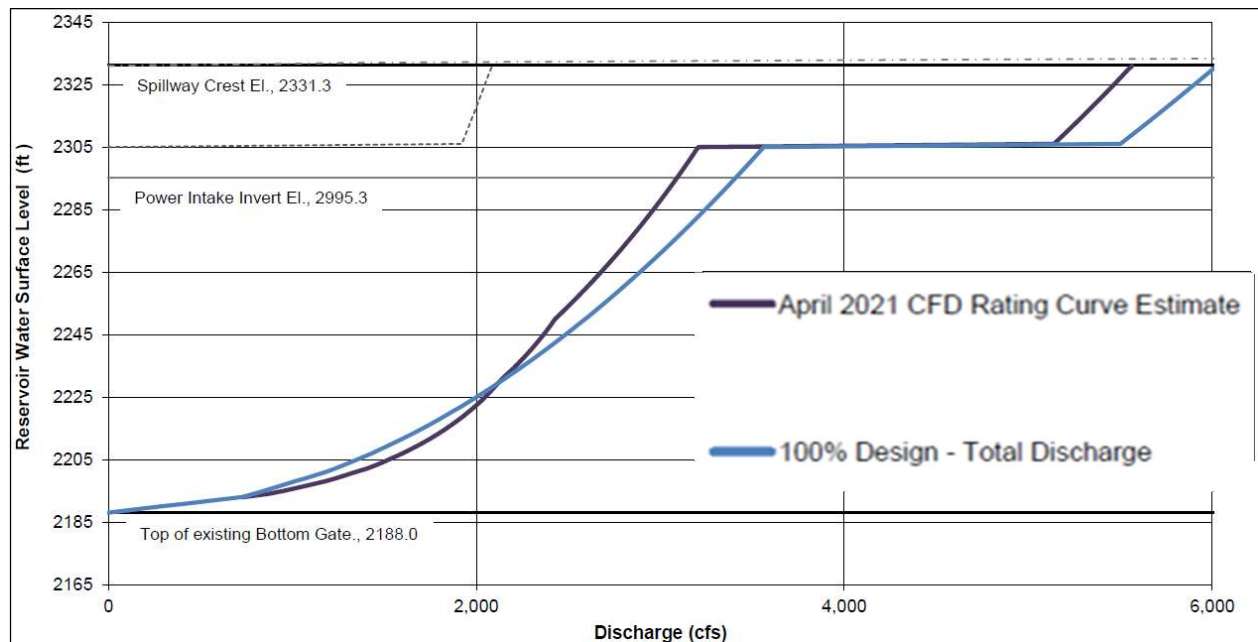


Figure 12 Discharge-Reservoir Level Rating Curve Study

The modeling change in tunnel design indicated reduced discharge efficiency at high head but improved discharge efficiency at low head. Further variation at low head was observed during decommissioning. This was speculated to be related to sedimentation in the section of tunnel upstream of the gate but could not be confirmed. While relatively minor and within what some might consider a reasonable margin of error for this type of analysis, the variation was enough to request support in managing the outflows from upstream facilities entering the IGD reservoir to ensure a safe breach water surface elevation could be achieved.

4 CONCLUSION AND LESSONS LEARNED

The successful evacuation of the Iron Gate Reservoir to support the Iron Gate Dam decommissioning as part of the Klamath River Renewal Project was made possible by repurposing the historic diversion channel and emergency environmental flow release channel. The tunnel upgrades originally proposed were schedule- and cost-prohibitive as they attempted to eliminate the uncertainty associated with hydraulic complexity, unknown data and historic equipment. This approach unavoidably increased and introduced other hydrologic and construction risks.

Risk management and a flexible, iterative design process supported by detailed CFD analysis enabled the design team to propose an effective solution while minimizing the new construction scope. Important observations made during the design and implementation of this project include:

- The incorporation of a reasonably accurate representation of the tunnel geometry in lieu of an idealized geometry, selected for conservatism, was pivotal. This result underscores the impact of macro geometric roughness on a hydraulic channel, specifically on an unlined tunnel intended for hydraulic conveyance. Often preliminary tunnel designs will consider a minimum cross section to be achieved to ensure hydraulic performance with a surface roughness assigned based on excavation methods and rock quality. The CFD analysis performed on the IGD diversion tunnel provides an example of how that would underestimate the friction losses therein. In this case the

added losses and energy dissipation was a notable benefit to the design goals but that is not always the case.

- Air-water flow interaction and air ventilation in the tunnel were key design considerations noting the irregular pattern of flows that are generated within the variable cross section when flowing near full. Proper ventilation was required to inhibit the buildup of negative pressures as the tunnel filled up with water to promote open-channel conditions in the closed-conduit flow.
- Given the many variables and inputs associated with analysis of this nature, it remains important to understand and acknowledge the limitations and incorporate them into the construction risk analysis.

This paper presents a brief summary of the discussions, analysis, design, and construction risk management completed by KP, NHC, Kiewit, and others to successfully reactivate the historic diversion tunnel to support reservoir drawdown and decommissioning.

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