# APPENDIX D1
# COPCO NO. 2 HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

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# 1.0 INTRODUCTION

This appendix includes a summary of data, design methodology, and other information used in the civil, hydrotechnical, and geotechnical design of the dam removal operations and structure evaluations at the Copco No. 2 Hydropower Facility.

Appendix D2 provides a summary of the computational fluid dynamic (CFD) modeling, and 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 DAM

### 2.2.1 RIVER CHANNEL

A remediated river channel is designed for the reach through the Copco No. 2 diversion dam that backfills the footprint of the dam excavation created by the removal of concrete directly below the channel to El. 2,453.5 ft, and excavates material upstream of the diversion dam to widen the thalweg of the river. The backfill of the channel invert is designed to minimize fill volume and connect the natural thalweg upstream of the intake structure on river left at El. 2,462 ft to the river invert downstream of the spillway apron at El 2,452 ft. An excavation upstream of the diversion dam on the right bank that will improve fish volition by widening the natural thalweg is included in the final channel grade at the request of the remediation Contractor. The resulting channel grade is 3.8% and creates a pool downstream of the existing apron due to a natural highpoint in the riverbed approximately 50 ft downstream of the sill. The channel grade geometry is developed in collaboration with the remediation Contractor to ensure adequate backfill will be placed over the remaining concrete abandoned in the channel. The excavation of the historic diversion dam upstream of the Copco No. 2 diversion dam is specified to be removed to match the adjacent riverbed invert and does not require backfill. The temporary channel excavation shown on Drawing C3520 and discussed in Section 2.5 must be backfilled if it is constructed by the Project Company. The final river channel is presented on Drawing C3234.

### 2.2.2 BACKFILL DESIGN

Backfill at the former dam site will comprise Erosion Protection (E7b) and 'Riverbed Material' as shown on Drawing C3234. "Riverbed Material' is unique to Copco No. 1 and 2 and is described on Drawing C3234.

The intent of the use of 'Riverbed Material' is to provide similar material to what is in the reach between Copco No. 1 and Copco No. 2, but to avoid finer material that was deposited during the reservoir impoundment.

The erosion protection is required to protect the backfilled slopes that overlay the concrete from the diversion dam that is abandoned in place, and discussed below in Section 2.5.3.

## 2.3 PRE-DRAWDOWN DIVERSION DAM WORKS AND CONTINGENCY REMOVAL METHOD

The Project Company plans to remove the entire diversion dam to the limits shown on Drawing C3221 during the pre-drawdown year by using Copco No. 1 to temporarily stop all flow into the Copco No. 2 reservoir, allowing for construction to occur in the dry. The removal of the historic diversion dam, the installation of the intake concrete plug, and the final channel grading would also occur during the Pre-drawdown year while the Copco No. 2 reach is dewatered. A diversion dam contingency removal method that spans both the Pre-drawdown and Drawdown years is shown on Drawings C3210, C3211, C3216 and C3217, with ancillary works to support the contingency removal method included on Drawings C3240 and C3520. Sections 2.4.1, 2.4.2, 2.4.3, 2.5, 2.6.1, 3.3.1, 3.4, and 3.5.1 include design summaries that pertain to works that support the diversion dam contingency removal method. These design summaries are only relevant if the contingency removal method is employed.

## 2.4 CONCRETE DIVERSION DAM REMOVAL STABILITY

### 2.4.1 CURRENT CONDITIONS OF UPSTREAM FACE

An underwater inspection was completed on November 7, 2019 to assess the current condition of the concrete diversion dam upstream face at Spillway Bay No. 1. The spillway bay was found to be in good condition, with no spalling concrete or exposed rebar. No open horizontal or vertical construction or expansion joints were observed. The riverbed was measured to be at El. 2,459.7 ft at the center of the bay. The riverbed material upstream of the spillway bay comprises fine to coarse sub-angular gravel and cobbles.

### 2.4.2 PRE-DRAWDOWN SPILLWAY CONCRETE PLUG

The diversion dam contingency removal method requires a portion of Spillway Bay No. 1 to be removed during the diversion dam Pre-Drawdown removal, as shown on Drawings C3210 and C3211. The removal would be a modification to the diversion dam that would be in place while Copco No. 2 continues to function as a water retaining dam and operate under normal power generating conditions. An analysis of the concrete plug to ensure the modified dam will meet all stability requirements for sliding stability and moment equilibrium is required.

The concrete ogee spillway does not contain any appreciable reinforcement and therefore American Concrete Institute (ACI) Code 350 does not apply. The plug is analyzed as a mass concrete structure as described in ACI Code 207.1R. Where as-built information is not available, the following conservative assumptions are incorporated into the analysis:

- The concrete plug acts as a mass concrete structure with a continuous unbonded horizontal construction joint and a vertical joint that passes through the entire ogee mass on the right side of Spillway Bay No. 1 between the ogee and the pier.
- The horizontal unbonded construction joint has a coefficient of internal friction of 1.0 and no cohesion (ACI, 2006). The assumed location of the horizontal unbonded joint is analyzed at four equally spaced elevations, starting at El. 2,459.5 ft (the elevation of the spillway apron) and ending at El. 2,467.5 ft.
- A concrete compressive strength of 3,000 psi.
- At least 50% of the resistance provided by one shear key contributes to the stability of the plug, provided a minimum of two shear keys remain fully intact.
- Shear strength of shear keys is estimated using methodology from literature (Curtis and Lum, 2008).

Two critical water levels are considered for the static case of the analysis:

- Case 1 – Headwater to top of radial gates at El. 2,487.5 ft and dry conditions downstream of the dam.
- Case 2 – Q100 flows producing a headwater elevation of 2,495 ft and a tailwater elevation of 2,479 ft.

One critical water level is considered for the OBE pseudo-static case of the analysis:

- Normal headpond operating level at El. 2,486.5 ft and dry conditions downstream of the dam.

The concrete plug is assessed for relevant gravity dam stability design criteria, to determine the minimum required thickness that could withstand the loading conditions listed above. The concrete plug thickness is governed by sliding stability, which requires a minimum plug thickness of 12 ft. The assumption that requires two shear keys remain intact precludes the concrete plug from being less than 17 ft, which is therefore selected as the design thickness.

### 2.4.3 LEFT BANK WING WALL STABILITY

The contingency diversion dam removal method requires Spillway Bay No. 1 to be removed down to El. 2,459.5 ft to complete drawdown at Copco No. 2, as shown on Drawings C3216 and C3217. The ogee spillway provides support to the left bank wing wall, which is required to remain stable after the ogee crest is removed to prevent erosion of the bank from river flows until the diversion dam is fully removed. The bay removal is considered to ensure the portion of the wing wall that spans the removed spillway will remain intact during the drawdown year.

The required structural stability assessment uses a SAP2000 finite element model to check the wall for shear and moment resistance, and reinforcement overstressing. The following inputs and assumptions are used in the structural analysis of the left wing wall:

- A concrete compressive strength equal to 3,000 psi and allowable compression stress equal to 1,350 psi.
- A reinforcement yield strength equal to 33,000 psi and allowable yield strength in tension equal to 16,000 psi. (JCR, 1924).
- Horizontal reinforcement in wall is 3/4 inch sq. bars in each face spaced at 24 inch on centers (Historic Drawing F-3730, F-3734).
- Vertical reinforcement in the wall is 1 inch sq. bars in each face spaced at 24 inch on centers (Historic Drawings F-3733,3768).
- Lap splices included for the reinforcement; 90-degree hooks were included on some of the rebars as per the historic drawings.
- Backfill over the upstream half of the analyzed span comprises lean concrete abutting the intake downstream wall to approximately El. 2,480.5 ft, which then slopes down to El. 2,476.5 ft at the approximate mid-way point in the analyzed wall span, as per Historic Drawing F-3730. A site reconnaissance identified the lean concrete on the ground surface in this area, and it is therefore assumed the drawing specifications were followed. The lean concrete backfill is assumed to be self-supporting and does not impose a load on the wall since it extends from the wall back to the counterfort along the intake downstream wall, however the groundwater behind the wall does impose a load.
- Backfill over the downstream half of the analyzed wall span comprises a coarse gravel/rock fill that was observed on surface during a site reconnaissance to approximately El. 2,476.5 ft. It is assumed the rock fill continues below surface to the invert of the wall.
- Groundwater behind the wall at El. 2,464.5 ft with no water in the spillway chute or stilling basin to simulate a rapid drawdown condition.
- Coarse gravel/rock backfill physical properties:
  - density = 130 pcf.
  - friction angle = 40°.
  - cohesion = 0 psf.
  - active earth pressure Ka = 1-sinØ.
- Both fixed and pinned edge conditions are evaluated since the historic drawings are not clear as to the degree of fixity on the bottom and upstream edge of the wing wall.

- The connection between the wing wall and the downstream apron wing wall is assumed to be monolithic and capable of transferring moment since the reinforcement extends across the construction joint and appears to form a lap splice (Historic Drawing F-4001).

It is considered to be more appropriate to use the specifications that coincide with the design standards at the time of original construction of the wall. The reinforcement is analyzed for an allowable tension stress of 16,000 psi, as recommended by the 1924 Joint Committee Report ‘Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete’ (CRSI, 2001).

Based on the assumptions and inputs listed above, the results of the analyses indicate that the tensile stress in the horizontal reinforcement do not exceed the yield strength of the reinforcement for the condition with the upstream and bottom edges of the left wing wall assumed to be pinned. The tensile stresses in the horizontal reinforcement do not exceed the yield strength of the reinforcement for the condition with the upstream edge assumed to be fixed and the bottom edge assumed to be pinned. The loading conditions for all pinned edge restraint conditions exceed a safety factor of 1.8. The tensile stress in the horizontal reinforcement will be less than the allowable tensile strength for the condition with the upstream and bottom edges assumed to be fixed. The tensile stresses in the vertical reinforcement will be less than the allowable tensile strength for all edge restraint conditions. The compression stresses in the concrete are less than the allowable compression strength for all edge restraint conditions. The shear stresses are also less than the allowable shear strength for all edge restraint conditions.

Based on the results of the structural analysis described above and the assumptions regarding the backfill loadings and edge restraint conditions, the spillway left wing wall should be stable for the condition with the entire ogee structure removed for the diversion of river flows.

#### 2.4.4 INTAKE DOWNSTREAM WALL STABILITY

The construction of the intake portal closure requires that the top slab of the intake, the trash rack, and concrete trash rack frame be removed. The intake downstream wall will be partially removed as part of the dam removal, as shown on Drawing C3221, but it is unclear if that will occur prior to the construction of the closure. The stability of the downstream intake wall is analyzed to ensure the removal of the structural components within the intake structure do not destabilize the downstream wall.

The required stability assessment uses a SAP2000 finite element model to check the wall for shear and moment resistances, and reinforcement overstressing. The following assumptions and inputs are used in the structural analysis of the intake downstream wall:

- The water level on the downstream side of the wall is assumed to be at 2,467.5 ft, which corresponds to a tailwater elevation that exceeds the 1% probable flood flows for July through September. The area inside the intake structure is assumed to be dry for the construction of the closure.
- A reinforcement yield strength equal to 33,000 psi and allowable tensile stress equal to 16,000 psi. (JCR, 1924).
- The intake downstream wall remains intact; however, the left wing wall is fully removed.
- Lean Concrete backfill on the downstream side of the wall (Historic Drawing F-3730).
- Earthquake loading associated with the Copco No. 1 OBE is applied to the wall (Appendix A).
- Connections with downstream buttress and intake slab are fixed.
- Top and northeast end of wall are free.

The results show very low stresses in the intake downstream wall for the water loading on the downstream face during demolition with the spillway left wing wall, top slab, and trash rack beams removed. Based on the assumptions and results of the structural analyses, the intake downstream wall should be stable for the loadings applied during the temporary condition described above.

### 2.4.5 INTAKE CONCRETE PLUG

A reinforced concrete wall is designed to be placed against the existing tunnel intake gate and provide a permanent plug for the tunnel. The plug will be supported along the bottom and sides with reinforcement dowels anchored into the existing concrete intake walls and extending into the new concrete plug. It is assumed the existing gate will deteriorate over time and not provide support to the concrete plug. The concrete plug will include reinforcement in both bending directions to resist the applied loads from the concrete rubble backfill and river and provide the required minimum flexural reinforcement.

The structural analysis of the intake concrete plug uses the following assumptions and inputs:

- The existing gate will be lowered and used as the downstream form for construction of the concrete plug.
- The upstream face of the plug will be angled at the same incline as the intake gate and therefore the plug will have a uniform thickness.
- The top of the concrete plug will form a watertight seal to prevent seepage into the tunnel.
- 1% probable annual flood water level is at El. 2483.0 (applying a conservatively high Manning's n value of 0.06) and concrete rubble fill level is at El. 2492.5.
- Bottom of concrete wall at El. 2458.5 and top at approximately El. 2479.5 (bottom of 12 inch thick curved curtain wall above).
- Earthquake loading is for a permanent structure using the Copco No. 1 Maximum Credible Earthquake (MCE) PGA of 0.26g (PacifiCorp, 2015b) and seismic coefficient equal to two thirds of the PGA.
- Existing concrete has a compressive strength equal to 3,000 psi. New concrete has a compressive strength equal to 4,000 psi and new has a reinforcement yield strength equal to 60,000 psi.
- Saturated unit weight of concrete rubble fill equal to 145 pcf, and bulk unit weight equal to 140 pcf.
- For the static loading assume at-rest earth pressure for the concrete rubble fill.
- For the earthquake loading assume active earth pressure for the concrete rubble fill.
- A waterstop will be installed around the perimeter of the concrete plug where it contacts the existing concrete to prevent seepage into the tunnel.
- The surface of existing concrete in contact with new concrete will be roughened to an amplitude of ¼ inch to create better bond and shear friction resistance.
- Strength design approach is used with ACI 318 load factors for reinforced concrete design.

The required thickness and reinforcement for the concrete plug is calculated for loads generated from the concrete rubble fill and river. The design considers the earthquake loading due to inertia forces from the concrete wall and dynamic earth pressures from the concrete rubble fill. The moments and forces in the wall are calculated using a two-dimensional finite element plate analysis subjected to varying and uniform loads from the concrete rubble fill and hydrostatic water loads. The plate is assumed to be free to deflect laterally at the top and hinged on the bottom and along both sides. The analysis also considers an alternative connection condition with the top of the plate pinned against lateral movement. Reinforcing bar dowels will be installed along the bottom and sides to provide lateral support and shear resistance.

Maximum shear occurs at the bottom of the plug in the center and controls the wall thickness. The thickness required to resist the maximum shear is 28 inches at the bottom of the plug. Maximum moment about the vertical axis occurs at the top center of the plug and creates tension on the downstream face in the horizontal direction. The reinforcement required to resist the maximum moment is #9 rebar at 12-inch spacing horizontally on the downstream face. Maximum moment about the horizontal axis occurs at about one-third of the plug height in the horizontal center of the wall and creates tension on the downstream face. The reinforcement required to resist the maximum moment is #7 rebar at 12-inch spacing vertically on the downstream face. Minimum reinforcement required for flexure is provided in the horizontal and vertical directions on the upstream face.

## 2.5 SPILLWAY APRON TEMPORARY WORK PLATFORM

The contingency removal method temporary spillway apron work platform is designed to provide a dry working surface for the removal of the remainder of the diversion dam after drawdown occurs, and is shown on Drawing C3520. The work platform forms a channel with the left bank wing wall that has an invert at the spillway apron and will allow water to flow freely through Spillway Bay No. 1 while demolition proceeds. The work platform elevation of 2,465.0 ft is designed to exceed the maximum tailwater elevation adjacent to the platform for a flow of 1,900 cfs, which corresponds to the 20% probable flood for the June 16-30 period, and is modelled to be 2,464.8 ft. The results of the CFD modelling are provided in Appendix D2 and are contingent on the construction of the temporary channel discussed below. The work platform fill will comprise General Fill (E9b) which is free draining and will provide a dry working surface. The side slopes of the platform will be stabilized and protected from erosion through the use of grout bags. Grout bags (or similar approved) will be designed to withstand flows up to predicted maximum river flow velocity against the work platform associated with the June 16 - 30, 20% probable flood, approximately between 8 and 13 ft/s, as reported in Appendix D2.

A highpoint in the river channel is present downstream of the concrete dam that has an invert higher than the spillway apron. A temporary channel excavation downstream of Spillway Bay No. 1 is required to prevent backwatering caused by the highpoint that will increase the water elevation at the temporary work platform during the dam removal works. A 26 ft wide channel will be constructed downstream of the dam that abuts the spillway apron at elevation 2,459.5 ft and continues downstream at 1% until daylighting into the natural river grade, approximately 240 ft downstream of the apron, as shown in Drawing C3520. The effect of the channel on the tailwater elevation during the dam removal is illustrated in Appendix D2. The Project Company may opt to not construct the channel; however, it will increase the design tailwater elevation at the platform approximately 2.5 ft, and the platform will have to be raised if a dry and stable working surface is desired. If the channel is constructed it should be excavated prior to constructing the temporary work platform and must be backfilled after the dam removal is complete to the original riverbed elevation using the same material that is excavated from the channel. The channel is not required if the primary diversion dam removal method is employed.

## 2.6 EROSION PROTECTION

### 2.6.1 SPILLWAY APRON

The potential for scour at the spillway apron if the contingency removal method is employed is presented in Section 3.4 and shows pockets of shear stress on the native bed material directly upstream and downstream of Spillway Bay No. 1. The probability of erosion of the spillway apron is assessed following the USACE Best Practices in Dam and Levee Safety Risk Analysis guidelines (USACE, 2015) and is a function of the quality of the concrete. The Erodibility Index of the concrete is estimated to vary between 6,400 and 11,520. The maximum stream power density is computed to be 8.43 HP/ft$^2$ (conversion1 KW/m$^2$ = 0.125 HP/ft$^2$). Based on the flow depth and velocity along the spillway apron, the scour potential for the conditions evaluated are below the threshold of 1% probability of erosion, as defined in the USACE guideline. Estimated stream power values are shown on Figure 2.1.

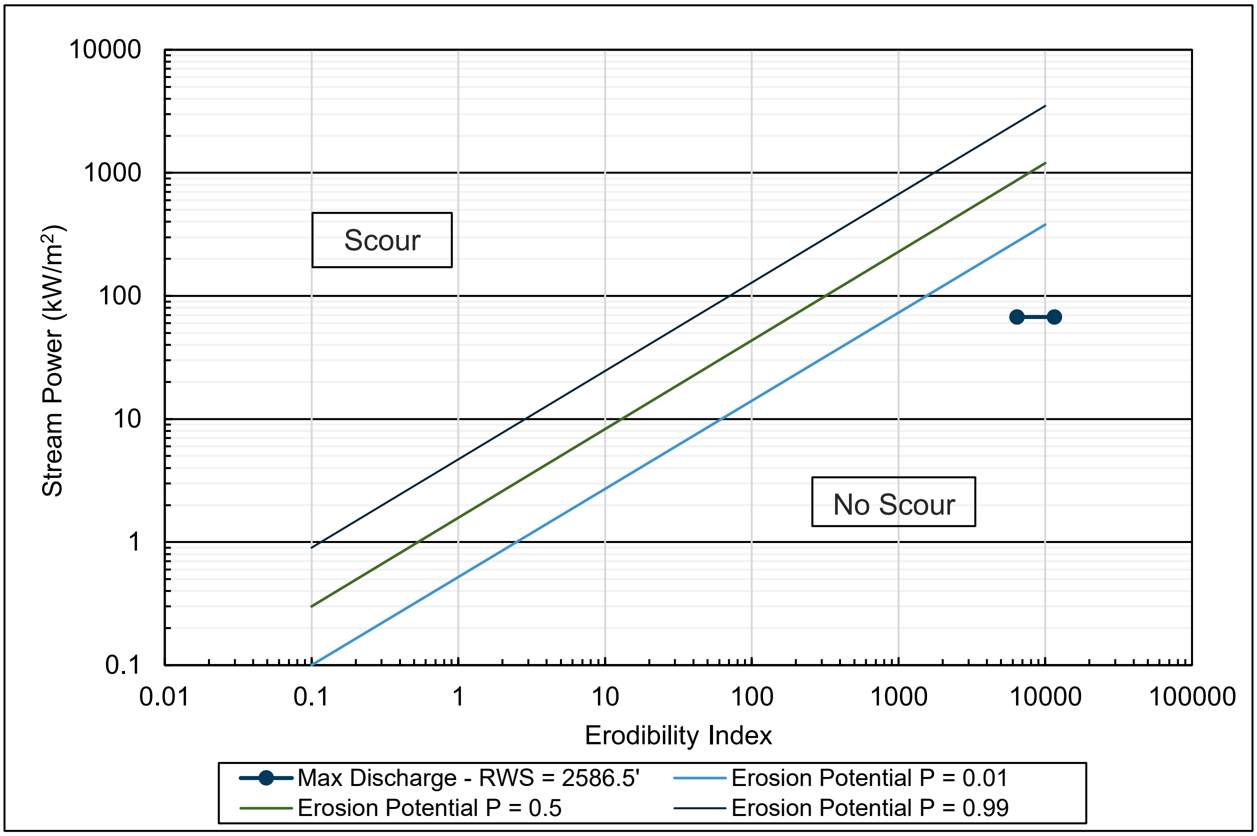



**Figure 2.1 Probability of Erosion of the Spillway Apron**

### 2.6.2 INTAKE STRUCTURE BACKFILL

The intake structure backfill may comprise both Concrete Rubble (CR2) and or 'Riverbed Material'. The maximum backfill limit of the concrete rubble is delineated on Drawing C3232. The limit is designed using the final channel grade to ensure the minimum blanket thickness of erosion protection, as shown on Drawing C3234, is present above any remaining concrete. All concrete buried in place above the maximum 1% probable annual flood river level will be covered with a minimum of 2 ft of General Fill (E9). The Concrete Rubble (C2) is coarse enough that a filter layer between the concrete rubble and the erosion protection is not required.

### 2.6.3 FINAL CHANNEL SLOPE EROSION PROTECTION

Erosion protection is required along the final river channel slopes that will overlay the buried concrete remaining after the diversion dam removal. Erosion protection for the final river channel at Copco No. 2 is designed for the post dam removal 1% probable annual flood with 3 ft of freeboard. Final channel characteristics and geometry are used to develop a HEC-RAS 2D model, which produces a velocity profile and water surface elevation that is used to determine the required erosion protection. The HEC-RAS 2D results conservatively do not consider the channel excavation upstream of the diversion dam on the right side of the channel.

The maximum river level during the 1% annual flood event is 2483.0 ft, when applying an upper bound Manning's n value of 0.06. The design velocity is divided into two zones, a lower and an upper zone, to account for the large range in velocities along the side slopes. The dividing line between the zones is set at elevation 2,466 ft. Based on a Manning's n of 0.04, the maximum design velocity is 22 ft/s at the toe of the erosion protection slope and 17 ft/s at the dividing line between the zones.

The modified-Maynord method is used to determine the size and thickness of erosion protection that is required to resist the design velocities.

Minimum $D_{50}$ sizes of 34 inches and 20 inches are calculated for the 22 ft/s and 17 ft/s design velocities, respectively. The boundary between zones is conservatively increased by 2 ft to El. 2,468 ft for the final design, and the upper limit of the upper zone is set to an elevation of 2,486 ft to provide a 3 ft freeboard.

The blanket thickness of the erosion protection is selected based on the equivalent Caltrans riprap class grading, which the E7 material grades mimic. To reduce the size of the lower zone minimum $D_{50}$, the thickness is increased to 8 ft. The upper zone thickness is 3.5 ft. Both material types are anticipated to be placed in the dry under the primary diversion dam removal method. The material thickness may need to be increased if placement of the erosion protection material occurs in-water.

The lateral extents of the erosion protection and typical section details are shown on Drawing C3234. The erosion protection material will conform to technical specification 31 05 00 Materials for Earthworks.

The underlying native bank material at the diversion dam is expected to comprise primarily cobbles and boulders with silt, sand, or gravel, based on the historic geological section and photographs (see Figure 2.2). If adequate large particle sizes are present within the native bank material, a filter material between the erosion protection and subgrade will not be required. Similarly, the material below the toe of the erosion protection at the intake area, as shown on C3232, is anticipated to comprise material with a particle gradation similar to the specified erosion protection, and therefore revetment toe protection is not required. Accordingly, in-river conditions will have to be assessed during the diversion dam excavation to ensure these two assumptions are applicable. The concrete and riverbed material that is to be placed in the intake structure on the channel bank has an approximate $D_{85}$ of 8 inches, which has an adequate filter relationship with the erosion protection cover material to prevent undermining of the erosion protection, as per USBR guidelines.

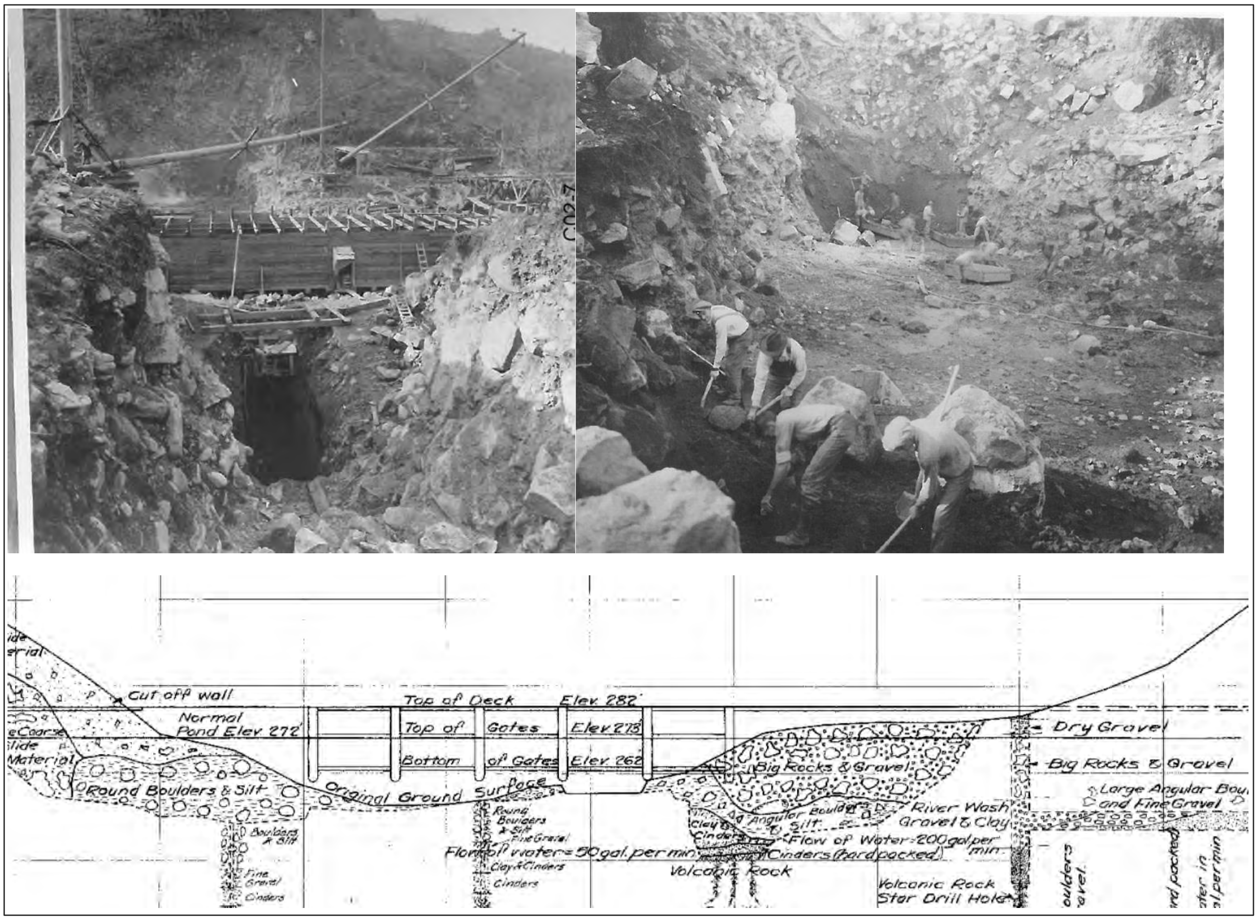



**Figure 2.2 Diversion Dam Excavation (Top Left and Right), Historic Geological Interpretation along Centerline of Dam**

### 2.6.4 POWERHOUSE TAILRACE BACKFILL

The concrete disposed in the tailrace will be capped with earthfill material so that it is not exposed on surface. The erosion protection assessment that considers the peak water surface level and maximum velocity of flow adjacent to the backfill slope (detailed in Section 3.6.3) suggests that minimal erosion protection is required due to the low design velocity across the majority of the backfill face. The concrete rubble, however, is still required to be covered. A 2 ft thick layer of E8 – Bedding material will be placed as cover, as it will provide erosion protection, and will limit the amount of material lost in the voids of the disposed concrete or to the river flow.

The north toe and edge of the tailrace backfill that abuts the subgrade of the removed right wing tailrace wall is an interface boundary between the main channel flows and the sheltered tailrace backfill. The flow velocities observed in the main channel decrease across this interface, as detailed in Section 3.6.2. Localized erosion protection will be used in the interface area of the backfill that may be subjected to an increased velocity, and it will extend into the tailrace backfill to an area where the lower velocities definitively govern. The design depth-averaged velocity and depth are selected to be 15 ft/s and 16 ft, respectively, as determined in Section 3.6.2.

The modified-Maynord method is used to determine the size and thickness of erosion protection that is required to resist the design velocities. The design velocities are determined using a HEC-RAS 2D model.

A minimum $D_{50}$ of 18 inches is calculated for the design velocity, so the E7b 'Erosion Protection' material is specified for localized erosion protection area. The majority of the erosion protection will be placed in-water, so the specified thickness is increased 50% from 3 ft to 4.5 ft.

The tailrace backfill is shown on Drawing C3420.

# 3.0 HYDROTECHNICAL

## 3.1 RESERVOIR DEPTH-AREA-CAPACITY

The depth-area-capacity relationships for the Copco No. 2 reservoir are based on the 2018 bathymetric survey (NAVD88 datum) and are shown on Drawing C3057. The reservoir capacity at elevations relevant to the Copco No. 2 facility are summarized in Table 3.1.

**Table 3.1 Reservoir Storage Capacity for Various Key Elevations**

| Key Elevation Description | Elevation (ft) | Capacity (acre-ft) |
|---|---|---|
| Normal Operating Level | 2,486.5 | 57.0 |
| Spillway Crest | 2,476.5 | 14.8 |
| Spillway Bay No. 1 Removed Invert | 2,459.5 | 0.2 |

## 3.2 OUTLET STRUCTURE RATING CURVES

The reservoir water surface levels will be managed during pre-drawdown using the conveyance system to the powerhouse to allow for dam modifications works. The drawdown of the reservoir will comprise discharge through the existing spillway gates and the removal of Spillway Bay No. 1. The development of the discharge rating capacities for the spillway gates and the removal of Spillway Bay No. 1 are detailed in Appendix D2 and are shown on Drawing C3057.

## 3.3 RESERVOIR SEQUENCING

The Copco No. 2 reservoir will be dewatered during the Pre-Drawdown year using the attenuation capacity of the Copco No. 1 reservoir to facilitate the removal of the concrete diversion dam. The Copco No. 1 reservoir will be drawn down to increase storage capacity, and the Iron Gate reservoir to be filled to increase flow release capacity prior to initiating the Copco No. 2 reach dewatering sequence. The process may repeat several times to accommodate the construction schedule for the planned work. The final channel grading will be in place for the Drawdown year.

The contingency removal method would leave the majority of the concrete dam in place for part of the drawdown year, which impacts the removal methods and ancillary works by having continuous flow through the dam site. The drawdown model (detailed in Appendix G) is used to assess the reservoir water surface levels during drawdown and post-drawdown under a range of hydrologic conditions when the contingency method is employed. Copco No. 2 is operated as a run-of-river facility with minimal storage volume; therefore, evacuation of the reservoir will occur quickly. During drawdown using the contingency removal method, the behavior of the reservoir will be a reflection of upstream conditions, particularly conditions at Copco No. 1.

The following section discuss the results of the drawdown model and the implications to the project.

### 3.3.1 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN FOR CONTINGENCY REMOVAL METHOD

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 drawdown model only considers the diversion dam contingency removal method configuration where Spillway Bay No. 1 is removed to the spillway apron. The results of the drawdown model are summarized in three ways:

- Individual year simulations are provided in the attached Copco No. 2 Simulated Drawdown Figures 1 through 36. These plots indicate the following:
    - Reservoir water surface levels.
    - Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway and power intake).
- Maximum daily reservoir water surface level daily non-exceedance percentiles (percentiles) are shown on Figure 3.1, and on Drawing C3057. This figure represents the results from all 36 model simulations as non-exceedance percentiles of reservoir water surface levels to summarize the distribution of the results on any given day of the simulations. These results to not represent a single simulation and 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.

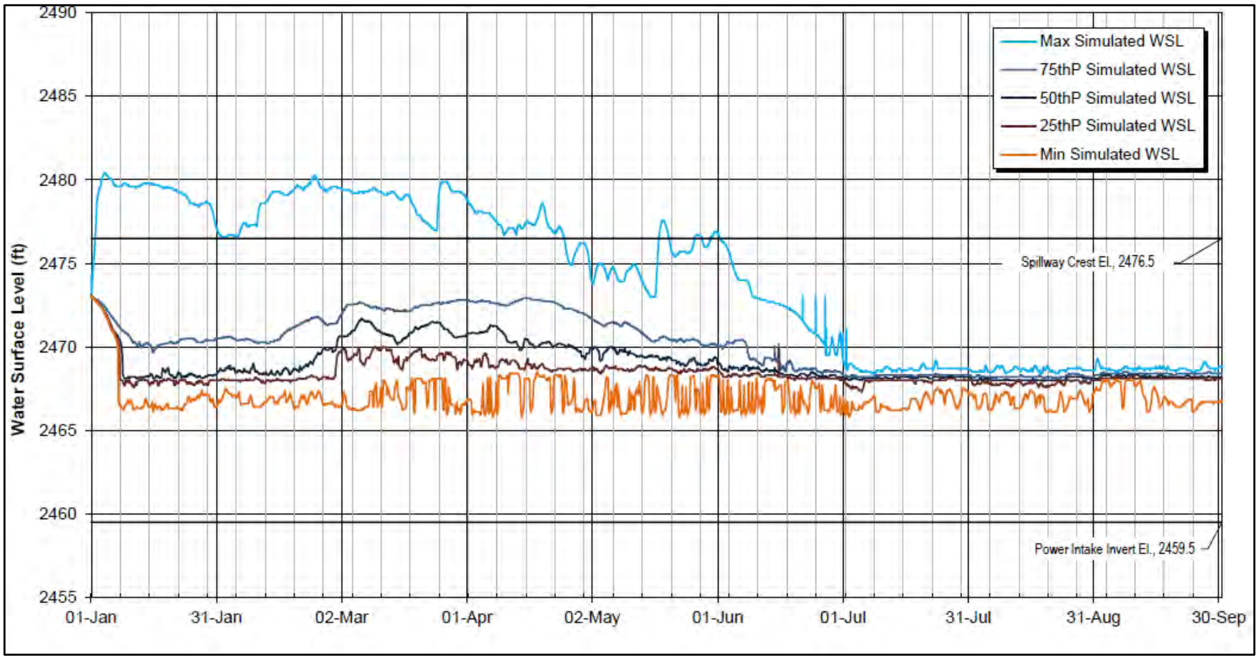



**Figure 3.1 Copco No. 2 Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles**

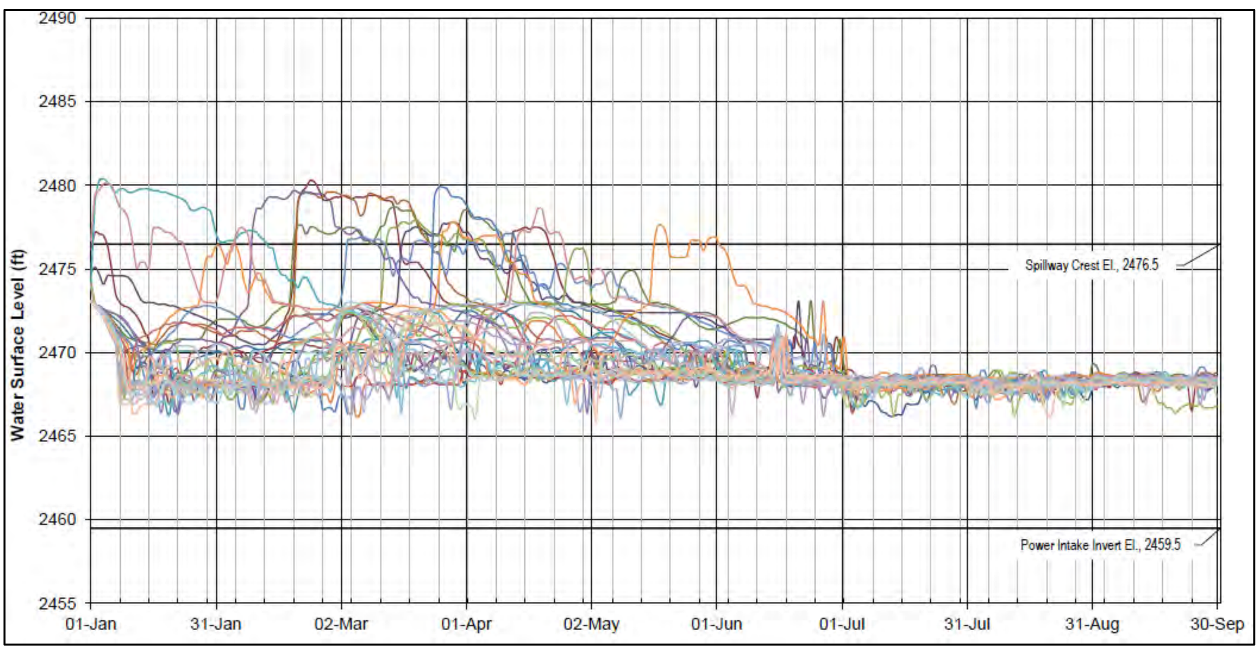



**Figure 3.2 Copco No. 2 Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot**

The simulated water surface levels on Figure 3.1 and Figure 3.2 are based on average daily conditions over the 36 year drawdown simulation period and show that there is a reduction in the reservoir water levels in mid-June with the majority of the simulated years achieving sustained low elevation water levels by the end of June. This is a function of inflow hydrology which indicates a reduction in streamflow for the second half of June (Appendix A6) and the timing of when the historic diversion tunnel is fully opened at Copco No. 1, which is targeted to be around June 15.

Figure 3.2 shows that there are large fluctuations in the reservoir water surface levels from January through June. Copco No. 2 is operated as a run-of-river facility with minimal storage volume; therefore, the reservoir water levels reflect the outflow conditions at Copco No. 1. The drawdown model results show that the flows may be discharged over the Copco No. 2 spillway between January and mid-June.

Lower reservoir levels will be sustained after July 1 depending on the hydrologic conditions and when the Copco No. 1 historic diversion tunnel is opened. The post-drawdown water surface levels 100 ft upstream of the dam face are within the range of 2,466.0 ft to 2,469.5 ft for all of the drawdown model simulations based on average daily conditions. Reservoir water surface levels would increase for low probability floods (i.e., the 1% and 5% probable monthly flood flows); however, no spillway overtopping is predicted for the period from June 16 to the end of September even in the case of a 1% probability flood occurring in this period, as shown in Table 1 on C3057.

Figure 3.3 shows reservoir drawdown distribution for the spillway crest elevation. This represents the cumulative percentage of model simulations and the dates when the reservoir water surface level is lower and sustained below the spillway crest. 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 level shown on Figure 3.3 is 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 this drawdown distribution. Occurrence of such events may shift the distribution to a later date. The following observations are made based on Figure 3.3:

- Elevation 2,476.5 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, 98% of the simulations by May 1, and 100% of the simulations by June 1. Spillway overtopping and flooding of the removal work area along the work platform is not likely to occur following June 1 based on average daily conditions for the 36 simulated model years.

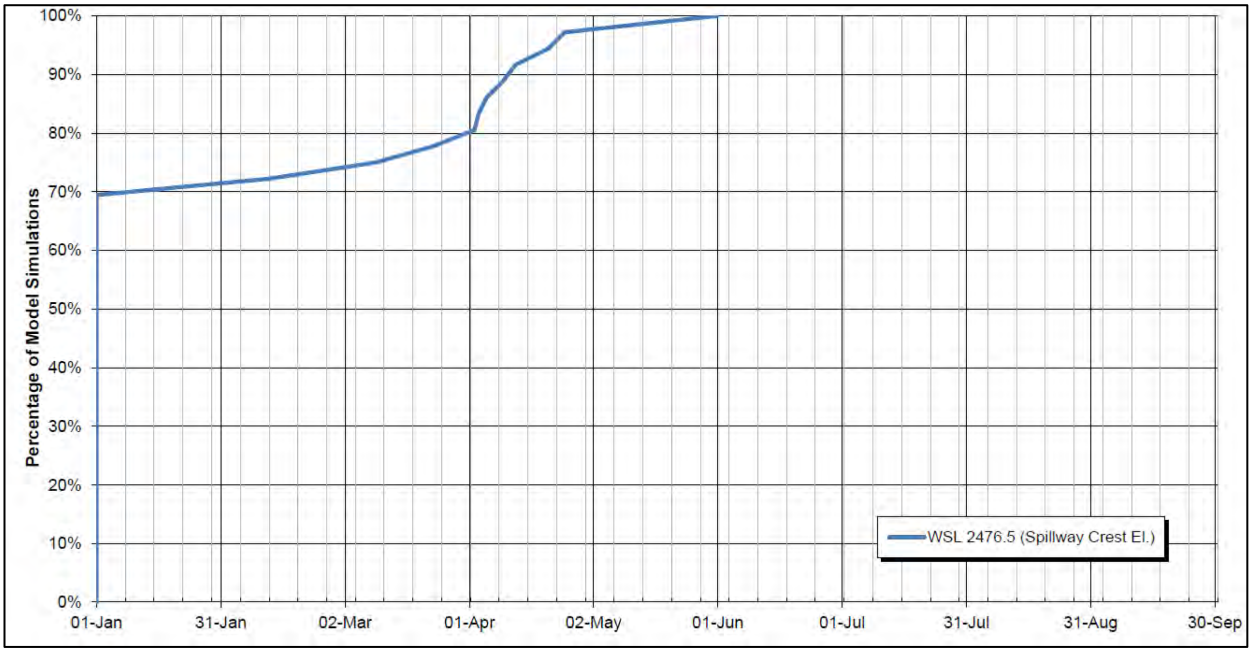



**Figure 3.3 Copco No. 2 Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below the Spillway Crest Elevation**

## 3.4 SCOUR POTENTIAL FOR CONTINGENCY REMOVAL METHOD

The computational fluid dynamics (CFD) model presented in Appendix D2 indicates that during the drawdown operation of the contingency removal method there will be varying levels of scour potential immediately upstream of the outlet structures. Shear stress magnitude figures are shown in Appendix D2.

Zones of high flow velocity coincide with zones of high bed shear stresses. These locations include the downstream end of the converging historic diversion dam, the left side of the historic diversion dam where flow can overtop, and immediately downstream of Spillway Bay No. 1.

It is anticipated that the flow will have the potential to scour and mobilize large cobbles (5 to 10 inches) throughout the deepest sections of the reservoir area. There are few localized areas where the flow would have the potential to mobilize medium to large boulders (20 to 35 inches). Given the 26 ft opening of the Spillway Bay No. 1, blockage from mobilized bed material is not anticipated.

## 3.5 STEADY-STATE WATER SURFACE LEVELS

### 3.5.1 CONTINGENCY REMOVAL METHOD WORK PLATFORM WATER SURFACE LEVELS

The contingency removal method requires a temporary construction work platform be built from the right bank onto the spillway apron to facilitate the diversion dam removal while water flows through Spillway Bay No. 1. The work platform is designed to have an elevation above the 20% probable flood between June 15th and September. The CFD modeling shown in Appendix D2 determines the flow velocities and water levels that the work platform is designed to withstand. A maximum water surface level of 2,464.8 ft and flow velocities up to 10 ft/s are anticipated adjacent to the work platform near the Spillway Bay No. 1 opening at the design discharge.

### 3.5.2 RESERVOIR DRAWDOWN AND POST-DRAWDOWN WATER SURFACE LEVELS AND TAILWATER LEVELS

Flood water surface levels at the dam and tailwater levels are shown on Drawing C3057 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 and Copco No. 1 facilities 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 and the average tailwater levels downstream of Spillway Bays No. 1 through 5 as detailed in Appendix D2 and shown on Drawing C3057.

### 3.5.3 HISTORIC DIVERSION DAM WATER LEVELS

The historic diversion dam located within the Copco No. 2 reservoir controls water surface levels upstream of the dam during low flows. A CFD model as outlined in Appendix D2 is used to determine the water surface levels upstream of the historic diversion dam under various flow conditions post-drawdown, to support construction methodology that requires equipment be used in the river. The water surface levels vary with location at a discharge of 990 cfs and decrease from approximately 2,477 ft just upstream of the historic diversion dam, to 2,475 ft through the narrow opening of the converging dykes, to 2,463 ft just upstream of the Copco No. 2 dam. It should be noted that the CFD model assumes the historic diversion is watertight. The historic diversion dam is anticipated to be porous given the original wood facing has deteriorated, which will ultimately lower the water surface elevations further.

### 3.5.4 TUNNEL NO. 1 STAGE-DISCHARGE CURVE

A stage-discharge curve for the Tunnel No. 1 is used to inform the process of dewatering of the Copco No. 2 reach. The curve shows the minimum water surface level that can be achieved for various flows by only diverting water through the water conveyance system. The stage-discharge curve uses a steady-state HEC-RAS model developed for Tunnel No. 1 and the reservoir intake bathymetry assuming fully open spillway gates. The key results from the model are as follows:

- Flow in Tunnel No. 1 has a steeper slope and the flow is supercritical at all discharges.
- Flow in the Wood-Stave and Tunnel No. 2 is subcritical due to their shallower slope and a hydraulic jump forms at the transition between Tunnel No. 1 and the Wood Stave.

- The stage-discharge curve is valid for flows up to 2,500 cfs with a high level of certainty. Up to this capacity, open channel flow conditions exist throughout all water conveyance segments (Tunnel No. 1, Wood Stave and Tunnel No. 2). For higher flows, the Wood-Stave and Tunnel No. 2 may enter full pipe flow conditions and the hydraulic jump may be pushed into the Tunnel No. 1 segment. Even though it is not expected that this condition would impact the water levels at the tunnel intake, full pipe flow conditions could not be modelled accurately with the existing HEC-RAS model, and as such, the rating curve is not shown for flows higher than 2,500 cfs.
- The stage-discharge relationship is set at the Spillway Bay No. 3 location, as this bay is found to have the highest water surface levels along the dam face. The stage-discharge curve is shown on Figure 3.4, and indicates that there is more than 2 ft of freeboard left at flows of 2,500 ft/s passing through the power intake. Spilling over the spillway bays is not likely to occur for flows lower than about 3,000 cfs; however, if construction activities are under way on the work platform downstream of the dam, these conditions will need to be closely monitored for the safety of the workers in that area.

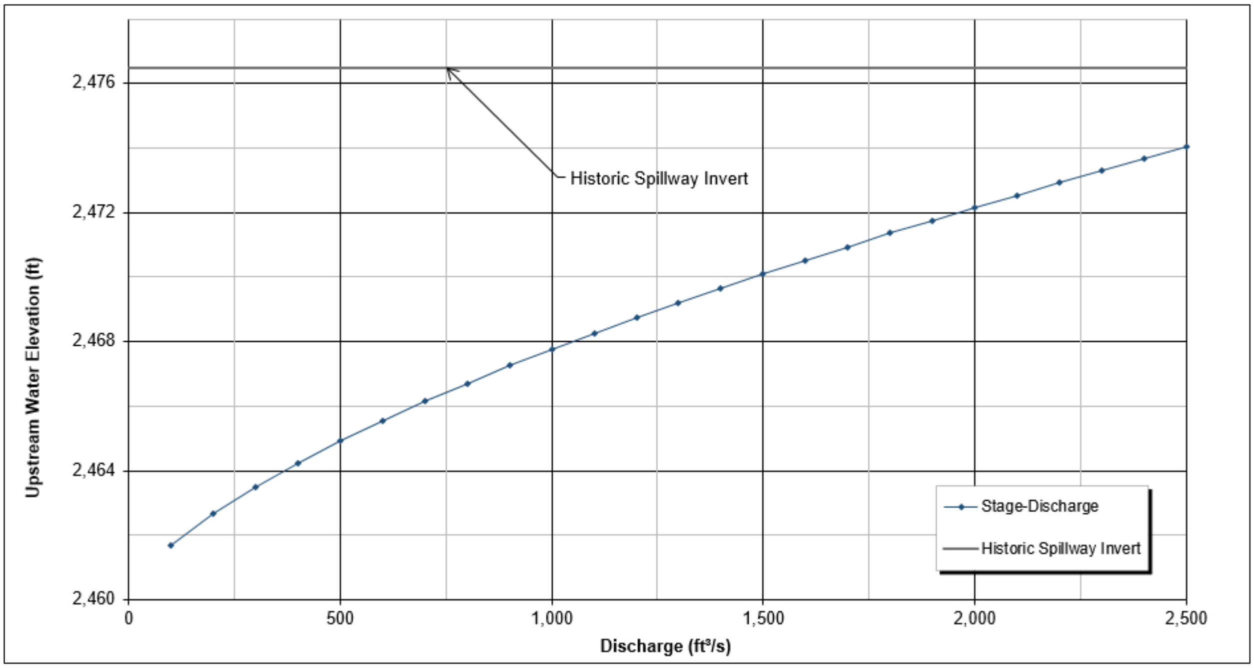



**Figure 3.4 Tunnel No. 1 Stage-Discharge Curve at Spillway Bay No. 3**

## 3.6 FINISHED GRADE HYDRAULIC CONDITIONS

### 3.6.1 FINAL RIVER CHANNEL WATER LEVELS

Channel characteristics and geometry of the Copco No. 2 final river channel presented on Drawing C3234 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 3.5 at the location of the current dam centerline. A sensitivity of the model uses Manning's n roughness of 0.03 and 0.06 to account for potential variability in roughness elements added to the channel to provide localized habitat. The results of the sensitivity analysis are included on Figure 3.5.

Dam removal construction activities in the vicinity of the final river channel may continue to occur 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 3.2 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 3.3 using the base model Manning's n value of 0.04.

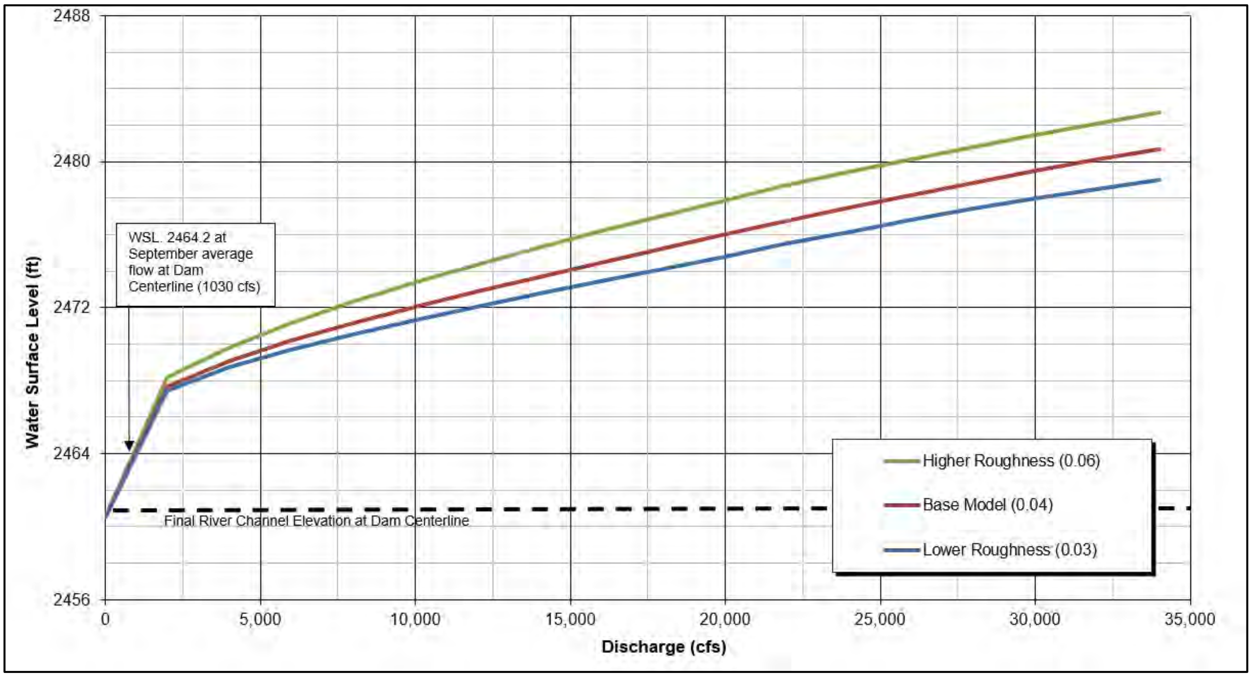



**Figure 3.5 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline**

**Table 3.2 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 | 1,900 | 2,100 | 5,700 | 6,300 | 7,400 | 8,400 |
| | 20% Probable Flood | 1,600 | 1,600 | 4,400 | 4,700 | 5,100 | 5,800 |
| | 50% Probable Flood | 1,300 | 1,300 | 3,600 | 3,600 | 3,900 | 4,200 |
| Mean Monthly Flow for Time Period | | 1,030 | 1,030 | 1,050 | 1,140 | 1,230 | 1,240 |
| Flow Condition | | Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline | | | | | |
| Statistical High Water (Flood Conditions) | 5% Probable Flood | 2,467.3 | 2,467.8 | 2,470.0 | 2,470.3 | 2,470.9 | 2,471.3 |
| | 20% Probable Flood | 2,466.3 | 2,466.3 | 2,469.3 | 2,469.5 | 2,469.7 | 2,470.1 |
| | 50% Probable Flood | 2,465.2 | 2,465.2 | 2,468.8 | 2,468.8 | 2,469.0 | 2,469.2 |
| Mean Monthly Flow for Time Period | | 2,464.2 | 2,464.2 | 2,464.2 | 2,464.3 | 2,464.6 | 2,465.0 |

**NOTES:**

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,460.6 FT.

**Table 3.3 Final River Channel Annual Steady-State Water Surface Levels at Existing Dam**

| 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 | 29,400 | 32,700 |
| | 5% Probable Flood | 18,200 | 24,300 |
| | 20% Probable Flood | 10,300 | 15,400 |
| | 50% Probable Flood | 7,100 | 11,200 |
| Annual Flow Duration 25% of Time Equaled or Exceeded | | 1,780 | 1,780 |
| Mean Annual Flow | | 1,710 | 1,710 |
| Annual Flow Duration 75% of Time Equaled or Exceeded | | 940 | 940 |
| **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 | 2,479.3 | 2,480.3 |
| | 5% Probable Flood | 2,475.3 | 2,477.6 |
| | 20% Probable Flood | 2,472.2 | 2,474.2 |
| | 50% Probable Flood | 2,470.7 | 2,472.6 |
| Annual Flow Duration 25% of Time Equaled or Exceeded | | 2,466.7 | 2,466.9 |
| Mean Annual Flow | | 2,466.4 | 2,466.7 |
| Annual Flow Duration 75% of Time Equaled or Exceeded | | 2,463.8 | 2,463.9 |

**NOTES:**

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,460.6 FT.

### 3.6.2 BACKFILLED POWERHOUSE AND TAILRACE CROSS-SECTION HYDRAULICS

The water surface elevations and velocities at the backfilled Copco No. 2 Powerhouse location (as shown on Drawing C3420) are calculated using a hydrodynamic model. The model uses HEC-RAS 2D with a Manning's n of 0.04 to evaluate a range of hydrologic conditions. The model considers the 1% probable annual flood flow of 32,700 cfs, the average August flow of 980 cfs, and the lowest flow possible for the river (based on required biological flows) of 850 cfs.

The resulting water surface elevations and depth-averaged velocities through the channel are provided in Table 3.4 and are shown on Figure 3.6 and Figure 3.7 for the 1% probable annual flood flow condition and the average August flow condition, respectively.

**Table 3.4 Water Surface Elevation and Velocity at Backfilled Powerhouse Location**

| Flow Case | Flow (cfs) | Water Surface El. (ft) | Maximum Depth-Averaged Velocity in Backwatered Area (ft/s) |
|---|---|---|---|
| 1% Probable Annual Flood Flow | 32,700 | 2,345.3 | 2.8 |
| August Average Flow | 980 | 2,332.0 | 0.2 |
| Low Flow | 850 | 2331.8 | 0.2 |

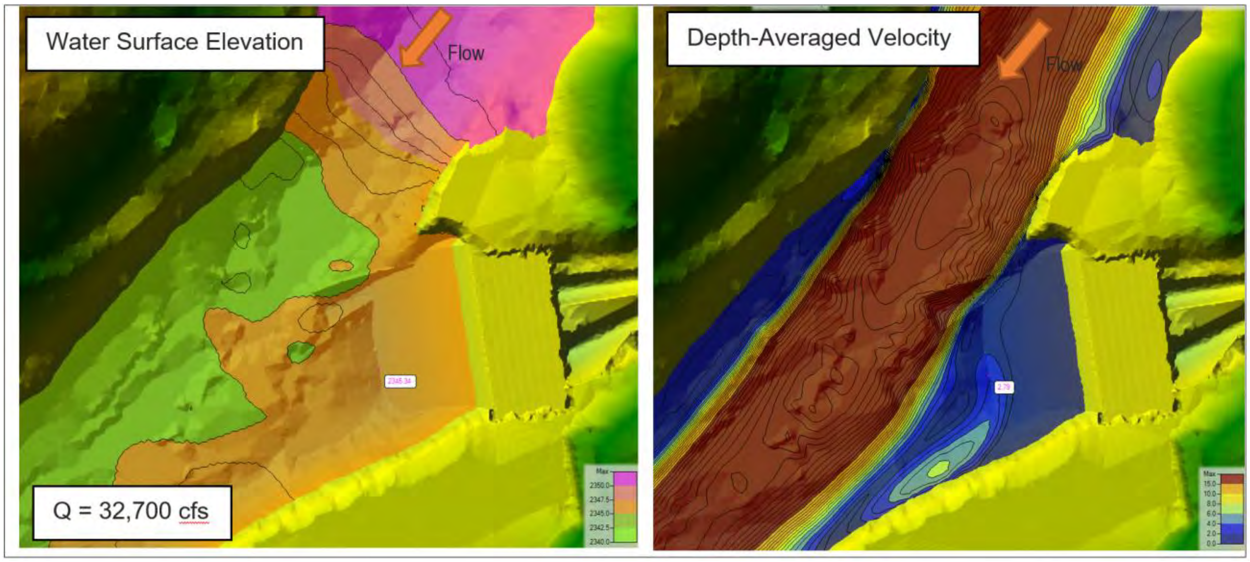



**Figure 3.6 Water Surface Elevation and Velocity at Backfilled Powerhouse Location for the 1% Probable Flood Flow Condition**

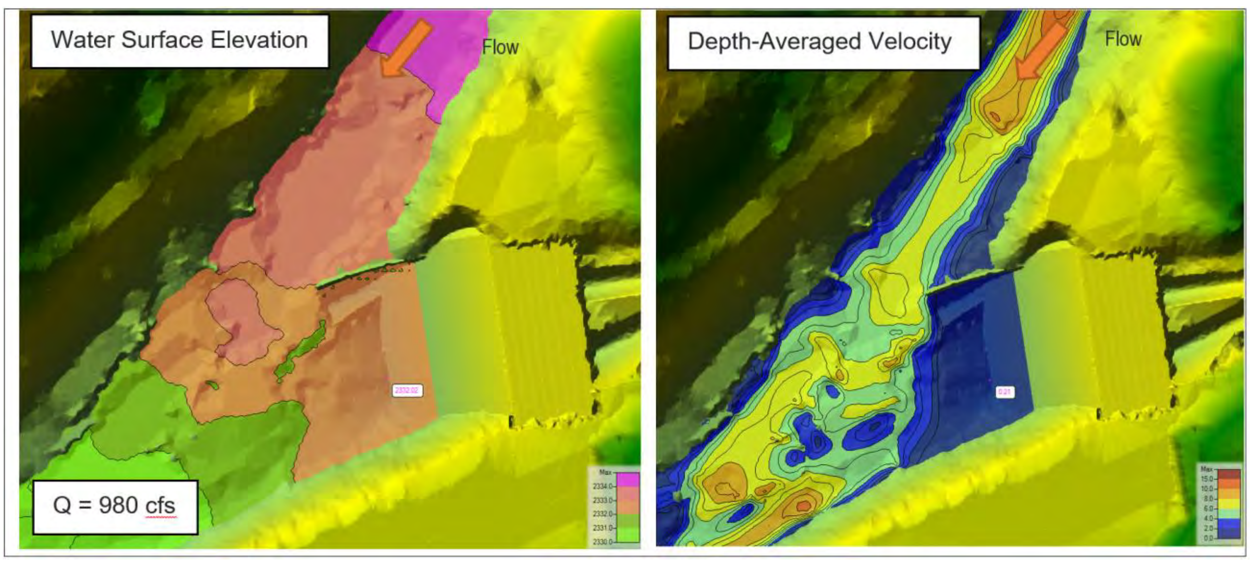



**Figure 3.7 Water Surface Elevation and Velocity at Backfilled Powerhouse Location for the Average August Flow Condition**

The model indicates that the average river level at the tailrace during the lower flow period when the tailrace is anticipated to be backfilled is at approximate elevation of 2,332.0 ft, as shown on Drawing C3420, while the maximum velocity of flow adjacent to the backfill slope is approximately equal to 3 ft/s.

The north toe and edge of the tailrace backfill that abuts the subgrade of the removed right wing tailrace wall is an interface boundary between the main channel flow and the flow in the sheltered tailrace backfill area. The velocity from the main channel decreases as the channel depth increases in the tailrace area and as it contacts the eddy flow that is anticipated to develop in the tailrace backfill area. The velocity contours of the tailrace area are shown on Figure 3.8. Based on the depth-averaged velocity contours, the edge of the fill in the interface area may be locally subjected to flows of 10 to 15 ft/s. To be prudent, it was determined to consider these interface velocities in the backfill design over the localized area. The erosion protection for this localized area, detailed in Section 2.6.4, is shown to expand well into the area of the tailrace backfill where the velocities are consistently within the maximum values presented in Table 3.4. The typical toe elevation of the backfill is approximately 2325 ft, so given the 1% annual probable flood flow elevation of 2345.3 ft provided in Table 3.4, the anticipated flow depth at the tailrace backfill is 20 ft.

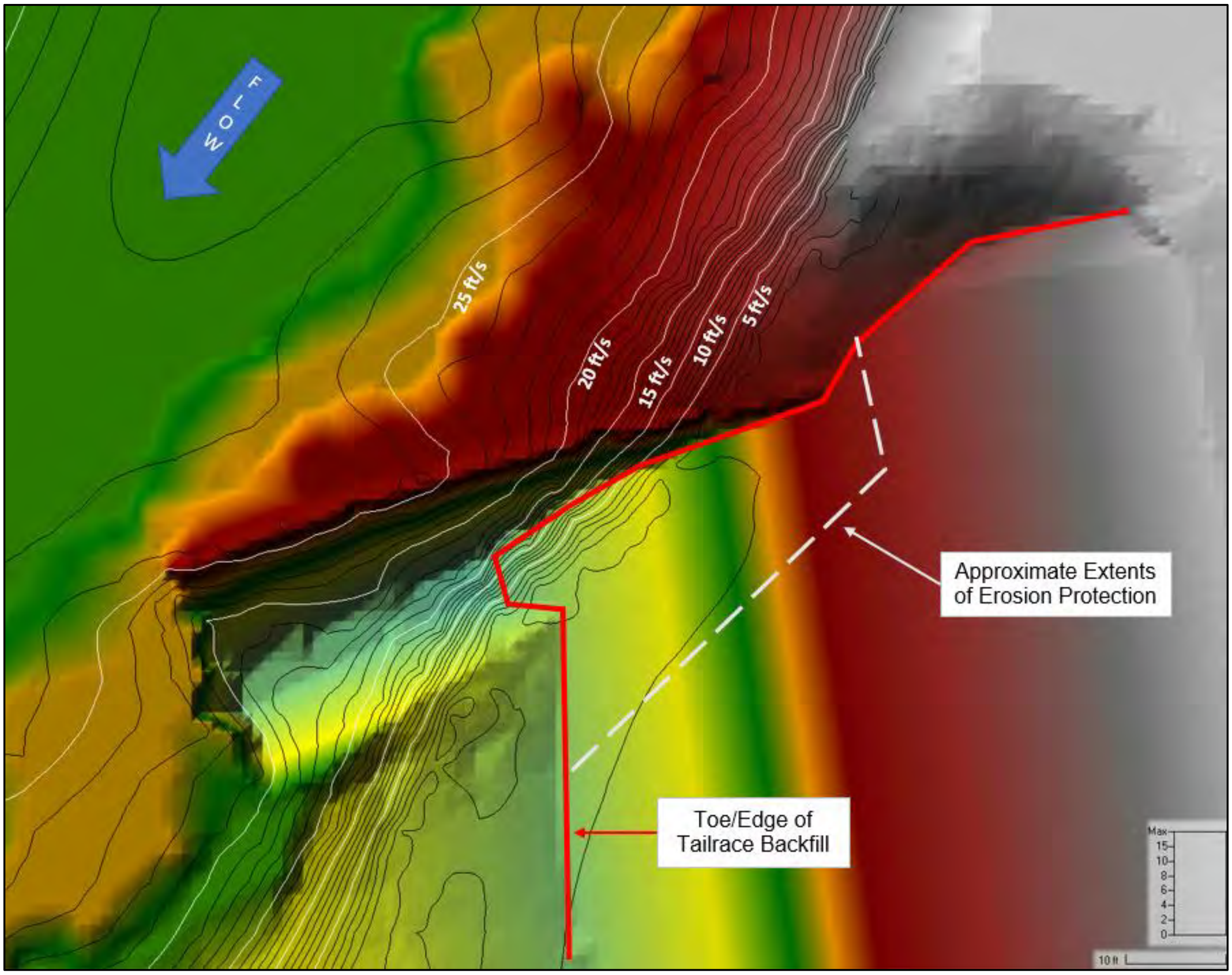



**Figure 3.8 Velocity Contours for the 1% Probable Flood Flow Condition at the Tailrace Backfill Area**

# 4.0 GEOTECHNICAL

## 4.1 DAM REMOVAL

Excavation will be required at the Copco No. 2 Diversion Dam to remove the concrete dam from the river channel. The left and right bank wing walls are the lateral extents of the concrete that need to be removed. Two temporary excavation slopes are required to sub-excavate the walls prior to the final river channel being backfilled, as shown on Drawing C3221. The slopes will be 1.5H:1V and will have a maximum height of 43 ft.

There is no subsurface site investigation data available for the site, only a historical geological interpretation of the native ground across the dam access that is included with the historic drawings. The assessment of the slope stability uses photographs of the dam construction to verify the type of material present and provide evidence of steep excavations. Recent site visits provide evidence that the surrounding material on the banks match the material shown in the geological interpretation and the historic photographs.

‘Big Rocks and Gravel’ and ‘Boulders and Silt’ are present on the right and left banks, respectively, according to the geological section shown in Historic Drawing D-3722. Historic photographs show that the excavations that occurred to construct the dam exposed poorly graded alluvium/colluvium with a majority component of cobbles and boulders. An example photograph is shown on Figure 4.1. The photographs also show that large historical sub-vertical excavations were possible at the time of construction.

The slope stability Limit Equilibrium Analysis (LEA) for the proposed excavation uses GeoStudio’s 2-Dimensional (2D) Limit-Equilibrium program Slope/W (GeoStudio, 2020). The material parameters are developed using the historic data detailed above. Based on the information available, a Leps low density poorly graded rockfill strength function is conservatively adopted for the abutment material. The excavation is assumed to be dry above, and partially saturated below, the normal operating reservoir level of 2,486.5 ft. The sensitivity to the degree of saturation in the excavation slope is determined by varying pore water pressures in the slope below El. 2,486.5 ft using an Ru coefficient until the target Factor of Safety (FOS) of 1.3 for a temporary excavation is achieved. The model indicates that if the phreatic surface is approximately 2.2 ft perpendicular from the cut face, or greater, then the excavation slope of 1.5H:1V is acceptable. In practice the phreatic surface will be dependent on how quickly the abutment material drains after the reservoir is drawn down. An engineer will inspect the water conditions in the actual excavation and determine if the minimum drawdown has occurred. Results of the slope stability LEA are shown in Figure 4.2.

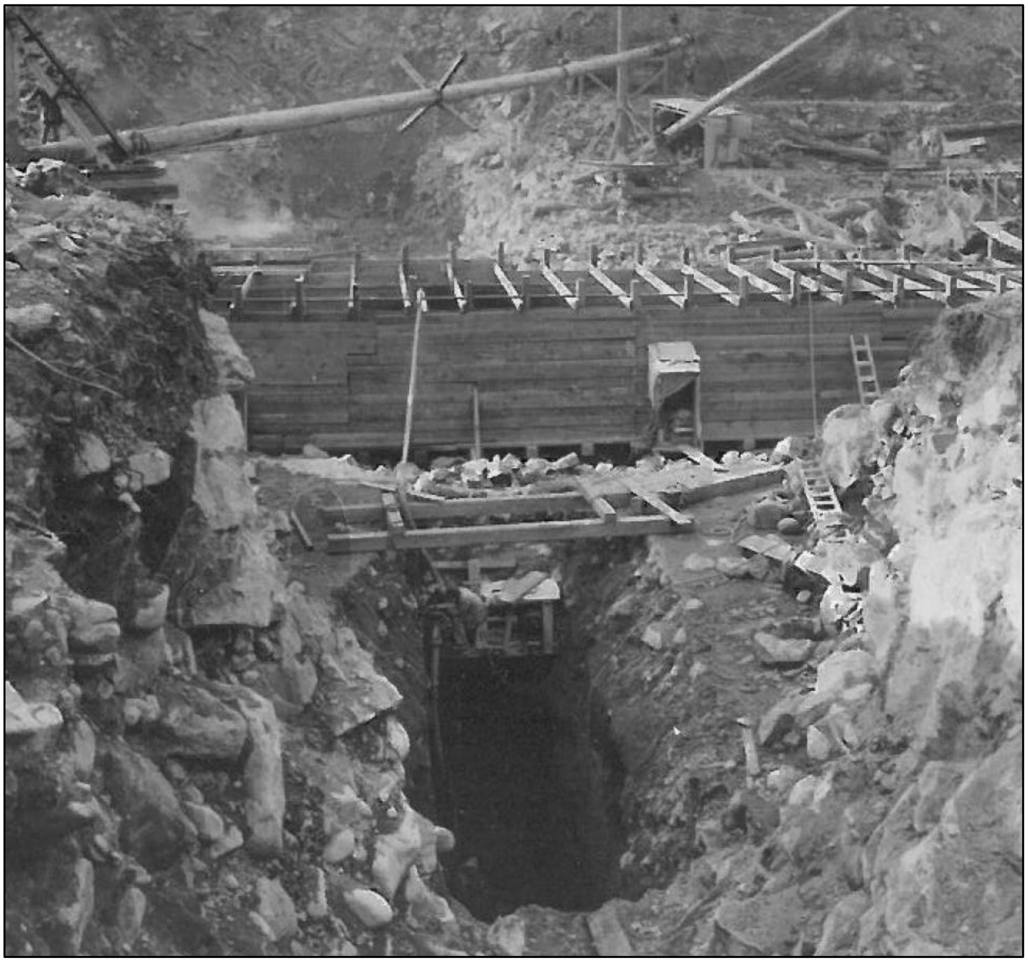


Figure 4.1 Diversion Dam Excavation Looking South from Right Bank

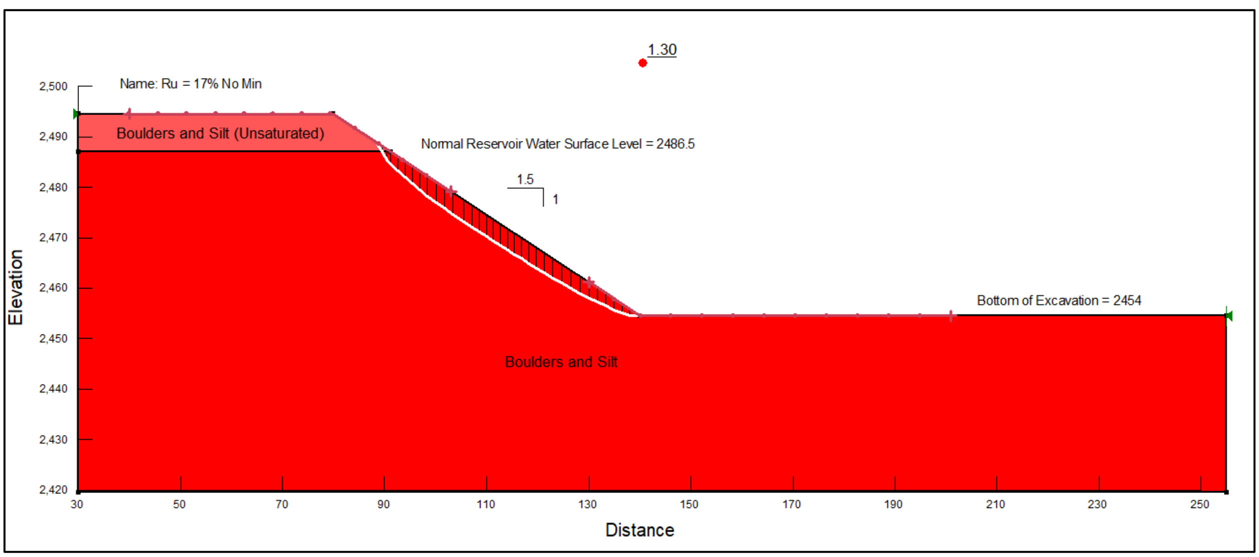



Figure 4.2 Diversion Dam Excavation Slope Stability Results

## 4.2 INTAKE STRUCTURE BACKFILL

The intake structure backfill will comprise concrete rubble that is placed to a maximum temporary slope of 1.5H:1V, prior to backfilling to the final channel grade. Concrete rubble is anticipated to have an internal friction angle similar to angular rockfill. For rockfill comprising sound rock, a reasonable friction angle can be as high as 45° (WSDOT, 2019). A temporary slope of 1.5H:1V is therefore acceptable. The concrete slope will be covered by the final channel backfill and therefore does not need to be considered for the long-term.

## 4.3 BORROW SITES

Borrow sites are required at the wood-stave penstock and the powerhouse area to provide General Fill (E9) backfilling material.

The wood-stave borrow site is shown on Drawing C3300 and is located within an existing borrow source location. The existing slopes within the targeted excavation area are approximately 1.3H:1V with slopes above and adjacent to the area as steep as 1.1H:1V. Recent site investigations show the targeted borrow material comprises silty gravel. The slopes are vegetated with no evidence of large-scale instability or unravelling. A photograph of the targeted borrow source is provided below on Figure 4.3.

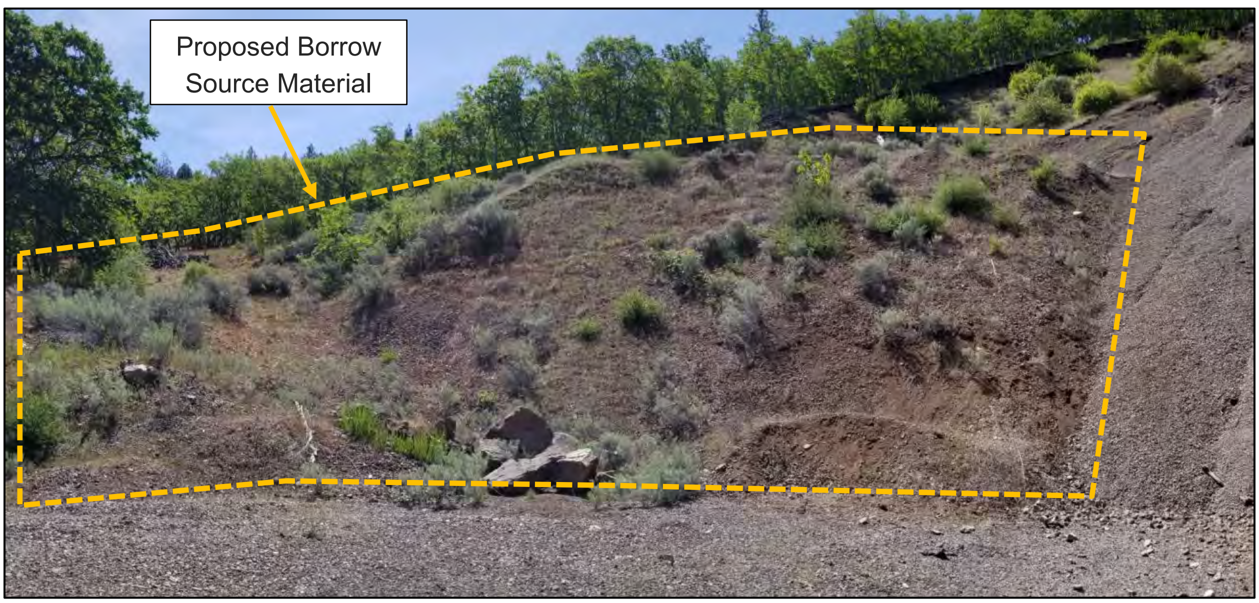



**Figure 4.3 Wood-stave Borrow Site**

A representative strength model for the material within the borrow site is determined by completing a LEA back analysis using Geostudio's 2D Limit Equilibrium program Slope/w (Geostudio, 2020). The Leps low density poorly graded strength function (Leps, 1970) is selected because of the gravel content of the borrow site and when applied to an assumed dry 1.3H:1V slope it produces a Factor of Safety (FOS) of approximately 1.25, which is appropriate given the observed condition of the slope. A 50 ft high 1.5H:1V cut slope representing the borrow site excavation is modelled using the same strength function and assumed dry conditions, and results in a FOS greater than 1.5, which is acceptable for a permanent excavation. Results of the slope stability LEA are shown on Figure 4.4. The excavated slopes will be monitored during construction and may be modified as required.

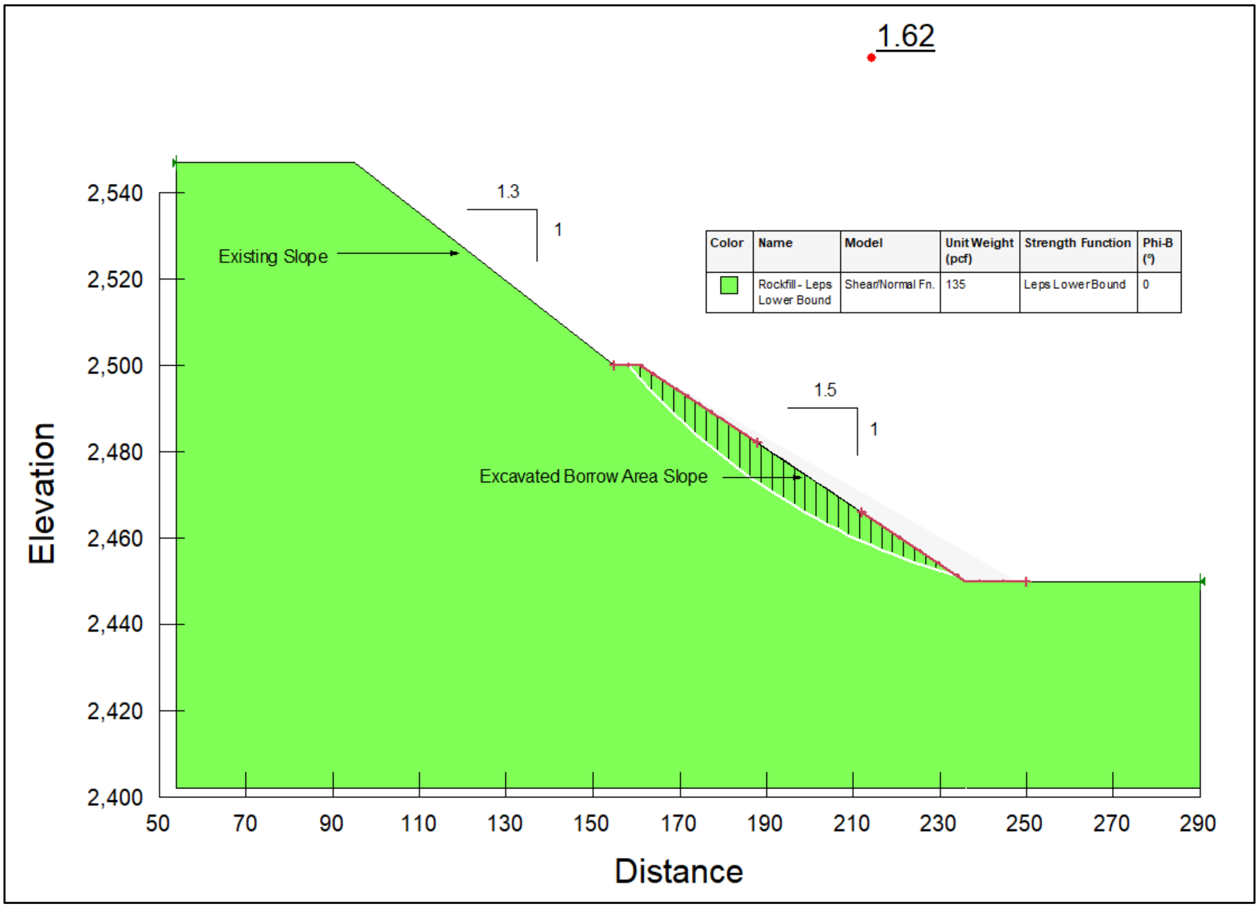



**Figure 4.4 Wood-stave Borrow Area Slope Stability Analysis Results**

The Copco No. 2 powerhouse borrow site is shown on Drawing C3332 and has limited geotechnical information. The existing slopes in the borrow area are approximately 2.5H:1V and it is anticipated based on historic photographs that the overburden comprises the excavated alluvial material from the powerhouse excavation. A conservative maximum excavation slope of approximately 4.5H:1V is selected for the borrow site and will be assessed by an engineer to determine if the encountered conditions support the proposed excavation slope.

## 4.4 TUNNEL #2 OUTLET PORTAL BACKFILL

The Tunnel #2 outlet portal will be barricaded with a General Fill (E9b) and Concrete Rubble (CR2) backfill as shown on drawing C3350. The backfill has a slope of 2H:1V that extends to the existing ground that has an approximate slope of 3H:1V. The slope stability of the backfill is analyzed with a 3-Dimensional (3D) LEA using Rocscience's program Slide3 (Rocscience, 2020) and applying the GLE (Morgenstern-Price) method. A 3D approach is used over a 2D approach due to the oblique geometry of the backfill, natural slope and the backfilled depression left from the removed penstocks. The backfill of the penstock depression adjacent to the tunnel portal backfill can be either General Fill (E9) or Concrete Rubble (CR2) as shown on drawing C3334. Concrete Rubble (CR2) however, is not considered in either the tunnel portal backfill or the penstock depression backfill to be conservative. The model assumes the backfill will be dry because General Fill (E9b) has less than 10% fines and therefore is assumed to be free draining. The surrounding slopes are also graded to prevent ponding water.

The existing ground immediately adjacent to the tunnel portal has overburden slopes of approximately 1.4H:1V to 1.5H:1V and appears to comprise rockfill and granular material. The existing ground is therefore represented by the Leps low density poorly graded strength function (Leps, 1970) in the 3D LEA. Bedrock is developed in the model by extending the exposed sub-vertical bedrock surfaces into the ground. A conservative horizontal bedrock overburden contact is modelled at an assumed elevation of 2300 ft to prevent bedrock from affecting the base of the slip surfaces. Bedrock is assumed to be high strength that slip surfaces cannot pass through to prevent failures developing in the competent sub-vertical bedrock behind the proposed backfill. The material parameters used in the LEA are summarized in Table 4.1.

**Table 4.1 Material Properties for Tunnel #2 Backfill Slope Stability LEA**

| Material | Unit Weight (pcf) | Effective Friction Angle (°) | Effective Cohesion (psf) |
|---|---|---|---|
| General Fill (E9b) | 135 | 36 | 0 |
| General Fill (E9) | 125 | 32 | 0 |
| Existing Ground | 127.5 | Leps Lower Density Strength Model | |
| Bedrock | 154 | High Strength | |

The slope stability LEA assess two loading conditions: 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. The design seismic loading is taken from STID-5 (PacifiCorp, 2015b), and is equal to the Copco No. 1 MCE of 0.26 g, and a maximum magnitude 7.5 earthquake is assumed based on the maximum estimated earthquake magnitudes in faults nearby as presented in the STID-5 (PacifiCorp, 2015b). Seismic displacements are approximated by the semi-empirical method, developed by Makdisi and Seed (1978).

The static FOS for the tunnel portal fill is 1.51, for a slip surface with a minimum depth of 5 ft as shown on Figure 4.6. The slip volume is approximately 220 CY and stays within the General Fill (E9b) material as shown in the cross section on Figure 4.6. Minor surficial slip surfaces with no minimum depth have a FOS of 1.45. The yield acceleration for the tunnel portal fill is determined to be 0.198g for a for a slip surface with a minimum depth of 5 ft. Displacement estimates predict the movement to be in the order of inches.

The analyses indicate the design slope of 2H:1V satisfies the requirements of the acceptance criteria. Smaller-scale slips are slightly below the acceptance criteria; however the volumes of the slip are small enough (<15 CY) that they can be disregarded.

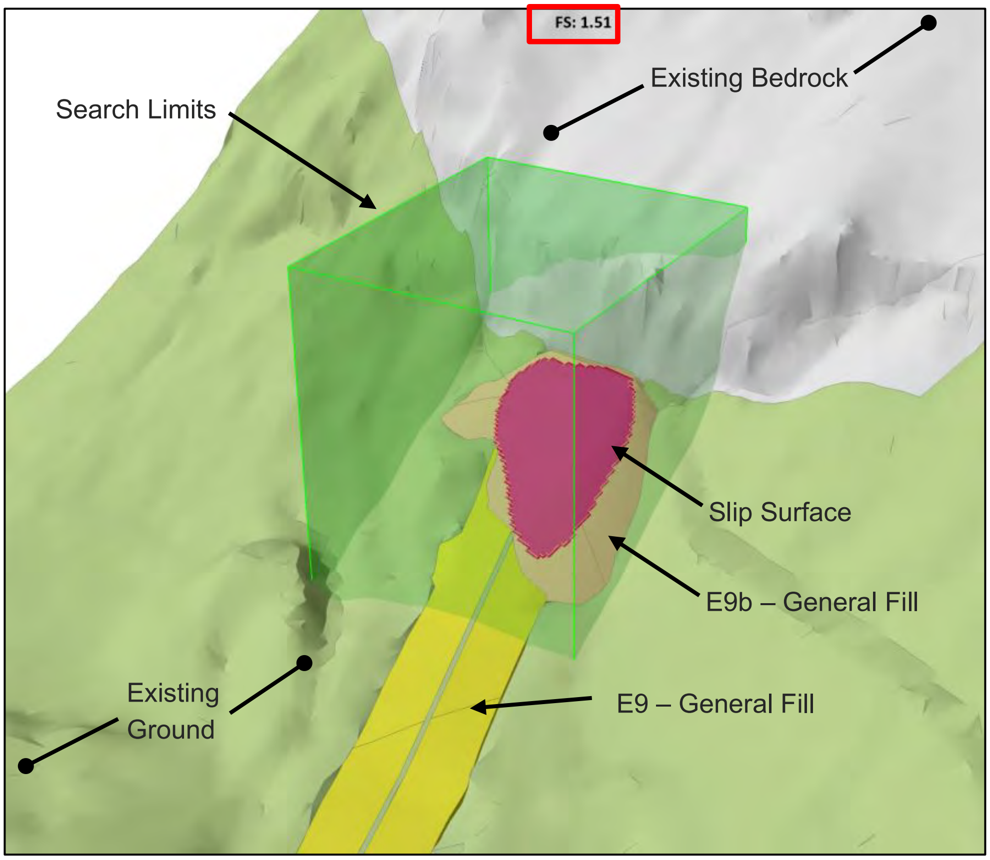
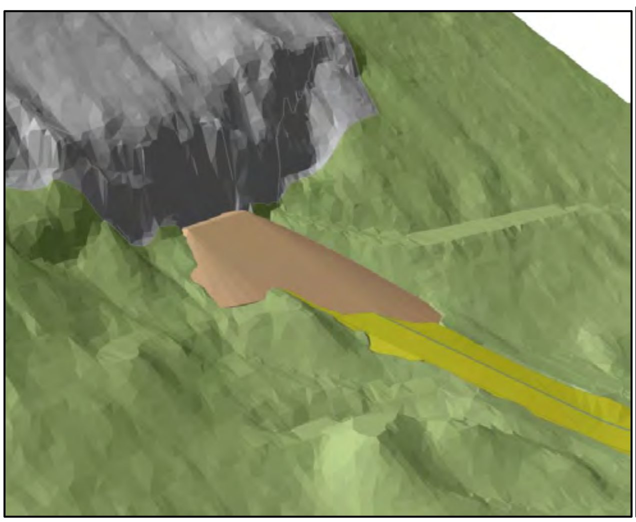



**Figure 4.5 Tunnel #2 Portal Backfill Slope Stability LEA Results**



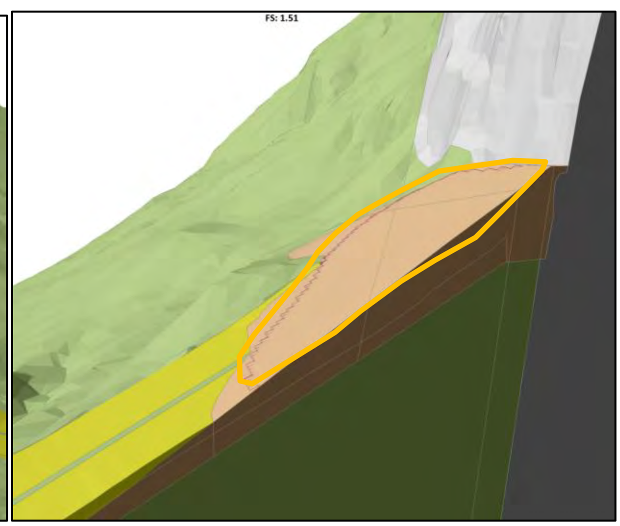



**Figure 4.6 Oblique View of Backfill with no Slip Surface (Right) Cross Section of Backfill with Slip Surface Highlighted (Left)**

## 4.5 POWERHOUSE TAILRACE BACKFILL

The Powerhouse Tailrace will be a disposal site for demolished concrete from the powerhouse, penstocks and tailrace. It will be backfilled with Concrete Rubble (CR2), or General Fill (E9a) if not enough concrete rubble is produced, to form a 2.5H:1V slope into the river. The design includes a minimum thickness of concrete rubble at the face of the fill to provide the required stability as determined by the slope stability LEA detailed in this section. The disposal site will be capped with a layer of Bedding (E8) material that protects the backfill from the river flows and covers the exposed concrete. The backfill 2D LEA uses Geostudio's Limit Equilibrium program Slope/w (Geostudio, 2020) and applies the GLE (Morgenstern-Price) method. Material parameters used in the analysis are summarized below in Table 4.2.

**Table 4.2 Material Parameters for Tailrace Backfill Slope Stability LEA**

| Material | Unit Weight (pcf) | Effective Friction Angle (°) | Effective Cohesion (psf) | Concrete Interface Friction Factor Tanδ (Sensitivity Check) |
|---|---|---|---|---|
| Concrete Rubble (CR2) | 130 | 45 | 0 | 0.55 |
| General Fill (E9a) | 115 | 28 | 0 | 0.45 |
| Bedding (E8) | 125 | 36 | 0 | 0.55 |
| General Fill (E9) | 125 | 32 | 0 | - |
| Cast-in-Place Concrete | 150 | 45 | 1000 | - |

The tailrace backfill slope stability LEA assesses three loading conditions: static long-term, rapid drawdown, and yield acceleration (ky) determination for approximating seismic displacement. The phreatic surface for the static and yield acceleration conditions ranges between the 1% probable flood elevation of 2345.3 ft and the assumed lowest possible river level elevation of 2331.8 ft (based on required biological flows) to find the critical water level. The drawdown analysis assumes that the Concrete Rubble (CR2) and General Fill (E9a) remain saturated during a rapid drawdown in river level from the 1% probable flood, to the July-September average flow elevation of 2332.0 ft. Use of the average July-September flow elevation is conservative since the 1% probable flood is anticipated to occur during the spring months. The concrete rubble is conservatively assumed not to drain in the drawdown condition to account for the General Fill (E9) material that will infill the interstitial voids in the concrete rubble, as required in the technical specifications. The acceptance criteria require a FOS of 1.5 for static long-term stability, a FOS of 1.2 for rapid drawdown stability, and FOS of 1.0 for yield acceleration determination without strength reduction. The design seismic loading is taken from STID-5 (PacifiCorp, 2015b), and is equal to the Copco No. 1 MCE of 0.26 g, and a maximum magnitude 7.5 earthquake is assumed based on the maximum estimated earthquake magnitudes in faults nearby as presented in the STID-5 (PacifiCorp, 2015b). Seismic displacements are approximated by a semi-empirical method, developed by Makdisi and Seed (1978).

A sensitivity analysis assesses the effect of a possible low strength interface between the cast-in-place concrete and the backfill material. Frictional factors for the interface of dissimilar materials are assigned based on values found in literature (NAVFAC, 1986), as detailed in Table 4.2, and are applied to the Static Long-term Loading Condition. The acceptance criteria require a FOS of 1.3 for sensitivity analyses.

The interface between Concrete Rubble (CR2) and General Fill (E9b) in the analyzed model determines the minimum Concrete Rubble backfill requirement that meets the acceptance criteria. An increase in concrete rubble beyond what is shown in the model would only increase the FOS. The minimum thicknesses of Concrete Rubble (CR2) as determined by LEA is 8 ft perpendicular to the slope below El. 2335 ft and 3 ft perpendicular to the slope above El. 2335 ft. The static long-term FOS is 1.62, as shown on Figure 4.7, the rapid drawdown FOS is 1.34, and the sensitivity check static long-term FOS is 1.39. The yield acceleration for the tailrace backfill is 0.21g. Displacement estimates predict the movement to be in the order of inches.

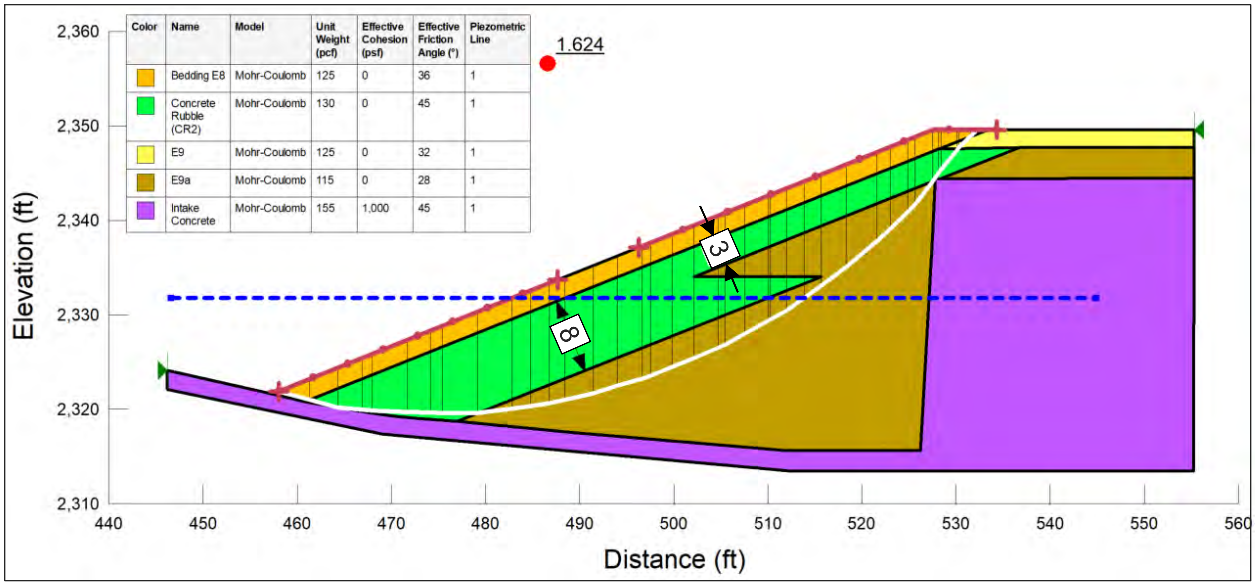



**Figure 4.7 Tailrace Backfill LEA Stability Results**

## 4.6 LEFT BANK ACCESS ROAD

The Left Bank Access Road is an option for the Project Company to construct, but is not required, and is shown on Drawings C3530 through C3534. It will start close to the wood-stave penstock and will follow an existing access road to the left bank of the diversion dam. The intent of the road is to provide access to the left bank for some equipment that will help facilitate the dam removal. It will not be used as a haul road, nor be heavily trafficked. The road is approximately 4,000 ft long and will require minor cuts and fills to ensure the road has a minimum width of 8 ft, but will target a road width of 12 ft where possible. The proposed road fill and cuts slopes for a typical section are 2H:1V and 1.5H:1V, respectively. For pinch point areas where the road is narrowed to 8 ft wide, the fill and cut slopes may be steepened to a maximum of 1.25H:1V and 1H:1V, respectively. The steeper cuts and fills are anticipated to occur only over short distances. The proposed slopes are considered preliminary and a full assessment must be completed by an engineer prior to construction if the Project Company chooses to construct the road.

The Copco No. 2 Dam Left Bank Access Road terrain hazard assessment identifies potential hazards affecting road users. The Study Area includes the upslope and downslope areas of natural terrain. No obvious large-sized areas of recent natural slope instability in the vicinity of the road are identified. A semi-circle-shaped convex slope break is identified down slope from the road alignment in the vicinity of STA. 10+00. This feature is possibly the back scarp of a relict landslide. If this feature is a landslide it is likely very old since there is no obvious accumulation of debris down slope from the convex slope break. The hazard assessment identifies relic rock fall zones above the road from STA 31+00 to the end of the road. The road, however, only crosses the path of the relic rock fall between STA. 31+00 to STA 35+00. The terrain hazard map is attached to this appendix, while the full terrain hazard assessment is presented in the Geotechnical Data Report.

The preliminary site reconnaissance of the road conducted by a KP geotechnical engineer further assesses the geohazards and determines if the proposed cuts and excavations are considered viable. The slopes above and below the proposed access road are observed to be gently vegetated. The surficial material consists of organic material overlying sandy material with some clay. Localized bedrock outcrops are present along the proposed alignment of the road. Preliminary results indicate the proposed cuts are minor and are not anticipated to affect the global stability of the road or slopes. If the Project Company opts to construct the road additional work is required to better classify the overburden material and determine if localized slope protection is required along the road.

No additional geohazards beyond what is identified in the desktop study are observed along the road alignment. The relic rock fall zones are identified where coarse talus blankets (boulder sized) are present below the cliff band above the road between STA 31+00 and the end of the road. The talus material is interpreted to provide a natural barrier for potential rock falls that will decrease the potential of rock falls reaching the road. The talus material is not present in the slope above the road between approximately STA. 38+00 to 39+00. Along this interval, given the projected traffic volume is low, the rock fall potential may be addressed by means of traffic controls, such as no stopping zones, and monitoring.

## 4.7 LEFT ABUTMENT CLIFF BAND

A steep cliff band overhangs the left abutment at the diversion dam. A timber crib is located along the cliff band which indicates the area has previously experienced rock fall hazards. It is understood that the crib was originally constructed to protect the crusher during construction of the dam. A KP geotechnical engineer has conducted a preliminary site reconnaissance of the crib to assess the area for geohazards related to worker safety. Figure 4.8 below shows the cliff band that extends above the intake structure at the Copco No. 2 diversion dam. The cliff band comprises four sub-cliffs, with the timber crib located below the upper cliff.

In general, it is noted that the cliff consists of medium strong to very strong, curved columnar structures. The columns seem to be well-defined and ordered, except for the upper part where these are disordered and distorted, as shown in Figure 4.9. The cliff band is vegetated with moss, grass, and small trees. It is evident that the columns have experienced raveling, indicating rock fall potential. The area behind the intake was inspected, and no recent signs of substantial rockfall were observed. Some minor rock fall hazard mitigation measures are expected to be required, such as barriers or safe work setback distances.

**Figure 4.8 Cliff Band and Timber Crib above Intake Structure**

**Figure 4.9 Upper Cliff Basalt Columns**

**Attachments:**

1 – Copco No. 2 Facility Simulated Drawdown

2 – Copco No. 2 Geohazard Terrain Maps

**References:**

American Concrete Institute (ACI) Committee, 1920. Standard Building Regulations for the Use of Reinforced Concrete. Journal Proceedings. Standard Specifications No. 23. Vol. 16. Issue 2. Feb 1. Pgs. 283-302.

American Concrete Institute (ACI) Committee 318, 1972. Building Code Requirements for Reinforced Concrete (ACI 318-71). Journal Proceedings. Volume 68. Issue 8. August 1. Pgs. 553-559.

American Concrete Institute (ACI) Committee 207, 2006. Guide to Mass Concrete, ACI 207.1R.

American Concrete Institute (ACI) Committee 318, 2014. Building Code Requirements for Structural Concrete, ACI 318.14, Chapter 11.

CRSI, 2001. Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures. Concrete Reinforcing Steel Institute. Engineering Data Report Number 48. Ref: H N H 0301/19M.

Curtis and Lum, 2008. Estimated Shear Strength of Shear Keys in Concrete Dams.

Du, Chongjiang, 2017. Design of Tunnel Plugs for Hydropower Projects. Hydro Review. Volume 36. Issue 5. June 1. https://www.hydroreview.com/2017/06/01/design-of-tunnel-plugs-for-hydropower-projects/#gref

Joint Committee on Standard Specifications for Concrete and Reinforced Concrete., American Society for Testing Materials. 1940. Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete: submitting recommended practice and standard specifications for concrete and reinforced concrete. Detroit: American Concrete Institute.

Kleinfelder, 2009. Geoseismic Evaluation, Copco No. 1 Dam, Siskiyou County. California, June 19, 2009

Knight Piésold (KP), 2022a. Existing Conditions Assessment Report Rev F. VA103-640/1-1. Vancouver, April 11, 2022.

Knight Piésold (KP), 2022b. Geotechnical Data Report Rev F. VA103-640/1-2. Vancouver, April 11, 2022.

Lang, B., 1999. Permanent Sealing of Tunnels to Retain Tailings or Acid Rock Drainage. IMWA Proceedings. International Mine Water Association.

Makdisi, F.I. and Seed, H.B., 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations. Journal of the Geotechnical Engineering Division. American Society of Civil Engineers. Vol 104, No. GT7. pp 849-867.

Naval Facilities Engineering Command (NAVFAC) 1986. Foundations and Earth Structures. Alexandria, Virginia, September 1986, Ref No. Design Manual 7.02.

Sorenson, A, 2014. Preliminary Evaluation of Feasibility for Water Impounding Concrete Bulkhead, Red and Bonita Mine, San Juan Country Colorado. Colorado Division of Reclamation, Mining and Safety. Ref No. 1287701.

PacifiCorp, 1925. Historic Photographs of Copco No. 2.

PacifiCorp, 2015a. Copco 1 Development – Supporting Technical Information Document (STID), Section 1 PFMA Report, Revision 2, 30 April 2015, Ref No. FERC P-2082-CA

PacifiCorp, 2015b. Copco 1 Development – Supporting Technical Information Document (STID), Section 5 Geology and Seismicty, Revision 1, 30 April 2015, Ref No. FERC P-2082-CA

USACE 1992. Design and construction of grouted riprap. Engineer Technical Letter No. 1110-2-334. U.S. Army Corps of Engineers, Washington DC.

USACE 1994. Hydraulic Design of Flood Control Channels. Engineering Manual EM 111-2-1601. Army Corps of Engineers, Washington DC.

USBR, 1984. Design of Small Dams.

USBR, 2014. Design Standards No. 14, Appurtenant Structures for Dams (Spillways and Outlet Works) Design Standard, Chapter 3: General Spillway Design Considerations.

USBR, 2015. Best Practices in Dam and Levee Safety Risk Analysis, Version 4.0, July 2015.

USBR, 2016. Technical Report SRH-2016-38, Dam Removal Analysis Guidelines for Sediment - Version 1, September 2016.

Washington State Department of Transportation, 2019. Geotechnical Design Manual Ref. No. M 46-03.12, Tumwater, WA. July 2019.

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

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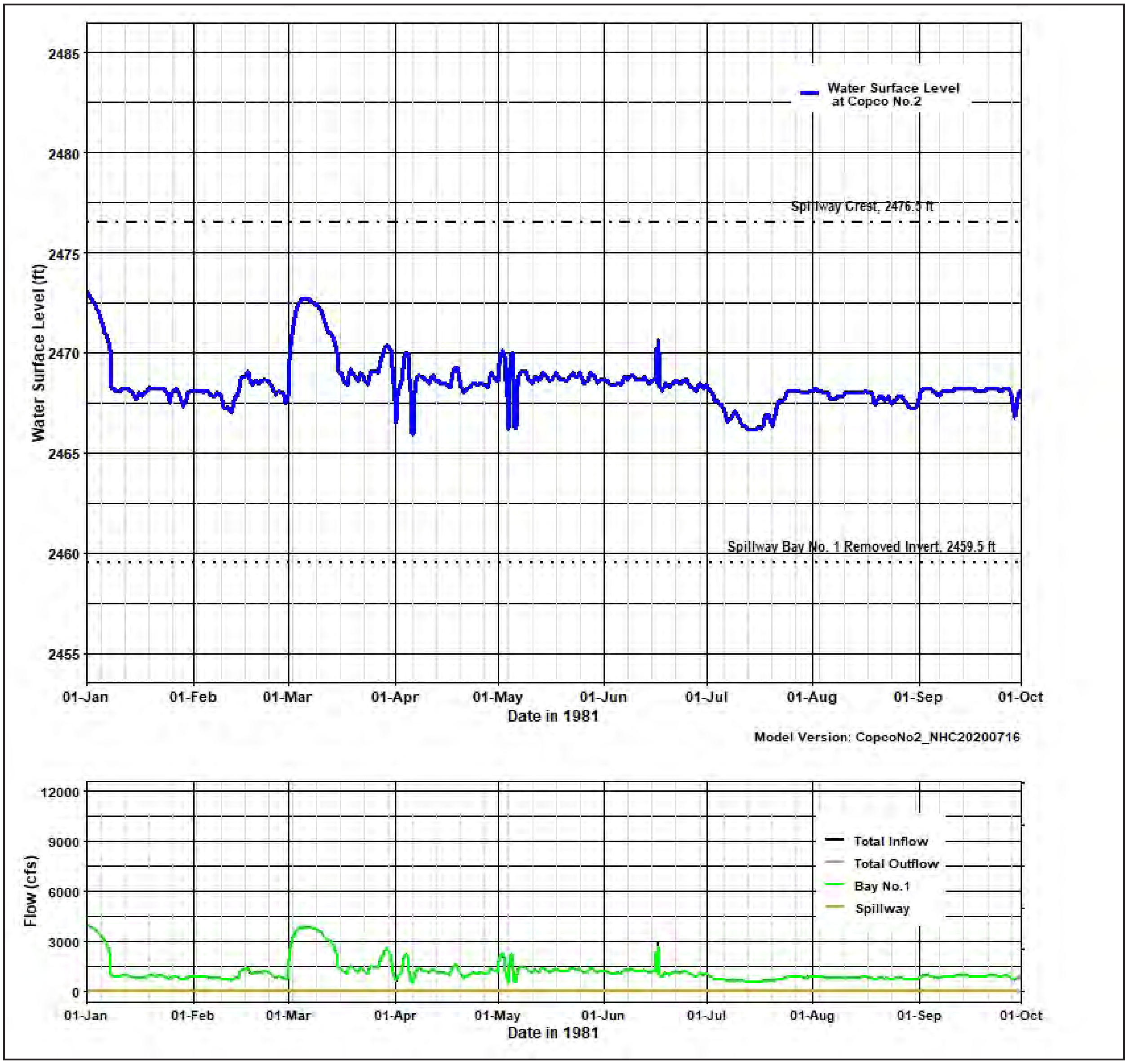


**Figure 1 -** COPCO No.2 Facility Simulated Drawdown - Year 1981

July 29, 2020

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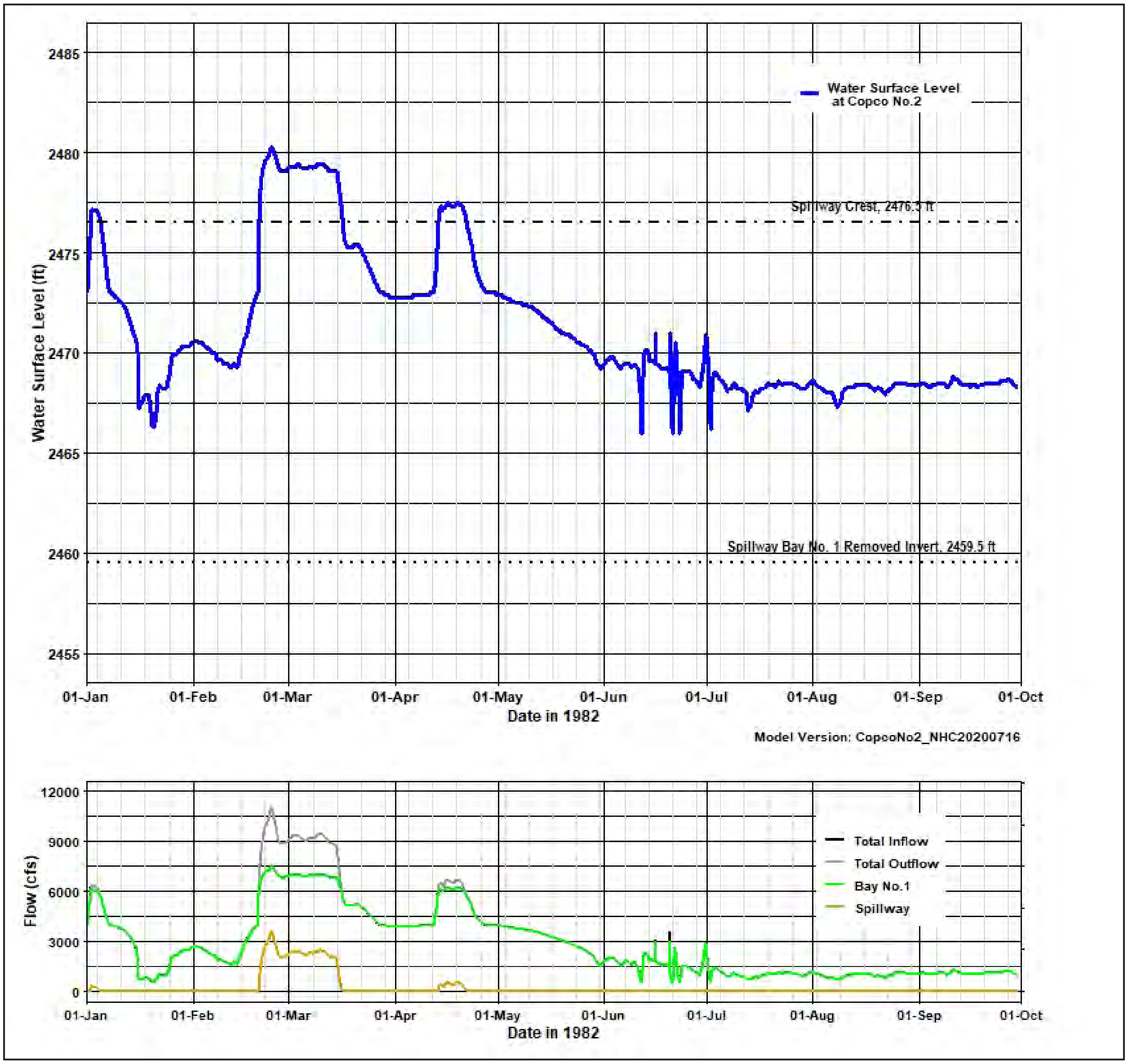


**Figure 2 -** COPCO No.2 Facility Simulated Drawdown - Year 1982

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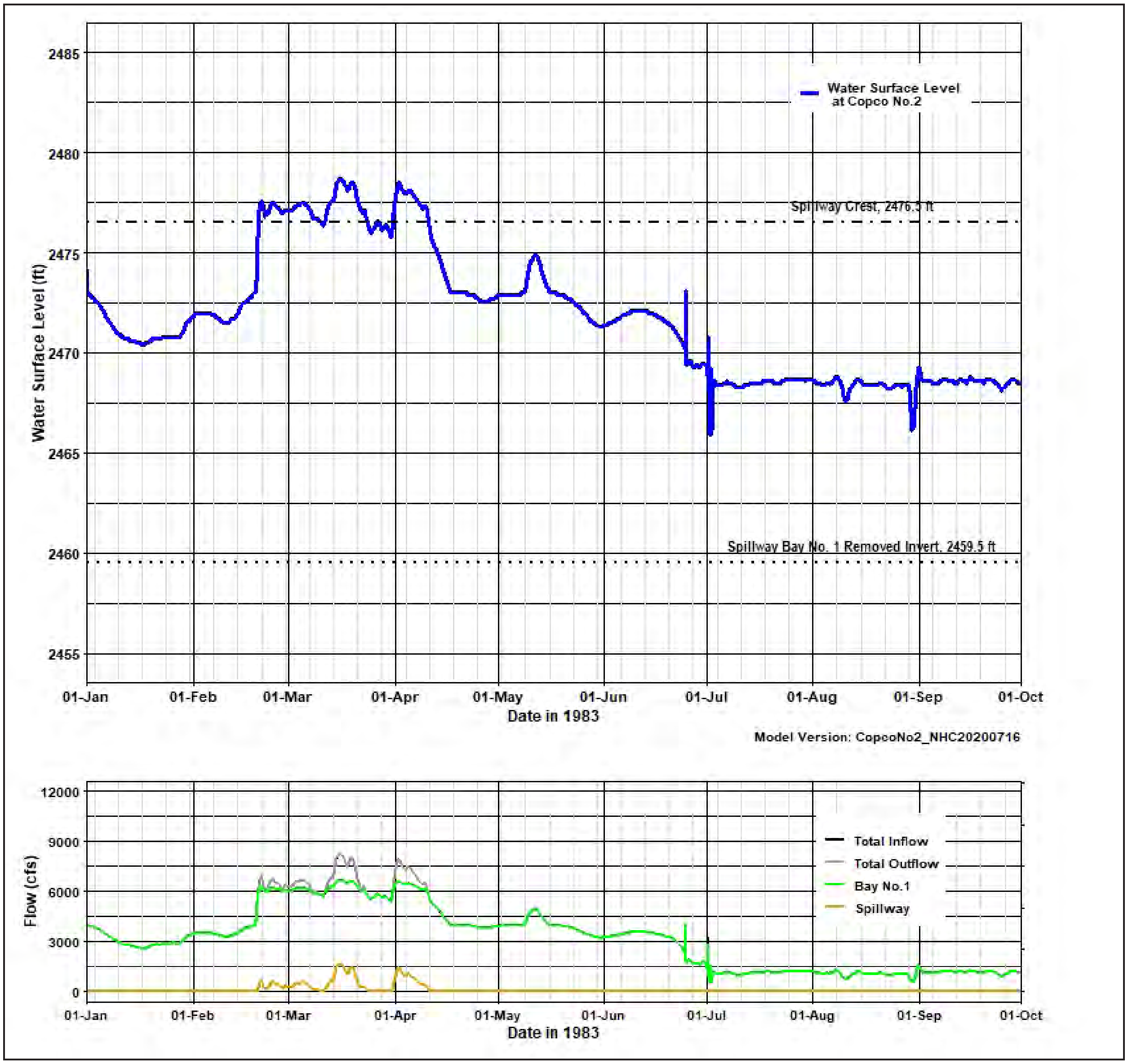


**Figure 3 -** COPCO No.2 Facility Simulated Drawdown - Year 1983

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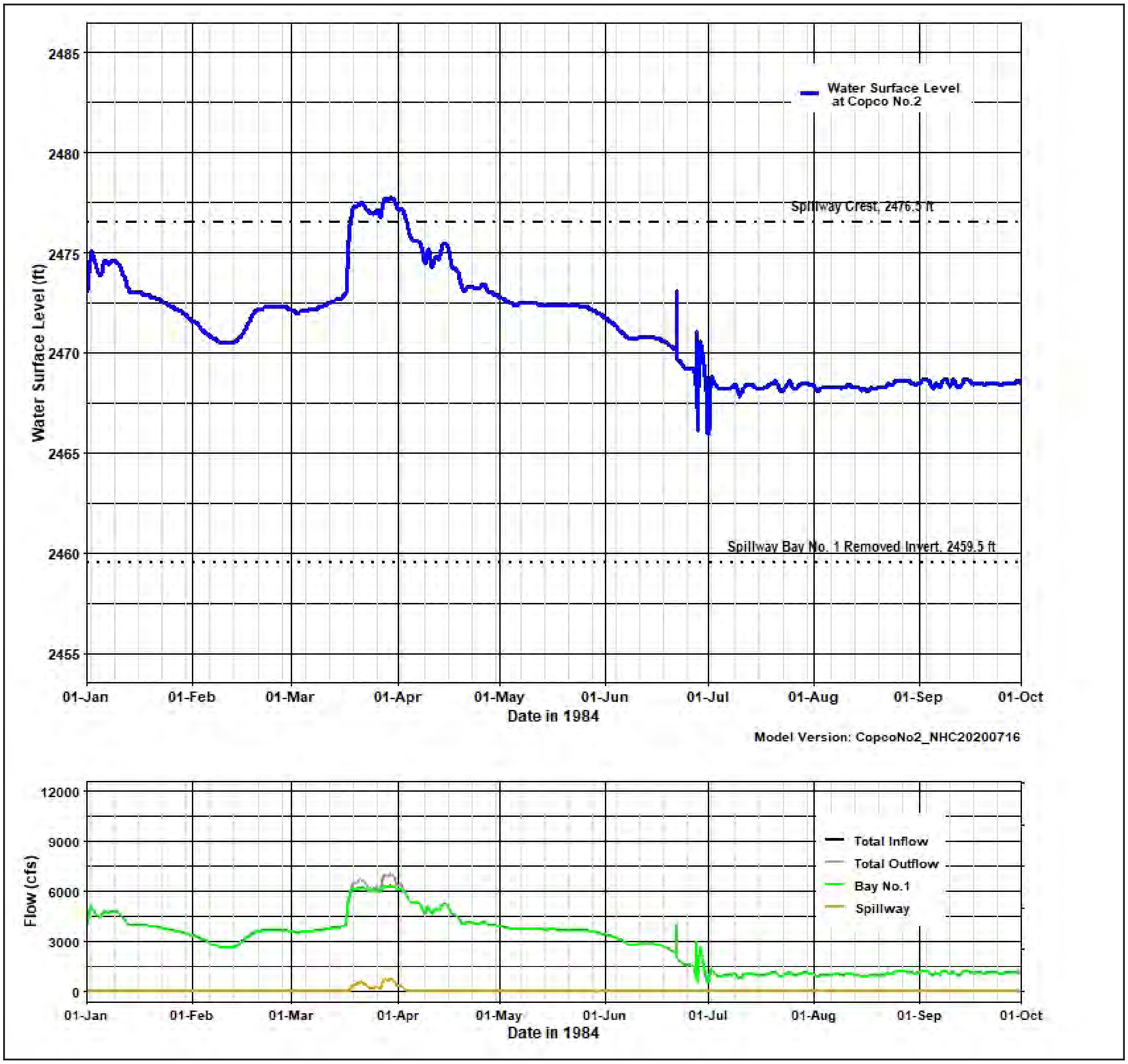


**Figure 4 -** COPCO No.2 Facility Simulated Drawdown - Year 1984

July 29, 2020

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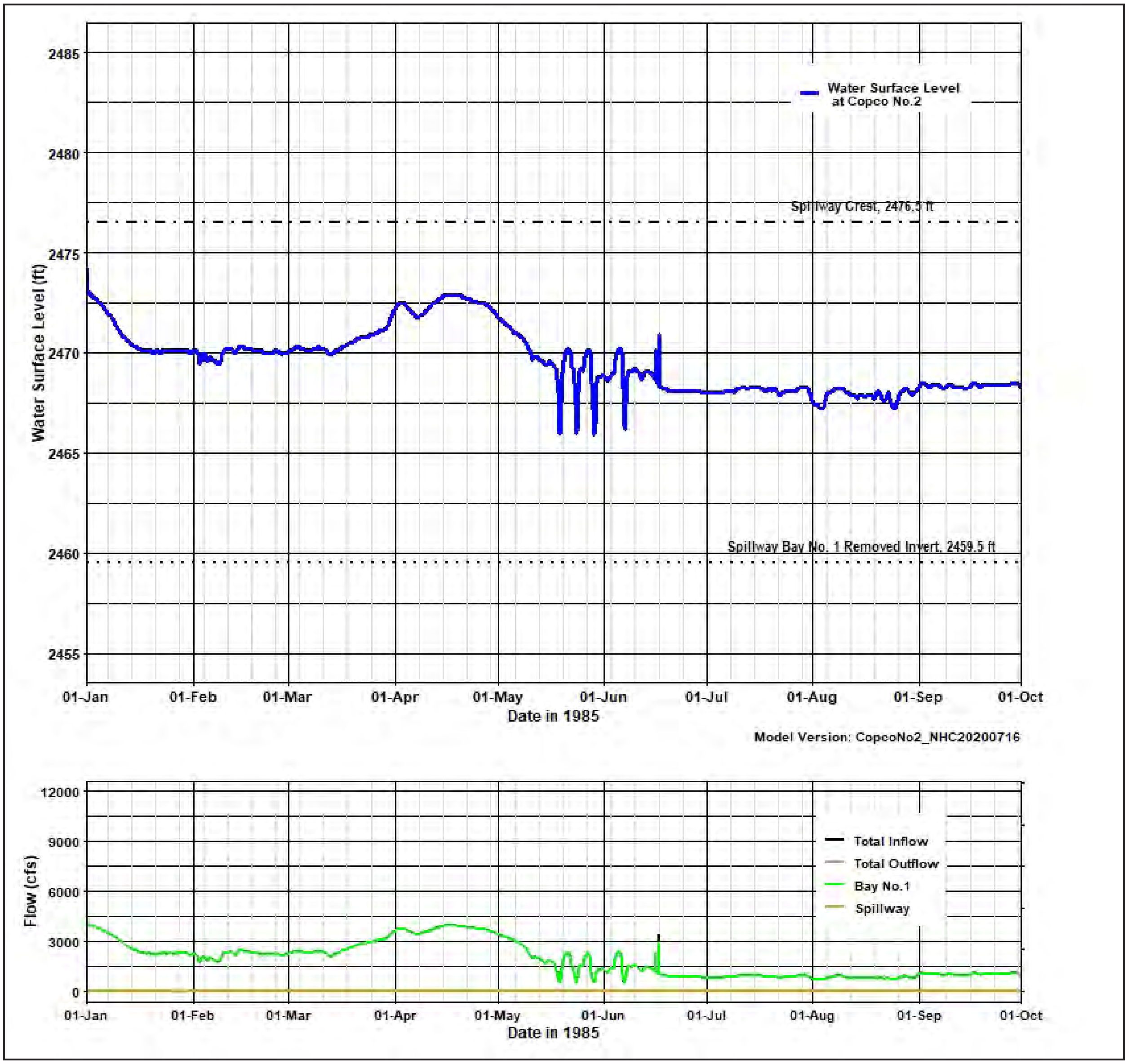


**Figure 5 -** COPCO No.2 Facility Simulated Drawdown - Year 1985

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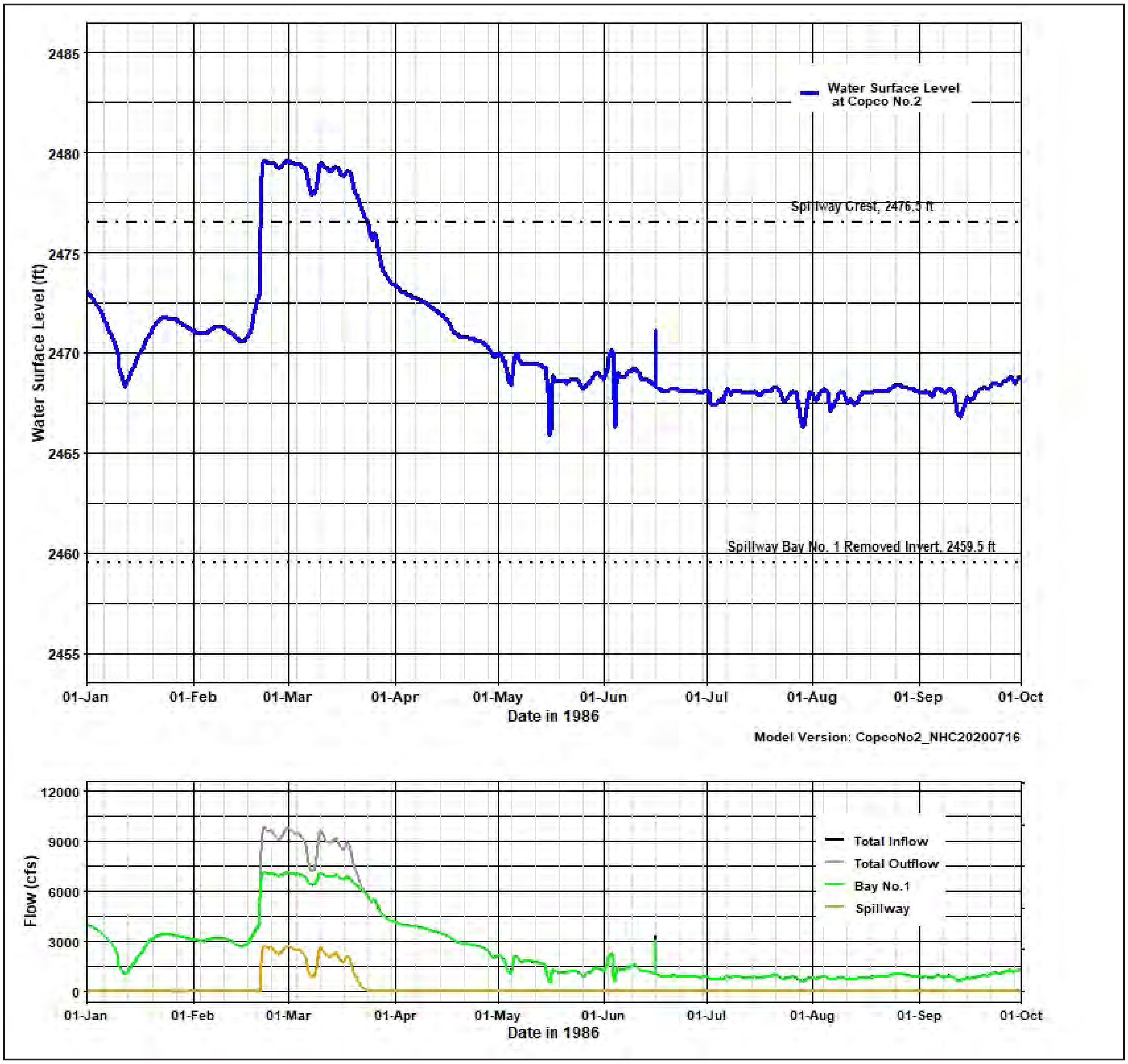


**Figure 6 -** COPCO No.2 Facility Simulated Drawdown - Year 1986

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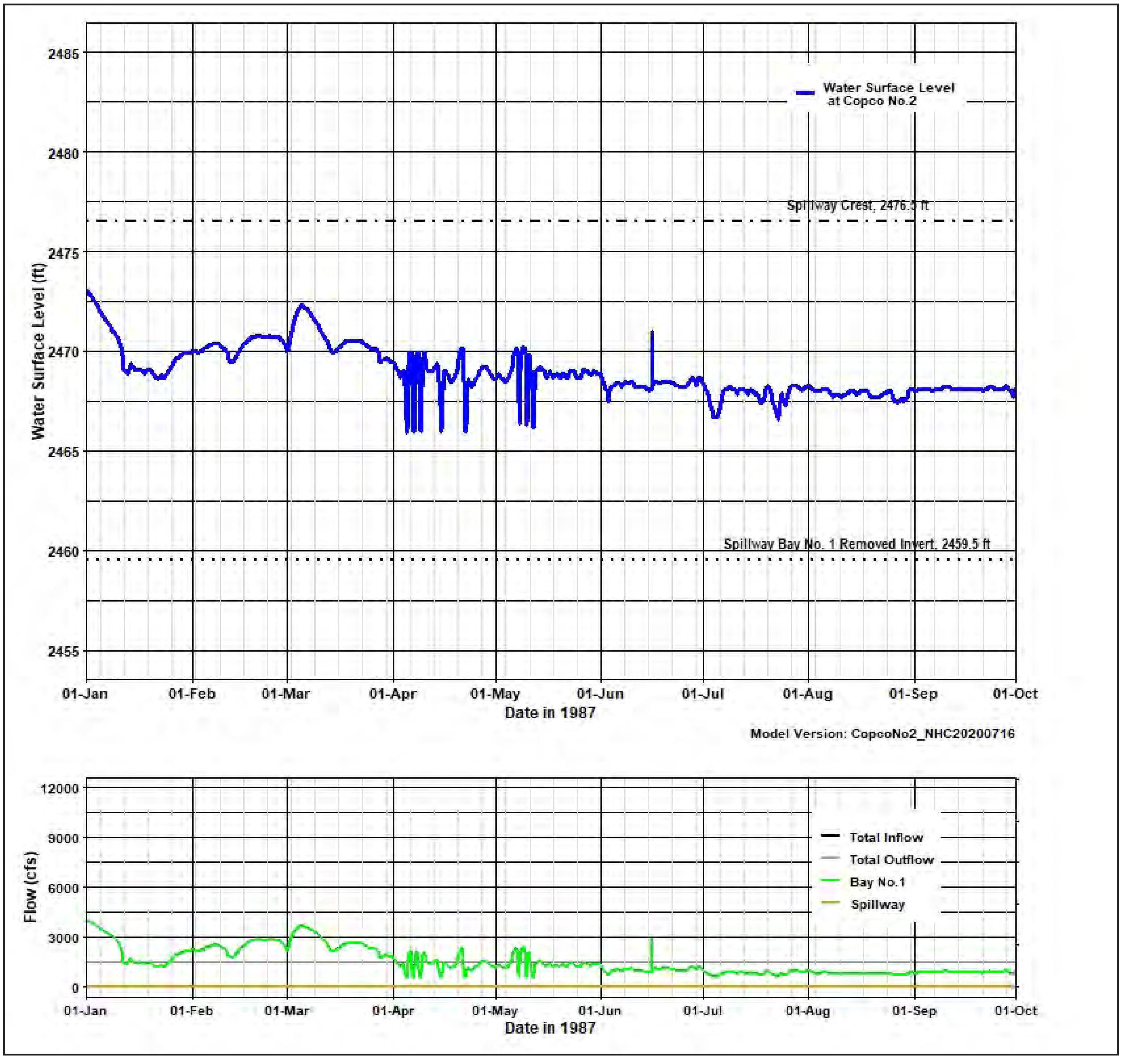


**Figure 7 -** COPCO No.2 Facility Simulated Drawdown - Year 1987

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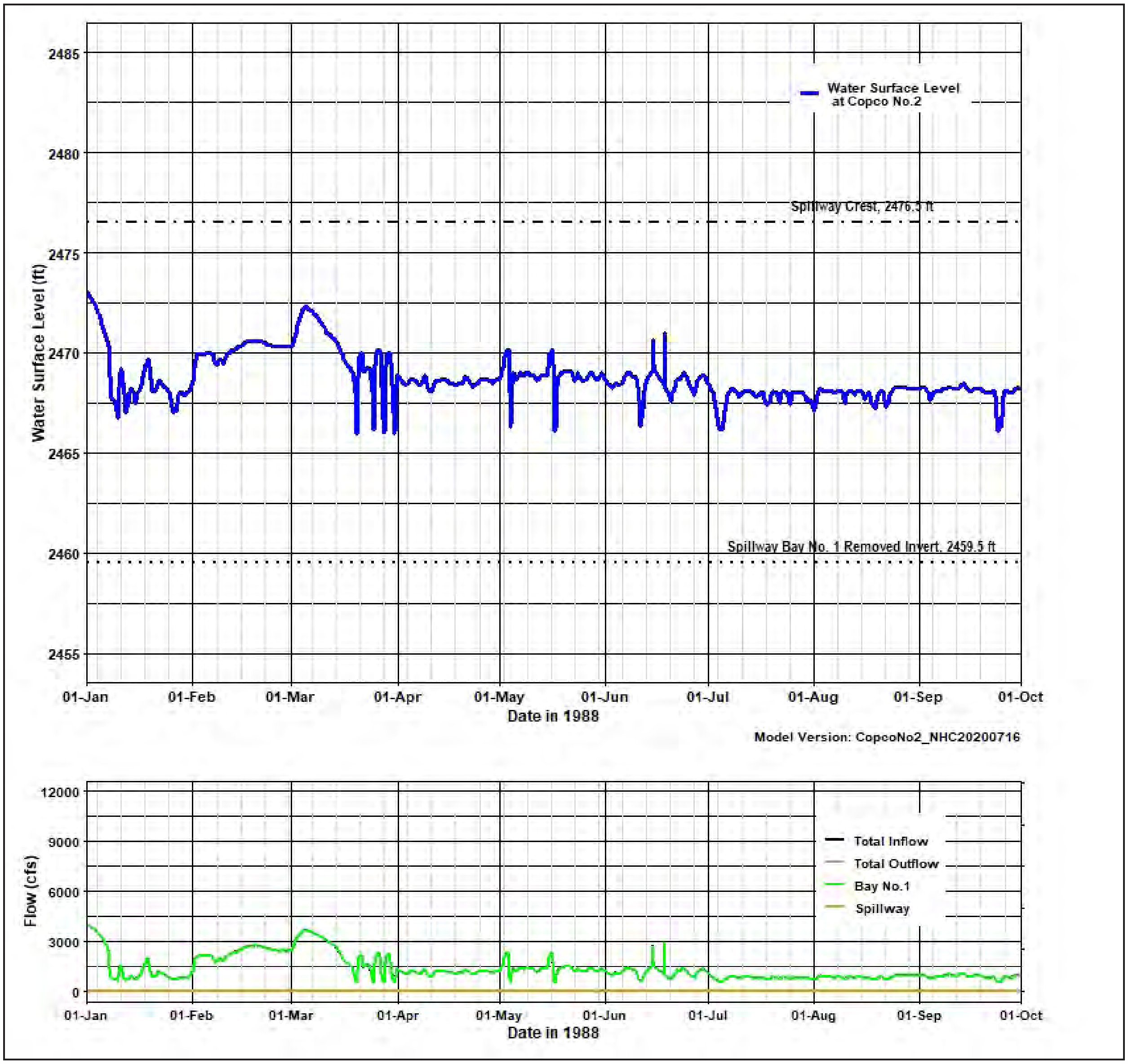


**Figure 8 -** COPCO No.2 Facility Simulated Drawdown - Year 1988

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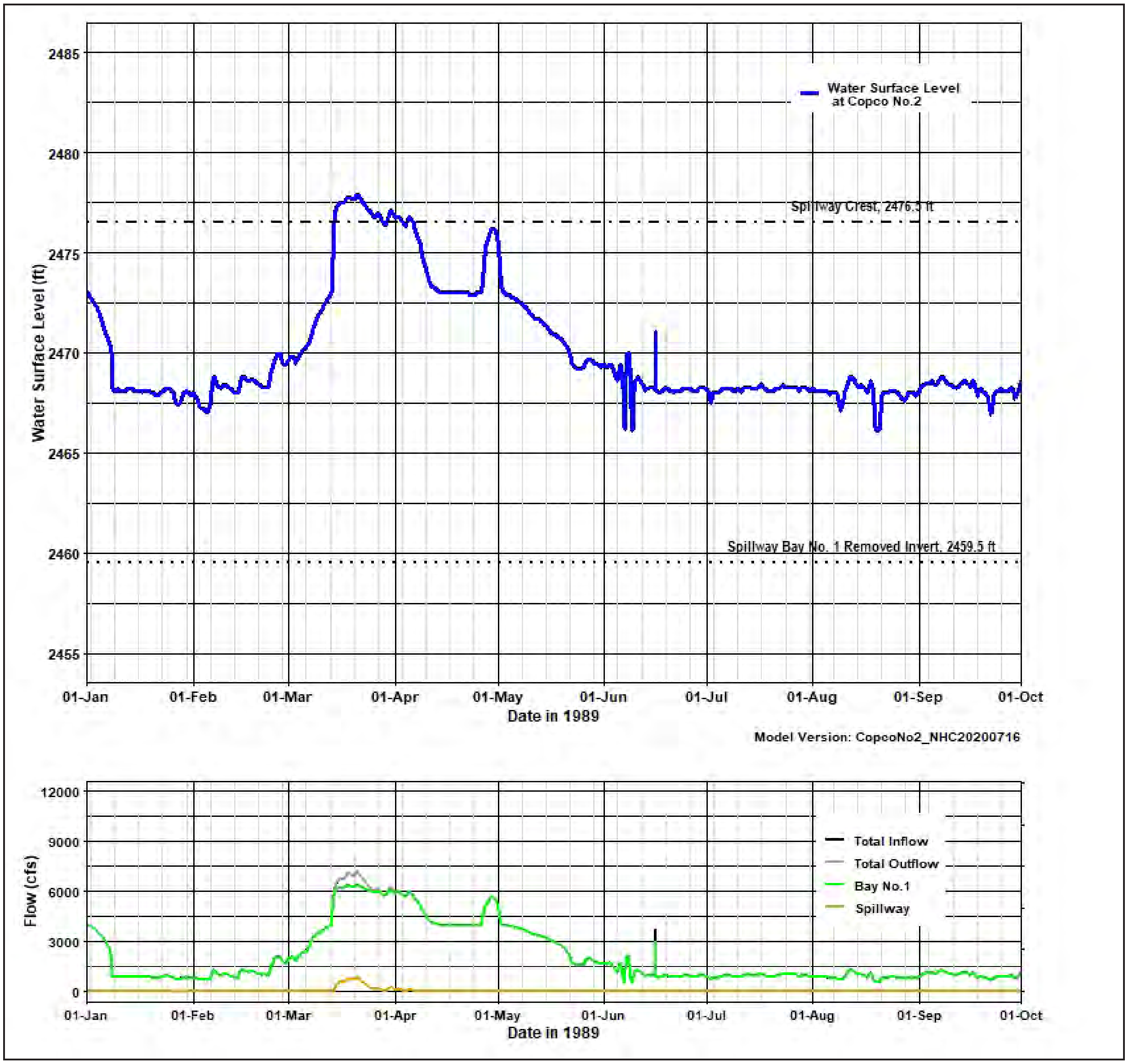


**Figure 9 -** COPCO No.2 Facility Simulated Drawdown - Year 1989

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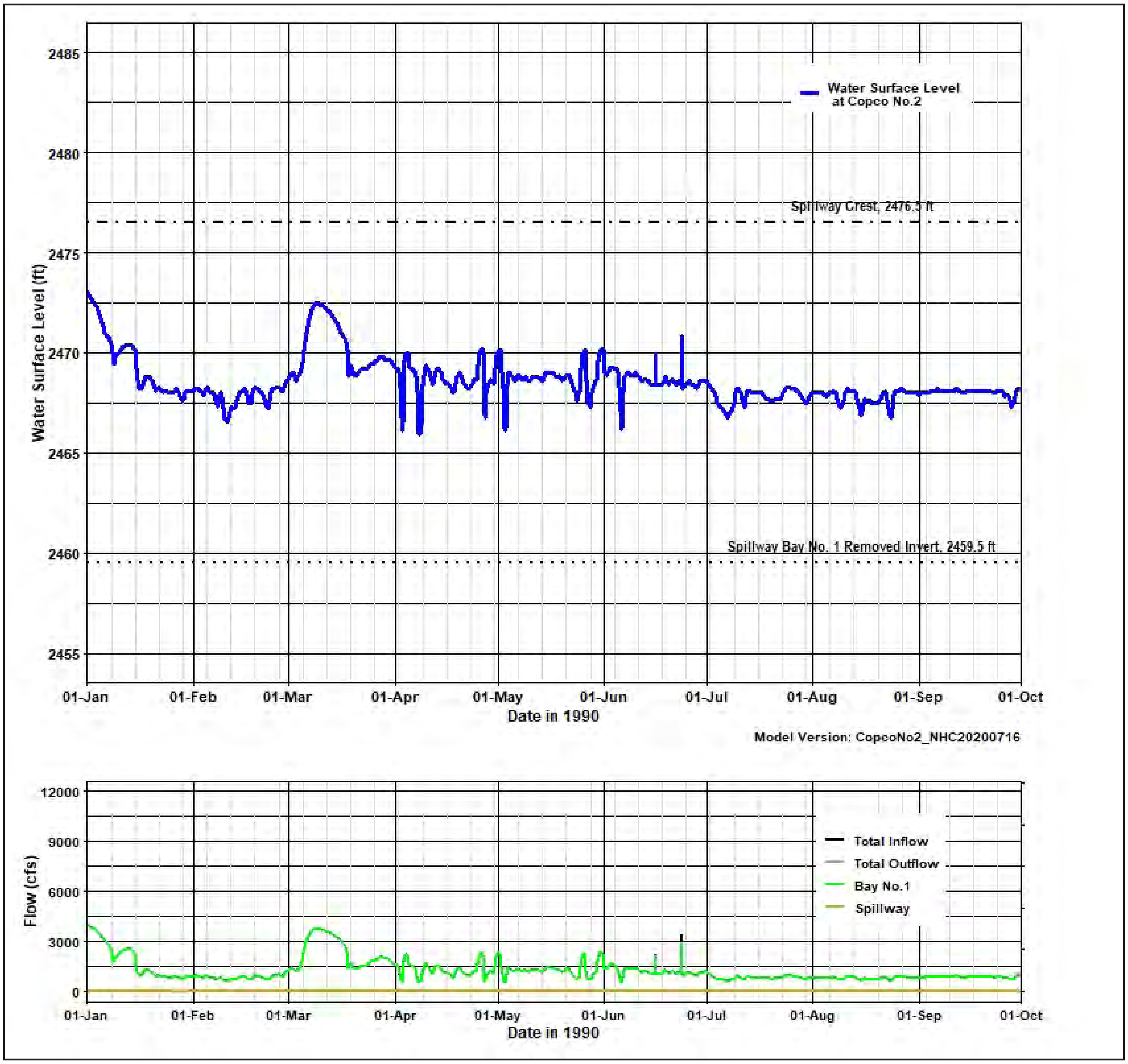


**Figure 10 -** COPCO No.2 Facility Simulated Drawdown - Year 1990

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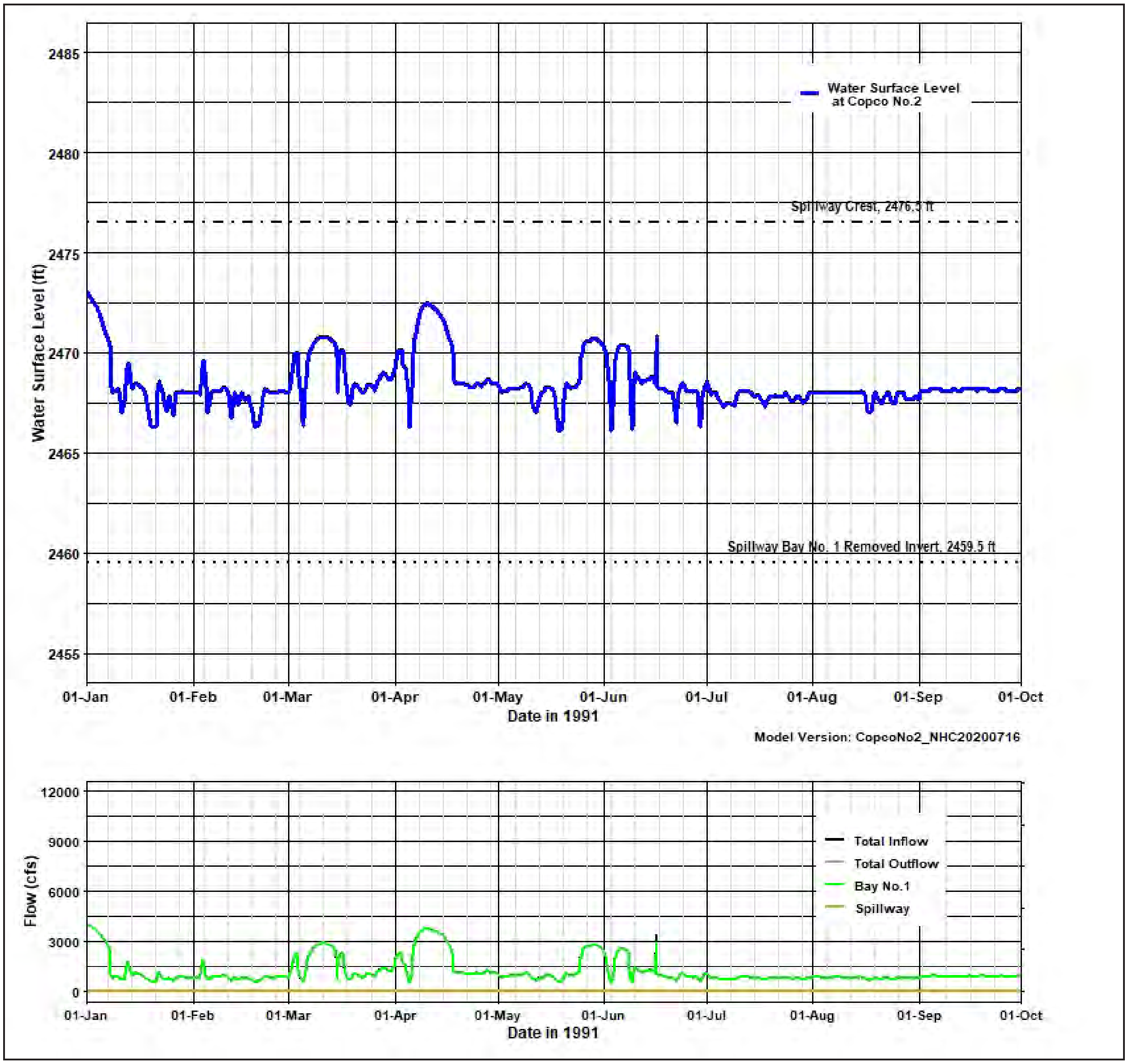


**Figure 11 -** COPCO No.2 Facility Simulated Drawdown - Year 1991

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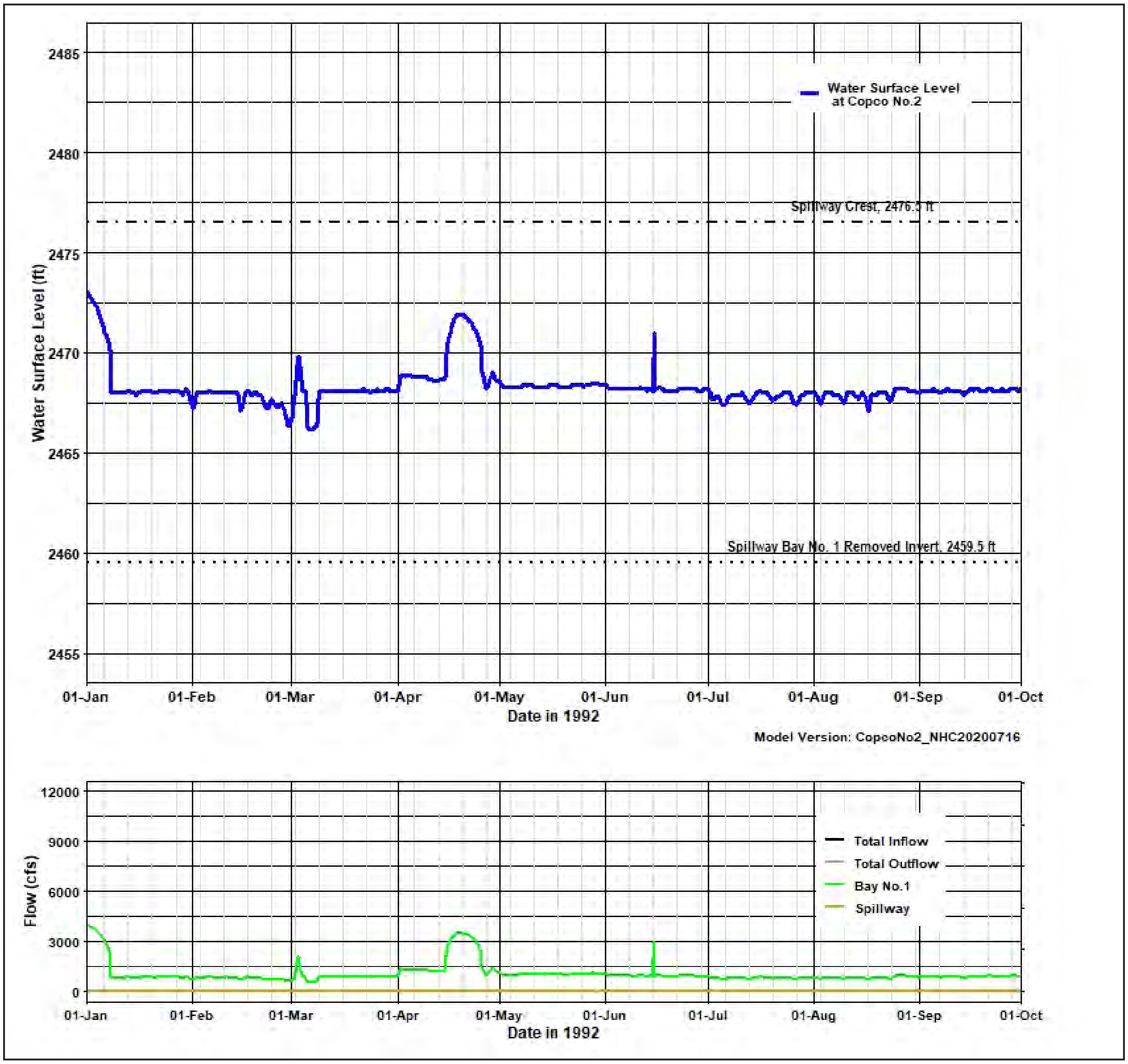


**Figure 12 -** COPCO No.2 Facility Simulated Drawdown - Year 1992

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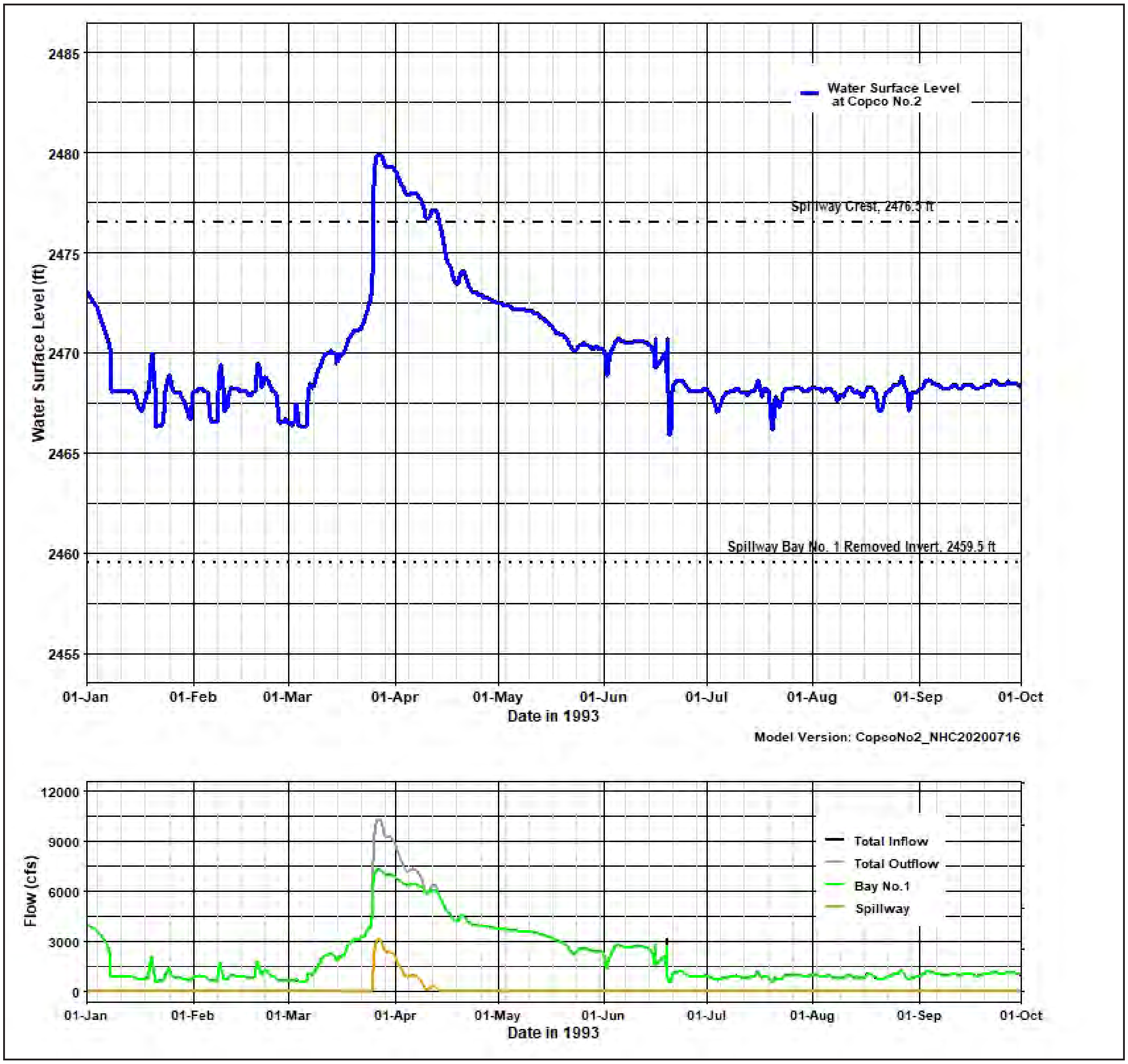


**Figure 13 -** COPCO No.2 Facility Simulated Drawdown - Year 1993

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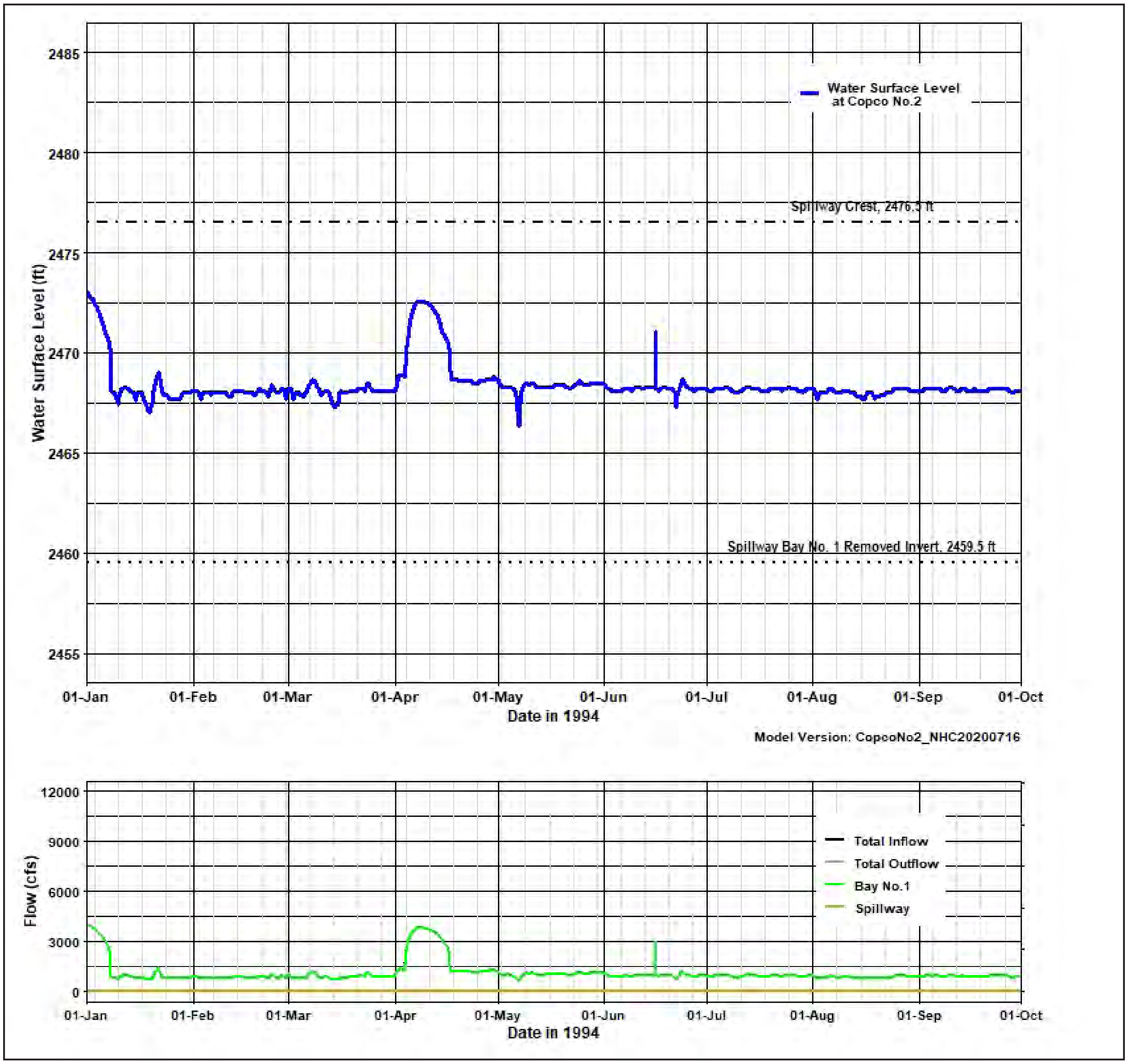


**Figure 14 -** COPCO No.2 Facility Simulated Drawdown - Year 1994

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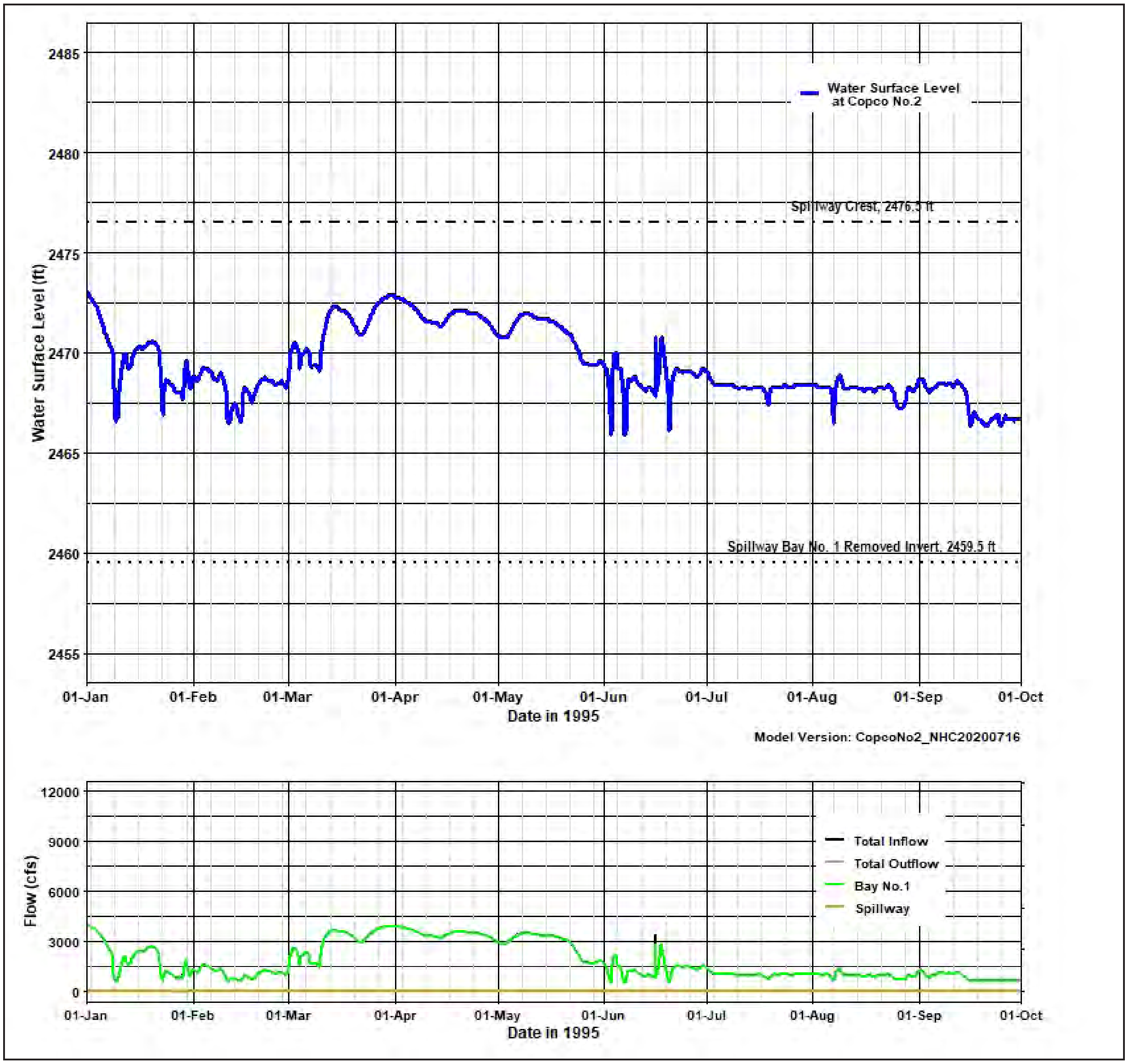


**Figure 15 -** COPCO No.2 Facility Simulated Drawdown - Year 1995

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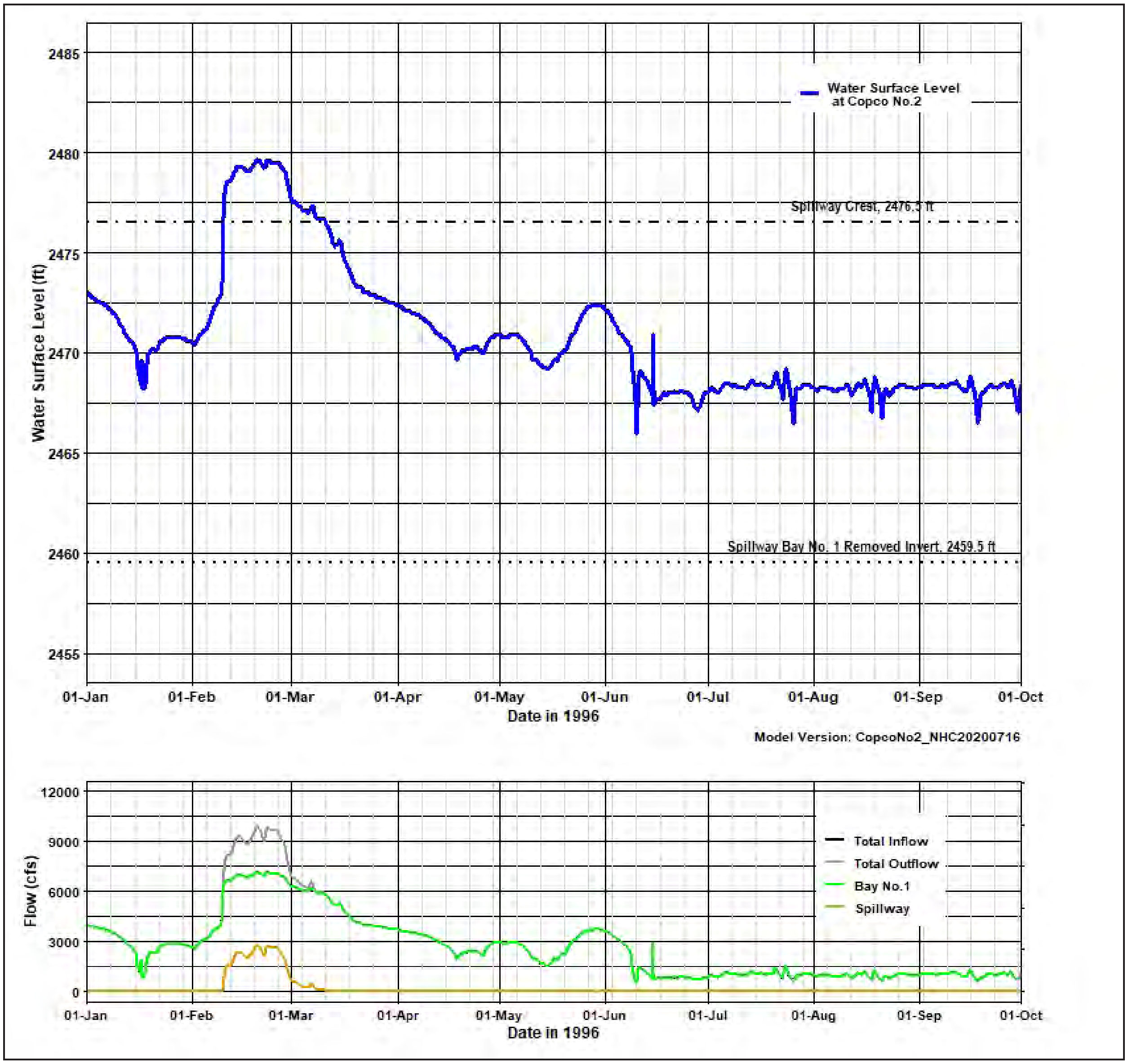


**Figure 16 -** COPCO No.2 Facility Simulated Drawdown - Year 1996

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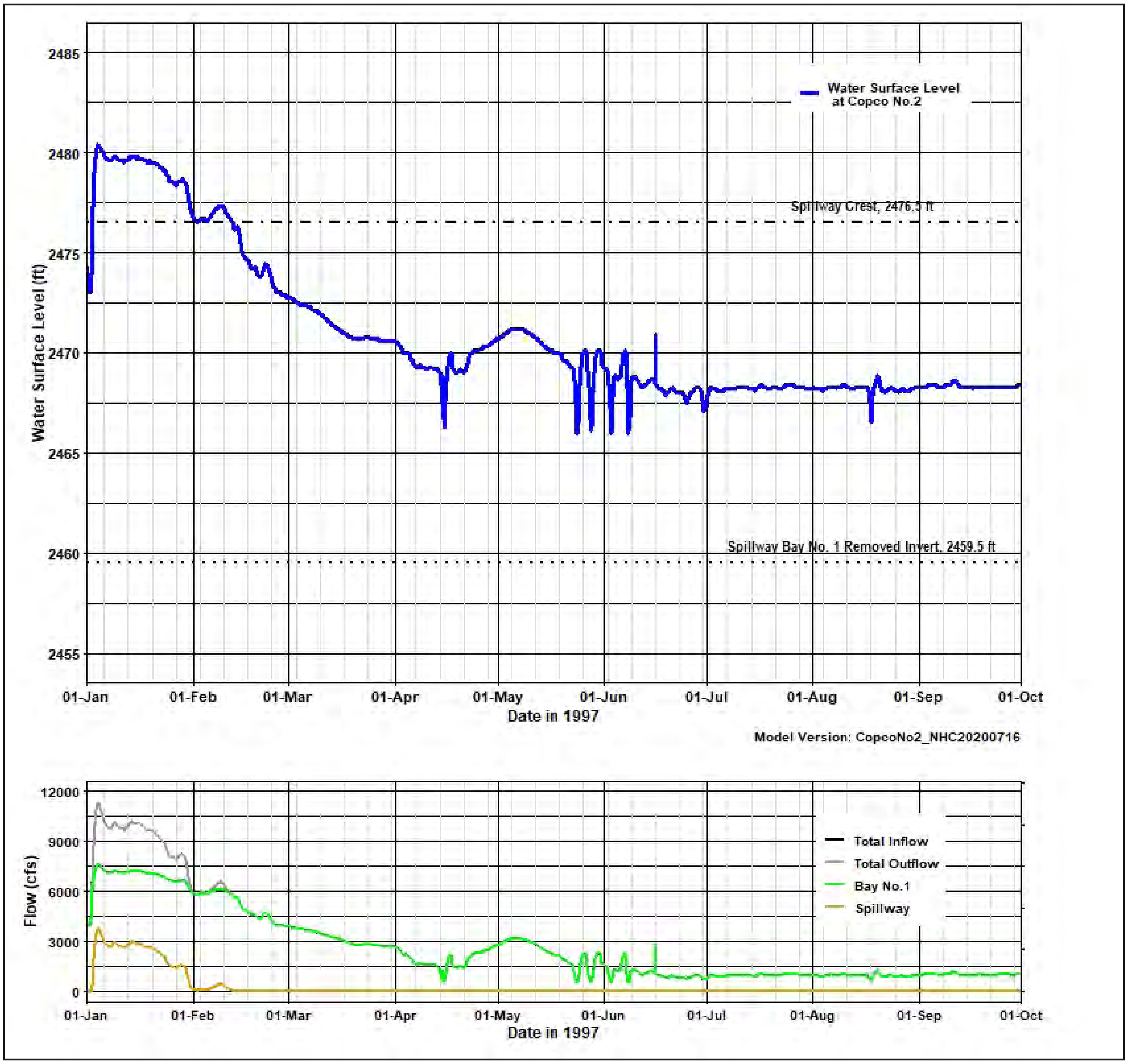


**Figure 17 -** COPCO No.2 Facility Simulated Drawdown - Year 1997

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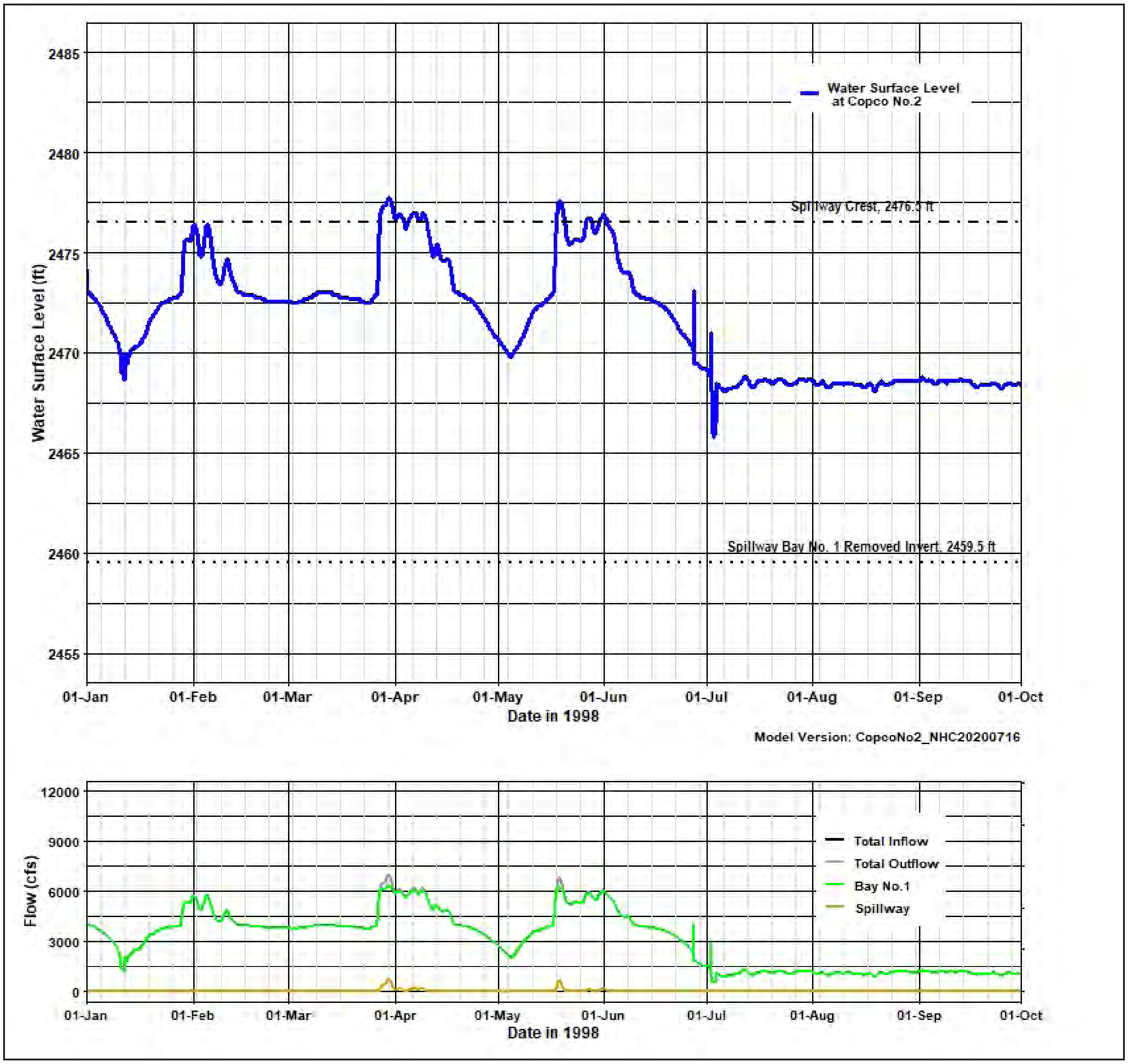


**Figure 18 -** COPCO No.2 Facility Simulated Drawdown - Year 1998

July 29, 2020

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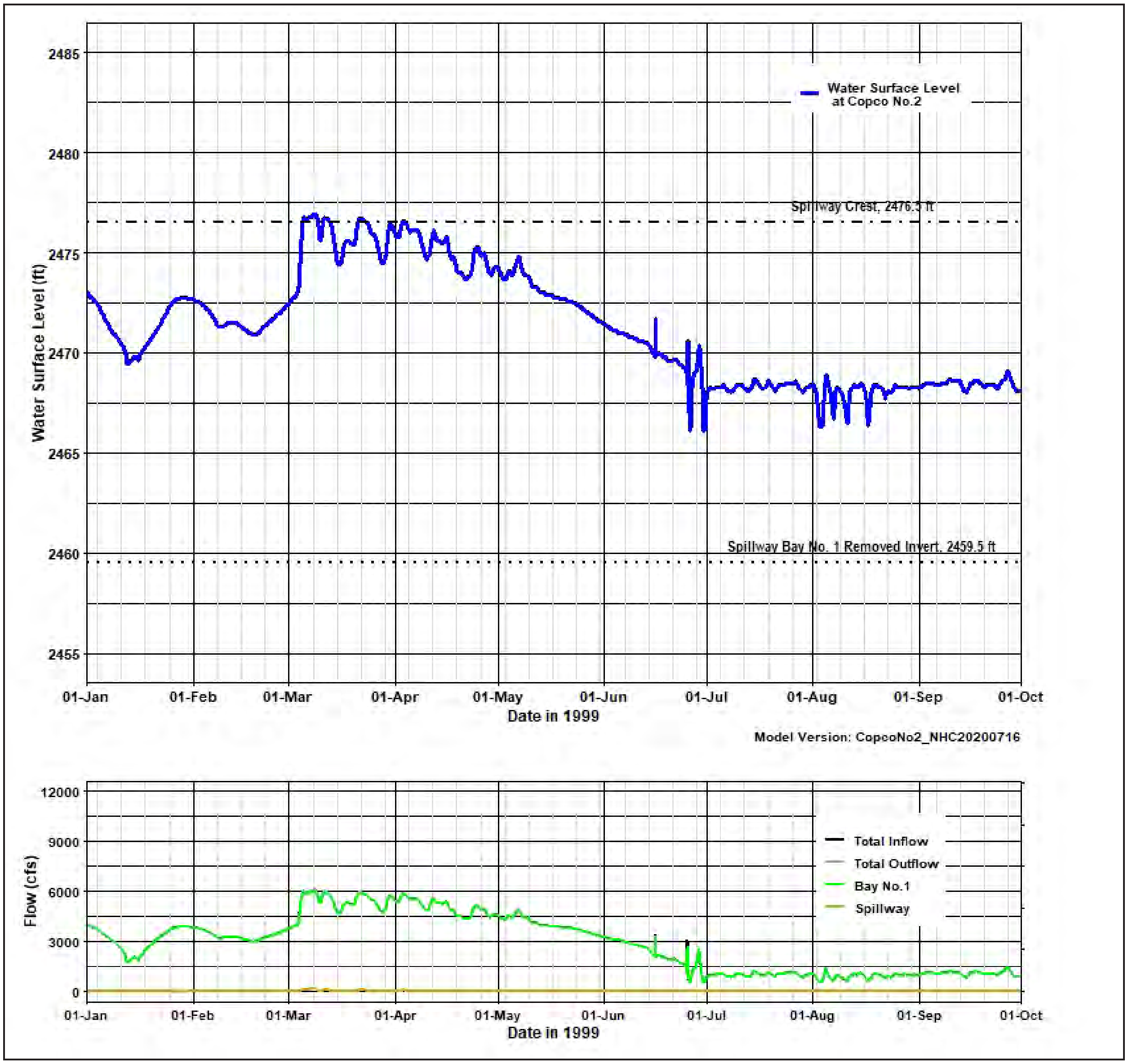


**Figure 19 -** COPCO No.2 Facility Simulated Drawdown - Year 1999

July 29, 2020

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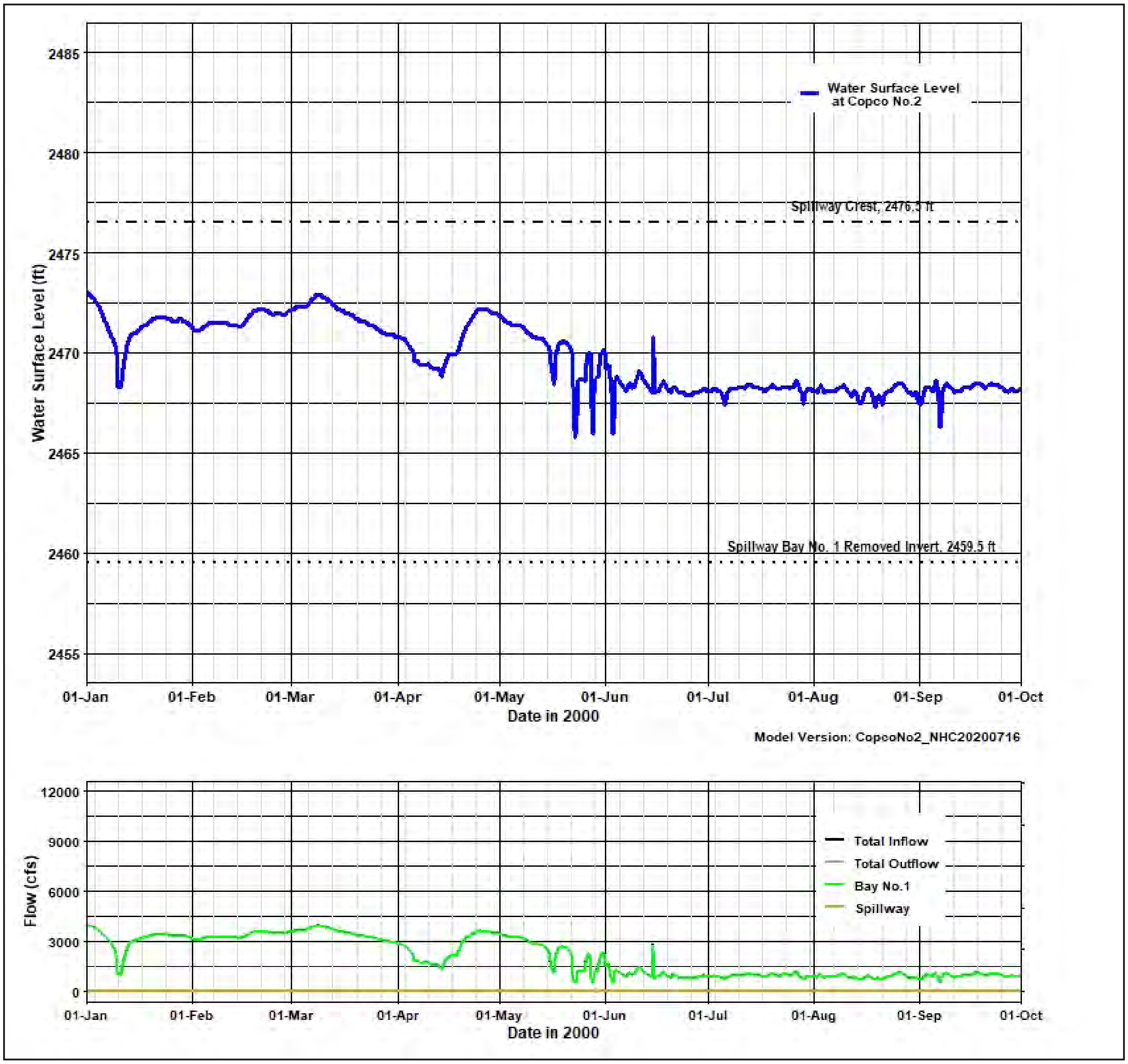


**Figure 20 -** COPCO No.2 Facility Simulated Drawdown - Year 2000

July 29, 2020

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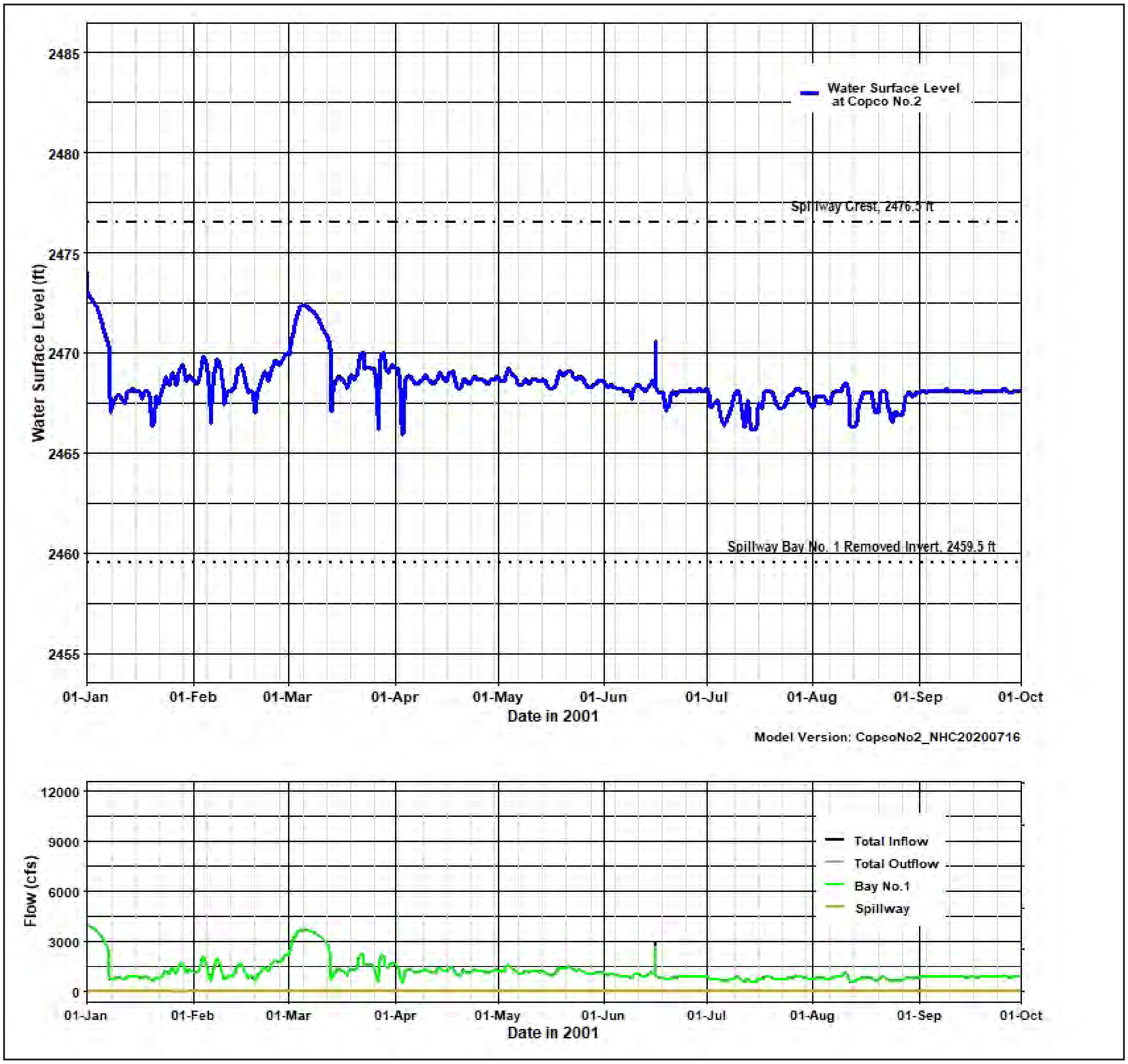


**Figure 21 -** COPCO No.2 Facility Simulated Drawdown - Year 2001

July 29, 2020

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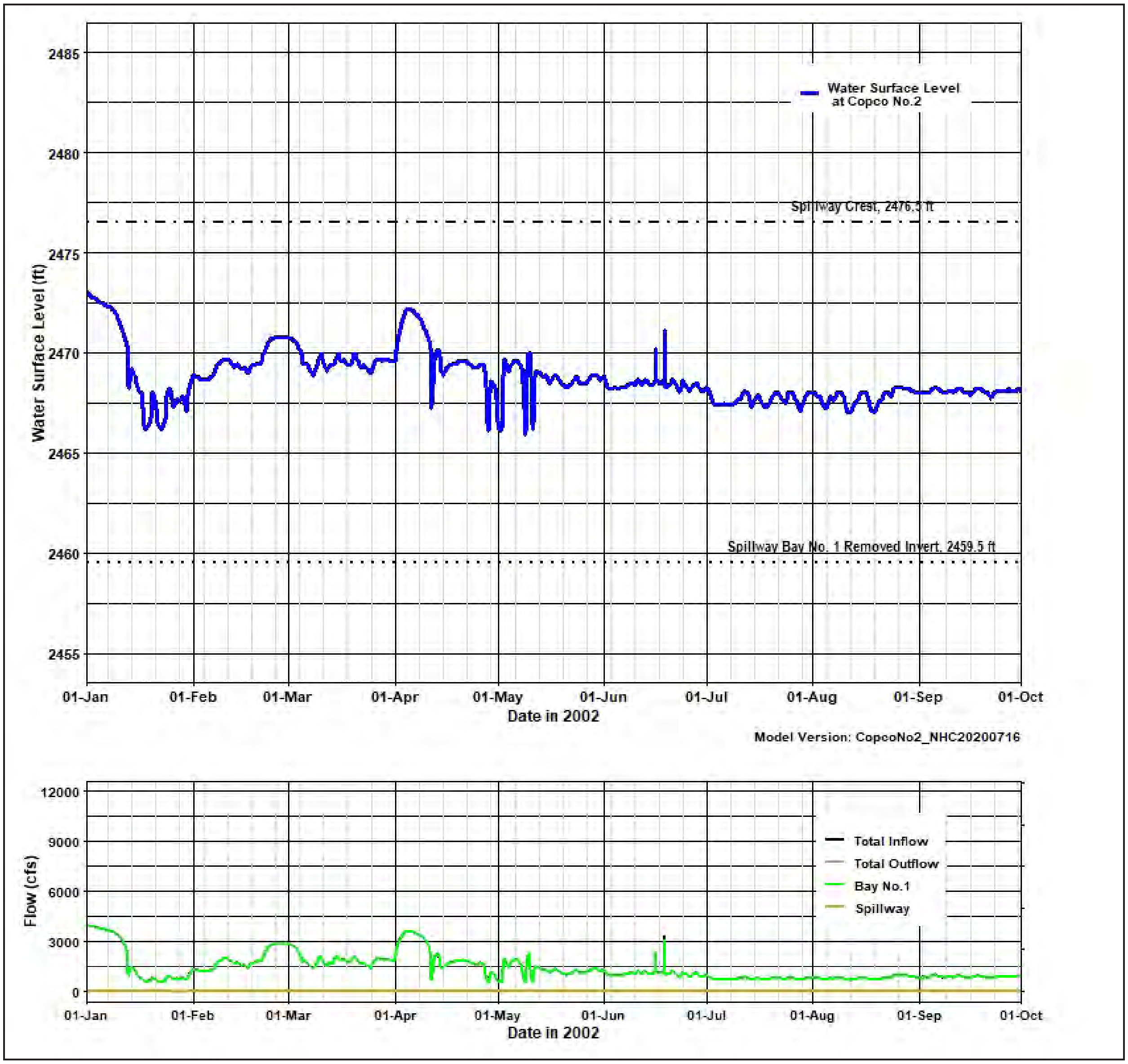


**Figure 22 -** COPCO No.2 Facility Simulated Drawdown - Year 2002

July 29, 2020

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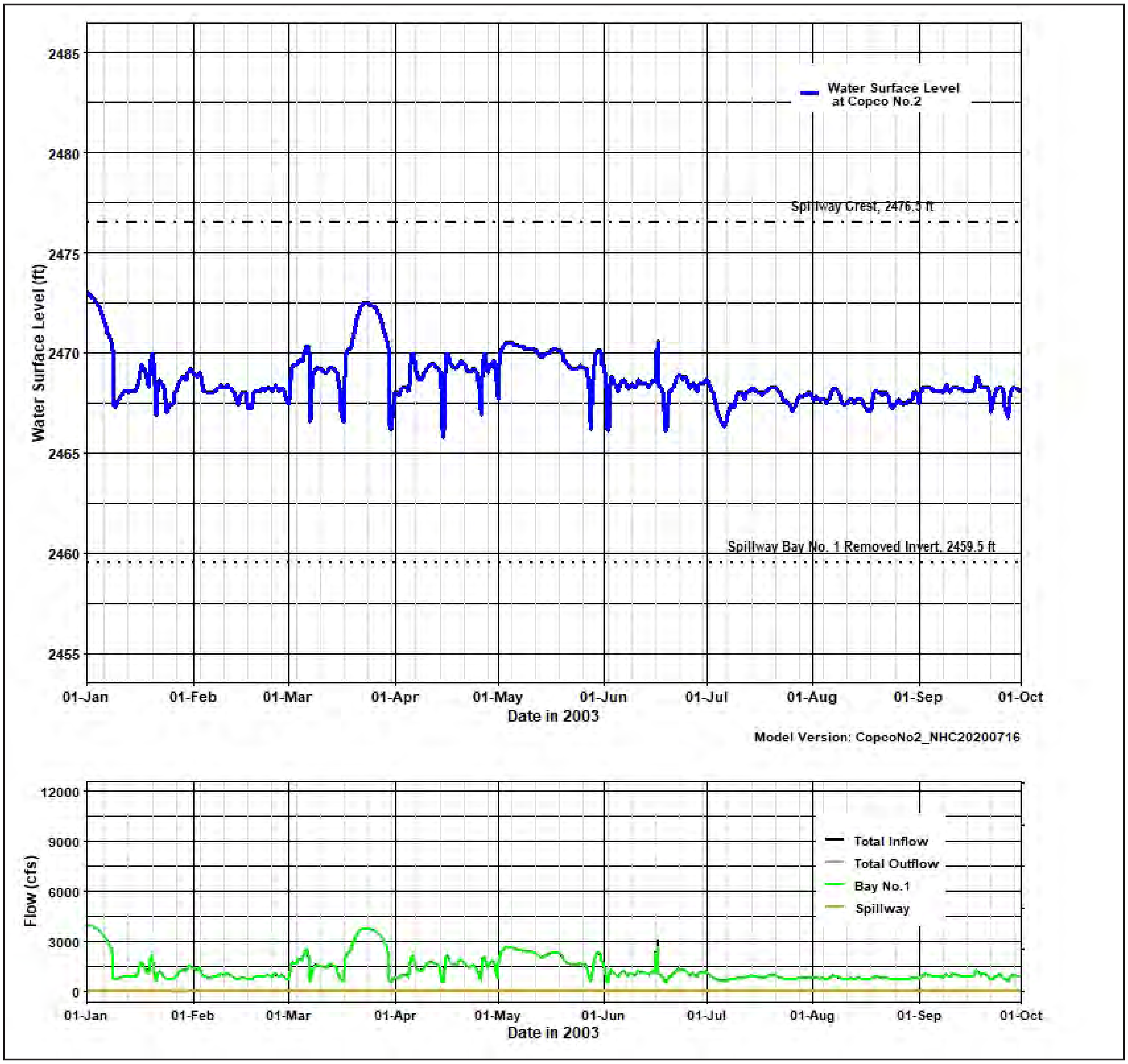


**Figure 23 -** COPCO No.2 Facility Simulated Drawdown - Year 2003

July 29, 2020

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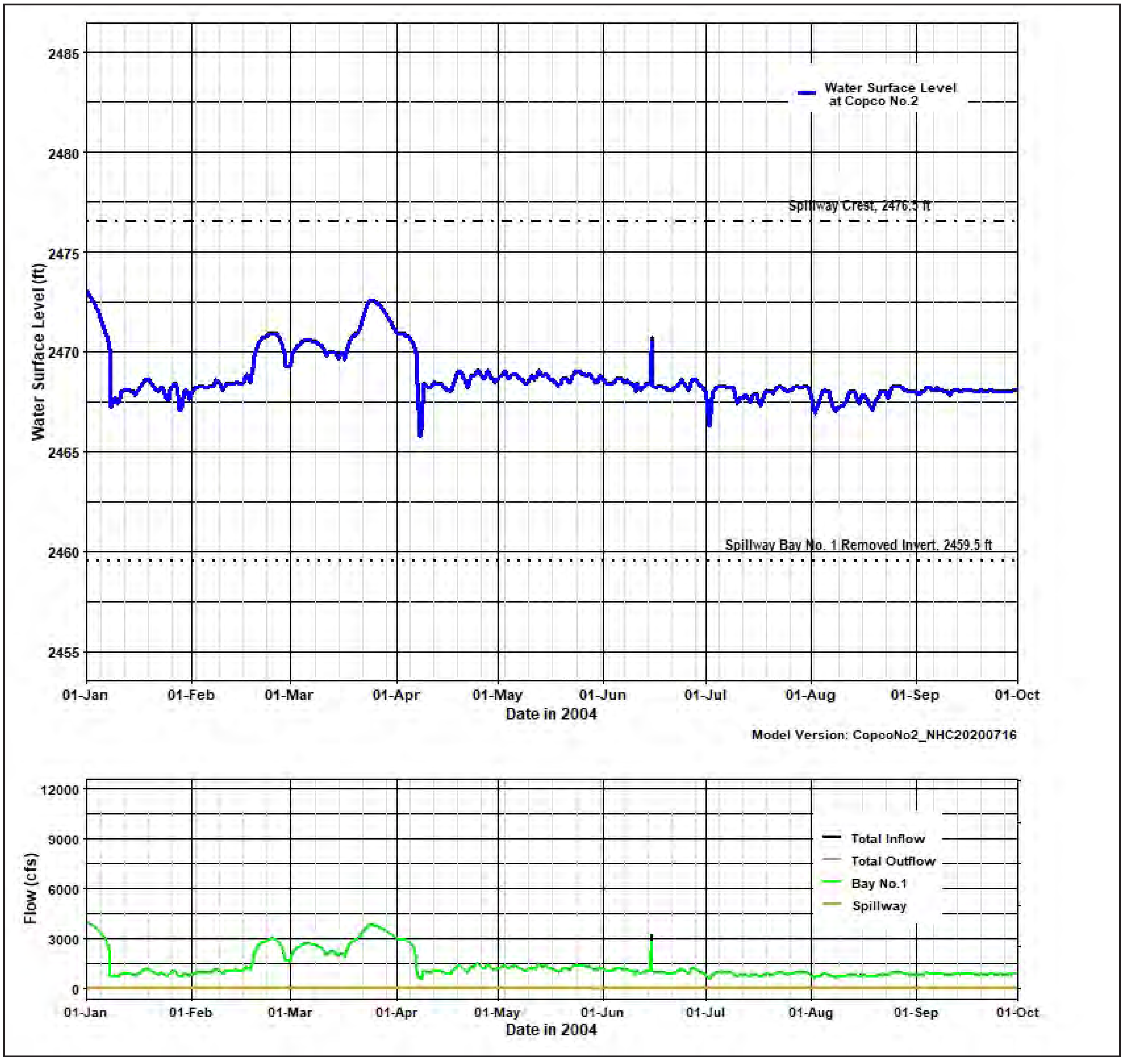


**Figure 24 -** COPCO No.2 Facility Simulated Drawdown - Year 2004

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

Knight Piésold CONSULTING

# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

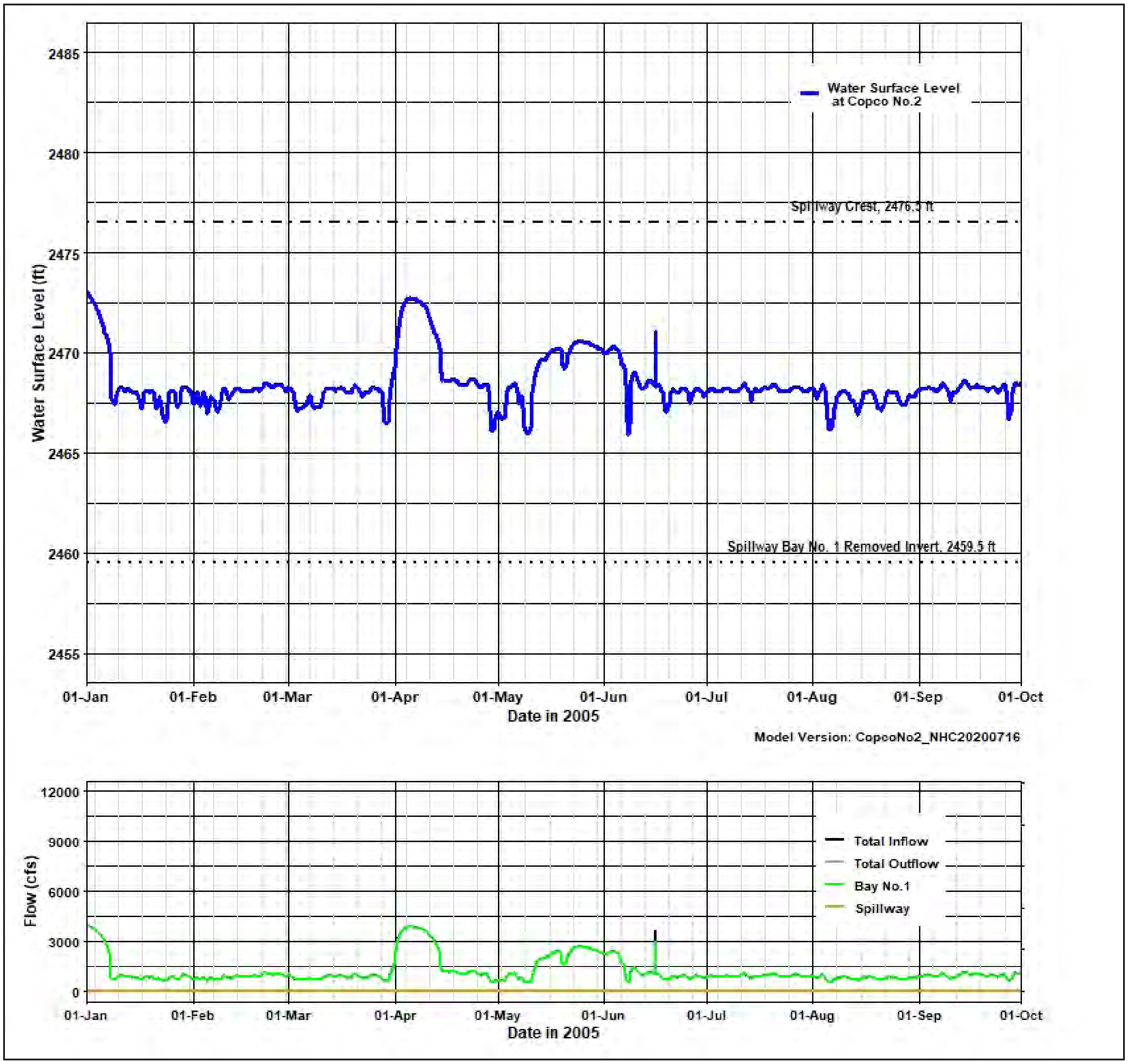


**Figure 25 -** COPCO No.2 Facility Simulated Drawdown - Year 2005

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

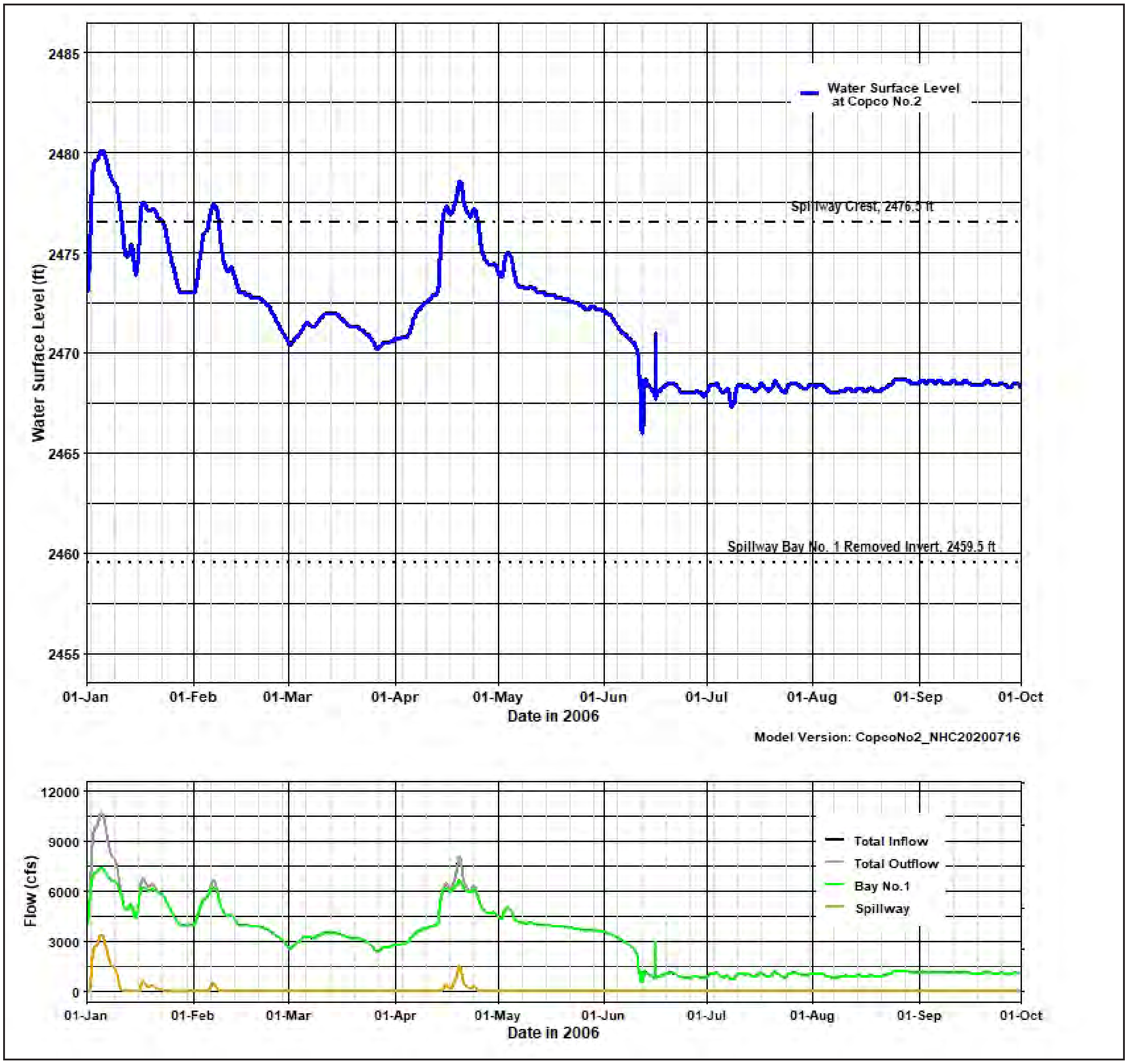


**Figure 26 -** COPCO No.2 Facility Simulated Drawdown - Year 2006

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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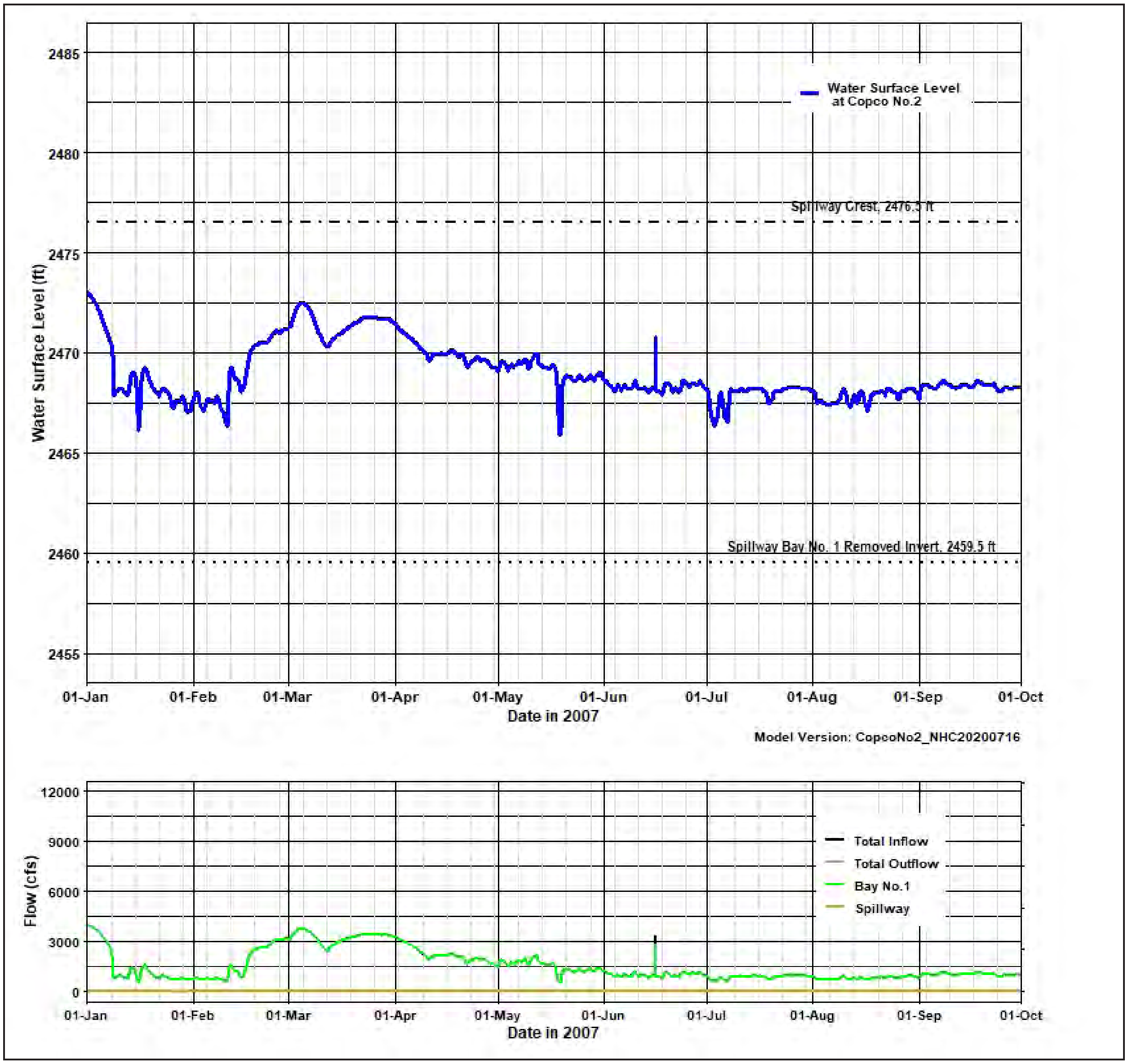


**Figure 27 -** COPCO No.2 Facility Simulated Drawdown - Year 2007

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

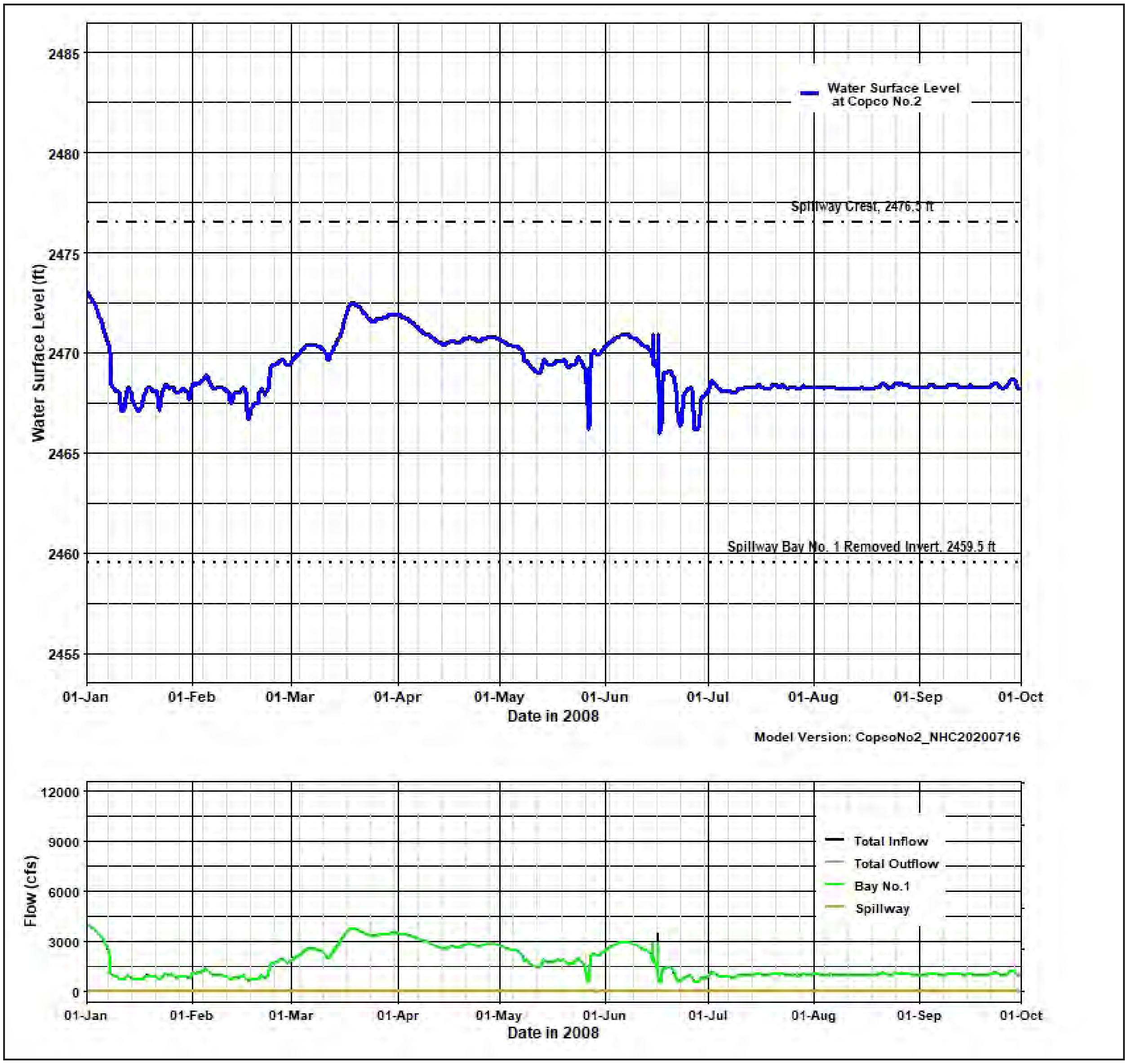


**Figure 28 -** COPCO No.2 Facility Simulated Drawdown - Year 2008

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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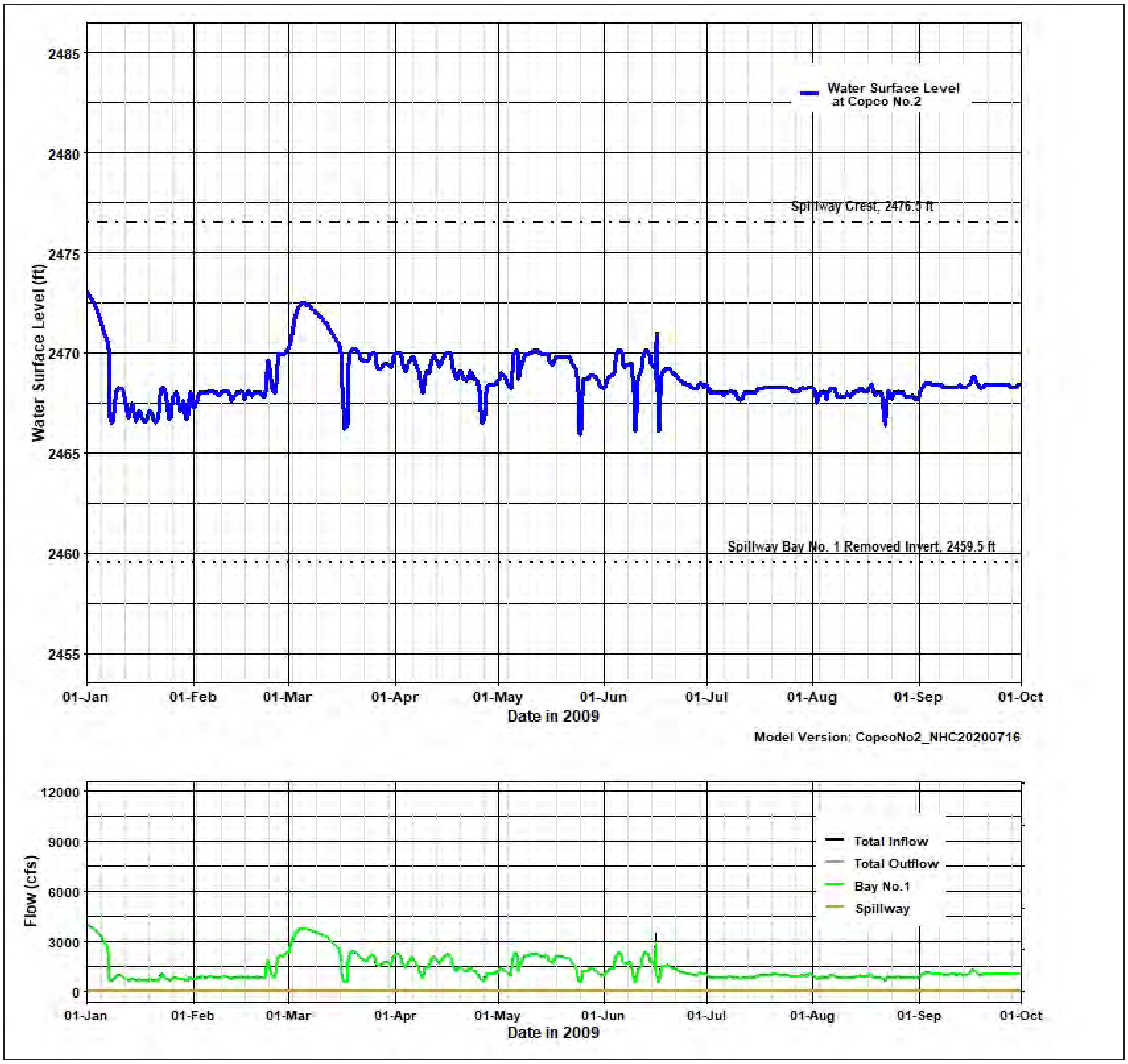


**Figure 29 -** COPCO No.2 Facility Simulated Drawdown - Year 2009

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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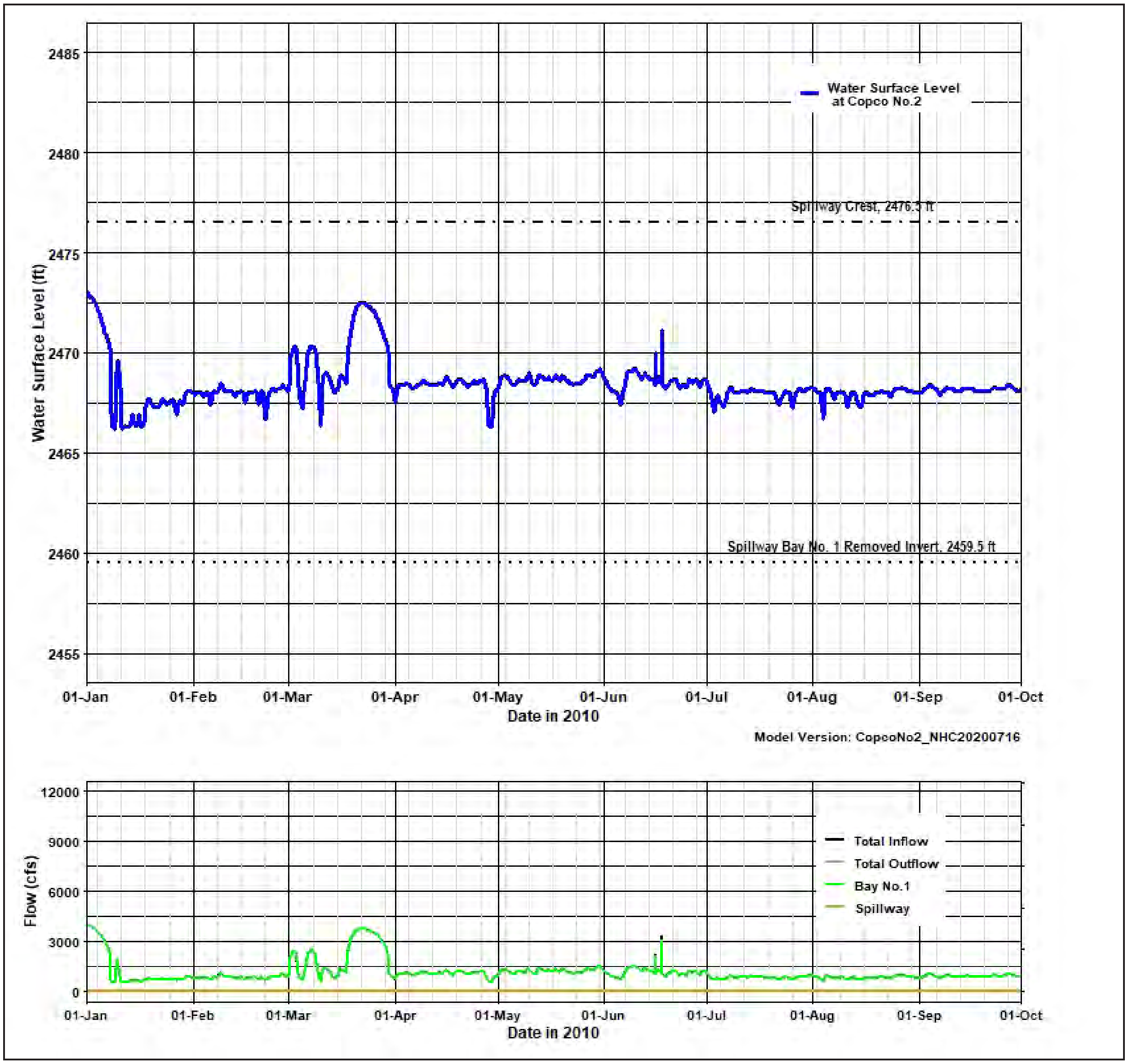


**Figure 30 -** COPCO No.2 Facility Simulated Drawdown - Year 2010

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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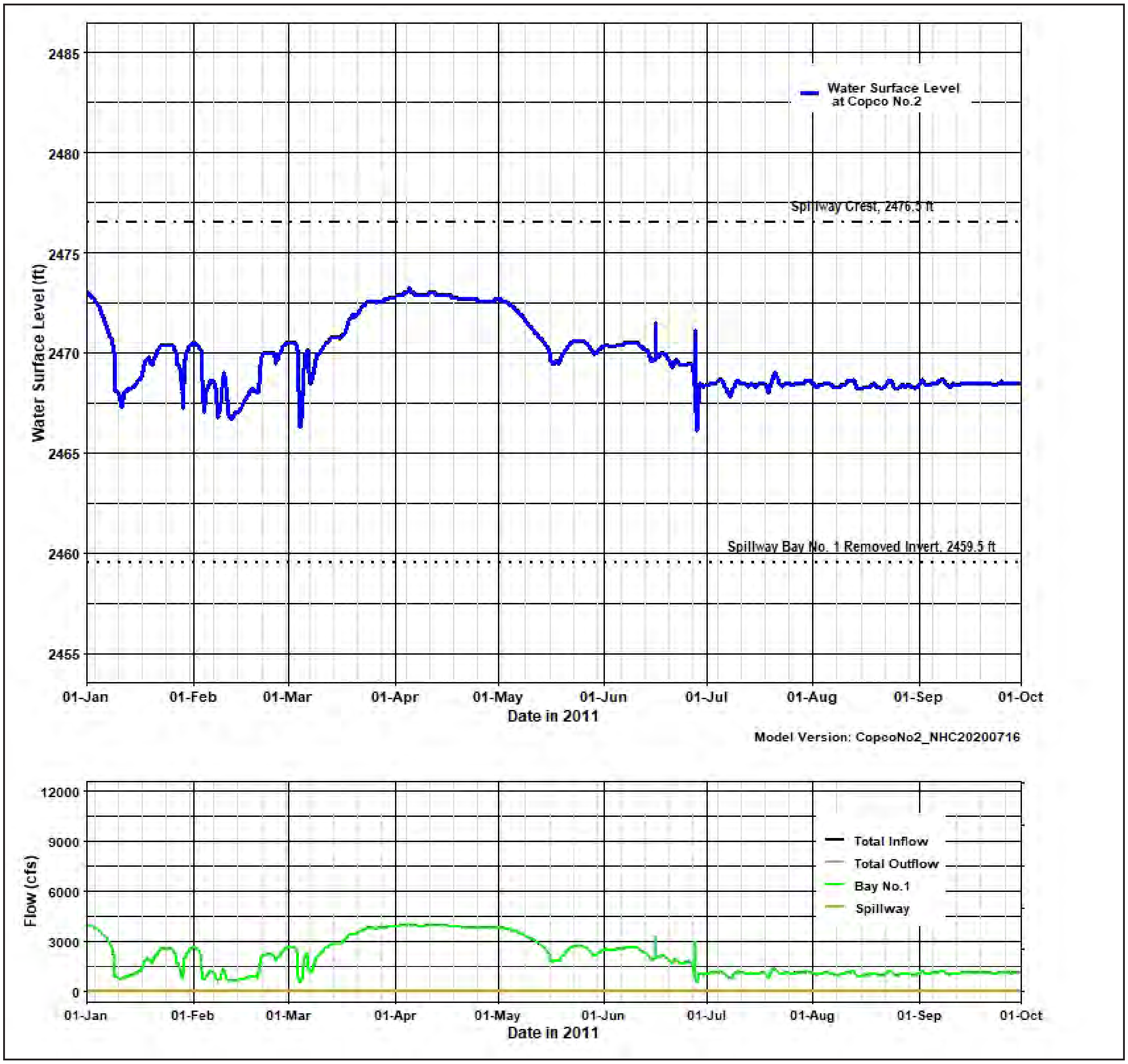


**Figure 31 -** COPCO No.2 Facility Simulated Drawdown - Year 2011

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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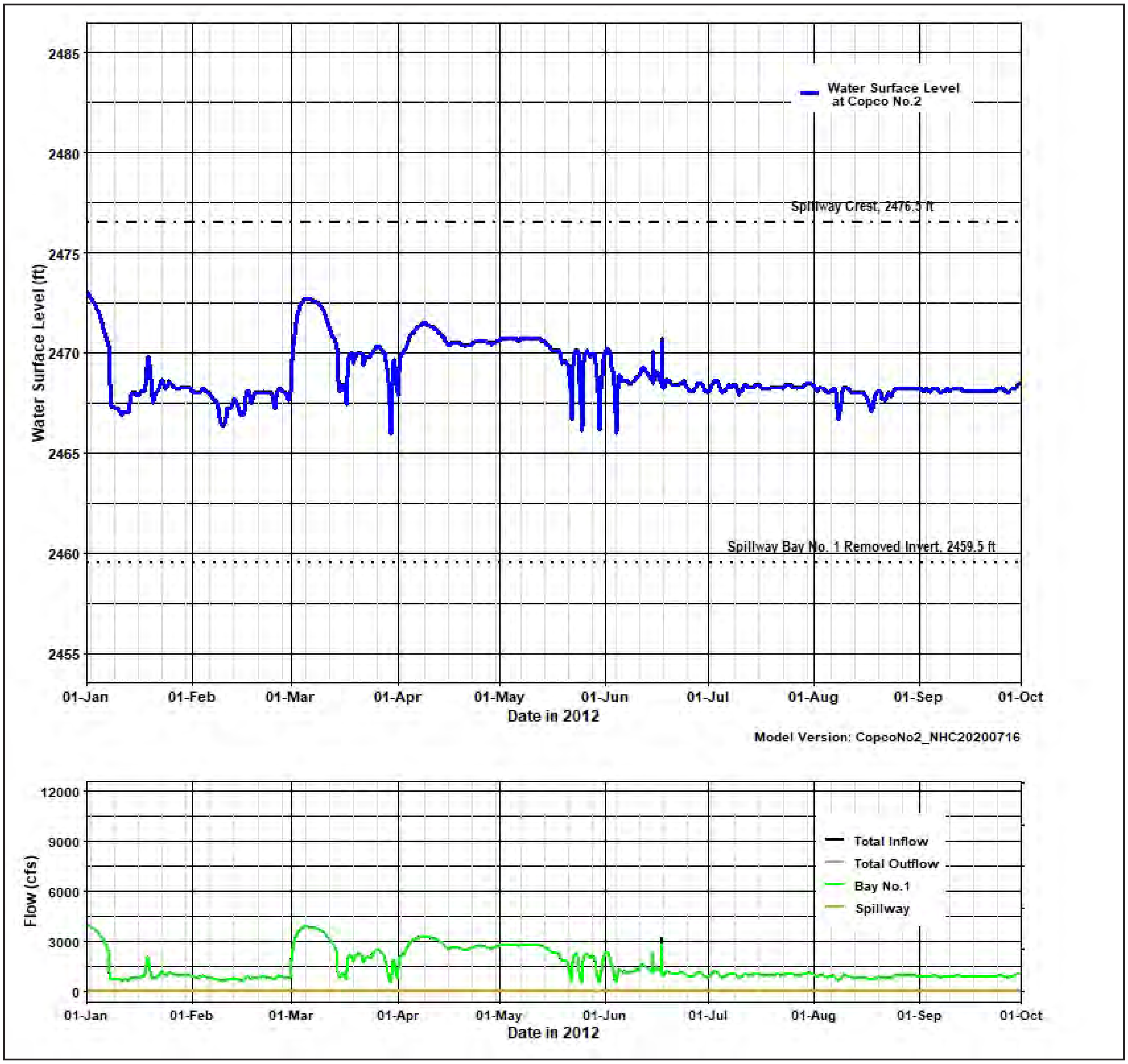


**Figure 32 -** COPCO No.2 Facility Simulated Drawdown - Year 2012

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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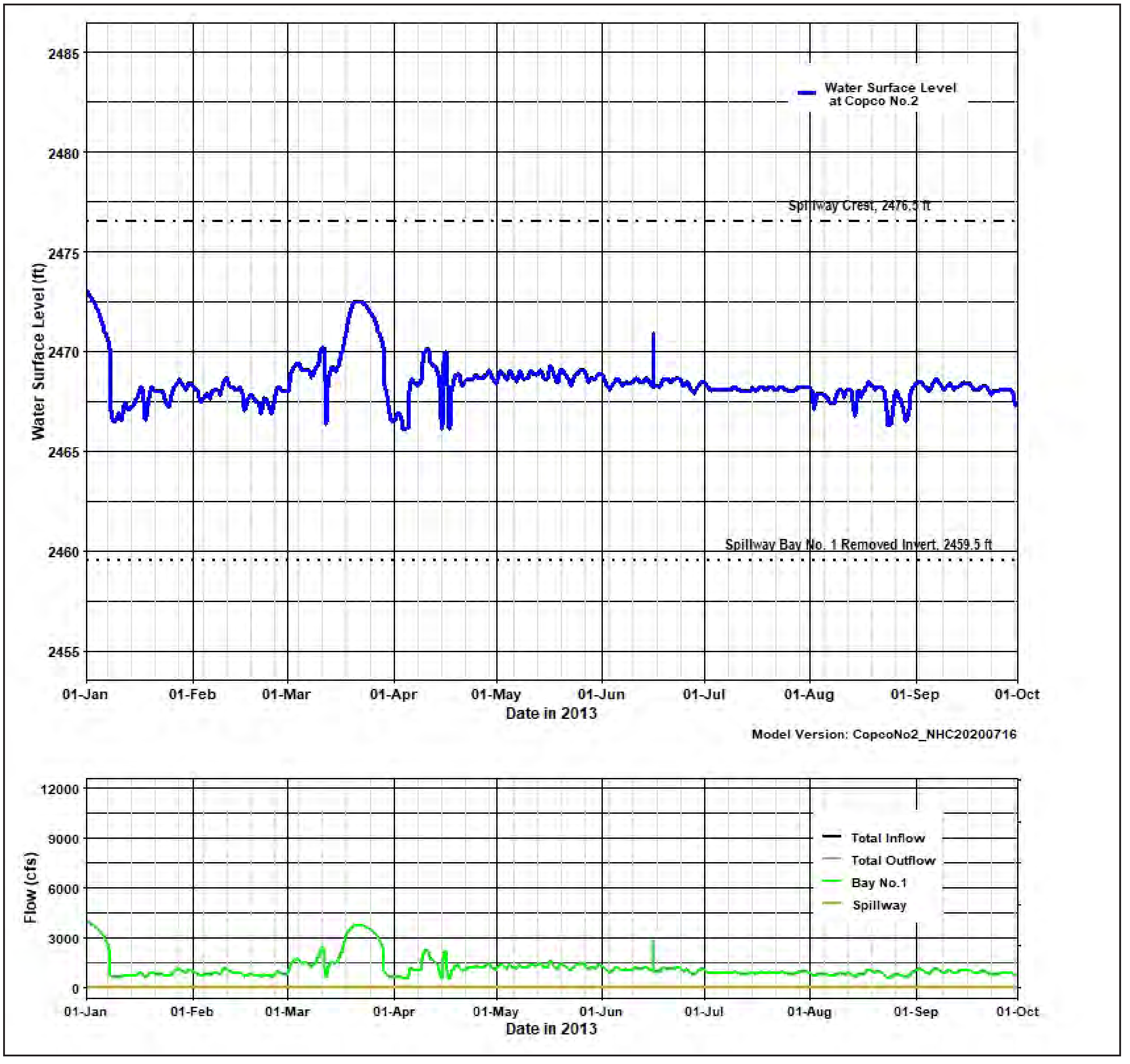


**Figure 33 -** COPCO No.2 Facility Simulated Drawdown - Year 2013

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

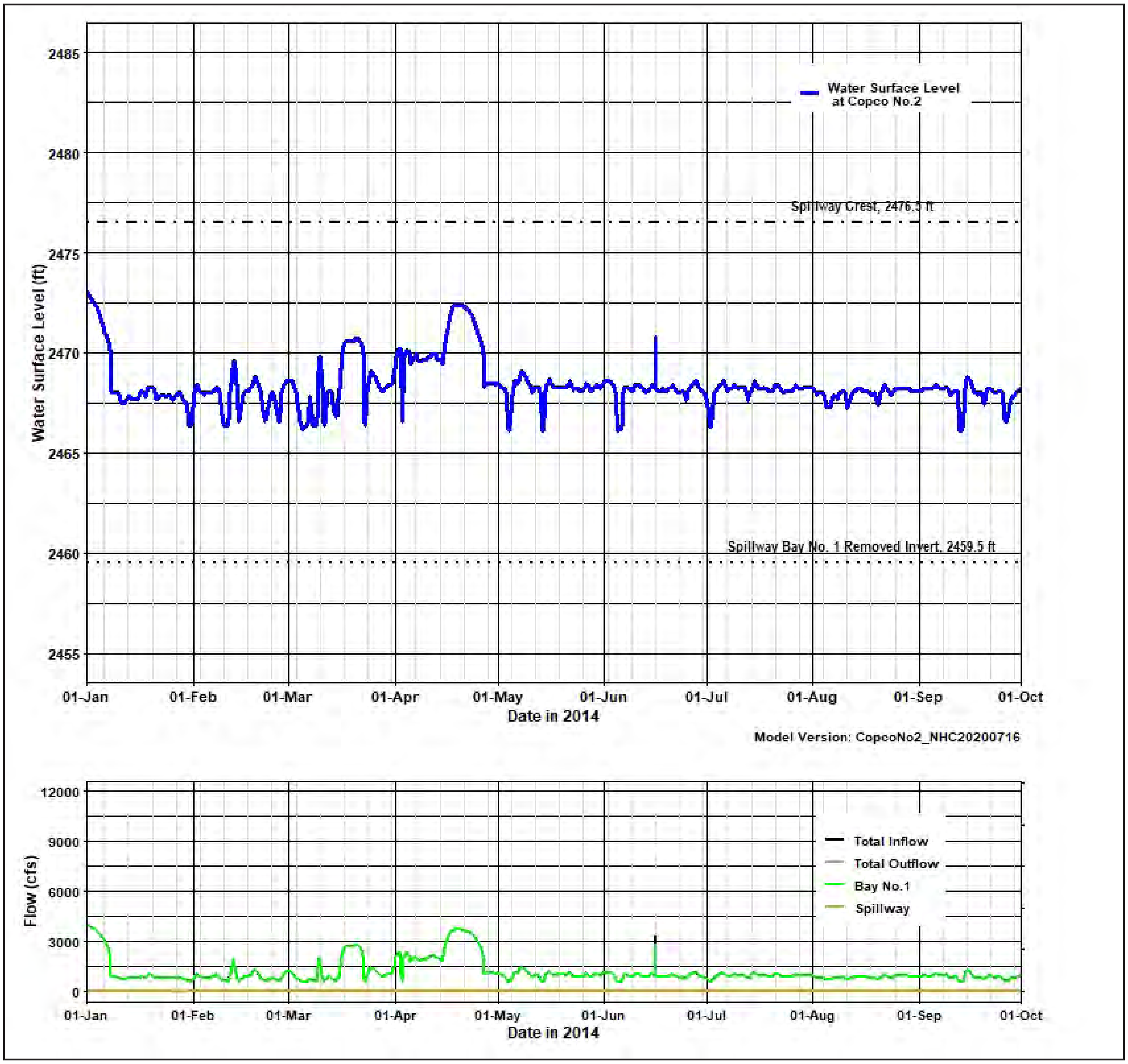


**Figure 34 -** COPCO No.2 Facility Simulated Drawdown - Year 2014

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

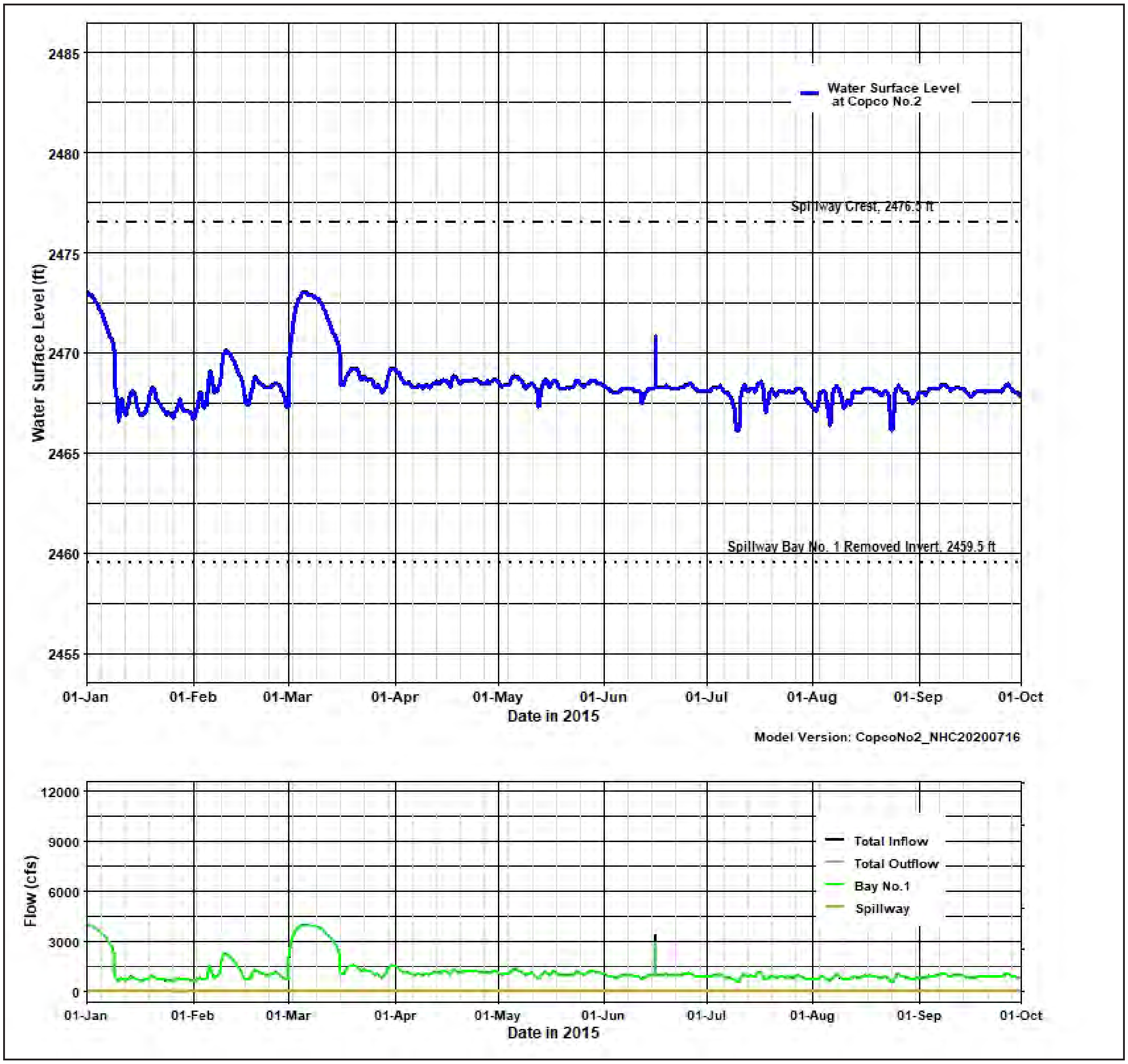


**Figure 35 -** COPCO No.2 Facility Simulated Drawdown - Year 2015

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

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# COPCO NO.2 FACILITY SIMULATED DRAWDOWN

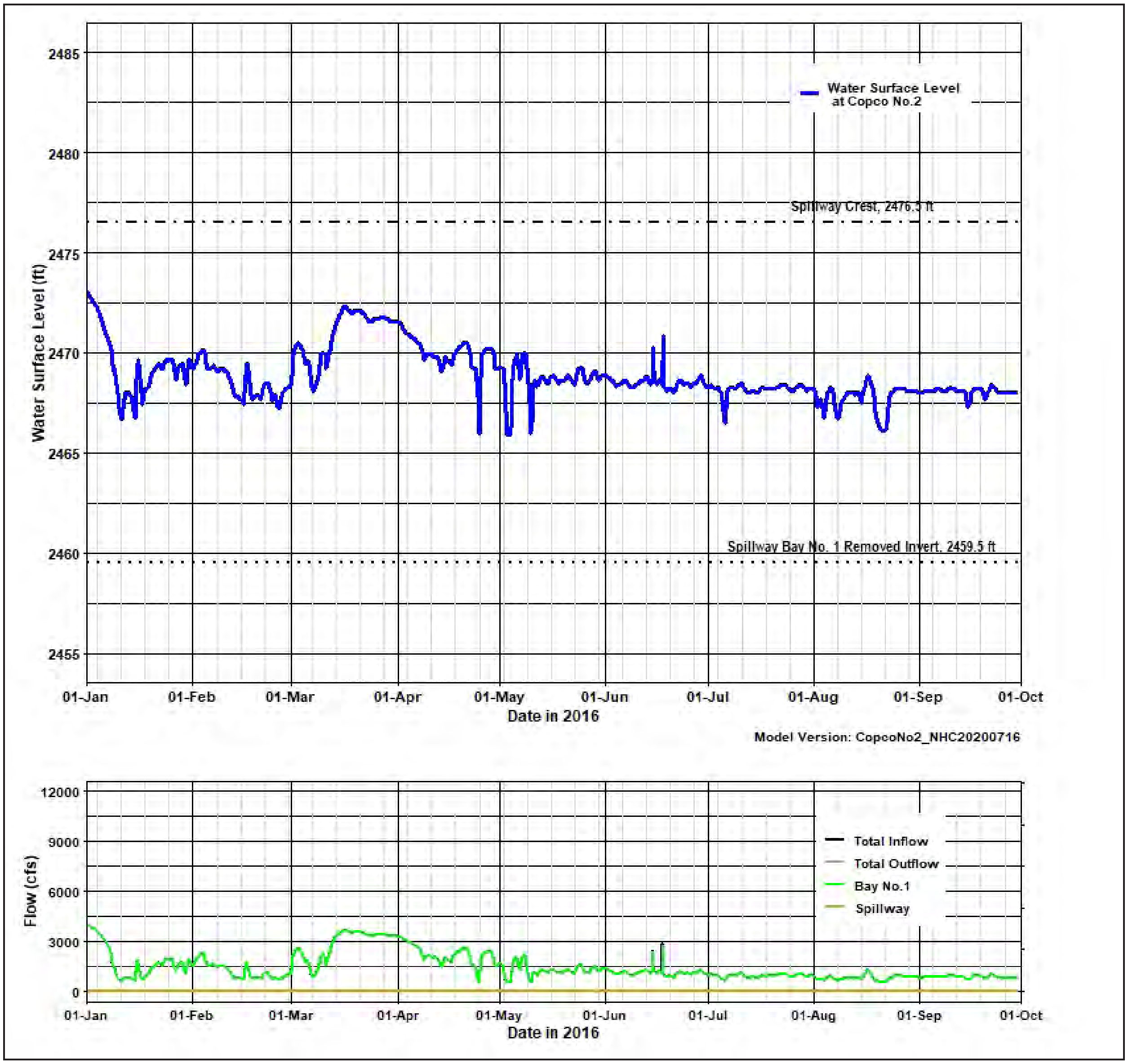



**Figure 36 -** COPCO No.2 Facility Simulated Drawdown - Year 2016

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo2_NHC20200716\individual_years

**ATTACHMENT A**

**100% FINAL Design Report_Appendix D1_May 28**

**Pages 73 and 74 of 74**

**REDACTED: Pages 73 and 74 of 100% FINAL Design Report_Appendix D1_May 28 consist in their 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).**

12787 Gateway Drive S. | Seattle, WA 98168 | 206.241.6000 | www.nhcweb.com

nhc
northwest hydraulic consultants

NHC Ref. No. 2004947.11.6

21 September 2020

**Knight Piésold | KRRP Project Office**
4650 Business Center Drive
Fairfield, California, USA, 94534

**Attention: Norm Bishop**

**Re: CFD Modeling of Copco No. 2 – 100% Design – Contingency Removal Method**

## DRAWDOWN OPERATIONS – 100% DESIGN – CONTINGENCY REMOVAL METHOD

The contingency removal method for the Copco No. 2 diversion dam includes three phases:

- Pre-Drawdown: Open radial gates fully to 11 feet, construct a temporary work platform downstream of the spillway and partial removal of ogee;
- Spillway Bay No. 1 Removal: Remove the concrete spillway crest at Bay No. 1 and drawdown reservoir; and,
- Diversion Dam Removal: Remove Spillway Bays No. 2 through 5.

## COMPUTATIONAL FLUID DYNAMICS MODELING

NHC conducted computational fluid dynamics (CFD) modeling of the Copco No. 2 Dam outlet facilities to verify spillway and Bay No. 1 conveyance capacities for a range of inflows and support the planning and designing of the contingency removal method. The Copco No. 2 spillway rating curve generated by the CFD modeling was used as input data for NHC's 100% design one-dimensional (1D) HEC-RAS drawdown model. The following conditions were simulated:

1. Pre-drawdown: Annual 1% probable flood flow of 29,400 cfs with five spillway gates fully open to 11 feet to verify headwater and tailwater elevations during the annual 1% probable flood flow.
2. Spillway Bay No. 1 Removal: Inflows vary from 29,400 cfs to 1,000 cfs with Spillway Bay No. 1 removed and the remaining four spillway gates fully open. These simulations are intended to determine a drawdown rating curve and velocities downstream of Bay 1 to evaluate erosion potential.
3. Diversion Dam Removal: 20% monthly peak inflows with Bay No. 1 excavated. These simulations are intended to determine headpond and tailrace elevations to support design of the temporary work platform during the removal of Spillway Bays No. 2 through 5.

## GEOMETRY AND ROUGHNESS

The model terrain includes topo-bathymetric data (GMA, 2018) of the Copco No. 2 Reservoir and Outlet Channel. Also included were the spillway, radial gates and earth embankment per the 1925 plans (Reference Drawings 3650, 3721, 3746, 3747, 3748, 3749, 3928, and 3930). All project elevations were converted from the project datum to NAVD88 using a +2,214.5 ft conversion. Three geometries were generated for CFD modeling of each project phase:

1. The Pre-Drawdown Copco No. 2 spillway comprised five 26-ft wide bays with crest at El. 2,476.5 ft and equipped with radial gates with a full opening height of 11 ft (fully open gate lip at El. 2,487.5 ft). The top elevation of the spillway deck was at El. 2,496.5 ft.
2. For Spillway Bay No. 1 Removal, the ogee spillway crest of Bay No. 1 was removed down to El. 2,459.5 ft (deck and gates were not included as they do not impact the flow).
3. For Diversion Dam Removal, the tailrace channel downstream of Bay No. 1 was excavated to reduce tailwater levels around the temporary work platform.

All simulations assumed the gates were fully open or removed and that the intake gate was closed, and no flow was diverted to the powerhouse (intake and powerhouse geometry were not included in the model).

The CFD model uses roughness height to evaluate friction losses. The roughness height for the terrain surrounding the structures was set to 3 ft, equivalent to Manning's n of approximately 0.04 to 0.05, depending on depth of flow. Roughness heights for the dam and work platform were set to 0.002 ft and 1 ft, respectively, based on the expected materials. Model sensitivity to roughness height was not tested as flow capacity is controlled by the spillway structure.

## MESH INDEPENDENCE

The 3D mesh was developed to balance model accuracy with computation time. Decreasing the mesh cell size increases model accuracy and computation time, though there is a mesh resolution for which additional refinement does not yield significant changes in the solution. Table 1 summarizes the results of spillway capacity sensitivity to mesh cell size. The 1-foot cell size around the spillway was considered optimal for further simulations. The 3D mesh used for subsequent simulations included 2-foot elements through the reservoir with a 1-foot refinement region around the spillway.

**Table 1. Mesh Independence Comparison - Spillway Capacity**

| Simulation | Spillway Cell Size (ft) | # cells (million) | CPU Time (h) | Discharge (cfs) | Relative difference |
|---|---|---|---|---|---|
| Pre-drawdown (5 gates) | 1.0 | 2.31 | 5 | 29,000 | - |
| Pre-drawdown (5 gates) | 0.5 | 2.76 | 12 | 29,200 | 0.7% |

## PRE-DRAWDOWN RESULTS

One simulation was run to verify the existing (pre-drawdown) Copco No. 2 Dam design capacity with the annual 1% probable flood flow of 29,400 cfs to the reservoir. The simulation considered conveyance through all five bays with the radial gates fully open to 11 feet and assumed no flow was diverted through the penstock/powerhouse.

Figures 1 and 2 illustrate water surface elevations for this simulation. Average headwater, taken approximately five feet upstream of the dam face, and tailwater elevations at approximately fifty feet downstream of the dam face, were at El. 2,494.8 ft and El. 2,480.1 ft, respectively. Although flow touched the lower lip of the fully open gate (El. 2,487.5 ft), the dam was capable of fully passing the annual 1% probable flood inflow to the reservoir without overtopping the deck (top El. 2,496.5 ft.).

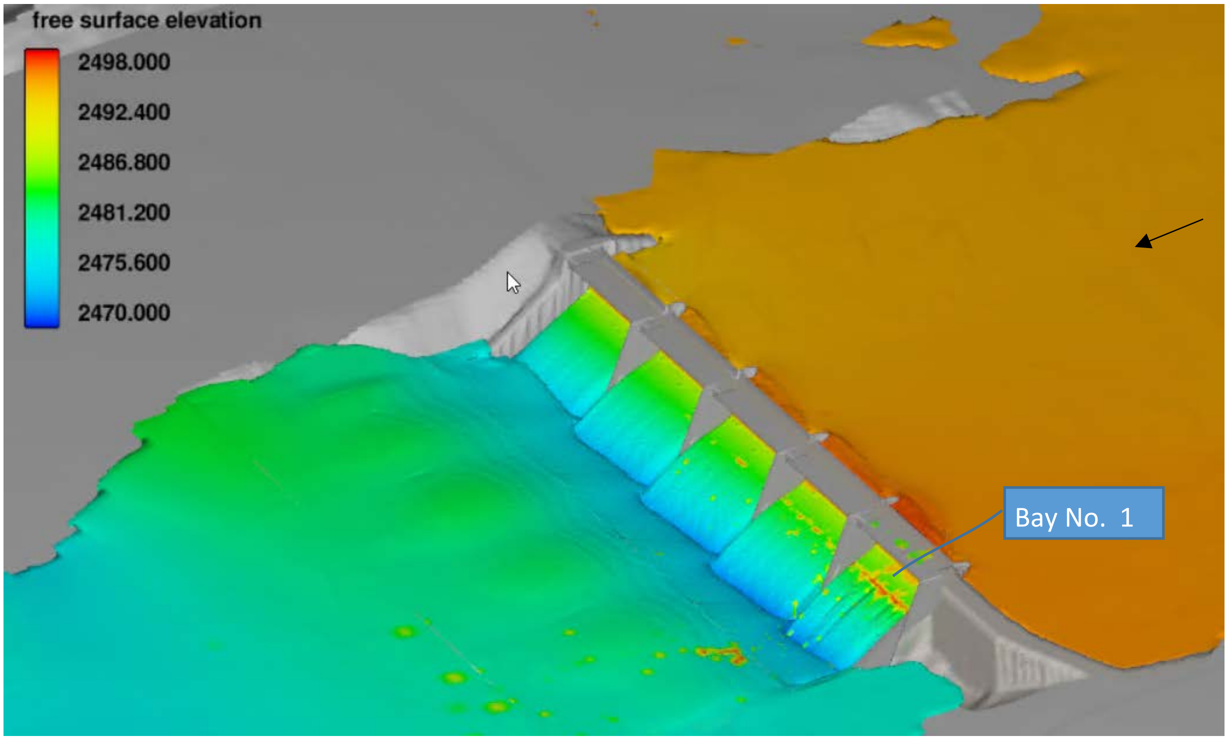



**Figure 1. Water Surface Elevation (ft) for a discharge of 29,400 cfs with five gates fully open.**

Figure 2 illustrates variations in water surface elevations both upstream and downstream of the spillway. The modeling showed that the distribution of flow was concentrated to the left side of the spillway near Bay No. 1, upstream of the spillway, which generated higher head to the left. The terrain downstream of the spillway was higher on the right side of the tailrace as compared to the left, which caused slightly higher tailwater levels near the right bank.

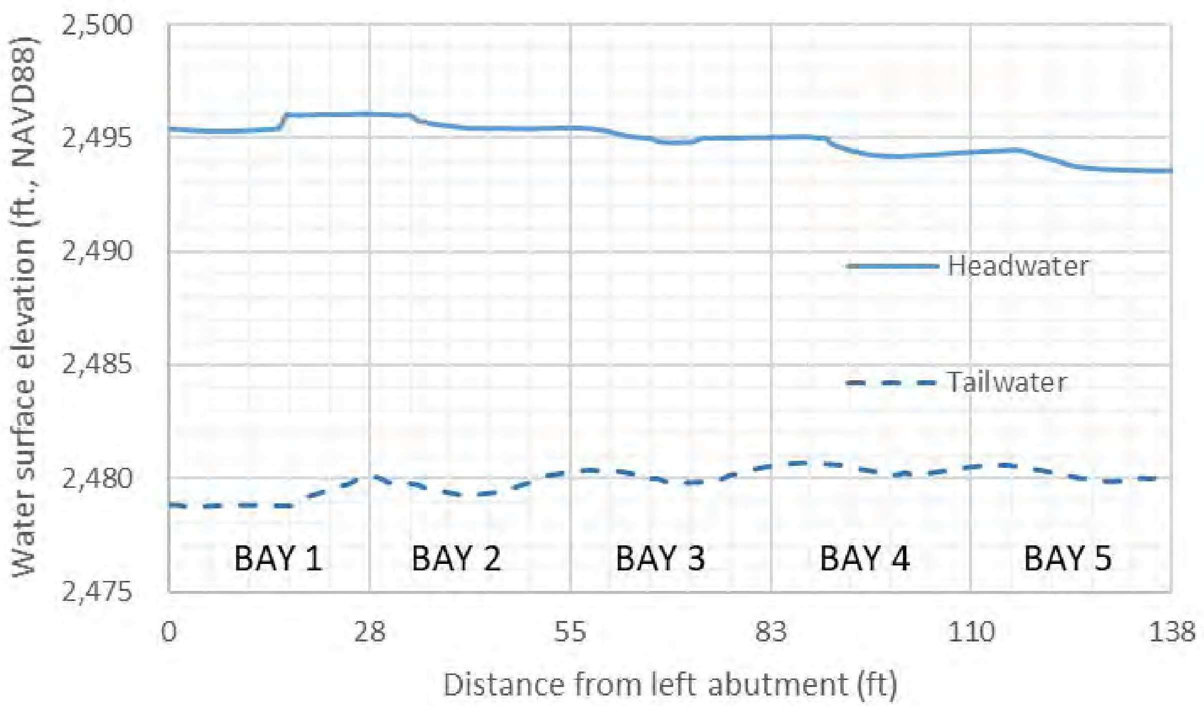



**Figure 2. Pre-Drawdown headwater and tailwater elevations for a discharge of 29,400 cfs with five gates fully open (view looking downstream).**

## SPILLWAY BAY NO. 1 REMOVAL RESULTS

Seven simulations were conducted to develop a rating curve for the dam with Bay No. 1 removed. The ogee crest spillway in Bay No. 1 was removed down to the invert elevation of the concrete stilling basin at El. 2,459.5 ft. The end sill at El. 2,462.4 ft was also removed from Bay No. 1 but retained for the four remaining bays. The temporary work platform to be placed on the stilling basin was not included in these simulations.

Figure 3 shows near-bed velocities at the dam during an inflow of 29,400 cfs. Maximum velocity over the concrete surface reached 30 ft/s on Spillway Bays 2 to 5 and 21 ft/s on Bay No. 1. Downstream from the spillway, velocities on the riverbed were typically around 6 to 7 ft/s.

Table 2 summarizes the simulation results, including average velocities over the water column (i.e. depth-averaged velocities). Reported depth-averaged velocities were at a location approximately 34 feet downstream of the dam face and headpond water surface elevations were at a location 100 feet upstream of the dam face and aligned with the centerline of the pier between Bay No.1 and Bay No. 2. Depth-averaged velocities ranged from approximately 6 to 27 ft/s depending on discharge. Within five feet of the left abutment, depth averaged velocities ranged from approximately 3 to 24 ft/s. Tailwater levels were reported right downstream of the concrete stilling basin and represent average values across the five bays.

**Table 2. Simulated Drawdown Conditions**

| Inflow (cfs) | Headpond El. (ft, NAVD88) | Bay No. 1 Velocity (ft/s) | Left Abutment Velocity (ft/s) | Average Tailwater Level Downstream of Bays No. 1-5 (ft, NAVD88) |
|---|---|---|---|---|
| 29,400 | 2,486.7 | 27.2 | 23.8 | 2,480.7 |
| 20,000 | 2,483.7 | 25.2 | 22.3 | 2,477.8 |
| 15,000 | 2,481.8 | 24.6 | 22.1 | 2,475.8 |
| 10,000 | 2,479.6 | 25.0 | 22.0 | 2,472.8 |
| 7,000 | 2,477.7 | 24.7 | 21.5 | 2,470.4 |
| 3,500 | 2,471.5 | 15.0 | 13.4 | 2,468.6 |
| 1,000 | 2,468.4 | 6.5 | 3.5 | 2,466.9 |

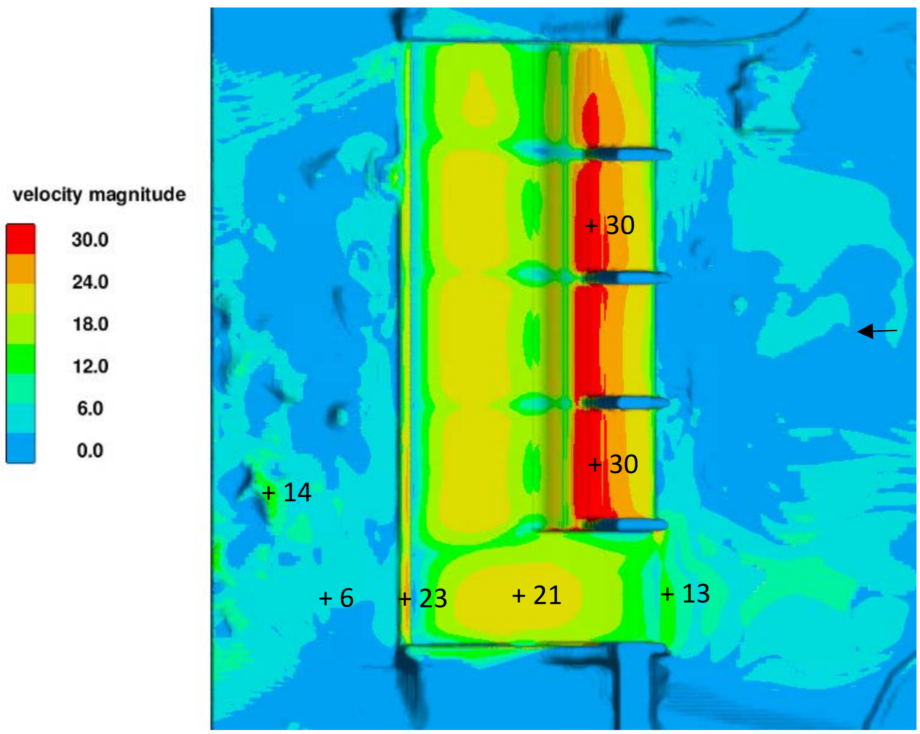



**Figure 3. Near-bed maximum velocity during drawdown for 29,400 cfs. Crosses indicate point velocity in ft/s.**

The gates and deck were not included in these simulations as they do not obstruct the flow. The Spillway Bay No. 1 Removal lowered the maximum headwater elevation during the annual 1% probable flood by 8.1 ft to El. 2,486.7 ft, and resulted in approximately one foot of headroom below the lip of the fully open gate (El. 2,487.5 ft). Under these conditions, and assuming approach velocity head was negligible in the equation $Q = Cs*b*H^{1.5}$, the weir discharge coefficient (Cs) for flow through Bay No. 1 was calculated to range from 3.0 to 3.5 $ft^{0.5}/s$, while the spillway discharge coefficient had an average value of Cs = 4.7 $ft^{0.5}/s$ and Cd = 0.80. NHC noted that if the velocity head was considered, then the discharge coefficient would be lower with a Cd = 0.67; this was not expected to change results of the rating curve as the rating curve was referenced only to a specific water level upstream.

## DIVERSION DAM REMOVAL RESULTS

Six simulations of a range of 20% monthly peak flows were evaluated to develop the Diversion Dam removal rating curve and to support the design of the temporary downstream work platform, which was initially assumed in the CFD model to fill the stilling basin of Bays No. 2 to 5 up to the elevation of the end sill at El. 2,462.4 ft. Approximately 140 feet downstream of the spillway, the wider tailrace pool constricts to a narrow channel opening with an adverse slope rising up to El. 2,463 ft. Since Bay No. 1 will be excavated down to El. 2,459.5 ft, the existing downstream bathymetry was anticipated to cause backwater and inundate the temporary work platform at lower flows. In an attempt to improve conveyance at lower flows, the simulations were evaluated again with the downstream reach excavated with a simple trapezoidal channel (26-foot bottom width, 1:1 side slopes at 1% longitudinal slope) from the spillway apron through the adverse slope reach to existing ground. Figure 4 compares the two geometries.

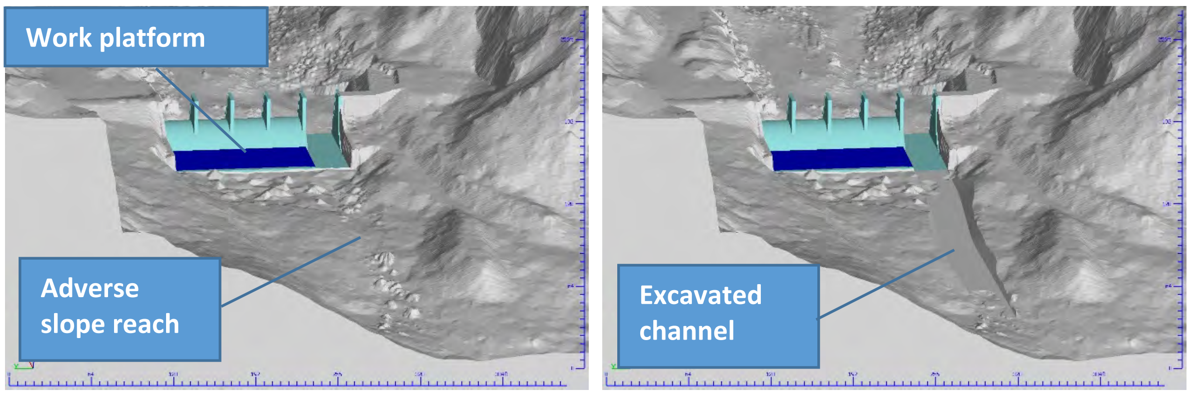



**Figure 4. Geometry comparison (left, existing conditions downstream; right, excavated downstream channel)**

Table 3 shows the results of the Diversion Dam Removal simulations. Headpond water surface elevations were reported approximately 100 feet upstream of the dam face. Tailwater elevations were the average water surface elevation across Bays 2 through 5, approximately 34 feet downstream of the dam face. Excavating the channel downstream caused negligible effects on headpond water levels; but decreased average tailwater levels across the temporary work platform (El. 2462.4 ft) by approximately 2.6 feet, on average.

**Table 3. Simulated Diversion Dam Removal Conditions**

| Inflow (cfs) | Headpond El. (ft, NAVD88) | | Avg. Tailwater El. Bay 2-5 (ft, NAVD88) | |
|---|---|---|---|---|
| | Existing Channel | Excavated Channel | Existing Channel | Excavated Channel |
| 8,500 | 2,478.7 | 2,478.7 | 2,470.6 | 2,468.4 |
| 5,800 | 2,476.6 | 2,476.6 | 2,469.2 | 2,467.4 |
| 4,500 | 2,474.2 | 2,474.0 | 2,468.7 | 2,466.3 |
| 2,900 | 2,470.8 | 2,470.7 | 2,468.2 | 2,465.6 |
| 1,700 | 2,469.6 | 2,469.5 | 2,467.6 | 2,465.1 |
| 800 | 2,468.2 | 2,468.1 | 2,466.6 | 2,462.6 |

This work was used to confirm the impacts of the excavated channel and to inform the required platform elevation of El. 2466 ft.

## WORK PLATFORM WATER LEVEL ANALYSIS

Since the previous analysis showed that even with an excavated downstream channel, a work platform at El. 2,462.4 ft could be flooded by discharges above 800 cfs (Table 3); additional analyses of water levels around the work platform was conducted by raising the work platform to El. 2,466.0 ft in the CFD model to ensure it remained dry under the following discharges:

1. 20% flood for June 16-30 of 1,900 cfs;
2. 20% flood for July/Aug of 1,300 cfs; and,
3. Mean Monthly July flow of 990 cfs.

Plan view plots of results for these discharges and the excavated channel condition are shown in Figure 5 for water surface elevation, Figure 6 for depth-averaged velocity, and Figure 7 for bed shear stress.

For the discharges simulated, the historical cofferdam constricted the flow and controlled water levels upstream, as shown in Figure 5. Figure 6 shows that the local velocity at the historical cofferdam constriction reached up to 18 ft/s for 1,900 cfs.

Flow accelerated when passing through Bay No.1, formed cross waves across Bay No. 1 (Figure 5) and reached velocities up to 18 ft/s for 1,900 cfs (Figure 6).

Figure 7 shows that areas of high bed shear stress coincided with the high-velocity spots shown in Figure 6. The critical shear stress values (in lbf/ft$^2$) needed to initiate the motion of a sediment particle of a given size (in feet) was added to the color scale in Figure 7 to provide an indication of sediment mobility and potential scour.

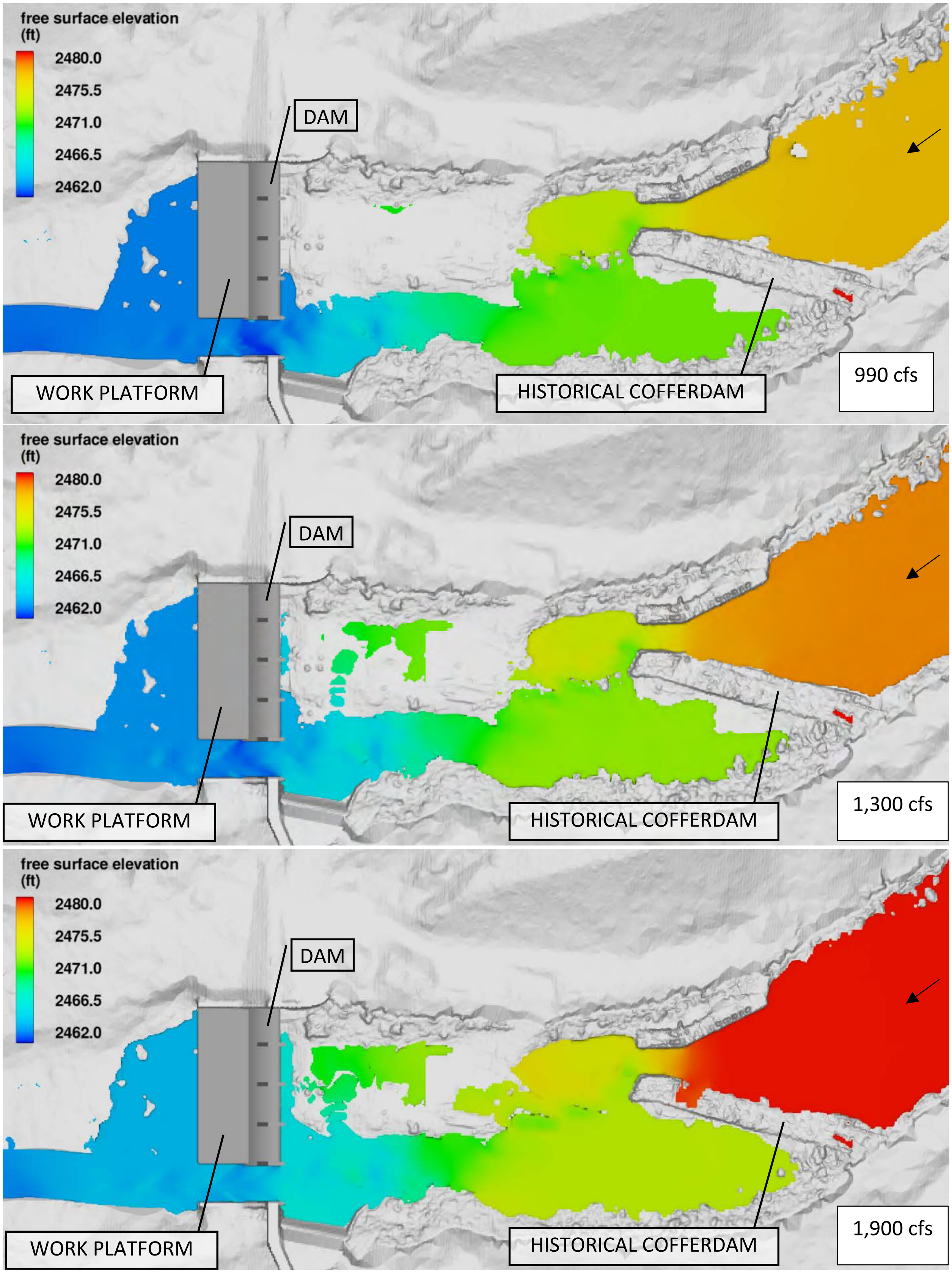



**Figure 5. Water surface elevation for the excavated channel condition and work platform at El. 2,466 ft**

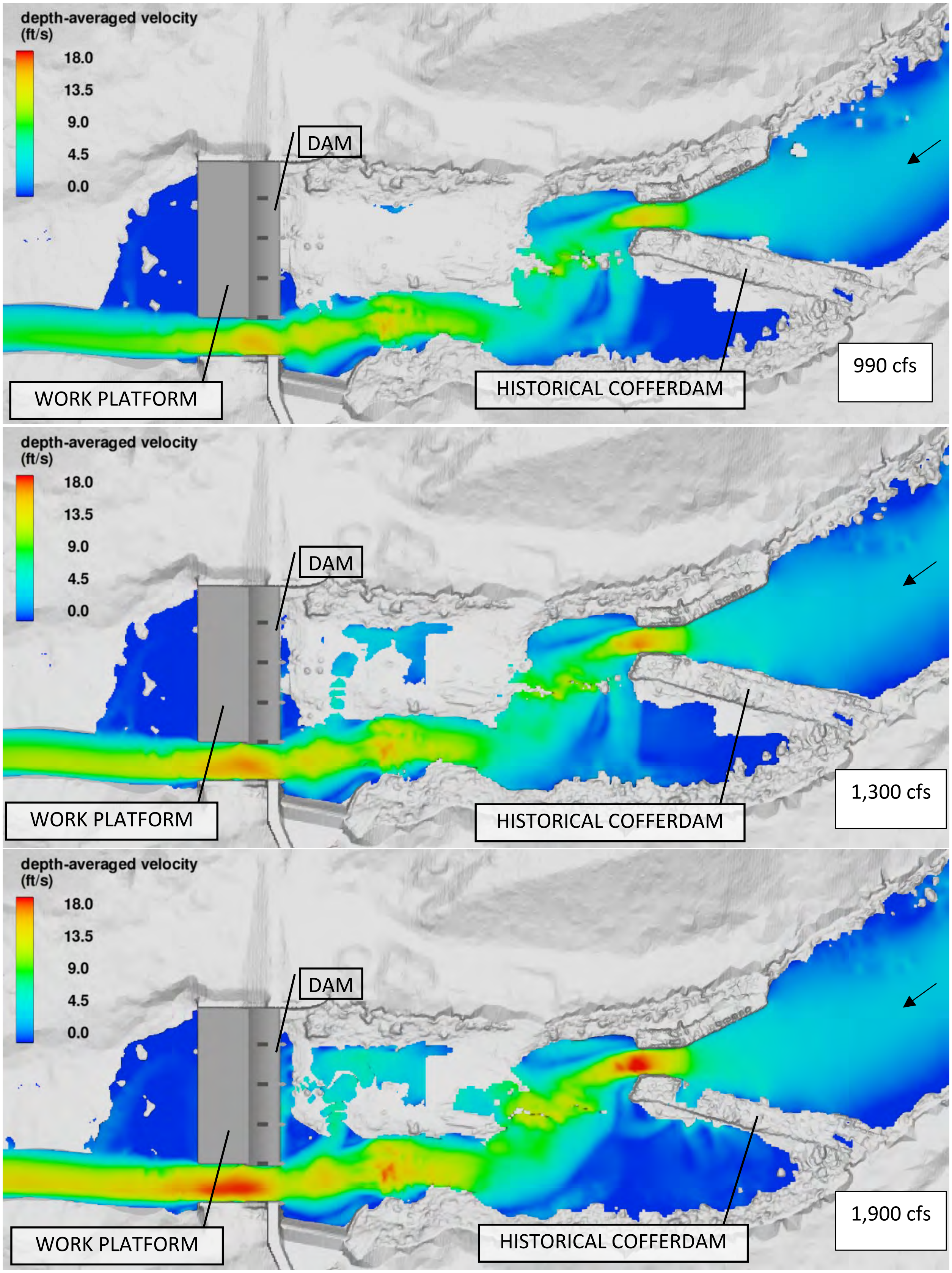



**Figure 6. Depth-averaged velocity for the excavated channel condition and work platform at El. 2,466 ft**

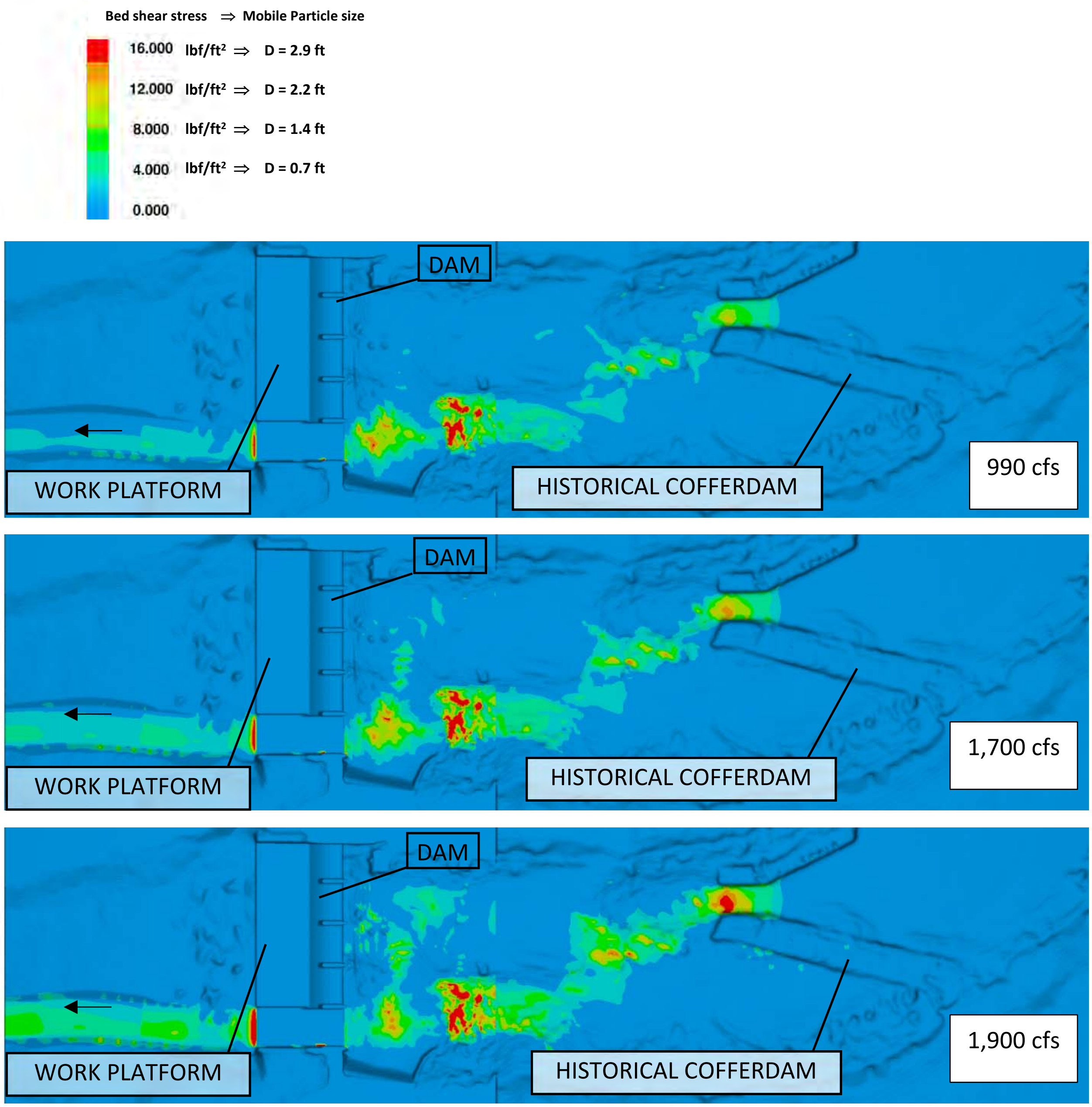



**Figure 7. Bed shear stress for the excavated channel conditions**

More detailed information on water levels and depth-averaged velocities around the work platforms on Bays No. 2 to 5 was extracted along three sections. Figure 8 shows Section A along the edge of the work platform adjacent to Bay 1. The stationing in Section A started at the downstream toe of the pier between Bay 1 and Bay 2 and advanced in the direction of the flow, towards Section B. Section B was located along the downstream edge of the work platform and was oriented towards the right bank. Section C runs parallel to the vertical face of the spillway and is located approximately 5 ft upstream of it, starting on the left abutment wall adjacent to Bay No. 1.

When extracting detailed information along these sections, two additional discharges of 1,800 cfs and 2,200 cfs were included.

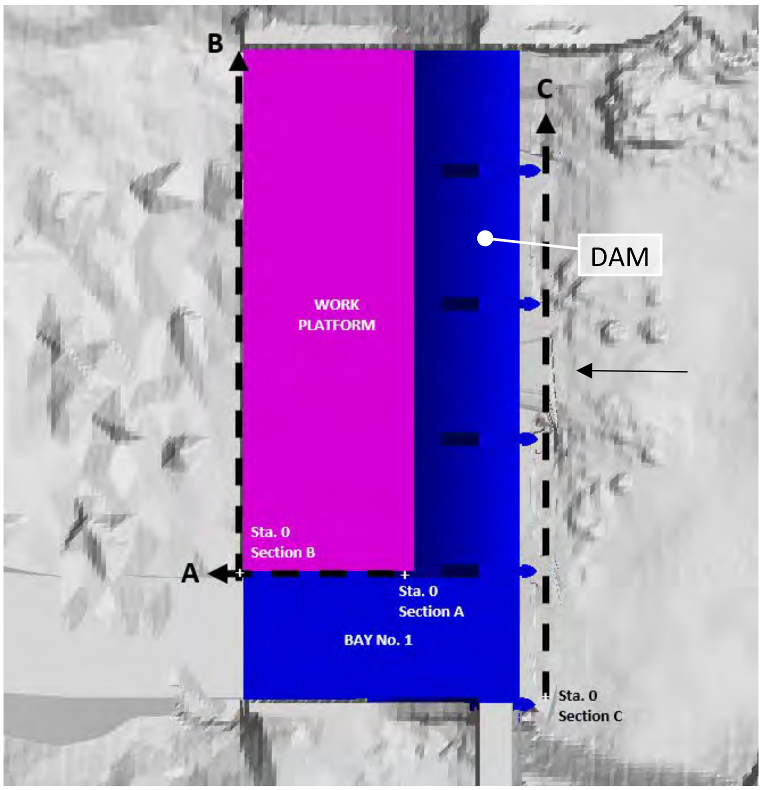
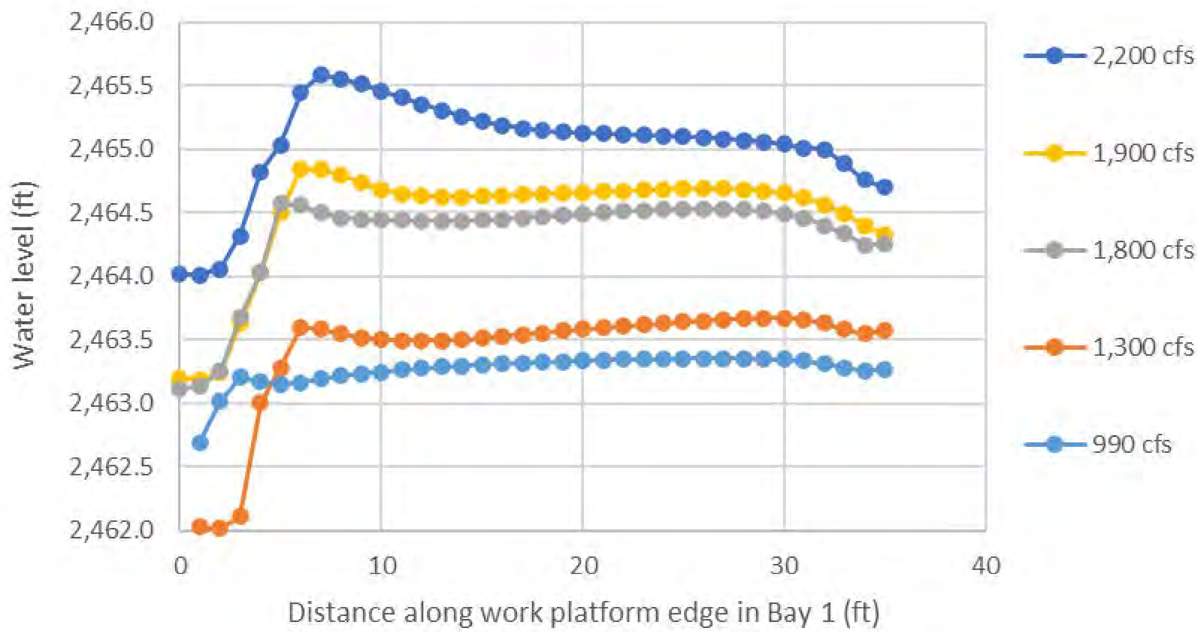



**Figure 8. Sections A, B, and C along perimeter of work platform.**

Profiles of water levels and depth-averaged velocity are shown in Figure 9 for Section A, Figure 10 for Section B and Figure 11 for Section C. Table 4 summarizes the maximum water levels at all sections for each of the five discharges simulated. Along the perimeter of the work platform, the maximum water level reached El. 2,465.6 ft for 2,200 cfs along Section A. Because a pool of almost stagnant water formed downstream of the work platform, water levels along Section B were practically flat, while velocities remained below 1 ft/s except near the corner intersection of Section A and Section B (Figure 10). Water levels and velocities were variable along Section A due to the cross waves formed along Bay No. 1 (Figure 5). Water levels along Section C are affected by the ground topography upstream of the dam (Figure 11).



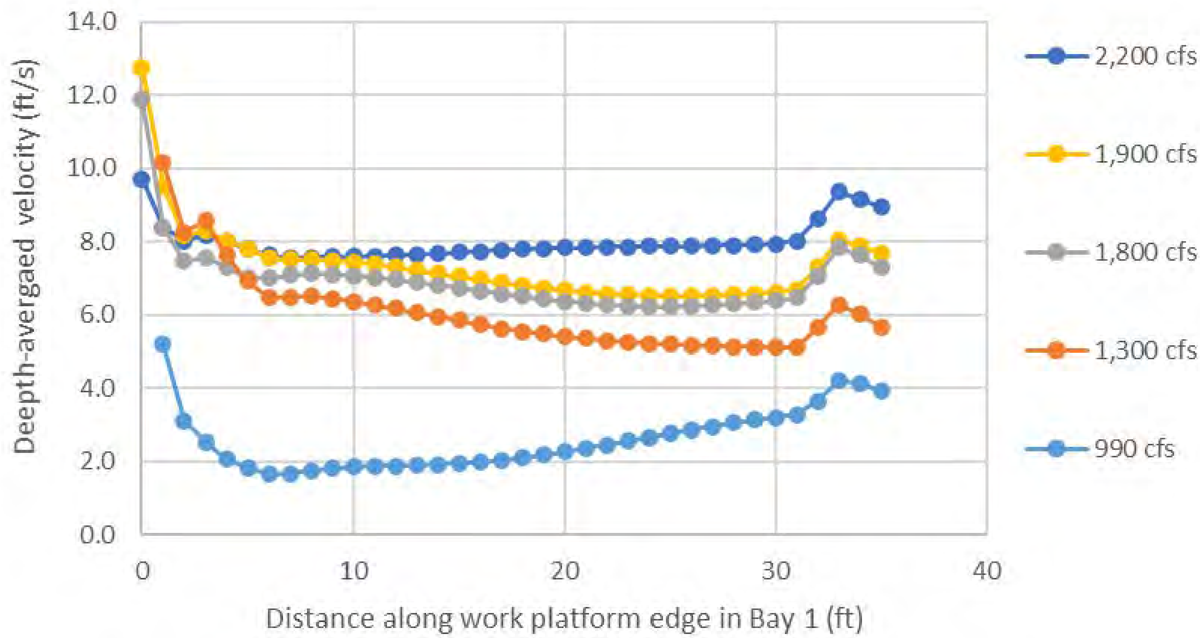




**Figure 9. Water levels and depth-averaged velocity along Section A.**

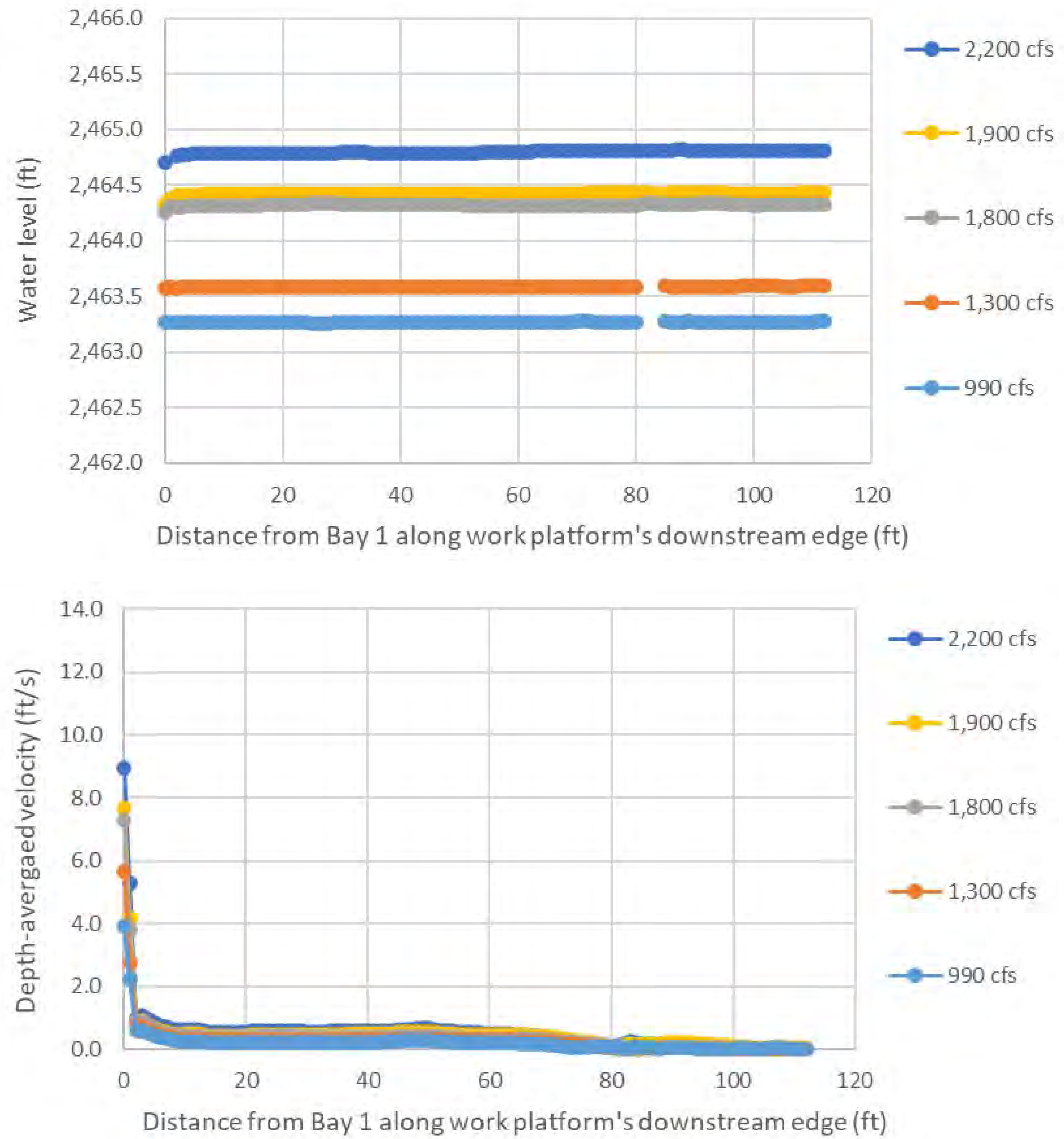
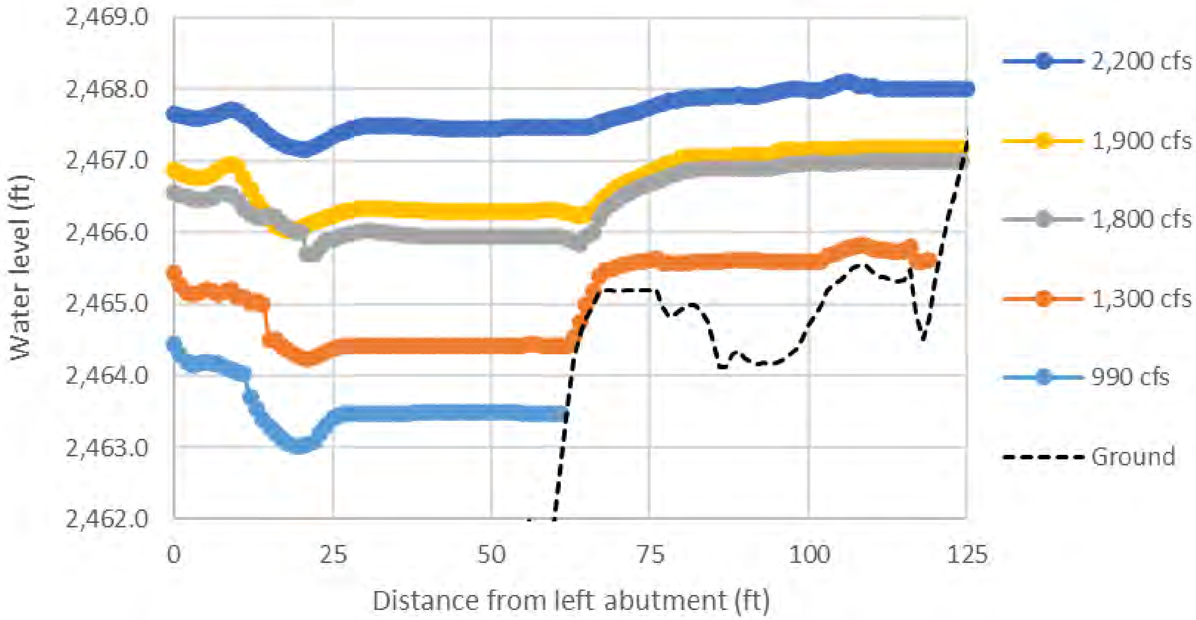



**Figure 10. Water levels and depth-averaged velocity along Section B.**



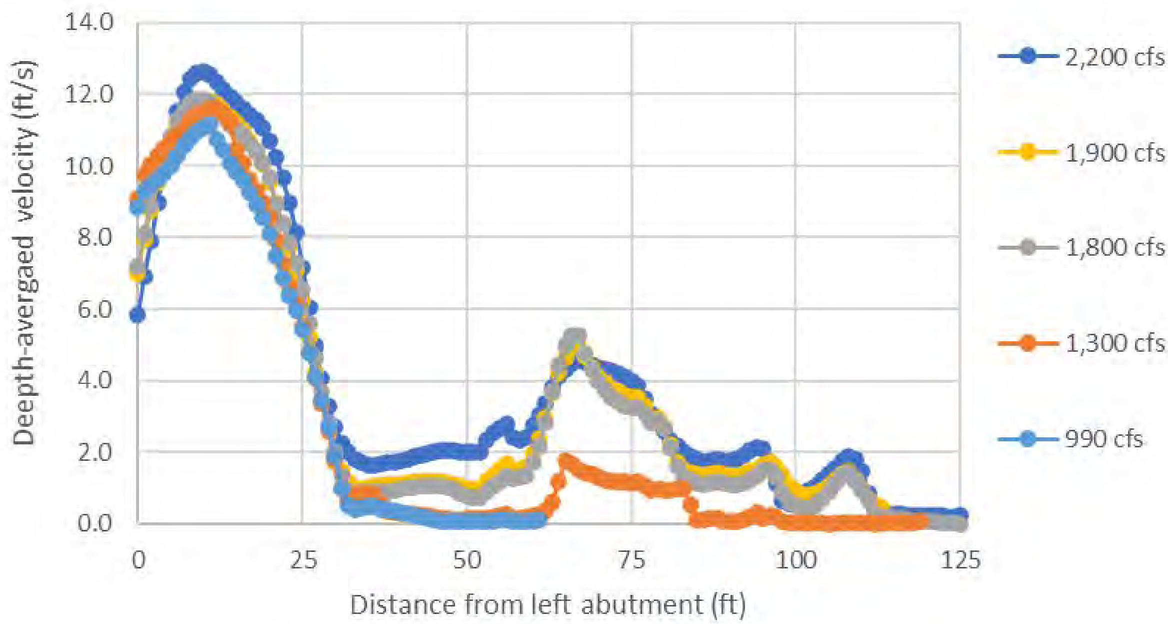




**Figure 11. Water levels and depth-averaged velocity along Section C.**

**Table 4. Maximum water levels around work platform**

| Inflow (cfs) | Maximum water level (ft) | | |
|---|---|---|---|
| | Section A | Section B | Section C |
| 990 | 2,463.4 | 2,463.3 | 2,464.4 |
| 1,300 | 2,463.7 | 2,463.6 | 2,465.8 |
| 1,800 | 2,464.6 | 2,464.3 | 2,467.0 |
| 1,900 | 2,464.8 | 2,464.4 | 2,467.2 |
| 2,200 | 2,465.6 | 2,464.8 | 2,468.1 |

## RATING CURVE

The results of the Spillway Bay No. 1 Removal and Diversion Dam Removal simulations were used to develop the composite headwater rating curve shown in Figure 12, combining flow through Bay No. 1 and over the spillways through Bays No. 2 to No. 5 with the gates fully open. The results plotted in Figure 12 used as input for the 1D drawdown model are also shown in tabular format in Appendix A. The solid blue line in Figure 12 was developed from the CFD results. Since Table 3 demonstrates that downstream channel excavation does not affect headwater levels 100 ft upstream of the dam face, CFD results from both Table 2 and Table 3 are plotted as dots in Figure 12.

The rating curve was zeroed at El. 2,459.5 ft, which is the invert of the Bay No. 1 excavation. However, 100 ft upstream of the dam ground elevation raises to approximately El. 2,464.5 ft. If Bay No. 1 is excavated to a slightly lower elevation, capacity will increase only if the channel is excavated further. The jog in rating curve slope at the spillway crest elevation (El. 2,676.5 ft, discharge above 5,800 cfs) is due to the change in flow regime. For water levels above El. 2,676.5 ft the crest of spillway Bays No. 2 through No. 5 influence flow and the rating curve exhibits typical weir features. Below El. 2,676.5 ft flow is mostly influenced by contraction through the Bay No. 1 opening. The water elevations will become progressively lower than what is presented in Figure 12 as additional spillway bays are removed.

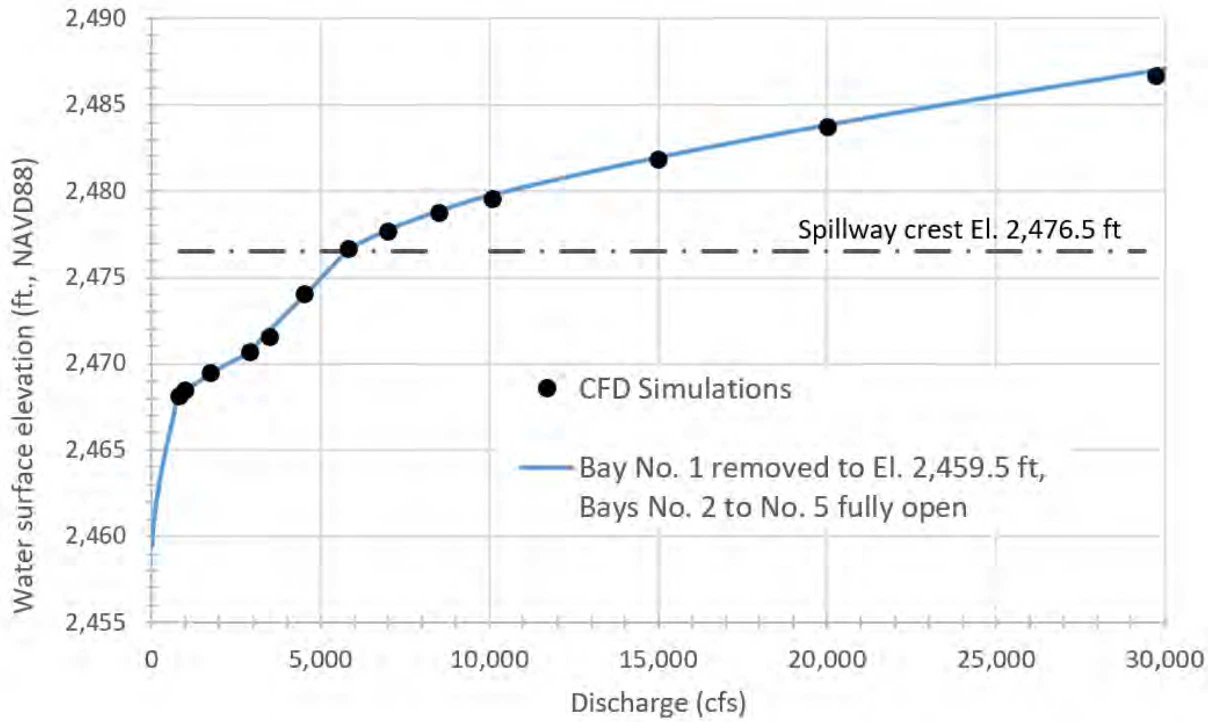



**Figure 12. Copco No.2 Dam - Headwater Rating Curve**

## REFERENCES

GMA Hydrology, Inc. (GMA). 2018. 2018 Klamath Dam Removal Project Topobathymetric Lidar & Sonar Technical Data Report. Project Number 60537920. December 2018.

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## APPENDIX A

### Copco No. 2 Post-Drawdown Rating Curve

| Elevation (ft., NAVD88) | Bay 1 (cfs) | Spillway 4 Gates (cfs) | Total (cfs) |
|---|---|---|---|
| 2,459.5 | 0 | | 0 |
| 2,460 | 11 | | 11 |
| 2,461 | 57 | | 57 |
| 2,462 | 123 | | 123 |
| 2,463 | 204 | | 204 |
| 2,464 | 298 | | 298 |
| 2,465 | 402 | | 402 |
| 2,466 | 517 | | 517 |
| 2,467 | 641 | | 641 |
| 2,468 | 773 | | 773 |
| 2,469 | 1,345 | | 1,345 |
| 2,470 | 2,170 | | 2,170 |
| 2,471 | 3,045 | | 3,045 |
| 2,472 | 3,490 | | 3,490 |
| 2,473 | 3,962 | | 3,962 |
| 2,474 | 4,464 | | 4,464 |
| 2,475 | 4,969 | | 4,969 |
| 2,476 | 5,496 | | 5,496 |
| 2,476.5 | 5,768 | 0 | 5,768 |
| 2,477 | 5,991 | 172 | 6,163 |
| 2,478 | 6,393 | 895 | 7,288 |
| 2,479 | 6,789 | 1,926 | 8,715 |
| 2,480 | 7,314 | 3,190 | 10,504 |
| 2,481 | 8,038 | 4,651 | 12,690 |
| 2,482 | 8,801 | 6,285 | 15,086 |
| 2,483 | 9,603 | 8,074 | 17,678 |
| 2,484 | 10,445 | 10,008 | 20,452 |
| 2,485 | 11,327 | 12,075 | 23,401 |
| 2,486 | 12,249 | 14,267 | 26,516 |
| 2,487 | 13,123 | 16,578 | 29,701 |
| 2,488 | 13,846 | 19,002 | 32,847 |
| 2,489 | 14,581 | 21,533 | 36,114 |
| 2,490 | 15,328 | 24,168 | 39,496 |