APPENDIX C1
COPCO NO. 1 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 operations required and structure evaluations for the dam removal at the Copco No. 1 Hydropower Facility.

Appendix C2 provides a summary of the hydrodynamic CFD modeling, Appendix C3 provides a summary of the stability evaluation, and drawdown modeling results are included as Appendix G.

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

2.1.1 GENERAL

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. Material types are discussed further in Section 4.1.

2.1.2 EXISTING CONCRETE

No records of compressive strength analysis are reported for the concrete of the dam. Construction drawings and photographs from Appendix J and K indicate that the main section of the dam has been constructed of a mixture of concrete and hand placed large stones. Review of the concrete mixes and cement content reported on the construction drawings indicates that the material properties summarized in Table 2.1 are conservative estimates:

- Cement efficiency of 10 psi/lb/cu yd for compressive strength is assumed (ACI, 1996)
- Main section of the dam: compressive strength of 4,000 psi
- Upstream and downstream cutoff wall: compressive strength of 3,000 psi

The construction drawings and photographs indicate the following:

- Wooden formwork was generally supported with tension cables or tie-rods within the forms and structural shoring on the exterior of the forms
- Construction joints were formed with shear keyed sidewalls and roughened by compressed air-drill

Table 2.1 Existing Concrete Material Properties

Properties	Values
Concrete Unconfined Compressive Stress	4,000 psi / 3,000 psi
Concrete Tensile Strength – Static [1]	430 psi
Concrete Tensile Strength – Dynamic [2]	640 psi

NOTES:
1. STATIC TENSILE STRENGTH IS BASED ON SPLITTING TENSILE TEST STUDIES BY RAPHAEL (1984).
2. DYNAMIC TENSILE STRENGTH IS BASED ON ACI (1996) GUIDELINES.

2.1.3 EXISTING REINFORCING STEEL

Construction drawings and photographs indicate that reinforcing steel used during construction is generally comprised of 30-pound rails and square bars.

- Concrete placement was completed in smaller concrete sections within the main section of the dam. Each concrete block is reinforced with 30-pound rails projecting on each construction joint face. Horizontal rails are placed at approximately 8 ft center to center and vertical rails are placed at approximately 12 ft center to center.
- The upper cutoff wall appears to have been reinforced with one layer of horizontal and vertical 30-pound rails at 4 ft centers.
- The spillway piers, deck and other sections requiring more complex shapes have been reinforced with 0.75 inches to 1.25 inches square bars.

The yield strength of existing reinforcing steel is not indicated in the construction drawings and is assumed to be 27 ksi in accordance with ACI 562 (2016).

2.2 LOW-LEVEL OUTLET

2.2.1 GEOMETRY

The low-level outlet will be constructed through the concrete dam as a 10.5 ft high and 10.5 ft wide D-shape tunnel with vertical sides, as shown on Drawings C2205 and C2225. The shape and profile is selected to facilitate construction, reduce stresses acting on the crown of the tunnel and to provide the internal cross section required to discharge the design flows. The outlet tunnel will be separated from the reservoir by a 10 ft orifice plug left in place until drawdown is initiated. The outlet tunnel will be sloped at a 10% grade to promote clearing of the concrete plug debris and sediment passage of bed material during the reservoir drawdown operation.

2.2.2 TUNNEL EXCAVATION

The excavation of the low-level outlet tunnel will terminate in the upper cutoff wall to leave a concrete plug separating the dry tunnel and the reservoir, as shown on Drawing C2225. Construction photos indicate that horizontal 30-pound rail dowels and downstream face reinforcement were used for this member. Further identification from historical design drawings found in Appendix K, and removal of the steel reinforcement will be required to ensure that the removal of the concrete plug is successful.

Probe drilling requirements during the tunnel excavation will depend on the homogeneity of the concrete. It is anticipated that at a minimum, the upper cutoff wall will be investigated by probe drilling to confirm the following design characteristics:

- Geometry of the upstream face
- Compressive strength of the concrete
- Water conditions
- Presence and location of concrete reinforcing bars

The concrete plug is designed as a monolithic unreinforced concrete structure sized to provide the plug length required for structural strength and a contingency length to account for possible fractures in the concrete mass induced by vibration during the excavation of the tunnel. The structural design is in compliance with the unreinforced concrete design guidelines of ACI 318 for the following loading conditions representative of the operation of the facility prior to the drawdown:

- Pre-drilled concrete plug
- Normal reservoir operation
- Normal reservoir operation level including loading resulting from the OBE seismic event

2.2.2.1 TUNNEL PLUG REMOVAL

The low-level outlet tunnel will be opened by removing the concrete plug by drilling and blasting from the dry downstream side of the tunnel. Access to remove the concrete plug is only possible from the outlet conduit vent pipe and will require predrilling of the blast holes at the time of the tunnel construction. The holes will be loaded and blasted on or after January 1 of the drawdown year. As the plug is blasted the difference in hydrostatic pressure from the reservoir side and the dry side will result in a surge wave propagating through the sloped tunnel entraining all concrete debris. Concrete debris will settle in the plunge pool. Complete opening of the plug is required to ensure the hydraulic characteristics of the low-level outlet are achieved.

The blast design, charging and detonation system will be completed by the Project Company and will require a formal submittal for review and approval.

2.2.2.2 UNLINED TUNNEL

The unlined tunnel's surface roughness is not critical to the hydraulic performance of the outlet. During operation, high-velocity flow and water containing sand or gravel will increase the potential for invert erosion. The probability of erosion of the tunnel invert is a function of the quality of the concrete and flow properties. The erodibility index of the existing concrete at the Copco No. 1 dam is estimated to vary between 6,400 for concrete with compressive strength of 3,000 psi and 11,520 for concrete with compressive strength of 4,000 psi. The maximum stream power density for the hydraulic conditions described in Section 3.0 is computed to be 20 HP/ft^2. The conditions evaluated are below the threshold of 1% probability of erosion as defined in the USACE guideline (USACE, 2015). Invert erosion is possible if the available stream power is greater than 34 HP/ft^2 at the erodibility index of 6,400.

The hydraulic conditions described in Section 3.0 indicate that cavitation potential is not critical. The following conditions will occur during the operation of the outlet:

- High velocity flow through the outlet: cavitation number is above 0.3.

The D-shape to circular concrete transition is designed to limit the cavitation potential in accordance with USBR guidelines (USBR, 2014).

2.2.2.3 DAM SAFETY

The dam stability evaluation with the low-level outlet has been analyzed using a three-dimensional finite element model to analyze the main center section. The analysis and evaluation indicate that the stresses due to the potential failure modes (PFM) (STID PacifiCorp, 2015a) loads and loading condition will be similarly low as identified by the stability analysis with the current dam arrangement and that little or no damage to the dam is expected during the PFM conditions.

The stability evaluation is summarized in Appendix C3.

2.2.3 OUTLET CONDUIT

The low-level outlet will have a circular 10.5 ft diameter steel outlet conduit extending from the D-shape tunnel to the spillway plunge pool, as shown on Drawing C2227. The outlet conduit will be protected by an earthfill apron allowing access to the existing spillway face during drawdown and diversion. In the event of a flood requiring the spillway to operate, the earthfill apron will be subjected to the hydraulic forces from the existing spillway stepped chute discharge.

The inclusion of the steel outlet conduit is not necessary to complete the reservoir drawdown. The Project Company desires access from the right abutment to the left abutment to access the downstream side of the historic diversion tunnel, and as such the plan is to install a 10.5 ft-diameter conduit through the spillway apron. The CFD simulations indicate that the hydraulic conditions of the operation of the low-level outlet are inlet controlled at all reservoir water surface levels, thus the inclusion or exclusion of the steel outlet conduit does not affect drawdown flow rates.

2.2.3.1 STEEL OUTLET CONDUIT

The steel outlet conduit is designed considering the ASCE (2012) Steel Penstock Design Guidelines. Reduced serviceability parameters are considered for the fabrication of the conduit and manhole as the usage intent is operating under no or low-pressure conditions and the design life of the steel conduit is short. Pressure vessel quality steels, with higher degree of uniformity in metallurgy are not required as the conduit will not be subjected to pressure fluctuations. Due to short service life, the steel components are specified as unlined and uncoated steel. The large diameter pipe is specified to be fabricated with steel plates. The material specifications can be substituted for steel materials with equivalent yield and tensile strength (i.e. ASTM pipe, API pipe, ASME flanges). The steel outlet conduit typical section is shown on Drawing C2227.

The hydraulics of the low-level outlet function acceptably with and without the steel conduit. The steel conduit affords the ability to access the downstream historic diversion tunnel portal during the pre-drawdown year and during reservoir drawdown. Should the Project Company provide an alternate access method or route to the downstream historic diversion tunnel portal, the Project Company at its option may implement an alternate which involves not installing the steel conduit and reducing the downstream work platform height.

2.2.3.2 AIR DEMAND

The manhole included within the outlet conduit will allow potential negative air pressures induced by the high-velocity flow to be vented to the atmosphere. Air demand CFD simulations indicate that the outlet will flow full if the air demand of the outlet is not satisfied by venting of the air zone between the free-surface flow from the downstream end. Maximum negative pressure along the outlet are located directly downstream of the inlet within the concrete tunnel section and do not exceed -30 psi. No damage to the concrete is expected. Repositioning of the flow's stream and potential vibrations from the variation in air demand can be observed in the following operating conditions:

- During operation of the outlet where the discharge is rapidly varied
- Low discharge and tailwater level where the air zone at the downstream end is restricted
- Manhole is blocked at reservoir levels lower than 2,530 ft

As the reservoir lowers to at or below 2,530 ft, potential adverse air demand conditions in the low-level outlet could result. Under this reduced head condition, air demand is critical between the inlet and the hydraulic jump that develops at the conduit discharge into the tailwater. The manhole is sized to meet this air demand and its air inlet must remain open until the historic diversion tunnel is opened and the reservoir is further lowered. The manhole is sized for the air velocity to be less than 150 ft/s (USACE, 1980). The manhole included within the outlet conduit is shown on Drawing C2228.

2.2.3.3 SPILLWAY APRON

The earthfill apron above the steel outlet conduit is designed for the greater of the surface loads resulting from the construction of the access track for removal of concrete rubble during dam demolition and the traffic load imposed by construction vehicles. The burial depth to resist conduit ovality, through-wall bending, side wall crushing and ring buckling due to the vertical loads is limited by controlling the conduit deflection under the vertical loads in accordance with API RP 1102 and AWWA M11 guidelines.

The pressure applied to the buried conduit by a concentrated surface loads has been evaluated using the Boussinesq method (AWWA, 2017). The allowable steel conduit ring deflection is selected to account for the impact loading of the falling concrete rubble and construction traffic.

The following parameters are considered:

- Maximum drop elevation for concrete rubble: 2,617 ft
- Unit weight of concrete: 150 lb/ft^3
- D50 of the concrete rubble: 36 in
- Impact factor 1.5
- Allowable steel conduit ring deflection of 4%
- Design vehicle – articulated haul truck, Cat 745

The concreted erosion protection lining is specified to protect the earthfill apron. The rigid apron is designed for the dynamic pressures caused by the resulting stresses from the high-velocity flow impacting the apron from the stepped chute and for the uplift forces acting on the concrete apron caused by the high-velocity flow over the apron. The flotation factor of safety when the structure is loaded with the maximum monthly 5% recurrent flood of July to December discharged from the spillway is designed to exceed 1.1. High flood discharge may result in severe damage to the spillway apron fill. This is identified as a construction risk and the spillway apron fill would be rebuilt if damaged.

2.3 HISTORIC DIVERSION TUNNEL

The historic diversion tunnel was used during the original construction of the dam as a free-flow diversion tunnel. An inlet structure with inlet valves was added to close the tunnel. Later a tunnel plug from the downstream end provided the final closure and the inlet structure was abandoned in place in the closed position. The tunnel is presently exhibiting signs of leakage upstream of the concrete plug (KP 2022a). The tunnel plug is leaking pressure flow through two existing small diameter grout pipes.

The diversion tunnel will be reopened to complete reservoir drawdown and provide river flow diversion during dam removal and river channel reclamation.

2.3.1 ROCK COVER ASSESSMENT

The tunnel rock mass is evaluated with the Norwegian Method (USACE, 1997) to assess the adequacy of the existing rock cover above the tunnel crown and adjacent tunnel walls for sustaining the hydrostatic loading within the rock mass when the inside of the tunnel is at atmospheric pressure (open channel flow conditions). The operation of the tunnel for drawdown with reservoir water levels above 2,545 ft is identified to require additional ground support to resist hydro jacking. The operation of the tunnel for river diversion is specified for hydrostatic pressure conditions below the allowable internal stress of the rock mass. The following operating parameters are considered:

- The diversion tunnel is opened once the reservoir is drawn down to elevation 2,530 ft
- The hydraulic control of the diversion tunnel is entrance controlled and the tunnel will flow partially full

2.3.2 HEADWORKS AND PLUG REMOVAL

The diversion tunnel will be re-opened by removing the intake structure under balanced hydraulic head. Removal of construction debris and concrete rubble to fully open the inlet to a minimum elevation of 2,505.8 ft will be required to prevent blockages and allow the drawdown to be completed. Removal of the intake structure under balanced head can commence at any time. It is however anticipated that it will only be possible to remove and clean the approach channel when the reservoir water level is at or below +/-2,530 ft. Heavy equipment access to remove the intake structure is possible from the left bank.

The diversion tunnel plug will be re-opened by removing the concrete plug by drilling and blasting from the downstream side of the plug. Removal of the plug will require drilling probe holes under the reduced reservoir hydrostatic head to determine the thickness of the tunnel plug. The plug drilling pattern will target the removal of concrete anchors and provide concrete fragmentation that will allow for the resulting concrete rubble to be evacuated from the tunnel with the surge of water after plug demolition.

The reopening of the diversion tunnel is designed to be completed without the use of rock traps due to the presence of the existing spillway plunge pool. As the tunnel plug is blasted the difference in hydrostatic pressure from the reservoir side will result in a surge wave removing concrete debris.

The tunnel plug blast design, charging and detonation system will be completed by the Project Company and will require a formal submittal for review and approval.

2.3.3 HISTORIC COFFERDAM

The historic rock filled crib cofferdam (diversion dam) was used during the construction of the main dam. Limited details are available of the cofferdam on the construction drawings and only a few photographs show the cofferdam. This structure is anticipated to be a small gravity concrete structure founded on rock with a minimum crest elevation of 2,515 ft. Once the water levels are diverted through the diversion tunnel, the Project Company will evaluate the local dewatering requirements to allow removal of the Copco concrete dam foundations to the elevations indicated on the drawings.

Subsequent removal of the cofferdam is required to establish the river channel final grade. It is anticipated that the structure will be removed by drilling and blasting during the excavation of the upstream historic construction spoil material. Blasting will require a formal submittal for review and approval.

2.3.4 ACCEPTANCE CRITERIA

The tunnel opening requirements are conservatively designed for the difference between the rock mass and the internal pressure and no ground pressure acting on the tunnel arrangement. The diversion tunnel is designed to have a factor of safety based on the ratio of minimum principal stress to the hydrostatic pressures exceeding 1.1 as per the USACE guidelines on Tunnels and Shaft construction (USACE, 1997). At reservoir level 2,530 ft or lower, the minimum principal stress acceptance criteria are met.

2.3.5 DEBRIS MANAGEMENT

Floating debris control measures will need to be implemented before the drawdown operation commences. PacifiCorp reported that the current debris management measures are successful at reducing the debris load seen at the facility. This includes:

- Debris survey from a boat

- Hand removal of loose woody debris on the shorelines

The use of a temporary debris boom upstream of the diversion tunnel inlet during the river diversion may be needed to prevent debris blockage by floating woody debris. The historic operation of the tunnel included a boom directly upstream of the inlet. Ground conditions will need to be evaluated once the reservoir is sufficiently drawn down to locate anchor points for a debris boom. It is anticipated that the anchorage will consist of grouted rock anchors or concrete gravity anchors. The temporary debris boom will be designed to reuse the facility's intake boom.

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

The depth-area-capacity relationships for the Copco No. 1 reservoir are based on the 2018 bathymetric survey (NAVD88 datum) and are shown on Drawing C2056. The reservoir capacity at elevations relevant to the Copco No. 1 facility are summarized in Table 3.1.

Table 3.1 Reservoir Storage Capacity for Various Elevations

Key Elevation Description	Elevation (ft)	Capacity (acre-ft)
Maximum Reservoir Operating Level	2,611.0	40,700
Normal Reservoir Operating Level	2,607.0	36,334
Normal Minimum Reservoir Operating Level	2,605.0	34,404
Minimum Reservoir Operating Level	2,596.0	26,500
Spillway Crest	2,597.1	27,404
Historic Cofferdam Crest	2,515.0	39
Low-Level Outlet Invert	2,492.5	0

3.2 OUTLET WORKS

3.2.1 DISCHARGE RATING CURVES

Discharges during the drawdown stages will be made through the newly constructed low-level outlet, the historic diversion tunnel, and spillway releases. The development of the discharge rating capacities for the outlets are detailed in Appendix C2. The Copco No. 1 discharge rating curves are presented on Drawing C2056.

The discharge capacity has not been modelled for the blockage condition or variation of discharge capacity due to sediment inflow. The low-level outlet is under greater than 40 ft of head until a reservoir surface level of 2,530 ft is reached. Blockage conditions where large quantities of debris and sediment-laden flow are moving through the outlet may reduce the discharge capacity temporarily, but is expected to be flushed due to the pressure differential between upstream and downstream conditions.

3.2.2 LOW-LEVEL OUTLET

The minimum flow conditions of the low-level outlet are evaluated with CFD simulations as described in Appendix C2. The simulations model the water surface immediately upstream and downstream of the outlet and assume clear water discharges. Discharge characteristics indicate the low-level outlet to be inlet-controlled at all reservoir elevations during drawdown. Flow conditions downstream of the inlet along the tunnel alignment and outlet conduit include the high-velocity flow jet exiting the circular inlet and open-channel flow along the tunnel profile. The CFD simulation indicates that the high-velocity jet passing through the inlet does not impact the top of the tunnel and breaks up on the tunnel apron leading to a free water surface in the outlet tunnel.

When the flow aeration requirements are not satisfied by the outlet's downstream air space and/or the opened manhole, flow will be discharged at or near full conduit condition. With this condition the hydraulic grade line along the conduit will not exceed elevation 2,510 ft.

3.2.2.1 LOW-LEVEL OUTLET OPERATION

At the maximum reservoir water surface elevation, computed flow depth and velocity in the tunnel will be up to 10 ft high and reach velocities of up to 80 ft/s. The CFD simulations show that vortex formation is possible below reservoir water surface elevations of 2,535 ft. Vortex formation could lead to floating debris accumulation near the dam, potentially leading to floating debris entrained through the outlet. The opening of the diversion tunnel at or below a reservoir water surface elevation of 2,530 ft will mitigate potential problems that could be caused by the vortex formation and accumulated debris.

Bed shear stress and velocity associated with the operation of the low-level outlet indicate that scour of small coarse material is possible. During the operation of the outlet and as the flow velocity increases near the bed upstream of the outlet, scour equilibrium will be reestablished and most sediment discharge will be due to the sediment laden flow. Large bed load material movement causing potential blockages is not expected.

The spillway plunge pool at Copco No.1 is a pre-excavated unlined channel with a base width of 45 ft and a slope of 5H:1V over a length of 165 ft towards the downstream end. The plunge pool is assumed to consist mainly of boulders as all moveable bed material would have been scoured during the operation of the spillway. No geotechnical investigation was conducted on the plunge pool bed materials. The following characteristics are assumed based on the construction drawings and photographs:

- Plunge pool is founded in a matrix of large, interlocked rock with sand and gravel
- Powerhouse tailrace is founded on andesite bedrock

The CFD study indicates that during operation of the outlet works the submerged discharge at the spillway plunge pool will dissipate a portion of the outlet flow energy. A concentrated area of high velocity from the outlet discharge jet will impact the plunge pool bed. The maximum jet velocity occurs during the operation of outlet #1 with flow velocity of 25 ft/s at the impact zone. The plunge pool erosion will be limited to the bank scour associated with the submerged jet and the turbulent flow conditions allowing movement of the bed material until an armor layer is formed, or the andesite foundation limits the scour progression.

3.2.3 DIVERSION TUNNEL

The diversion tunnel has a square shaped section of approximately 16 ft wide and 18 ft high and is located through the left abutment, as shown on Drawing C2100. The following design parameters are considered for the design:

- The tunnel initial opening at elevation 2,505.8 ft is specified to limit the discharge capacity. Flow characteristics at this opening satisfy the submergence requirements for inlet controlled discharge.
- Full conduit discharge may occur at the initial tunnel reopening if the inlet is fully opened. Minor reduction of discharge capacity would occur with no effects to the flow conditions.
- The fully open tunnel will operate partly full below inlet water levels of approximately 2,515 ft where the submergence requirements for inlet control discharge are satisfied.

The tunnel operates under inlet control for all inlet water levels below 2,530 ft. The capacity of the river reach downstream of the tunnel outlet is larger than the tunnel diversion capacity. Reduction of the capacity due to backwatering is not expected. During the operation of the diversion tunnel in the summer months, the discharges and flow depths will cause lower critical shear stresses indicating that there would be little or no scouring as the channel equilibrium would have been reached during the operation of the low-level outlet. Large bed load material movement causing potential blockages is not expected.

3.3 RESERVOIR DRAWDOWN

3.3.1 RESERVOIR DRAWDOWN CRITERIA

The reservoir drawdown outlet works, and its operation is designed to achieve the following:

1. Outlet facilities for reservoir drawdown will be designed to discharge reservoir drawdown flows and natural inflows during the emptying period up to flow events with 25% chance of exceedance between January 1 and June 15 of the drawdown year.
    a. Drawdown outlet discharge capacity is designed to lower the reservoir levels at a reasonably constant rate for elevations above 50% of the hydraulic height of the dam.
    b. Reservoir refill can occur when natural inflows exceed the drawdown outflows.
2. Rapid reservoir drawdown will occur when the capacity of the drawdown discharge outlet exceeds the natural inflow. This condition is achieved when:
    a. The reservoir volume is equal to the average inflow multiplied by one day (inactive storage).
    b. Storage capacity is less than 10% of the capacity at the long-term normal reservoir operation level.

3.3.2 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN

The operations of the Copco No. 1 reservoir during drawdown will achieve successful evacuation of the reservoir impoundment and provide the required flood control. The reservoir drawdown and river diversion will be completed using the spillway, newly constructed low-level outlet, and the historic diversion tunnel. The drawdown model (detailed in Appendix G) is used to assess the drawdown sequencing in terms of reservoir water surface levels under a range of hydrologic conditions.

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

- Individual year simulations are provided in the Attachment 1 - Copco No. 1 Facility Simulated Drawdown, Figures 1 through 36. These plots indicate the following:
  - o Reservoir water surface levels.
  - o Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway, low-level outlet, and flows through the historic diversion tunnel).
- Maximum daily reservoir water surface level non-exceedance percentiles (percentiles) are shown on Figure 3.1, and on Drawing C2056. This figure represents the results from all 36 model simulations as non-exceedance percentiles of the thirty-six resulting reservoir elevations to summarize the distribution of the results on any given day of the simulations. These results do not represent a single simulation and are based on all the 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 year model simulation.

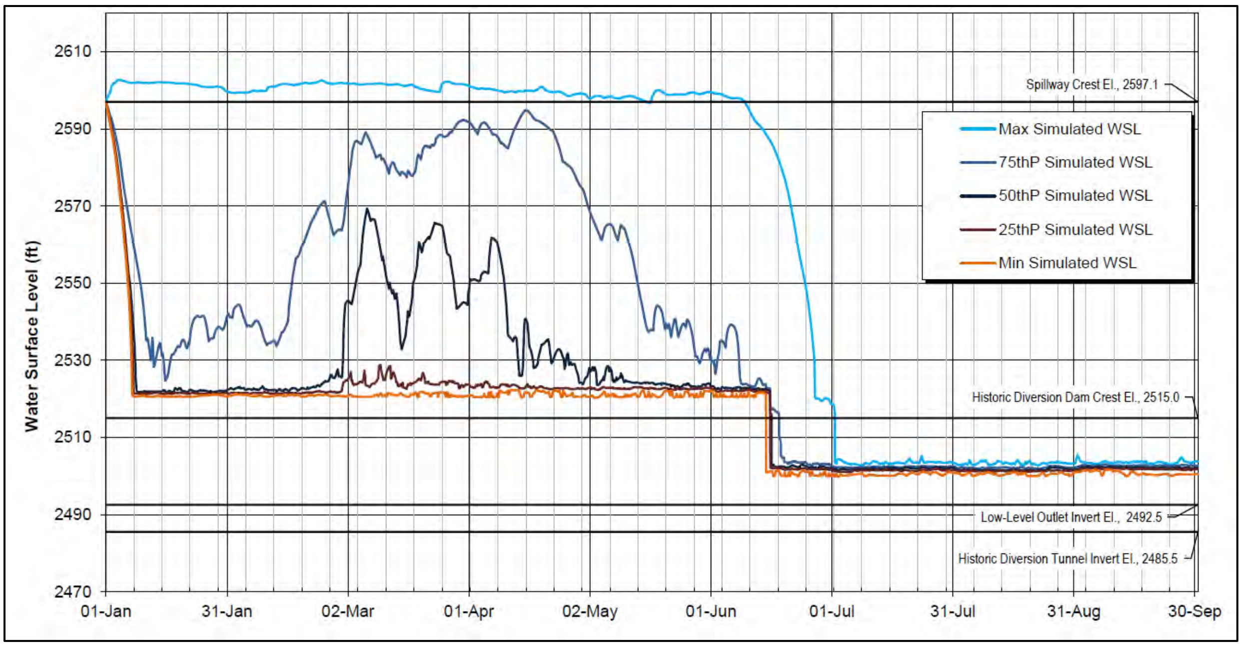

Figure 3.1 Copco No. 1 Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

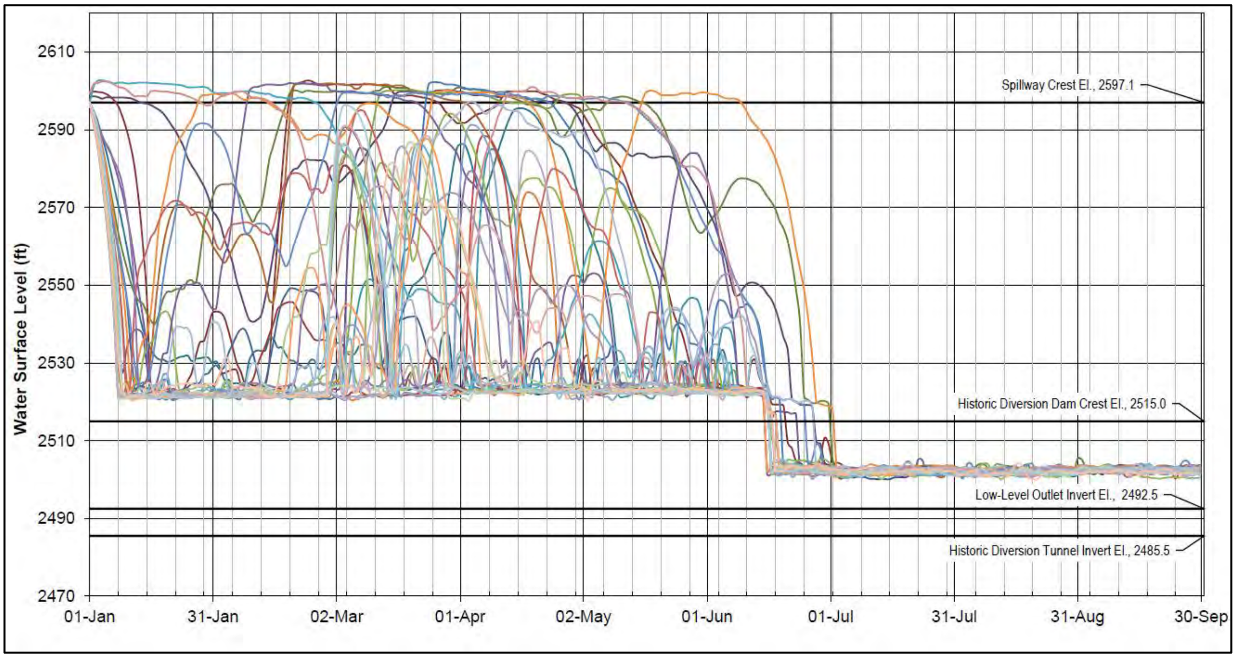

Figure 3.2 Copco No. 1 Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

The simulated water surface levels on Figure 3.1 show that the reservoir water levels drop below the crest of the historic diversion dam in mid-June for the 75th percentile, while the remaining model simulations achieve a lowered reservoir water level by the beginning of July.

Figure 3.2 shows that approximately 80% of the model simulations drawdown to a water surface elevation of approximately 2,520 ft in January, which is the lowest water surface elevation achievable using the low-level outlet prior to the historic diversion tunnel opening. The reservoir, however, refills in the higher flow months of February through May. There can be large fluctuations in the reservoir water surface levels from March through June. Spillway flows are observed after January for 31% of the simulations.

The reservoir water surface level can rapidly rise in March, April and May resulting from large inflow events. Examples of this are seen in simulation years 1981, 1989, and 1993, where the reservoir water surface level was at approximately 2,520 ft in January but then rapidly rose in response to the high inflows. These inflows may be a function of required flushing flows from the upstream irrigation project as described in USBR (2018) or are influenced by the flows from unregulated tributaries entering the Copco No. 1 reservoir.

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

- Elevation 2,597.1 ft – represents the spillway crest. Approximately 67% of the drawdown model simulations have reservoir water levels sustained below the spillway crest by January 2, 75% of the simulations by April 1, and 90% by May 5. All model simulations have reservoir levels below the spillway crest by June 10.
- Elevation 2,575 ft – represents the power intake invert. Only approximately 7% of the drawdown model simulations are at or below the power intake invert by March 5, 50% of the simulations by April 5, 80% of the simulations by May 10, and 100% by June 18.
- Elevation 2,557 ft – represents the top of dam removal lift 3. The removal of the top of the dam down to elevation 2,525 ft. can start after this date. Approximately 30% of the model simulations are sustained below elevation 2,557 ft by April 1, 63% of the simulations by May 1, 85% by June 1, and 100% by June 20.
- Elevation 2,530 ft – represents the highest water surface elevation at which the historic diversion tunnel can be opened. Currently in the drawdown model, the historic diversion tunnel opens after June 15 once the reservoir water surface level is at or below 2,530 ft, which is approximately 20 ft above the top of the existing intake structure. Initially, a 5 ft diversion tunnel opening is assumed and once the water surface level drops below 2,516 ft an 18 ft opening is assumed. The drawdown model indicates that approximately 50% of the simulations have reservoir water levels below 2,530 ft by June 1, with approximately 30% of the simulations achieving this as early as May 1, and 100% by the end of June. There is potential to open the historic diversion tunnel earlier in the year based on the drawdown model results, but this will be dependant on the hydrological conditions in the drawdown year as well as the flood risk to be assumed. River forecasting will be required as the reservoir levels need to be maintained below 2,530 ft once the historic diversion tunnel has been opened.
- Elevation 2,515 ft – represents the crest of the historic diversion dam. Drawdown is achieved when the water surface level is maintained below the diversion dam crest, which can only be achieved after the historic diversion tunnel is opened. The drawdown model indicates that approximately 80% of the simulations have reservoir water surface levels sustained below the crest of the historic diversion dam within a few days (June 19) of the historic diversion tunnel opening on June 15, with 100% of the simulations achieving this by July 2.

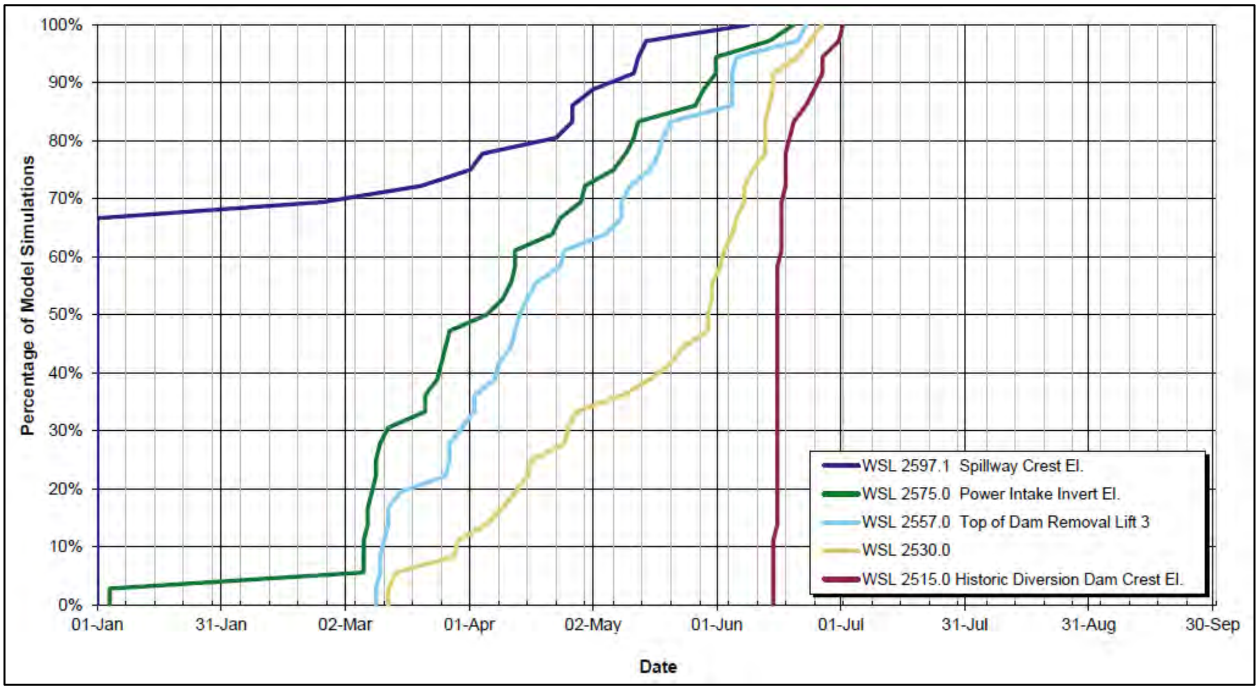

Figure 3.3 Copco No. 1 Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below Various Relevant Elevations

3.3.2.1 POST-DRAWDOWN/RIVER DIVERSION

The diversion tunnel can be opened when the reservoir water surface elevation of 2,530 ft is reached on or after June 15th of the drawdown year. The discharge capacity of the tunnel will depend on the opening size and the reservoir water level:

- The diversion tunnel partially opened to elevation 2,505.8 ft has a capacity greater than 1,775 cfs for reservoir level of 2,530 ft.
- The diversion tunnel fully opened has a capacity greater than 3,885 cfs for reservoir level of 2,514 ft, (elevation of the abandoned diversion dam with 1 ft freeboard).
- The fully opened diversion tunnel can bypass all inflows in the period June 15 to the end of September under all hydrological years evaluated.
- At the first opening of the diversion tunnel, outflow surges of up to 5,675 cfs are predicted occur.

River diversion is achieved when all the inflows are passed through the diversion tunnel with negligible attenuation in the post-drawdown period (i.e., the outflows are roughly equal to the inflows). The drawdown model summarized on Figure 3.1 and Figure 3.2 indicate that the post-drawdown water surface levels will range between 2,500 ft and 2,505 ft for average daily conditions evaluated in the drawdown model. These levels do not account for the low probability flood flows (i.e., the 1% and 5% probable flood flows); however, the fully opened diversion tunnel capacity of greater than 3,885 cfs can pass more than the 5% probable monthly flood with no ponding during the post drawdown period from June 15 to the end of September. The 1% probable flood of 4,400 cfs in the second half of June may cause limited ponding, up to an approximate maximum elevation of 2,516 ft, as shown in Table 1 on Drawing C2057.

3.4 STEADY-STATE WATER SURFACE LEVELS

3.4.1 RESERVOIR AND TAILWATER LEVELS

Flood water surface levels for the reservoir and tailwater are shown on Drawing C2057 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 reservoirs provide flow attenuation from January through June 15 of the drawdown year. Once river diversion has been achieved at these facilities, the flood flows will no longer be attenuated, therefore, no flow attenuation is assumed for the flood flows between June 16 and December of the drawdown year, as discussed in Appendix A6. The levels are calculated using the discharge rating curves developed for the outlet structures as detailed in Appendix C2 and shown on Drawing C2056.

3.4.2 REMOVAL TIMING

The earliest dates for key work items in relation to the post drawdown reservoir level and the tailwater level are determined based on the variability of the inflows and assume that work can occur up to the 5% probable inflow with freeboard. The earliest dates are presented in Table 3.2.

Table 3.2 Key Structure Elevations and Removal Timing

Work Item	Lowest Structure Elevation (ft)	Earliest Removal Date	Removal Volume	Design Flood Event	Comments	Disposal Location
Removal of spillway gates and ancillary items	2,597.1	-	-	1% max. monthly flood	Spillway gates can be removed in the dry after drawdown is complete to El. 2,530 ft.	Off-site, to be confirmed.
Removal of concrete dam and intake structure	2,472.1	-	52,000 cu yd	5% max. monthly flood	Allow impoundment of the 1% probable flood level with 3 ft freeboard.	Copco No. 1 Disposal Site.
Removal of gatehouses and intake mechanical items	+/-2,570	After low-level outlet operation	-	1% max. monthly flood	Can be removed in the dry after drawdown is complete, 5% probable flood level with 3 ft freeboard after June 1 is 2,534 ft.	Masonry and reinforced concrete to be disposed at Copco No. 1 Disposal Site. Mechanical and electrical items to be disposed off-site, to be confirmed.
Removal of penstock #1, #2, and #3	2,575	-	-	1% max. monthly flood	Can be removed at any time after the flow diversion through the turbine and generator unit is no longer required. Intake gates to be in the closed position and leakage controlled if the removal of the penstock exposes the downstream area to uncontrolled release of water.	Off-site, to be confirmed.
Re opening of the diversion tunnel	2,488	-	1,500 cu yd	After freshet flows	Final opening to occur when reservoir level is at or below elevation 2,530 ft.	Copco No. 1 Disposal Site.
Removal of upstream historic construction spoil materials	+/-2,490	-	-	-	Can be removed any time where water level allows access or alternatively by dredging.	Dredge materials will be disposed at the open-water disposal site, excavated materials will be used for river channel final grading, or disposed at Copco No. 1 Disposal Site.

Work Item	Lowest Structure Elevation (ft)	Earliest Removal Date	Removal Volume	Design Flood Event	Comments	Disposal Location
Removal of the powerhouse	2,465	-	-	-	Can be removed at any time after the flow diversion through the turbine and generator unit is no longer required. In-water work to occur after the California in-water work date (June 1).	Masonry and reinforced concrete to be disposed at Copco No. 1 Disposal Site Mechanical and electrical items to be disposed off-site, to be confirmed.
Removal of in-river concrete	2,472	June 16	-	-	Drawdown is complete and all flows are diverted through the diversion tunnel.	Copco No. 1 Disposal Site and tailrace openings.
Removal of spillway work platform and river channel final grading	2,477	-	-	-	In-water work is required, lowest water levels occur during August and September.	Suitable materials will be used for river channel final grading, or disposed at Copco No. 1 Disposal Site.
Construction of tunnel portal plugs	+/- 2,494 (inlet) / 2,475 (outlet)	August	-	-	In-water work is required, lowest water levels occur during August and September.	-

NOTES:

1. REMOVAL OF THE FACILITY'S WATER RETAINING STRUCTURES INCLUDING THE DAM, INTAKE GATES AND THE SPILLWAY GATES BEFORE JUNE 1 AND WHEN THE RESERVOIR WATER SURFACE LEVEL IS ABOVE ELEVATION 2,538 ft EXPOSES THE WORK SITE AND DOWNSTREAM AREA TO THE RISKS OF THE 1% PROBABLE ANNUAL FLOOD.
2. SHOULD THE DIVERSION TUNNEL BE REOPENED BEFORE JUNE 15, THE DESIGN WATER LEVELS AND EARLIEST REMOVAL DATE SHOULD BE REEVALUATED.

3.5 FINAL GRADING HYDRAULIC CONDITIONS

Channel characteristics and the geometry of the Copco No. 1 final river channel presented on Drawings C2230, C2231 and C2232 were used to develop a hydrodynamic model to determine the stage-discharge relationship for the post-dam removal period. The model uses HEC-RAS 2D with a Manning's n of 0.04.

The resulting stage-discharge relationship is shown on Figure 3.4 at the location of the current dam centerline. Model sensitivity is evaluated using Manning's n roughness of 0.03 and 0.06 to account for potential variability in roughness elements that are added to the channel to provide localized habitat. The results of the sensitivity analysis are included on Figure 3.4.

Dam removal construction activities in the vicinity of the final river channel are scheduled to continue into the fall. Steady-state water surface levels for probable floods and mean monthly flows for specified periods in September through November, are provided for reference in Table 3.3 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 3.4 using the base model Manning's n value of 0.04.

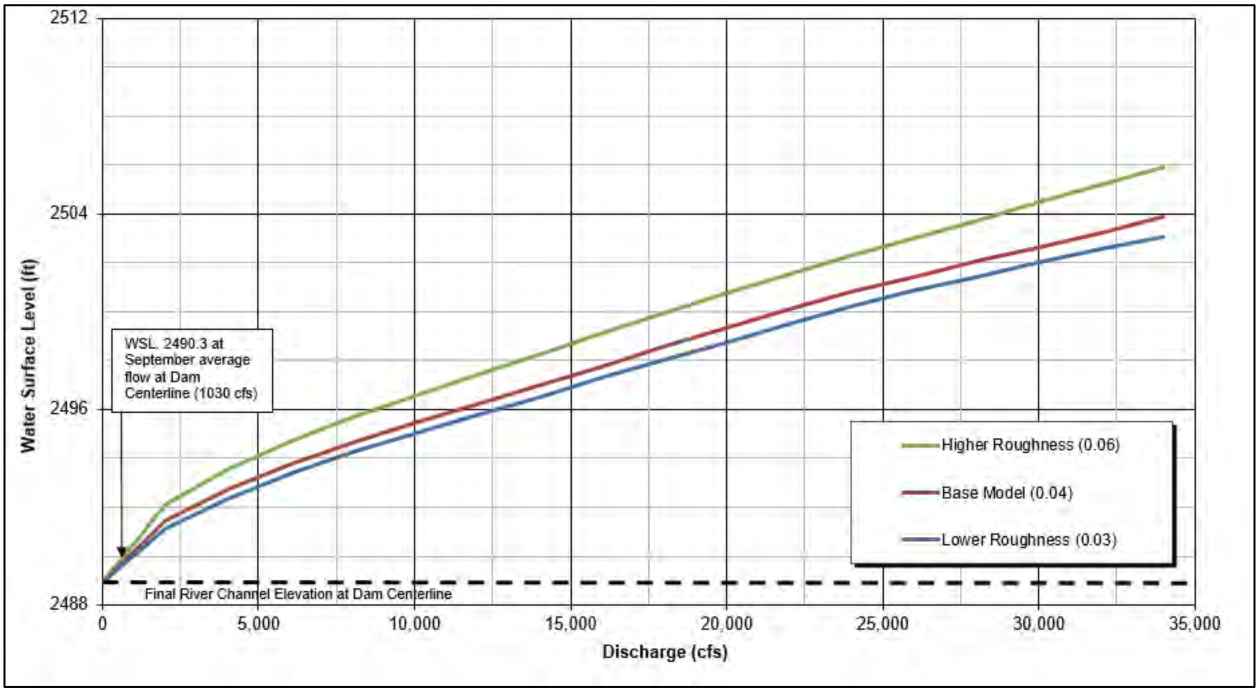

Figure 3.4 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

Table 3.3 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	2,491.4	2,491.5	2,493.6	2,493.9	2,494.4	2,494.8
	20% Probable Flood	2,491.0	2,491.0	2,492.9	2,493.1	2,493.3	2,493.7
	50% Probable Flood	2,490.6	2,490.6	2,492.5	2,492.5	2,492.7	2,492.8
Mean Monthly Flow for Time Period		2,490.3	2,490.3	2,490.3	2,490.3	2,490.4	2,490.5

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,489 ft.

Table 3.4 Final River Channel Annual Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge with Attenuation from Upstream Facilities (cfs)	Discharge with No Attenuation from Upstream Facilities (cfs)
		Annual	Annual
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 (1)**	**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline (1)**
		Annual	**Annual**
Statistical High Water (Flood Conditions)	1% Probable Flood	2,502.4	2,503.5
	5% Probable Flood	2,498.6	2,500.9
	20% Probable Flood	2,495.6	2,497.5
	50% Probable Flood	2,494.2	2,495.9
Annual Flow Duration 25% of Time Equaled or Exceeded		2,491.2	2,491.2
Mean Annual Flow		2,491.1	2,491.1
Annual Flow Duration 75% of Time Equaled or Exceeded		2,490.1	2,490.1

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 2,489 ft.

4.0 GEOTECHNICAL

4.1 MATERIAL PROPERTIES

4.1.1 FILL MATERIAL PROPERTIES

The following materials will be encountered during facility demolition and will require engineered storage or will be sourced locally for use in establishing temporary access as required for the project. These materials are anticipated to be sourced from the following locations:

- General Fill (Type E9/E9a) and Select Fill (Type E4) will be sourced from the road improvement excavation and riverbed excavation (historic spoil materials). Minimal to no processing of these materials is anticipated.
- Erosion Protection (Type E7) will be selectively sourced from the road improvement excavation, riverbed excavation (historic spoil materials) and the Copco No. 2 borrow site. Separation during the excavation of these materials is anticipated.

- Concrete Rubble (Type CR1 and CR2) will be produced from demolition of the intake, power canal, forebay and powerhouse. Fragmentation will be as specified on the Technical Specifications.
- Random Fill (Type E10): a small quantity of construction waste consisting of reinforcing steel, wood, organic materials, and other materials is expected to be encountered during the removal of the facility. Any materials that are not earthfill, rockfill, or concrete rubble will be disposed in accordance with the project requirements.

4.1.2 LOW-LEVEL OUTLET APPROACH CHANNEL

The low-level outlet approach channel dimensions and depth are based on the review of the construction photographs, it is assumed that during construction the river channel was clear and approximately 110 ft wide at the dam site, with andesite bedrock at surface on both banks. The area upstream of the upper cut-off wall was filled with sand, gravel, and boulders from the excavation of the dam foundation and bedrock clearing of the dam abutments. The area could also have logs and/or construction debris. This fill could be approximately 30 ft deep. No geotechnical dredging site investigations have been completed.

It is anticipated that material dredging will be limited to upstream of the abandoned diversion dam and will gradually slope to an elevation lower than the invert of the low-level outlet to form a shallow rock/debris trap to reduce the bed load discharge at time of the low-level outlet plug removal.

Stability of the excavation side slopes will be designed for the underwater condition.

4.1.3 DIVERSION TUNNEL APPROACH CHANNEL

The diversion tunnel approach channel excavation is required to re-establish flow passage to the diversion tunnel. The diversion tunnel approach channel was excavated during the early stages of construction of the facility, dimensions are not shown on the construction drawings. Construction photographs show naturally stable steep slopes of andesite rock. It is anticipated that clearing with a dredge or excavator will be limited to the previous channel dimensions and depth. No geotechnical dredging site investigations have been completed. Bathymetry investigations completed in 2020 indicate that the intake structure and gate shaft pedestals appear to be partially buried when compared to the historic photograph of the structure.

4.1.4 RIVER CHANNEL

The re establishment of the river channel will require the excavation and relocation of historic excavation material spoiled upstream of the dam. Most of the excavation is located on the right riverbank through historic excavation material spoils with local areas of shallow bedrock associated with hillside spurs located in basalt rock. The excavation slopes are anticipated to be in alluvial basalt material and in historic material spoils. The stability of the historic material spoils is anticipated to be satisfactory as these areas have been under +/-90 ft of water for a long duration. Construction photos show the following:

- Material spoils are comprised of large boulders and cobbles removed from the excavations and most likely end-dumped up to an approximate elevation of 2,545 ft.
- Left riverbank slopes are comprised of natural steep slopes of andesite rock.

No bedrock will be excavated. The excavation lines and grades are intended to provide a stable river channel section and where indicated expose the bedrock limits.

Prior to drawdown, the Project Company may consider dredging some of the upstream areas that would be hard to access once the drawdown is complete.

4.2 TEMPORARY WORK PLATFORMS AND ACCESS TRACKS

The removal works will require a work platform at the base of the spillway to allow the construction of the low-level outlet and access the diversion tunnel outlet portal. Various temporary access tracks will also be required during dam and intake removal to access the facility at different elevations and to allow the excavation of the upstream river channel. The access tracks which are critical to the construction progress are:

- Left bank access track through private property, equipment only (not designed)
- Downstream dam access to elevation 2,557 ft
- Upstream access to historic spoil at elevation 2,530 ft (not designed)
- Ford access to the historic diversion tunnel inlet and outlet areas

Timing of the temporary access tracks will vary based on the Project Company's schedule and progression of the work activities.

4.2.1 RIGHT BANK ACCESS ROAD IMPROVEMENTS

The right bank access roads will provide construction access for the removal of both the Copco No.1 and Copco No. 2 dams. The roads will upgrade the existing Copco access road and establish additional spurs to allow for construction traffic to move more efficiently. The proposed road designs are shown on Drawing C2510 and as detailed in Appendix F5. The road alignment has been selected based on a terrain hazard assessment.

A terrain hazard assessment has been undertaken for the Copco No. 1 Dam Access Road to identify potential hazards affecting road users. The Study Area includes the upslope and downslope areas of natural terrain. The assessment identifies rock fall hazards between approximately STA. 8+50 and 11+00 on the Copco No. 1 Access Road and approximately STA 4+00 and 5+00 on the Copco Disposal Site Access Road. The rock fall hazard along the Copco No. 1 Access Road will need to be addressed by the Project Company during the construction stage. The Copco Disposal Site Access Road will be positioned a minimum of 15 ft away from the cliff face and will be elevated above the existing ground. The gap between the cliff face and the road will form a rock catchment area for any potential rock fill. In addition to the rock fall hazards, a recent debris slide was mapped at STA 4+50 of the Copco No. 1 Access Road. It is approximately 50 ft wide, 50 ft long, and 8 ft deep with a volume of approximately 20,000 ft^3. Historic air photo interpretation shows the landslide developed before 1991 and there was no significant change between 1991 and 2016. The slide is currently being managed using an existing gabion block catchment system, and no changes are proposed to the structure. Maintenance of the existing gabion catchment system is currently being performed during normal road maintenance. This maintenance practice should continue during construction. The full terrain hazard assessment is presented in the Geotechnical Data Report.

4.2.2 SPILLWAY ACCESS TRACK

The spillway access track will provide construction access to the spillway plunge pool and diversion tunnel outlet portal. Access during the pre-drawdown year will be via a low-water access from Copco No. 2 along the Klamath river during periods of controlled discharge from Copco No. 1. Access after the drawdown period, when operation of the Copco No. 1 powerhouse is no longer necessary will be through the powerhouse. This access track will be improved to be utilised as the main haul-out route during the dam removal activities.

Fill construction in open water is required within the plunge pool. A low-water access road will be developed to be used when the Copco No.1 tailrace level/Copco No. 2 forebay levels are low. Reduced water discharge from Copco No.1 dam and releases from Copco No.2 dam will be required to lower the water levels. An access track upstream of Copco No.2 will allow for intermittent placement of general fill – Type E9 material /rockfill from the road improvement excavation to construct the access track and for materials to be placed and compacted around the outlet conduit to form the spillway apron.

The spillway access track will have to be constructed to elevation 2,511 ft on top of the concreted erosion protection layer of the spillway apron. Usage of this access can not obstruct the operation of the air vent as outlined in Section 2.0. A temporary work platform of concrete rubble will be installed up to elevation 2,525 ft on top of the access track during the removal of the dam.

4.3 RIVER CHANNEL

The channel geometry is designed to convey the 1% probable annual flood event. The granular native materials are considered erosion resistant where the channel is excavated to the approximate grade and lines of the historic Klamath River channel. Unlined channel sections are specified where the channel has uniform flow conditions, slope and cross section and is located within the historic Klamath River channel. Unlined channel sections are designed to be stable at the the final channel grade up to the post-drawdown 1% annual flood event.

Temporary thalwegs and fords may be required to allow excavation of the river channel, riverbed material placement below the water level and access to the riverbanks. Upstream controlled releases from Keno Dam can lower daily flows to 500 cfs to allow low-flow river crossings with depth of roughly 2 ft.

4.3.1 EROSION PROTECTION DESIGN

Erosion protection of fill construction is provided to prevent scour resulting from high-velocity and/or turbulent flow. Erosion protection material is designed in compliance with the USACE (1994) guidelines with a minimum safety factor of 1.2. To prevent erosion of fine material located below the slopes lined with erosion protection material, a layer of bedding material will be placed to provide the appropriate filter relationship with the subgrade material.

Erosion protection at the south-east corner of the abandoned in place powerhouse substructure and the tunnel portal barriers is designed for the post-drawdown 1% annual flood event. Channel characteristics and geometry were used to develop a HEC-RAS 2D model, which produced the channel cross sectional area at critical location for the design of erosion protection. The hydraulic characteristics at locations where erosion protection is designed are estimated to be:

- Upstream tunnel portal barrier: maximum river level during the 1% probable annual flood event is estimated to be at El. 2,512.5 ft (approximately 20 ft above the channel bottom) and the average velocity in the design reach is computed to be 19.5 ft/s, assuming a Manning's n value of 0.04.
- Downstream tunnel portal barrier and south-east corner of the powerhouse: maximum river level during the 1% probable annual flood event is computed to be at El. 2,501 ft (approximately 15.5 ft above the

channel bottom) and the average velocity in the design reach is estimated to be 18 ft/s, assuming a Manning's n value of 0.04.

The modified Maynord method (USACE, 1994) was used to determine the rock size and thickness of erosion protection that is required to resist the flow velocity. Rock density used for the design of erosion protective layers at Copco No. 1 is assumed as to be 170 lb/ft^3 (specific gravity of 2.74 based on laboratory testing from the surrounding proposed borrow sources). The specified rock size and erosion protection layer thickness is shown on Drawing C2175 and C2230. Erosion protection will be extended 3 ft above the design water surface.

The required riprap gradations are presented in the Section 31 05 00 – Materials for Earthwork specification and are also shown on Drawings G0050 and G0051.

4.3.2 DIVERSION TUNNEL PORTAL CLOSURE

The closure plug in the tunnel portal is required to minimize possibility of future diversion of the river channel through the abandoned in place diversion tunnel. The upstream tunnel portal closure is designed for the following conditions:

- Hydraulic head associated with the 1% probable annual flood water level.
- Hydraulic conductivity of the rock mass surrounding the plug is limited to less than 3.2^{-7} ft/s.
- Maximum exit gradient of 0.25 in the downstream toe of the earthfill to prevent piping.

The downstream tunnel portal closure is designed to allow relief of any seepage water that accumulates between the upstream and downstream closures. A high conductivity material is specified to allow drainage.

4.4 DISPOSAL SITES

A number of borrow and disposal areas will be required for the construction of the project. Borrow and spoil areas are design and developed with stable permanent slopes and suitable surface drainage requirements.

A total of approximately 136,000 cu. yd. of material comprised mainly of concrete rubble, general fill and select construction waste will be disposed for the dam removal Works. Two disposal sites will be used for the project, the main disposal site near the current operator's houses and the open water site for any materials removed by dredging upstream of the dam.

The main disposal site, No. 1, will have a maximum height of approximately 55 ft and is sized to contain approximately 189,000 cu. yd. of removed concrete from the dam, penstock, powerhouse and associated structures and the concrete removed from the Copco No. 2 dam. The disposal site may also contain dam earthfill material from Copco No. 2 and non-hazardous construction material from the removed facility buildings. The final volume of disposed material will vary based on the placed volume and actual quantities removed from the sites.

The proposed dredged material disposal site is generally shown on Drawing C2272. The dredged material will be free dumped onto the reservoir bottom, in open water. The dredged material may contain previously excavated materials and abandoned construction materials. The material placed in the disposal site will be nominally compacted and graded once the area is accessible after reservoir drawdown. Any construction debris that is not earthfill, rockfill, or concrete rubble will be removed and disposed in accordance with Project disposal requirements. A summary of the removed material is shown on Table 4.1. Small quantities of fill material are required at the powerhouse site and for channel final grade construction.

Table 4.1 Disposal Area and Disposal Material Sources

Source	Neat Line Volume (cu. yd.)	Disposal Area
Concrete from Copco No. 1 dam	52,000	Disposal Site No. 1
Previously excavated material from the reclaimed river channel footprint	27,000	Disposal Site No. 1
Concrete removed from Copco No. 1 powerhouse and penstock	2,000	Disposal Site No. 1
Excavated materials from road improvements	70,000	Disposal Site No. 1/River Channel
Concrete and earthfill removed from Copco No. 2 dam	5,000	Disposal Site No. 1
Low-Level Outlet and Diversion Tunnel	1,945	Disposal Site No. 1
Total	**130,945**	
Low-level outlet approach channel	3,000	Open Water Disposal Site
Diversion tunnel approach channel	2,000	Open Water Disposal Site
Total	**5,000**	
Overall Total	**135,945**	

The stability of permanent fill slopes was assessed for two sites: the disposal site located above the dam site and the powerhouse site. Limit Equilibrium Analysis (LEA) was completed in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. The acceptance criteria require a factor of safety (FOS) of 1.5 for static long-term stability, FOS of 1.0 for pseudo-static screening with 20% strength reduction and 50% MCE, and FOS of 1.0 for yield acceleration determination without strength reduction. In addition, STID-8 (PacifiCorp, 2015a) indicated displacements of 2 ft are acceptable according to a FERC guideline for the operating dam. Seismic displacements were approximated by the semi-empirical method, developed by Makdisi and Seed (1978). The design seismic loading was defined by STID-5 (PacifiCorp, 2015c), for a Maximum Credible Earthquake (MCE) of 0.26 g, and a magnitude 7.5 earthquake was assumed.

The disposal site stability analysis considered two units, a 2-ft capping layer of General fill (E9) material overlying disposal fill of concrete rubble (CR1) and/or General Fill (E9). Given the amount of concrete rubble expected in the disposal fill, the Leps (1970) lower-bound shear-normal strength function was assumed for both units. The unit weight of the General fill (E9) capping layer was assigned 125 pcf and the unit weight of the concrete rubble (CR1) and/or General Fill (E9). fill was assigned 130 pcf. Dry conditions were assumed. The stability results indicate static (FOS greater than 3.0) and pseudo-static (FOS greater than 1.0) stability is not expected to be a concern.

The powerhouse site consists of a 4-ft cap of Select Fill (E4) overlying modified General Fill (E9), which requires a higher level of compaction than unmodified E9. The unit weights assumed were 125 pcf for E4 and 130 pcf for E9. The strength assumed for E4 was 32° with zero cohesion. The Leps (1970) lower-bound shear-normal strength function was assumed for the modified E9 and the E7 erosion protection. The stability result shown on Figure 4.2, for a full-height slip that marginally achieves the target FOS of 1.5, indicates the critical area of concern is the steep 1.5H:1V slope face. The slip involves mostly the E4 capping layer and intersects the top foot or two of the modified E9 fill. Shallow or surficial slips within the E4 cap are possible but these should involve small volumes (tens of cubic yards). The yield acceleration was estimated at 0.06 for a shallow half-height slip (about 2 ft deep and less than 50 yd^3) in the steep

1.5H:1V slope face and is estimated to displace greater than 2 ft. For a full-height slip in the steep 1.5H:1V slope face, the yield acceleration is 0.11 and the estimated seismic displacement is less than 2 ft. The volumes associated with the predicted full-height slips are in the order of hundreds of cubic yards, which could be accommodated on the bench at the base of the 1.5H:1V slope. Space is limited in this area and necessitates a steep slope for the final grade. The size of the slip will be controlled by the surface of the concrete foundation and/or bedrock at the toe of the steep slope, the compaction effort of the E4 cap and modified E9 fill, and the thickness of the riprap erosion protection placed in the channel.

Figure 4.1 Powerhouse Site - Stability Analysis Results

4.5 EXCAVATED SLOPE STABILITY

Two design excavated slopes were analyzed for static and pseudo-static stability. The first slope excavation is located along the final river channel right bank. The second slope excavation is located at the base of the existing excavation at the historic borrow site.

Stability of the excavated slopes was analysed using Slope/W (GeoStudio, 2020) and the Morgenstern-Prive (GLE) method of slices. The target FOS for static is 1.5. The target for pseudo-static is 1.0 for a 20% reduction in strength and 50% MCE. The design seismic loading was defined by STID-5 (PacifiCorp, 2015c), for a Maximum Credible Earthquake (MCE) of 0.26 g.

Material properties used in the LEA were assumed based on limited information. A sensitivity check for lower strength was also completed for the borrow excavation. The right bank river channel excavation is expected to involve historical fill placed for a rail line utilized during dam construction. Conservative base case properties were selected accordingly. Base case assumptions for both slopes comprised a unit weight of 135 pcf and the Leps (1970) lower-bound shear-normal function. Dry conditions were assumed for both slopes.

The static LEA for both slopes resulted in FOS greater than 1.9 and the pseudo-static resulted in FOS greater than 1.0. The sensitivity check for reducing the strength to the Leps (1970) Angular Sand function for the borrow excavation indicated static stability is not expected to be a concern (FOS greater than 1.5) and pseudo-static stability would be marginal. The consequence of failure of the excavation slopes at the borrow excavation during a seismic event is low due to its location, well above the river channel.

Attachments:

1 – Copco No. 1 Facility Simulated Drawdown

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

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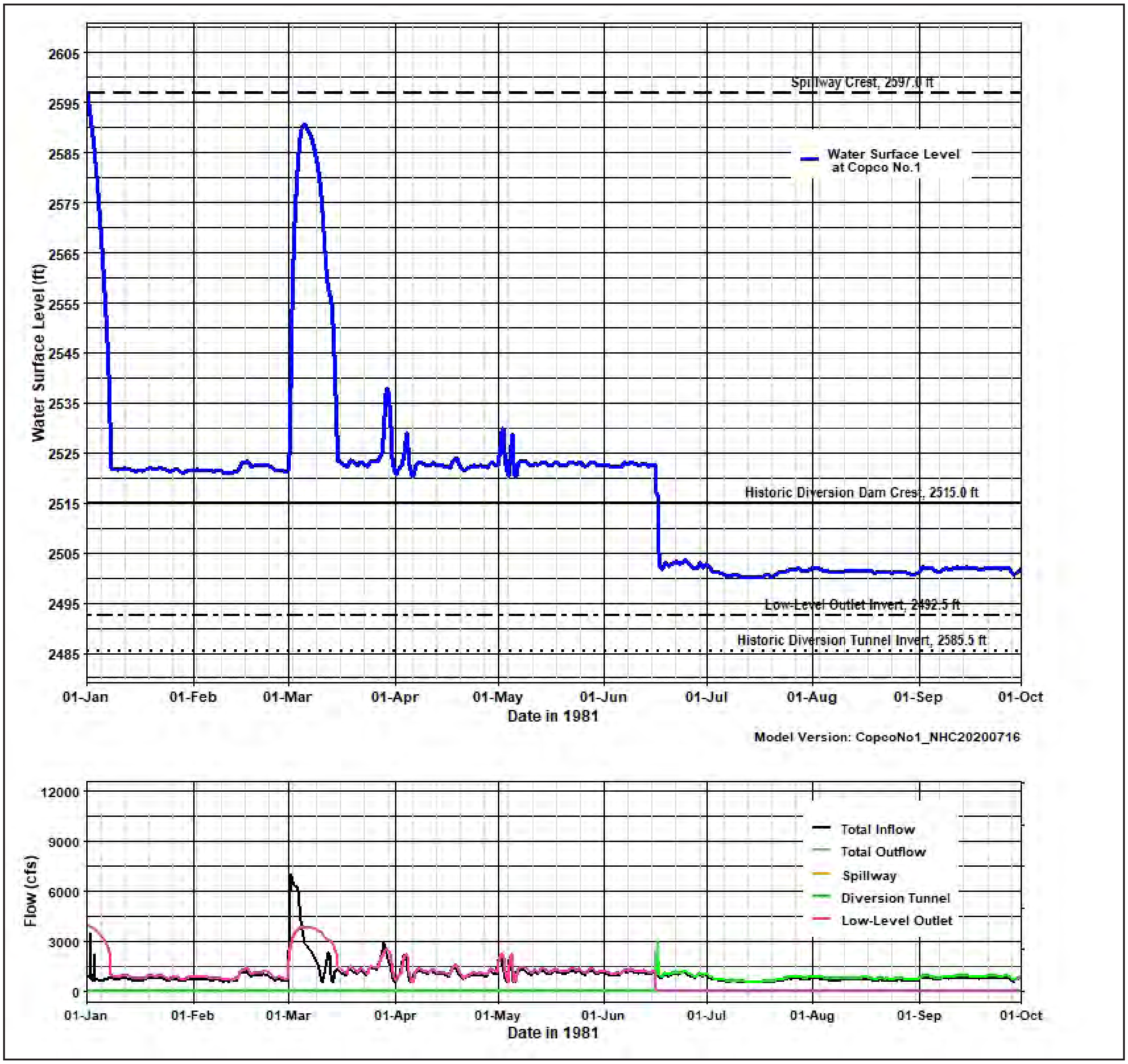

Figure 1 - COPCO No.1 Facility Simulated Drawdown - Year 1981

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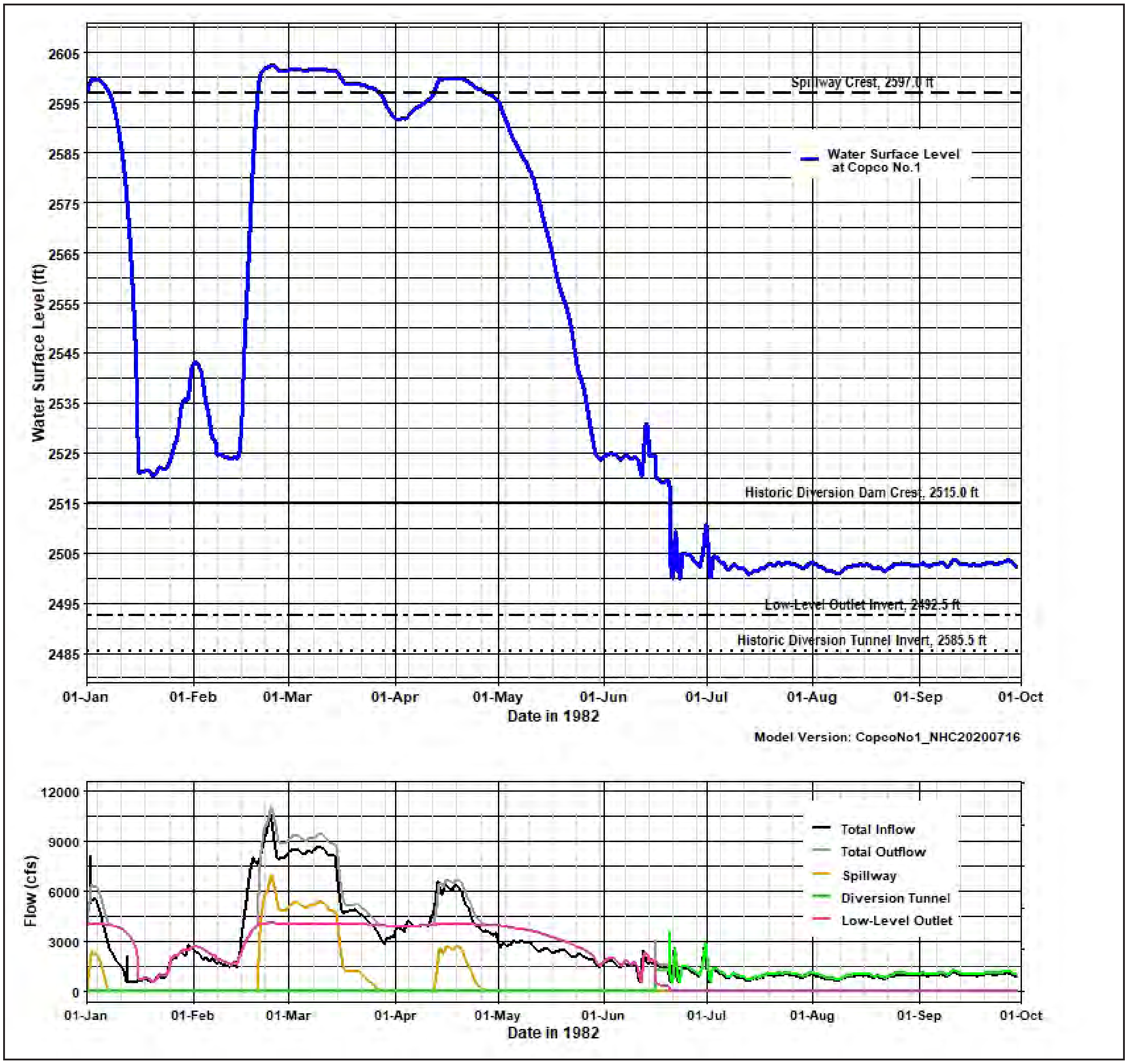

Figure 2 - COPCO No.1 Facility Simulated Drawdown - Year 1982

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

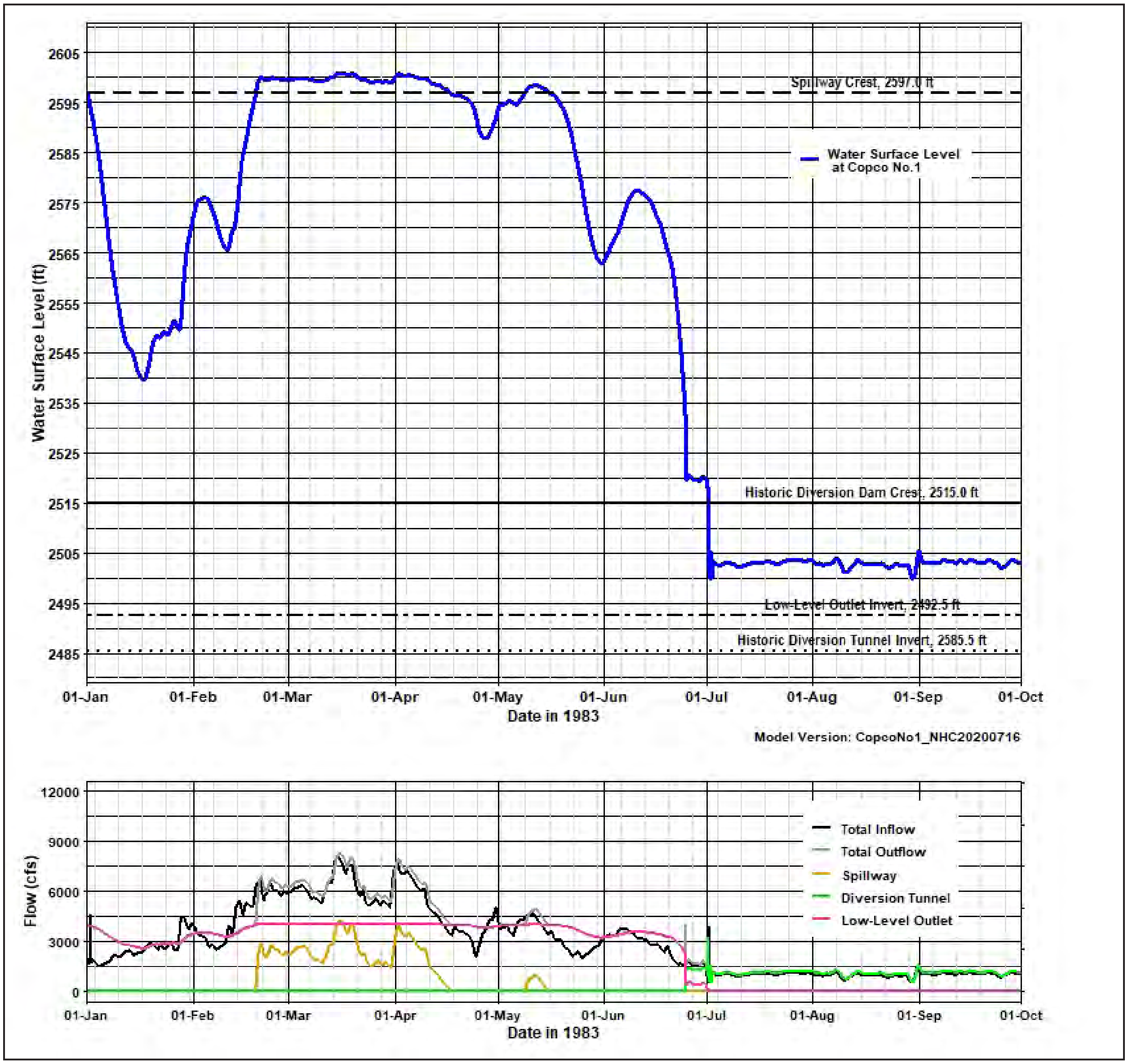

Figure 3 - COPCO No.1 Facility Simulated Drawdown - Year 1983

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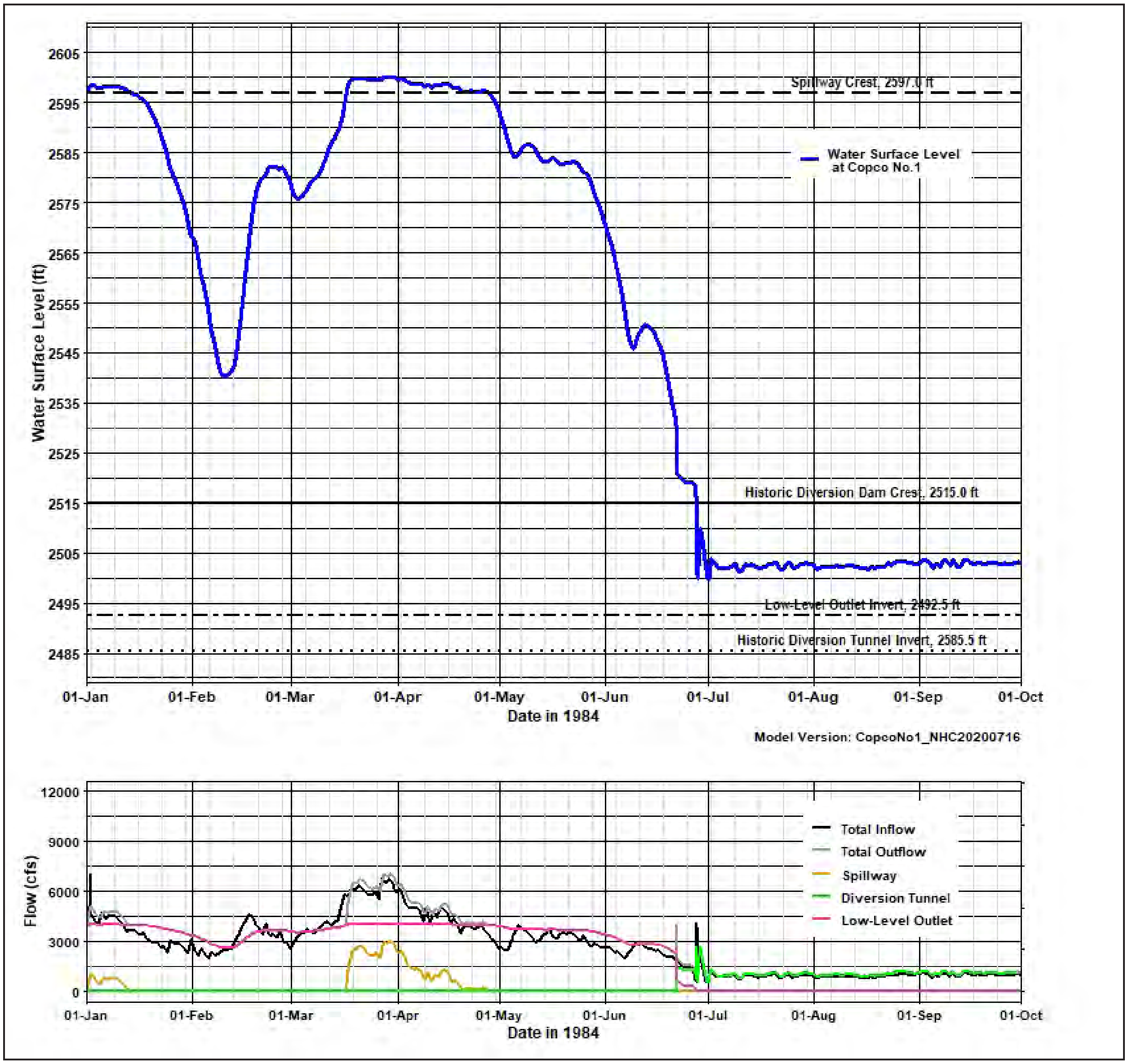

Figure 4 - COPCO No.1 Facility Simulated Drawdown - Year 1984

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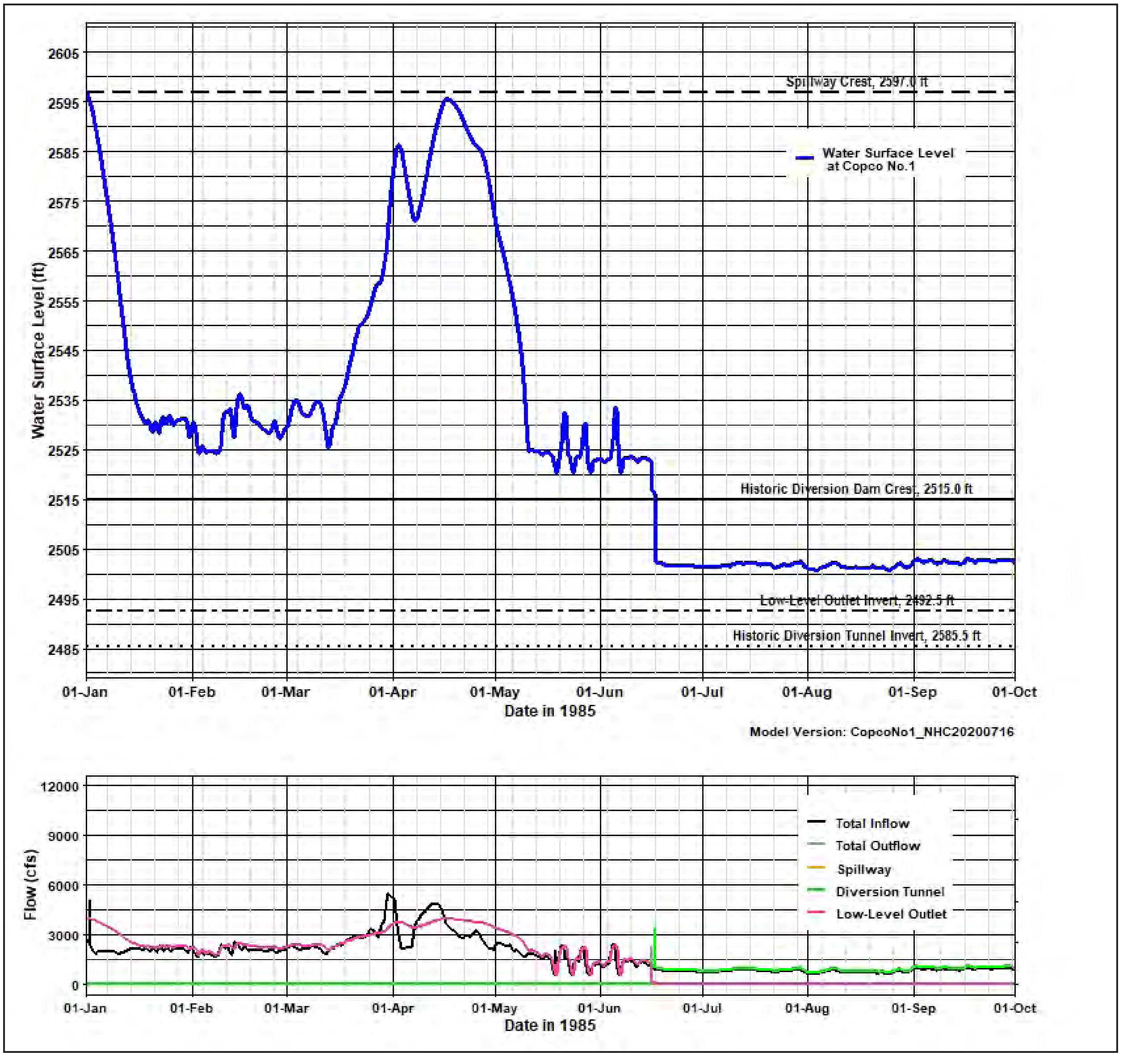

Figure 5 - COPCO No.1 Facility Simulated Drawdown - Year 1985

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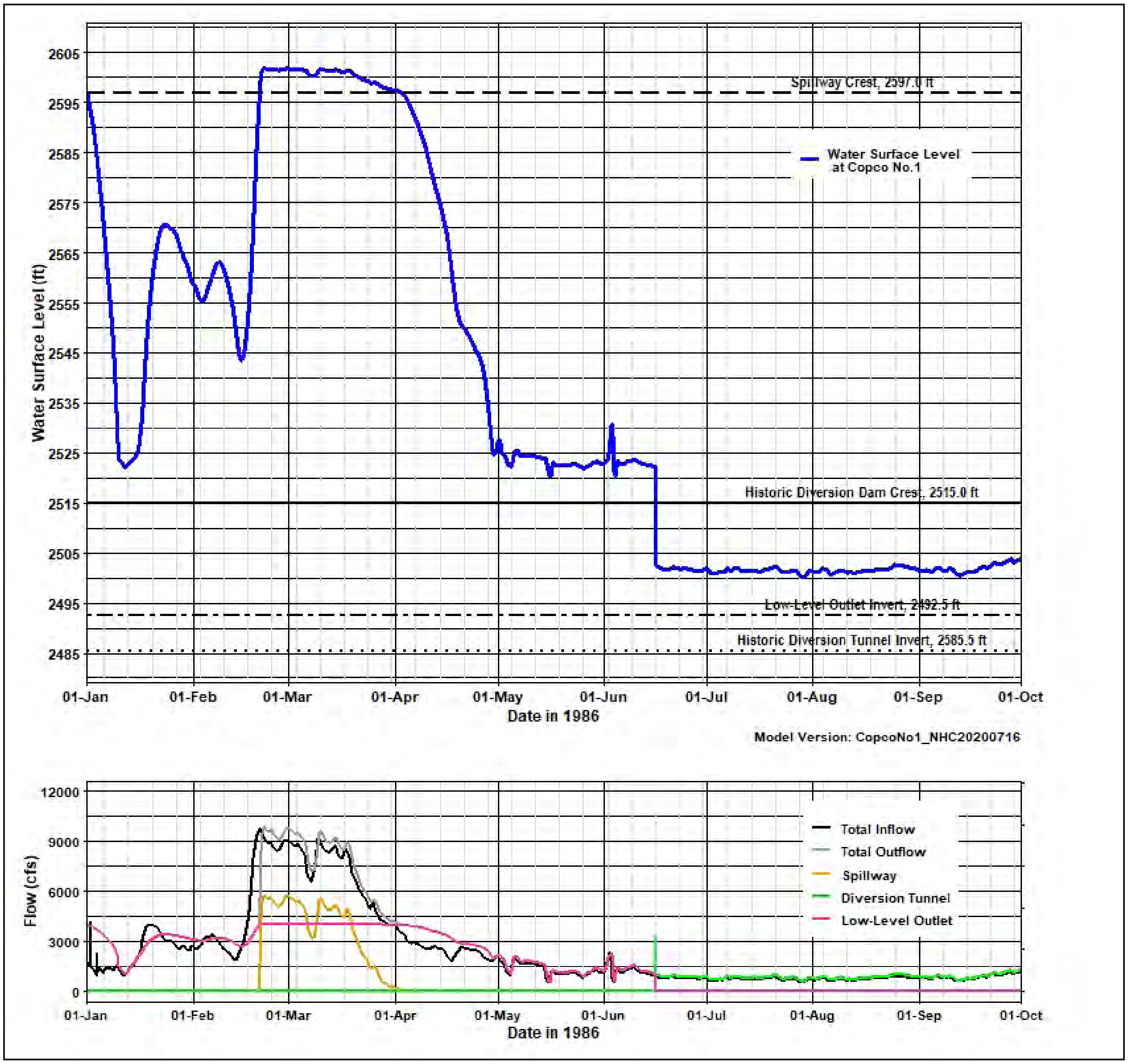

Figure 6 - COPCO No.1 Facility Simulated Drawdown - Year 1986

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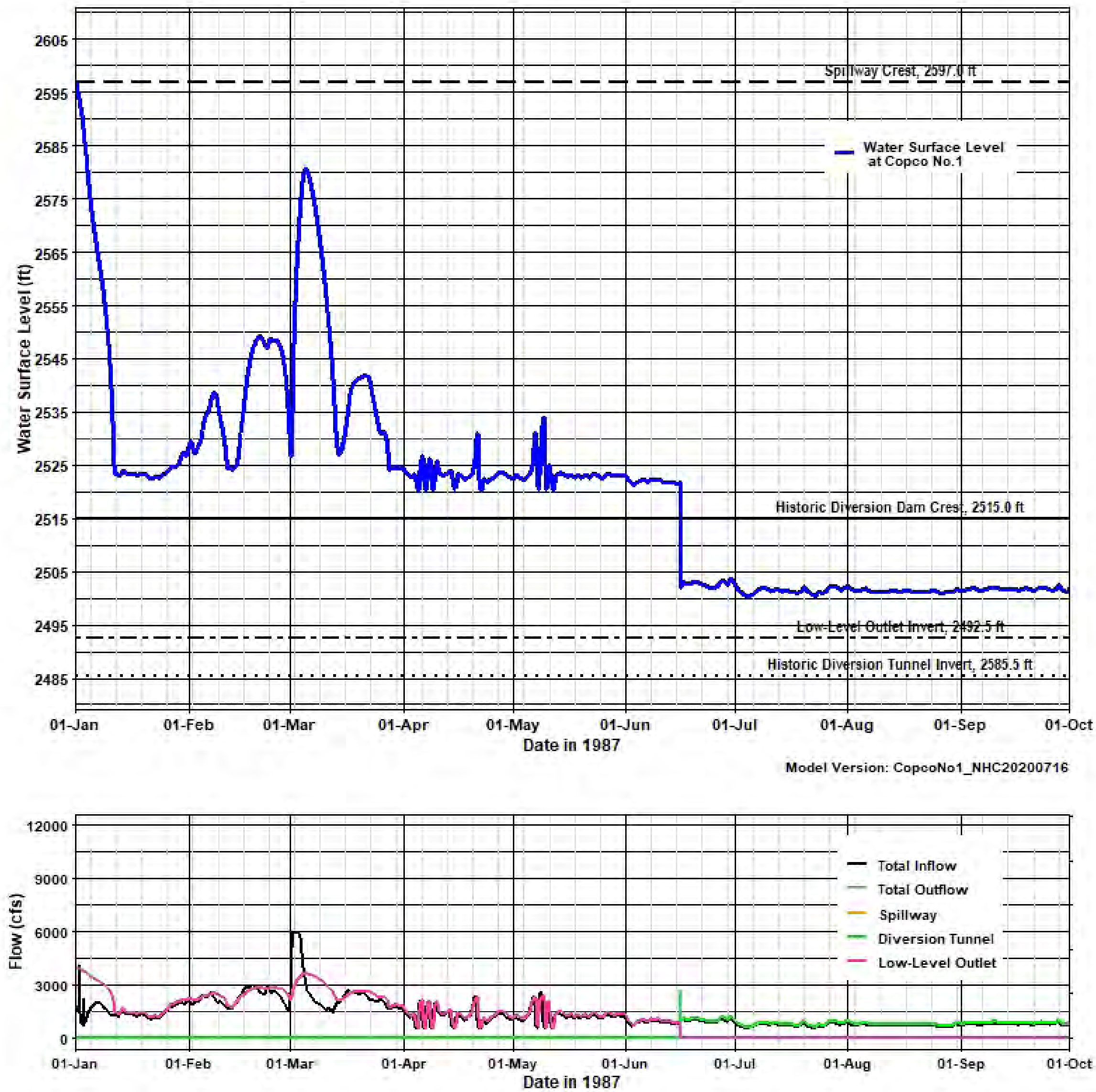

Figure 7 - COPCO No.1 Facility Simulated Drawdown - Year 1987

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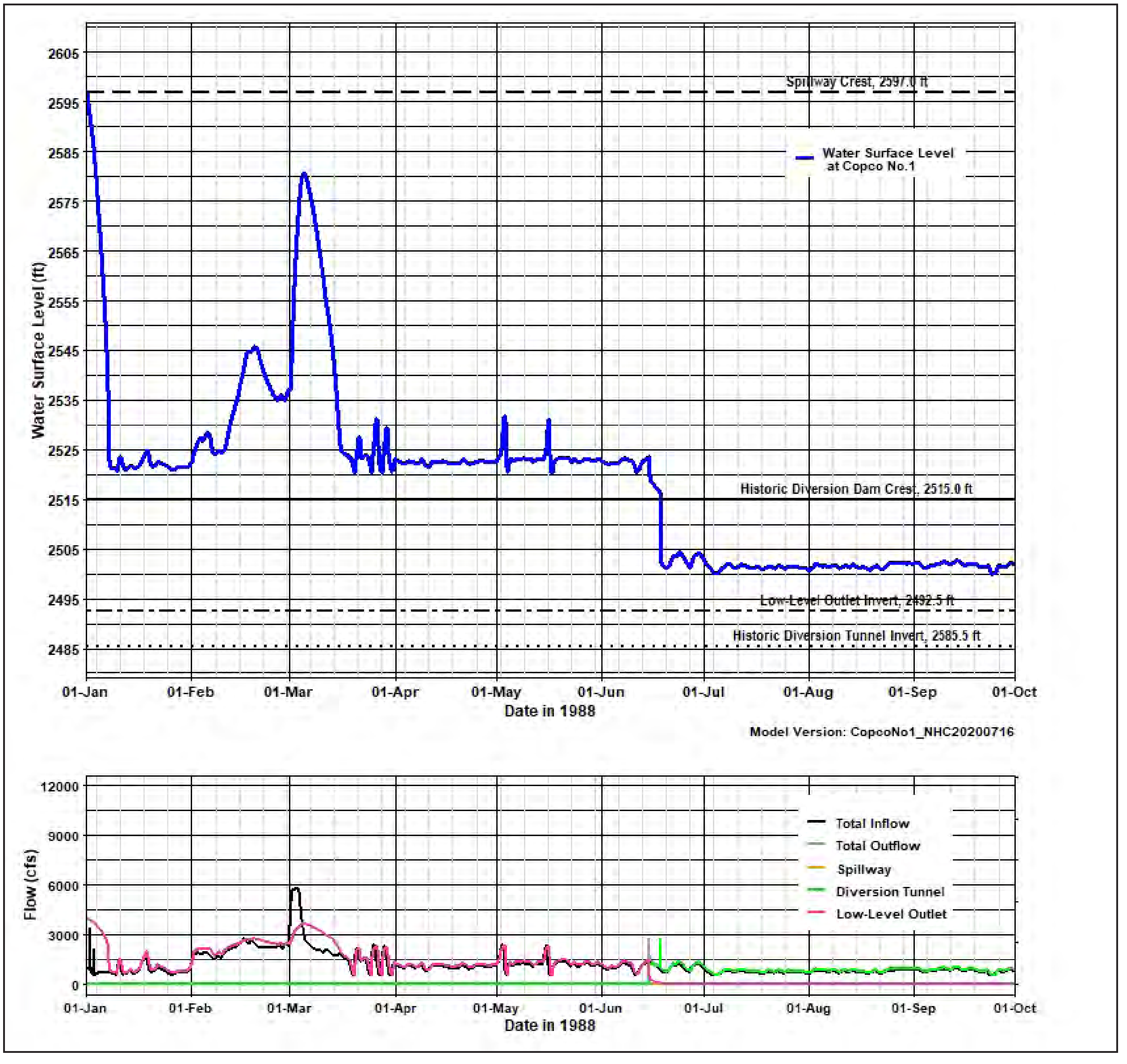

Figure 8 - COPCO No.1 Facility Simulated Drawdown - Year 1988

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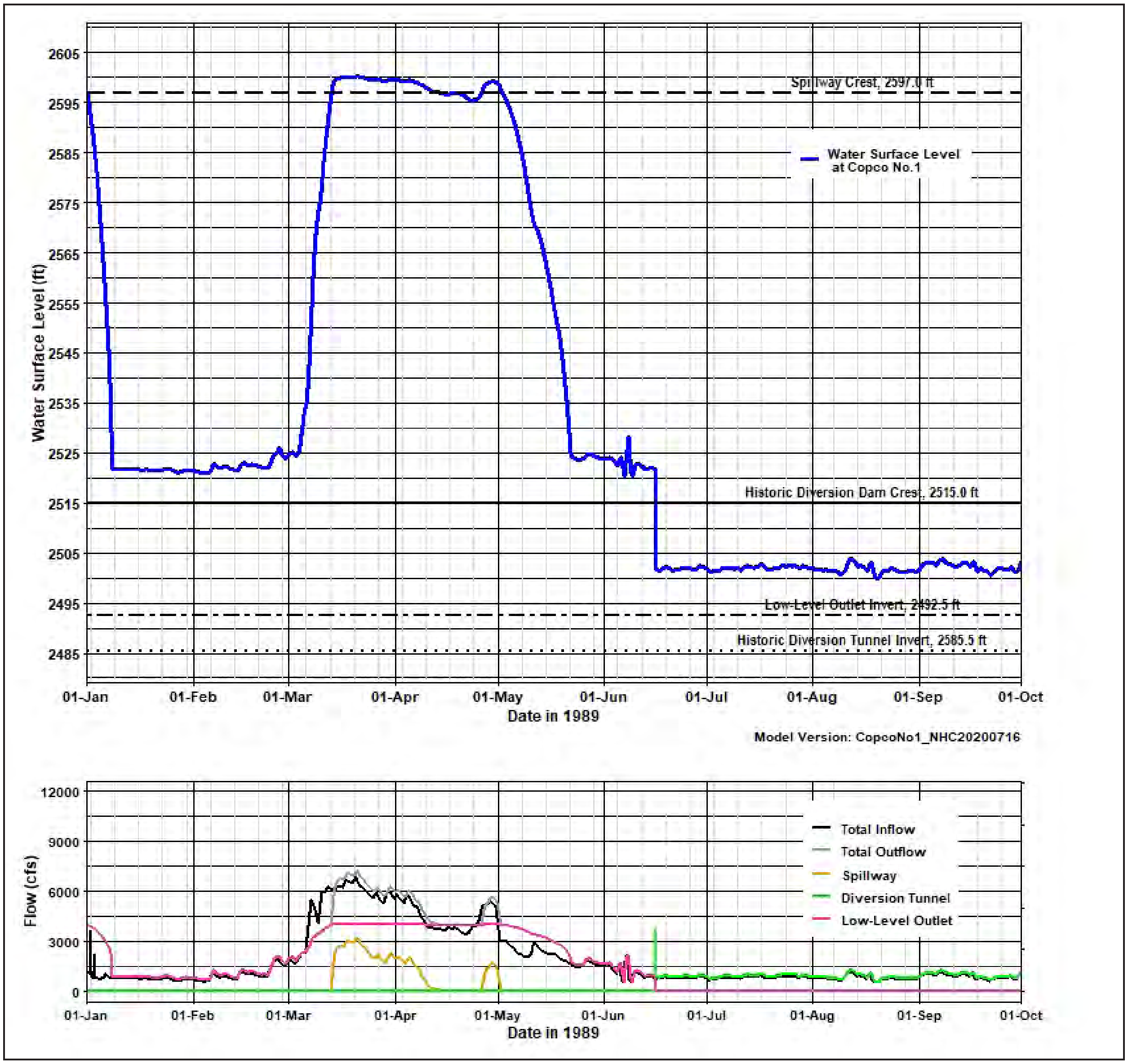

Figure 9 - COPCO No.1 Facility Simulated Drawdown - Year 1989

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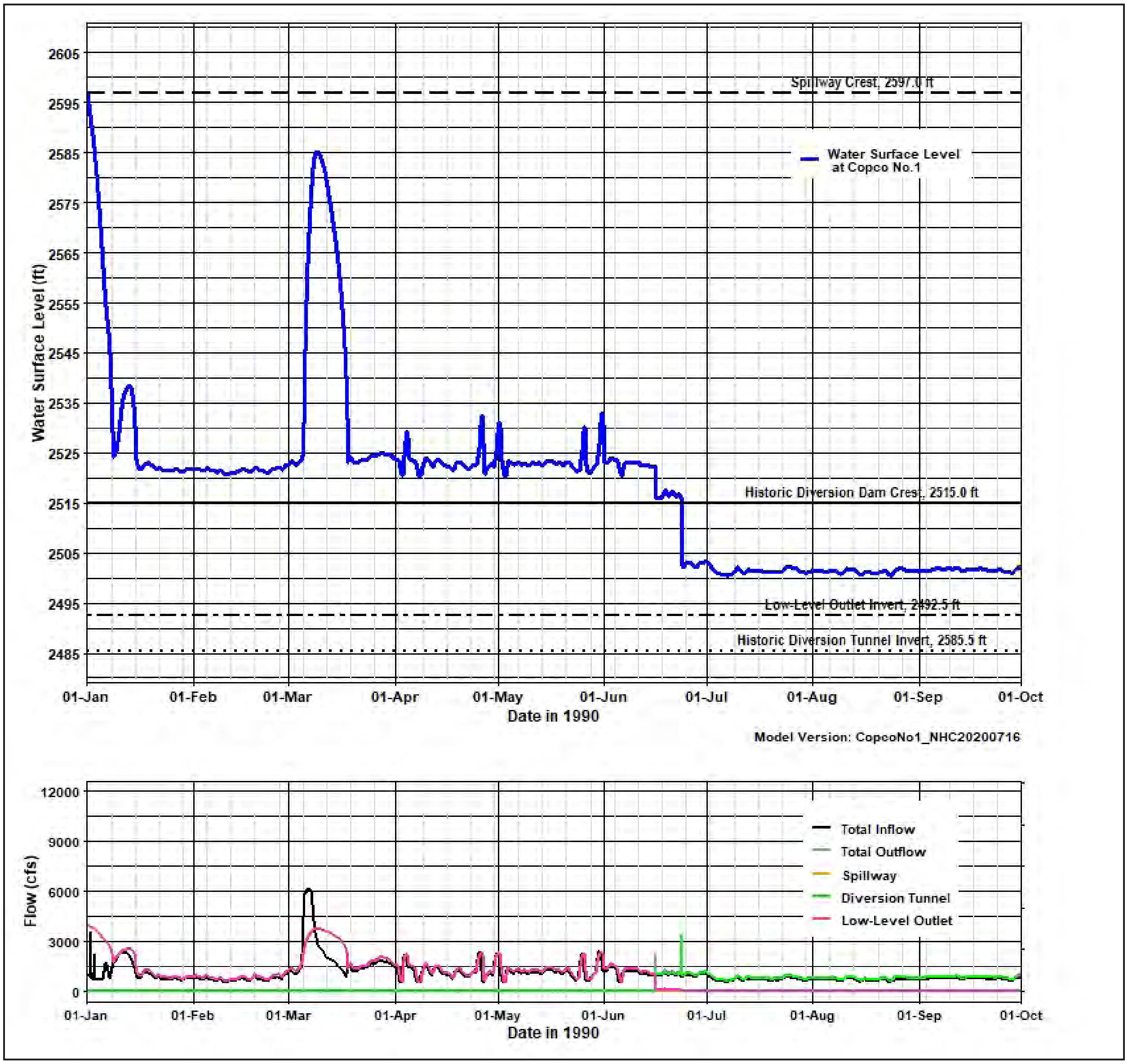

Figure 10 - COPCO No.1 Facility Simulated Drawdown - Year 1990

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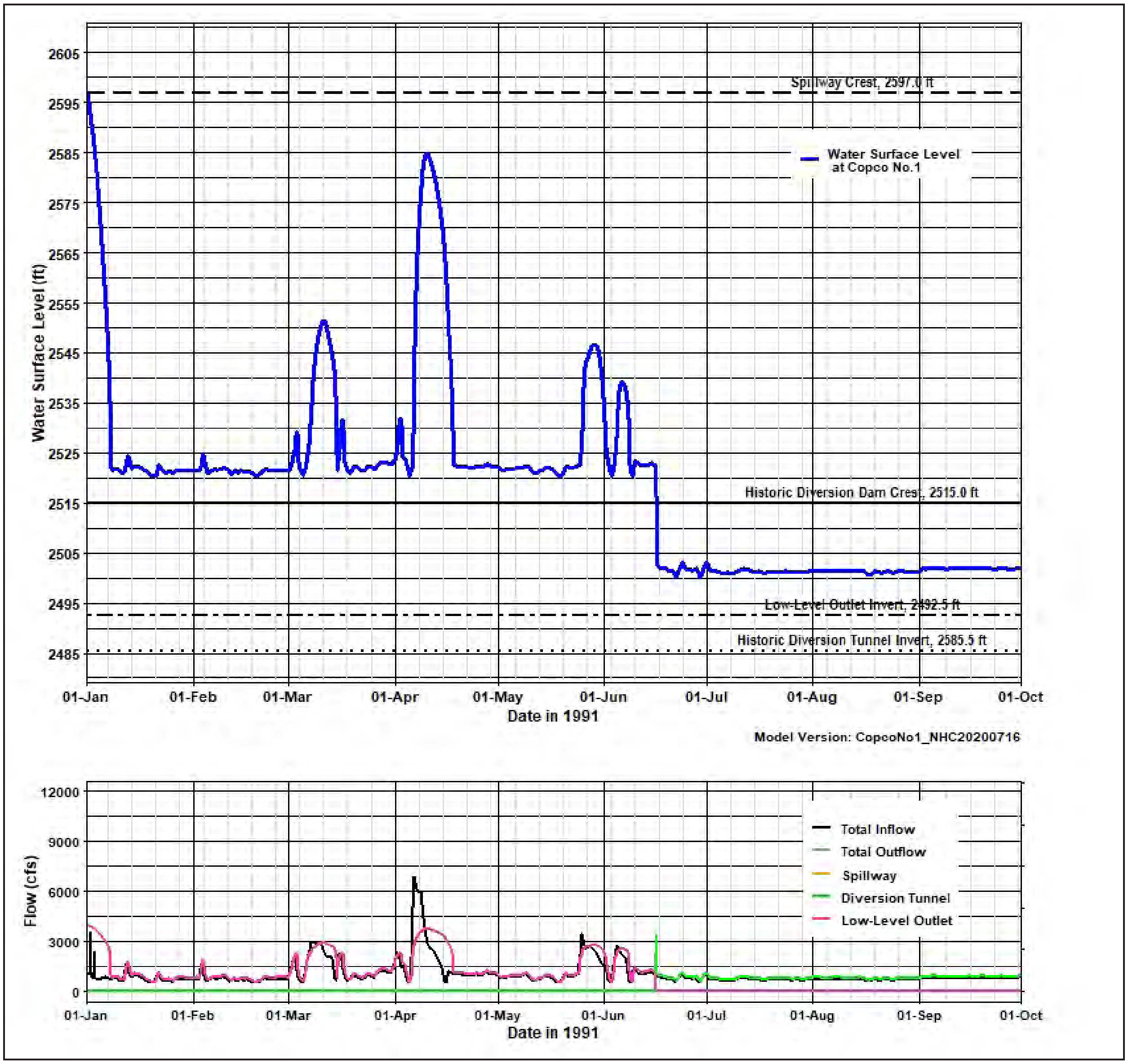

Figure 11 - COPCO No.1 Facility Simulated Drawdown - Year 1991

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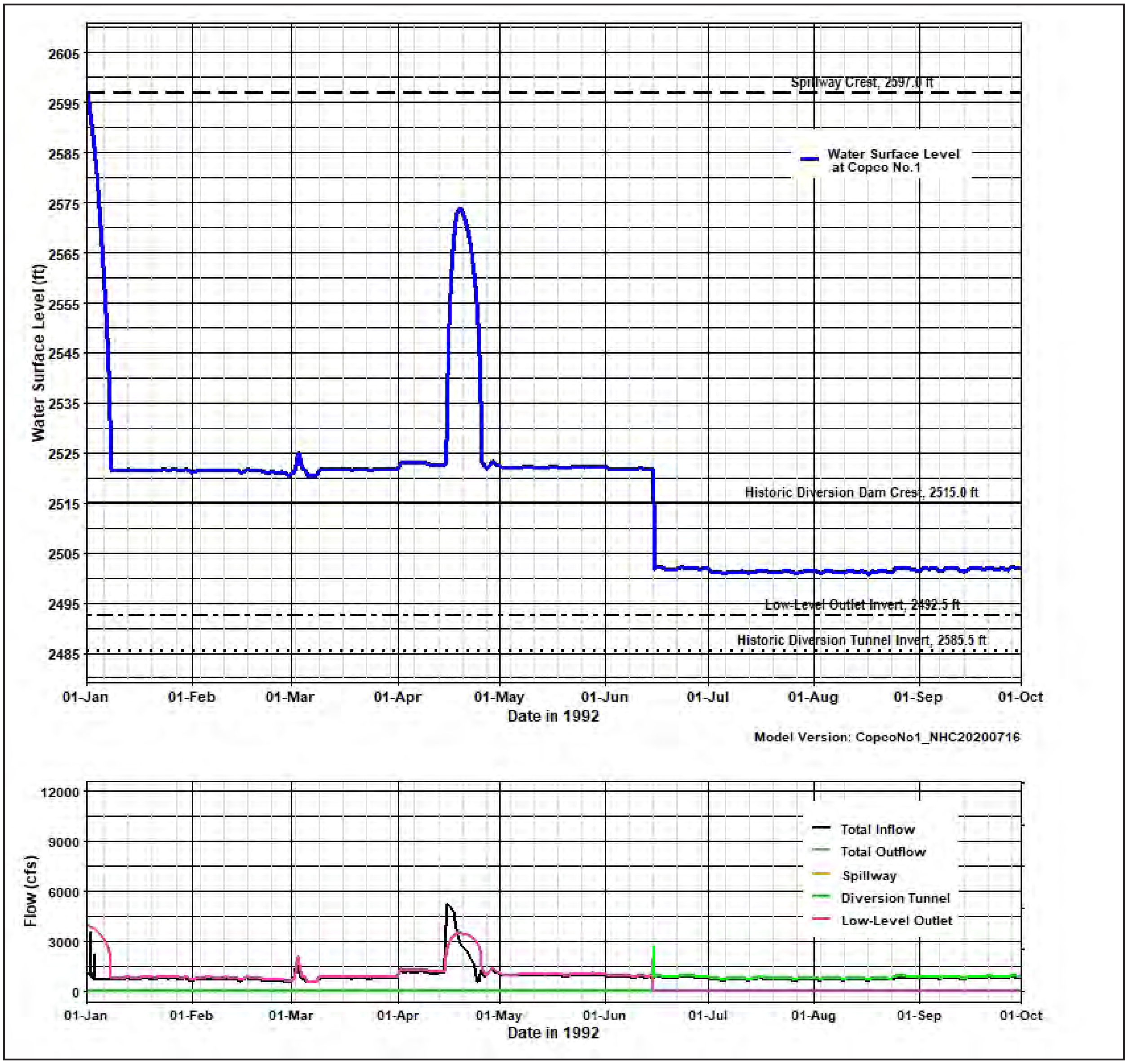

Figure 12 - COPCO No.1 Facility Simulated Drawdown - Year 1992

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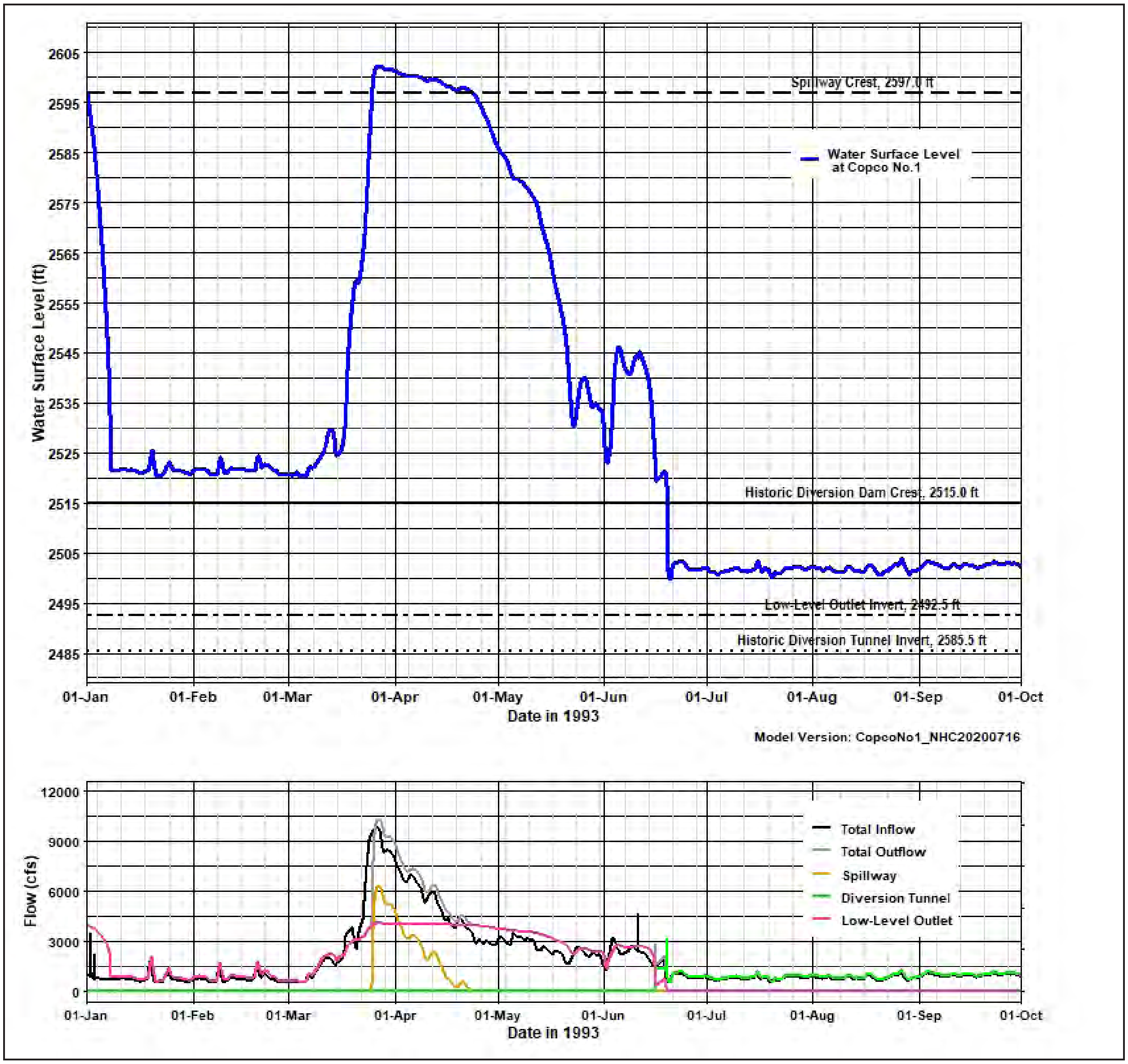

Figure 13 - COPCO No.1 Facility Simulated Drawdown - Year 1993

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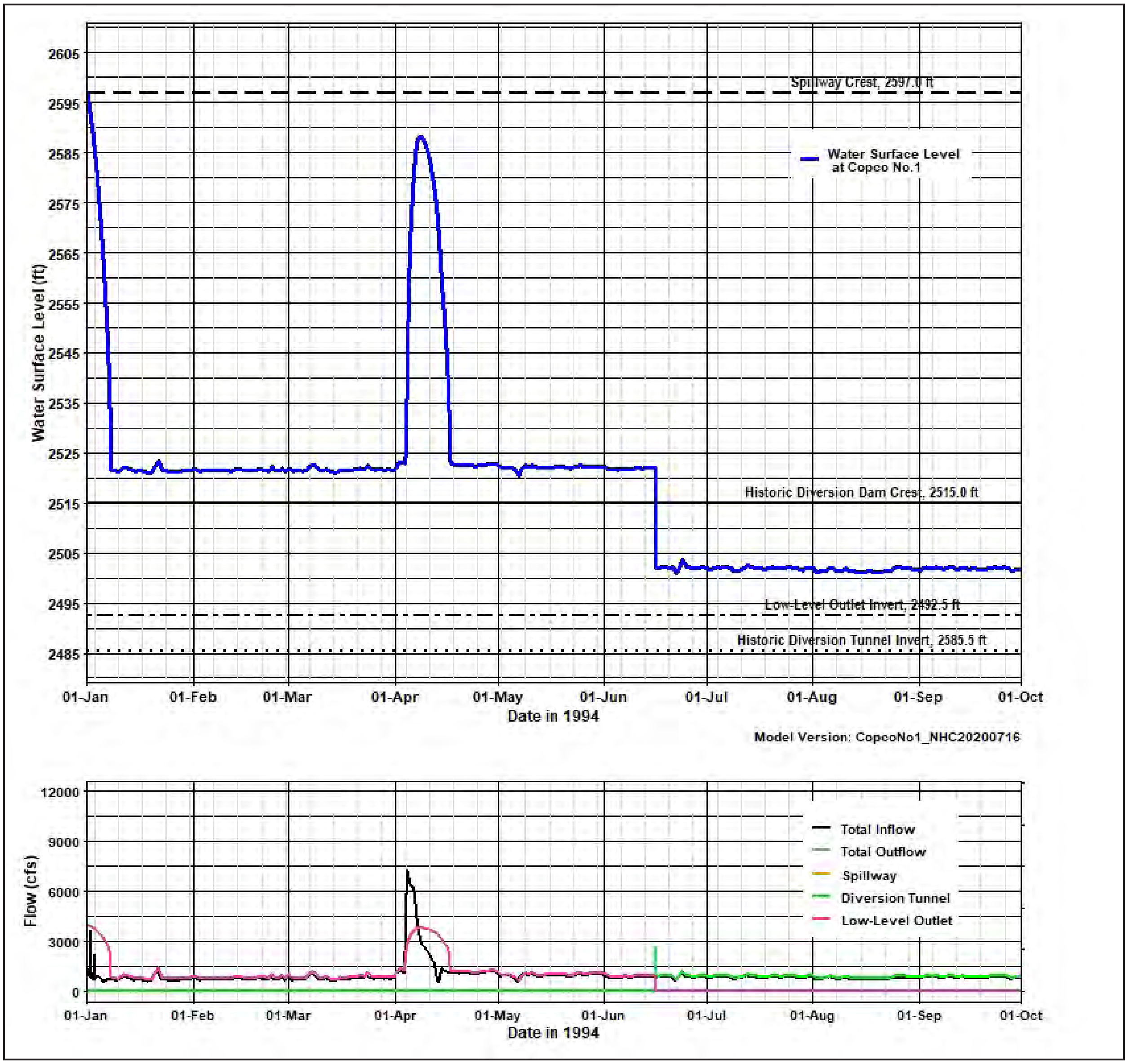

Figure 14 - COPCO No.1 Facility Simulated Drawdown - Year 1994

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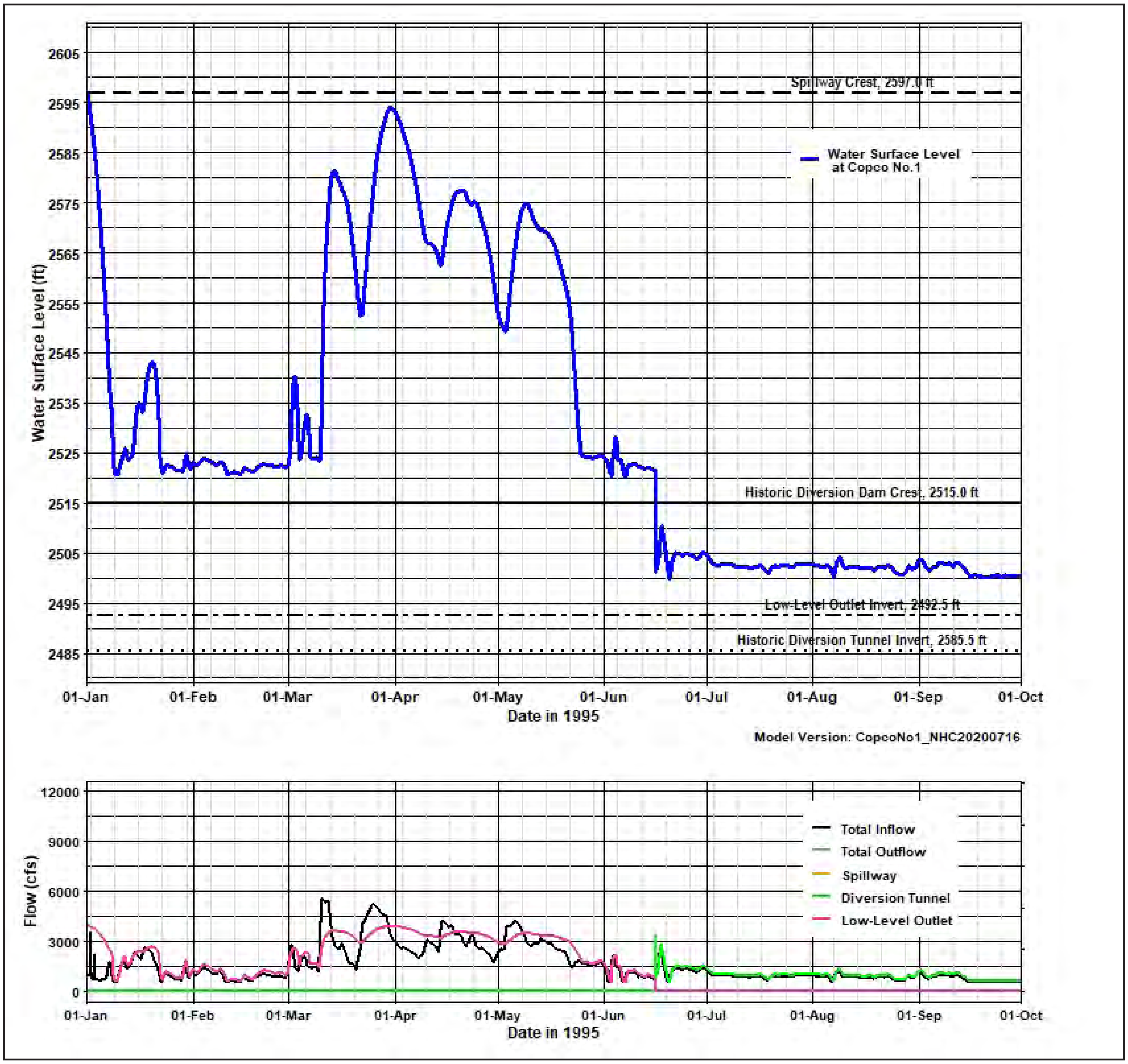

Figure 15 - COPCO No.1 Facility Simulated Drawdown - Year 1995

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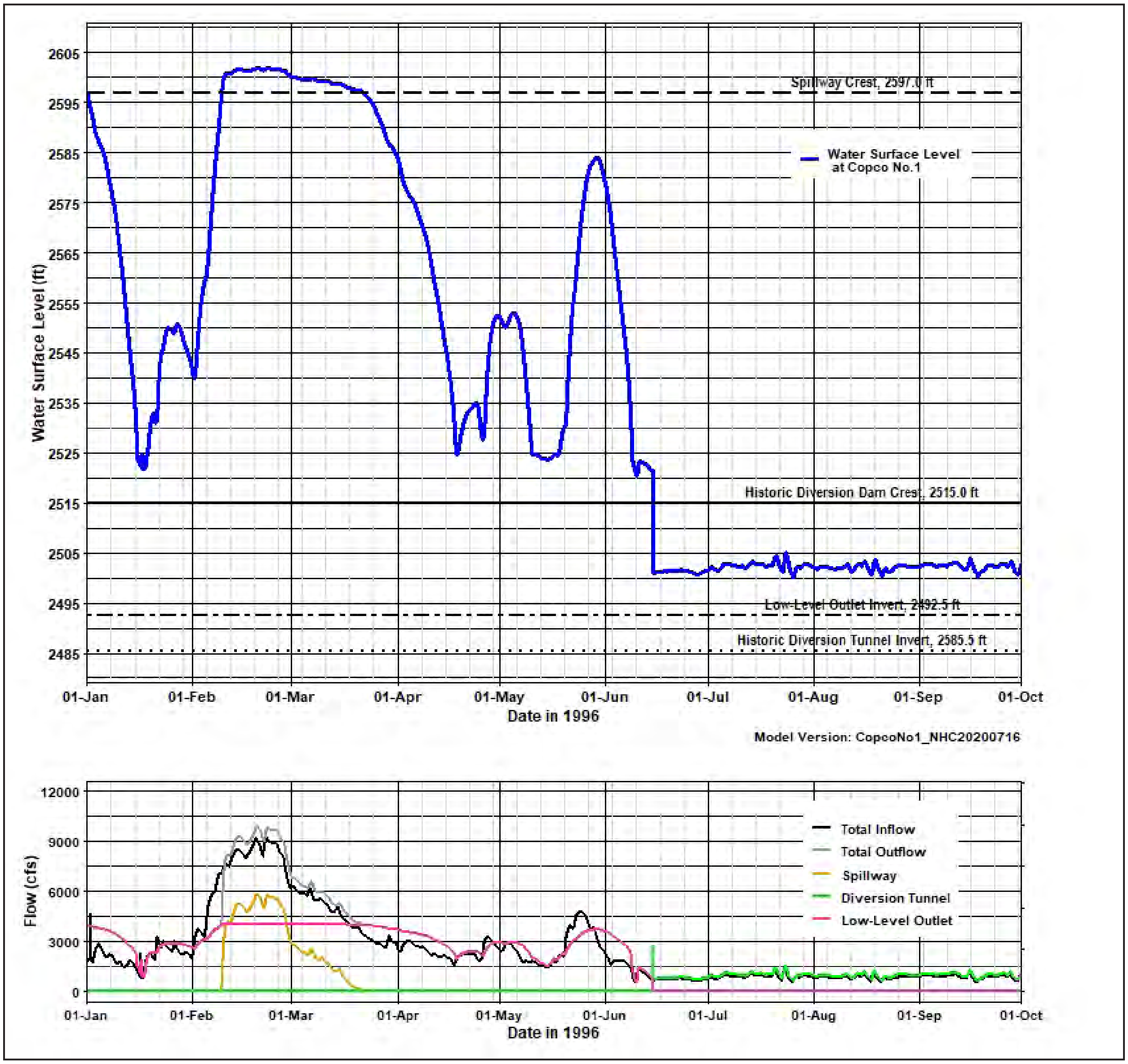

Figure 16 - COPCO No.1 Facility Simulated Drawdown - Year 1996

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

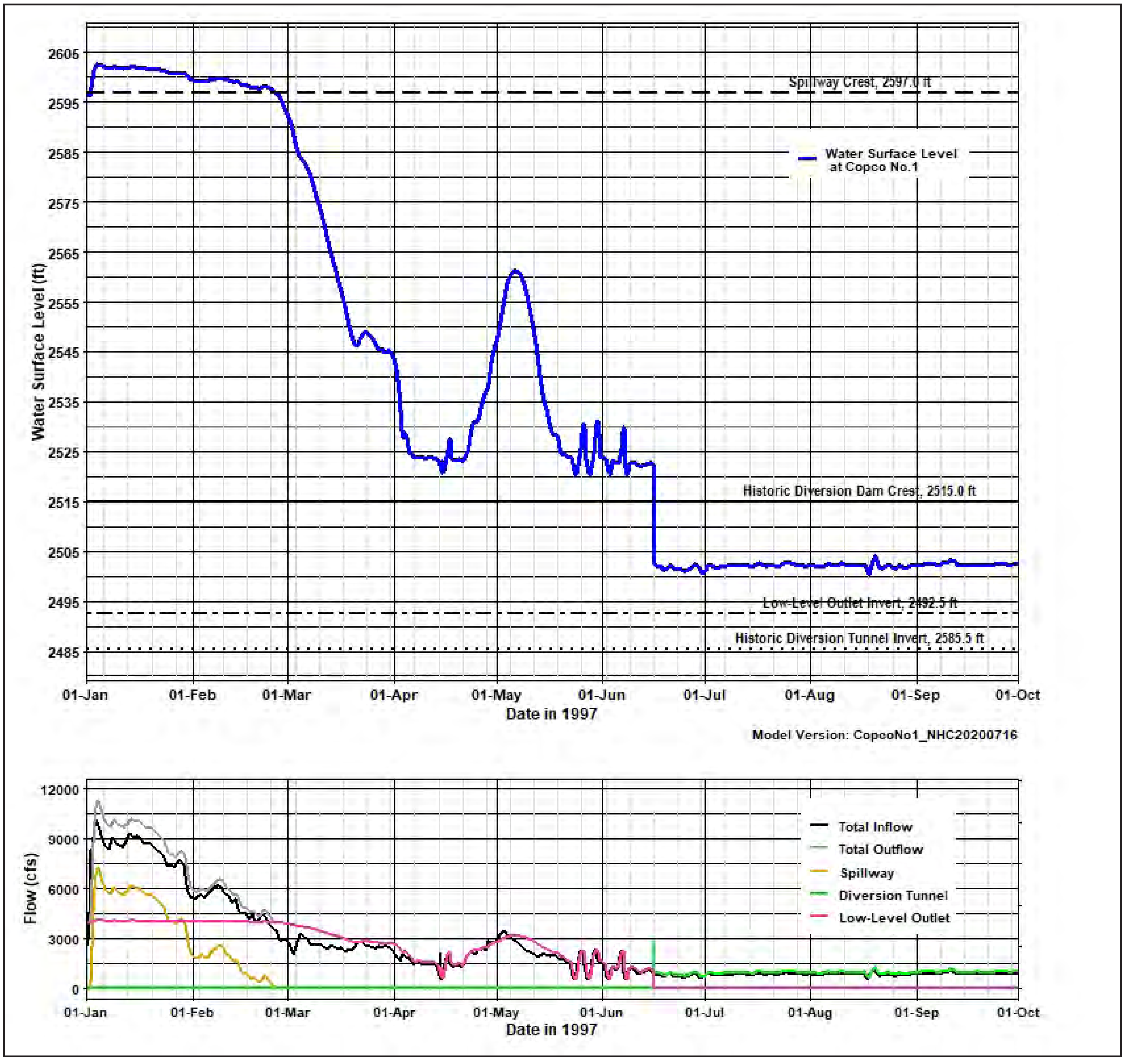

Figure 17 - COPCO No.1 Facility Simulated Drawdown - Year 1997

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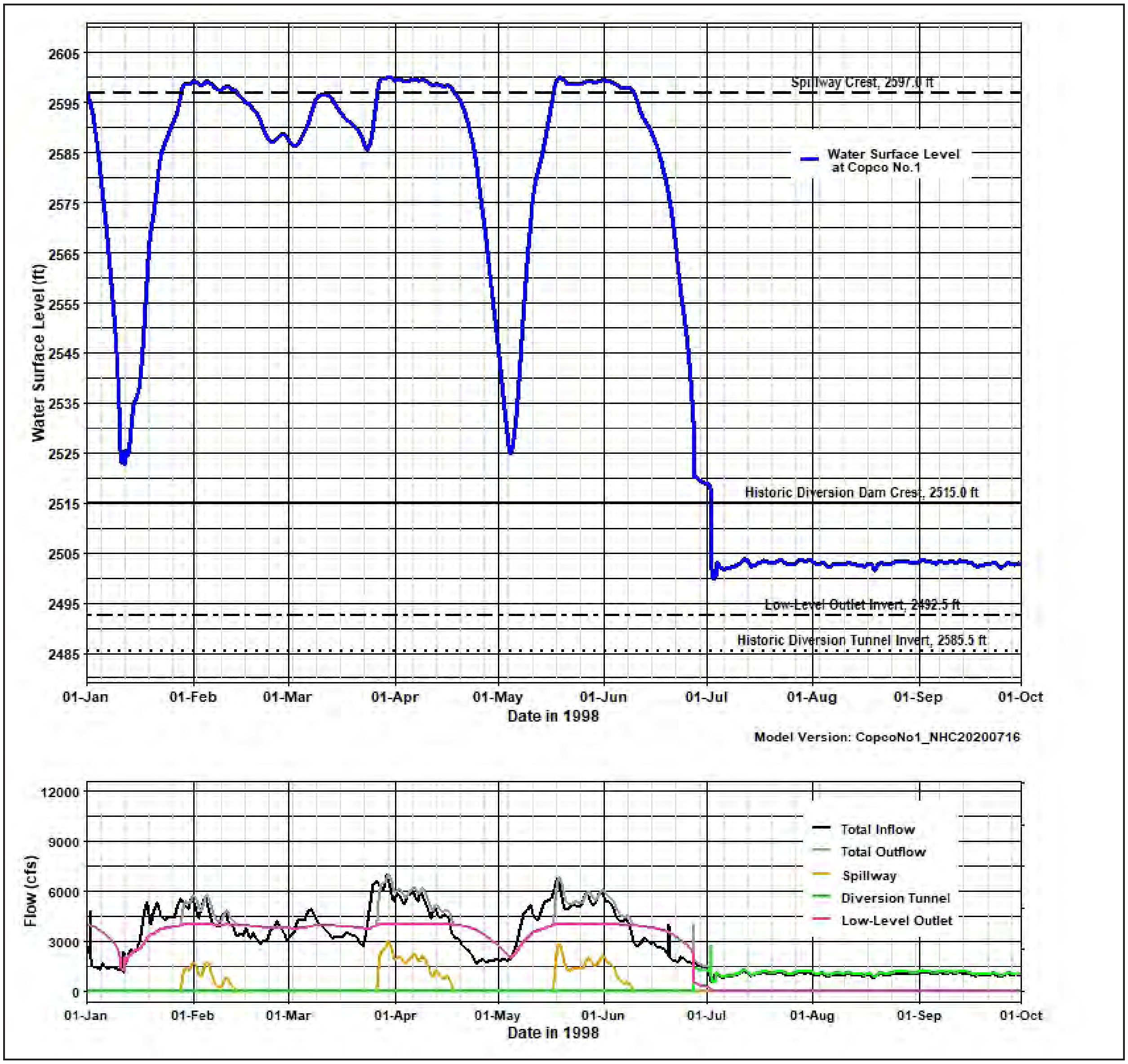

Figure 18 - COPCO No.1 Facility Simulated Drawdown - Year 1998

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

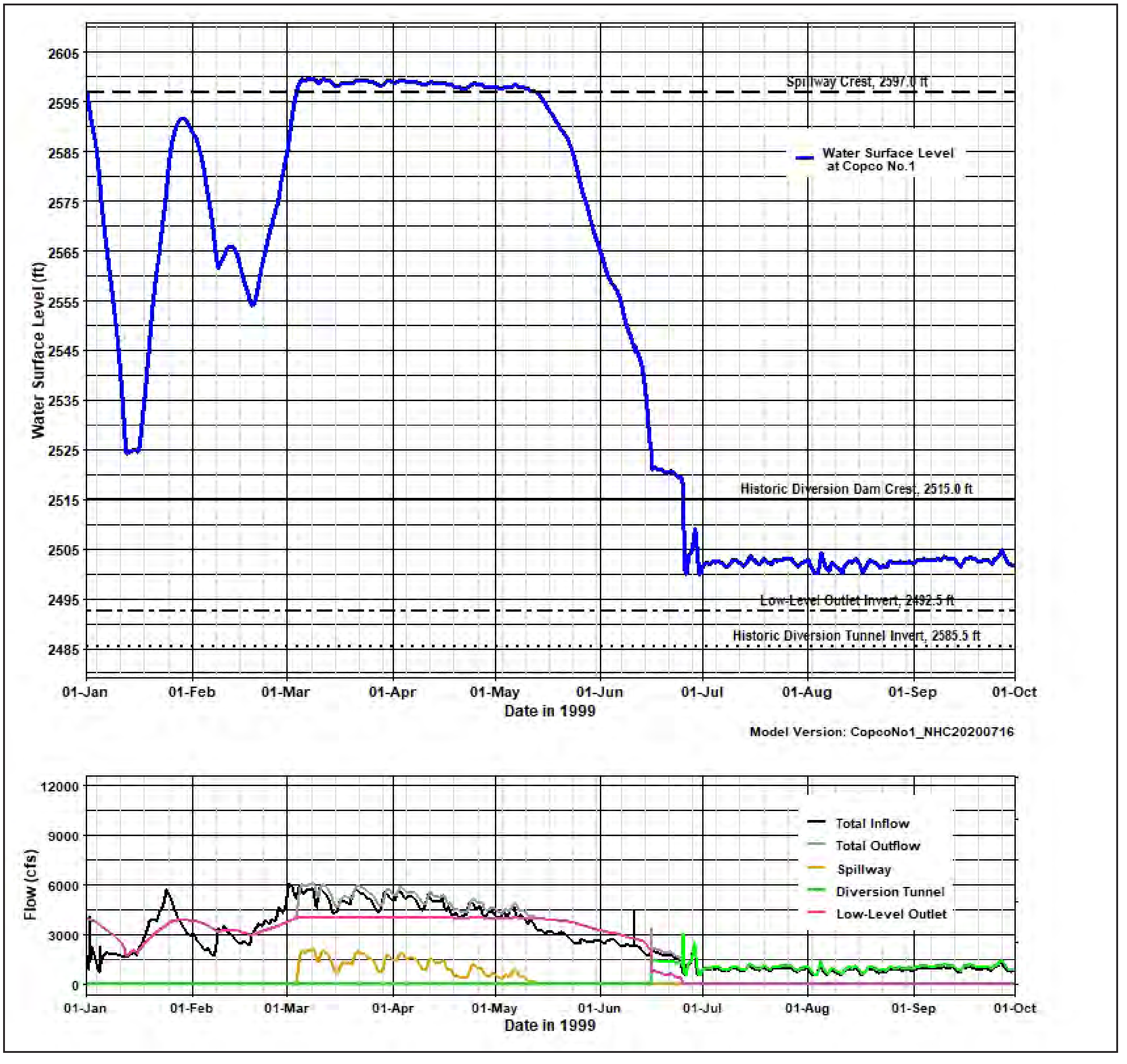

Figure 19 - COPCO No.1 Facility Simulated Drawdown - Year 1999

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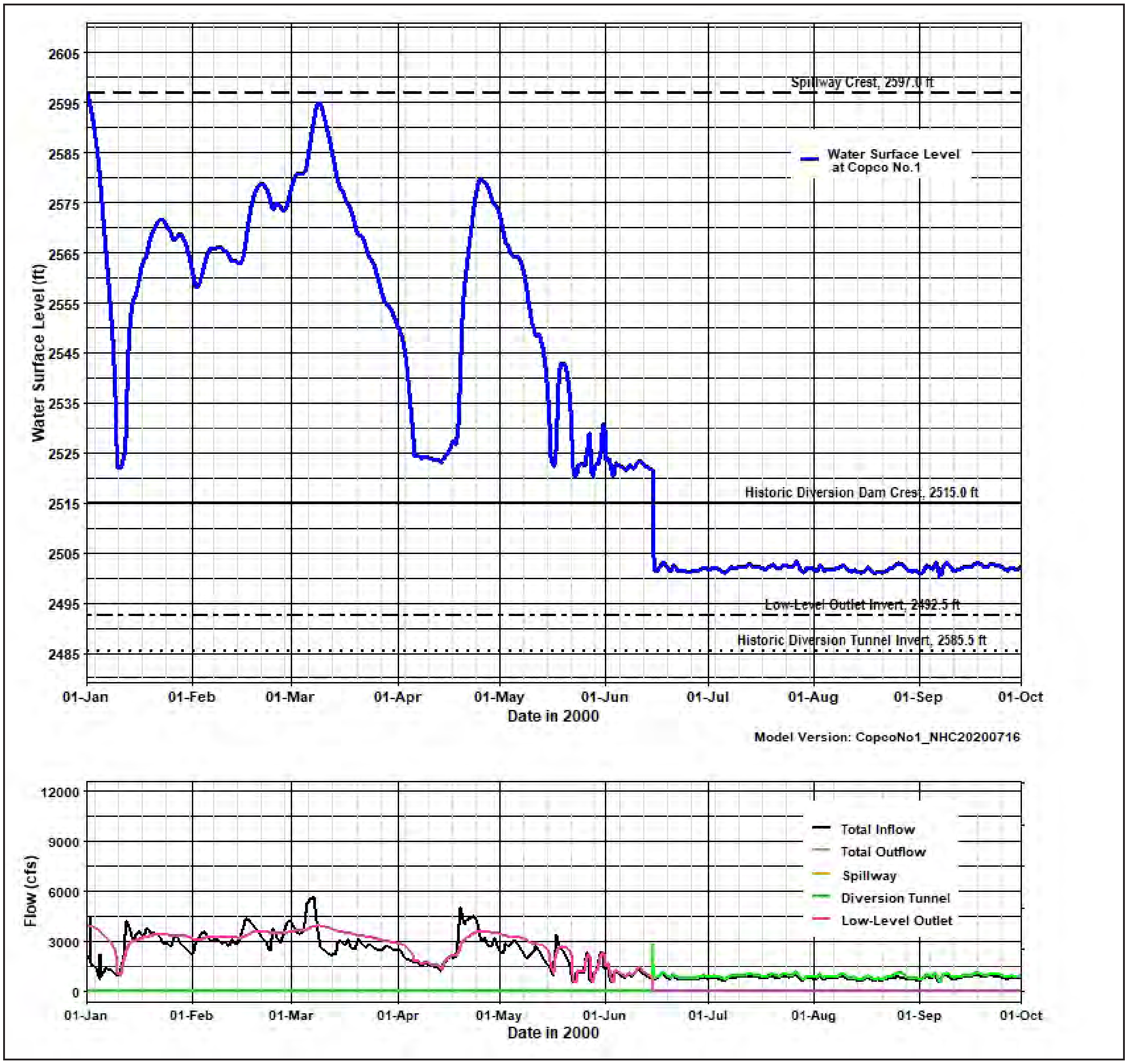

Figure 20 - COPCO No.1 Facility Simulated Drawdown - Year 2000

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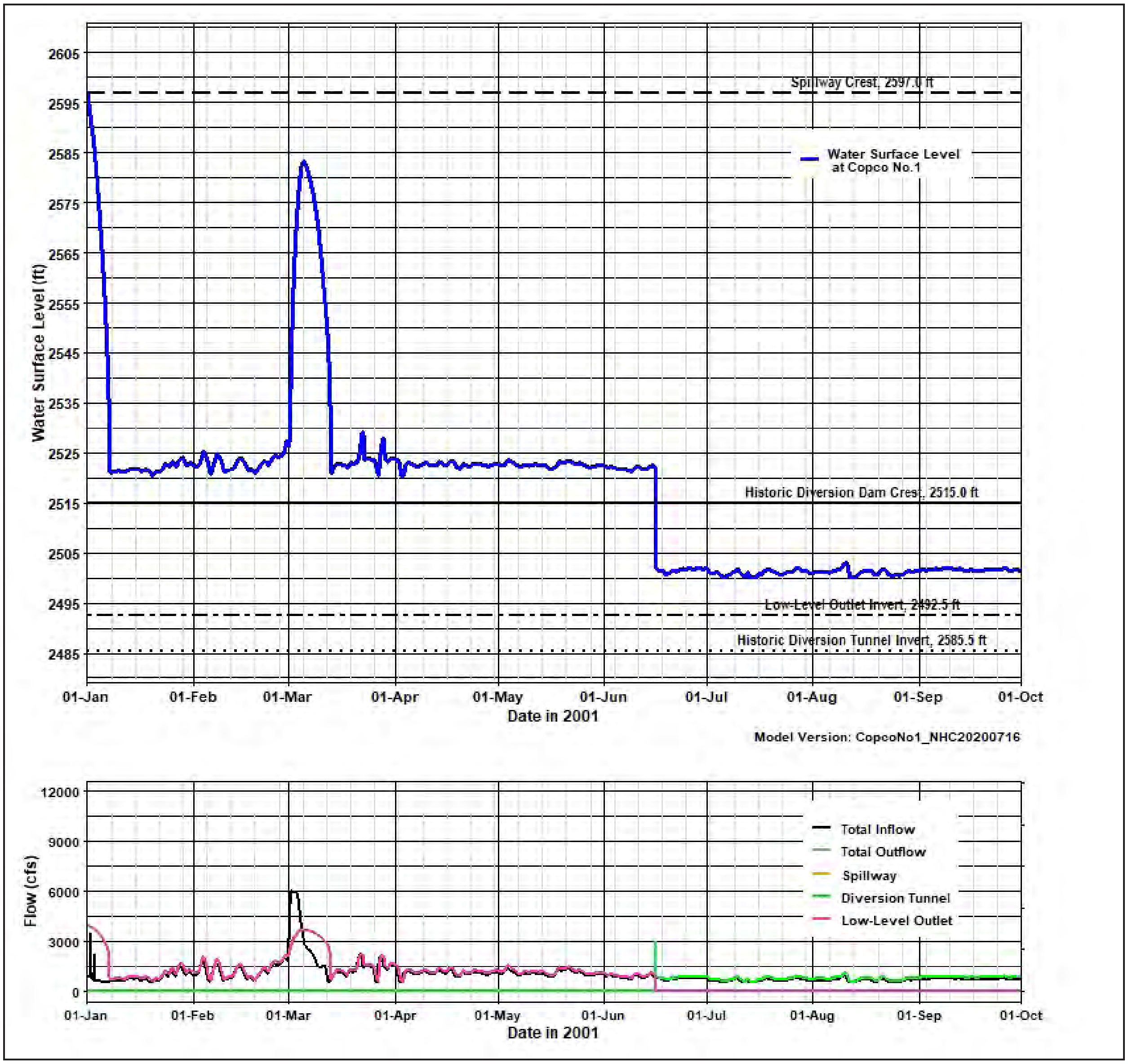

Figure 21 - COPCO No.1 Facility Simulated Drawdown - Year 2001

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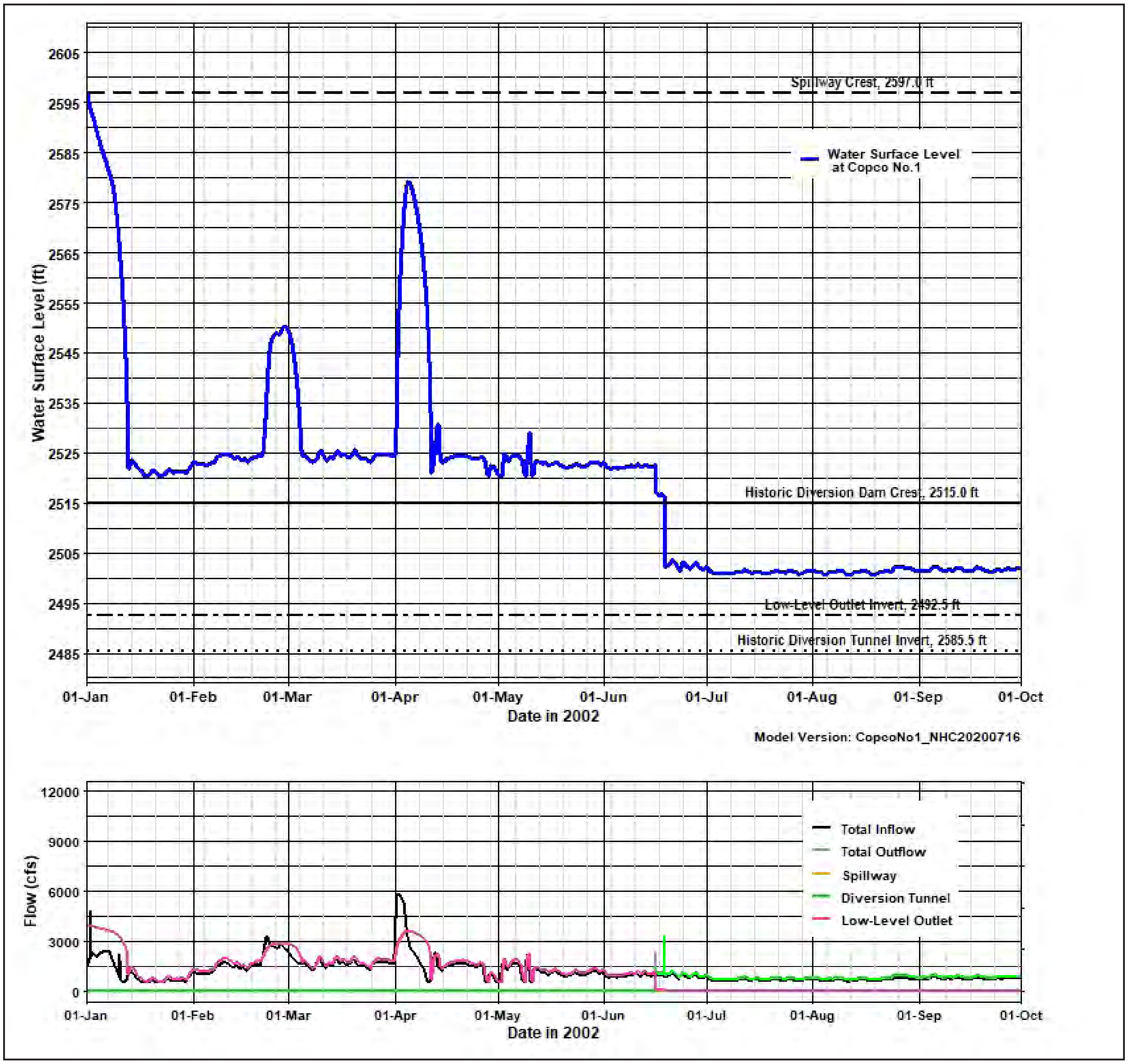

Figure 22 - COPCO No.1 Facility Simulated Drawdown - Year 2002

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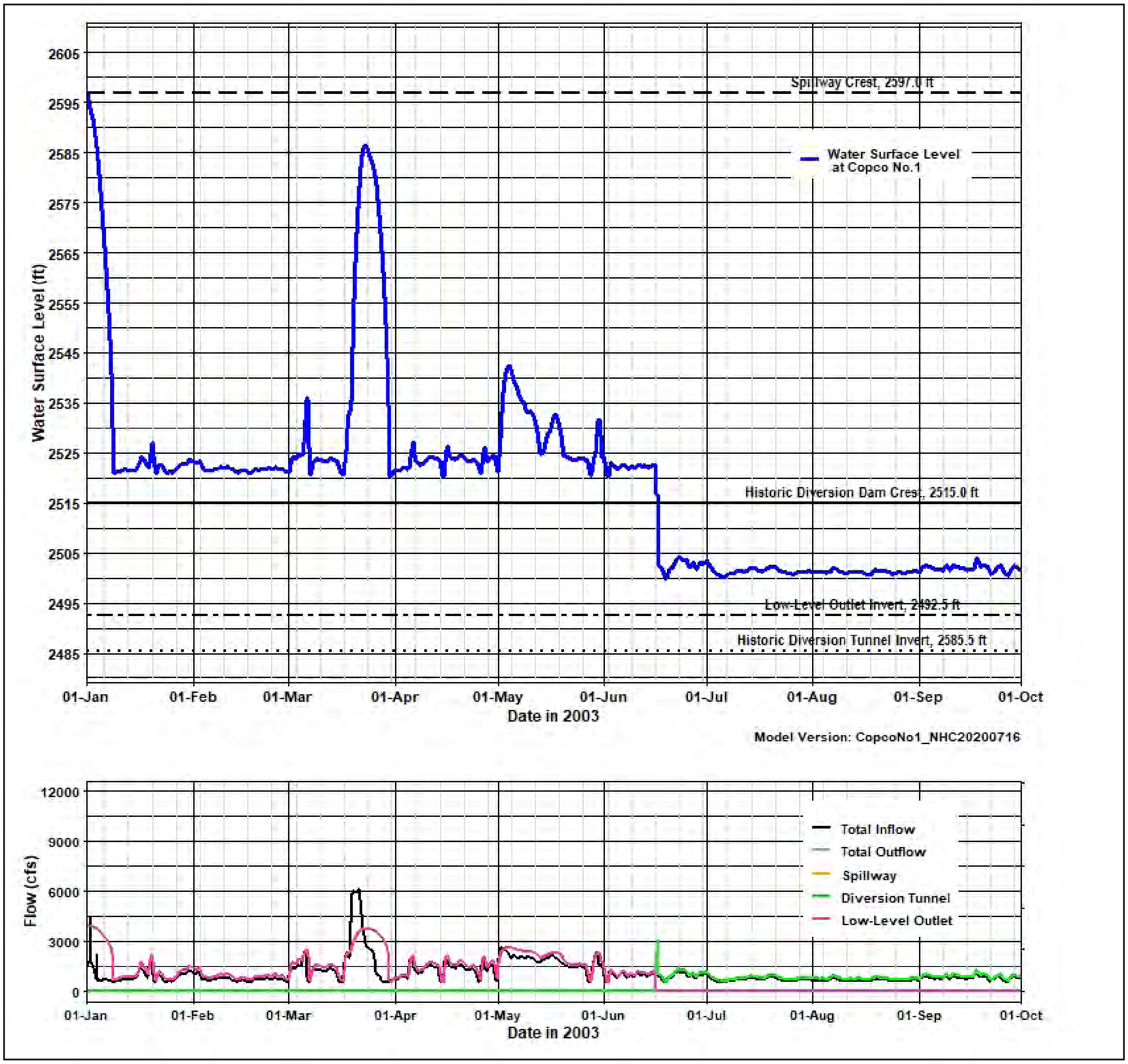

Figure 23 - COPCO No.1 Facility Simulated Drawdown - Year 2003

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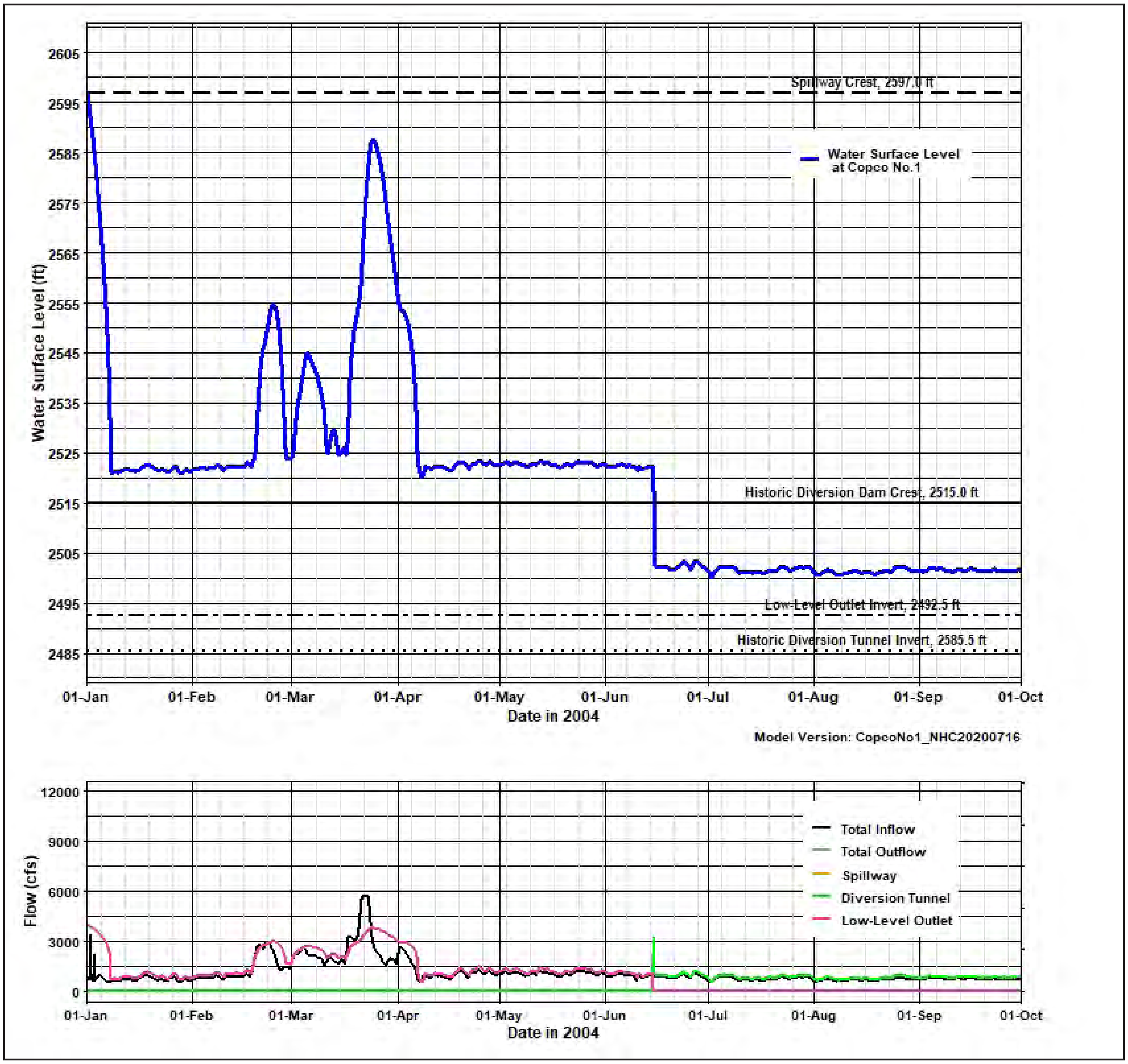

Figure 24 - COPCO No.1 Facility Simulated Drawdown - Year 2004

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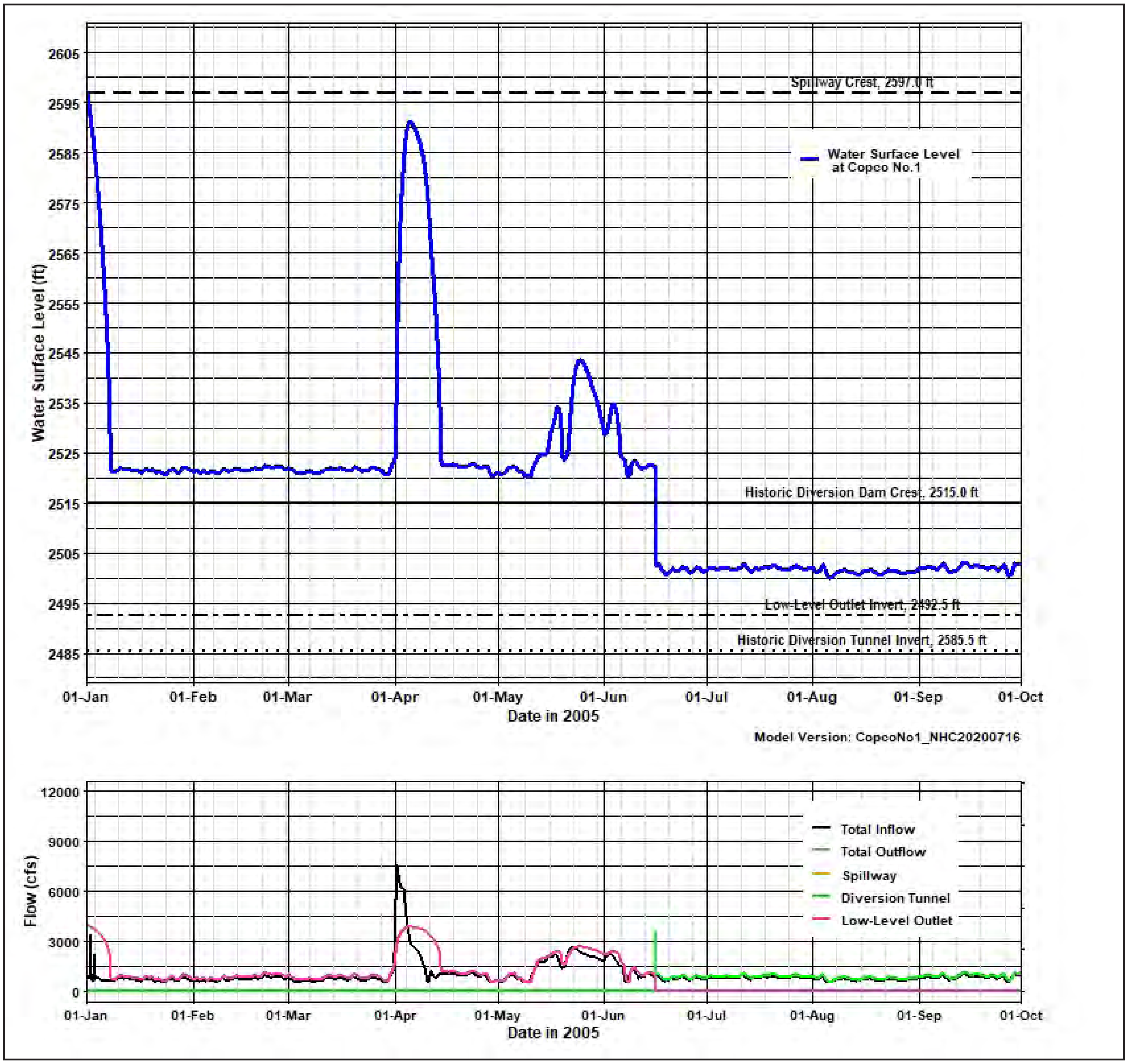

Figure 25 - COPCO No.1 Facility Simulated Drawdown - Year 2005

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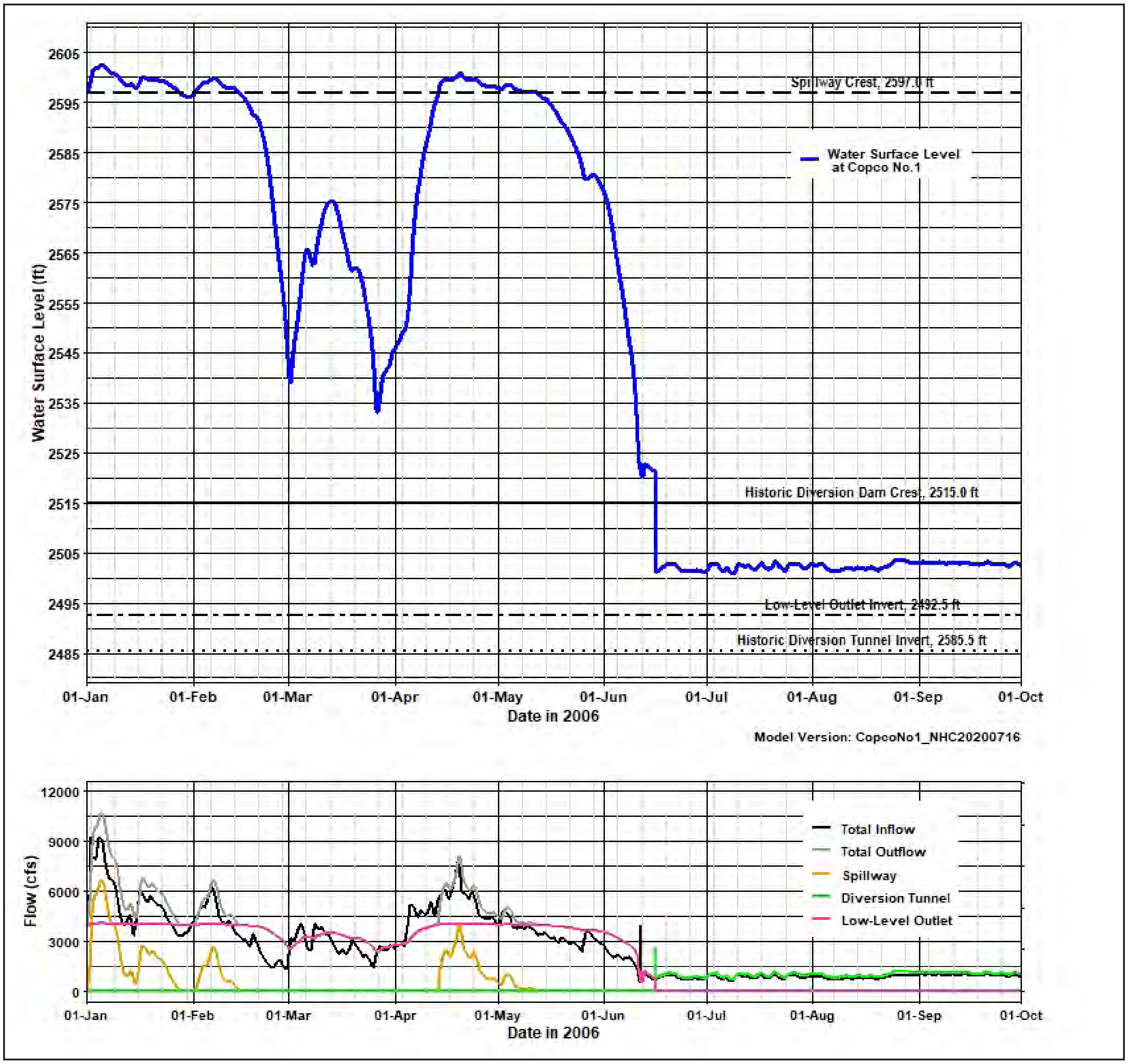

Figure 26 - COPCO No.1 Facility Simulated Drawdown - Year 2006

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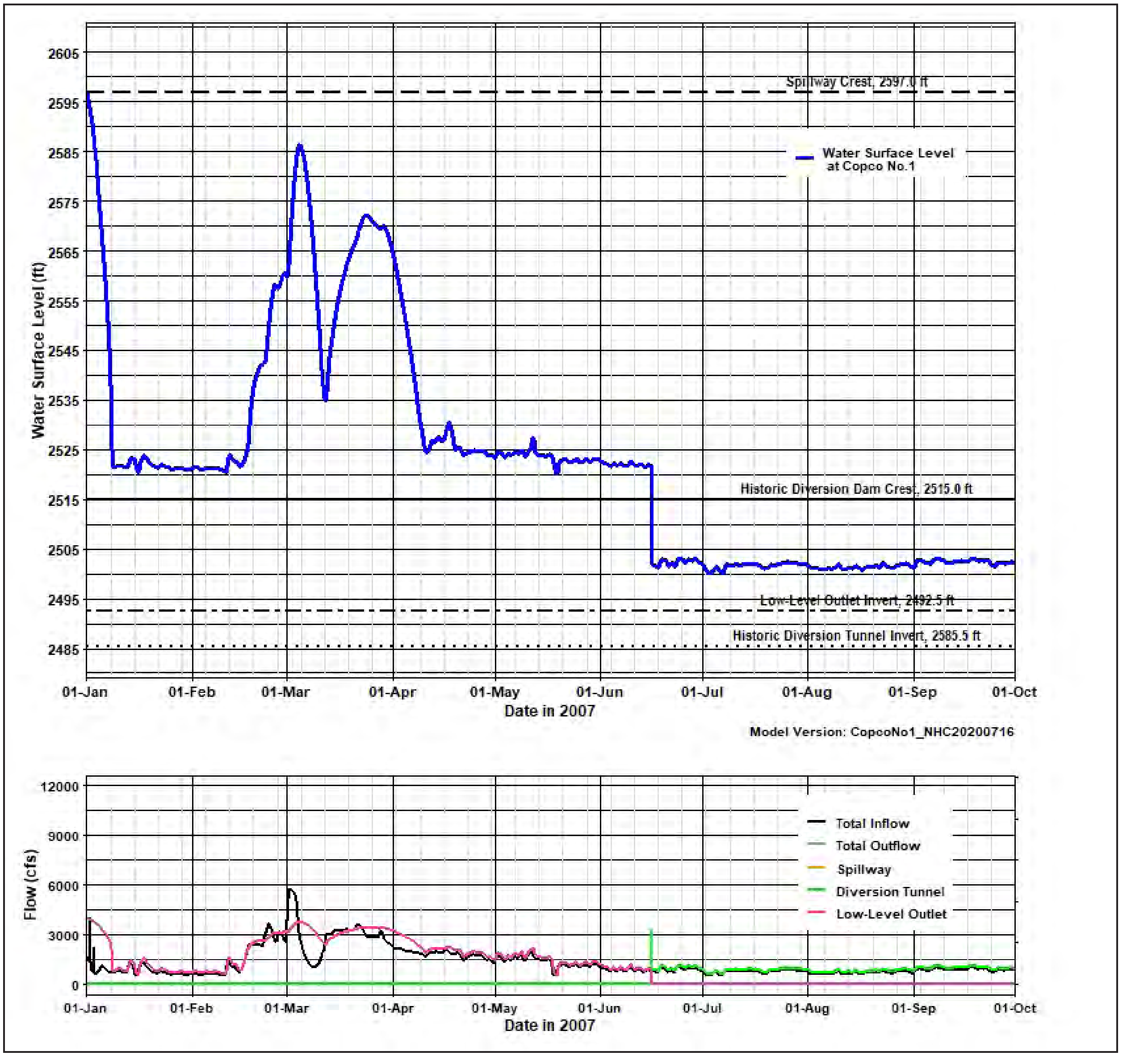

Figure 27 - COPCO No.1 Facility Simulated Drawdown - Year 2007

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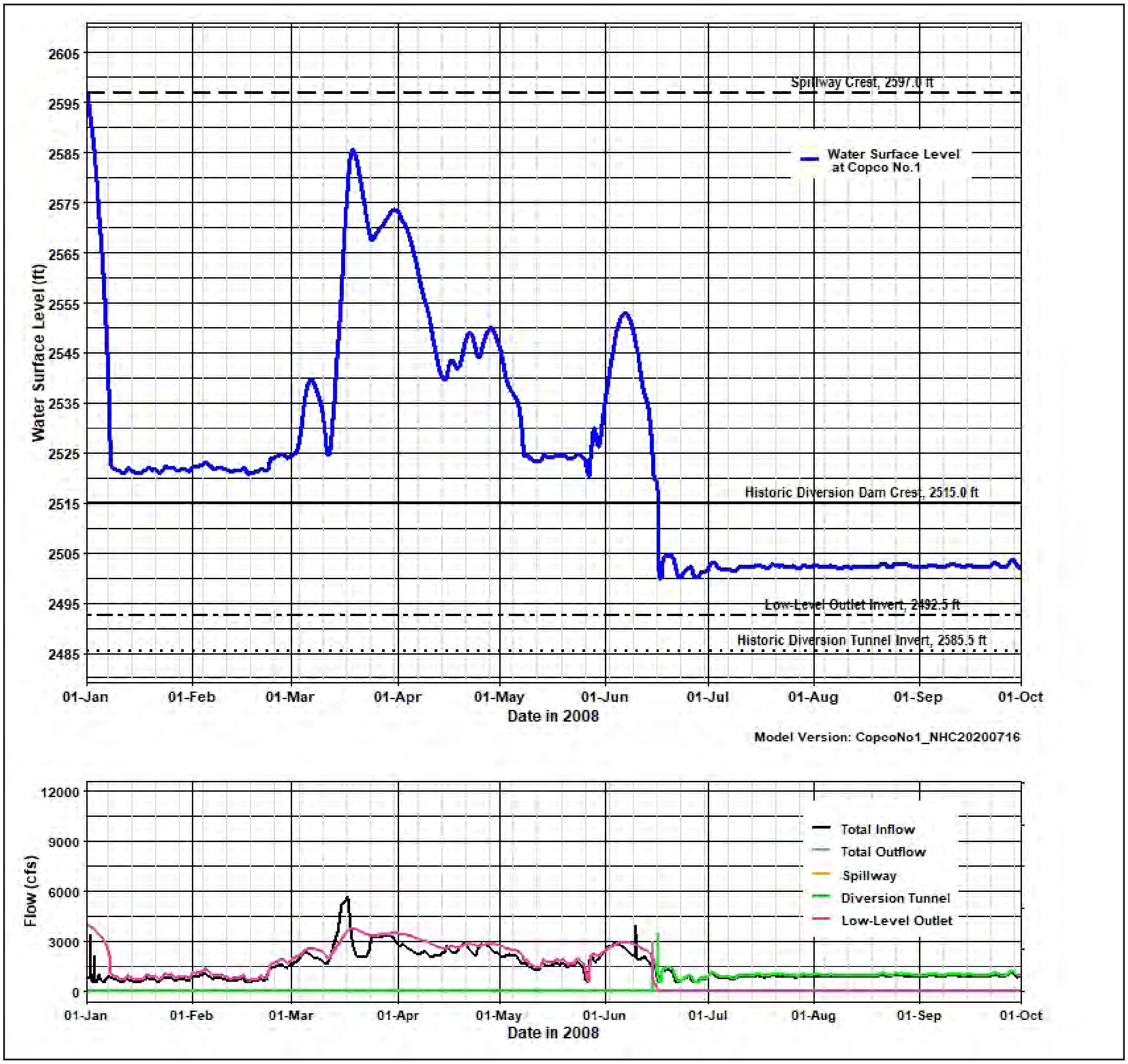

Figure 28 - COPCO No.1 Facility Simulated Drawdown - Year 2008

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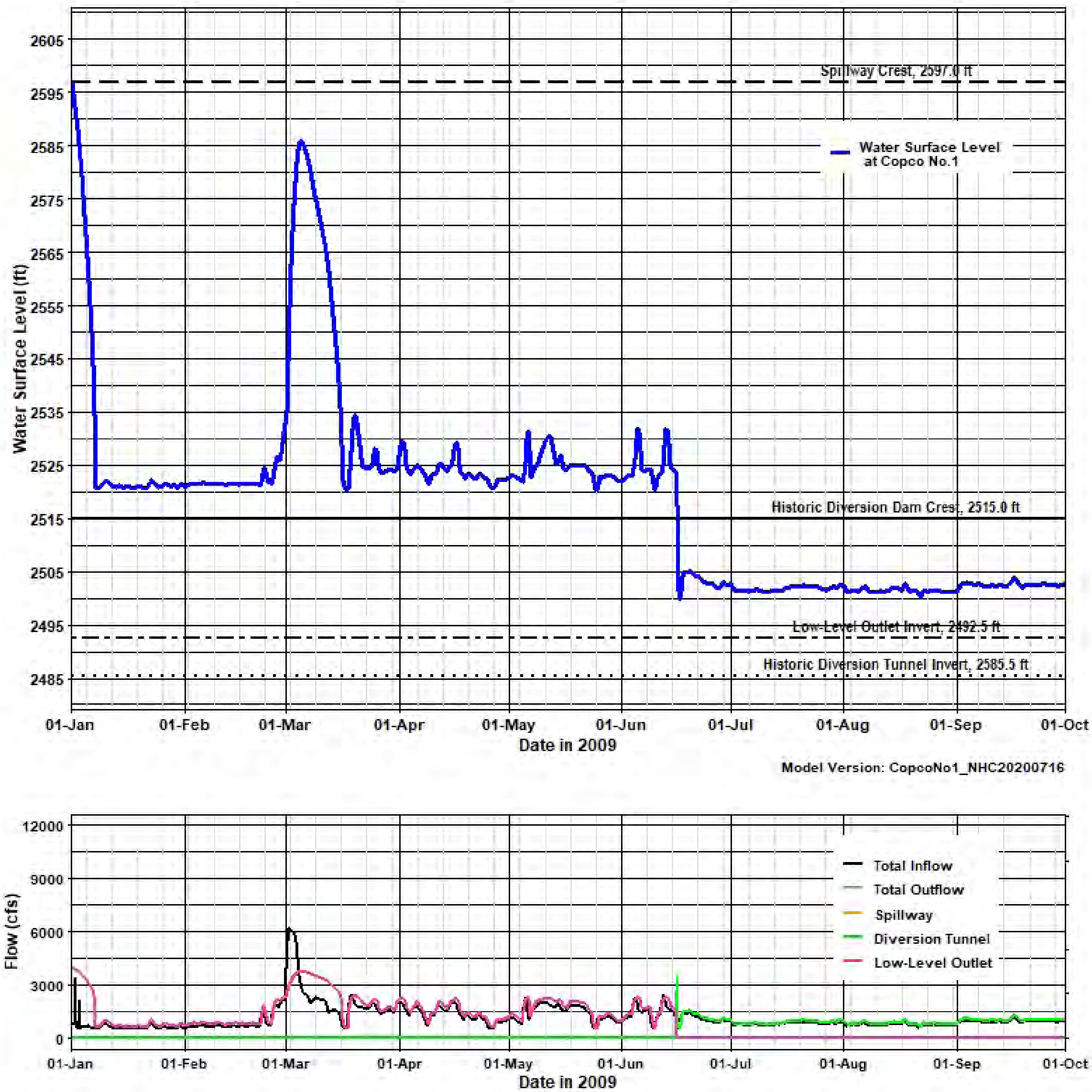

Figure 29 - COPCO No.1 Facility Simulated Drawdown - Year 2009

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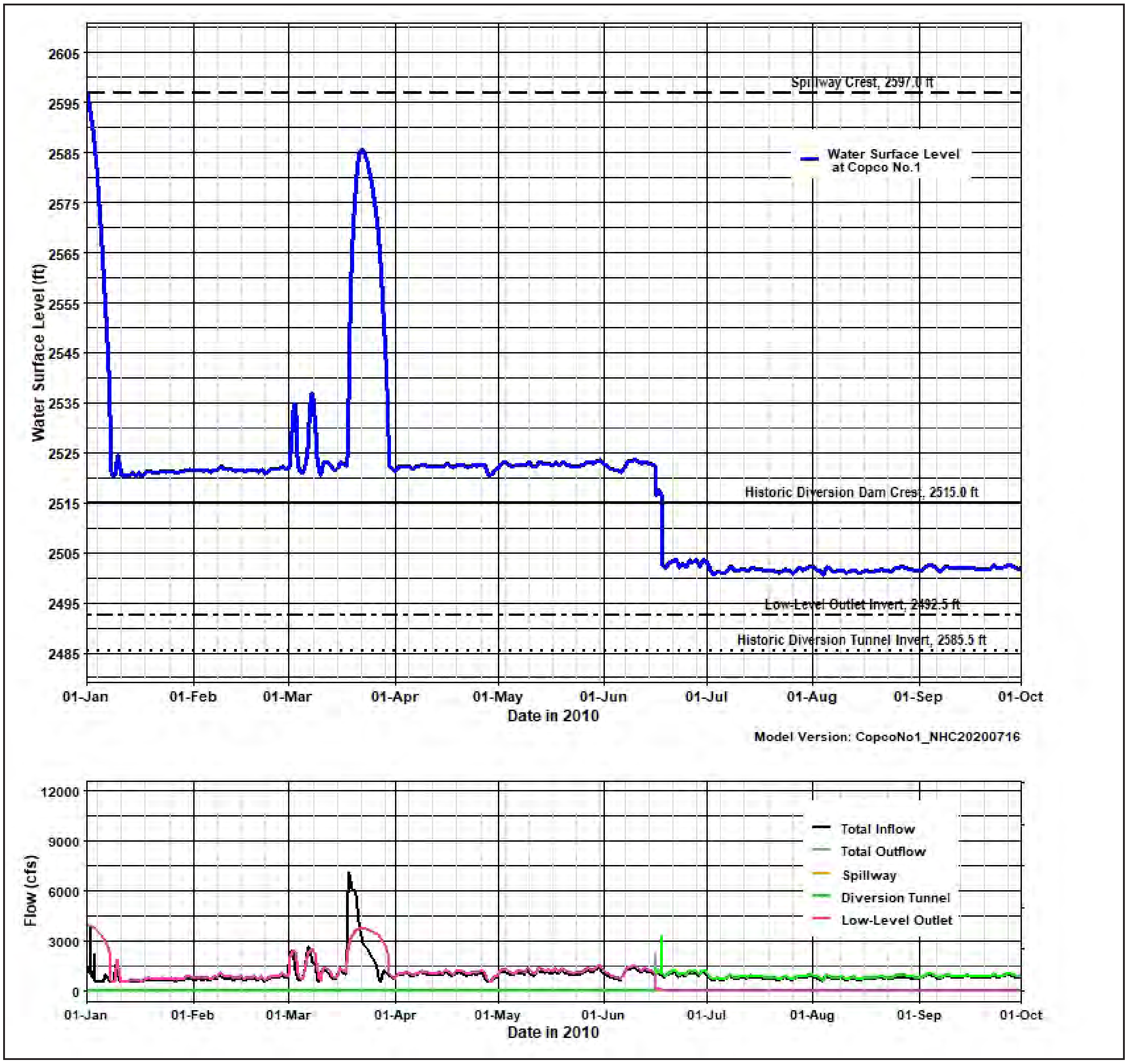

Figure 30 - COPCO No.1 Facility Simulated Drawdown - Year 2010

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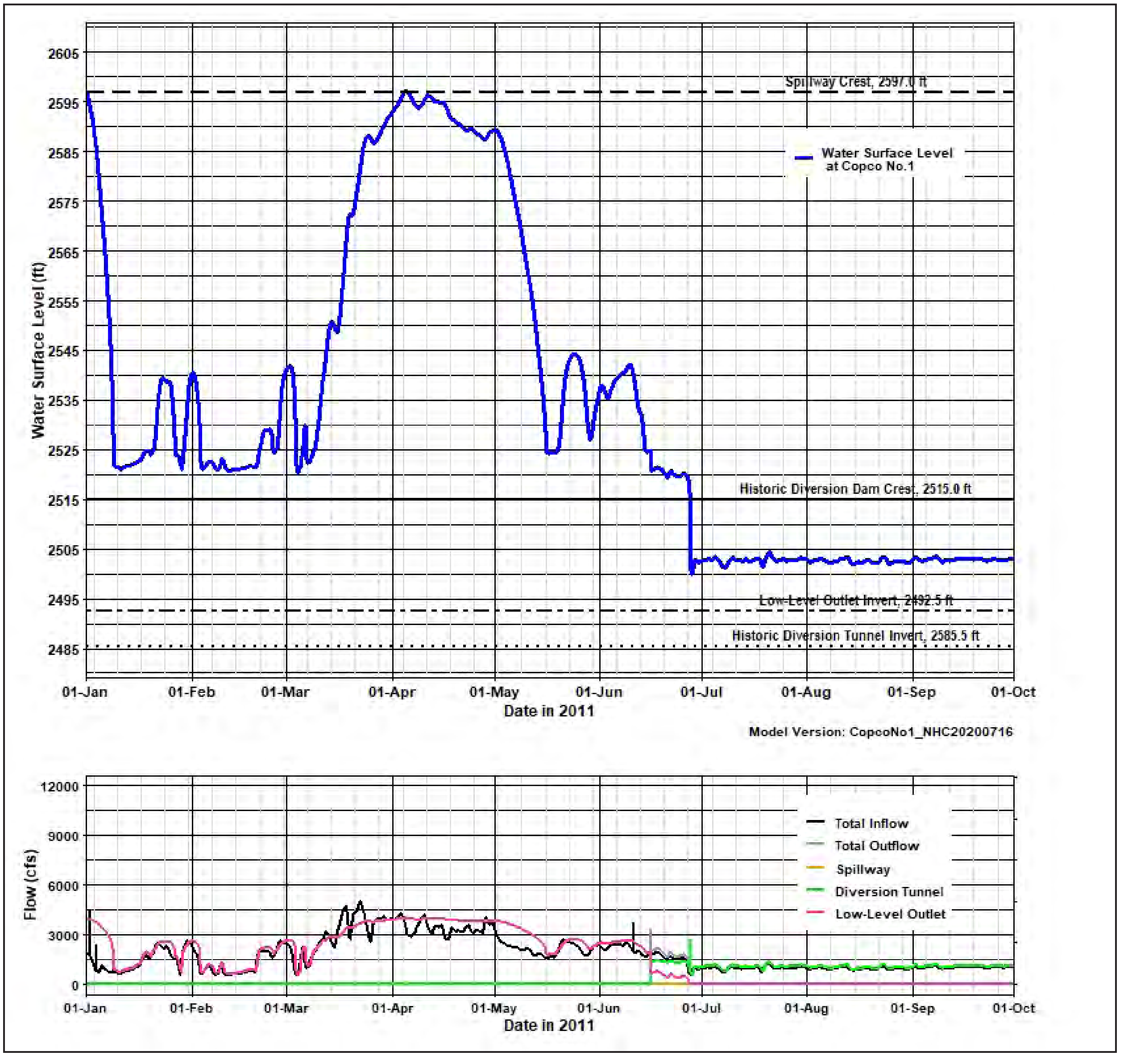

Figure 31 - COPCO No.1 Facility Simulated Drawdown - Year 2011

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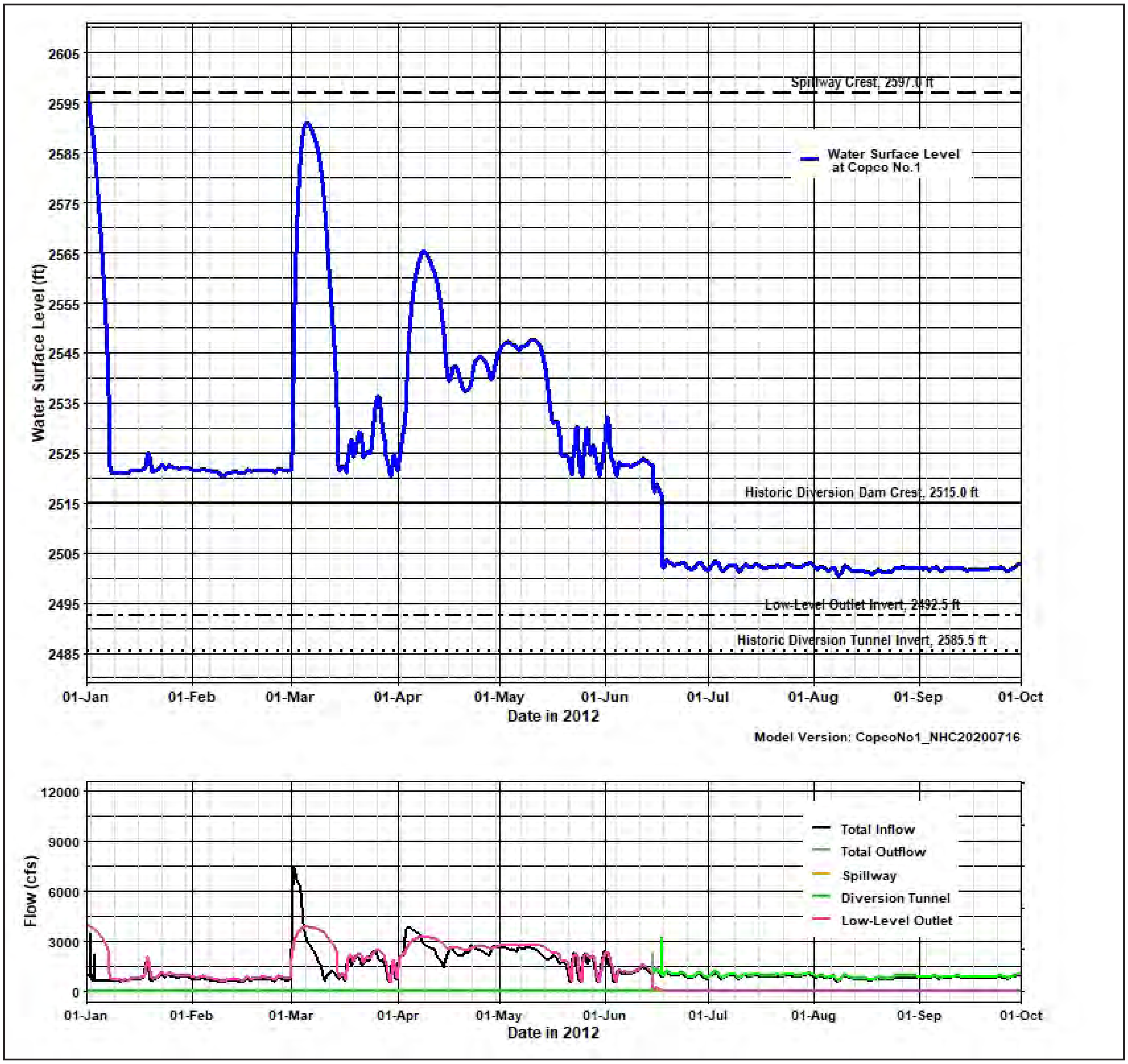

Figure 32 - COPCO No.1 Facility Simulated Drawdown - Year 2012

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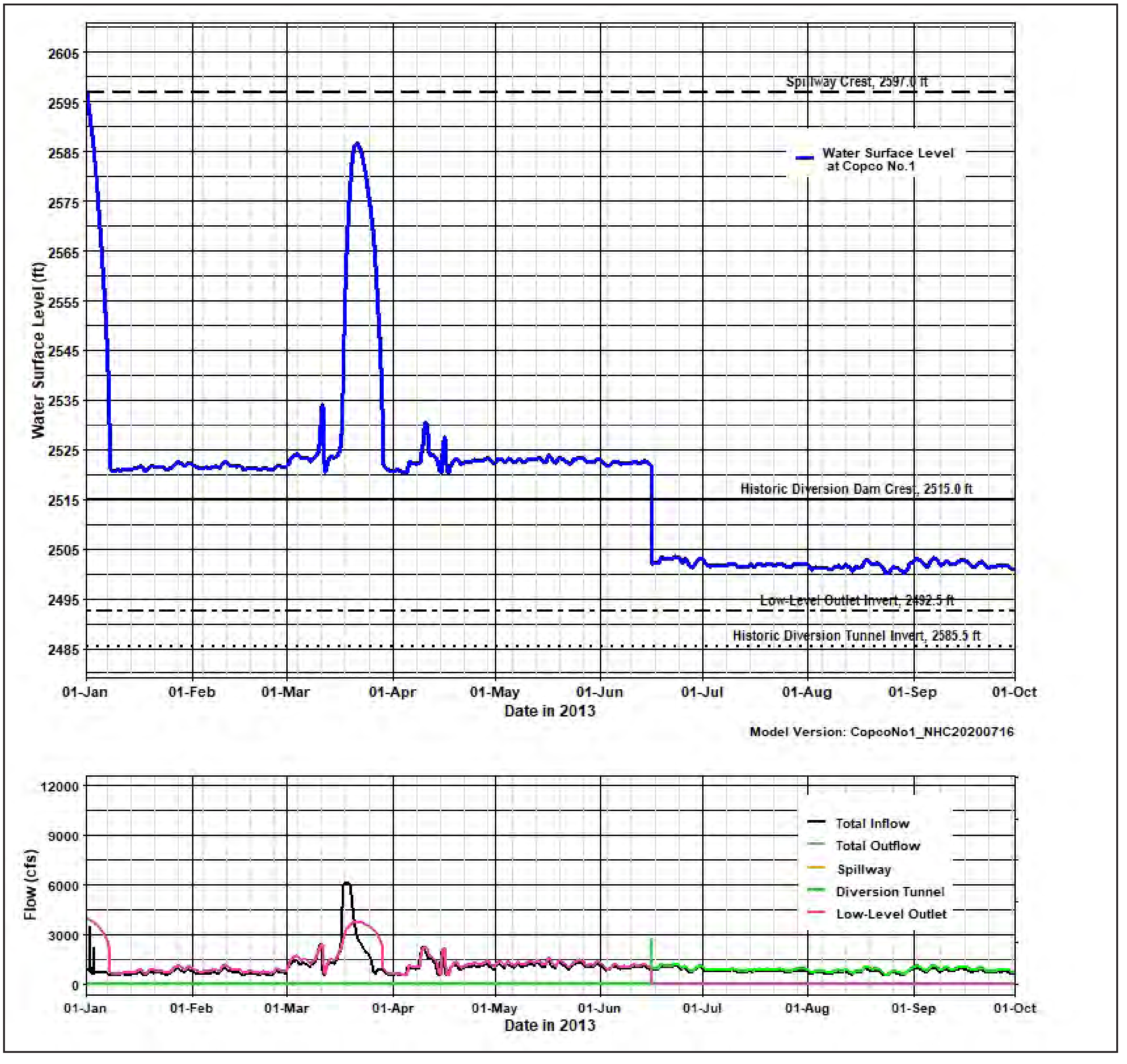

Figure 33 - COPCO No.1 Facility Simulated Drawdown - Year 2013

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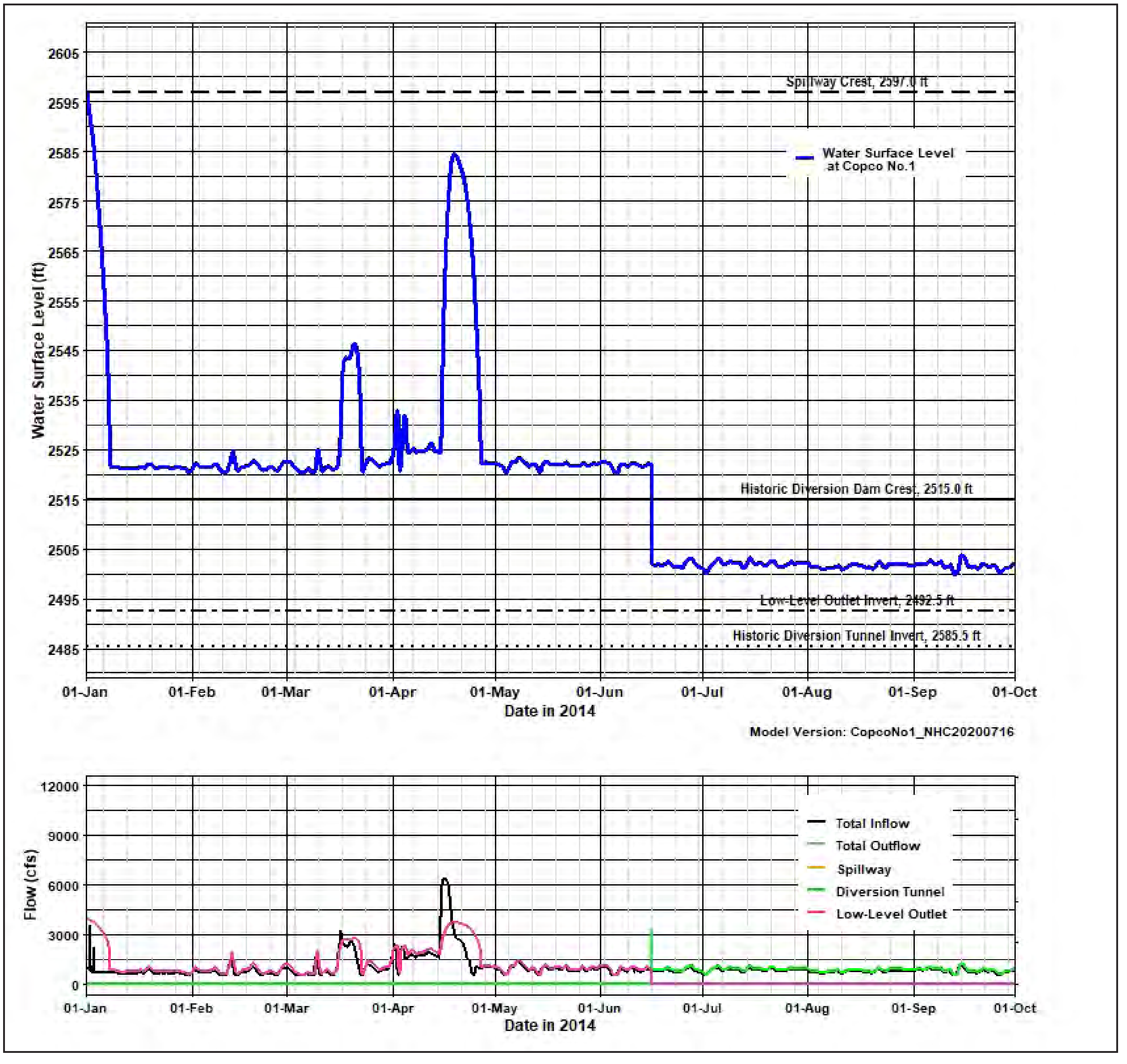

Figure 34 - COPCO No.1 Facility Simulated Drawdown - Year 2014

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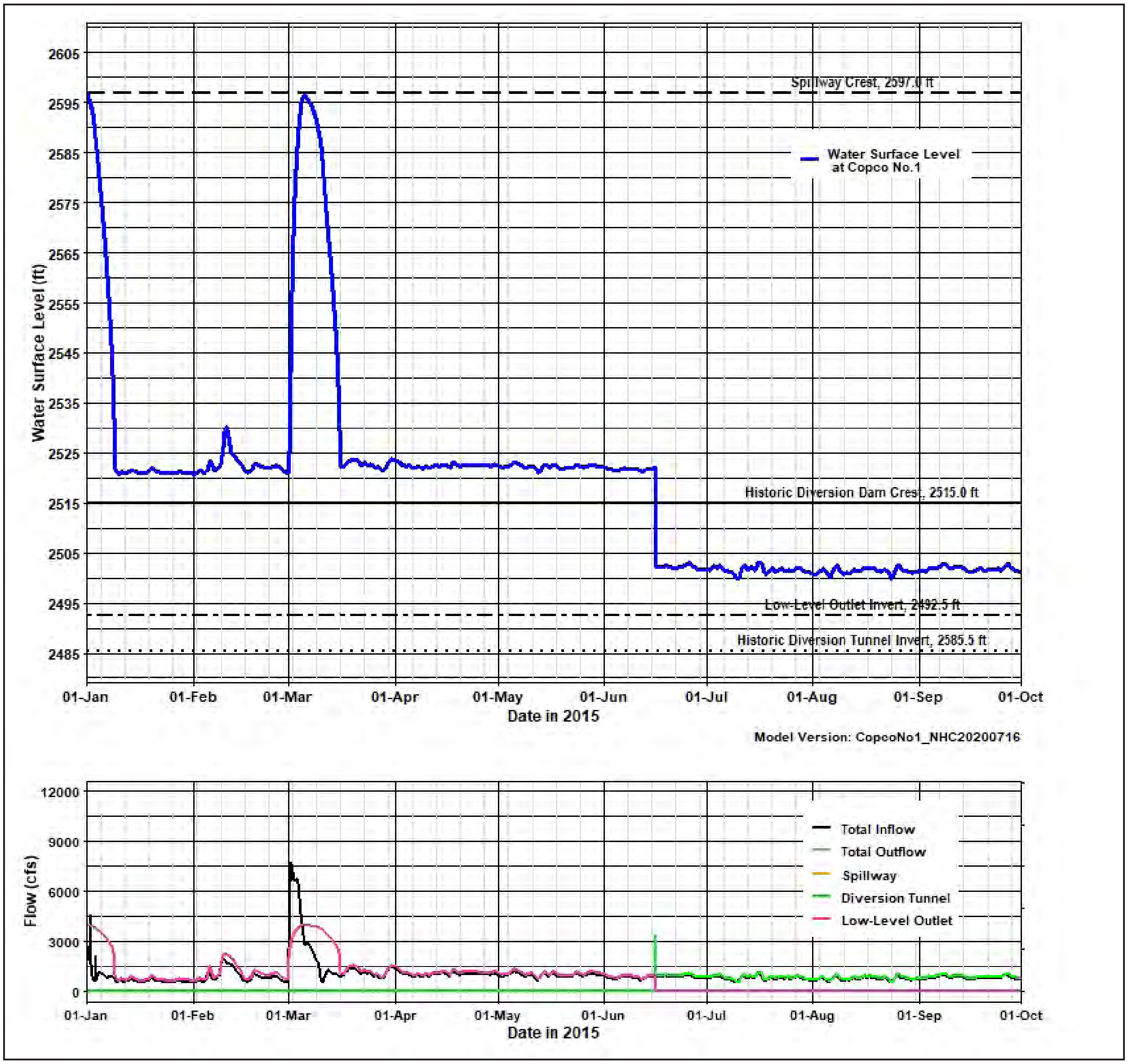

Figure 35 - COPCO No.1 Facility Simulated Drawdown - Year 2015

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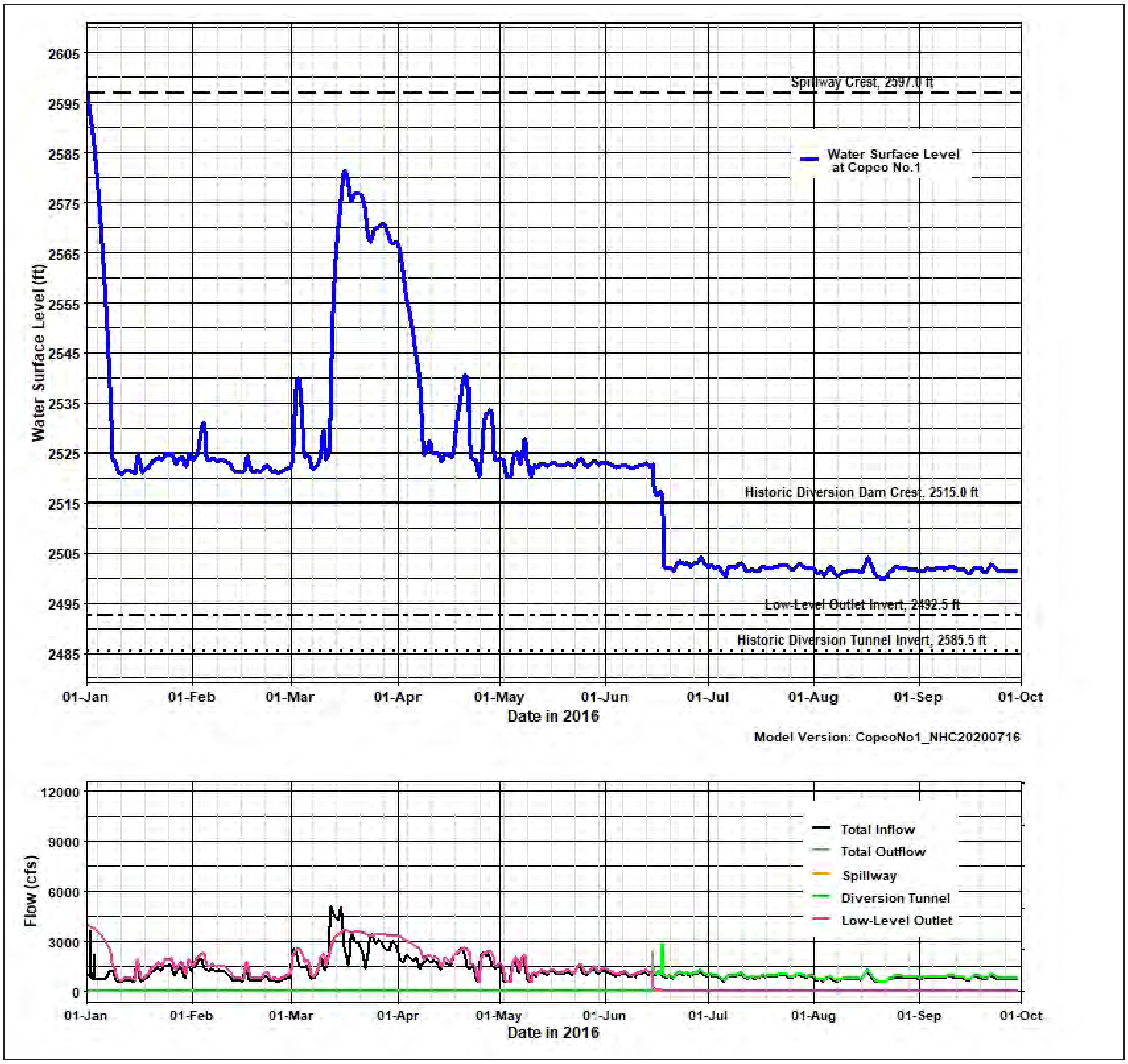

Figure 36 - COPCO No.1 Facility Simulated Drawdown - Year 2016

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nhc
northwest hydraulic consultants

NHC Ref. No. 2004947.11.3

21 September 2020

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

Attention: **Norm Bishop**

Re: CFD Modeling of Copco No.1 Dam, 100% Design

1 INTRODUCTION

Copco No. 1 reservoir will be drawn down using a 10.5-ft by 15 ft concrete and 126-inch diameter steel low-level outlet (LLO). The upstream D-shape portion of the LLO will be drilled through the dam's concrete body and then connected to a steel pipe downstream (Figure 1). The drilling will leave a 10-ft circular plug at the dam's upstream face to be removed when needed for reservoir drawdown (pre-drawdown maximum reservoir level is 2,597.1 ft). The LLO's invert elevations are El. 2,492.5 ft at the inlet section and El. 2,477.3 ft at the outlet section. The 100% design is shown in KP Figures C2205 – General Arrangement; C2210 – Approach Channel; C2227 – Outlet Conduit (steel Details); C2225 – Outlet Tunnel (Concrete Outline); and, C2226 – Outlet Transition (D-shape to outlet conduit square to round transition).

NHC conducted numerical modelling to:

1. Develop pre- and post-construction tailwater rating curves at the Copco No. 1 plunge pool.
2. Develop a rating curve for the Copco No. 1 LLO.
3. Assess sediment mobility in the Copco No.1 Reservoir when the LLO and Historic Diversion Tunnel operate.

The tailwater rating curves were developed using a two-dimensional (2D) depth-averaged flow model. The LLO rating curve development and assessment of sediment mobility were conducted using a three-dimensional (3D) computational fluid dynamics (CFD) model.

The tailwater rating curves were used as boundary condition for the CFD model. The LLO rating curve is used as input data for NHC's 100 percent design one-dimensional (1D) HEC-RAS drawdown model.

The hydraulics of the Spillway, Powerhouse and Historic Diversion Tunnel are not part of the present CFD analysis. As the single-phase CFD model only deals with water, air demand and venting requirements of the LLO were not analyzed.

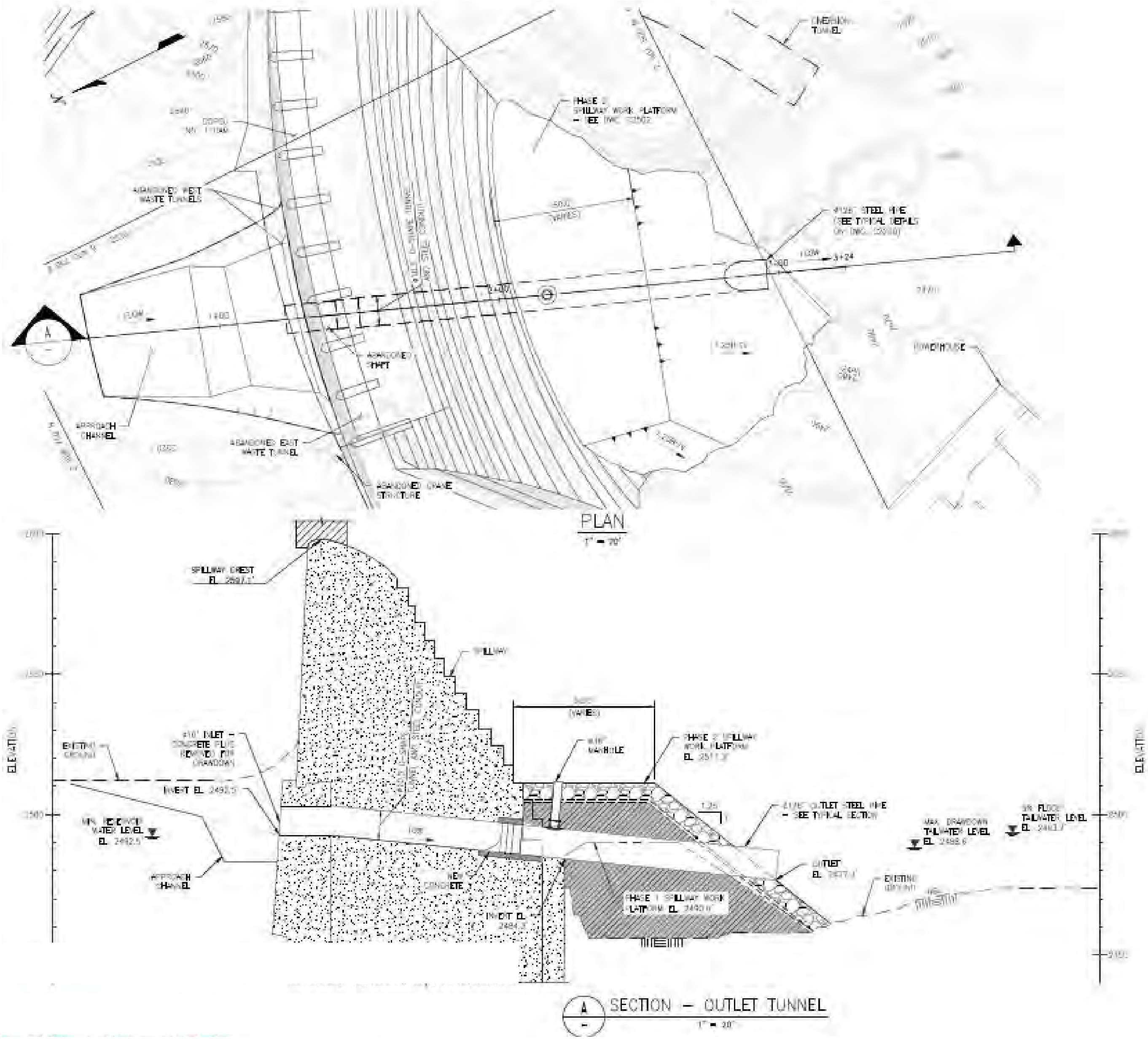

Figure 1. Plan view and section of proposed Copco No. 1 drawdown low-level outlet, KP Drawing C2205 (07/22/2020)

2 TAILWATER RATING CURVES

Figure 2 shows the 1,600-ft long reach of the Klamath River between the upstream side of Copco No. 2 Dam and the toe of Copco No. 1 Spillway, where a deep plunge pool is located. A River2D hydrodynamic model (Steffler and Blackburn, 2002) of this reach was applied to develop the Copco No. 1 tailwater rating curves for both pre-deconstruction and post-deconstruction of Copco No. 2 conditions.

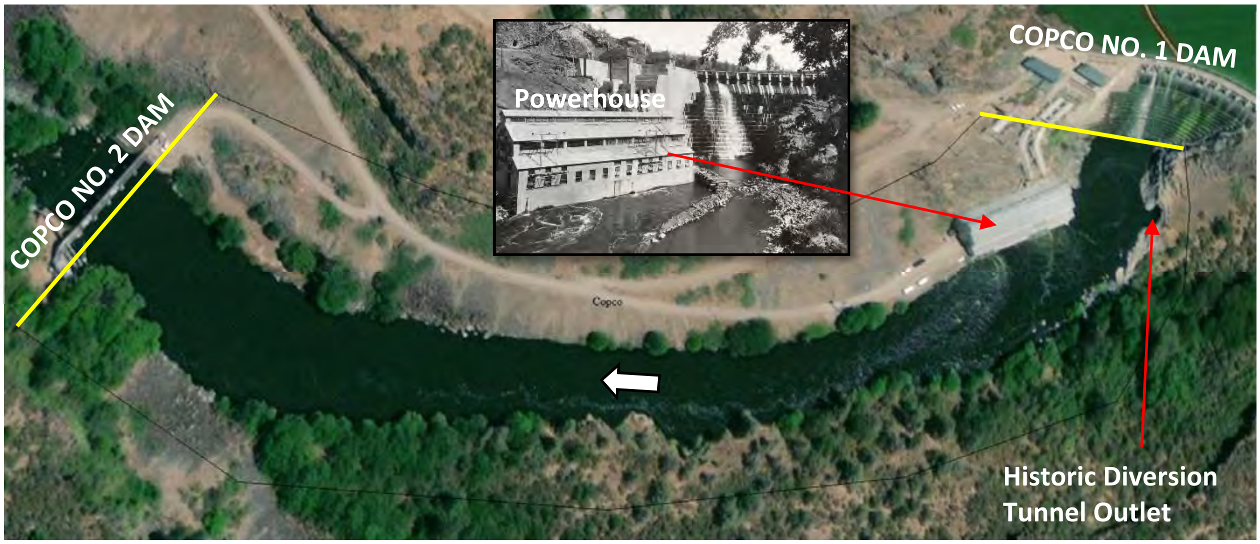

Figure 2. Extent of 2D hydrodynamic model between Copco No. 1 and Copco No. 2 Dams. Insert depicts historical photograph of Copco No. 1 powerhouse and cofferdam. Yellow lines show inflow and outflow sections of the model.

Pre-deconstruction conditions correspond to the existing river bathymetry with Copco No. 2 Dam operating and historic cofferdams in place (Figure 3). The pre-deconstruction bathymetry was based on a 3-ft resolution 2018 digital elevation model (DEM), (GMA, 2018).

For post-deconstruction conditions it is assumed that Copco No. 2 Dam is removed, and the Copco No. 2 historic cofferdam is lowered down to El. 2,472 ft, which is approximately the elevation of the surrounding riverbed (Figure 4).

The pre-deconstruction channel bathymetry in Figure 3 shows three bed features relevant for the hydraulics of the reach between the two dams: Copco No. 2 historic cofferdam, high point near Copco No. 1 Powerhouse and Copco No. 1 plunge pool:

- The V-shaped submerged cofferdam located approximately 70 ft upstream of Copco No. 2 Dam is a remnant of the original historic Copco No. 2 cofferdam built during dam construction. The crest of the cofferdam reaches up to approximately El. 2,481.6 ft and is currently fully submerged when Copco No. 2 Reservoir is at its normal operating water level at El. 2,486.5 ft.

- The historical photograph in Figure 2 shows what appears to be a cofferdam or dyke adjacent to the Copco No. 1 Powerhouse Tailrace. This is believed to be the high point in Figure 3; however, this bed feature could also be in part sediment eroded from the Copco No. 1 plunge pool located upstream.

- There is a roughly 20-ft deep pool located at the toe of Copco 1 Spillway (bottom ~ El. 2,454 ft). This pool might have been initially excavated, and also could have been scoured during large floods that spilled over Copco No. 1 Spillway. It seems plausible that some of that scoured sediment was deposited at the high point located downstream.

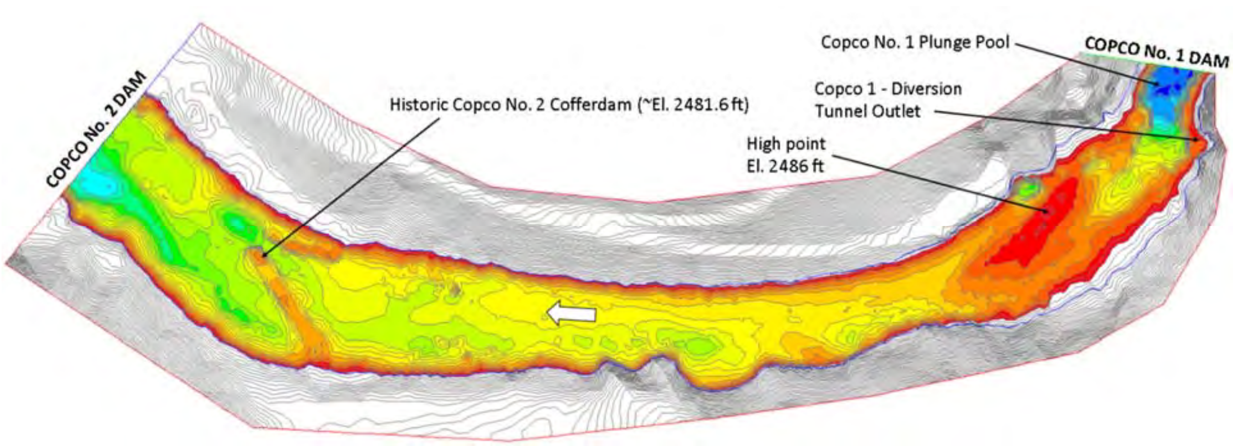

Figure 3. Existing pre-deconstruction bathymetry of Klamath River between Copco No. 1 and Copco No. 2 Dams (from *Topo_2018DEM_3ft.tif*)

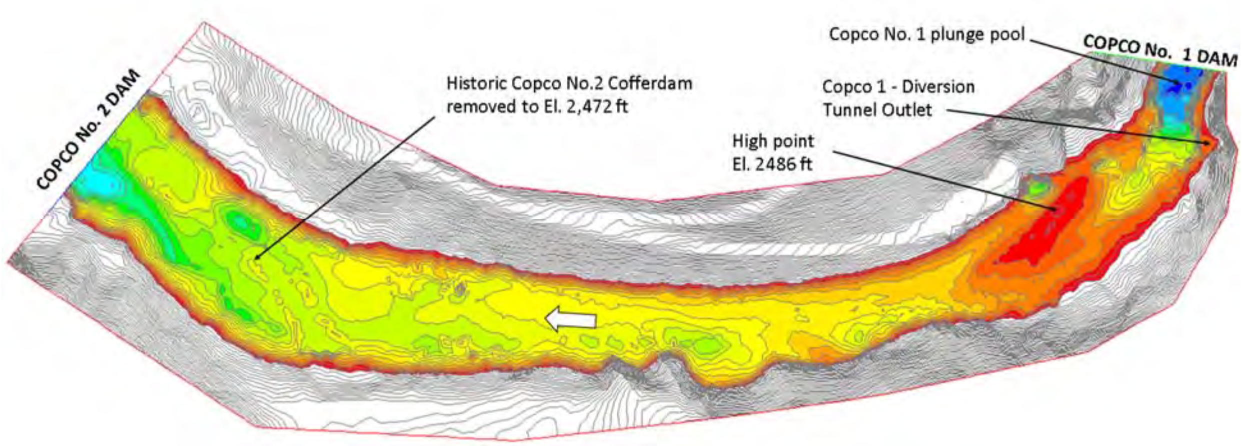

Figure 4. Modified post-deconstruction bathymetry of Klamath River between Copco No. 1 and Copco No. 2 Dams with historic Copco No. 2 cofferdam removed down to El. 2,472 ft.

Because water levels in the reach between the dams are controlled by the Copco No. 2 historic cofferdam and high point features mentioned above, water levels are not very sensitive to roughness. A preliminary sensitivity analysis for a discharge of 13,000 cfs showed that varying roughness height between 1 ft (Manning's n = 0.032) and 3 ft (n=0.040) only changed water levels by 0.5 ft at Copco No. 1 plunge pool. Roughness height in the River2D model was set to 3 ft, which approximates the roughness used in the 1D drawdown hydraulic model.

The estimated Copco No. 2 Spillway discharge capacity at the normal operating level of El. 2,486.5 ft is 13,500 cfs with the five spillway gates fully open. A CFD model of Copco No. 2 Dam (NHC, 2020) found that for the 100-year flow of 29,400 cfs, water levels at the dam reached El. 2,494.8 ft with the five spillway gates fully open. Based on this CFD result, it was estimated that for a discharge of 23,400 cfs, water level at Copco No. 2 Dam should be approximately at El. 2,490.3 ft. This information was used to set the water levels at Copco No. 2 Dam for the pre-deconstruction River2D model. For inflow discharges of 13,000 cfs and lower, water levels downstream at Copco No. 2 Dam were maintained constant at El. 2,486.5 ft. Figure 5 shows the pre-construction longitudinal water surface profiles computed by the 2D model along the Klamath River between Copco No. 1 and Copco No. 2 Dams.

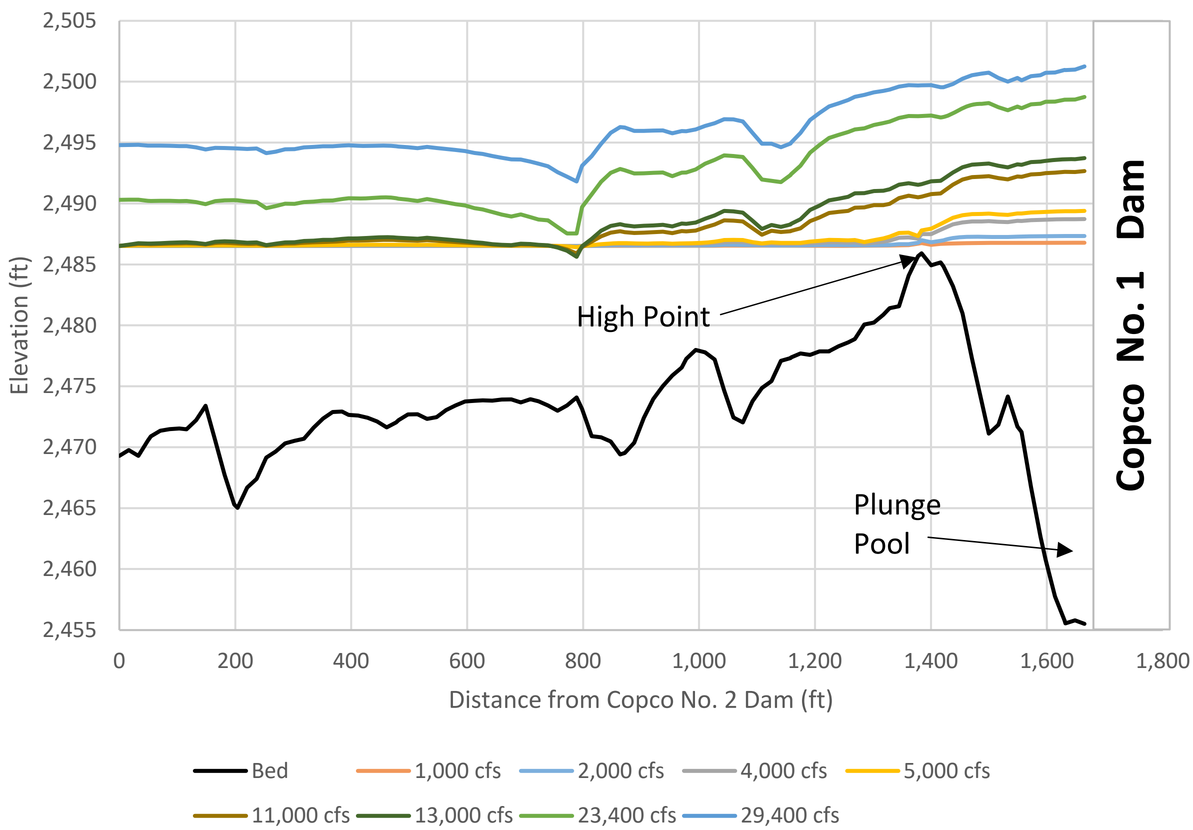

Figure 5. Pre-deconstruction water surface profile along Klamath River between Copco No. 1 and Copco No. 2 Dams

For the post-deconstruction River2D model with Copco No. 2 Dam removed, downstream water levels at the outflow section were estimated by the 1D hydrodynamic model. Figure 6 shows the post-deconstruction longitudinal water surface profiles computed by the 2D model along the Klamath River between Copco No. 1 and Copco No. 2 Dams.

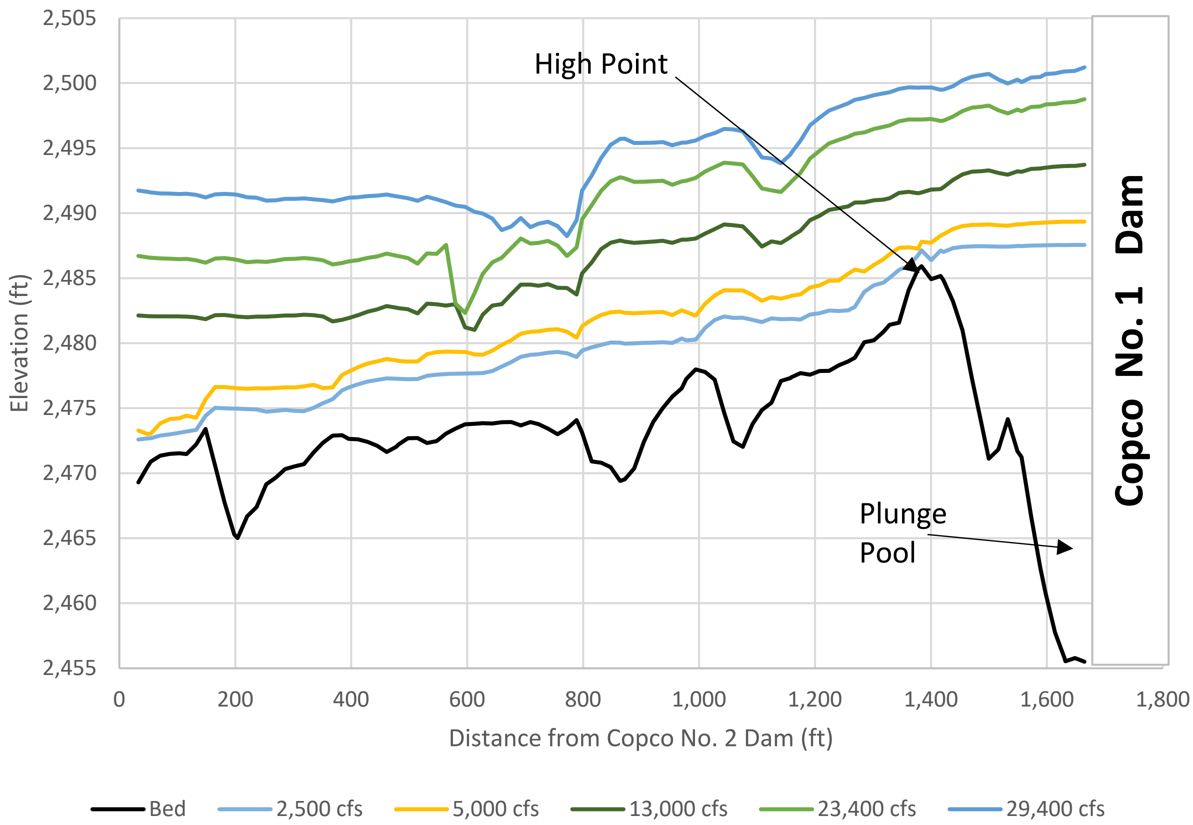

Figure 6. Post-deconstruction water surface profile along Klamath River between Copco No. 1 and Copco No. 2 Dams

For the pre-deconstruction conditions, Copco No. 2 Dam does not control water levels at the toe of Copco No. 1 Dam; but instead the water levels are controlled by the high point (Figure 5). This remains true even after Copco No. 2 Dam is removed for the post-deconstruction conditions (Figure 6). The effect of the high point on the tailwater rating curve at Copco No. 1 Dam is evident in Figure 7, which shows that the pre- and post-deconstruction rating curves are practically identical; i.e. the removal of Copco No. 2 Dam has no discernible effect on tailwater levels at Copco No. 1 Dam. Table 1 shows the rating curve in tabular format. This rating curve was used to set the downstream boundary condition in the CFD model described next.

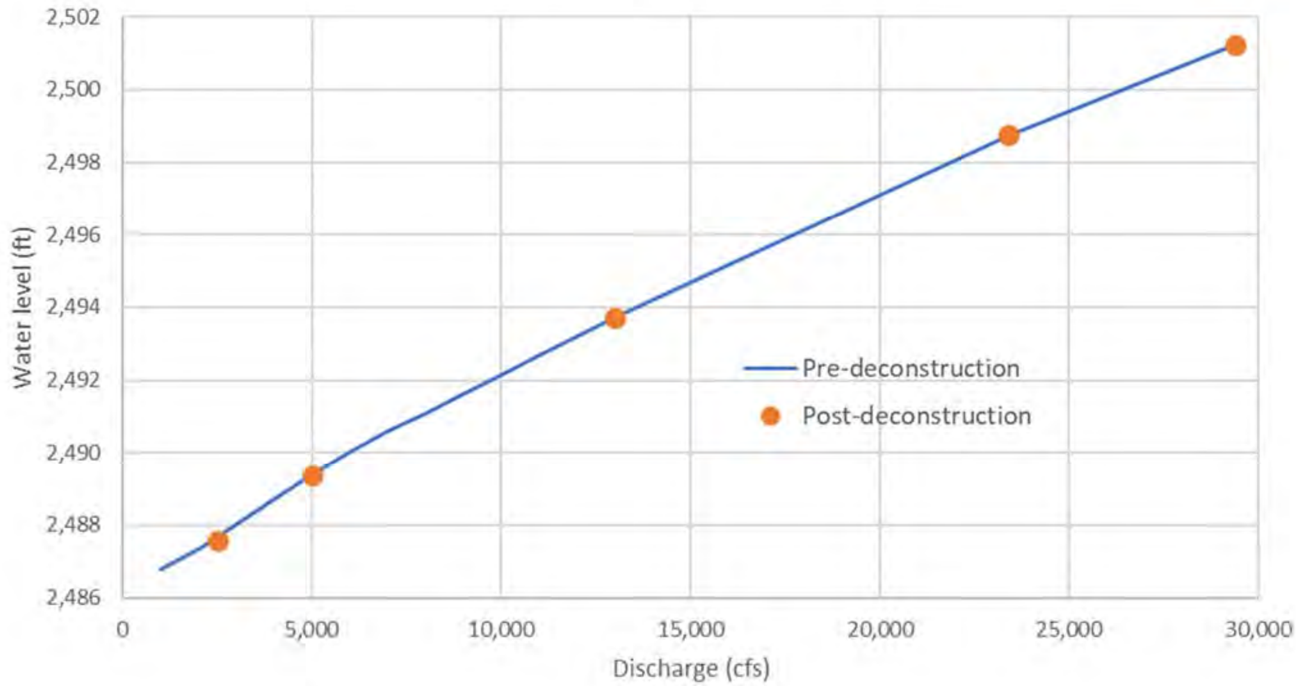

Figure 7. Copco No. 1 Dam tailwater rating curve

Table 1. Tailwater rating curve at Copco No. 1 Dam

Discharge (cfs)	Water Level (ft)
1,000	2,486.8
2,000	2,487.3
3,000	2,488.0
4,000	2,488.7
5,000	2,489.4
6,000	2,490.0
7,000	2,490.6
8,000	2,491.1
9,000	2,491.6
10,000	2,492.1
11,000	2,492.7
12,000	2,493.2
13,000	2,493.7
23,400	2,498.8
29,400	2,501.3

3 LOW-LEVEL OUTLET

3.1 CFD Model development

In addition to the bathymetry (GMA, 2018), the other information required to generate the 3D geometry for the CFD model was provided by KP on June 16, 2020 as:

- One AutoCAD drawing:
  - CP1-OUTL-TUNL-3DBJ.dwg;
- Two XML Files:
  - CP1-APRH-CHAN-UPST-EXCV_2020.06.15.xml; and,
  - CP1-WPAD-STEL-LINR-PHAS-02 2020.06.15.xml.

During construction of the Copco No. 1 Dam, a cofferdam was built upstream to divert water towards the Historic Diversion Tunnel. The cofferdam was left in place and is presently submerged in the reservoir. Figure 8 shows the cofferdam, which has a saddle point at approximately El. 2,515 ft, or 22.5 ft above the LLO's inlet invert elevation. When reservoir water levels become low enough (~El. 2,525 ft), the cofferdam would start acting as a weir controlling flows.

The CFD software selected for the project was FLOW-3D, developed by Flow Science Inc. FLOW-3D has an excellent track record for modeling complex free surface flows, as those found in the present application. NHC has considerable experience using FLOW-3D and has verified it in several instances using experimental data. FLOW-3D numerical settings included the effects of gravity and turbulence, the latter using the RNG k-epsilon turbulence model. The single-phase Volume-of-Fluid (VOF) method was used to track the free surface, which neglects air dynamics and assumes atmospheric pressure at the water surface, implying the model assumes flow is adequately vented. Roughness height was adopted as 2.5 inches based on information provided by KP, although this is believed to be of secondary importance because flow is controlled at the inlet.

The geometry of the CFD model (Figure 8) encompasses a region approximately 850 ft long, 520 ft wide and 180 ft high, which includes the entire LLO, dam and work platform; a 150-ft long reach downstream of the dam; plus a 550-ft long reach of the upstream reservoir (Figure 9). The Historic Diversion Tunnel is not included in the model.

The CFD mesh size was 8 ft within the reservoir away from the dam, 4 ft in the reservoir near the dam, 1 ft in the downstream reach and 0.5 ft within the LLO. Since flow capacity is controlled at the inlet, the mesh was further refined to 0.125 ft (1.5 inches) around the inlet (orifice) to the LLO, providing 80 cells to resolve details of the geometry and flow, which is considered adequate for developing the rating curve.

The downstream outflow boundary condition was set using the tailwater levels downstream of Copco No. 1 Dam shown in Figure 7. The upstream inflow boundary condition was the reservoir water surface (RWS) elevation. Six RWS elevations upstream of the historic cofferdam were used to develop the rating curve: El. 2,597.1 ft, El. 2,565 ft, El. 2,535 ft, El. 2,530 ft, El. 2,525 ft, and El. 2,520 ft.

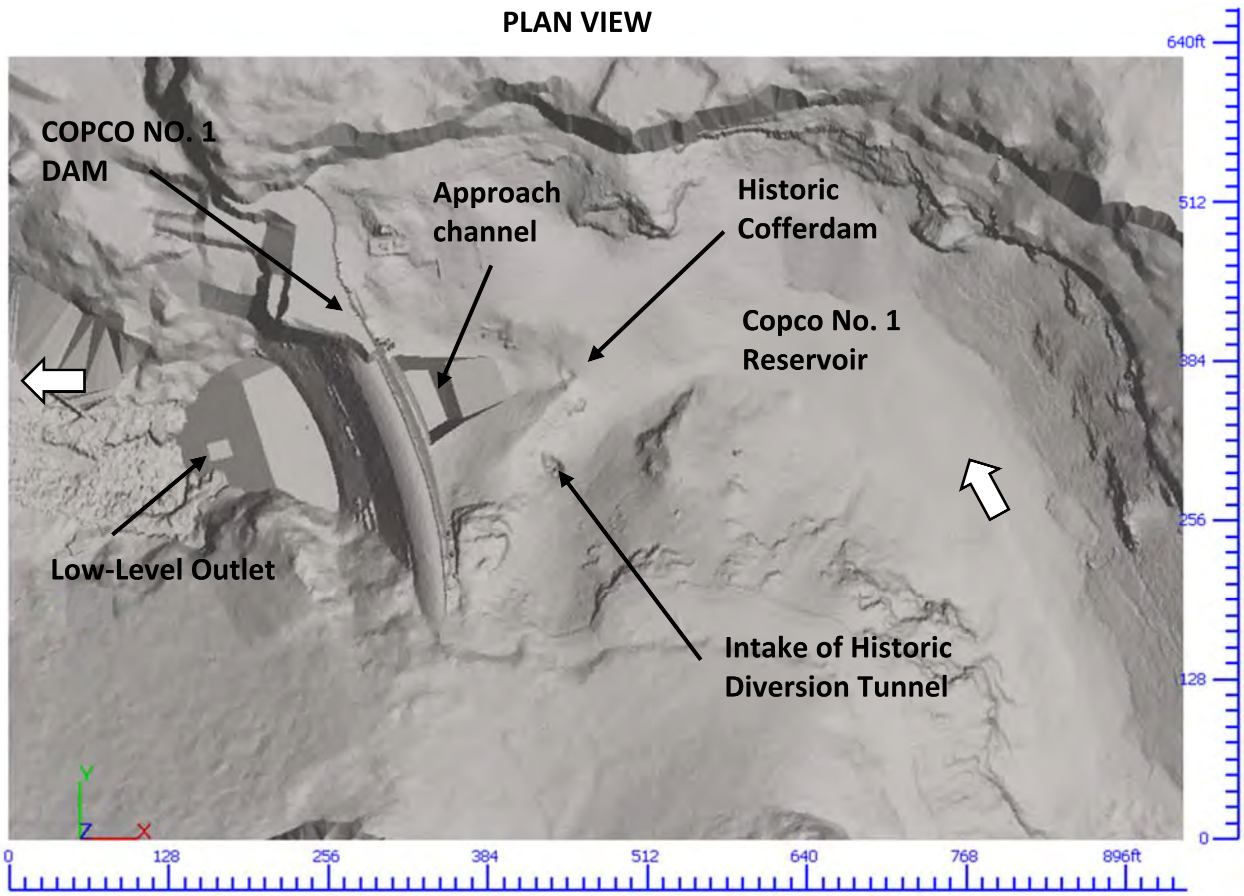

Figure 8. Geometry of 100% Design Copco No. 1 Dam CFD model

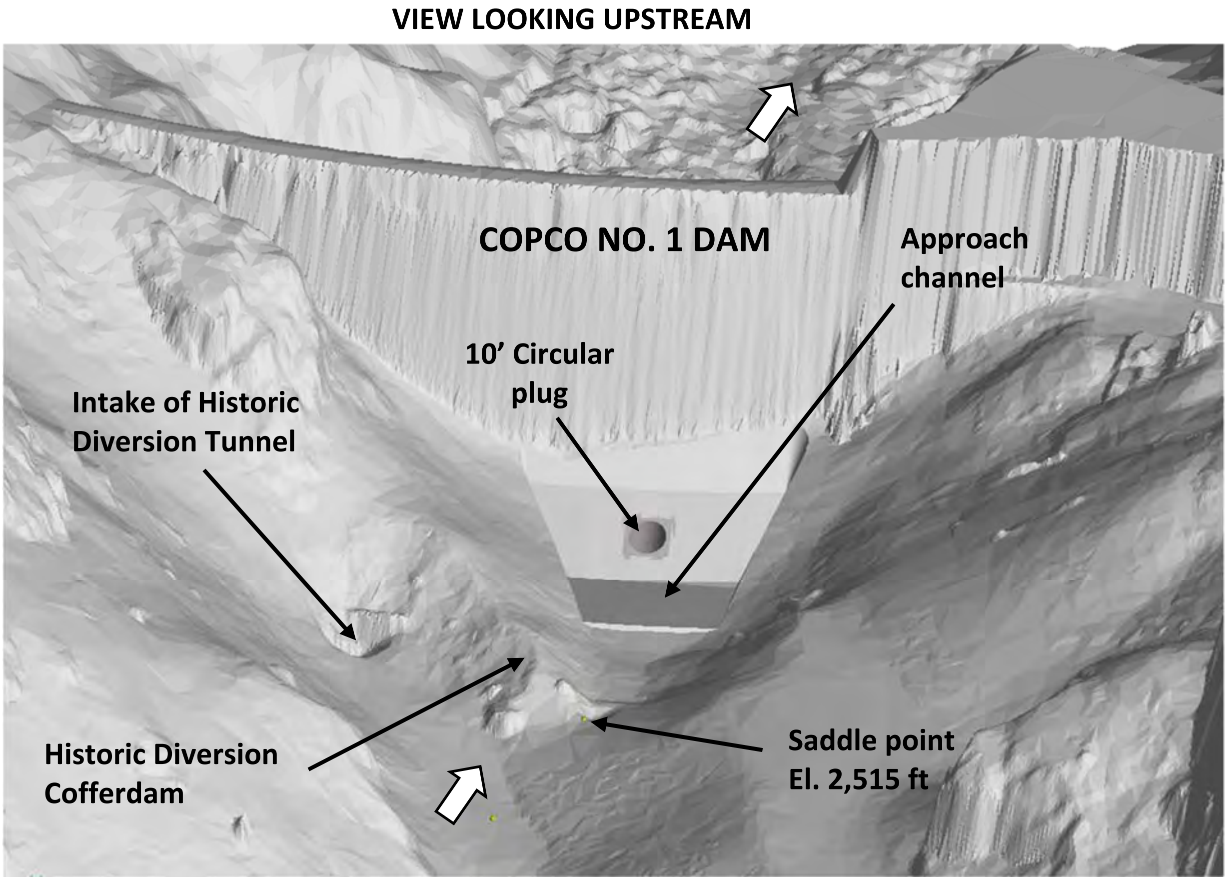

Figure 9. Historic Diversion Cofferdam located upstream of Copco No. 1 Dam (image looking downstream)

3.2 RESULTS

Because the LLO alignment is not perpendicular to the dam's upstream face, flow is skewed upon entering the LLO and immediately detaches downstream of the LLO inlet section, as illustrated in Figure 10 for the highest RWS 2,597.1 ft. NHC noted that the flow entering the LLO is a transition zone and that FLOW-3D is not modeling air entrainment and bulking effects. The flow conditions as presented are conservative in terms of predicted discharges in the LLO. If the flow entering the LLO is not adequately vented, then the discharges entering the LLO may be higher.

Figure 11 shows the velocity field along a vertical plane cutting through the centerline of the LLO for five of the simulated flow conditions. The hydraulic jump near the outlet could move farther upstream depending on the amount of air entrainment and air bulking.

Figure 12 shows three-dimensional inclined views of the LLO. Figure 13 and Table 2 show the LLO rating curve. Under all flow conditions, flow detaches at the LLO inlet and remains as open-channel flow (not pressurized). Below RWS El. 2,535 ft a vortex starts developing between the historic cofferdam and the LLO.

For RWS around El. 2,525 ft and higher, the LLO flow is inlet controlled and the rating curve can be approximated by an orifice equation with discharge coefficients between 0.62 and 0.63. For RWS below El. 2,525 ft, the cofferdam starts acting as a weir controlling flows to RWS El. 2,515 ft, which is the saddle point of the cofferdam. When water levels drop below the cofferdam's saddle point, water cannot flow towards the dam anymore and can only flow towards the Historic Diversion Tunnel.

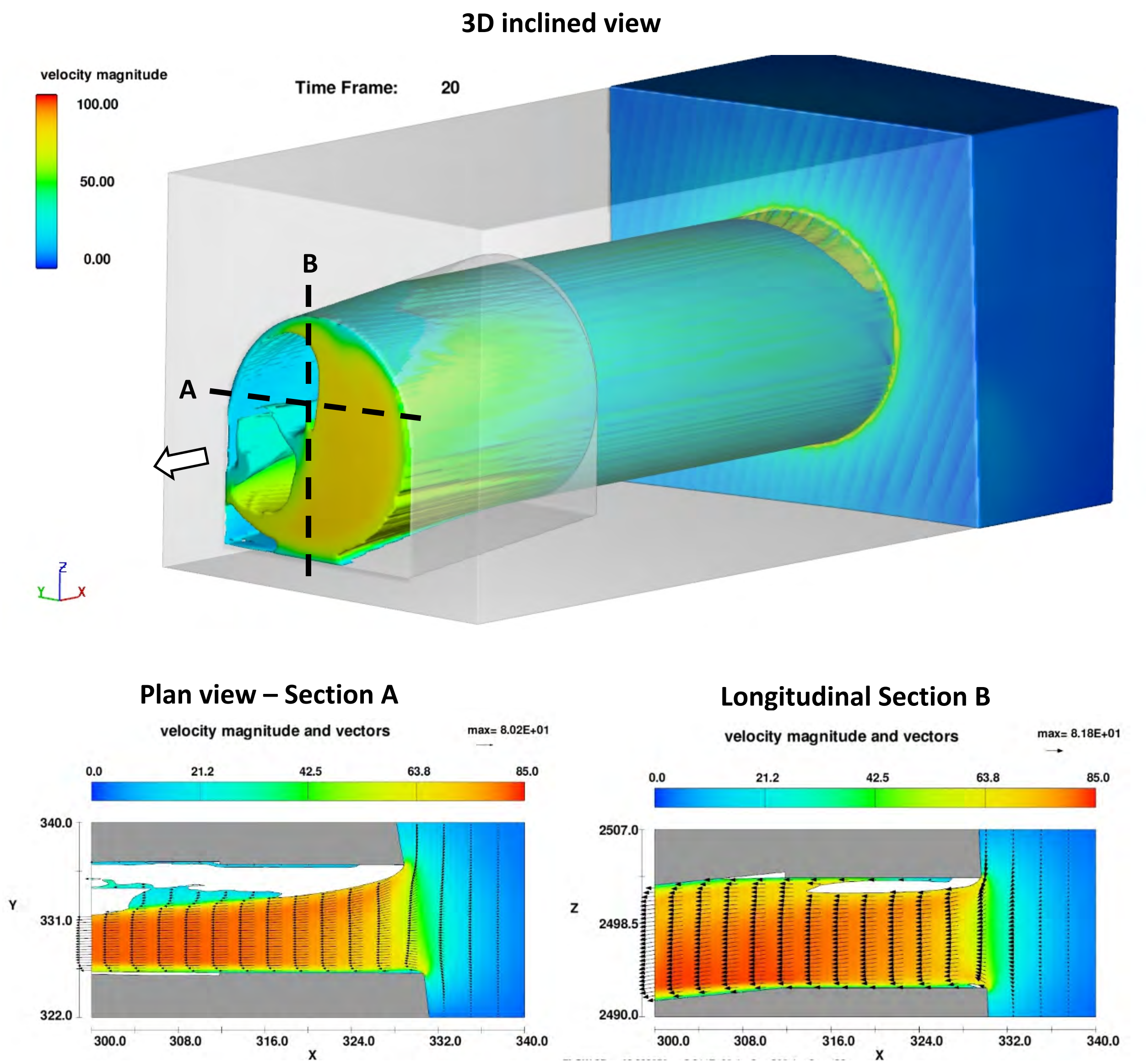

Figure 10. Close view of flow in the LLO inlet for RWS 2,597.1 ft. 3D inclined view and two sections cutting through the center of LLO's inlet orifice

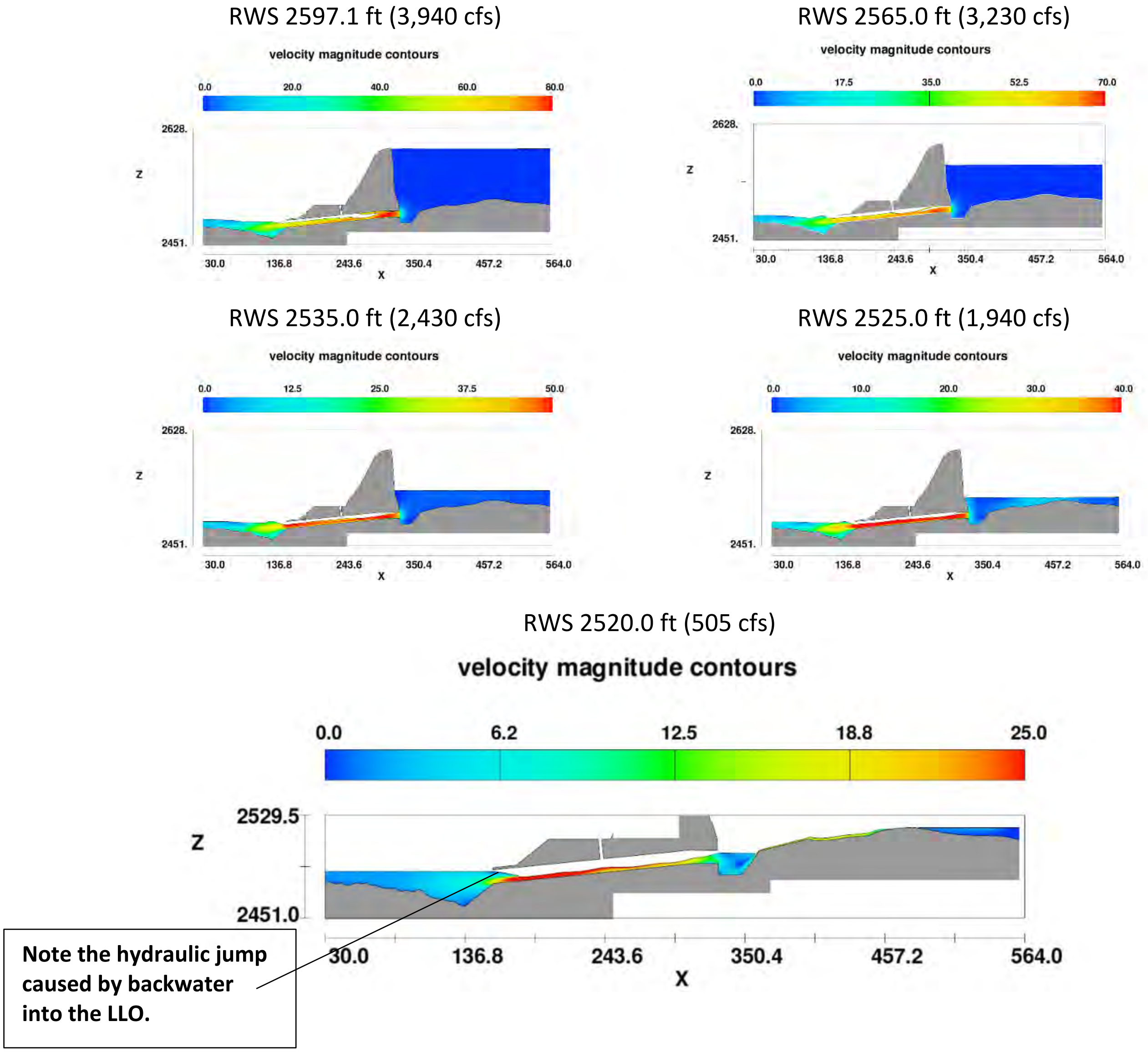

Figure 11. Longitudinal profiles along Copco No. 1 Low-Level Outlet for various reservoir water surface (RWS) elevations. Velocities presented in ft/s

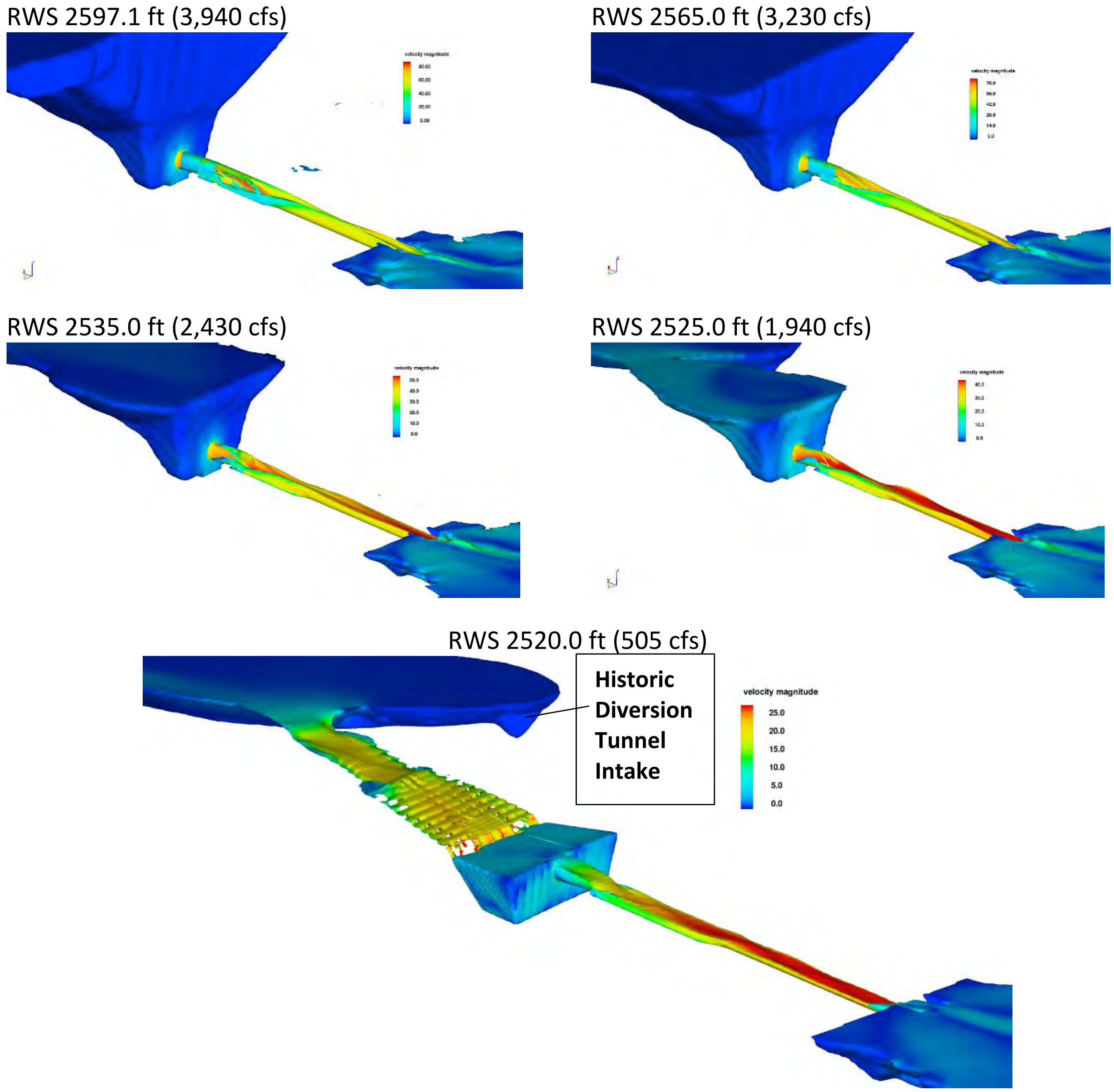

Figure 12. Three-dimensional views of Copco No. 1 Low-Level Outlet for various reservoir water surface (RWS) elevations.

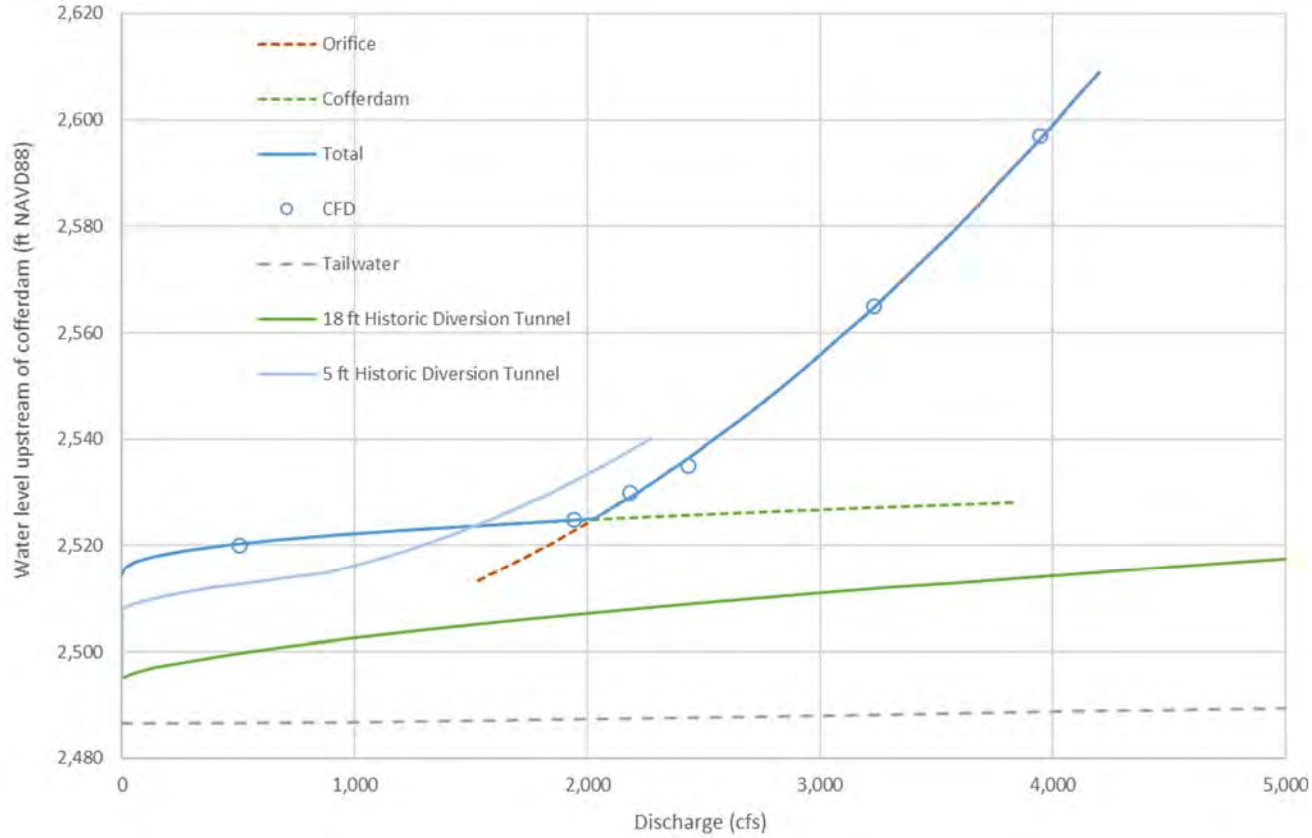

Figure 13. Rating curve of Copco No. 1 Low-Level Outlet (showing tailwater rating curve and Historic Diversion Tunnel Rating Curves for reference).

Table 2. Rating curve of Copco No. 1 Low-Level Outlet

Water level (ft)	Discharge (cfs)
2,609	4,197
2,604	4,099
2,599	3,998
2,594	3,894
2,589	3,789
2,584	3,680
2,579	3,568
2,574	3,453
2,569	3,334
2,564	3,211
2,559	3,084
2,554	2,951
2,549	2,813
2,544	2,667
2,540	2,546
2,539	2,515
2,538	2,483
2,537	2,451
2,536	2,419
2,535	2,386
2,534	2,353
2,533	2,319
2,532	2,285
2,531	2,250
2,530	2,215
2,529	2,179
2,528	2,143
2,527	2,106
2,526	2,069
2,525	2,031
2,524	1,653
2,523	1,264
2,522	934
2,521	659
2,520	500
2,519	268
2,518	144
2,517	63
2,516	18
2,515	0

4 SEDIMENT MOBILITY

The Historic Diversion Tunnel is expected to start operating when water levels in the Copco No. 1 Reservoir drop to El. 2,530 ft (Knight Piesold, 2020). The sediment mobility upstream of Copco No. 1 Dam at this reservoir water level was assessed for two conditions, assuming the LLO operates alone and both the LLO and Historic Diversion Tunnel operate simultaneously.

Sediment mobility was assessed by comparing the bed shear stress of the flow predicted by FLOW-3D with the critical bed shear stress shown in Table 3. The critical shear stress values presented in Table 3 approximately represent the minimum shear force applied by the flow per unit area of the bed, needed to initiate the motion of a sediment particle of a given size, which is surrounded by particles of the same size resting on a flat bed. In order to improve the estimates of bed shear stress, the mesh in the reservoir was refined to 2 ft.

Table 3. Critical shear stress for initiation of sediment motion on flat bed. Adapted from Julien (2002).

Sediment class name	Particle size		Critical shear stress	
	(mm)	(ft)	(Pa)	(lbf/ft^2)
Very large boulder	2048	6.7	1790	37.4
Large boulder	1024	3.4	895	18.7
Medium boulder	512	1.7	447	9.3
Small boulder	256	0.8	223	4.7
Large cobble	128	0.4	111	2.3
Small cobble	64	0.2	53	1.1
Very coarse gravel	32	0.1	26	0.54
Coarse gravel	16	0.05	12	0.25
Medium gravel	8	0.03	5.7	0.12
Fine gravel	4	0.01	2.71	0.057
Very fine gravel	2	0.01	1.26	0.026
Very coarse sand	1	0.003	0.47	0.010
Coarse sand	0.5	0.002	0.27	0.006
Medium sand	0.25	0.001	0.194	0.004
Fine sand	0.125	0.0004	0.145	0.003
Very fine sand	0.063	0.0002	0.110	0.002
Coarse silt	0.031	0.0001	0.083	0.002

4.1 LLO only

Figure 14 shows the bed stress computed in the reservoir upstream of Copco No. 1 Dam. The red color shading indicates areas where shear stress equals or exceeds 0.01 lb/ft^2, high enough to entrain all sand sizes according to Table 3.

With a reservoir level at El. 2,530 ft, flow velocity and bed shear stresses in the upstream reservoir will be high enough to mobilize all sand sizes up to roughly 230 ft upstream of the LLO. Fine sand and silt should be mobilized even farther upstream of the LLO.

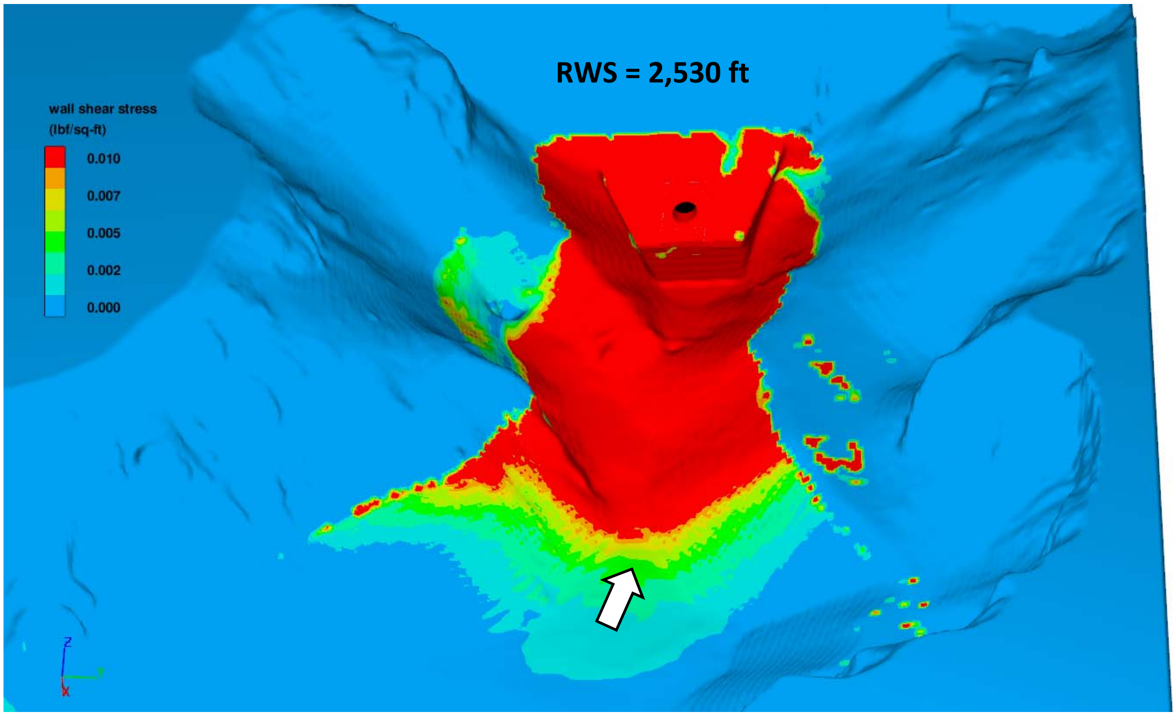

Figure 14. Bed shear stress in Copco No. 1 reservoir for RWS 2,530 ft with the LLO only (red color shading indicates areas where sand could be mobile)

4.2 LLO and Historic Diversion Tunnel

Knight Piésold (2020) developed rating curves for the Historic Diversion Tunnel Copco No. 1 up to RWS 2,530 ft. The rock tunnel is currently hydraulically sealed by an intake structure and tunnel plug (Figure 9). The intake structure and tunnel plug will be removed. The initial opening will be small and once the reservoir level is lowered to El. 2516 ft., the remaining of the intake structure will be removed to the full 18 ft.

An additional simulation was conducted for RWS 2,530 ft assuming both the LLO and 5-ft open Historic Diversion Tunnel were operating. Since the geometry of the Historic Diversion Tunnel is not explicitly included in the CFD model, it was modelled as a 16-ft wide by 5-ft high sink that extracts 1,845 cfs, based on the Knight Piésold (2020) rating curve.

Figure 15 shows the depth-averaged velocity in Copco No. 1 Reservoir under these simulated conditions. With the exception of the flow velocity over the historic cofferdam and right next to the inlets to both the LLO and Historic Diversion Tunnel, flow velocity remains below 4 ft/s.

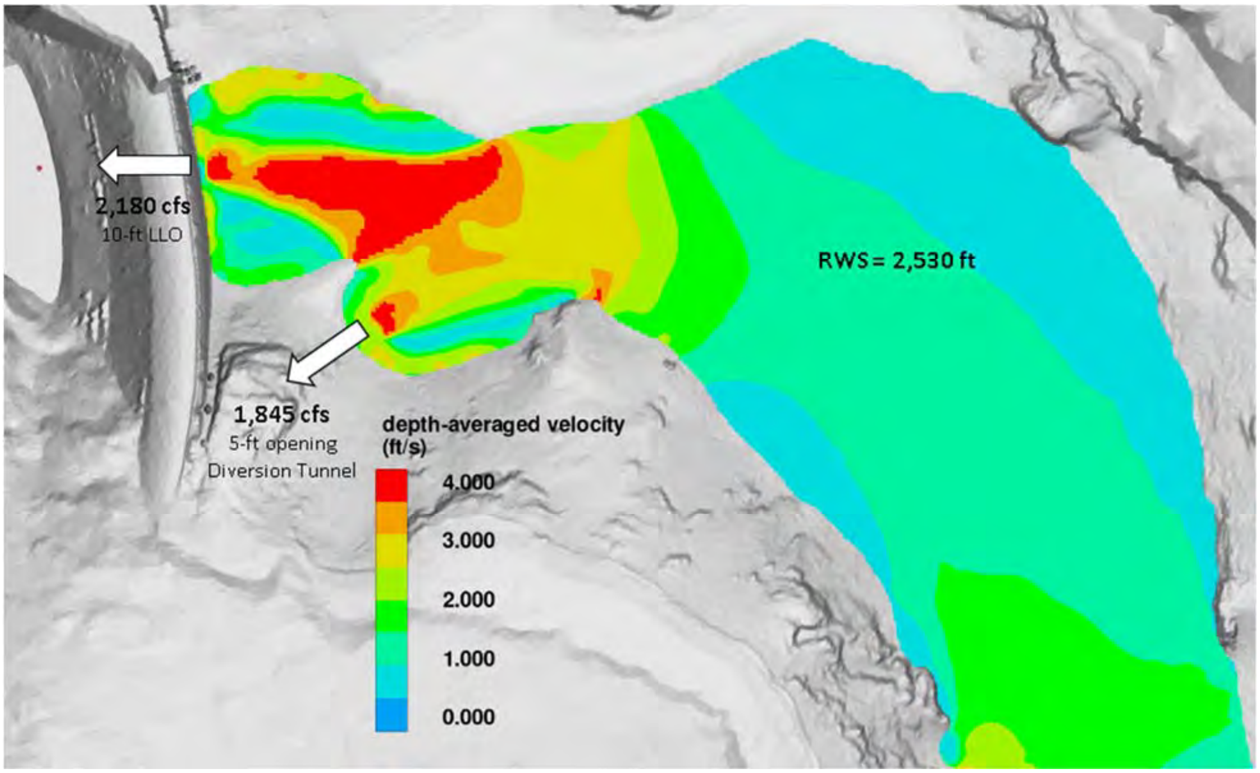

Figure 15. Depth-averaged velocity in Copco No. 1 reservoir for RWS 2,530 ft with the LLO and 5-ft opening Historic Diversion Tunnel operating.

Because sediment sizes between sand and boulders vary over several orders of magnitude (Table 3), it is difficult to find good color scale for bed shear stress valid for all these sizes. Figure 16 shows the bed stress computed in the reservoir upstream of Copco No. 1 in four plots where the maximum stress was truncated at values of 0.010, 0.025, 0.050 and 1.0 lbf/ft^2 so that the mobility of various sizes is more discernible. Red color in each plot represents the maximum value in the scale and provides an indication of the area where a given size (e.g. very coarse sand (VCS), fine gravel (FG), etc.), could be mobile. Table 4 shows the distance from Copco No.1 Dam up to where a given sediment size could be mobile.

It can be concluded that when the LLO and 5-ft opening Historic Diversion Tunnel operate, sand of all sizes should be mobile in the upstream reservoir (bed shear stress = 0.01 lbf/ft^2); while boulders would not be mobile because velocity in the reservoir remains low (< 4 ft/s).

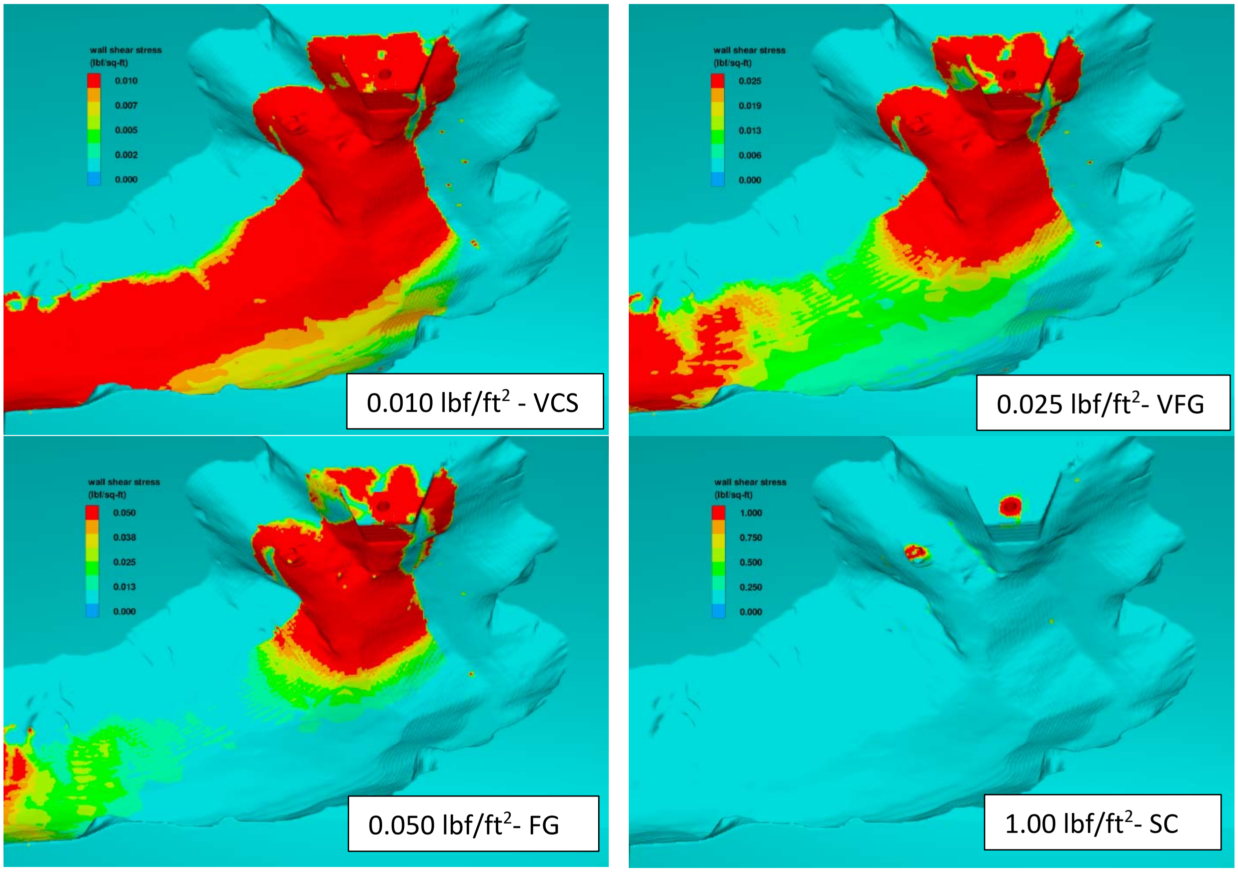

Figure 16. Bed shear stress in Copco No. 1 reservoir for RWS 2,530 ft with the LLO and 5-ft opening Historic Diversion Tunnel operating. Maximum bed shear stress in each plot was truncated at the values shown in text box to better visualize mobility of given size (see Table 4).

Table 4. Distance from Copco No. 1 Dam where given sediment class is mobile when LLO and 5-ft opening Historic Diversion Tunnel operate.

Sediment class name	Critical shear stress (lbf/ft^2)	Particle size (ft)	Distance from dam (ft)
Medium boulder	9.3	1.7	-
Small boulder	4.7	0.8	-
Small cobble (SC)	1.1	0.2	-
Medium gravel (MG)	0.12	0.03	160
Fine gravel (FG)	0.057	0.013	230
Very fine gravel (VFG)	0.026	0.007	280
Very coarse sand (VCS)	0.010	0.003	>500

5 REFERENCES

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Steffler, P. and Blackburn, J. (2002). "River2D. Two-Dimensional Depth Averaged Model of River Hydrodynamics and Fish Habitat". University of Alberta.

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APPENDIX C3
COPCO NO. 1 - STABILITY EVALUATION

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1.0 SCOPE

The stability analysis of the Copco 1 concrete dam has been undertaken to evaluate the safety of the existing dam with the modifications proposed comprising of the proposed low-level outlet within the dam center section (the outlet tunnel). The objective of the analysis is to evaluate if the current stability analyses require to be revised and if the dam modifications result in an unacceptable structural response and risk to the operation of the facility. The analysis is focused on the potential failure modes (PFM) related to the main dam section, where the dam modifications could have deleterious effects to the overall stability or structural response of the dam.

1.1 PREVIOUS STABILITY ANALYSES

The latest supporting documents related to the stability evaluation of the dam are summarized as follows:

- Kleinfelder (2009) performed a site-specific seismotectonic study. The study recommended that the operating basis earthquake (OBE) at the dam site be increased to 0.16 g and the maximum credible earthquake (MCE) be increased to 0.26 g.
- Black & Veatch (1996 & 2011) performed a static and dynamic stability analysis of the dam in 1996. In 2011, the 1996 dynamic stability analyses were reviewed and scaled to represent the seismic loading recommended by Kleinfelder (2009). The study concluded that the dam had excess structural capacity, adequate factors of safety, but made recommendations to evaluate the intake block section and the spillway piers with loading resulting from the MCE.
- URS (2013) performed a static and dynamic stability analysis of the intake block section and spillway piers. The study concluded that the intake block section and spillway piers have adequate structural capacity and adequate factors of safety.
- PacifiCorp (2015) revised the potential failure mode analysis (PFMA) originally developed by Black & Veatch in 2004. 15 potential failure modes (PFM) are identified and categorized in accordance with FERC guidelines. Twenty-three PFM are characterized and described.

2.0 INFORMATION AND ASSUMPTIONS

The stability analysis was undertaken using the information documented in the previous stability analyses and engineering assessments completed for PacifiCorp and summarized in Section 1.1 and submitted to FERC as part of the licensing requirements.

The following assumptions for this stability analysis are made:

- A 3-dimensional (3D) finite element model (FEM) has been developed based on the historical drawings, supplemented using ground levels from the latest LiDAR terrain data received.

- The dam axis is modelled on a single curvature with upstream radius of 493.4 ft (B&V, 2011). The terrain data indicates that the upstream radius is not uniform and may be nominally larger than 500 ft. The difference is small considering the large upstream radius and this should not significantly impact the global results obtained.
- There has been a change in survey datum, and the ground levels documented in the previous stability reports. The datum used by B&V and URS is the National Geodetic Vertical Datum of 1929 (NGVD29), elevations adjustment for the North American Vertical Datum of 1988 (NAVD88), used for this analysis is adjusted by adding 3.5 ft to the NGVD29 elevations.
- The spillway piers above the spillway ogee crest have not been modelled.
- The stability of an arch relies on effective arch thrust transfer into the abutment rock mass. No definitive foundation/excavation footprint plan for the dam is available. However, photographs of both abutments taken during construction reveal that rock foundation preparation was undertaken. The foundation was shaped to provide a rough, interlocking footing to key the dam to the foundation. In this regard, for the analyses completed herein, a radial arch/abutment contact has been assumed.
- Material properties for the concrete and rock mass are based on the values documented in the previous stability assessments.
- Pseudo dynamic seismic stability evaluations have been undertaken in the previous stability assessments, which have shown that the concrete gravity-arch dam performs exceptionally well under seismic load.

2.1 DAM CLASSIFICATION

The Copco No. 1 Dam is located on the Klamath River at river mile 202.2 and has the following State of California Division of Safety of Dams (DSOD) identification:

- DSOD Jurisdictional Dam Number: 91.000 / U.S. Army Corp of Engineers' National Inventory of Dams (NID) number: CA00323
- County: Siskiyou County
- Certified Status: Certified
- Downstream Hazard: high
- Condition Assessment: Satisfactory

3.0 POTENTIAL FAILURE MODE ANALYSIS

The PFM's identified in the Potential Failure Modes Analysis (PFMA) Report (PacifiCorp, 2015) has been used to guide the previous stability evaluations. KP has identified the following potential failure modes (PFM) directly related to the stability and safety of the main dam section:

- PFM 1A: Arch Dam Structural Failure during Flood; Category IV
- PFM 6A: Arch Dam Failure; Category IV
- PFM 10A: Arch Dam Failure during Seismic Loading; Category II

The stability evaluation and stress analysis of the existing structure with the proposed low-level outlet tunnel will evaluate the loads and loading conditions as defined for the potential failure modes identified and developed below. The potential failure modes related to the dam modification are intended to supplement the PFMA (PacifiCorp, 2015) are summarized in Table 3.1 and Table 3.2.

Table 3.1 Dam Center Section Failure due to Construction of Proposed Low-Level Outlet

PFM Description	During operation of the existing dam under normal reservoir operating level, construction surcharges in the form of vibration from the drilling and blasting causes longitudinal cracking at the concrete plug. Under loading the cracked sections are destabilized causing failure of the arch cantilever between spillway bay 2 to 5. Uncontrollable reservoir releases will occur.
Adverse Factors	Unknown dam construction, reinforcement, and construction joint location.
Positive Factors	Arch stresses developed under existing conditions are low. The tunnel will be excavated from the downstream face, blasting performance can be critically monitored, evaluated and the methodology improved as necessary for each successive round length during the advance. Should excessively cracking to the surrounding periphery develop as a result of the blasting and mucking, crack injection and related repairs can be implemented safely as part of the ongoing construction activity. Operation of the reservoir at a lowered operating level will reduce hydrostatic loading applied.
Risk Reduction Actions	Survey of upstream dam face location by drilling and grouting exploratory holes. Establishing peak particle velocity (PPV) criteria and monitoring program. Undertake controlled blasts using smooth blasting techniques.
FERC PFMA Classification	Category I.

Table 3.2 Dam Center Section Failure due to Construction of Proposed Low-Level Outlet Tunnel during Seismic Loading

PFM Description	During operation of the existing dam under normal reservoir operating level and OBE seismic event, failure of the tunnel concrete plug could lead to uncontrollable reservoir releases.
Adverse Factors	Unknown dam construction, reinforcement, and construction joint location.
Positive Factors	Arch stresses developed under existing conditions are low. The tunnel will be excavated from the downstream face, blasting performance can be critically monitored, evaluated and the methodology improved as necessary for each successive round length during the advance. Should excessive cracking to the surrounding periphery develop as a result of the blasting and mucking, crack injection and related repairs can be implemented safely as part of the ongoing construction activity. Operation of the reservoir at a lowered operating level will reduce hydrostatic loading applied.
Risk Reduction Actions	Survey of upstream dam face location by drilling and grouting exploratory holes.
FERC PFMA Classification	Category I.

3.1 DAM MODIFICATION TIMELINE

The dam modifications are intended to be constructed while the facility is operational. Tunnel construction work is anticipated to occur in the summer and the removal of the concrete plug (by lake tap method) will occur in the winter period of the reservoir drawdown year. The reservoir drawdown will be completed in the spring to early summer of the drawdown year with no future reservoir impoundment during the removal of the dam.

3.2 DAM EMPTY CONDITION

The intent of the dam safety evaluations is to analyze the overall stability of the structure at the serviceability limit state (SLS). The SLS conditions are those considered to cause failure or structural / mechanical damage that may impact public safety or result in unintended releases of water during operation under specific critical governing loads (operating water level, flood, earthquake, etc.). During the reservoir drawdown and dam demolition the serviceability of the dam is not threatened as the loading acting on the dam is reduced considerably. Load reduction measures include:

- Reservoir lowering – reduction of the lateral load contributing to the axial forces until all impounded water is diverted through the dam and the diversion tunnel.
- Dam demolition – reduction of the vertical load (mass), geometry of the structure limits the construction loads to be applied within the effective area of the structure's foundation.

The construction loads are small compared to the removed lateral loads and vertical loads; therefore, this stability condition has not been considered.

4.0 EVALUATION APPROACH

The stability analyses are based on the 3D FEM using 10-node solid tetrahedral elements. Two models were compiled, one for the as-built case and the other for the case including the proposed low-level outlet tunnel for comparative purposes. The stresses obtained from each model are evaluated against target criteria, followed by an interpretation of the computed structural behavior. In addition, sliding stability is evaluated by calculating the indicative factor of safety against sliding on the dam foundation.

All structural evaluations were undertaken using SolidWorks Simulation Finite Element Modelling package using linear elastic analyses.

5.0 MATERIAL CHARACTERISTICS

5.1 FOUNDATION ROCK MASS

The foundation material properties applied in the model are based on the previous structural evaluations (B&V 1996 and 2011) and URS (2013), which provide a detailed discussion on the derivation thereof. Only a short summary will be provided here.

The compressive strength of the andesite rock mass foundation is expected to exceed 10,000 psi and does not appear to have any planes of weakness in the abutments. Shearing through intact rock or concrete would be required for the structure to slide in any direction. All previous evaluations assumed that the rock and concrete have the same deformation modulus.

The URS report also contains indicative cohesion and friction angles for the rock mass that were used for sliding evaluations of the intake works.

The applied rock mass properties incorporated in these analyses are summarized below.

- Density1 = 0 lb/ft^3
- Unconfined Compressive Strength = 4,000 lb/in^2
- Poisons Ratio = 0.2

- Deformation modulus = 3,000,000 lb/in^2
- Base Joint Cohesion = 0 lb/in^2
- Base Joint Friction Angle = 54.5°

NOTES:

1. IT IS COMMON PRACTICE TO APPLY A MASSLESS FOUNDATION FOR FINITE ELEMENT MODELLING, CONSIDERING ONLY THE DEFORMATION MODULUS AND NEGLECTING ITS INERTIA AND DAMPING EFFECTS.
2. IT SHOULD BE NOTED THAT ZERO COHESION HAS BEEN CONSIDERED IN THE PREVIOUS ANALYSES, WHICH IS CONSIDERED VERY CONSERVATIVE. THIS IS, HOWEVER, ASSUMED WITH A CORRESPONDINGLY HIGH FRICTION ANGLE, AND WILL BE RETAINED IN THIS ANALYSIS.

5.2 CONCRETE CHARACTERISTICS

Existing concrete characteristics for the stability evaluations have been applied as follows:

• Density[1]	150 lb/ft^3
• Unconfined Compressive Strength	4,000 lb/in^2
• Poisons Ratio	0.2
• Coefficient of thermal expansion	5x10^{-6} /°F
• Deformation modulus (sustained)	3,000,000 lb/in^2
• Tensile Strength[2]	430 lb/in^2

NOTES:

1. THE CONCRETE PROPERTIES ARE ADOPTED AS PER THE 1996, 2011 AND 2013 STABILITY EVALUATIONS. COMPRESSIVE TEST RESULTS OR TESTING PROGRAM AIMED AT CONFIRMING THE CONCRETE PROPERTIES WERE NOT UNDERTAKEN TO SUPPORT THE ASSUMPTIONS OF THE PREVIOUS STABILITY EVALUATIONS.
2. THE TENSILE STRENGTH IS BASED ON SPLITTING TENSILE TEST STUDIES PERFORMED BY RAPHAEL (1984).

6.0 LOADING

Loads are defined as per Chapter 3 of ASCE Standard No. 7 (ASCE, 2017) and Chapter 3 of USACE, 2016.

6.1 DEAD LOAD

Dead loads include self-weight and any additional weight that is fixed to the structure (formwork and equipment). No dead load surcharge is applied to the analyses as the dam modifications are reducing the overall self-weight.

6.2 HYDROSTATIC LOADING

The following hydrostatic loading conditions were considered:

- Normal reservoir water surface level: 2,607.5 ft
- Reservoir maximum drawdown water surface level: 2,487.5 ft
- Normal operation tailwater level: 2,487.5 ft

6.3 UPLIFT LOADING

No dedicated drainage facilities are included for the dam. The development of pore pressure along the base of the structure is assumed to follow the conventional triangular distribution as illustrated below on Figure 6.1.

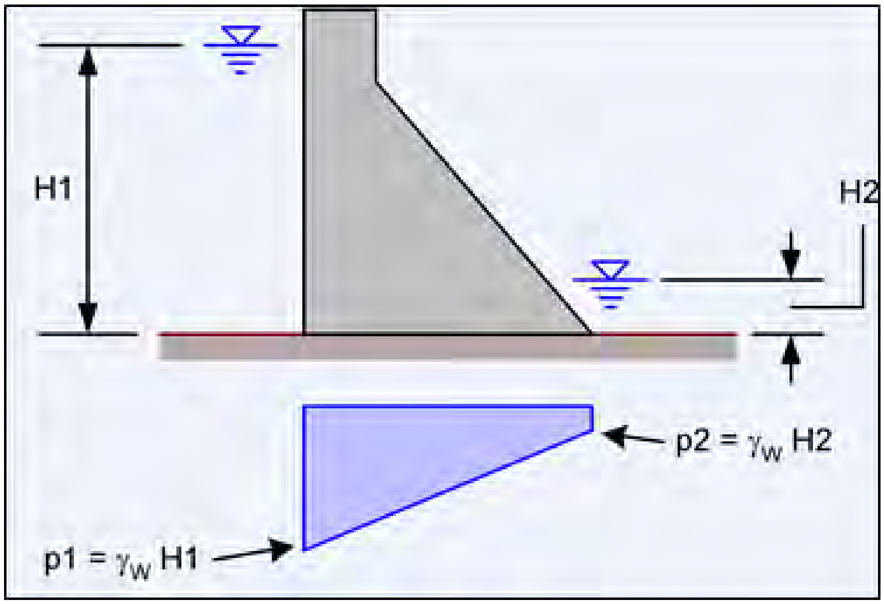


Figure 6.1 Uplift Pressure Distribution Diagram

The linear pressure differential is applied to the dam and foundation interface.

6.4 SILT LOADING

The previous stability evaluations did not include a sediment study and conservatively assumed a sediment load up to the invert of the intakes at elevation 2,575 ft., equal to an additional pressure of 22.6 lb/ft^3 acting on the structure. Silt loading for the stability evaluation has assumed the same loading, applied as a linearly distributed lateral static load with the following mechanical properties:

- Silt active pressure coefficient = 0.333
- Silt effective unit weight = 86.5 lb/ft^3

The 2018 bathymetry surveys indicate that sediment level has accumulated only to elevation 2,512 ft.

6.5 TEMPERATURE LOADING

A temperature differential load case was also considered to evaluate the structural impact associated with the long-term thermal shrinkage. Shrinkage may reduce lateral stress transfer through arching, placing more emphasis on cantilever action.

Considering the age of the structure, internal core temperature due to heat of hydration will already have dissipated. The temperature load is imposed by seasonal temperature changes with respect to a reference temperature at the time the dam was constructed.

As per (B&V, 1996), the stress-free temperature of 41 °F is retained as the reference temperature and a temperature drop load of 4 °F below the reference temperature has been considered.

There are no transverse contraction joints in the dam. To evaluate the effect of long-term shrinkage, a reference temperature of 41 °F was defined in the model and a transient temperature load of +37 °F is considered.

6.6 CONSTRUCTION LOADING

Construction loading is not considered for the analyses. Control of peak particle velocity (PPV) criteria associated with the tunnel excavation operation will be specified and vibration monitoring of the structure will be implemented during the construction of the tunnel to monitor and record particle velocities in longitudinal (radial), transverse, and vertical axes. Minimum PPV criteria will be developed to limit the effects to the dam and foundation rock mass in accordance with U.S. Bureau of Mines (USBM) and Office of Surface Mining and Reclamation Enforcement (OSMRE). No construction surcharge is applied to the analyses.

6.7 EARTHQUAKE LOADING

The Operating Basis Earthquake (OBE) considered for the facility during the dam modification period and drawdown period is defined as the probabilistic horizontal acceleration corresponding to a 50 percent chance of being exceeded in 100 years (or a 144-year return period). A peak ground acceleration of 0.12 g corresponding to the OBE is used for the analyses (Kleindfelder, 2009).

6.7.1 PSEUDO-STATIC ANALYSIS

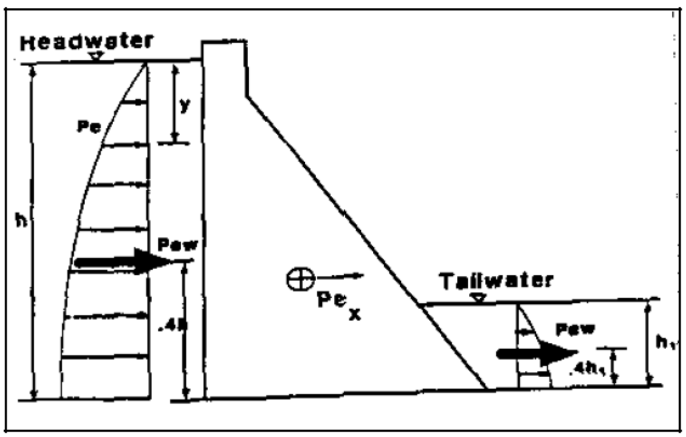
A pseudo static analysis has been completed using the traditional approach (sometimes called the seismic coefficient method of analysis) for the purpose of evaluating overall stability of the structure subject to earthquake loading.

Only the horizontal PGA earthquake ground motion parameter has been used, and the sustained design seismic coefficient has been accepted as 2/3 PGA. Using this approach, the earthquake forces are treated as pure lateral static forces, which are combined with the usual hydrostatic, silt and uplift pressures and gravity load. The lateral earthquake forces are associated with the weight of the dam and the dam impoundment, expressed as a product of the seismic coefficient, which is held constant over the height of the structure.

Inertia of reservoir for horizontal earthquake acceleration induces an increased pressure on the dam concurrently with concrete inertia forces. This load has been approximated using the Westergaard formula and applied as a parabolic pressure distribution to the upstream face as shown in Figure 6.2.

The structure is analysed for sliding stability only as response characteristics of the dam-reservoir-foundation system and the characteristics of the earthquake ground motion itself are not considered. The dynamic stresses computed therefore have little resemblance to the dynamic response of the dam. As such, no stress results are presented or interpreted, and the output relates specifically to overall stability during an earthquake event.

Considering the relatively low PGA associated with the OBE and that for the case of the gravity-arch dam, horizontal cracking at the dam faces cannot develop into a failure mode, the importance of reviewing the stress patterns and critical stress values under transitory seismic action is considered reduced, but can be evaluated using response spectra or modal time history dynamic analysis. This will provide the full dynamic response behaviour with representative dynamic stress output.



$$Pe_x = Ma_x = \frac{W}{g}\alpha g = W\alpha$$

where

Pe_x = horizontal earthquake force

M = mass of dam

a_x = horizontal earthquake acceleration = g

W = weight of dam

g = acceleration of gravity

α = seismic coefficient

Figure 6.2 Seismically loaded dam monolith diagram (EM 1110-2-2200)

7.0 LOAD COMBINATIONS

Loading combinations used for the evaluation of the stability conditions and to evaluate the stresses within the dam during the dam modifications are defined in accordance with the USACE (1994) guidelines and by evaluation of the effects of the critical loads. The loading conditions related to the construction of the proposed low-level outlet have been identified and are summarized:

- Usual Load Combinations (USLC-1): Empty reservoir, self-weight.
- Usual Load Combinations (USLC-2): reservoir water surface elevation 2,607.0 ft, self-weight, sediment elevation 2,575.0 feet, and uplift.
- Usual Load Combinations (USLC-3): temperature drop, reservoir water surface elevation 2,607.0 ft, self-weight, and uplift.
- Pseudo Static (Stability).
- Unusual Load Combination (UNLC-1): reservoir water surface elevation 2,607.0 feet, self-weight, sediment elevation 2,575.0 feet, uplift, and OBE earthquake.

The analyses considered the following loading combinations, as indicated in Table 7.1.

Table 7.1 Stability Loading Combinations

Load Case	Loading Condition	Upstream Water	Silt	Self-Weight	Uplift	Temperature	Earthquake
USLC-1	Usual - Empty	TWL	No	Yes	No	No	No
USLC-2	Usual	FSL	Yes	Yes	Yes	No	No
USLC-3	Usual	FSL	Yes	Yes	Yes	Yes	No
UNLC-1	Unusual	FSL	Yes	Yes	Yes	No	Yes

NOTES:

1. STRESS BASED ANALYSES HAVE AT THIS STAGE ONLY BEEN COMPLETED FOR LOADING COMBINATIONS (USLC-1, USLC-2, AND USLC-3) TO ASSESS THE ANTICIPATED STRUCTURAL BEHAVIOR SUBJECT TO USUAL OPERATING CONDITIONS.

8.0 FINITE ELEMENT ANALYSIS

8.1 FINITE ELEMENT MODEL

The FEM developed for the analyses of the structure included a 3D representation of the dam, the concrete cut-off wall below the dam and a portion of its foundation rock mass. There are no continuous joints in the dam and each component was modelled as a monolithic entity. It is also noted that the section above the cutoff wall contains 30-pound railroad rails, which have been arranged to act as reinforcing steel on the upstream face. The contribution of the steel has not been considered.

A high-density mesh was applied, incorporating 4 ft wide, 10-node solid tetrahedral elements in the dam, with larger elements applied to the foundation. For sliding evaluations, contact elements were defined at the dam and foundation interface. The contact element prevents interference between component faces but allows them to move away from each other to form gaps, thus preventing tension and allowing only compressive forces to develop. The normal and shear forces are resolved on each face and used as input to the sliding safety factor calculations. A FE model for the existing state (without the tunnel) and with the tunnel was compiled. The model and mesh are illustrated in Figures 8.1 and 8.2.

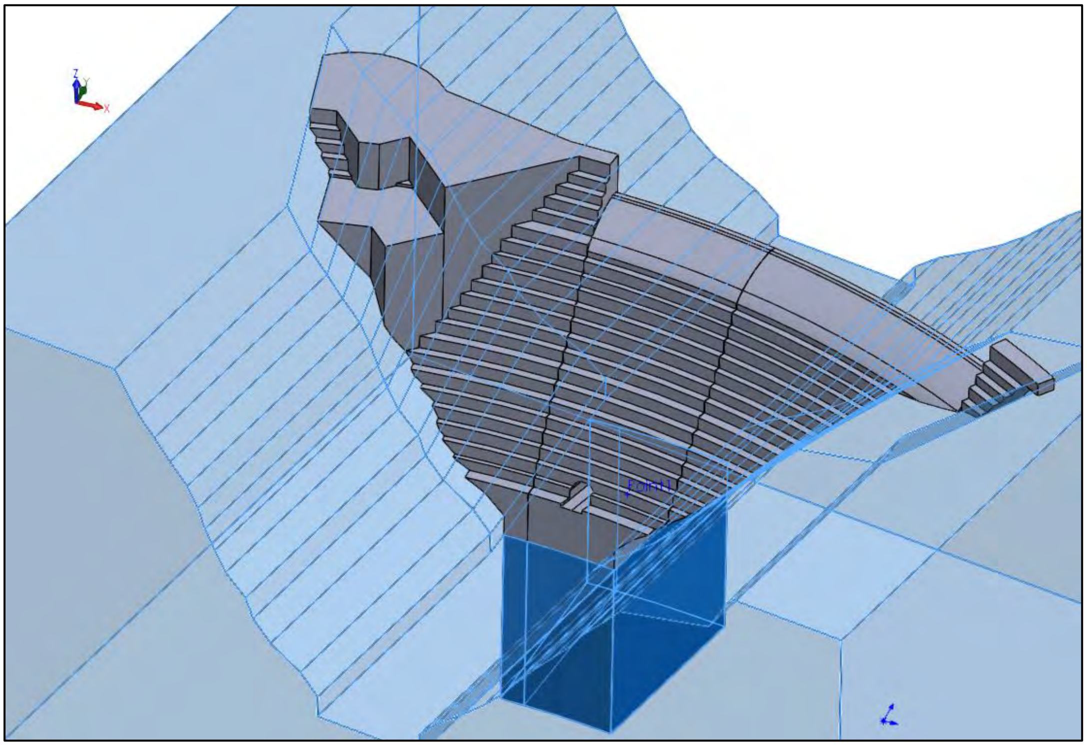


Figure 8.1 3D Model of Structure

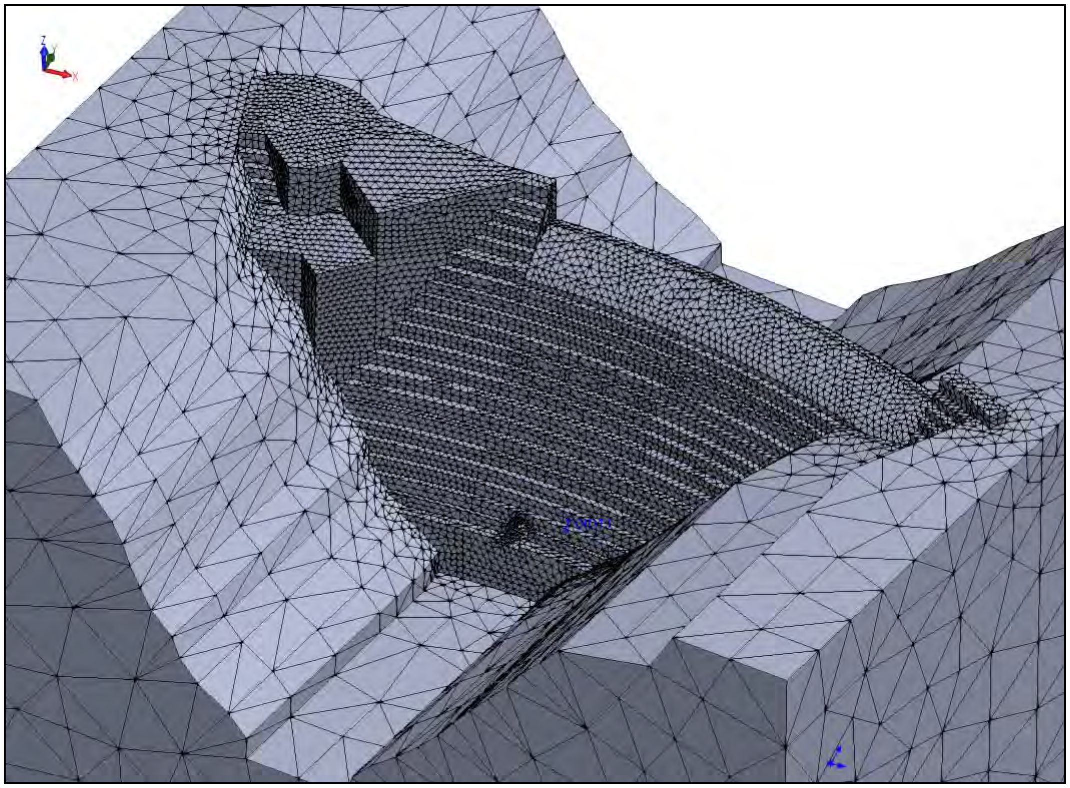


Figure 8.2 Tetrahedral Meshing of Structure

The bottom face of the foundation was fixed against all translational and rotational degrees of freedom and all lateral faces of the foundation block were constrained by defining restraints that prohibit out-of-plane displacements.

8.1.1 GRAVITY LOAD

A constant gravitational acceleration of 32.2 ft/s^2 was applied to the weight of the materials for the analyses.

8.2 STRESS CRITERIA

In the absence of any compressive strength test results for the concrete, experienced based criteria have been defined. The allowable stress criteria for evaluation of the FEM are summarized below in Table 8.1. The applied criteria are considered conservative.

Table 8.1 Allowable Stress Criteria for Concrete

Concrete Strength Criteria	Allowable Stress (psi)
Compressive Strength	1,800
Tensile Strength	240

It is important to recognize the fact that elastic finite element models will always indicate concentrated and exaggerated local stresses at significant physical discontinuities. This condition is particularly apparent at the rigid contact boundaries for which component stiffness changes are evident, such as at the interface between the dam and the foundation. It can also be seen that the smaller the elements used, the larger the computed stresses at these locations. When locally concentrated areas of high stress are apparent on an FE model, it is usually expedient to review average stresses over a larger area and it is correspondingly essential to adopt realistic stress criteria, which take full cognizance of the analysis methods and materials models applied.

8.3 SLIDING STABILITY CRITERIA

Indicative factors of safety against sliding at the base of the dam were evaluated at discrete locations using the resultant shear and normal thrust forces, obtained by summing the free-body forces from the FEM analysis at the foundation contact. These are related to the basic shear-friction sliding safety factor (SSF) formula given as Equation 1:

$$SSF = \frac{\left(\Sigma \overline{V} + U\right)\tan\phi + c\,A_C}{\Sigma H} \tag{EQ. 1}$$

Where: SSF = Sliding Safety Factor

- ΣV = Sum of vertical forces excluding uplift pressure
- U = Uplift pressure force resultant
- φ = friction angle (peak value or residual value)
- c = cohesion (peak value or residual value)
- A_C = Base area in compression
- ΣH = Sum of horizontal forces

The required factors of safety against sliding for the dam structure are indicated below in Table 8.2 as extracted from the previous URS (2013) stability report. These SSF's assume that apparent cohesion is not relied upon for stability.

Table 8.2 Sliding Safety Factors

Stability Condition		Loading Condition	
		Normal	Unusual
Base Joint	Required SSF	1.5	1.3

The stability criteria for use in the stability analysis of the modifications to the current dam arrangement are in accordance with the FERC guidelines (FERC 2016 and 2017) and USACE guidelines (USACE, 1994).

8.4 RESULTS

8.4.1 STRESS RESULTS

The main results of the finite element analyses are presented in Table 8.3 and discussed in this section. Positive stresses refer to computed tensions, while negative stresses denote computed compressions.

Vector plots of the principal stresses reflect both stress magnitude and direction. These are accordingly best suited for the interpretation of the stress distribution patterns and for the overall behavior mechanisms of the structure under analysis. In view of the fact that contour plots illustrate only the maximum magnitudes of the surface stresses, it is important to review the respective values against an interpretation and understanding of the actual structural behavior. The third principal (P3) stresses are generally associated with the largest compressive stresses, whilst the first principal (P1) stresses generally include the maximum tensions occurring within the dam body.

Table 8.3 Result of Stress Evaluations

Load Case	Cantilever Stress (psi)		Arch Stress (psi)		P1's (psi)		P3's (psi)		Max. Crest Displacement ($x10^{-3}$ ft)
	U/S Heel	D/S Toe	U/S Face	D/S Face	U/S Face	D/S Face	U/S Face	D/S Face	
USLC-1a	-80 (-200)	-18 (-60)	-20	-40	In Compression		-200 (-350)	-45 (-140)	-2,5
USLC-1b	-75 (-180)	-20 (-80)	-22	-35	In Compression		-200 (-350)	-40 (-120)	-2,5
USLC-2a	-80 (2,2)	-80 (-320)	-83 (-200 @ RB NOC Interface)	-10 (16)	In Compression	20	-100	-125 (-420)	4,29
USLC-2b	-82 (2,2)	-114 (-380)	-89 (-220 @ RB NOC Interface)	-10 (106 @ tunnel crown)	In Compression	60 (120 @ Tunnel Crown)	-86 (-320)	-125 (-410) (-200 in tunnel sidewalls)	4,35
USLC-3a	-27 (214)	-52 (-152)	-40 (-120)	42 (60)	80 (350)-Dam heel	48 (60)	-55 (-100)	-60 (-170)	5,67
USLC-3b	-22 (220)	-70 (-145) (-290 @ tunnel base)	-45 (-100)	35 (220 @ tunnel crown)	80 (350)-Dam Heel	60 (250 @ Tunnel Crown)	-65 (-95)	-62 (-160) (-200 in tunnel sidewalls)	5,7

NOTES:

1. LOAD CASES FOLLOWED BY (A) DENOTES EXISTING ARRANGEMENT CASES.
2. LOAD CASES FOLLOWED BY (B) DENOTES DAM ARRANGEMENT WITH PROPOSED LOW-LEVEL TUNNELS.
3. STRESS VALUES INDICATED IN PARENTHESES ARE INDICATIVE OF THE PEAK NODAL STRESSES THAT ARE COMPUTED LOCALLY OR AT THE DISCONTINUOUS EDGES OF THE MODEL. THE ELEVATED MAGNITUDE OF THIS STRESS IS ACCORDINGLY CONSIDERED TO BE ASSOCIATED WITH A SINGULARITY AND TO BE EXAGGERATED.

As is apparent from the stress results obtained, the gravity-arch dam structure has significant strength reserves and generally operates under low compressive stress levels throughout the structural section for the load cases considered.

The dam empty scenario represents the condition following full dewatering of the reservoir pool. The displacement results indicate minor upstream movement due to settlement on the flexible foundation when only gravity load is imposed. This deformation leads to compressive stress concentration at the upstream heel, which peak at approximately 80 psi. Under this condition, the structure operates primarily under cantilever action.

With hydrostatic loading applied, the structure indicates its maximum downstream displacement at the crest of the crown cantilever, as is typically desired for a gravity-arch dam. The displacement contour plots (Figure 8.3) indicate near symmetrical deformation, skewed slightly toward the right bank as a result of the overall stiffness provided by the intake structure. The crest displacements are all small and there is no notable change in computed displacement with or without the proposed low-level outlet. Given these low displacements and considering that the structure is 157.5 ft high above the cutoff wall, it is quite evident that an elastic materials response is observed, and that the global rigidity of the gravity-arch dam structure is clearly demonstrated.

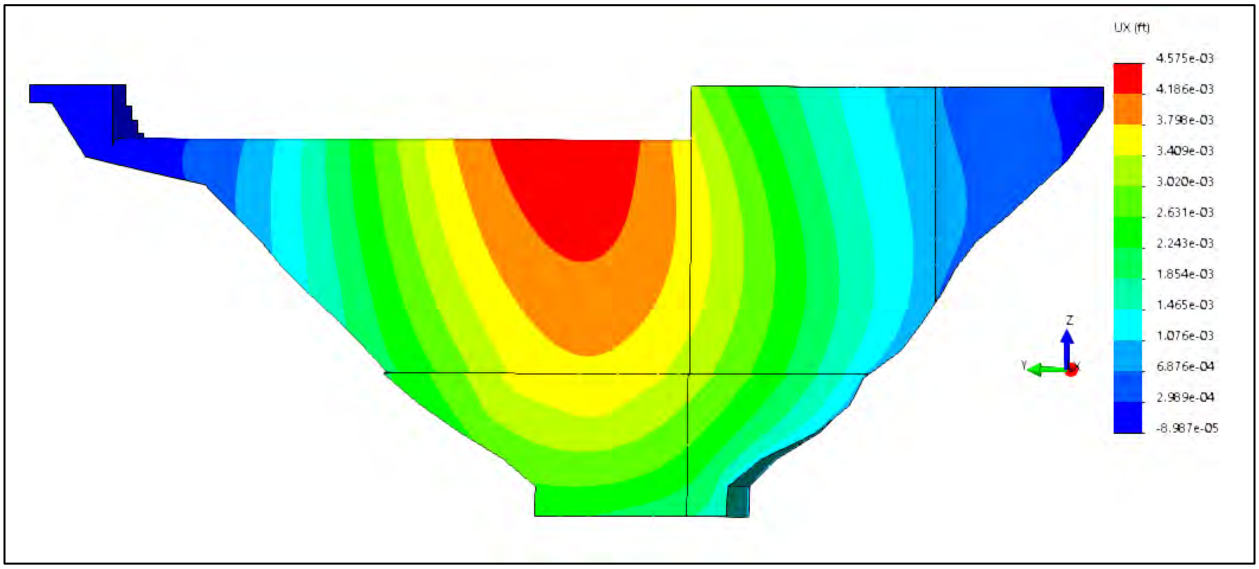


Figure 8.3 Displacement Contours, Case USLC-2

The ability of the structure to transfer 3D stresses due to its curved axis is also apparent. Arch action originates from the crown cantilever, peaking at the crest and indicating near horizontal stress transfer across the spillway section, reducing to cantilever action toward the base. The compressive arch stress levels are approximately 80 to 100 psi, which is low. No high tensile stress concentrations were observed in the relatively massive structural section of the dam, as expected.

When the transient temperature load is applied, the analysis indicates a slightly reduced arch stress magnitude, to approximately 60 psi, with a small increase in crest displacement. Three-dimensional structural behavior is, however, apparent for all load cases considered.

The vector plot of P3 compressive stresses shown on Figure 8.4 illustrates the overall stress transfer mechanism through the dam.

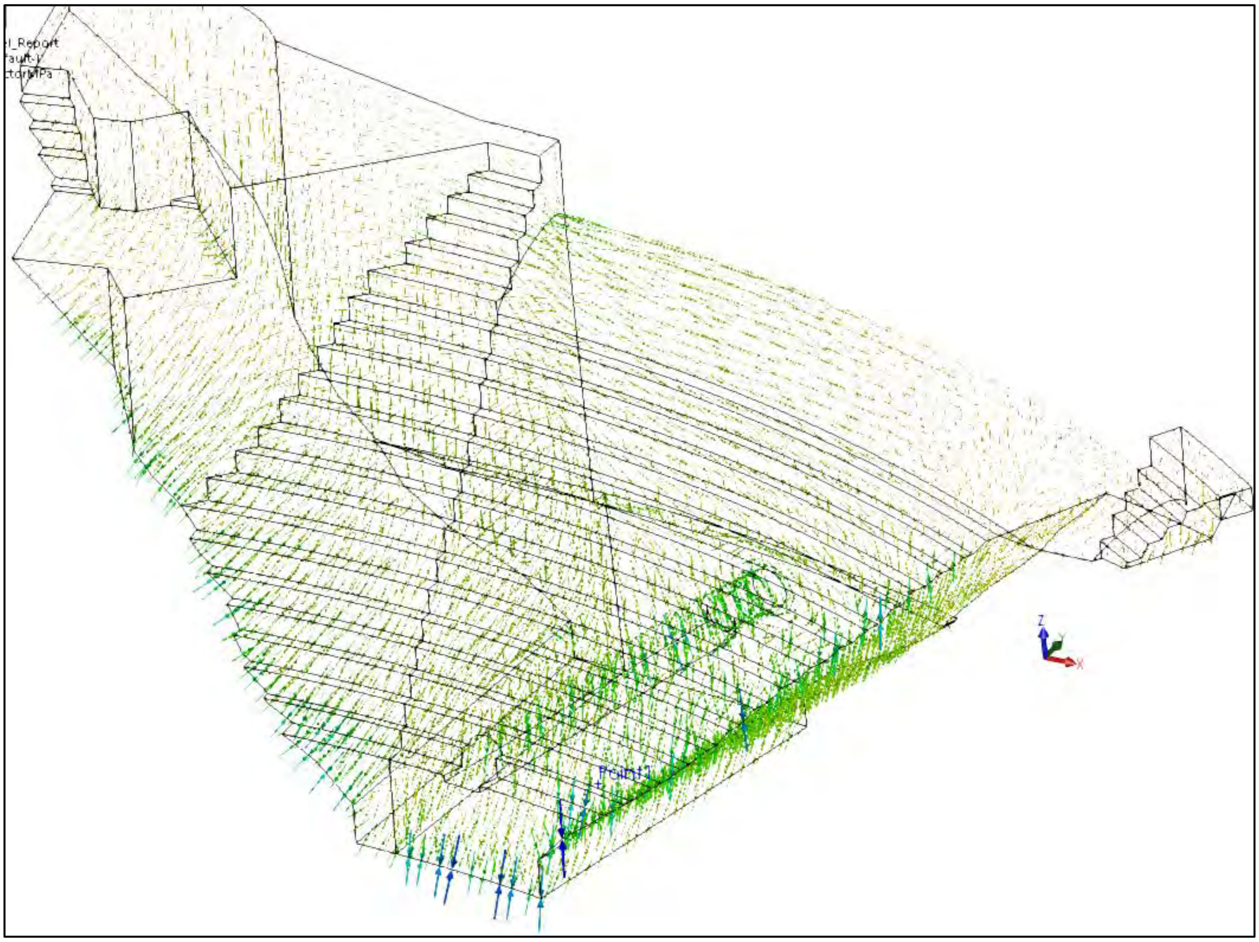
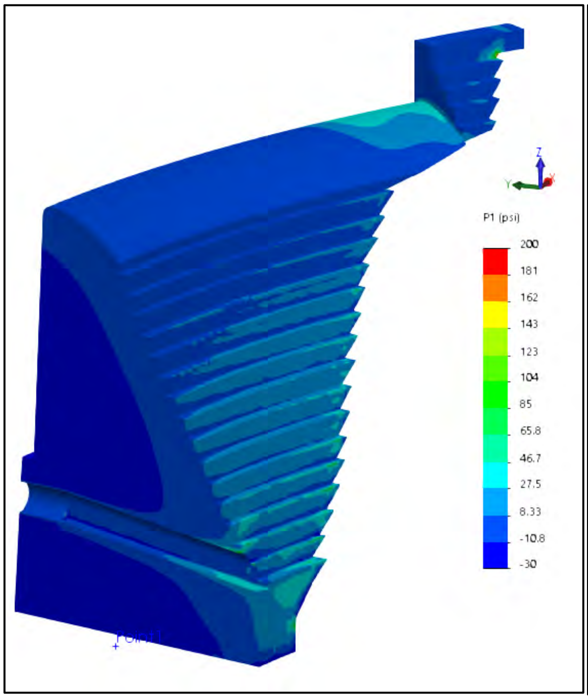


Figure 8.4 P3 Compressive Stresses, USLC-2b

8.4.2 STRUCTURAL STRENGTH

Where the tunnel is excavated, compressive stress develops in a concentric pattern around the excavated periphery, having a magnitude of under 200 psi, which is an indication of a stable profile. None of the computed compressions are problematic and the stress results show negligible effect on the global stress transfer in the dam.

First and third principle stress plots along a cross section through the tunnel centerline indicates the tensile and compressive stresses that develop (Figure 8.5). Horizontal tensile stress is computed at the tunnel crown on the downstream side over the first 10 ft. This is due to the reduced effective section depth above the crown at this location. Tension is below 100 psi, which does not exceed the concrete tensile strength, but will require further investigation to evaluate whether additional support may be necessary to prevent cracking during excavation and when the tunnel is put into operation, to resist the additional internal pressure.



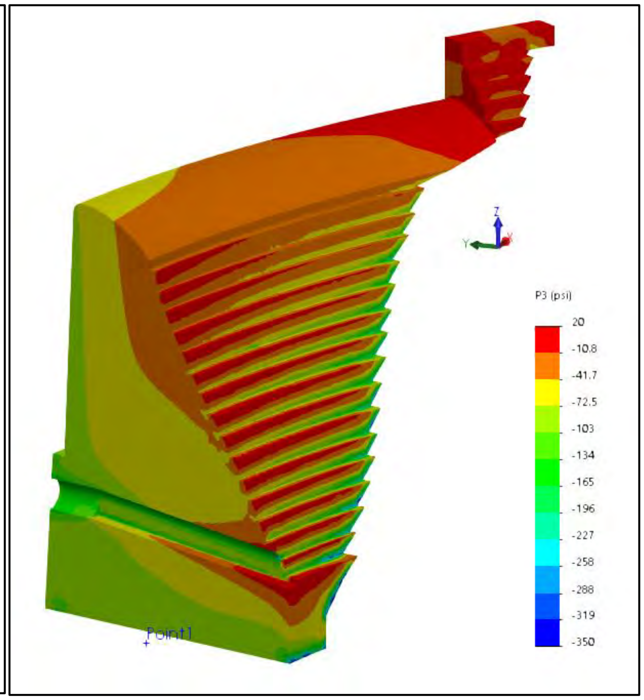


Figure 8.5 Stress Contours, Low-Level Outlet, Centerline, USLC-2b

Overall, the gravity-arch structure utilizes 3D structural behavior to resist and transfer upstream loads effectively. Small crest displacements are consistently computed, and the dam is shown to operate primarily under low levels of compressive stress magnitude.

The results indicate that the dam is conservatively proportioned and will continue to function efficiently when the tunnel is being excavated.

8.4.3 SLIDING STABILITY ANALYSIS

Sliding stability on a gravity-arch structure is seldom problematic due to wedge action that is provided between the dam and foundation. The 3D effect through the development of arch action in the structure provides significant additional strength capacity against sliding, being much greater than those of an independent gravity monolith.

The analysis of the arch effect is an extension to the 3D field using the 2D simple assessment criterion. The evaluation considers sliding safety of the whole dam body, assuming that elastic and unilateral restraint is exerted by the foundation (i.e. only compressive reaction forces) and that the ratio (‘f’) between the tangential and perpendicular reaction forces is uniform along the whole dam-foundation interface (ICOLD, 2004).

In this regard, the free body contact and friction forces acting on contact elements of the structure were obtained directly from the FE model, computed across its 3D footprint. The contact elements only allow compressive forces to develop. The free body forces on each face are summed to obtain the respective global normal passive thrust and total friction force resolved in the upstream to downstream direction, at the base of the structure.

For the sliding analysis, the intention is to comparatively evaluate the effect on stability, before and after including the tunnel. Since the intake works on the right bank are wide and massive, only its restraint stiffening effect is considered in the evaluation. A contact element was defined on the leading face at the commencement of the spillway section, and the free body forces were summed along the foundation faces from this position to the left abutment as illustrated with Figure 8.6 and the results are presented in Table 8.4.

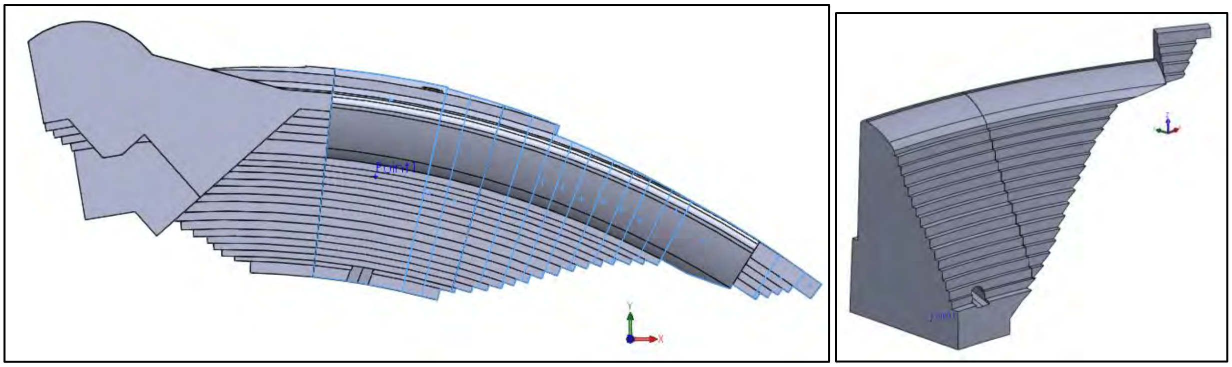


Figure 8.6 Single-Wedge Sliding Model

Table 8.4 Sliding Analysis

Load Combination	Condition	Comparative SSF
USLC-1 - Gravity Only	Current (No Tunnels)	24.50
	Following Tunnel Excavation	23.18
USLC-2i - FSL + Silt	Current (No Tunnels)	2.82
	Following Tunnel Excavation	2.76
USLC-2ii - FSL+Uplift	Current (No Tunnels)	2.16
	Following Tunnel Excavation	2.07
USLC-3 - FSL+Uplift+Silt	Current (No Tunnels)	1.82
	Following Tunnel Excavation	1.75
UNLC-1 - FSL+Uplift+Silt+ OBE	Current (No Tunnels)	1.60
	Following Tunnel Excavation	1.59

NOTES:

1. COHESION IS ASSUMED AS 0 PSI.
2. THE FRICTION ANGLE IS TAKEN AS 54.5º (B&V, 2011 AND URS, 2013).

The sliding stability results demonstrate that even under the very conservative assumption of zero cohesion, the dam structure exceeds the required factor of safety against sliding for all combinations considered, indicating a safe design. Furthermore, compared to the existing case, the effect on the sliding safety factors after including the draw-down tunnel is very small. The tunnel has minor impact on the overall stability of the dam wall.

9.0 STABILITY OF SPILLWAY PIERS

Review of the PFM conditions related to the stability of the spillway piers (URS, 2013) indicate that under the combinations evaluated above for the dam with the proposed low-level outlet tunnel, the factor of safety exceeds the criteria. The increase in the factor of safety is considered minimal and little or no damage is expected to the spillway piers from loading resulting from the OBE.

No further analyses on the spillway piers were completed.

10.0 CONCLUSIONS AND RECOMMENDATIONS

The stability analyses of Copco No.1 concrete dam with the proposed low-level outlet tunnel indicate that the structure satisfies the stability and structural strength criteria for the loading conditions associated with the dam modification. The concrete structure, when analysed for the load and load combinations of the PFM conditions has adequate stability and allowable stresses to withstand the loading conditions associated with the dam modification.

Maximum compressive and tensile stresses are low, but 3-dimensional stress transfer is observed, confirming that the structure functions well as a conservatively proportioned gravity-arch dam. 3-dimensional behaviour increases the structural rigidity and allows the dam to resist static and dynamic loadings better than the case of a gravity structure.

Once the tunnel is excavated, the section is subject to low levels of ring compression. Bursting tensile stress is computed at the crown and invert at the downstream face, where cover is low. This stress does not exceed the concrete tensile strength. The stress and stability results have demonstrated that inclusion of the tunnel has minimal impact on the global stability of the structure.

Stability of the dam, during excavation of the low-level outlet and when there is no impoundment is not critical to public safety.

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