APPENDIX A1
PROJECT NOTATION, UNITS, AND CONVERSION

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1.0 PROJECT NOTATION

1.1 STANDARD UNITS

The standard units for the design of the project will be the following US Customary Units:

- Length: inch (in), feet (ft) and mile (mi)
- Area: acres
- Volume (reservoir): acre-feet (acre-ft)
- Volume (fluid): US gallons, million US gallons (gal, Mgal)
- Volume (concrete, earthfill): cubic yard (yd3)
- Mass: pound (lb), short tons (tons)
- Density: pounds per cubic foot (pcf)
- Pressure: pound-force per square foot (psf)
- Temperature: degrees Fahrenheit (°F)
- Power: horsepower (hp)
- Flow rate: cubic foot per second (cfs), cubic foot per minute (cfm) gallons per minute (gpm)

1.2 CONVERSIONS TO OTHER US CUSTOMARY UNITS

Other US Customary Units will also be used for preparation of the design. These units and conversion factors from the standard units (unless otherwise indicated) will be the following:

- Length: 1 ft = 12 inches (in)
- Length: 1 yard (yd.) = 3 ft
- Length: 1 mile (mi) = 5,280 ft
- Area: 1 acre = 43,560 square feet (sq. ft)
- Volume: 1 acre-ft = 43,560 cubic feet (ft^3)
- Volume: 1 acre-ft = 1,613 cubic yards (yd^3)
- Fluid volume: 1 Mgal = 1,000,000 gallons (gal)
- Mass: 1 ton = 2,000 pounds (lbs)
- Density: 1 short ton per cubic yard ($tons/yd^3$) = 74 pcf
- Pressure: 1 pound-force per square inch (psi) = 144 psf

- Pressure: 1 kilopound per square inch (ksi) = 1,000 psi

1.3 CONVERSIONS TO INTERNATIONAL SYSTEM OF UNITS (SI)

Typical conversion factors to the International System of Units (SI) from the standard units for the project are the following:

- Length: 1 ft = 0.305 meters (m)
- Length: 1 yd. = 0.914 m
- Length: 1 mi = 1.61 kilometers (km)
- Diameter: 1 in = 25.4 millimeters (mm)
- Area: 1 acre = 4,047 square meters (m^2)
- Area: 1 acre = 0.405 hectare (ha)
- Volume: 1 acre-ft = 1,233 cubic meters (m^3)
- Volume: 1 yd^3 = 0.765 m^3
- Volume: 1 ft^3 = 0.028 m^3
- Fluid volume: 1 gal = 3.785 litres (L)
- Fluid volume: 1 Mgal = 3,785 m^3
- Mass: 1 ton = 907 kilograms (kg)
- Mass: 1 ton = 0.907 tonnes (t)
- Density: 1 pcf = 16 kilograms per cubic meter (kg/m^3)
- Density: 1 pcf = 0.016 tonnes per cubic meter (t/m^3)
- Density: 1 $tons/yd^3$ = 1.19 tonnes per cubic meter (t/m^3)
- Pressure: 1 psf = 0.048 kilopascal (kPa)
- Pressure: 1 psi = 6.89 kilopascal (kPa)
- Power: 1 hp = 746 watts (W)
- Flow rate: 1 gpm = 0.227 cubic meters per hour (m^3/hr)
- Flow rate: 1 gpm = 0.063 litres per second (L/s)

APPENDIX A2
MAPPING, SURVEYS, AND SITE CONTROLS

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1.0 OVERVIEW

Project area mapping to document the existing site conditions across the project site was undertaken by the US Department of the Interior (USDOI) in 2009. LiDAR and 3D break-lines for approximately 170 miles on the Klamath River from Link River Dam, OR to the confluence with Elk Creek south of Happy Camp, CA, and surveys along with above and in-water cross-sections at each of nine bridges, were included in the study area (USDOI, 2010). The map projection for the project is as follows:

- Projection: California State Plane:
  - Zone: 1
  - FIPS zone: 0401
  - Vertical Datum: NAVD 1988
  - Horizontal Datum: NAD83
  - Unit: Feet

Site control will be established and verified by the Contractor. Scale factors will be established for the entire site for use in ground to UTM coordinate conversions if required.

Survey control will be established through surveyed benchmarks across the site. Benchmarks are expected to be established at the intake locations, along the penstock routes and at the powerhouse & switchyard locations. Benchmarks will also be established along the transmission line alignments and at major bridge and road crossings.

The Contractor will establish any other control points and benchmarks necessary to set out and construct the Works.

APPENDIX A3
GEOLOGICAL SETTING

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1.0 GENERAL

The Klamath River traverses multiple physiogeographic provinces starting in the Basin and Range Province of Oregon, traversing the High and Western Cascades, Klamath Mountains Province and the Coastal Ranges of northern California, and reaching the Pacific Ocean at Requa, 16 miles south of Crescent City. The Project area is predominantly contained in the Western and High Cascades. The Klamath River pre-dates the formation of the Cascade Mountain Range and maintained a relatively similar course through the mountain building events.

The bedrock of the Project Area comprises volcanic rocks (up to 45 million years old) and includes basalt and andesite lava flows, tuffs, tuff-breccias and volcaniclastic sandstone. The volcanic rocks are intruded by numerous dikes and plugs of andesite, rhyolite, and basalt. Many of the volcanoes associated with the Western Cascades have since eroded, but large shield volcanoes and vents of the High Cascades remain and are still active in present times.

Large deposits of coarse alluvium were deposited along the Klamath River during the period of the last glaciation when the river had a higher discharge. Lacustrine deposits were laid down in former temporary lakes that were created at the present-day sites of the Copco No. 1 and J.C. Boyle Reservoirs when the Klamath River was temporarily ‘dammed’ by volcanic activity.

2.0 J.C. BOYLE HYDROELECTRIC FACILITY

The topography in the area of the J.C. Boyle hydroelectric facility is predominantly a low-gradient bowl with gently rolling terrain. The steepest topography exists in the river canyons upstream and downstream of the reservoir. All the bedrock units in the area are estimated to be younger than 5 million years and associated with High Cascades volcanism from large stratovolcanic complexes and smaller shield volcanoes and vents; these are typically basaltic flows interlayered with volcaniclastics and hydrovolcanic deposits, leading to highly complex geology from a large variety of sources.

Faulting is very prominent in the J.C. Boyle Reservoir area and appears to be associated with extensional tectonics of the Basin and Range Province that began approximately 1.5 to 2.0 million years ago. The bowl topography of the reservoir area likely formed as a dropped-down basin. At least one fault splay is predicted to extend into the dam area (PanGEO, 2008).

The surficial deposits at the reservoir comprise lacustrine deposits as well as river alluvium and local colluvial deposits. The lacustrine deposits comprise older sediments that were laid down in a former lake that was created when the river was temporarily ‘dammed’ by volcanic activity and recent sediments, which were deposited within the reservoir.

3.0 COPCO NO. 1 AND COPCO NO. 2 HYDROELECTRIC FACILITIES

The area surrounding the Copco No. 1 and Copco No. 2 reservoirs is characterized by hillsides comprised of low gradient lava flows from surrounding shield volcanoes. The Copco Basalt (0.14 million years) makes up the vertical upper walls of the canyon in the vicinity of the dam site. The Copco Basalt was created by volcanic flows from vents on both sides of the river, which led to damming of the river and the formation of a lake in the same area as the present-day reservoir. The Western Cascades Volcanics underlie most of the slopes on the shoreline of the reservoir. This unit comprises andesite with interstratified tuff-breccia, volcaniclastic sandstone and tuffs.

Small faults that have been historically mapped in the area of the Copco No. 1 and No. 2 hydroelectric facilities typically trend west to northwest south of the river. Limited structural mapping of faults north of the river shows a northward trend.

The surficial deposits at the Copco No. 1 Reservoir comprise lacustrine deposits as well as river alluvium and local colluvial deposits. The lacustrine deposits mainly comprise sediments that were laid down in a former lake that was created when the river was temporarily ‘dammed’ by volcanic activity. Fine sediments, comprising silts and diatomite (siliceous skeletal remains of diatoms) were deposited in the lake. The formation of the lake resulted in fluvial terraces and fans developing further still from the contemporary course of the river. Recent lacustrine deposits have accumulated within the reservoir since its construction. Colluvium occurs locally around the shoreline of the Copco No. 2 Reservoir.

Natural groundwater springs can be observed and typically exist in the tuffaceous layers between impermeable lava flows and along lithological contacts. The rapidly cooled more porous lava flow tops and bottoms are common aquifers in the region.

4.0 IRON GATE HYDROELECTRIC FACILITY

The Iron Gate Dam and its reservoir lie entirely within the Western Cascades Geologic Province. The bedrock around the shoreline comprises andesite and basalt with volcanic breccia, tuff, tuffaceous siltstones, and sandstones. The Western cascades strata dip gently towards the east. Surficial deposits around the reservoir shoreline include colluvium and local alluvial deposits at drainage line intersections.

Natural springs are also found in numerous locations on the valley slopes surrounding the Iron Gate Reservoir.

References:

PanGEO Incorporated (PanGEO), 2008. Geotechnical Report – Klamath River Dam Removal Project. August. Seattle, Washington, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015a. J.C. Boyle Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015b. Copco No. 1 Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015c. Iron Gate Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

APPENDIX A4
SEISMICITY

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1.0 DESIGN PARAMETERS FOR TEMPORARY STRUCTURES

A standard and guideline review of DSOD, the California Water Code, Caltrans, USACE, ASCE, FEMA, FERC, USBR, and Uniform Building Code documents did not yield clear design criteria for the seismic design of temporary structures. KP has also reviewed the latest Supporting Technical Information Documents (STIDs) provided by PacifiCorp as they pertain to geology and seismicity at J.C. Boyle, Copco No. 1, Copco No. 2, and Iron Gate. It was determined from these documents that the site-specific ground motion parameters for permanent structures were developed by Kleinfelder West Inc. (Kleinfelder) and Black & Veatch using the 2002 United States Geological Survey (USGS) database. The seismic design parameters presented in this appendix have been determined using the updated USGS seismic hazard database in conjunction with a design life equal to or less than one year. The current data provided by the USGS seismic hazard database is based on the 2014 model which incorporates the latest ground motion prediction models for shallow crustal earthquakes (known as the Next Generation Attenuation Models).

The probability of exceedance for the Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE) events were assessed to quantify the risk associated with structures having a design life of 1 year. The probability of exceedance was calculated using the following equation:

$$Q = 1 - e^{-L/T}$$

Where: Q = probability of exceedance

L = design life (years)

T = return period (years)

The resulting probabilities of exceedance are as-follows:

- OBE (1/475-year event): 0.2% probability of exceedance
- MCE (1/2475-year event): 0.04% probability of exceedance

The OBE event was selected for the design of temporary structures having a design life of one year or less. The spectral accelerations corresponding to the OBE event at each site are presented with the OBE PGAs in Table 1.1.

Table 1.1 Selected Seismic Design Parameters for Temporary Structures at Each Site

Site	Return Period (years)	2014 USGS[1] PGA (g)	2014 USGS[1] Sa (0.2 s)	2014 USGS[1] Sa (1.0 s)
J.C. Boyle	475	0.17	0.39	0.14
Copco No. 1	475	0.12	0.26	0.10
Copco No. 2	475	0.12	0.26	0.10
Iron Gate	475	0.11	0.25	0.10

NOTES:

1. PGA AND SPECTRAL ACCELERATION VALUES TAKEN FROM THE USGS UNIFIED HAZARD TOOL DABATASE (USGS).

2.0 DESIGN PARAMETERS FOR PERMANENT SLOPES

Permanent slopes are designed to the MCE values provided in the STIDs for the hydropower facilities. The STIDs are presented in Appendix J.

References:

Black & Veatch, 2010. Copco No. 1 Development Klamath River Hydroelectric Project, FERC Project No. 2082 – Seismic Analysis of Structures. January 12.

Black & Veatch, 2009. Technical Memorandum – Time Histories for J.C. Boyle Dam. September 4.

Black & Veatch, 2004. 5.A Seismicity – Iron Gate. September 15.

Kleinfelder West Inc. (Kleinfelder), 2009a. Geoseismic Evaluation Report – J.C. Boyle Dam. June 19. Salt Lake City, Utah, USA.

Kleinfelder West Inc. (Kleinfelder), 2009b. Geoseismic Evaluation Report – Copco No. 1 Dam. June 19. Salt Lake City, Utah, USA.

Kleinfelder West Inc. (Kleinfelder), 2009c. Geoseismic Evaluation Report – Iron Gate Dam. June 19. Salt Lake City, Utah, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015a. J.C. Boyle Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015b. Copco No. 1 Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

PacifiCorp Energy Inc. (PacifiCorp), 2015c. Iron Gate Development: Supporting Technical Information Document, Section 5 – Geology and Seismicity. April 30. Portland, Oregon, USA.

United States Geological Survey (USGS). Earthquake Hazards Program: Uniform Hazard Tool. (Accessed from: https://earthquake.usgs.gov/hazards/interactive/

APPENDIX A5
CLIMATE

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1.0 OVERVIEW

The Project sites are located in predominantly rural areas of southern Oregon and northern California, along the riparian corridors of the Klamath River and its tributaries. The local climate is characterized by cool, wet winters and warm, dry summers. Cold air temperatures generally occur from November through March and warmer air temperatures and drier conditions occur from April through October with summer air temperatures highest in July, August, and September. The summers are dry with occasional isolated thunderstorms from July to September (Oregon Watershed Enhancement Manual, 2001).

The area is characterized by varying precipitation with a drier climate near Klamath Falls, Oregon and a wetter climate in northern California. Most precipitation occurs in the winter months of November, December and January (Oregon Watershed Enhancement Manual, 2001). Due to generally high elevations, the upper plateau has cool temperatures and receives a substantial amount of snow, which accumulates into moderately deep snowpack (Oregon Watershed Enhancement Manual, 2001). At its higher elevations (above 5,000 feet), the Klamath Basin receives rain and snow during the late fall through to spring.

2.0 AVAILABLE DATA

The National Oceanic and Atmospheric Administration (NOAA) operate several cooperative climate stations in the region. The regional climate datasets most relevant to the Project sites are:

- Keno, Oregon: NCEI COOP #354403 (6 miles from J.C. Boyle facility)
- Copco Dam No. 1, California: NCEI COOP #041990 (located at Copco No. 1 facility)

The location of the regional climate stations and the Project sites are shown on Figure 2.1.

Figure 2.1 Regional Climate Station Locations and Project Locations

2.1 TEMPERATURE

Data from the regional climate station within the closest proximity to each site was selected to represent the temperatures at that Project site. Available temperature data for the regional climate stations are presented in Table 2.1. The mean annual air temperature range is 44 °F to 52 °F between Keno, Oregon climate stations and Copco Dam No. 1, California. The months with the highest mean temperatures for the stations are July through September with maximum monthly mean temperatures ranging between 68 °F and 75 °F. The lowest minimum monthly mean temperatures are in January and December ranging between 29 °F and 36 °F.

Table 2.1 Measured Regional Temperature Data Summary

Station Details[1]	Unit	Keno, OR	Copco Dam No. 1, CA
Station Number	-	35-4403	04-1990
Latitude	° ' "	42° 7' 46.92" N	41° 58' 46.92" N
Longitude	° ' "	121° 55' 46.92" W	122° 20' 16.08" W
Elevation	ft	4,116	2,703
Distance from Site			
Nearest Project Site(s)	-	J.C. Boyle	Copco No. 1, Copco No. 2, Iron Gate
Distance from Site	mi	6.2	6.0 from Iron Gate
Period of Record[2]	-	1927-2019	1959-2019
Measured Values[3, 4]			
Mean Annual	ºF	44.4	52.1
Mean Annual High	ºF	58.5	65.7
Mean Annual Low	ºF	29.1	38.6
Maximum Monthly Mean	ºF	68.4	75.3
Minimum Monthly Mean	ºF	29.0	35.9
Maximum Recorded Daily	ºF	103	115
Minimum Recorded Daily	ºF	-20	-2

NOTES:

1. DATA OBTAINED FROM NOAA ATLAS 14 – PRECIPITATION-FREQUENCY ATLAS OF THE UNITED STATES (2014).
2. THE PERIOD OF RECORD IDENTIFIES WHEN THE FIRST AND LAST MEASUREMENTS WERE TAKEN AND DOES NOT REPRESENT A CONTINUOUS PERIOD OF DATA COLLECTION.
3. MEASURED TEMPERATURE VALUES OBTAINED FROM NOAA REGIONAL CLIMATE CENTERS (ACIS, 2015).
4. MEASURED TEMPERATURE VALUES REPRESENT RECORDED DATA ONLY.

2.2 PRECIPITATION

Precipitation values for the project sites were derived in a similar manner to the temperature values, with the nearest regional climate station data providing the representative values for each specific project site. The wettest months are November through January. The proportion of precipitation falling as snow is directly correlated to temperature, which varies with each location within the Project region. In the upper watershed, snow is the primary form of precipitation for elevations above 5,000 feet.

The maximum daily rainfall range observed (recorded) at the regional climate stations is 3.0 inches and 6.0 inches for the Copco Dam No. 1 and Keno climate stations, respectively. The daily rainfall was converted to an equivalent 24-hr rainfall using a standard factor of 1.13 (Hershfield, 1961) resulting in maximum 24-hr rainfall of 3.4 inches to 6.8 inches for the Copco Dam No. 1 and Keno climate stations, respectively. The precipitation values are summarized in Table 2.2 and the mean monthly precipitation values are summarized in Table 2.3.

Table 2.2 Measured Regional Precipitation Summary[1, 2]

	Unit	Keno, OR	Copco Dam No. 1, CA
Period of Record[3]	-	1927-2019	1959-2019
Mean Annual Precipitation	in.	18.6	19.7
Mean Total Annual Rainfall	in.	13.4	18.0
Percentage of Annual Precipitation as Rain	%	72%	91%
Mean Total Annual Snowfall	in.	51.5	16.8
Mean Total Annual SWE[4]	in.	5.1	1.7
Maximum Recorded 24-hour Precipitation[5]	in.	6.8	3.4

NOTES:

1. DATA OBTAINED FROM NOAA REGIONAL CLIMATE CENTERS (ACIS, 2015).
2. MEASURED PRECIPITATION VALUES REPRESENT RECORDED DATA ONLY.
3. THE PERIOD OF RECORD IDENTIFIES WHEN THE FIRST AND LAST MEASUREMENTS WERE TAKEN AND DOES NOT REPRESENT A CONTINUOUS PERIOD OF DATA COLLECTION.
4. SWE – SNOW WATER EQUIVALENT. VALUES DETERMINED ASSUMING SNOW WATER EQUIVALENCY CONVERSION FACTOR OF 0.1 (NRCS).
5. MAXIMUM RECORDED 24-HOUR PRECIPITATION WAS DETERMINED BY APPLYING A 1.13 FACTOR (HERSHFIELD, 1961) TO THE MAXIMUM RECORDED DAILY PRECIPITATION.

Table 2.3 Measured Regional Mean Monthly Precipitation

	Keno, OR	Copco No. 1 Dam, CA	Keno, OR	Copco No. 1 Dam, CA	Keno, OR	Copco No. 1 Dam, CA
	Average Precipitation (in)		Average Number of Days with Precipitation >0.5 in		Average Total Snowfall (in)	
Jan	2.9	3.0	4	3	14.8	5.4
Feb	2.0	2.2	3	3	9.8	2.8
Mar	1.9	2.1	4	3	6.1	1.6
Apr	1.3	1.6	3	2	1.9	0.5
May	1.2	1.3	3	2	0.2	-
Jun	0.8	0.8	2	1	-	-
Jul	0.3	0.3	1	1	-	-
Aug	0.5	0.4	1	1	-	-
Sep	0.6	0.6	1	1	-	-
Oct	1.5	1.3	2	2	0.5	-
Nov	2.5	2.9	3	3	5.8	1.7
Dec	3.2	3.4	4	3	12.8	5.1
Mean Annual	18.6	19.7	32	24	51.5	16.8

The intensity duration frequency (IDF) data for the Copco Dam No. 1 climate station were provided by NOAA's Precipitation Frequency Data Server (NOAA, 2017). NOAA provides data for recurrence periods from 1 to 1,000 years with durations ranging from 5 minutes to 60 days. The IDF data for the Copco Dam No. 1 climate station is tabulated in Table 2.4 and are representative of the Copco No. 1, Copco No. 2, and Iron Gate Project Sites.

Table 2.4 IDF Data for Copco Dam No. 1 Climate Station (inches)

Duration	Recurrence Interval (yrs)									
	1-yr	2-yrs	5-yrs	10-yrs	25-yrs	50-yrs	100-yrs	200-yrs	500-yrs	1,000-yrs
5-min	0.10	0.14	0.20	0.24	0.31	0.36	0.41	0.47	0.62	0.77
10-min	0.15	0.20	0.28	0.35	0.44	0.51	0.59	0.68	0.89	1.10
15-min	0.18	0.25	0.34	0.42	0.53	0.62	0.72	0.82	1.07	1.33
30-min	0.24	0.33	0.45	0.55	0.70	0.82	0.95	1.09	1.42	1.76
60-min	0.32	0.44	0.60	0.74	0.94	1.10	1.27	1.46	1.91[1]	2.36[1]
2-hr	0.45	0.59	0.77	0.92	1.13	1.30	1.47	1.65	1.93[1]	2.38[1]
3-hr	0.55	0.70	0.90	1.07	1.30	1.47	1.65	1.84	2.09	2.41
6-hr	0.79	0.98	1.23	1.43	1.70	1.91	2.12	2.34	2.63	2.85
12-hr	1.10	1.36	1.70	1.98	2.36	2.66	2.96	3.26	3.68	4.01
24-hr	1.57	1.96	2.47	2.90	3.50	3.98	4.47	4.99	5.70	6.28
2-day	1.98	2.50	3.20	3.78	4.61	5.26	5.94	6.67	7.68	8.50
3-day	2.29	2.91	3.76	4.46	5.46	6.24	7.07	7.94	9.16	10.10
4-day	2.48	3.18	4.11	4.89	5.97	6.83	7.71	8.65	9.95	11.00
7-day	2.90	3.73	4.81	5.69	6.90	7.83	8.78	9.77	11.10	12.10
10-day	3.22	4.15	5.34	6.31	7.61	8.59	9.59	10.60	12.00	13.00
20-day	4.16	5.40	6.98	8.22	9.86	11.10	12.30	13.50	15.10	16.30
30-day	5.07	6.61	8.53	10.00	12.00	13.40	14.90	16.30	18.10	19.50
45-day	6.42	8.36	10.80	12.60	15.10	16.80	18.50	20.20	22.40	24.00
60-day	7.56	9.80	12.60	14.70	17.40	19.40	21.30	23.20	25.60	27.40

NOTES:

1. THE 500-YR AND 1,000-YR 60-MIN AND 2-HR VALUES WERE FLAGGED AS POTENTIALLY ERRONEOUS DUE TO MINIMAL INCREASE IN RAINFALL WITH INCREASE IN STORM DURATION.
2. IDF DATA TAKEN FROM NOAA'S PRECIPITATION FREQUENCY DATA SERVER (NOAA, 2017).

The IDF curves for the Keno climate station were determined using information provided by the Oregon Department of Transportation (ODOT) and supplemented by data available through the Western Regional Climate Center (WRCC). Intensity Duration Recurrence (IDR) information is dictated by the Oregon Rainfall IDR Curve Zone Map as stipulated in the ODOT Hydraulics Manual (ODOT, 2014). The Rainfall IDR Curve Zone Map is shown in Figure 2.2.

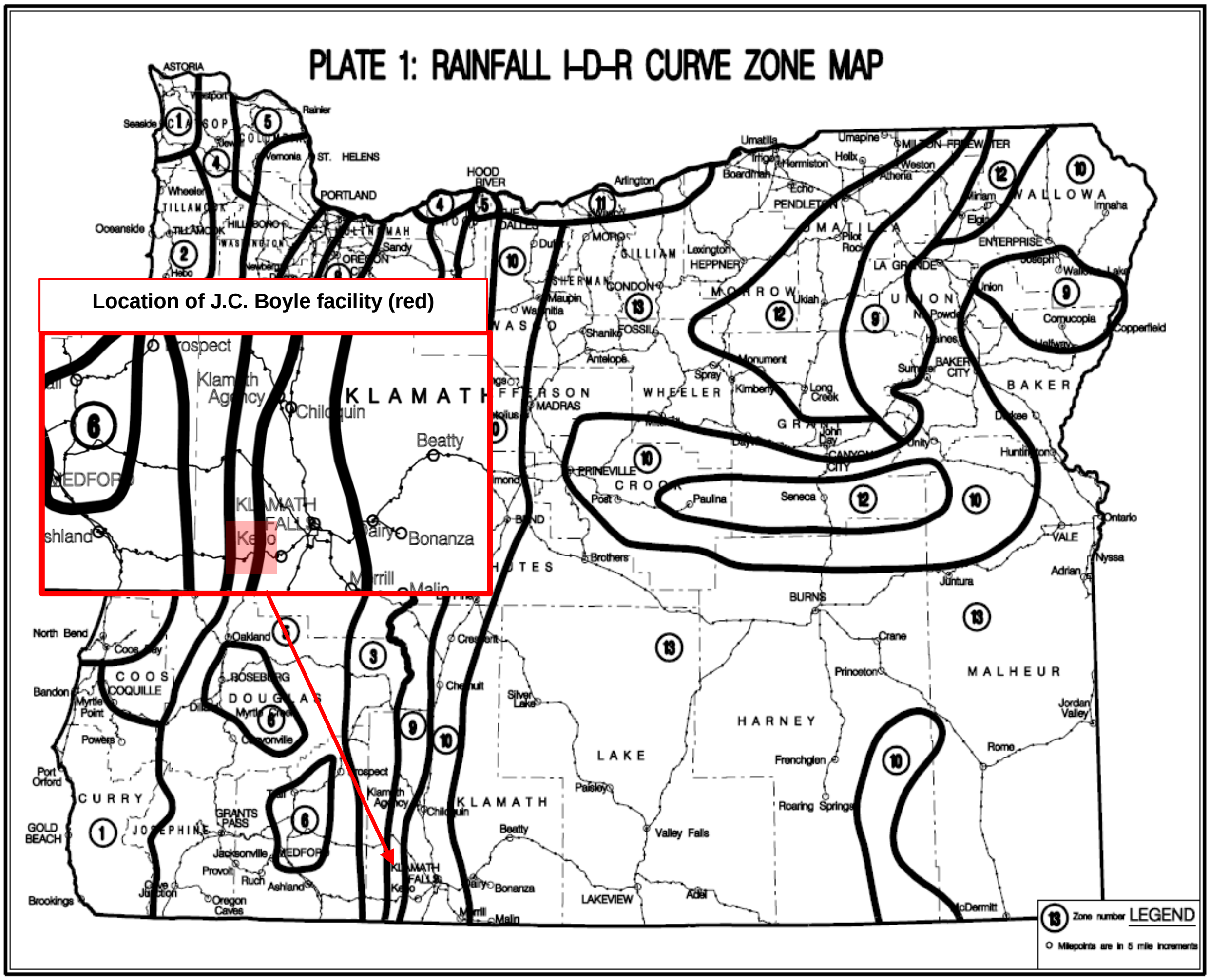

Figure 2.2 ODOT Rainfall IDR Curve Zone Map and Approximate Location of J.C. Boyle (ODOT, 2014)

The zoning map is used to identify which IDR data should be applied to a site. Zone 9 has been selected as representative of the IDR data for the J.C. Boyle project site based on the site location. The IDR rainfall intensity data for Zone 9 is tabulated in Table 2.5.

Table 2.5 IDR Data for Oregon Zone 9 (inches)

Duration	Recurrence Interval (yrs)					
	2-yrs	5- yrs	10-yrs	25-yrs	50-yrs	100-yrs
5-min	0.13	0.18	0.21	0.25	0.29	0.34
10-min	0.19	0.28	0.33	0.40	0.44	0.52
15-min	0.25	0.35	0.41	0.50	0.58	0.66
30-min	0.34	0.48	0.58	0.70	0.80	0.90
60-min	0.44	0.64	0.73	0.88	1.05	1.15
2-hr	0.58	0.82	0.90	1.04	1.20	1.38
3-hr	0.72	0.96	1.08	1.23	1.38	1.59
6-hr	1.02	1.32	1.50	1.62	1.80	2.04
24-hr	2.00	2.50	2.80	3.20	3.80	4.00

NOTES:
1. DATA FOR RECURRENCE PERIODS FROM 2 TO 100 YEARS WITH DURATIONS RANGING FROM 5 MINUTES TO 6 HOURS PROVIDED BY ODOT (ODOT, 2014).
2. 24-HOUR DURATION EVENT DATA PROVIDED BY WRCC PRECIPITATION FREQUENCY MAPS PUBLISHED IN NOAA ATLAS 2 AND REPRESENTS THE IDF DATA FOR THE WHOLE STATE OF OREGON (WRCC, 1973).

2.3 WIND

Regional wind data was not available for the Copco Dam No. 1 and Keno climate stations at the time of the preparation of this report. Wind is a design parameter required for the design of bridges and piers. The American Association of State Highway and Transportation Officials (AASHTO) requires a wind velocity at 30 ft (V_{30}) above low ground/above design water level and recommends the adoption of V_{30} = 100 mph in the absence of site-specific wind data (AASHTO, 2012). This value has been adopted for the design. Alternative wind velocities may be considered to evaluate freeboard requirements specific to wave run-up and set-up considerations.

References:

American Association of State Highway and Transportation Officials (AASHTO), 2012. Load and Resistance Factor Design Bridge Design Specifications. 6th Edition. Washington, DC, USA.

Applied Climate Information System (ACIS), 2015. Single Station Daily Data Listing. NOAA Regional Climate Centers. Retrieved from: http://scacis.rcc-acis.org/ (accessed June 21, 2019).

Hershfield, D. M., 1961a. Rainfall Frequency Atlas of the United States. Technical Paper No. 40, Weather Bureau, United States Department of Commerce, Washington, DC.

Natural Resources Conservation Services Oregon (NRCS). What is Snow Water Equivalent? United States Department of Agriculture (USDA). Retrieved from: https://www.nrcs.usda.gov/wps/portal/nrcs/detail/or/snow/?cid=nrcs142p2_046155 (accessed July 4, 2019)

NOAA's National Weather Service - Hydrometeorological Design Studies Center, 2017. NOAA Atlas 14 Point Precipitation Frequency Estimates: CA. Retrieved from: https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca (accessed June 14, 2019).

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APPENDIX A6
HYDROLOGY

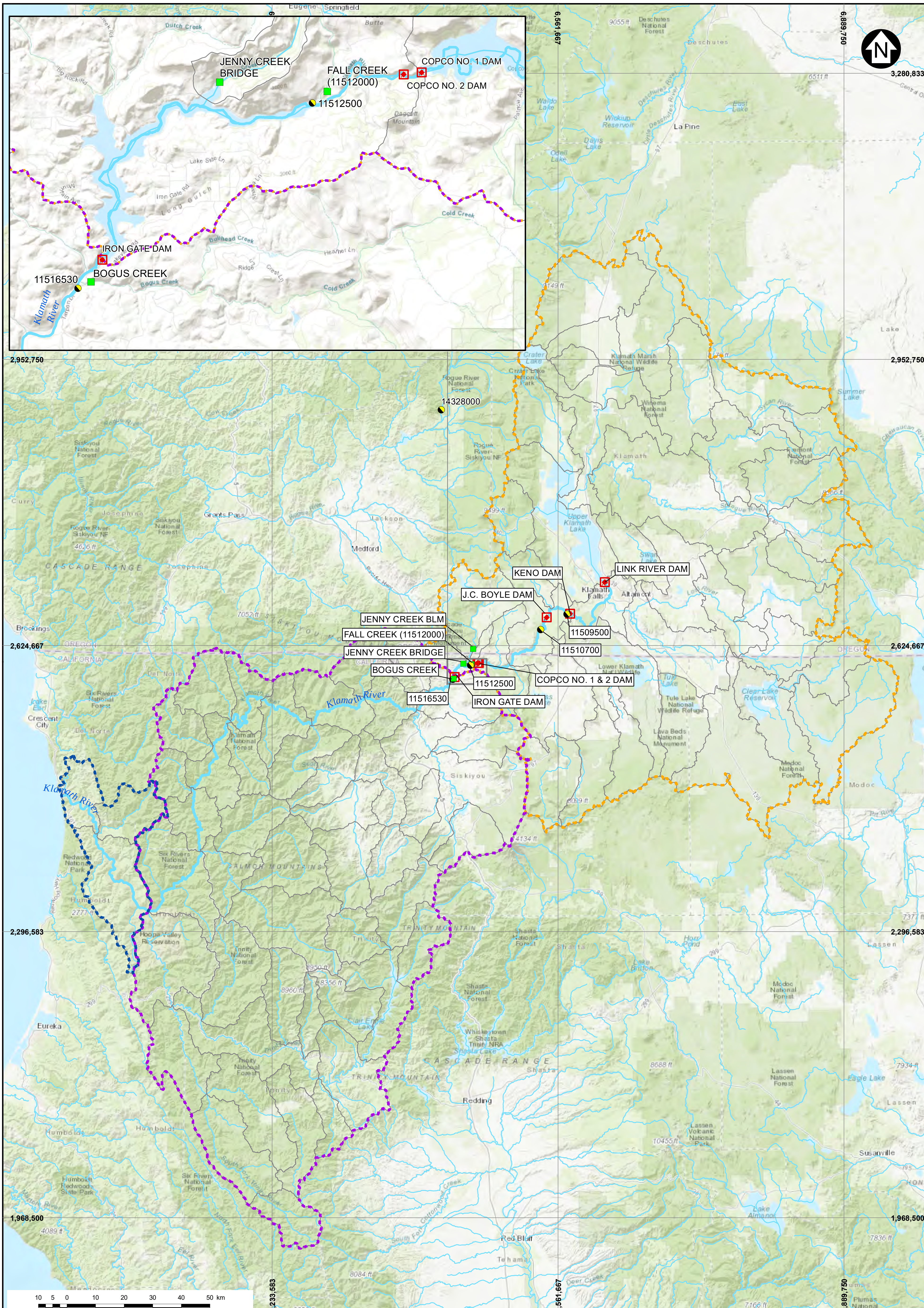
TABLE OF CONTENTS

1.0 WATERSHED DESCRIPTION

The Klamath River originates at the outlet of Upper Klamath Lake in southern Oregon and flows approximately 250 miles southwest through the Cascade Mountains of southern Oregon and northern California to the Pacific Ocean. The Upper Klamath Basin has five main lakes: Crater Lake, Upper Klamath Lake, Lower Klamath Lake, Clear Lake, and Tule Lake. The Upper Klamath Basin contains all the hydroelectric developments on the Klamath River, including the Klamath River Renewal Project (KRRP) sites. The Middle Klamath Basin extends 150-miles from Iron Gate Dam downstream to the Trinity River confluence. The Lower Klamath Basin starts at the Trinity River confluence and extends 43 miles downstream to the Pacific Ocean.

The Upper Klamath Basin has broad valleys shaped by volcanoes and active faulting. The fault-bounded valleys contain all the large, natural lakes and large wetlands of the Klamath Basin. The Klamath River flows through mountainous terrain from J.C. Boyle Dam to Iron Gate Dam. Downstream of Iron Gate Dam, and for most of the river's length from there to the Pacific Ocean, the river maintains a relatively steep, high-energy channel (NRC, 2004).

A map of the reach containing the four PacifiCorp dams covered by the KRRP is given on Figure 1.1.



LEGEND:

- ADDITIONAL HYDROLOGY STATION
- USGS STATION
- DAM LOCATION
- KLAMATH RIVER
- LOWER KLAMATH BASIN
- MIDDLE KLAMATH BASIN
- UPPER KLAMATH BASIN
- SUB-CATCHMENT

NOTES:

1. BASE MAP: ESRI ONLINE TOPOGRAPHIC MAP.
2. COORDINATE GRID IS IN METRES.
COORDINATE SYSTEM: NAD 1983 2011 STATEPLANE CALIFORNIA I FIPS 0401 FT US.
3. THIS FIGURE IS PRODUCED AT A NOMINAL SCALE OF 1:1,250,000 FOR 11x17 (TABLOID) PAPER. ACTUAL SCALE MAY DIFFER ACCORDING TO CHANGES IN PRINTER SETTINGS OR PRINTED PAPER SIZE.

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KLAMATH RIVER RENEWAL PROJECT

KLAMATH WATERSHED BASIN

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA103-640/1	9

FIGURE 1.1 REV 0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	27MAY'22	ISSUED WITH REPORT	ELK	KK	AS

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2.0 KLAMATH RIVER AVERAGE MONTHLY FLOW CONDITIONS

The US Bureau of Reclamation (USBR) stores, diverts, and conveys the waters of the Klamath and Lost Rivers to serve authorized Klamath Irrigation Project (Irrigation Project) purposes. The Bureau is required to meet contractual obligations in compliance with state and federal laws and to carry out the activities necessary to maintain the Irrigation Project and maintain its proper long-term functioning and operation. Biological assessments have been prepared to evaluate the potential effects of the continued operation of the Irrigation Project on species listed as threatened or endangered under the Endangered Species Act (ESA). The biological assessments have been prepared pursuant to Section 7(a)(2) of the ESA of 1973, as amended (16 United States Code [USC.] § 1531 et seq.).

Several Section 7 Consultations and Biological Opinions (BiOp's) have governed the operation of Upper Klamath Lake (UKL) and the Irrigation Project since the 1990's (USBR, 2012). The consultations involve the National Marine Fisheries Service (NMFS), also known as NOAA Fisheries, as well as the US Fish and Wildlife Service (FWS) and the USBR. The USBR currently meets its obligations under the ESA by operating the Irrigation Project in accordance with the latest FWS and NMFS BiOp, dated March 29, 2019. This BiOp is based on information provided in the USBR's Final Biological Assessment (USBR, 2018) and is effective April 1, 2019 through March 31, 2029. The latest BiOp operating conditions will govern the Klamath River during the dam removal and reclamation activities of the KRRP.

The USBR uses results generated by the Water Resources Integrated Modeling System (WRIMS) to identify the Klamath River and Upper Klamath Lake hydrographs that are likely to occur due to implementing the proposed operations across the full range of reasonably foreseeable annual precipitation and hydrologic patterns. WRIMS is a generalized water resources modeling system for evaluating operational alternatives of large, complex river basins. USBR has developed a WRIMS model specific to the Klamath Basin, which is referred to as the Klamath Basin Planning Model (KBPM). The KBPM incorporates the 2019 BiOp operating conditions and models the Klamath River flows. WRIMS is used to estimate mainstem Klamath River flows at the US Geological Survey (USGS) gages located near the Keno and Iron Gate Dam facilities. While the KBPM captures the hydrology under a wide range of plausible conditions, the unique sequencing and patterns of climatological and hydrological events that will occur in the future cannot be predicted.

There are 36 years (October 1980-November 2016) of daily average flows for the Keno and Iron Gate USGS gages as modeled using the KBPM (USBR, 2018). These daily flows were used to calculate the monthly average inflows for each of the four KRRP facilities. The Keno values were prorated by the ratio of the respective drainage areas to generate values for J.C. Boyle. The Iron Gate values were prorated by drainage area to generate values for Copco No. 1 and Copco No. 2. Area proration is a conventional method to determine flows at ungaged locations, particularly for locations on the same river system (Maidment, 1993). The monthly average flows for the four KRRP sites are shown in Table 2.1 and on Figure 2.1 for each facility. In addition to the monthly average flows for the period of record, Figure 2.1 also includes the range of average monthly flows at each facility for the 36 years of BiOp flows used in the KBPM model. Figure 2.2 is an example ensemble plot of daily average flows at the Iron Gate USGS gage on which each line represents a single year (also referred to as a spaghetti plot). This figure overlaps 36 years of BiOp flows on a common x-axis that spans January 1 to December 31, and highlights the variability of maximum daily flows in each month.

Table 2.1 Monthly Average Flows at Project Sites

Facility	Keno[1]	J.C. Boyle[2]	Copco No. 1[2,3]	Iron Gate[1]
Drainage Area (mi^2)	**3,920**	**4,080**	**4,370**	**4,630**
Month	**Monthly Average Flow (cfs)**			
January	1,450	1,500	1,910	2,030
February	1,820	1,900	2,360	2,500
March	2,690	2,800	3,230	3,430
April	2,270	2,370	2,790	2,950
May	1,690	1,760	2,110	2,230
June 1 – 15	1,280	1,330	1,620	1,720
June 16 – 30	920	960	1,210	1,280
July 1 – 15	710	740	990	1,050
July 16 – 31	730	760	990	1,050
August	730	760	980	1,040
September 1 – 15	780	810	1,030	1,090
September 16 – 30	760	790	1,030	1,090
October 1 – 15	780	810	1,050	1,120
October 16 – 31	860	890	1,140	1,210
November 1 – 15	940	980	1,230	1,300
November 16 – 30	910	950	1,240	1,310
December	1,070	1,110	1,490	1,580
Average Annual Flow (cfs)	1,330	1,390	1,710	1,820
Average Annual Unit Flow (cfs/mi^2)	0.34	0.34	0.39	0.39

NOTES:

1. 2019 BIOP FLOWS (USBR, 2018) WERE USED AS THE REPRESENTATIVE INCOMING FLOWS TO THE FACILITY BASED ON THE PERIOD OF RECORD FROM 1980 - 2016.
2. J.C. BOYLE INFLOWS WERE CALCULATED USING THE 2019 BIOP FLOWS AT THE USGS KENO GAGE USING LINEAR AREA PRORATION. COPCO NO. 1 INFLOWS WERE CALCULATED USING THE 2019 BIOP FLOWS AT THE USGS IRON GATE GAGE USING LINEAR AREA PRORATION.
3. MONTHLY AVERAGE INFLOWS AT COPCO NO. 2 ARE ASSUMED TO BE THE SAME AS THE MONTHLY AVERAGE INFLOWS AT COPCO NO. 1.

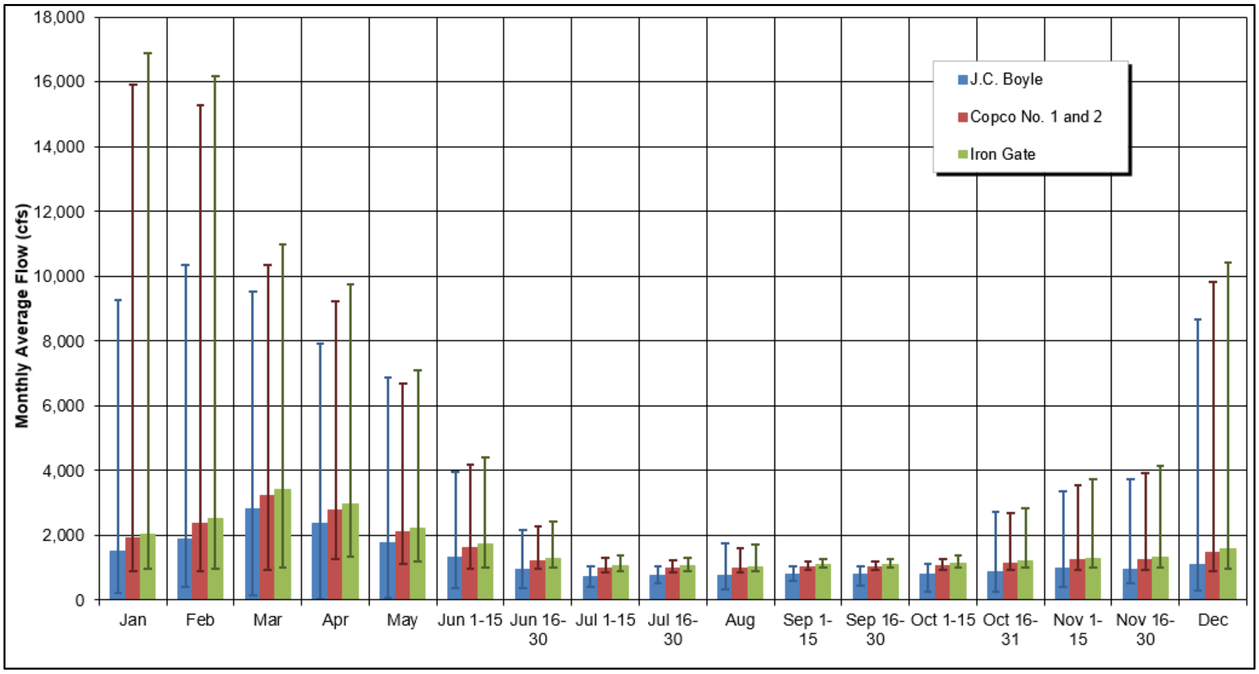

Figure 2.1 Monthly Average BiOp Flows at Project Sites

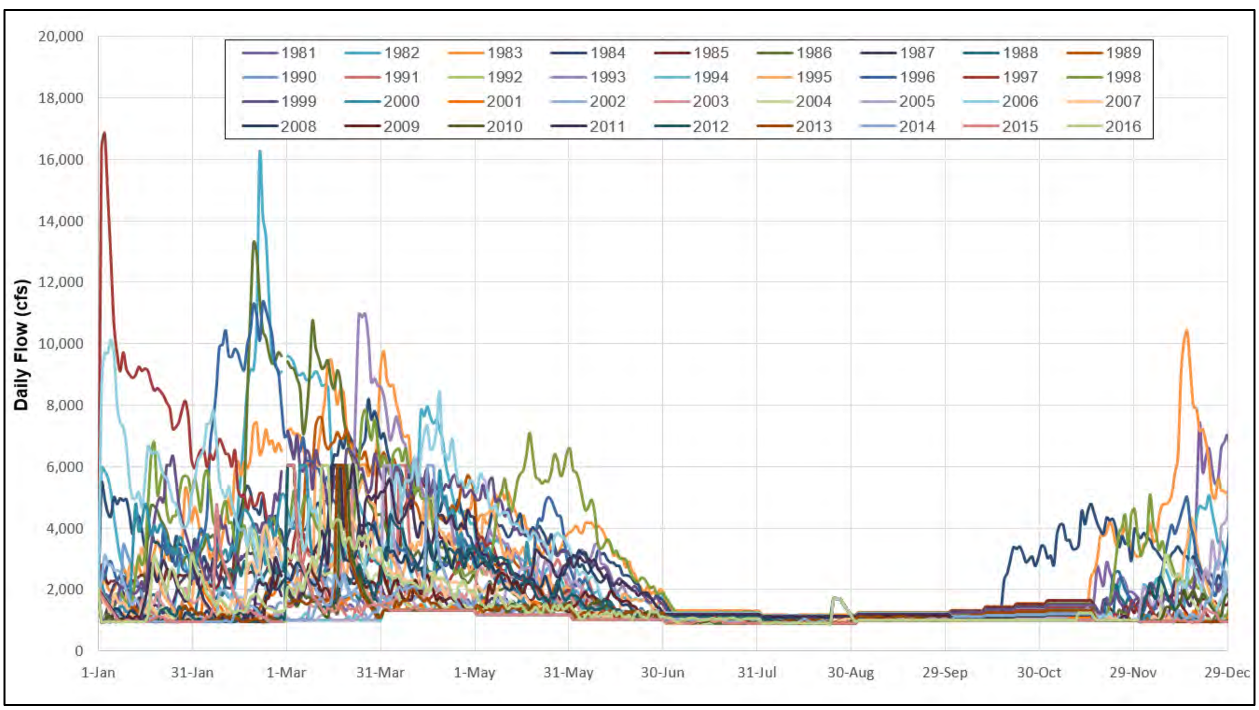

Figure 2.2 Daily Average BiOp Flows at the Iron Gate USGS Gage

The annual patterns of stream flows apparent in the above hydrographs are characterized by the following throughout the Klamath basin:

- High flows in the spring (March and April) due to spring snowmelt runoff (freshet), in the Upper Klamath basin and unregulated tributaries.
- Lower flows in mid-summer to late fall (July through October) due to reduced precipitation during the summer months.
- Increasing flows throughout the winter months (November through February) due to progressively increasing precipitation (which falls as snow in the upper elevations and rain in the lower elevations).

The regulation of Upper Klamath Lake is done with respect to the streamflow patterns seen on Figure 2.1.

- The reservoirs are not designed to mitigate floods and are typically full during the annual peak flows due to the timing of these events and, therefore, attenuation of these storms is limited. During the summer months when the reservoirs have more storage capacity the flood attenuation potential is greater.

The tributary flows contribute high flows during freshet that cannot be mitigated compared to much lower flows during the summer period when flow is mostly from the mainstem. The annual hydrograph on Figure 2.1 indicates that the highest monthly average flows occur in March during spring runoff, but the largest peak flow events generally occur in January and February, as indicated by the maximum range of daily flows shown on Figure 2.2. These peak flows are driven by rain on snow events and govern the annual flood events.

The peak floods at Iron Gate can be substantially greater than the peak floods at J.C. Boyle due to the tributaries that enter the Klamath River between the two facilities. The largest tributary between the Keno and Iron Gate facilities is Jenny Creek which contributes a high amount of flow during the late winter and spring snowmelt months. The hydrology of Jenny Creek is further described in Section 5.1.

3.0 KLAMATH RIVER PEAK FLOODS FOR EXISTING CONDITIONS

3.1 ANNUAL PEAK FLOODS

3.1.1 METHODOLOGY

Various return period design flood estimates, representing existing conditions, are required for design purposes. Peak flood estimates for the Project area were developed using both the historical USGS gage streamflow data and the developed 2019 BiOp flow data (USBR, 2018). Annual peak flows were determined from both datasets and used to estimate the annual return period peak flows. Flood frequency analyses were performed on the annual peak flow data using the HEC-SSP software, following the Bulletin 17B method for Log-Pearson Type III distribution (USGS, 1982). A detailed description of the analyses for each dataset is outlined in the sections below.

3.1.2 HISTORIC USGS GAGE DATA

The USGS operates several stream gages on the Klamath River within proximity of the Project area. The station details of the regional datasets most relevant to the KRRP are provided in Table 3.1 and shown on Figure 1.1.

Table 3.1 USGS Regional Streamflow Gaging Stations

USGS Gaging Station No.	Station Name	Drainage Area (mi²)	Longitude	Latitude	Period of Record
11509500	Klamath River at Keno, OR	3,920	42°08'00"	121°57'40"	1905-1913 1930-2017
11510700	Klamath River below John C. Boyle Power Plant near Keno, OR	4,080	42°05'05"	122°04'20"	1959-2017
11512500	Klamath River below Fall Creek near Copco, CA	4,370	41°58'20"	122°22'05"	1923-1961
11516530	Klamath River below Iron Gate Dam, CA	4,630	41°55'41"	122°26'35"	1960-2017

The annual peak flow data for the USGS gages was imported to the United States Army Corps of Engineers' (USACE) HEC-SSP software (V2.1) and used for the flood frequency analyses. A low flow threshold, below which flows did not fit the distribution, were determined by assessing the flood-frequency curves. The data visually fit within the 95 percent confidence limit of the distribution for all locations except J.C. Boyle. Accordingly, the J.C. Boyle data below 3,400 cfs was identified as low flow outliers and the Bulletin 17B procedures were followed to adjust the flood probabilities to account for these low outliers.

The period used for the peak flow analysis is from 1960 onwards. The USGS records for the J.C. Boyle and Iron Gate Dam gages begin after 1960 and account for the effects of many of the reservoirs within the Klamath River basin. This period also includes the flood of record for the Klamath region, which occurred in December 1964 (water year 1965). Copco No. 1 has a peak flow record for the period of 1923 to 1961, which is outside the selected period of analysis. Accordingly, the return period peak flows for Copco No. 1 were calculated by scaling the flood flows at Iron Gate according to the methodology described in "Estimation of Peak discharges for Rural, Unregulated streams in Western Oregon" (USGS, 2005). This approach, which indicates direct linear scaling with an exponent 1.0, results in conservative flood estimates for Copco No. 1 since the peak floods at Iron Gate are substantially greater than the peak floods at J.C. Boyle due to the tributary flows that enter the Klamath River between the two facilities.

Annual peak flood results using the historical USGS data are presented in Table 3.2.

3.1.3 2019 BIOLOGICAL OPINION DATA

The 2019 BiOp flows (USBR, 2018) are comprised of 36 years (1980-2016) of average daily flows for both the USGS gages at Keno and Iron Gate. The daily flows were converted to instantaneous peak floods using conversion factors that were calculated by comparing the annual maximum instantaneous flows to the corresponding daily flows using data available from the USGS gages located downstream of J.C. Boyle (11510700, Klamath River BLW John C Boyle Powerplant, Nr Keno OR) and downstream of Iron Gate Dam (11516530, Klamath River below Iron Gate Dam, CA). The locations of these gages are shown on Figure 1.1. The comparisons indicate that the annual maximum instantaneous floods are approximately 10% higher than the daily flows for the same day. Conversion factors of 1.10 and 1.12 were used to adjust the available 2019 BiOp daily flows into instantaneous peak floods for the Keno and Iron Gate data, respectively. The instantaneous peak flood data at Keno and Iron Gate were used for the flood frequency analyses.

The J.C. Boyle and the Copco No. 1 annual peak floods were calculated using the area proration methodology described in "Estimation of Peak discharges for Rural, Unregulated streams in Western

Oregon" (USGS, 2005), based on the annual BiOp flood frequency results for the Keno and Iron Gate facilities, respectively. The peak flood results from the Iron Gate facility were used in preference to those at Keno to estimate flood values at the Copco No. 1 facility because the Iron Gate flows demonstrate proportionally greater flood flows than the flows at the upstream facility and therefore better represent the effects of the relatively large peak flow contributions from the mostly unregulated tributary creeks and rivers that inflow between the upstream facility and Copco No. 1.

Annual peak flood results using the 2019 BiOp flow data are presented in Table 3.2.

3.1.4 ANNUAL PEAK FLOOD VALUES FOR DESIGN

The historic USGS data and the 2019 BiOp data were both used to estimate annual return period floods at the Klamath River hydroelectric facilities under existing conditions. The 2019 BiOp operating conditions may change the timing and/or volumes of the Klamath River and, therefore, needed to be included in the peak flood analysis in addition to the historical flows seen at the USGS gages. The 2019 BiOp operating conditions are especially important for the monthly peak floods as these floods are more influenced by the regulation of the Klamath River from the upstream facilities. The flood values selected as the recommended design values are the maximum values between these two datasets, as shown in Table 3.2. The annual return period floods at Copco No. 1 are also used as representative of the annual return period floods for Copco No. 2.

Table 3.2 Annual Peak Floods for Existing Conditions

Location	Drainage Area (mi^2)	Annual Percent Probable Flood (cfs)							
		50%	20%	10%	5%	2%	1%	0.50%	0.20%
Historic USGS Data									
J.C. Boyle	4,080	5,300	8,500	10,300	11,700	13,300	14,200	15,000	15,800
Copco No. 1	4,370	5,600	10,300	14,000	18,200	24,200	29,400	35,000	43,200
Iron Gate	4,630	5,900	10,900	14,900	19,300	25,700	31,200	37,100	45,800
2019 Biological Opinion Data									
J.C. Boyle	4,080	7,000	8,400	9,500	10,400	11,800	12,900	14,100	15,600
Copco No. 1	4,370	7,100	9,400	11,500	14,000	17,800	21,300	25,500	32,100
Iron Gate	4,630	7,500	10,000	12,200	14,800	18,900	22,600	27,000	34,100
Recommended Design Values									
J.C. Boyle	4,080	7,000	8,500	10,300	11,700	13,300	14,200	15,000	15,800
Copco No. 1	4,370	7,100	10,300	14,000	18,200	24,200	29,400	35,000	43,200
Iron Gate	4,630	7,500	10,900	14,900	19,300	25,700	31,200	37,100	45,800

3.1.4.1 ANNUAL FLOWS WITH HIGH PROBABILITY OF EXCEEDANCE

The 2019 BiOp data were used to estimate the annual peak floods at the Klamath River hydroelectric facilities that have high probabilities of exceedance that will occur more frequently. These values were determined as per the methodology described in Section 3.1.1 and are summarized in Table 3.3. The annual percent probable floods at Copco No. 1 are used as representative of the annual percent probable floods for Copco No. 2.

Table 3.3 Flows with High Probabilities of Exceedance

Location	Drainage Area (mi²)	Annual Percent Probable Flood (cfs)		
		99.9%	80.0%	66.7%
J.C. Boyle[1]	4,080	4,600	5,900	6,400
Copco No. 1[2]	4,370	5,200	5,900	6,400
Iron Gate	4,630	5,500	6,300	6,800

NOTES:

1. CALCULATED BASED ON KENO RESULTS (USING 2019 BIOP FLOWS) USING METHODOLOGY DESCRIBED IN "ESTIMATION OF PEAK DISCHARGES FOR RURAL, UNREGULATED STREAMS IN WESTERN OREGON" (USGS, 2005).
2. CALCULATED BASED ON IRON GATE RESULTS (USING 2019 BIOP FLOWS) USING METHODOLOGY DESCRIBED IN "ESTIMATION OF PEAK DISCHARGES FOR RURAL, UNREGULATED STREAMS IN WESTERN OREGON" (USGS, 2005).

3.2 PEAK FLOODS FOR MONTHLY TIME PERIODS

3.2.1 GENERAL

A flood frequency analysis was performed for monthly periods to better define the risk of flooding events occurring during the dam removal period. The flood frequency analysis used to determine monthly return period peak flows was the same as that used for the annual return period flows, as described in previous sections. The data indicate that the areal extent of freshet snowmelt contributing to peak flows diminishes greatly in the second half of June, and therefore the month of June was divided into two periods for peak flood analysis purposes: June 1 to June 15 and June 16 to June 30. Additional months that were subdivided into two periods include July, September, October, and November. These months were subdivided to support the proposed construction schedule.

3.2.2 HISTORIC USGS GAGE DATA

Daily data for the USGS stations (J.C. Boyle and Iron Gate Dam, Table 3.1) were used to calculate the monthly peak floods. Daily discharge data from January 1960 up until the most recent data available were used for the monthly flood frequency analyses.

The Iron Gate data source was USGS station 11516530. The J.C. Boyle data source was USGS station 11510770 and flows below 3400 cfs were treated as low flow outliers due to the influence of upstream activity. The daily flows of both datasets were converted to equivalent instantaneous 24-hr floods using the conversion factors developed for each site during the annual flood frequency analysis, as discussed above. It is recognized that the instantaneous to daily ratios would tend to vary monthly depending on the source of the flood flows and the amount of upstream flow regulation, but the regulation from upstream reservoirs would tend to limit the size of the ratios to less than the annual peak ratios, so use of annual ratios results in reasonably conservative instantaneous peak flow estimates.

A flood frequency analysis was performed on the monthly peak flows using the HEC-SSP software (V2.1), following the Bulletin 17B method for Log-Pearson Type III distributions (USGS, 1982). The monthly peak floods for Copco No. 1 were calculated using non-linear proration with calculated Iron Gate monthly peak values using the methodology described in “Estimation of Peak Discharges for Rural, Unregulated Streams in Western Oregon” (USGS 2005). Table 3.4 provides the flood frequency results for the specified time periods.

The historic USGS flows are regulated flows and are influenced by the operation of the reservoirs on the Klamath River. This regulation makes it possible for some monthly peak flows to be higher at J.C. Boyle than at Iron Gate.

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TABLE 3.4

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PEAK FLOODS FOR SPECIFIED TIME PERIOD USING HISTORIC USGS GAGE DATA

Location	Drainage Area (mi²)	Month	Instantaneous Peak Floods for Specified Time Period (cfs)							
			50% Probable Flood	20% Probable Flood	10% Probable Flood	5% Probable Flood	2% Probable Flood	1% Probable Flood	0.5% Probable Flood	0.2% Probable Flood
		Jan	2,600	4,400	6,000	8,000	11,100	14,000	15,000	15,800
		Feb	2,700	4,900	6,900	9,200	13,000	14,200	15,000	15,800
		Mar	3,500	6,300	8,500	10,900	13,300	14,200	15,000	15,800
		Apr	3,400	5,700	7,400	9,200	11,600	13,600	15,000	15,800
		May	2,600	4,300	5,500	6,800	8,500	9,900	11,300	13,400
		Jun 1 - 15	1,500	2,400	3,200	4,200	5,800	7,300	9,100	12,100
		Jun 16 - 30	1,200	1,700	2,200	2,700	3,400	4,100	4,800	5,900
		Jul 1 - 15	1,000	1,400	1,700	2,100	2,700	3,200	3,900	4,900
J.C. Boyle[1]	4,080	Jul 16 - 31	1,000	1,200	1,400	1,500	1,600	1,700	1,800	2,000
		Aug	1,400	1,500	1,600	1,700	1,800	1,800	1,800	1,900
		Sep 1 - 15	1,400	1,700	1,900	2,100	2,400	2,500	2,700	3,000
		Sep 16 - 30	1,500	1,900	2,200	2,400	2,800	3,000	3,200	3,500
		Oct 1 - 15	1,700	2,200	2,500	2,900	3,400	3,800	4,200	4,700
		Oct 16 - 31	1,700	2,400	2,800	3,300	4,000	4,600	5,200	6,100
		Nov 1 - 15	1,800	2,600	3,200	3,800	4,700	5,500	6,300	7,500
		Nov 16 - 30	2,000	2,900	3,600	4,400	5,400	6,300	7,200	8,500
		Dec	2,500	3,900	5,100	6,300	8,200	9,900	11,700	14,400
		Jan	3,000	5,800	8,400	11,800	17,600	23,400	30,500	42,800
		Feb	3,000	5,800	8,400	11,800	17,600	23,400	30,500	42,800
		Mar	4,100	7,400	10,200	13,000	17,100	20,500	23,900	29,000
		Apr	3,600	6,500	8,900	11,100	14,400	17,000	19,700	23,400
		May	2,600	4,500	5,900	7,400	9,400	11,000	12,700	15,100
		Jun 1 - 15	1,500	2,500	3,400	4,500	6,400	8,200	10,500	14,100
		Jun 16 - 30	1,200	1,800	2,200	2,700	3,500	4,100	4,900	6,100
		Jul 1 - 15	900	1,200	1,600	2,000	2,600	3,200	4,100	5,300
Copco No. 1[2]	4,370	Jul 16 - 31	900	1,000	1,100	1,200	1,300	1,400	1,500	1,600
		Aug	1,100	1,300	1,500	1,600	1,800	2,000	2,100	2,400
		Sep 1 - 15	1,300	1,600	1,800	1,900	2,100	2,200	2,300	2,500
		Sep 16 - 30	1,300	1,600	1,900	2,100	2,400	2,500	2,700	3,000
		Oct 1 - 15	1,500	2,000	2,500	2,900	3,700	4,300	5,100	6,200
		Oct 16 - 31	1,500	2,200	2,700	3,300	4,200	5,100	6,000	7,500
		Nov 1 - 15	1,700	2,500	3,300	4,100	5,400	6,600	7,900	10,000
		Nov 16 - 30	1,900	3,000	4,000	4,900	6,500	7,800	9,300	11,700
		Dec	2,500	5,000	7,400	10,700	16,600	22,600	30,500	43,200
		Jan	3,200	6,100	8,900	12,500	18,700	24,800	32,400	45,400
		Feb	3,200	6,100	8,900	12,500	18,700	24,800	32,400	45,400
		Mar	4,300	7,900	10,800	13,800	18,100	21,700	25,400	30,800
		Apr	3,800	6,900	9,400	11,800	15,300	18,000	20,900	24,800
		May	2,800	4,800	6,300	7,900	10,000	11,700	13,500	16,000
		Jun 1 - 15	1,600	2,600	3,600	4,800	6,800	8,700	11,100	15,000
		Jun 16 - 30	1,300	1,900	2,300	2,900	3,700	4,400	5,200	6,500
		Jul 1 - 15	1,000	1,300	1,700	2,100	2,800	3,400	4,300	5,600
Iron Gate[3]	4,630	Jul 16 - 31	1,000	1,100	1,200	1,300	1,400	1,500	1,600	1,700
		Aug	1,200	1,400	1,600	1,700	1,900	2,100	2,200	2,500
		Sep 1 - 15	1,400	1,700	1,900	2,000	2,200	2,300	2,400	2,600
		Sep 16 - 30	1,400	1,700	2,000	2,200	2,500	2,700	2,900	3,200
		Oct 1 - 15	1,600	2,100	2,600	3,100	3,900	4,600	5,400	6,600
		Oct 16 - 31	1,600	2,300	2,900	3,500	4,500	5,400	6,400	8,000
		Nov 1 - 15	1,800	2,700	3,500	4,400	5,700	7,000	8,400	10,600
		Nov 16 - 30	2,000	3,200	4,200	5,200	6,900	8,300	9,900	12,400
		Dec	2,700	5,300	7,900	11,300	17,600	24,000	32,400	45,800

M:\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\2_Flood Frequency Analysis\[Flood Frequency Analysis - Monthly.xlsm]Table - Monthly_USGS_b

NOTES:

1. DATA SOURCE USGS STATION 11510770 "KLAMATH RIVER BLW JOHN C.BOYLE PWRPLNT, NR KENO,OR", PERIOD OF RECORD 1959 TO 2019. PERIOD OF RECORD USED IN ANALYSIS 1960 TO 2019 TO COINCIDE WITH THE IRON GATE PERIOD OF RECORD. FLOWS BELOW 3,400 cfs WERE CENSORED LOW FLOW OUTLIERS DUE TO THE INFLUENCE OF UPSTREAM DAM ACTIVITIES.
2. CALCULATED USING NON-LINEAR PRORATION WITH IRON GATE USING METHODOLOGY DESCRIBED IN "ESTIMATION OF PEAK DISCHARGES FOR RURAL, UNREGULATED STREAMS IN WESTERN OREGON" (USGS, 2005).
3. DATA SOURCE USGS STATION 11516530 "KLAMATH R BL IRON GATE DAM CA", PERIOD OF RECORD 1960 TO 2019. PERIOD OF RECORD USED IN ANALYSIS 1960 TO 2019.
4. ANALYSIS USES HISTORIC USGS GAGE DATA. THESE FLOWS ARE INFLUENCED BY THE OPERATION OF THE RESERVOIRS ON THE KLAMATH RIVER AND ARE, THEREFORE, REGULATED. THE REGULATION MAKES IT POSSIBLE FOR PEAK FLOWS TO BE HIGHER AT J.C. BOYLE THAN AT IRON GATE.
5. THE DATA INDICATE THAT FOR SOME MONTHS THERE IS A TRANSITION IN THE HYDROLOGY IN THE MIDDLE OF THE MONTH. MONTHS WHEN THIS OCCURS INCLUDE JUNE, JULY, SEPTEMBER, OCTOBER, AND NOVEMBER. FOR ANALYSIS PURPOSES THESE MONTHS HAVE BEEN DIVIDED INTO TWO PERIODS: 1st TO 15th AND 16th TO 30th/31st OF EACH MONTH.

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3.2.3 2019 BIOLOGICAL OPINION DATA

The 2019 BiOp daily flows for the Keno and Iron Gate facilities were used to estimate the monthly peak floods for the KRRP hydroelectric facilities. The peak daily flow in each specified period was determined and converted to an instantaneous peak flow using the conversion factor of 1.10. A flood frequency analysis was performed on these peak floods using HEC-SSP (V2.1), following the Bulletin 17B method for Log-Pearson Type III distributions (USGS, 1982).

The peak floods for specified time periods at J.C. Boyle and Copco No. 1 were calculated using the methodology described in USGS (2005), based on the results for the Keno and Iron Gate facilities, respectively. The return period floods for specified periods at Copco No. 1 are used as representative for Copco No. 2. Table 3.5 provides the flood frequency results for the specified time periods.

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TABLE 3.5

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

PEAK FLOODS FOR SPECIFIED TIME PERIOD USING 2019 BIOLOGICAL OPINION DATA[1]

Location	Drainage Area (mi²)	Month	Instantaneous Peak Floods for Specified Time Period (cfs)							
			50% Probable Flood	20% Probable Flood	10% Probable Flood	5% Probable Flood	2% Probable Flood	1% Probable Flood	0.5% Probable Flood	0.2% Probable Flood
		Jan	2,000	3,700	5,400	7,400	10,600	13,700	17,400	23,500
		Feb	2,200	4,500	6,700	9,300	13,700	18,000	23,100	31,600
		Mar	6,000	7,700	8,400	8,900	9,200	9,400	9,500	9,600
		Apr	4,300	6,500	7,800	9,000	10,500	11,500	12,500	13,700
		May	2,700	4,000	4,800	5,600	6,600	7,300	7,900	8,800
		Jun 1 - 15	1,800	2,800	3,500	4,200	5,300	6,100	7,100	8,400
		Jun 16 - 30	1,300	1,800	2,200	2,700	3,600	4,400	5,300	6,800
		Jul 1 - 15	900	1,100	1,200	1,200	1,300	1,400	1,500	1,600
Keno[2]	3,920	Jul 16 - 31	900	1,000	1,100	1,200	1,300	1,400	1,400	1,500
		Aug	1,000	1,200	1,200	1,300	1,400	1,400	1,500	1,600
		Sep 1 - 15	1,000	1,100	1,100	1,200	1,200	1,300	1,300	1,400
		Sep 16 - 30	1,000	1,100	1,100	1,200	1,300	1,300	1,400	1,400
		Oct 1 - 15	1,000	1,100	1,200	1,300	1,400	1,400	1,500	1,600
		Oct 16 - 31	1,000	1,200	1,400	1,700	2,400	3,000	3,900	5,400
		Nov 1 - 15	1,000	1,400	1,800	2,300	3,400	4,400	5,800	8,500
		Nov 16 - 30	1,100	1,800	2,500	3,500	5,300	7,200	9,700	14,400
		Dec	1,800	3,200	4,400	5,800	8,000	10,100	12,500	16,300
		Jan	2,100	3,900	5,600	7,700	11,000	12,900	14,100	15,600
		Feb	2,300	4,700	7,000	9,700	11,800	12,900	14,100	15,600
		Mar	6,300	8,000	8,800	9,300	9,600	9,800	9,900	10,000
		Apr	4,500	6,800	8,100	9,400	10,900	12,000	13,000	14,300
		May	2,700	4,200	5,000	5,800	6,900	7,600	8,200	9,200
		Jun 1 - 15	1,800	2,800	3,500	4,400	5,500	6,400	7,400	8,800
		Jun 16 - 30	1,400	1,800	2,300	2,800	3,600	4,400	5,000	6,300
		Jul 1 - 15	900	1,100	1,300	1,300	1,400	1,500	1,600	1,700
J.C. Boyle[3]	4,080	Jul 16 - 31	900	1,000	1,100	1,200	1,300	1,300	1,300	1,400
		Aug	1,000	1,200	1,200	1,400	1,500	1,500	1,600	1,700
		Sep 1 - 15	1,000	1,100	1,000	1,200	1,200	1,400	1,400	1,500
		Sep 16 - 30	1,000	1,100	1,000	1,200	1,300	1,400	1,500	1,500
		Oct 1 - 15	1,000	1,100	1,300	1,400	1,500	1,500	1,600	1,700
		Oct 16 - 31	1,000	1,200	1,500	1,800	2,500	3,100	3,900	5,300
		Nov 1 - 15	1,000	1,400	1,800	2,300	3,400	4,400	5,900	8,600
		Nov 16 - 30	1,100	1,900	2,600	3,600	5,300	7,200	9,600	14,000
		Dec	1,900	3,300	4,600	6,000	8,300	10,500	13,000	15,600
		Jan	2,400	4,500	6,800	9,600	14,600	19,700	25,500	32,100
		Feb	2,900	5,800	8,500	11,800	17,400	21,300	25,500	32,100
		Mar	6,500	8,500	9,200	9,800	10,200	10,400	10,600	10,700
		Apr	4,600	6,900	8,500	10,000	11,900	13,200	14,500	16,100
		May	2,900	4,300	5,400	6,400	7,900	9,000	10,300	11,900
		Jun 1 - 15	1,900	2,900	3,700	4,500	5,600	6,600	7,700	9,400
		Jun 16 - 30	1,400	1,900	2,400	2,900	3,600	4,400	5,100	6,400
		Jul 1 - 15	1,100	1,300	1,500	1,600	1,800	2,000	2,200	2,500
Copco No. 1[4]	4,370	Jul 16 - 31	1,100	1,200	1,300	1,300	1,400	1,400	1,400	1,500
		Aug	1,200	1,300	1,300	1,400	1,400	1,400	1,500	1,500
		Sep 1 - 15	1,100	1,200	1,300	1,300	1,400	1,400	1,400	1,500
		Sep 16 - 30	1,100	1,200	1,300	1,300	1,400	1,400	1,400	1,500
		Oct 1 - 15	1,200	1,300	1,400	1,500	1,600	1,600	1,700	1,800
		Oct 16 - 31	1,100	1,400	1,600	2,000	2,600	3,200	4,000	5,400
		Nov 1 - 15	1,200	1,500	1,900	2,400	3,500	4,500	6,000	8,700
		Nov 16 - 30	1,300	2,000	2,700	3,700	5,400	7,200	9,700	14,000
		Dec	2,000	3,800	5,700	8,100	12,400	17,100	22,900	32,100
		Jan	2,500	4,800	7,200	10,200	15,500	20,900	27,000	34,100
		Feb	3,100	6,100	9,000	12,500	18,500	22,600	27,000	34,100
		Mar	6,900	9,000	9,800	10,400	10,800	11,000	11,200	11,300
		Apr	4,800	7,300	9,000	10,600	12,600	14,000	15,400	17,100
		May	3,000	4,600	5,700	6,800	8,400	9,600	10,900	12,600
		Jun 1 - 15	2,000	3,000	3,800	4,600	5,900	7,000	8,200	10,000
		Jun 16 - 30	1,500	2,000	2,500	3,000	3,700	4,400	5,200	6,500
		Jul 1 - 15	1,200	1,400	1,600	1,700	1,900	2,100	2,300	2,600
Iron Gate[5]	4,630	Jul 16 - 31	1,200	1,300	1,400	1,400	1,500	1,500	1,500	1,600
		Aug	1,300	1,400	1,400	1,500	1,500	1,500	1,600	1,600
		Sep 1 - 15	1,200	1,300	1,400	1,400	1,500	1,500	1,500	1,600
		Sep 16 - 30	1,200	1,300	1,400	1,400	1,500	1,500	1,500	1,600
		Oct 1 - 15	1,300	1,400	1,500	1,600	1,700	1,700	1,800	1,900
		Oct 16 - 31	1,200	1,500	1,700	2,100	2,700	3,300	4,100	5,500
		Nov 1 - 15	1,300	1,600	2,000	2,500	3,600	4,600	6,100	8,800
		Nov 16 - 30	1,400	2,100	2,900	3,800	5,500	7,300	9,800	14,000
		Dec	2,100	4,000	6,000	8,600	13,200	18,100	24,300	34,100

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NOTES:

1. 2019 BIOLOGICAL OPINION FLOWS (USBR, 2018) WERE PROVIDED FOR THE PERIOD FROM 1981 TO 2016. FLOWS WERE PROVIDED AT KENO (USGS GAGE 11509500) AND IRON GATE (USGS GAGE 11516530).
2. CALCULATED USING 2019 BIOP FLOWS AT KENO. A FACTOR OF 1.10 WAS APPLIED TO ADJUST DAILY AVERAGE FLOW TO DAILY PEAK FLOW.
3. CALCULATED USING NON-LINEAR AREA PRORATION WITH 2019 BIOP FLOWS AT KENO USING METHODOLOGY DESCRIBED IN "ESTIMATION OF PEAK DISCHARGES FOR RURAL, UNREGULATED STREAMS IN WESTERN OREGON" (USGS, 2005).
4. CALCULATED USING NON-LINEAR AREA PRORATION WITH 2019 BIOP FLOWS AT IRON GATE USING METHODOLOGY DESCRIBED IN "ESTIMATION OF PEAK DISCHARGES FOR RURAL, UNREGULATED STREAMS IN WESTERN OREGON" (USGS, 2005).
5. CALCULATED USING 2019 BIOP FLOWS AT IRON GATE. A FACTOR OF 1.12 WAS APPLIED TO ADJUST DAILY AVERAGE FLOW TO DAILY PEAK FLOW.
6. THE DATA INDICATE THAT FOR SOME MONTHS THERE IS A TRANSITION IN THE HYDROLOGY IN THE MIDDLE OF THE MONTH. MONTHS WHEN THIS OCCURS INCLUDE JUNE, JULY, SEPTEMBER, OCTOBER, AND NOVEMBER. FOR ANALYSIS PURPOSES THESE MONTHS HAVE BEEN DIVIDED INTO TWO PERIODS: 1st TO 15th AND 16th TO 30th/31st OF EACH MONTH.
7. THE CEREMONIAL FLOW RELEASES FOR THE YUROK BOAT DANCE CEREMONY WILL BE DEFERRED FOR THE DRAWDOWN YEAR. THESE FLOWS HAVE, THEREFORE, BEEN REMOVED FROM THE DATASET.

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3.2.4 MONTHLY PEAK FLOOD RESULTS

The Historic USGS data and 2019 BiOp data were both used to determine the monthly peak floods at the Klamath River reservoirs under existing conditions. The flood values selected as the recommended design values are the maximum calculated values, as shown in Table 3.6 for J.C. Boyle, Copco No. 1 and Iron Gate. An example visual interpretation of Table 3.6 for selected time periods is shown for Iron Gate on Figure 3.1. The monthly return period floods at Copco No. 1 are used as representative of the monthly return period floods for Copco No. 2.

The results show that for all facilities the peak floods for specified time periods decrease from April through to August. The peak flood results then increase from September through to March.

When considering the application of the monthly peak floods in relation to deconstruction activities near the river or reservoirs, embankment dam removal periods, or instream works, the designer/contractor should carefully consider the flows, water levels, and risk levels associated with the probable flood events in the time period that the work will take place or the time period that the structure will remain in place.

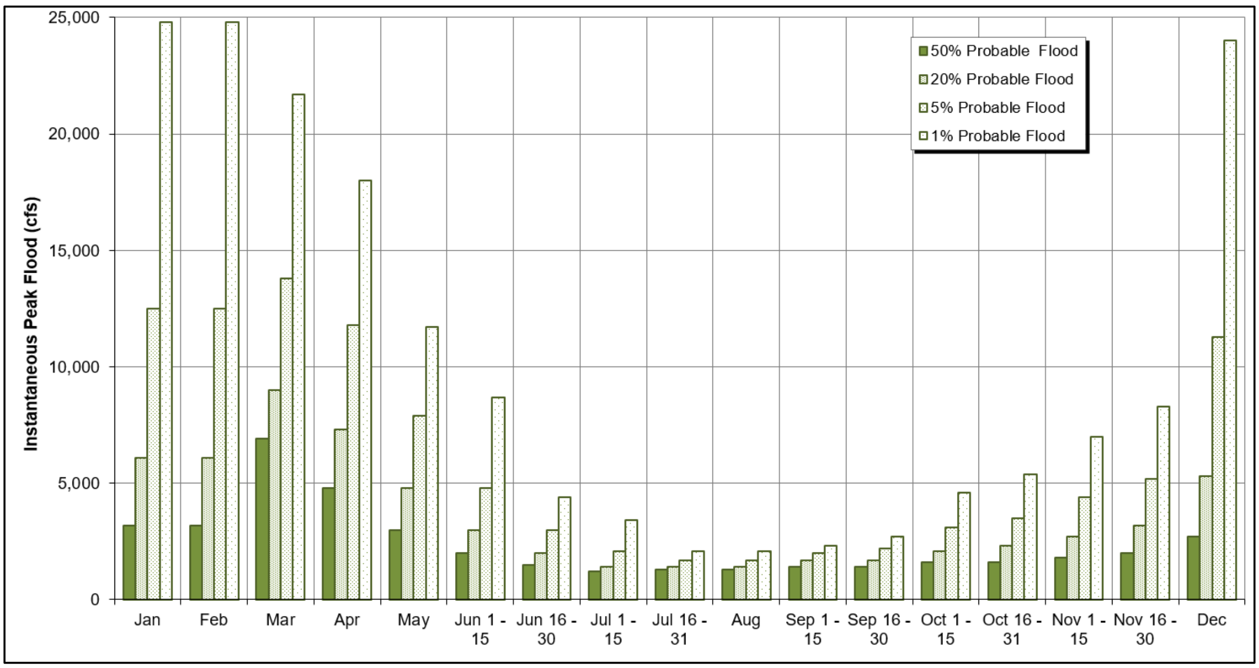

Figure 3.1 Iron Gate Peak Floods per Specified Time Period

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TABLE 3.6

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

RECOMMENDED DESIGN VALUES OF MONTHLY PEAK FLOODS

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Location	Drainage Area (mi^2)	Month	Instantaneous Peak Floods for Specified Time Period (cfs)								Average Monthly Flow (cfs)
			50% Probable Flood	20% Probable Flood	10% Probable Flood	5% Probable Flood	2% Probable Flood	1% Probable Flood	0.5% Probable Flood	0.2% Probable Flood	
		Jan	2,600	4,400	6,000	8,000	11,100	14,000	15,000	15,800	1,500
		Feb	2,700	4,900	7,000	9,700	13,000	14,200	15,000	15,800	1,900
		Mar	6,300	8,000	8,800	10,900	13,300	14,200	15,000	15,800	2,800
		Apr	4,500	6,800	8,100	9,400	11,600	13,600	15,000	15,800	2,370
		May	2,700	4,300	5,500	6,800	8,500	9,900	11,300	13,400	1,760
		Jun 1 - 15	1,800	2,800	3,500	4,400	5,800	7,300	9,100	12,100	1,330
		Jun 16 - 30	1,400	1,800	2,300	2,800	3,600	4,400	5,000	6,300	960
		Jul 1 - 15	1,000	1,400	1,700	2,100	2,700	3,200	3,900	4,900	740
J.C. Boyle	4,080	Jul 16 - 31	1,400	1,500	1,600	1,700	1,800	1,800	1,800	2,000	760
		Aug	1,400	1,500	1,600	1,700	1,800	1,800	1,800	1,900	760
		Sep 1 - 15	1,400	1,700	1,900	2,100	2,400	2,500	2,700	3,000	810
		Sep 16 - 30	1,500	1,900	2,200	2,400	2,800	3,000	3,200	3,500	790
		Oct 1 - 15	1,700	2,200	2,500	2,900	3,400	3,800	4,200	4,700	810
		Oct 16 - 31	1,700	2,400	2,800	3,300	4,000	4,600	5,200	6,100	890
		Nov 1 - 15	1,800	2,600	3,200	3,800	4,700	5,500	6,300	8,600	980
		Nov 16 - 30	2,000	2,900	3,600	4,400	5,400	7,200	9,600	14,000	950
		Dec	2,500	3,900	5,100	6,300	8,300	10,500	13,000	15,600	1,110
		Jan	3,000	5,800	8,400	11,800	17,600	23,400	30,500	42,800	1,910
		Feb	3,000	5,800	8,500	11,800	17,600	23,400	30,500	42,800	2,360
		Mar	6,500	8,500	10,200	13,000	17,100	20,500	23,900	29,000	3,230
		Apr	4,600	6,900	8,900	11,100	14,400	17,000	19,700	23,400	2,790
		May	2,900	4,500	5,900	7,400	9,400	11,000	12,700	15,100	2,110
		Jun 1 - 15	1,900	2,900	3,700	4,500	6,400	8,200	10,500	14,100	1,620
		Jun 16 - 30	1,400	1,900	2,400	2,900	3,600	4,400	5,100	6,400	1,210
		Jul 1 - 15	1,100	1,300	1,600	2,000	2,600	3,200	4,100	5,300	990
Copco No. 1	4,370	Jul 16 - 31	1,200	1,300	1,500	1,600	1,800	2,000	2,100	2,400	990
		Aug	1,200	1,300	1,500	1,600	1,800	2,000	2,100	2,400	980
		Sep 1 - 15	1,300	1,600	1,800	1,900	2,100	2,200	2,300	2,500	1,030
		Sep 16 - 30	1,300	1,600	1,900	2,100	2,400	2,500	2,700	3,000	1,030
		Oct 1 - 15	1,500	2,000	2,500	2,900	3,700	4,300	5,100	6,200	1,050
		Oct 16 - 31	1,500	2,200	2,700	3,300	4,200	5,100	6,000	7,500	1,140
		Nov 1 - 15	1,700	2,500	3,300	4,100	5,400	6,600	7,900	10,000	1,230
		Nov 16 - 30	1,900	3,000	4,000	4,900	6,500	7,800	9,700	14,000	1,240
		Dec	2,500	5,000	7,400	10,700	16,600	22,600	30,500	43,200	1,490
		Jan	3,200	6,100	8,900	12,500	18,700	24,800	32,400	45,400	2,030
		Feb	3,200	6,100	9,000	12,500	18,700	24,800	32,400	45,400	2,500
		Mar	6,900	9,000	10,800	13,800	18,100	21,700	25,400	30,800	3,430
		Apr	4,800	7,300	9,400	11,800	15,300	18,000	20,900	24,800	2,950
		May	3,000	4,800	6,300	7,900	10,000	11,700	13,500	16,000	2,230
		Jun 1 - 15	2,000	3,000	3,800	4,800	6,800	8,700	11,100	15,000	1,720
		Jun 16 - 30	1,500	2,000	2,500	3,000	3,700	4,400	5,200	6,500	1,280
		Jul 1 - 15	1,200	1,400	1,700	2,100	2,800	3,400	4,300	5,600	1,050
Iron Gate	4,630	Jul 16 - 31	1,300	1,400	1,600	1,700	1,900	2,100	2,200	2,500	1,050
		Aug	1,300	1,400	1,600	1,700	1,900	2,100	2,200	2,500	1,040
		Sep 1 - 15	1,400	1,700	1,900	2,000	2,200	2,300	2,400	2,600	1,090
		Sep 16 - 30	1,400	1,700	2,000	2,200	2,500	2,700	2,900	3,200	1,090
		Oct 1 - 15	1,600	2,100	2,600	3,100	3,900	4,600	5,400	6,600	1,120
		Oct 16 - 31	1,600	2,300	2,900	3,500	4,500	5,400	6,400	8,000	1,210
		Nov 1 - 15	1,800	2,700	3,500	4,400	5,700	7,000	8,400	10,600	1,300
		Nov 16 - 30	2,000	3,200	4,200	5,200	6,900	8,300	9,900	14,000	1,310
		Dec	2,700	5,300	7,900	11,300	17,600	24,000	32,400	45,800	1,580

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NOTES:

1. RECOMMENDED DESIGN VALUES ARE BASED ON THE MAXIMUM VALUES BETWEEN THE ANALYSIS COMPLETED USING THE HISTORIC USGS GAGE DATA AND THE 2019 BIOP FLOW DATA.
2. HISTORIC USGS DATA SOURCE FOR ANALYSIS: USGS STATION 11516530 "KLAMATH R BL IRON GATE DAM CA", PERIOD OF RECORD 1960 TO 2019. PERIOD OF RECORD USED IN ANALYSIS 1960 TO 2019.
3. 2019 BIOP FLOW DATA SOURCE FOR ANALYSIS: 2019 BIOLOGICAL OPINION FLOWS (USBR, 2018) PROVIDED FOR THE PERIOD 1981 TO 2016. FLOWS WERE PROVIDED AT IRON GATE (USGS GAGE 11516530).
4. THE DATA INDICATE THAT FOR SOME MONTHS THERE IS A TRANSITION IN THE HYDROLOGY IN THE MIDDLE OF THE MONTH. MONTHS WHEN THIS OCCURS INCLUDE JUNE, JULY, SEPTEMBER, OCTOBER, AND NOVEMBER. FOR ANALYSIS PURPOSES THESE MONTHS HAVE BEEN DIVIDED INTO TWO PERIODS: 1st TO 15th AND 16th TO 30th/31st OF EACH MONTH.
5. RECOMMENDED DESIGN VALUES FOR THE SECOND HALF OF JULY ARE DICTATED BY THE AUGUST PEAK MONTHLY FLOOD VALUES FOR DAM SAFETY PURPOSES.

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4.0 KLAMATH RIVER ANNUAL DAILY FLOW DURATION

Daily flow duration curves show the percentage of time that a flow is likely to equal or exceed a specified value on an annual or monthly basis. The flow duration curves for the KRRP hydroelectric facilities were created with the following inputs:

- Developed using the 2019 Biological Opinion Flows (2019 BiOps) provided by USBR (2018).
- 2019 BiOps for USGS gage 11509500 Klamath River at Keno, OR were translated to the J.C. Boyle facility using linear area proration.
- 2019 BiOps for USGS gage 11516530 Klamath River below Iron Gate Dam, CA were translated to the Copco No. 1 facility using linear area proration. The flows for the Copco No. 1 facility were also used for the Copco No. 2 facility.

The annual and monthly daily flow duration curves based on the 2019 BiOp flows are shown below in Tables 4.1 to 4.4 and on Figures 4.1 to 4.3 for the KRRP facilities.

Table 4.1 Flow Duration Flows Based on 2019 BiOp Flows – Annual

% of Time Equaled or Exceeded	Discharge (cfs)			
	Keno	J.C. Boyle	Copco No. 1	Iron Gate Dam
99%	300	320	850	900
95%	500	530	850	900
90%	570	590	900	950
80%	640	660	940	1,000
75%	660	690	940	1,000
70%	690	720	970	1,030
60%	760	790	1,050	1,110
50%	820	860	1,110	1,180
40%	920	950	1,250	1,320
30%	1,130	1,170	1,540	1,630
25%	1,400	1,460	1,780	1,880
20%	1,770	1,840	2,210	2,340
10%	2,860	2,980	3,430	3,630
5%	4,140	4,310	4,780	5,060
1%	6,680	6,960	7,630	8,080

Table 4.2 Flow Duration Flows Based on 2019 BiOp Flows – Monthly – J.C. Boyle

% of Time Equaled or Exceeded	Discharge (cfs)																	
	Monthly																	Annual
	Jan	Feb	Mar	Apr	May	Jun 1 - 15	Jun 16 - 30	Jul 1 - 15	Jul 16 - 31	Aug	Sep 1 - 15	Sep 16 - 30	Oct 1 - 15	Oct 16 - 31	Nov 1 - 15	Nov 16 - 30	Dec	
99%	360	440	290	230	190	210	250	380	500	460	570	410	240	240	300	510	420	320
95%	470	510	550	770	740	620	590	530	580	550	660	560	610	580	620	560	490	530
90%	520	540	690	890	860	740	660	590	610	590	690	660	690	670	680	590	520	590
80%	580	600	1,060	1,000	940	800	710	640	650	620	720	690	740	740	760	630	570	660
75%	600	630	1,220	1,040	980	820	730	660	670	630	730	710	750	760	790	650	590	720
70%	620	650	1,440	1,120	1,030	860	750	670	690	650	750	720	770	780	820	660	610	720
60%	660	720	1,800	1,450	1,140	940	780	700	730	670	770	760	790	820	870	680	650	790
50%	720	940	2,220	1,870	1,410	1,030	820	740	760	700	800	790	810	850	910	710	680	860
40%	970	1,580	2,650	2,330	1,720	1,170	890	760	790	740	830	810	840	880	940	740	740	950
30%	1,530	2,220	3,350	2,840	2,110	1,440	970	790	820	790	860	860	870	930	990	780	970	1,170
25%	1,850	2,540	3,880	3,390	2,330	1,670	1,020	810	830	810	880	880	890	950	1,030	820	1,240	1,840
20%	2,160	2,980	4,770	3,790	2,530	1,950	1,080	840	850	850	910	900	910	980	1,090	910	1,530	1,840
10%	3,500	4,320	5,840	4,920	3,180	2,490	1,520	900	930	1,000	950	950	1,000	1,120	1,250	1,560	2,350	2,980
5%	4,870	6,010	6,660	5,670	3,870	2,910	1,830	960	980	1,360	1,010	980	1,060	1,220	1,370	3,090	3,250	4,310
1%	8,280	8,880	8,560	6,860	5,290	4,350	2,580	1,120	1,060	1,560	1,070	1,060	1,170	3,090	3,630	3,970	5,640	6,960

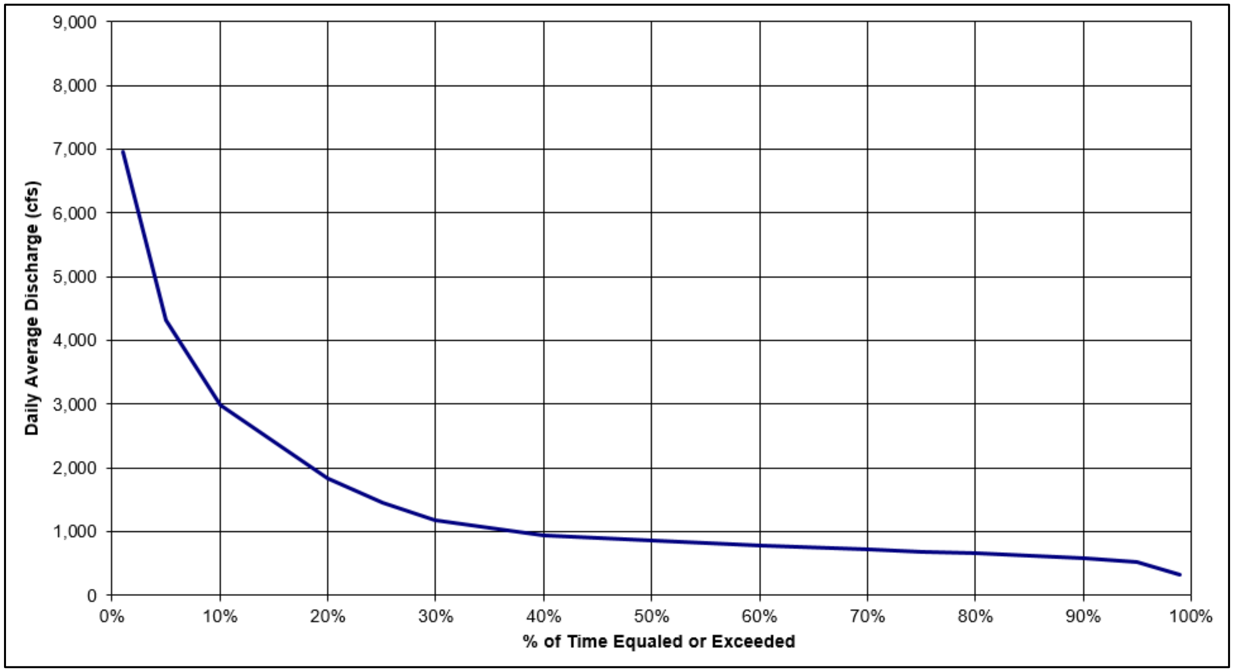


Figure 4.1 J.C. Boyle Annual Flow Duration Curve

Table 4.3 Flow Duration Flows Based on 2019 BiOp Flows – Monthly – Copco No. 1 and Copco No. 2

% of Time Equaled or Exceeded	Discharge (cfs)																	
	Monthly																	Annual
	Jan	Feb	Mar	Apr	May	Jun 1 - 15	Jun 16 - 30	Jul 1 - 15	Jul 16 - 31	Aug	Sep 1 - 15	Sep 16 - 30	Oct 1 - 15	Oct 16 - 31	Nov 1 - 15	Nov 16 - 30	Dec	
99%	900	900	940	1,250	1,110	970	960	850	850	850	940	940	940	940	940	940	900	850
95%	900	900	940	1,250	1,110	970	970	850	850	850	940	940	940	940	940	940	900	850
90%	900	900	1,080	1,250	1,110	970	970	850	850	850	940	940	940	940	940	940	900	900
80%	900	900	1,520	1,280	1,240	1,040	970	850	850	850	940	940	940	940	1,000	940	900	940
75%	900	940	1,630	1,410	1,290	1,080	970	860	870	850	940	940	960	990	1,050	940	900	940
70%	900	990	1,800	1,540	1,350	1,130	970	890	900	850	940	940	990	1,020	1,100	940	900	970
60%	970	1,120	2,210	1,810	1,430	1,200	990	930	950	850	940	940	1,030	1,070	1,130	940	900	1,050
50%	1,120	1,390	2,640	2,230	1,700	1,300	1,050	970	990	890	1,010	1,000	1,030	1,090	1,150	940	930	1,110
40%	1,420	1,980	3,120	2,780	2,080	1,480	1,120	1,000	1,000	960	1,070	1,080	1,060	1,110	1,160	940	1,060	1,250
30%	1,930	2,570	3,850	3,320	2,470	1,660	1,190	1,060	1,050	1,040	1,100	1,100	1,080	1,150	1,240	1,020	1,440	1,540
25%	2,280	2,920	4,430	3,920	2,700	1,840	1,230	1,080	1,060	1,050	1,100	1,110	1,100	1,190	1,260	1,080	1,600	1,780
20%	2,580	3,400	5,200	4,270	2,940	2,140	1,410	1,110	1,080	1,060	1,130	1,130	1,130	1,220	1,300	1,220	1,860	2,210
10%	3,980	4,820	6,080	5,260	3,620	2,830	1,770	1,160	1,160	1,110	1,160	1,160	1,200	1,350	1,460	1,960	2,800	3,430
5%	5,340	6,980	7,110	5,750	4,250	3,250	2,050	1,180	1,180	1,460	1,160	1,160	1,250	1,440	1,550	3,300	4,020	4,780
1%	9,070	10,460	8,920	7,220	5,430	4,560	2,780	1,410	1,250	1,600	1,190	1,190	1,350	3,020	3,870	4,070	6,770	7,630

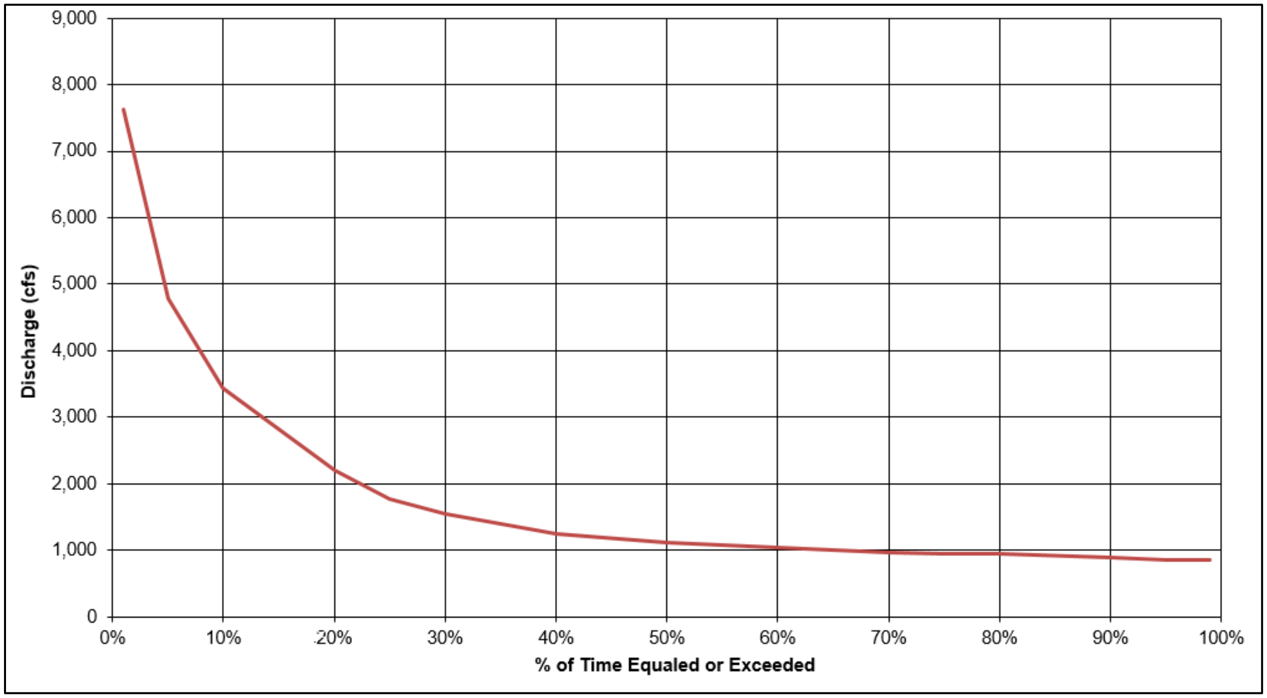


Figure 4.2 Copco No. 1 and Copco No. 2 Annual Flow Duration Curve

Table 4.4 Flow Duration Flows Based on 2019 BiOp Flows – Monthly – Iron Gate Dam

% of Time Equaled or Exceeded	Discharge (cfs)																	
	Monthly																	Annual
	Jan	Feb	Mar	Apr	May	Jun 1 - 15	Jun 16 - 30	Jul 1 - 15	Jul 16 - 31	Aug	Sep 1 - 15	Sep 16 - 30	Oct 1 - 15	Oct 16 - 31	Nov 1 - 15	Nov 16 - 30	Dec	
99%	950	950	1,000	1,330	1,180	1,030	1,020	900	900	900	1,000	1,000	1,000	1,000	1,000	1,000	950	900
95%	950	950	1,000	1,330	1,180	1,030	1,030	900	900	900	1,000	1,000	1,000	1,000	1,000	1,000	950	900
90%	950	950	1,150	1,330	1,180	1,030	1,030	900	900	900	1,000	1,000	1,000	1,000	1,000	1,000	950	950
80%	950	950	1,610	1,360	1,320	1,100	1,030	900	900	900	1,000	1,000	1,000	1,000	1,060	1,000	950	1,000
75%	950	1,000	1,730	1,500	1,370	1,150	1,030	910	930	900	1,000	1,000	1,010	1,040	1,110	1,000	950	1,000
70%	950	1,050	1,910	1,640	1,430	1,190	1,030	950	960	900	1,000	1,000	1,050	1,080	1,110	1,000	950	1,030
60%	1,030	1,180	2,340	1,920	1,520	1,270	1,050	980	1,010	900	1,000	1,000	1,090	1,130	1,190	1,000	950	1,110
50%	1,180	1,470	2,800	2,360	1,810	1,380	1,110	1,030	1,040	940	1,070	1,080	1,100	1,160	1,210	1,000	980	1,180
40%	1,500	2,090	3,310	2,950	2,200	1,570	1,180	1,050	1,060	1,020	1,130	1,140	1,120	1,180	1,230	1,000	1,120	1,320
30%	2,040	2,730	4,080	3,520	2,620	1,760	1,260	1,120	1,110	1,100	1,160	1,160	1,150	1,220	1,320	1,080	1,520	1,630
25%	2,420	3,100	4,700	4,150	2,860	1,950	1,300	1,140	1,120	1,110	1,170	1,170	1,170	1,260	1,330	1,150	1,700	1,880
20%	2,730	3,600	5,510	4,530	3,110	2,270	1,490	1,180	1,150	1,130	1,200	1,190	1,200	1,290	1,380	1,300	1,970	2,340
10%	4,220	5,110	6,450	5,570	3,840	2,990	1,870	1,230	1,230	1,180	1,230	1,230	1,270	1,430	1,540	2,070	2,960	3,630
5%	5,650	7,390	7,530	6,090	4,500	3,440	2,180	1,250	1,250	1,550	1,230	1,230	1,330	1,520	1,640	3,500	4,260	5,060
1%	9,600	11,080	9,450	7,650	5,760	4,830	2,950	1,490	1,320	1,700	1,260	1,260	1,430	3,200	4,110	4,310	7,170	8,080

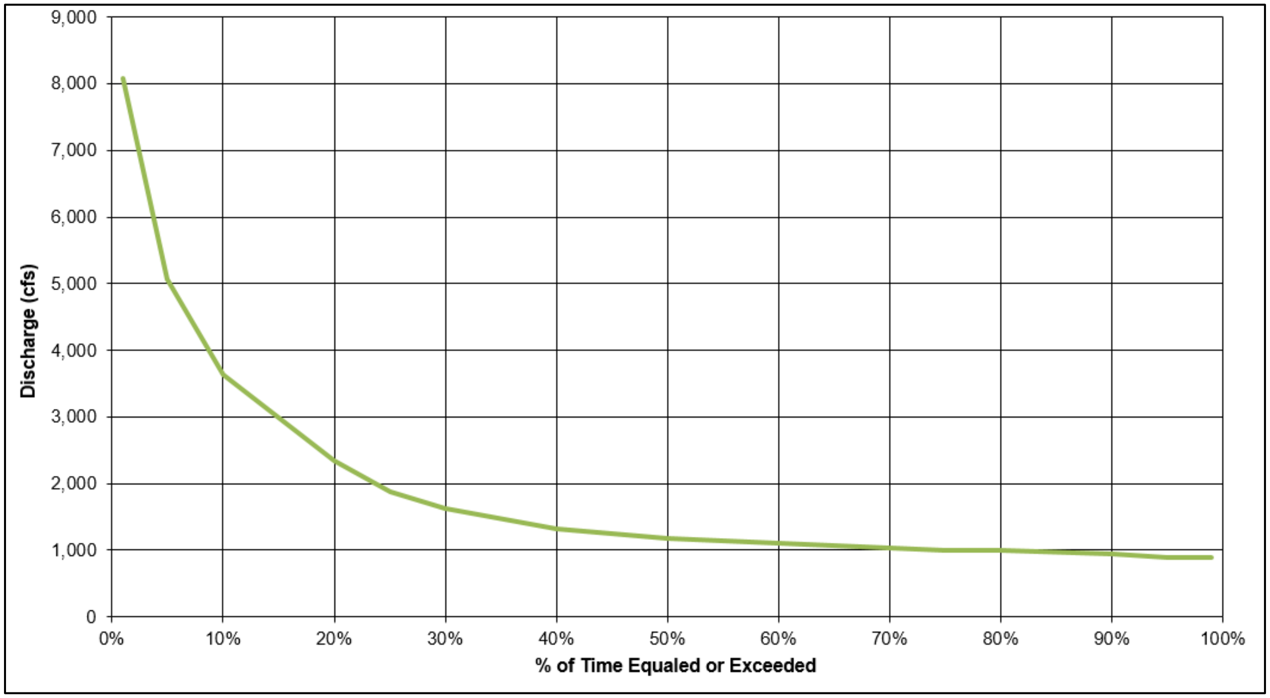


Figure 4.3 Iron Gate Dam Annual Flow Duration Curve

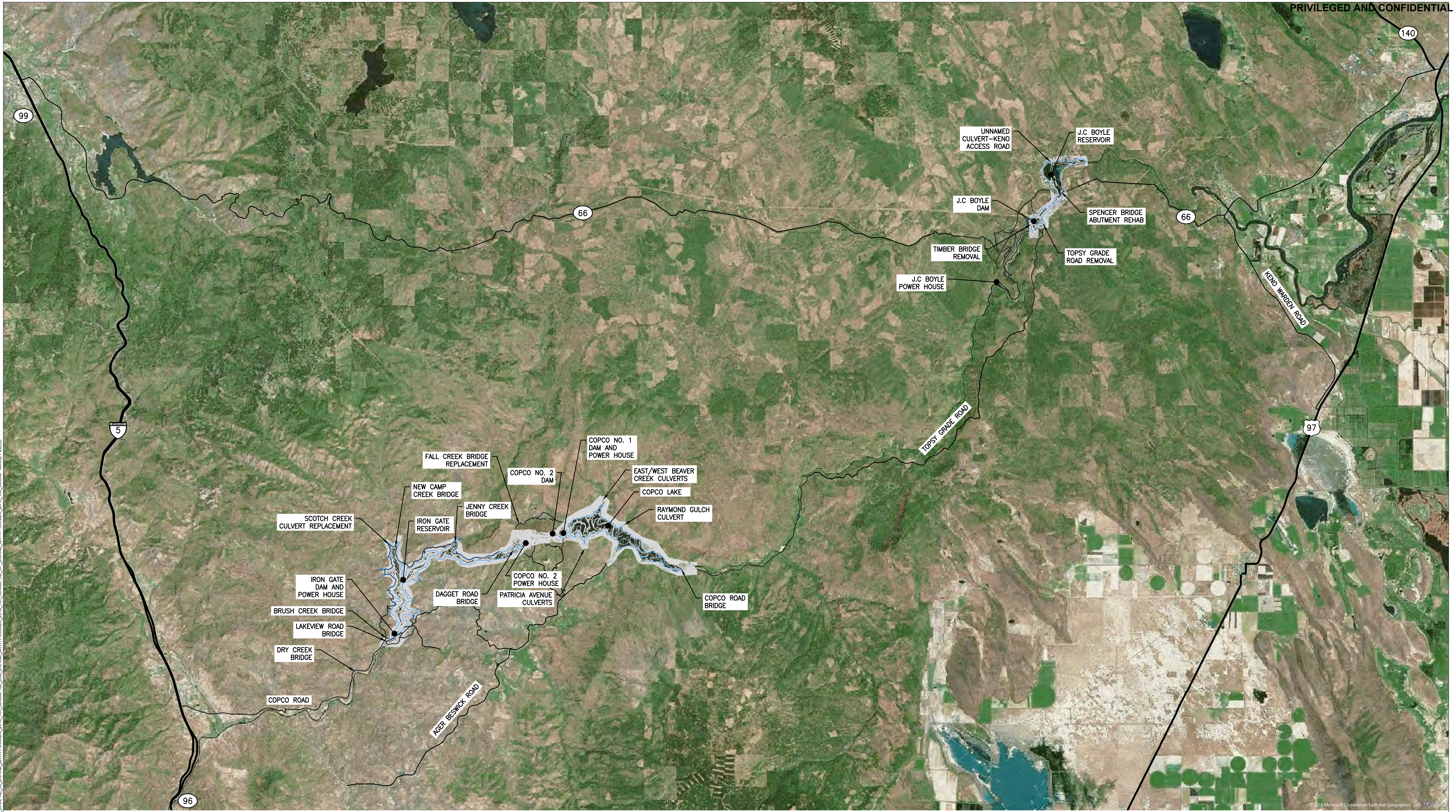
5.0 FLOWS FOR ROADS AND BRIDGE CROSSINGS

Located within the KRRP area are various roads, bridges, and culvert crossings. The locations of road, bridge, and culvert sites identified for improvement, monitoring, or construction purposes are identified on Figure 5.1.

The primary design goal for the roads, bridges, and culverts component of the KRRP is to modify the existing transport infrastructure to accommodate safe construction access throughout the KRRP site and to maintain existing public access during all stages of the project, from initial construction through to final removal of the hydroelectric facilities, and subsequent restoration. To facilitate this transportation design goal, design flood estimates for ungaged locations within the KRRP area are required.

Most of the transportation points of interest (POIs) are located on tributaries to the Klamath River, with the remaining POIs located directly on the Klamath River. The peak design floods at the ungaged locations were estimated by characterizing the tributary flows within the Klamath Basin between the J.C. Boyle and Iron Gate facilities. The Jenny Creek tributary represents a substantial portion of the incoming flows between the J.C. Boyle and the Iron Gate facilities. While Jenny Creek does have irrigation diversions and the flows are therefore partially regulated, this regulation effect is much smaller than that caused by the reservoirs on the mainstem of the Klamath River, and likely has little impact on the highest peak flows.

Many of the other larger tributary streams to the Klamath River are also regulated with irrigation structures, but as with Jenny Creek, the effects of these regulations on the largest peak flows is likely limited. The return period peak design flows calculated for all tributary streams are based on flow records for unregulated streams.



PLAN
1"= 7500'

0 7500 15000 22500
HORIZ 1" =7500', VERT 1" = 7500'

KIEWIT INFRASTRUCTURE WEST CO.

KLAMATH RIVER RENEWAL PROJECT

LOCATION OF TRANSPORTATION POINTS OF INTEREST (POI)

Knight Piésold CONSULTING

P/A NO.	REF NO.
VA103-640/1	9

FIGURE	REV
FIGURE 5.1	0

REV	DATE	DESCRIPTION	DESIGNED	DRAWN	REVIEWED
0	27MAY'22	ISSUED WITH REPORT	JCO	ELG/WAL	SRM

SAVED: M:\1\03\00640\01\A\Acad\FIGS\B03, 5/20/2022 11:43:53 AM, WLAHODA PRINTED: 5/20/2022 11:44:27 AM, Key Plan, WLAHODA ACAD VERSION: 23.1S (LMS TECH)

5.1 JENNY CREEK TRIBUTARY

Jenny Creek is a tributary to the Klamath River that discharges into the Iron Gate reservoir. The flow at Jenny Creek represents approximately 40% of the tributary and overland flow area between J.C. Boyle and Iron Gate facilities. There is an inactive USGS hydrology station located at the outlet of Jenny Creek (USGS Station JENNY C NR COPCO CA, 11516500); however, peak flow data for this gage are only available from 1923 to 1928, and the quality of the data is uncertain. This station has a drainage area of 205 mi^2 (210 mi^2 at the Jenny Creek bridge), and the records indicate annual peak flows ranging from 420 cfs to 1,960 cfs, with a six-year average of about 1,000 cfs. Relative to peak flows recorded at other creeks in the region, these values seem low.

The Bureau of Land Management (BLM) has a hydrology gage on Jenny Creek (located below Spring Creek at UTM 10T 0553140 / 4652570 (Lat/Long: 42.02335, -122.35817) with a drainage area of approximately 195 mi^2. The BLM data consists of average daily flows and annual peak flows for the period of 1998 to 2018. BLM notes that the rating curve may not be applicable and may require updating. The information for this gage has not undergone QA/QC procedures and is therefore provisional. Nonetheless, the data are believed to be the best Jenny Creek specific flow data currently available, and as such, these data were used to complete a hydrologic analysis for Jenny Creek.

5.1.1 AVERAGE MONTHLY FLOW

The average monthly flows for Jenny Creek at the Jenny Creek Bridge were calculated, as presented in Table 5.1 and on Figure 5.2. These data were prorated from the BLM gage location to the Jenny Creek bridge.

Table 5.1 Monthly Average Flow for Jenny Creek at Jenny Creek Bridge (Provisional)

Month	Monthly Average Flow (cfs)
January	121
February	181
March	305
April	225
May	136
June	41
July	16
August	15
September	16
October	19
November	29
December	87

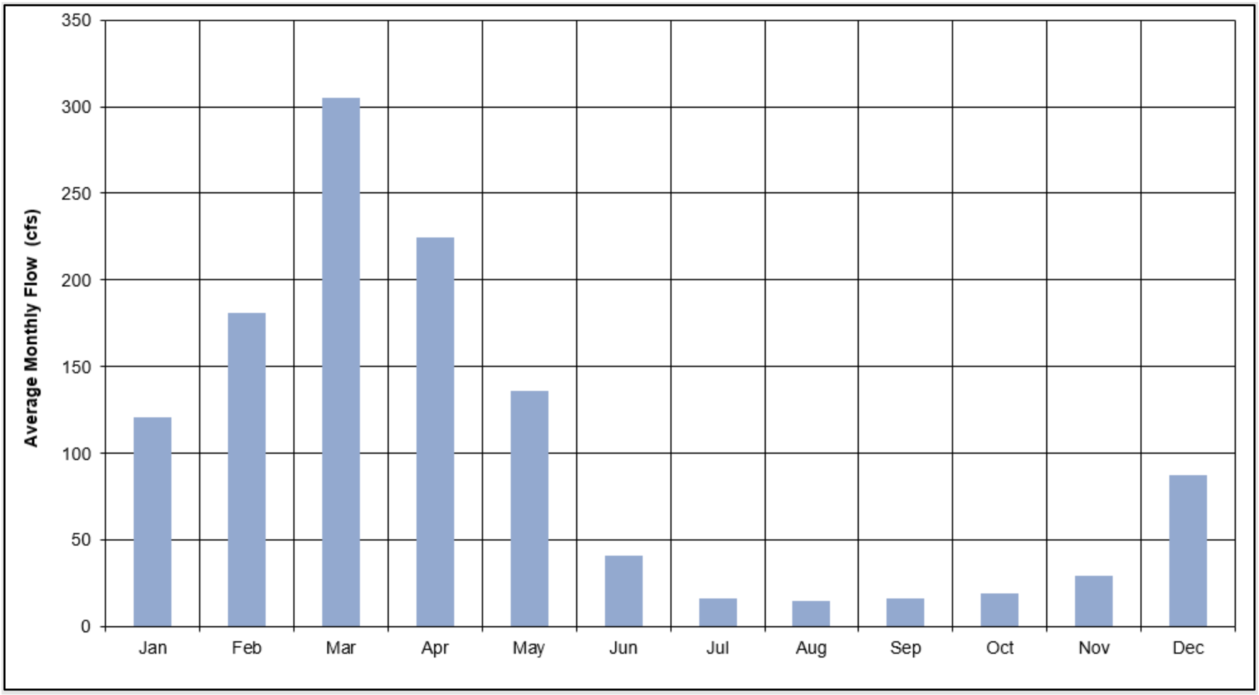


Figure 5.2 Monthly Average Flow for Jenny Creek at Jenny Creek Bridge (Provisional)

5.1.2 ANNUAL PEAK FLOODS

A summary of the available stream gage data used for the regional hydrology assessment of the tributary streams is provided in Table 5.2 below, and the station locations are shown on Figure 5.3.

Table 5.2 Summary of Streamflow Gage Records

Gage	Gage Operator/ Number	Basin Area (mi^2)	Period of Record	Notes
Klamath Tributary near Keno, OR	USGS 11509400	1.02	1964-1981	Annual peak flow estimates only. Includes the 1964 flood.
Fall Creek at Copco CA	USGS 11512000	14.6	1928 - 1959	Peak streamflow available. Does not include 1964 flood.
Fall Creek at Copco CA	PacifiCorp	14.6	2015 - 2017	Hourly data available. Not QA/QC'd. Does not include 1964 flood.
Bogus Creek	PacifiCorp	53.7	2014 - 2018	15-minute data available. Not QA/QC'd. Does not include 1964 flood.
Jenny Creek	BLM	195	1998 - 2018	15-minute data available. Not QA/QC'd. Does not include 1964 flood.
Rogue River above Prospect, OR	USGS 14328000	312	1909 - 2017	15-minute data available. Includes 1964 flood record.

Figure 5.3 Regional Streamflow Gage Locations

A regional flow assessment was performed on available peak flow data for the stream gages listed in Table 5.2. The characteristics of the gaged basins as well as the lengths of available streamflow records were considered when determining the suitability of a gage for estimating flood flows for Jenny Creek. The PacifiCorp gages on Bogus Creek and Fall Creek were excluded due to insufficient stream gage data for the analysis. The USGS gage data for Fall Creek at Copco and the Klamath Tributary near Keno were excluded because their drainage areas are outside of the range of 0.50 to 1.50 times the size of the Jenny Creek drainage area, as recommended by the USGS (2005). Data for the USGS stream gage on Rogue River above Prospect (gage number 14328000) were selected as the most appropriate dataset for calculating return period peak flows for Jenny Creek because of the similarity of Rogue River and Jenny Creek watersheds in terms of drainage area and mean basin elevation. In addition, Rogue River has a lengthy period of record, which dates from 1909 to 2017 and includes the flood of record for the Klamath region (December 1964).

A flood frequency analysis was completed for the entire period of record for the Rogue River using the HEC-SSP (V2.1), following the Bulletin 17B method for the Log-Pearson Type III distribution (USGS, 1982).

The Rogue River flood frequency results were then transposed using the area proration methodology described in "Estimation of Peak discharges for Rural, Unregulated streams in Western Oregon" (USGS, 2005) to calculate the peak flood flows for Jenny Creek at the bridge. A scaling exponent of 1.0 was used for the transposition, as recommended in USGS (2005).

A flood frequency analysis was also performed on the BLM Jenny Creek annual peak flood data using HEC-SSP (V2.1), following the Bulletin 17B method for the Log-Pearson Type III distribution (USGS, 1982). The calculated peak flood values were prorated to the Jenny Creek bridge location using the methods outlined in USGS (2005) and a scaling exponent of 1.0.

The flood frequency analysis results based on both the USGS Rogue River and the BLM Jenny Creek datasets are presented in Table 5.3.

Table 5.3 Flood Frequency Analysis for Jenny Creek Bridge

Percent Probable Flood	Jenny Creek Bridge Peak Floods (cfs)	
	Design Values - Prorated from Rogue River USGS gage, 1909 - 2017	Prorated from Jenny Creek BLM gage, 1998 - 2017
50%	3,100	1,400
20%	5,000	2,700
10%	6,500	4,000
5%	8,000	5,500
2%	10,100	8,000
1%	11,900	10,400
0.5%	13,900	13,200
0.2%	16,600	17,700

The two sets of values agree reasonably well for events greater than the 5% probable flood, while the Rogue River values are higher for events smaller than the 5% probable flood. Flood events greater than the 5% probable flood are typically used for the design of hydraulic structures.

5.2 ANNUAL PEAK FLOODS FOR LOCATIONS OTHER THAN JENNY CREEK

Design flood estimates for ungauged locations for road, bridge, and culvert crossings within the KRRP area were determined by scaling regional peak flows according to the crossing location. For ungaged locations located on the Klamath River, the annual peak floods were determined based on the design flood estimates from the closest appropriate dam facility, which were linearly prorated by the ratio of the respective drainage areas to the location of interest.

For ungaged locations on tributary streams of the Klamath River, the annual peak floods were calculated based on the annual peak flood values for the USGS gage on Fall Creek (gage number 11512000) using non-linear drainage area proration. The Fall Creek stream gage data were selected for the analysis based on drainage area size and mean basin elevation, which are generally representative of the watersheds pertaining to the majority of the POI's that are located on tributary streams much smaller than Jenny Creek. In addition, the Fall Creek record length is reasonably long, at 32 years, and though it is dated (1928 to 1959), it is the most appropriate record available for small streams.

A flood frequency analysis was performed on the Fall Creek annual peak flood data using HEC-SSP (V2.1), following the Bulletin 17B method for the Log-Pearson Type III distribution (USGS, 1982). The calculated peak floods were then non-linearly prorated to the POI locations. The scaling exponent for drainage area was investigated to determine the appropriate value to use for the smaller drainage areas of the POIs. A review of the various USGS regional regression equations for determining peak floods for Oregon and California for the Klamath region indicates scaling exponents ranging from 0.5 to 1.0, although most of the values tend to be towards the upper end of the range, and therefore a value of 0.9 was selected for design purposes.

Preliminary design flood values estimated for roads, bridges, and culverts are provided on a site-by-site basis in Table 5.4.

Table 5.4 Annual Peak Floods for Roads, Bridges, and Culvert Structures

Location	Drainage Area (mi^2)	Annual Percent Probable Flood (cfs)[6]						
		50%	10%	5%	2%	1%	0.5%	0.2%
Scotch Creek Culvert[1]	17.9	170	450	600	850	1,070	1,320	1,710
New Camp Creek Bridge[1]	19.8	180	490	660	930	1,170	1,440	1,870
Jenny Creek Bridge	210	1,400	4,000	5,500	8,000	10,300	13,100	17,700
Timber Bridge Removal[2,3]	4,080	7,000	10,300	11,700	13,300	14,200	15,000	15,800
East/West Beaver Culverts[1]	5.6	60	160	210	300	370	460	600
Raymond Gulch Culvert[1]	2.5	28	80	103	140	180	220	291
Patricia Avenue Culverts[1]	0.4	5	15	20	28	35	43	56
Copco Road Bridge[2,3]	4,340	7,100	13,900	18,100	24,000	29,200	34,800	42,900
Unnamed Culvert Keno Access Road[1]	12.2	120	320	430	600	750	930	1,210
Spencer Bridge[2,3]	4,050	6,900	10,200	11,600	13,200	14,100	14,900	15,700
Topsy Grade Road Culvert[1]	2.2	30	70	90	130	160	200	260
Daggett Road Bridge[2,3,4]	4,370	7,100	14,000	18,200	24,200	29,400	35,000	43,200
Fall Creek Bridge[1]	12.2	120	320	430	600	750	930	1,210
Brush Creek Bridge[1]	5.0	50	140	190	270	340	420	540
Lakeview Road Bridge[2,3,5]	4,630	7,500	14,900	19,300	25,700	31,200	37,100	45,800
Dry Creek Bridge[1]	8.9	90	240	320	450	570	700	910

NOTES:

1. VALUES ARE CALCULATED BASED ON FALL CREEK ANNUAL PEAK FLOOD RESULTS USING NON-LINEAR DRAINAGE AREA PRORATION WITH A SCALING FACTOR OF 0.9 (USGS, 2005).
2. VALUES ARE BASED ON ANNUAL PEAK FLOOD RESULTS FROM THE CLOSEST APPROPRIATE DAM FACILITY, WHICH WERE LINEARLY PRORATED BY THE RATIO OF THE RESPECTIVE DRAINAGE AREAS.
3. THE SITE IS LOCATED ON THE KLAMATH RIVER AND THEREFORE THE FLOW DATA ARE REGULATED.
4. THE DRAINAGE AREA OF THE COPCO NO. 1 FACILITY WAS USED FOR THE DRAINAGE AREA OF POINT OF INTEREST.
5. THE DRAINAGE AREA OF THE IRON GATE FACILITY WAS USED FOR THE DRAINAGE AREA OF POINT OF INTEREST.

6.0 POST-DAM REMOVAL PEAK FLOODS

The KRRP dams currently create upstream reservoirs and pass flood flows through spillways. The routing of flows through the reservoirs and over the spillways necessitates a rise in the reservoir levels and the associated temporary storage of flow volumes, which results in an attenuation of flood peak discharges. With the removal of the dams, there will be no more flood attenuation, which will impact the flood magnitudes in the future. This section presents post dam removal peak flows for use in designing permanent features at the former dam sites.

A hydrologic model was developed to estimate the change in the magnitude of the peak floods post-dam removal, which simulates flows in the Klamath River from downstream of the Keno Dam to downstream of the Iron Gate Dam, as described in Attachment 1 (KP Memo VA22-00403). The model was set up using HEC-HMS (v 4.3) to route the flows through the Copco No. 1 reservoir and spillway and then through the Iron Gate reservoir and spillway. Routing effects from the J.C. Boyle and Copco No. 2 reservoirs and spillways were omitted as these reservoirs have negligible active storage volumes. Once the model was calibrated using tributary inflows for various recorded storm events for the pre-dam removal case, the same storms were modelled again with the dams removed.

6.1 ANNUAL PEAK FLOODS

Two empirical equations were developed from the post-dam removal modeling results to aid in estimating the effects on peak floods that may result from the removal of the dams, as discussed in Attachment 1 (KP Memo VA22-00403). Using these empirical equations and the annual peak floods from Table 3.2 (that include attenuation), the post-dam removal annual peak floods were calculated per facility and are shown in Table 6.1.

Table 6.1 Post-Dam Removal Annual Peak Floods

Location	Drainage Area (mi^2)	Annual Percent Probable Flood (cfs)							
		50%	20%	10%	5%	2%	1%	0.5%	0.2%
J.C. Boyle	4,080	7,000	8,500	10,300	11,700	13,300	14,200	15,000	15,800
Copco No. 1	4,370	11,200	15,400	19,900	24,300	29,400	32,700	36,800	45,400
Iron Gate	4,630	11,700	16,200	20,900	25,400	30,500	33,600	39,000	48,100

The J.C. Boyle Dam reservoir provides minimal attenuation of peak floods, therefore there is negligible increase to the peak flood events. As such, the annual peak floods in Table 3.2 are also used to represent the post-dam removal floods for this facility. The annual return period floods at Copco No. 1 are used as representative of the annual return period floods for Copco No. 2.

6.2 PEAK FLOODS FOR MONTHLY TIME PERIODS

The post-dam removal empirical equations are applicable to peak events that result from snowmelt and/or rain-on-snow events, including the annual peak events. When there is less rainfall during the low flow summer months, the monthly peak flood events are primarily driven by releases from Upper Klamath Lake and there is less contribution from tributary and overland sources. Accordingly, peak flows during the summer months tend to be sustained for extended periods and there is little attenuation as these flows pass through the power generation facilities to the downstream. As such, the empirical equations developed for post-dam removal peak flows are not applicable to high flows that occur during the period between June 16 and September 30. The post-dam removal high flows during this period will likely be similar to the existing conditions.

The monthly peak floods from Table 3.6 were used to calculate the post-dam removal monthly peak floods per facility by applying the empirical equations (see Attachment 1) to the flows between October 1 to June 15, and by adopting the current values (Table 3.6) for flows from June 15 to September 30. The estimated post-dam removal flows are shown in Table 6.2.

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TABLE 6.2

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

POST-DAM REMOVAL MONTHLY PEAK FLOODS

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Location	Drainage Area (mi²)	Month	Instantaneous Peak Floods for Specified Time Period (cfs)								Average Monthly Flow (cfs)
			50% Probable Flood	20% Probable Flood	10% Probable Flood	5% Probable Flood	2% Probable Flood	1% Probable Flood	0.5% Probable Flood	0.2% Probable Flood	
J.C. Boyle	4,080	Jan	2,600	4,400	6,000	8,000	11,100	14,000	15,000	15,800	1,500
		Feb	2,700	4,900	7,000	9,700	13,000	14,200	15,000	15,800	1,900
		Mar	6,300	8,000	8,800	10,900	13,300	14,200	15,000	15,800	2,800
		Apr	4,500	6,800	8,100	9,400	11,600	13,600	15,000	15,800	2,370
		May	2,700	4,300	5,500	6,800	8,500	9,900	11,300	13,400	1,760
		Jun 1 - 15	1,800	2,800	3,500	4,400	5,800	7,300	9,100	12,100	1,330
		Jun 16 - 30	1,400	1,800	2,300	2,800	3,600	4,400	5,000	6,300	960
		Jul 1 - 15	1,000	1,400	1,700	2,100	2,700	3,200	3,900	4,900	740
		Jul 16 - 31	1,400	1,500	1,600	1,700	1,800	1,800	1,800	2,000	760
		Aug	1,400	1,500	1,600	1,700	1,800	1,800	1,800	1,900	760
		Sep 1 - 15	1,400	1,700	1,900	2,100	2,400	2,500	2,700	3,000	810
		Sep 16 - 30	1,500	1,900	2,200	2,400	2,800	3,000	3,200	3,500	790
		Oct 1 - 15	1,700	2,200	2,500	2,900	3,400	3,800	4,200	4,700	810
		Oct 16 - 31	1,700	2,400	2,800	3,300	4,000	4,600	5,200	6,100	890
		Nov 1 - 15	1,800	2,600	3,200	3,800	4,700	5,500	6,300	8,600	980
		Nov 16 - 30	2,000	2,900	3,600	4,400	5,400	7,200	9,600	14,000	950
		Dec	2,500	3,900	5,100	6,300	8,300	10,500	13,000	15,600	1,110
Copco No. 1	4,370	Jan	5,800	9,600	12,900	17,300	23,700	28,800	33,200	44,900	1,910
		Feb	5,800	9,600	13,000	17,300	23,700	28,800	33,200	44,900	2,360
		Mar	10,500	13,000	15,300	18,700	23,200	26,400	29,200	32,500	3,230
		Apr	8,000	11,000	13,500	16,400	20,300	23,100	25,700	28,800	2,790
		May	5,700	7,900	9,700	11,600	14,200	16,300	18,400	21,100	2,110
		Jun 1 - 15	4,200	5,700	6,800	7,900	10,300	12,600	15,600	20,000	1,620
		Jun 16 - 30	1,400	1,900	2,400	2,900	3,600	4,400	5,100	6,400	1,210
		Jul 1 - 15	1,100	1,300	1,600	2,000	2,600	3,200	4,100	5,300	990
		Jul 16 - 31	1,200	1,300	1,500	1,600	1,800	2,000	2,100	2,400	990
		Aug	1,200	1,300	1,500	1,600	1,800	2,000	2,100	2,400	980
		Sep 1 - 15	1,300	1,600	1,800	1,900	2,100	2,200	2,300	2,500	1,030
		Sep 16 - 30	1,300	1,600	1,900	2,100	2,400	2,500	2,700	3,000	1,030
		Oct 1 - 15	3,600	4,400	5,100	5,700	6,800	7,600	8,700	10,100	1,050
		Oct 16 - 31	3,600	4,700	5,400	6,300	7,500	8,700	9,800	11,700	1,140
		Nov 1 - 15	3,900	5,100	6,300	7,400	9,100	10,600	12,200	15,000	1,230
		Nov 16 - 30	4,200	5,800	7,200	8,400	10,500	12,100	14,600	19,900	1,240
		Dec	5,100	8,500	11,600	15,900	22,700	28,200	33,200	45,400	1,490
Iron Gate	4,630	Jan	6,100	9,900	13,500	18,100	24,800	29,900	34,100	47,700	2,030
		Feb	6,100	9,900	13,700	18,100	24,800	29,900	34,100	47,700	2,500
		Mar	11,000	13,700	16,000	19,700	24,200	27,500	30,300	33,400	3,430
		Apr	8,300	11,500	14,200	17,300	21,300	24,100	26,800	29,900	2,950
		May	5,800	8,300	10,200	12,200	15,000	17,200	19,300	22,100	2,230
		Jun 1 - 15	4,400	5,800	6,900	8,300	10,800	13,300	16,400	21,000	1,720
		Jun 16 - 30	1,500	2,000	2,500	3,000	3,700	4,400	5,200	6,500	1,280
		Jul 1 - 15	1,200	1,400	1,700	2,100	2,800	3,400	4,300	5,600	1,050
		Jul 16 - 31	1,300	1,400	1,600	1,700	1,900	2,100	2,200	2,500	1,050
		Aug	1,300	1,400	1,600	1,700	1,900	2,100	2,200	2,500	1,040
		Sep 1 - 15	1,400	1,700	1,900	2,000	2,200	2,300	2,400	2,600	1,090
		Sep 16 - 30	1,400	1,700	2,000	2,200	2,500	2,700	2,900	3,200	1,090
		Oct 1 - 15	3,800	4,500	5,300	6,000	7,100	8,000	9,100	10,600	1,120
		Oct 16 - 31	3,800	4,800	5,700	6,500	7,900	9,100	10,300	12,300	1,210
		Nov 1 - 15	4,100	5,400	6,500	7,800	9,400	11,100	12,900	15,800	1,300
		Nov 16 - 30	4,400	6,100	7,500	8,800	11,000	12,700	14,900	19,900	1,310
		Dec	5,400	8,900	12,200	16,700	23,700	29,300	34,100	48,100	1,580

M:\1\03\00640\01\A\Data\Task 2200 - Project Support\Hydrology Update Post Removal\Flood Frequency Analysis Update\[Post-Dam Removal Peak Floods - Monthly - 100% DCD.xlsm]Table_Post-Dam Removal Monthly

NOTES:
1. PEAK FLOOD CALCULATIONS ARE BASED ON METHODOLOGY PRESENTED IN KP MEMO "REVISED KLAMATH RIVER FLOOD HYDROLOGY – POST DAM REMOVAL" (VA22-00321, MARCH 2022).
2. PRE-DAM REMOVAL ANNUAL PEAK FLOOD VALUES WERE REQUIRED FOR CALCULATIONS AND ARE TAKEN FROM TABLE 3.2, APPENDIX A6, 100% DESIGN REPORT (VA103-640/1-9, REV 0).
3. THE POST-RIVER DIVERSION MONTHLY PERCENT PROBABLE FLOODS AT COPCO NO. 1 ARE USED AS REPRESENTATIVE OF THE POST-RIVER DIVERSION ANNUAL PERCENT PROBABLE FLOODS FOR COPCO NO. 2.
4. THE PEAK FLOODS HAVE NOT BEEN ADJUSTED FOR THE SUMMER PERIOD BETWEEN JUNE 16 TO SEPTEMBER 30. THE PEAK FLOODS DURING THIS PERIOD ARE ASSUMED TO BE UNAFFECTED POST-RIVER DIVERSION.
5. THE PEAK FLOODS HAVE NOT BEEN ADJUSTED FOR THE J.C. BOYLE FACILITY AS THE RESERVOIR PROVIDES MINIMAL ATTENUATION OF PEAK FLOODS. THE PEAK FLOODS IN TABLE 3.2, APPENDIX A6, 100% DESIGN REPORT (VA103-640/1-9, REV 0) ARE ASSUMED TO BE UNAFFECTED POST-RIVER DIVERSION.

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Attachments:

VA22-00403 Revised Klamath River Flood Hydrology – Post Dam Removal

References:

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APPENDIX A7
DESIGN CRITERIA

TABLE OF CONTENTS

1.0 INTRODUCTION

The following design criteria were developed in a collaborative manner by the Klamath River Renewal Project Team (i.e. KRRC, Kiewit, KP, RES, and Camas).

These design criteria provide the agreed basis for KP's design of Kiewit's reservoir drawdown and dam removal scope of work, and related activities including construction access improvements.

The design criteria are presented in the following tables:

- Table A7.1 – Diversion Tunnels
- Table A7.2 – Reservoir Drawdown
- Table A7.3 – Embankment Dam Removal
- Table A7.4 – Concrete Dam and Structures Removal
- Table A7.5 – Roads, Bridges, and Culverts
- Table A7.6 – Material Deposition
- Table A7.7 –Dam Site Permanent Works

Overarching design criteria and roles for the following key topics are addressed below:

- Flood design criteria for embankment dam removal
- Final dam breach criteria for Iron Gate Dam
- River channel design criteria

2.0 FLOOD DESIGN CRITERIA FOR EMBANKMENT DAM REMOVAL

Embankment dam crest elevations during the various stages of removal shall meet the following criteria with regards to flood passage. Design of the excavations for J.C. Boyle and Iron Gate dams shall provide for a dam section that can safely retain water, meet stability criteria, and have a crest elevation that is 3 feet greater than needed to allow for passage of a 1% probable flood for that time of year. As embankment removal advances, a point is reached where in advance of breach where the crest elevation is no longer required to be above the 1% probable flood elevation, and instead it is to be kept above a 5% probable flood elevation for that time of year.

3.0 FINAL DAM BREACH CRITERIA FOR IRON GATE DAM

The final dam breaches will be timed to avoid periods of high inflow to limit the magnitude of peak outflow through the breaches.

The final breach of Iron Gate Dam is unique because it will have the largest impounded water volume at the time of final breach and because it is located farthest downstream of the four facilities being removed. The specific target for peak outflow discharge is approximately 6,000 cfs, as measured at USGS Gaging Station No. 11516530, Klamath River below Iron Gate Dam. This criterion is based on the estimated bankfull discharge of 5,000 to 6,000 cfs in the Klamath River downstream of Iron Gate Dam, as provided by the Yurok Tribe.

KRRC is responsible for the following aspects related to the final dam breaches, which are not addressed in these design criteria:

- Public safety, including public communication and public access restriction outside of Kiewit controlled construction areas (as required).
- Assessment and mitigation (as required) of potential downstream impacts associated with the final breach outflow wave, including sediment transport and deposition.

4.0 RIVER CHANNEL DESIGN CRITERIA

The design criteria and roles for the final river channels through the existing dam sites are further described below:

- Final channel, floodplain and canyon wall geometry throughout the removal extents shall provide a geomorphically appropriate transition between cross sections, that is passable to fish species of concern, immediately upstream and downstream of the previous dam location.
- The KRRP Team has collectively agreed on specific criteria related to the geomorphically appropriate transition of the final river channel, floodplain, and canyon walls, including depth of concrete removal below the remediated river channel, thickness of riverbed fill material to be placed over concrete structures left in place below the remediated riverbed, lateral extent of dam structure removals, and the upstream and downstream extents and elevations for dam removal excavations. These agreed criteria are documented in Tables A7.4 and A7.7.
- Kiewit/KP's scope for design of the final fish volition channels comprises the footprints of the existing dams and historic cofferdams. The Habitat Contractor will review the designs and provide acceptance for volitional fish passage and will be responsible for scope outside the footprint limits of the existing dams and historic cofferdams.

The limits of excavation at each of the dam sites is based on the site foundation geology.

- J.C. Boyle: The bedrock at the foundation is rough with ridges and high points. The volitional fish passage channel bottom will be on top of the encountered rock features. Channel roughening does not require dental cleaning between the rough rock ridges and high points as these features should be preserved. Boulders and large rocks from the historic dam construction will be encountered at the downstream toe of the dam as it is excavated. These are recognized roughening features and will be graded to the channel configuration.

- Copco No. 1: The dam site is within a narrow rock-walled canyon. The rock walls undulate, and resident talus material is located between the rock formation. The rock walls and foundations at the concrete dam will be excavated to bedrock or the agreed concrete excavation limit, and then backfilled. Upstream of the dam, there is a combination of construction waste material (soil, rock, and other construction debris). The construction waste material will be removed to the higher of bedrock, stable talus, or the designed longitudinal channel bottom profile. The designed longitudinal channel profile will tie into existing channel bathymetry upstream of the historic cofferdam and the existing channel profile downstream of the Copco No. 1 powerhouse.
- Copco No.2: The dam site is on a soil foundation. The concrete dam will be excavated to the concrete excavation limit and then backfilled to match the adjacent channel.
- Iron Gate: The dam site is a U-shaped rock-walled and bottomed canyon. The dam will be excavated to the higher of bedrock limits or to the designed longitudinal channel bottom profile.

Erosion protection:

- Erosion protection will be provided for permanent fill slopes within the dam excavation footprints. Erosion protection is not required on bedrock slopes.
- Additional rock or other materials requested by the Habitat Contractor for aquatic habitat purposes to be shown on the Habitat Contractor design documents.

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TABLE A7.1

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

DIVERSION TUNNELS
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 TUNNEL IMPROVEMENTS			
Hydrostatic and Hydrodynamic Pressure	• Conditions that will occur when reservoir level is consistent with the 1% flood event.	• Maintain balaced hydrostatic pressures across tunnel liner or rock consistent with the existing conditions. • This criteria applies to the tunnel and all associated works and apertanances including valves, gates, and venting.	• USACE EM 1110-2-2901, 1997
Diversion Tunnel Water Velocity	• For all drawdown hydrologic requirements: - Unlined Rock: <10 ft/sec - Concrete: <20 ft/sec	• The diversion tunnel operation during drawdown and deconstruction are about 10 months in duration, reinforced concrete will be used for short sections of tunnel where velocities over 20 ft/s are required	•USBR Design Standards No. 3, Chapter 4: Tunnels, Shafts and Caverns (2014)
Diversion Tunnel Air Flow	• Natural air flow within tunnel or installed venting shall be designed to mitigate adverse pressure conditions and cavitation that may compromise tunnel integrity for all drawdown or hydrologic scenarios up to and including the 1% Flood Event	• Dr. H.Falvey is the project reviewer	• Engineering Monograph No. 41 (Falvey, 1980)
Tunnel Ground Support	• Safe Construction Access	• Where modifications are not required for hydraulic drawdown criteria above, ground support shall be provided for safe construction access	• USACE EM 1110-2-2901, 1997
Portal Slope Protection	• Safe Construction Access	• Where modifications are not required for hydraulic drawdown criteria above, ground support shall be provided for safe construction access	• USACE EM 1110-1-2908, 1994 • USACE EM 1110-2-1902, 2003
Tunnel/Shaft Closure (Post Drawdown)	• Ensure no public access, pedestrian or vehicle is possible following drawdown. • Include provision for tunnel seepage		

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TABLE A7.2

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

RESERVOIR DRAWDOWN
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 OPERATING REQUIREMENTS			
Daily Minimum Downstream Flows	Downstream of Iron Gate as measured at the USGS Gage: • Sept through Nov, March 1,000 cfs • Dec through Feb 950 cfs • April 1,325 cfs • May 1,175 cfs • June 1,025 cfs • July and Aug 900 cfs	• Minimum flows will be dictated by USBR requirements which may supersede the Biological Opinion flows as set out.	• USBR, BIOP 2019
Normal Maximum Operating Surface Elevation (ft msl)	• J.C. Boyle = 3,796.7 ft • Copco Lake = 2,611.0 ft • Iron Gate = 2,331.3 ft		FERC Licence Application - Exhibit A (2004) - NAVD88 Elevations
Normal Minimum Operating Surface Elevation (ft msl)	• J.C. Boyle = 3,791.7 ft • Copco Lake = 2,604.5 ft • Iron Gate = 2,327.3 ft		
3.0 DRAWDOWN			
Initial Drawdown	• To begin on or about January 1 of the drawdown year.		
Reservoir Drawdown Rate	• Target drawdown water surface level rate approximately 5 ft/day	• Each facility is unique relative to reservoir area capacity and proposed drawdown. Actual drawdown will be based on the actual inflow conditions during the applicable water year	
4.0 GEOTECHNICAL REQUIREMENTS			
4.1 Slope Stability of Reservoir Rim			
Minimum Required FOS	• Drawdown = 1.2	• Reservoir Drawdown criterion applies to existing dam embankment slopes	• USBR Design Standard No. 13 • USACE EM 1110-2-1902, 2003
	• Long-term, Post Drawdown = 1.5		• USBR Design Standard No. 13 • USACE EM 1110-2-1902, 2003
Design Earthquake for Temporary Construction	• Refer to Seismicity design criteria found in Appendix A4		

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TABLE A7.3

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

EMBANKMENT DAM REMOVAL
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 PRE EMBANKMENT REMOVAL REQUIREMENTS			
Iron Gate Dam STID	• STID Section 8 - Stability and Stress Analyses		• PacifiCorp, Klamath Hydroelectric Project FERC No. P-2082 Iron Gate Hydroelectric Development (NatDam: CA00325), Supporting Technical Information Document(STID) Rev.2 (4-30-2015)
JC Boyle Dam STID	• STID Section 8 - Stability and Stress Analyses		• PacifiCorp, Klamath Hydroelectric Project FERC No. P-2082 J.C. Boyle Hydroelectric Development, Supporting Technical Information Document(STID) Rev.2 (4-30-2015)
2.0 EMBANKMENT REMOVAL REQUIREMENTS			
Minimum Freeboard Elevation (embankment)	• Dam deconstruction will be staged to provide a remaining dam section that can safely retain water and meet stability and stress requirements		• See Project STID
	• Freeboard will be provided during dam deconstruction of 3 ft or greater for a 1% probable flood at that time of year.		• USBR Design Standard No. 13
	• In the late stages of dam deconstruction, freeboard will be provided of 3 ft or greater for a 5% probable flood at that time of year.		
Final Dam Breach Rate	• J.C. Boyle and Iron Gate final dam breaches shall be designed to maximize the amount of material removal by the flow of the Klamath River • The timing of final dam breaches will avoid periods of high inflow • The target peak outflow for the final breach of Iron Gate Dam is approximately 6000 cfs, as measured at USGS Gaging Station No. 11516530, Klamath River below Iron Gate Dam.	The impounded water surface level at the time of final dam breach will depend on hydrologic conditions during the drawdown period. The peak outflow discharge is based on estimated bankfull discharge of 5000 to 6000 cfs downstream of Iron Gate Dam.	DJ Bandowski, Yurok Tribe, e-mail correspondence, March 10, 2022.
Design Earthquake for Temporary Construction	• Design earthquake to be per Appendix A4		
3.0 SLOPE STABILITY			
3.1 Minimum Factors of Safety for Temporary Slopes			
Reservoir Drawdown	• FOS = 1.3		• USBR Design Standard No. 13 • USACE EM 1110-2-1902, 2003

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TABLE A7.4

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

CONCRETE DAM AND STRUCTURES REMOVAL
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 PRE CONCRETE DAM REMOVAL REQUIREMENTS			
Copco No.1 Dam STID	• STID Section 8 - Stability and Stress Analyses		• PacifiCorp, Klamath Hydroelectric Project FERC No. P-2082 Copco 1 Hydroelectric Development, Supporting Technical Information Document(STID) Rev.2 (4-30-2015)
Copco No.2 Dam	• Low-hazard potential rated structure, not required to have an STID		
2.0 COPCO NO. 1 CONCRETE DAM PROPERTIES			
Cement Efficiency	• 10 psi/lb/cu yd		• ACI (1996)
Concrete Unconfined Compressive Strength	• Main section of dam = 4000 psi (minimum) • Upstream and downstream cutoff wall = 3000 psi (minimum)	• No records of compressive stress analysis are reported for the concrete of the dam • Construction drawings and photographs indicate the main section of the dam is constructed of a mixture of concrete and hand-placed large stones	• Construction Drawings and Photographs
Concrete Tensile Strength	• Static = 430 psi	• Based on splitting tensile test studies	
	• Dynamic = 640 psi		• ACI (1996)
Existing Reinforcing Steel	• 30-pound Rails	• Horizontal rails are placed at 8 ft center to center • Vertical rails are placed at 12 ft center to center • Upper cutoff wall construction consists of one layer of horizontal and vertical rails	• Construction Drawings and Photographs
	• 0.75" - 1.25" square bars	• Used in the construction of spillway piers, deck, and other sections requiring more complex shapes	
	• Yield strength: Fy = 27 ksi		• ACI 562 (2016)
3.0 STRUCTURE REMOVAL AND DEMOLITION			
In-Channel Concrete Removal	• Concrete in river channel will be removed to a depth intended to prevent future development of fish passage impediments, as reviewed and agreed by KRRP Habitat Contractor		
	• Copco No. 1: The elevation for concrete removal at the base of the concrete dam within the dam footprint fish volition channel is 2,472.1 ft. The specific agreed thickness of riverbed fill placement over the final concrete surfaces within the dam footprint fish volition channel is 10 ft.		
	• Copco No. 2: The elevation for concrete removal within the dam footprint fish volition channel is 2,453.5 ft. Riverbed fill material will be placed to blend with natural riverbed material at the fill extents.		
Out-of-Channel Concrete Removal	• Concrete removal depth and final grading to blend with natural topography. Concrete should not be removed where concrete is necessary for rock integrity and stability	• Removal depth to be confirmed during dam deconstruction	
Cutoff Wall Removal	• The cutoff walls that protrude above the river bed surface under the J.C. Boyle and Iron Gate embankments will be removed		
	• Gunite Cutoff Wall at Copco No 2. will be partially removed and buried, as reviewed and agreed by KRRP Habitat Contractor		
4.0 DAM STRUCTURAL STABILITY CRITERIA			
Stability and Stress Analyses	• Copco No.1 reservoir pre-drawdown dam modification analyses to follow STID		• PacifiCorp, Klamath Hydroelectric Project FERC No. P-2082 Copco 1 Hydroelectric Development, Supporting Technical Information Document(STID) Rev.2 (4-30-2015)

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TABLE A7.5

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

ROADS, BRIDGES, AND CULVERTS
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 SITES AND ENVIRONMENT			
1.1 Hydrology and Hydraulics			
Temporary Bridge Soffit Minimum Freeboard Requirements	• Minimum freeboard for temporary bridges will be 1 ft during 5% annual probable flood. Distance is measured from water surface elevation to the lowest point on the bridge deck.	• Minimum freeboard for temporary bridges will be 1 ft during 5% Flood Events.	
Design Storm/Discharge Data	• Temporary Structures = 5% annual probable flood • Permanent Structures = 1% annual probable flood		• AASHTO
Scour	• Temporary Structures = 5% annual probable flood • Permanent Structures = Per AASHTO		• AASHTO
1.2 Seismicity			
Temporary Bridge	• Seismic Design Spectra is 10% probability of exceedence in 10 years. Site Modified spectral response for lateral acceleration = 0.082 (period = 0.2s) (USGS)		• Caltrans LRFD - Memo to Designers (May 2011) Site Seismicity for Temporary Bridges and Stage Construction.
Permanent Box Culverts	• MCE - 2% Probablity of Exceedence in 50 years • Site modified Peak Ground Acceleation (PGA_M) = 0.452		ASCE7-16
Permanent Steel Plate Arch Culvert	• MCE - 2% Probablity of Exceedence in 50 years • Site modified Peak Ground Acceleation (PGA_M) = 0.237		ASCE7-17
2.0 ROADS			
2.1 Basic Design Policies			
Temporary and Construction Access Roads Speed Limits	• 15 mph		
2.2 Roadway Geometry Design and Structure Standards			
Permanent Roads	• Match to existing per agreed to MOUs based on pre-job video as agreed to.		
Temporary Roads	• Per The Project Company		
2.3 Temporary Construction Access at Dam Sites (General)			
Design Vehicle	• 45 ton off-highway articulated haul truck	• CAT 745	
Minimum Lane Width	• 15 ft		
Safety Berm	• 3 ft where exposed to side slope.		
Minimum Curve Radius	• 35 ft		
Road Grade	• Normal road grade = ≤7% • Maximum road grade = 15%	• An exception to maximum road grade is made at the J.C. Boyle facility for portions of the lower penstock access road in order to minimize slope cuts.	
Surfacing Water Management	• As required in order to maintain safe and effective contruction access.		
2.4 Temporary Construction Access at Dam Sites (Specific)			
Copco No. 1 Right Bank Construction Access / Haul Roads	See specific design criteria memo: KP Ref VA21-00436, found in Appendix F5.		
Iron Gate Haul Road	See specific design criteria memo: KP Ref VA22-00428, found in Appendix F6.		
2.5 Public Roadway Geometric Cross Section			
Lane Width	• 11 ft minimum, or match existing width		
Number of Lanes During Construction	• Maintain one lane minimum with traffic control; • Temporary full lane closure as needed with prior approval		
Temporary roadway max turning radius	• Outside turning radius of 65'	Supplier provided turning radii.	• Per The Project Company
2.6 Pavement Design - Copco Road Rehabilitation			
Replacement of Paved Road Surfaces	• Match to existing per agreed to MOUs based on pre-job video as agreed to.		• AASHTO 1993
Replacement of Gravel Road Surfaces	• Match to existing per agreed to MOUs based on pre-job video as agreed to.		
2.7 Roadside Design			
Cut/Fill Slopes	• 1V:3H or flatter	• Embankment slopes no steeper than 1V3H wherever practical and, ideally, 1V6H or flatter.	
3.0 BRIDGES AND CROSSINGS			
General	• Replacement bridges, box culverts, and steel plate arch culvert crossings will be standard prefabricated structures, designed and supplied by a supplier.		• Per The Project Company
Strength I	• For modular highway bridges, and modular construction bridges carrying vehicular traffic and crossing over state highways, local roads, or railroads, the design vehicular live load must be HL-93 as specified in AASHTO-CA LRFD BDS Article 3.6.1.2.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Strength II	• For modular construction bridges, the design vehicular live load and special equipment loads are specified by the contractor. Load factors for Strength II as specified in AASHTOCA LRFD BDS must be applied.		• Caltrans - Memo to Designers 12-9 (Sep 2018)

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KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

ROADS, BRIDGES, AND CULVERTS
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
Strength III	• For modular highway and construction bridges, wind load must be as specified in AASHTOCA LRFD BDS Article 3.8.1.2 multiplied by a reduction factor of 0.84 corresponding to 10% probability of exceedance in 10 years.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Strength V	• For modular highway and construction bridges, the wind load must be as specified in AASHTO-CA LRFD BDS Article 3.8.1.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Fatigue I	• For modular highway bridges, and modular construction bridges carrying vehicular traffic and crossing over state highways, local roads, or railroads, the infinite fatigue life design requirements as specified in AASHTO-CA LRFD BDS Article 6.6.1.2.2 must be applied.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Extreme Event I	• For modular bridges designated as “standard”, seismic load must be as specified in Caltrans Memo to Designers 20-2 “Site Seismicity for Temporary Bridges and Stage Construction”. • Force capacities must be based on the expected material properties in accordance with Caltrans Seismic Design Specifications for Steel Bridges.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Extreme Event II	• Vehicular railing must be designed for TL-4 design forces as specified in AASHTO-CA LRFD BDS Article A13.2. The regulatory speed limit must be posted for 45 MPH or less. • All components in the load path of the modular bridge system must be designed for TL-4 design forces as specified in AASHTO-CA LRFD BDS Article A13.2.”		• Caltrans - Memo to Designers 12-9 (Sep 2018)
Service I	• For modular highway bridges designated as “standard”, the vehicular live load HL-93 deflection must not exceed the limit of span length/800.		• Caltrans - Memo to Designers 12-9 (Sep 2018)
3.2 Temporary Bridge Strengthening			
Fall Creek	Temporary intermediate support system to accommodate HL93 Vehicle Loads.		
Dry Creek	Temporary intermediate support system to accommodate HL93 Vehicle Loads.		
Bridge Access	• Open to public		
Impact Loads on Foundations	• Impact load of floating debris = 1000 lbs • Maximum Impact Force of Woody Debris on Floodplain Structures (USACE, 2002)		Technical Report ERDC/CRREL TR-02-2 - (USACE, 2002)
3.3 Temporary Construction Access Bridge - Daggett			
Roadway width	• 1 lane (18 ft)		
Foundations	• Designed to accommodate construction loads during bridge installation (loads provided by supplier) • Design Vehicle = HL93 • Maximum bearing reactions to be provided by supplier. • Check flood for analyzing structural stability at the extreme event limit state = 5% event • Abutment design as per AASHTO Section 11.6.		AASHTO
Erosion Protection	• As per California Bank and Shore Rock Slope		• California Bank and Shore Rock Slope Protection Design (2000)
Bridge Access	• Construction Traffic Only at Daggett Road Temporary Bridge.		
3.4 Materials			
Structural steel			ASTM A709
Minimum Tensile Yield Strength	• fy = 36 ksi		
Minimum Ultimate Yield Strength	• fu = 65 ksi		
Unit Weight	• γ_{STEEL} = 0.284 lb/in^3		
Cast-in-place concrete (CIPC)			AASHTO - 5.4.2.1.
28-day min. Compressive strength	• f'c = 4 ksi		
Unit Weight	• γ_{CONC} = 0.145 kcf	• Normal Weight with f'c ≤ 5.0 ksi	
Pre-cast reinforced concrete	By suppliers		
Reinforcing steel for CIPC			AASHTO - 5.4.3.1
Minimum Yield strength	• f'y = 60 ksi		
Unit Weight	• γSTEEL = 0.490 kcf		
8.0 AQUATIC ORGANISM PASSAGE			
Design Flows	High Design Flow Adult Salmonids: • 1% annual probable flood or 0.5*Q2; High Design Flow Juvenile Salmonids: • 10% annual probable flood or 0.1*Q2; Low Design Flow Adult Salmonids: • 50% annual probable flood or 3 cfs; Low Design Flow Juvenile Salmonids: • 95% annual probable flood or 1 cfs		• NMFS 2019 • CDFW Part IX

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TABLE A7.5

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

ROADS, BRIDGES, AND CULVERTS
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
Maximum Culvert Velocities	Stream Simulation Design Method - Mimic upstream hydraulic conditions Hydraulic Design Criteria for Max Juvenile Velocity: • 1 fps Max Adult Velocity: • varies with culvert length		• NMFS 2019 • CDFW Part IX
Minimum Flow Depth	Adult Salmonids minimum depth: 1 ft Juvenile Salmonids minimum depth: 0.5		• NMFS 2019 • CDFW Part IX
Crossing Criteria - Channel Form and Slope	• Crossing width < 1.5 Active Channel Width • Channel slope < 6% • Conveys sediment and debris		• CDFW Part XII • Technical White Paper 2017.06.20 • NOAA 2011 • NMFS 2019
8.1 Stream Design			
Rock Scour and Slope Protection	• Stable Rock Gradation based on USACE equations, • Side Slope >/= to 1.5 H :1 V • USACE Method for Steep Slopes for bed slopes >2% • Minimum blanket thickness >1.5*d50 or d100		• NCHRP Report 568 • USACE EM 1110-2-1601
Large Wood Structures	• Meet criteria described in Reservoir Restoration for Bank Stabilization		• USBR & ERDC National Large Wood Manual
Stream stabilization	• Design Flood = 1% PPE • Engineered Stream Bed Material sized using CDFW methodology • Active channel width equal to active channel width in unimpaired reaches • Overbanks <0.5*Active chanel width		• CDFW Part XIII Fish Passage Design and Implementation
9.0 CULVERTS			
9.1 Temporary/Permanent Culverts			
General	• Temporary Culverts and Permanent Culverts shall be designed in accordance with the appropriate references for each state.		• AASHTO
9.2 Hydraulic Capacity			
Permanent Culvert Design Flow	• 2% annual probable flood		• AASHTO
Permanent Culvert Check Flood	• 1% annual probable flood		• AASHTO
Temporary Bypass Flows	• Monthly 5% annual probable flood		
9.3 Design Loads			
Vehicle Load	• Culverts shall be designed for HL-93 vehicle loads		• AASHTO
9.4 Existing Culverts			
Existing culvert replacement	• Replace in kind when needed.		

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REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	27MAY'22	ISSUED WITH REPORT VA103-640/1-9	JOR	NB

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TABLE A7.6

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

MATERIAL DISPOSAL
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 DISPOSAL REQUIREMENTS AND LOCATIONS			
1.1 J.C. Boyle			
Excavated Embankment Materials	• Shall be disposed in the J.C. Boyle disposal sites	• Embankment riprap will be excavated and stockpiled for later use	
Concrete Rubble	• Concrete rubble shall be disposed in scour hole below power canal spillway		
	• Concrete rubble from J.C. Boyle powerhouse and penstock anchors shall be disposed in the J.C. Boyle tailrace and covered with native materials to blend with surrounding topography		
	• Concrete rubble in the scour hole shall be covered with a 4 ft minimum thickness cover		
1.2 Copco No. 1 and No. 2			
Concrete Rubble	• Concrete rubble from Copco No. 1 and Copco No. 2 dam shall be disposed in Copco disposal site • The disposal site shall be stripped of subsoil prior to rubble placement, and stockpiled to be used later to cover the disposal site		
	• Concrete footings from Copco No. 2 Woodstave Penstock shall be laid down and buried on site using the adjacent access road material		
	• Concrete rubble from Copco No. 2 powerhouse and penstock anchors shall be disposed in the Copco No. 2 tailrace and covered with native materials to blend with surrounding topography		
Woodstave Penstock	• Wood from the woodstave penstock will be transported off site and disposed of in a licenced facility		
1.3 Iron Gate			
Excavated Embankment Materials	• Excavated embankment materials shall be disposed in the spillway and in the disposal sites. The spillway shall be filled first to the maximum extent possible while still meeting the requirements for stability		
Concrete Rubble	• Concrete rubble shall be disposed of in the disposal sites and covered with a minimum 3 feet of excavated embankment material		
1.4 Common Criteria			
Partially Removed Concrete Structures	• Partially removed concrete structures shall be covered with a minimum of 2 ft of stable fill		
Cover	• The disposal sites shall be covered with fill and shall be designed to meet the ecological design criteria and blend into the landscape as naturally as possible		
Slope Stability	• Minimum required FOS = 1.5 for Long-term slope stability • Design earthquake for permanent construction		
Drainage	• Maximum exit gradient for seepage • Design storm for surface drainage and erosion control/protection design		

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0	27MAY'22	ISSUED WITH REPORT VA103-640/1-9	SDR	NB
REV	DATE	DESCRIPTION	PREP'D	RVW'D

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TABLE A7.7

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

DAM SITE PERMANENT WORKS
DESIGN CRITERIA

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Feature/Consideration	Criteria	Remarks	Reference
1.0 GENERAL			
Design Life	• 50 years		
Design Flood for River Channel Erosion Protection	• 1% probable flood	• Erosion protection will be provided for permanent fill slopes within the dam excavation footprints. Erosion protection is not required on bedrock slopes. • Habitat features: additional rock or other materials requested by Habitat Contractor for aquatic habitat purposes to be shown on the Habitat Contractor design documents.	
Seismic Parameters	• As per the STID for the respective sites		

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REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	27MAY'22	ISSUED WITH REPORT VA103-640/1-9	SDR	NB

APPENDIX B1
J.C. BOYLE HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

TABLE OF CONTENTS

1.0 INTRODUCTION

The J.C. Boyle Design appendix includes a summary of data design methodology, and other information used in the civil, hydrotechnical, and geotechnical design of the dam removal operations and structure evaluations at the J.C. Boyle Hydropower Facility.

Appendix B2 provides a summary of the hydrodynamic modeling completed to support the hydrotechnical design at J.C. Boyle, including CFD modeling and scour potential. Drawdown modeling results are included as Appendix G.

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

2.1.1 GENERAL

Material properties are 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) and the Existing Conditions Assessment Report (VA103-640/1-1). Earthwork Division 31 Technical Specifications provide material specifications for the construction materials that will be used for the project. Gradation curves for the construction materials are provided on Drawings G0050 and G0051.

2.1.2 MATERIAL TYPES

Excavated materials will require engineered storage and locally sourced materials will be used in establishing temporary access or providing cover for concrete as required for the project. These materials are sourced and utilized in the following applications:

- General Fill (Type E9/E9a/E9b) and Random Fill (E10) will be sourced from the embankment, historic cofferdam, forebay, power canal, penstock, and powerhouse excavations. They will be placed in the disposal sites, tailrace and scour hole and will also be used to cover concrete. These materials are assumed to require no processing.
- Erosion Protection (Type E7a/E7b/E7c) and Bedding materials (Type E6/E8) will be used to line the final river channel and for general erosion and sediment control best management practices. These materials will require sorting by particle size or processing to create conforming materials to the gradations as shown on Drawings G0050 and G0051.
- Select Fill (Type E4) and Class II Aggregate Base (Type E11) will be used for road construction. These materials will require processing to create conforming materials to the gradations as shown on Drawings G0050 and G0051.
- Concrete Rubble (Type CR1 and CR2) will be produced from demolition of the intake, power canal, forebay and powerhouse. The demolished concrete particle size requirements are shown on Drawing G0051, but generally these materials will not require any processing.

2.1.3 EVALUATION OF EXISTING CONCRETE

Evaluation of existing concrete and reinforcing steel is not required for the J.C. Boyle demolition and removal works.

2.2 RIVER CHANNEL DESIGN

2.2.1 CHANNEL PARAMETERS

A final graded river channel has been developed for the reach through the excavated J.C. Boyle embankment that will provide long-term fish passage. The final river channel meanders and undulates through the reach with an average grade of 1% and a minimum width of 90 ft over a channel length of approximately 700 ft, and is presented on Drawings C1230 and C1232. It is expected that all embankment fill, historic cofferdam fill, and remaining sediment will be mechanically removed. The final channel will utilize the natural channel rock outcrops on the right bank, removing the need for erosion protection on this bank. A portion of the embankment and concrete cutoff wall is to remain on the left bank and will require armoring. The final river channel is designed to a 1% probable annual flood wetted perimeter. On the left bank within the embankment footprint, above the storm flood wetted perimeter, the channel slopes will be continued at a maximum slope of 3H:1V and areas expected to be inundated during the 1% probable annual flood will be lined with bedding and erosion protection materials to mitigate scour. See Section 2.2.2 for further details. Final stabilization measures are required for any exposed Zone I core material, as shown on C1620.

The channel invert begins at the upstream historic cofferdam toe and ends at the known downstream toe of the embankment, as indicated by lidar imagery. If encountered conditions differ from those assumed in this design, the rockfill toe located on the downstream side of the embankment is to be evaluated and graded as required. The concrete cutoff wall will be removed down to bedrock.

2.2.2 EROSION PROTECTION DESIGN

Erosion protection is designed 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 in situ ground 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.

Riprap designs shown on the Drawings were based on 2D hydraulic modeling. Erosion protection for the final river channel at J.C. Boyle is designed for the post-drawdown 1% flood event. Channel characteristics and geometry were used to produce a velocity profile on the left bank and a range of expected velocities from the bottom of the channel to the water surface elevation. It is a requirement that the remaining portion of the embankment be stable, and therefore the fill and cut slopes of the channel must be protected against the long-term design flood. The hydraulics of the final channel were modelled to determine the design parameters for the required slope erosion protection.

The modified Maynord method was used to determine the size and thickness of erosion protection that is required to resist the maximum computed velocity, and the results are verified using other accepted methods. Rock density used for the design of erosion protective layers at J.C. Boyle is assumed to be 165 lb/ft^3. The resulting erosion protection, including D_{50}, layer thickness, key-in and required bedding is shown on Drawing C1230. At the upstream end of the erosion protection reach, the erosion protection will be extended up the bank from a minimum elevation of 3,739.8 ft, where the channel bed begins, to the maximum anticipated river elevation, plus 3 ft of freeboard, of 3,753.0 ft. At the downstream end of the erosion protection reach, the erosion protection will be extended up the bank from a minimum elevation of 3,736.9 ft, where the channel bed begins, to the maximum anticipated river elevation, plus 3 ft of freeboard, of 3,749.0 ft. The expected flood elevations are further discussed in Section 3 of this appendix. The lateral extents of the erosion protection details are shown on Drawing C1230.

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. The United States Department of Agriculture – Part 633 National Engineering Handbook methodology was used to determine bedding size requirements.

2.3 FINAL GRADING

In general, all areas disturbed by construction of the project components will be restored to final lines and grades as soon as practical. All disturbed slopes below the 1% flood elevations will be stabilized with erosion protection or other suitable means.

2.4 DIVERSION CULVERT STOPLOG REMOVAL

2.4.1 BLASTING STOPLOGS

Prior to initiation of Stages 3 and 4 drawdown, charges will be set from the downstream side of the diversion culvert stoplogs. To access the downstream side of the diversion culverts, the spillways must be inactive. To protect the downstream workers and to provide increased reservoir capacity, the spillway gates are to be closed. The timing of the work will depend on inflows into the J.C. Boyle reservoir. The holes will be drilled, and charges set in a manner that conforms with both blasting and demolition technical specifications.

Locations and dimensions of the diversion culvert stoplogs are shown on Drawings C1220 and C1221. The blast design, charging and detonation system will be completed by the Project Company.

2.4.2 DEBRIS MANAGEMENT

Floating debris control measures will 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 water surface levels are expected to stabilize at the staged drawdown surface elevations as defined in Section 3 of this appendix. Debris management and control measures may be implemented at these times to remove any deleterious material that could pose a potential risk to the drawdown and diversion operation.

2.5 CONSTRUCTION ACCESS

2.5.1 GENERAL

Two access roads are designed as part of the 100% DCD: the Powerhouse Road realignment, and the lower penstock access road rehabilitation. The lower penstock access road rehabilitation is an optional temporary upgrade and is defined as road reconstruction. The Powerhouse Road realignment (permanent) are defined as new road construction. The road construction design basis information is included in Design Criteria Appendix A7. It should be noted that an exemption was made to the design criteria grades for portions of the lower penstock access road. The existing maximum road grade is greater in some sections than the maximum design grade stated in the Design Criteria of Appendix A7.

The road construction details are shown on Drawings C1500, C1501, C1511, and C1512.

2.5.1.1 ROAD RECONSTRUCTION

Road reconstruction is the re-opening and upgrade of existing site access roads. Reconstruction upgrades will include brush clearing, ditch cleaning, widening of the road prism, surfacing, and drainage structure installation. Generally, the existing road location will be utilized, minimizing cut-and-fill activities and material quantities. No wearing course or aggregate materials are specified for temporary road reconstruction. The road clearing width will require the removal of vegetation on both sides of the road prism to allow the road surface to dry more readily and to improve visibility on the road. Clearing widths will be minimized, but still allow for safe and stable reconstruction of the road.

2.5.1.2 NEW ROAD CONSTRUCTION

New construction may include falling right-of-way, clearing, and grubbing, stripping, log decking, stump removal, and construction of sub-grade, ditches, and drainage structures. The location of the new roads will be based on topographic features and surface material, and the control point of where that road is accessing. Road grades will be constructed for easier access during construction and to reduce long-term maintenance cost. Similar to reconstruction, new roads will require the establishment of a clearing width or right-of-way.

Subgrade construction consists of using local material to construct a stable surface to form the base of the road. The new roads are assumed to be constructed using a cut-and-fill technique. Material is cut from the uphill side of the road and is placed as fill on the downhill side of the road. The material is then compacted using the tracks of the excavator or bulldozer. New roads are to be capped with Type E11 material with typical sections and details shown on Drawing C6001.

2.5.2 ROAD DRAINAGE

Existing road drainage structures located on project roads (roads to be used for the construction of the project) will be maintained by the Project Company for the duration of the construction period. New drainage structures include drainage swales and drainage culverts as shown on Drawings C1620 to C1624. Swales are required around any design fills to minimize erosion and culverts are required along the power canal alignment to facilitate drainage over the buried concrete. Construction of a matching swale is required on the uphill side of the newly constructed Powerhouse Road Realignment. A culvert exists at the powerhouse which diverts flow from the base of the penstocks and conveys it to the tailrace. If this culvert is removed or blocked during cut-and-fill activities, the Project Company is to replace this culvert with one of equal capacity to convey slope runoff through the powerhouse fill and into the Klamath River. Typical details for bedding and slope of culverts are shown on C1622.

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

The depth-area-capacity relationships for the J.C. Boyle reservoir are based on the 2018 bathymetric survey (NAVD88 datum) and are shown on Drawing C1056. The reservoir capacity at elevations relevant to the J.C. Boyle 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 Normal Operating Level	3,796.7	3,168
Minimum Normal Operating Level	3,791.7	1,758
Spillway Crest	3,785.2	858
Power Intake Invert	3,771.7	160
Historic Cofferdam Crest	3,770.0	124
Diversion Culvert Invert	3,755.2	0.1

3.2 OUTLET STRUCTURE DISCHARGE RATING CURVES

Discharges during the drawdown stages will be made through the existing outlets at the intake structure: three spillway bays, the power intake, and the two diversion culverts. No alterations will be made to the existing outlets except for the removal of the concrete stoplogs upstream of the two diversion culverts. The development of the discharge rating capacities for the outlets are detailed in Appendix B2. The J.C. Boyle discharge rating curves are presented on Drawing C1056.

3.3 RESERVOIR DRAWDOWN SEQUENCING

The operations of the J.C. Boyle reservoir during drawdown and post-drawdown will achieve successful lowering of the reservoir impoundment and provide the required flood control. The reservoir drawdown sequencing will be completed over four stages as outlined in the design report. The drawdown model (detailed in Appendix G) assesses the drawdown sequencing in terms of reservoir water surface levels under a range of hydrologic conditions. The following sections discuss the results of the drawdown model and the implications to the project.

3.3.1 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN

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

- Individual year simulations are provided in the attached J.C. Boyle Simulated Drawdown Figures 1 through 36. These plots indicate the following:
  - Reservoir water surface levels.
  - Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway, power intake, and flows through the diversion culverts).
- Maximum daily reservoir water surface level daily non-exceedance percentiles (percentiles) are shown on Figure 3.1, and on Drawing C1056. This figure represents the results from all 36 model simulations as non-exceedance percentiles to summarize the distribution of the results on any given day of the simulations. These results do not represent a single simulation, but are based on all model simulations.

- Ensemble figures with each line representing a single model simulation for a different year, (also referred to as spaghetti figures) are shown on Figure 3.2. This figure overlaps the simulated reservoir water surface levels on a common x-axis that spans January 1 to September 30. Each line represents a single model simulation.

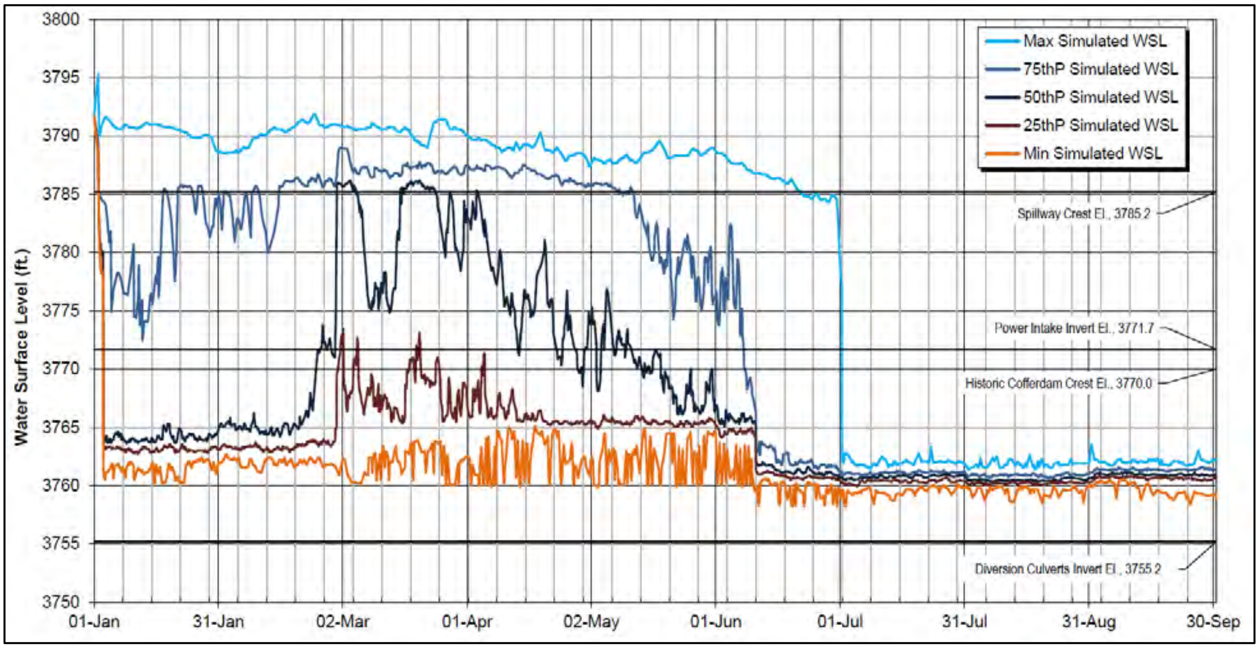


Figure 3.1 J.C. Boyle Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

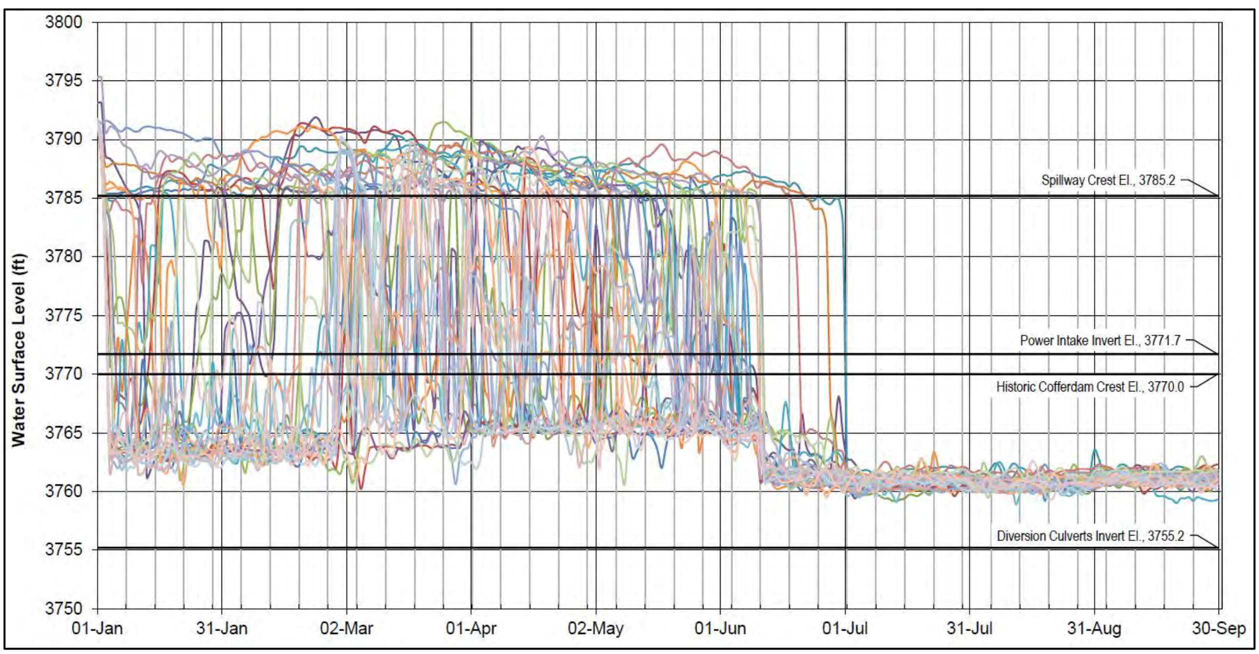


Figure 3.2 J.C. Boyle Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

The simulated water surface levels on Figure 3.1 and Figure 3.2 show that there is a substantial reduction in the reservoir water levels in mid-June with the majority of the simulated years achieving sustained water levels below the historical cofferdam crest in early June. This is a function of initiating Stage 4 of drawdown on June 10 and the inflow hydrology which indicates a reduction in streamflow for the second half of June (Appendix A6). There are three model years (1983, 1984, and 1998) that show elevated reservoir water surface levels past June 15. However, in these years, the reservoir water surface levels do drop below the crest of the historic cofferdam prior to July 1. Stage 4 can be initiated as early as January once the charges are set on diversion culvert #2 and Stage 3 drawdown is complete, so extending the initiation until June 10 is not a requirement of this design.

Figure 3.2 shows that there are large fluctuations in the reservoir water surface levels from January through June as a function of the inflow hydrology and the J.C. Boyle reservoir. The J.C. Boyle reservoir has a small storage capacity and the reservoir can refill quickly during the higher flow months typically in January through May resulting in spillway flows. Lower reservoir levels will be sustained below the crest of the historic cofferdam during Stage 4 and after June 1 depending on the hydrologic conditions.

Figure 3.3 shows reservoir drawdown distributions for various relevant facility components, which represent cumulative percentages of model simulations indicating the dates when the reservoir water surface level is lower and sustained below a certain elevation. The actual date when the water surface elevation will be sustained in the drawdown year can be different than shown on Figure 3.3, depending on the hydrological conditions and the drawdown sequencing applied. The water levels shown on Figure 3.3 are based on average daily conditions for the 36 drawdown model simulations. Low probability flood flows (e.g., the 5% or 1% probable flood flows) may have not 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 3,792.1 ft – represents embankment phase 2 crest, at which point the embankment removal down to the June 1 1% probable flood at elevation 3,790.0 ft can start. Approximately 97% of the simulations have reservoir water levels sustained below the embankment Phase 2 crest by January 2, and 100% of the simulations by January 3.
- Elevation 3,785.2 ft - represents the spillway crest. Approximately 45% of the drawdown simulations have reservoir water levels sustained below the spillway crest by April 20, 91% of the simulations by June 10, and 100% of the simulations by June 20. This indicates that diversion culverts become more accessible during the late spring and summer months.
- Elevation 3,771.7 ft – represents the power intake invert. Approximately 40% of the simulations have reservoir water levels sustained below the power intake invert by June 1, 91% of the simulations by June 10, and 100% of the simulations by July 1.
- Elevation 3,770.0 ft – represents the crest of the historic cofferdam. Approximately 40% of the simulations have reservoir water levels sustained below the crest of the historic cofferdam by June 1, 91% of the simulations by June 10, and 100% of the simulations by July 1. This indicates that the height of the assumed cofferdam is appropriate for diverting flows during the anticipated embankment removal construction window.

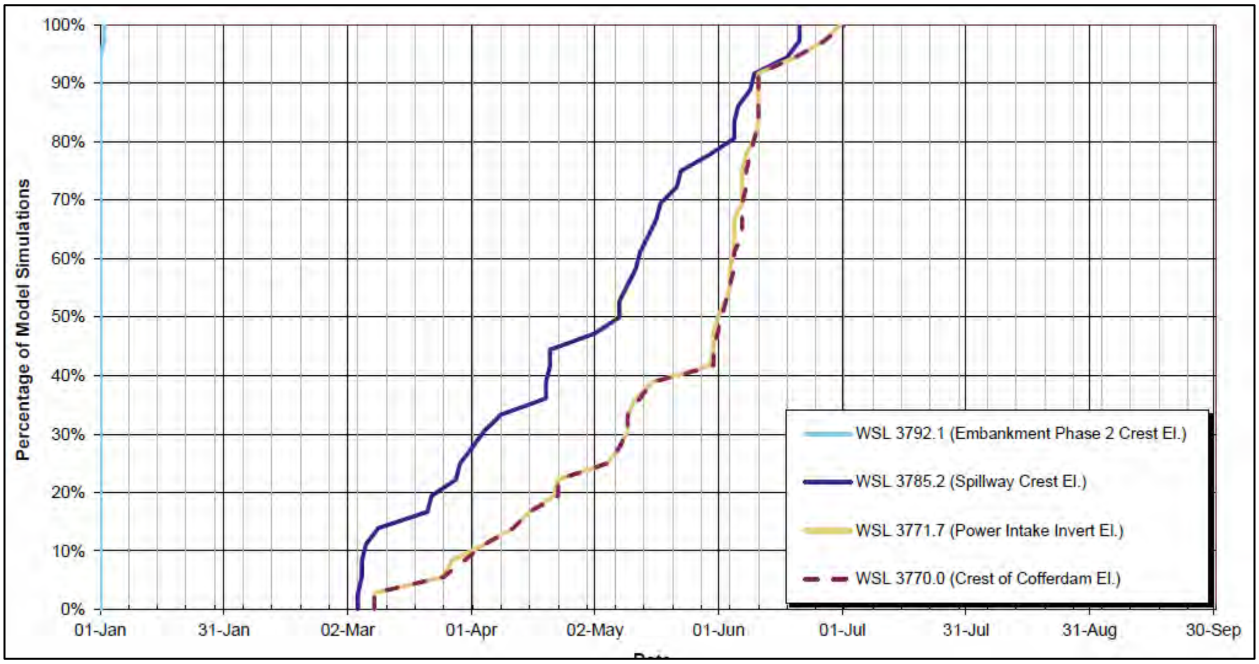


Figure 3.3 J.C. Boyle Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below Various Relevant Elevations

The results of the reservoir drawdown model are outlined below for each stage of drawdown. It should be noted that the rules set for the drawdown model do not include coordination with the Upper Klamath River Basin (Keno Dam and/or Klamath Lake) or initiation of Stage 4 drawdown prior to June 10 which is acceptable if Stage 3 drawdown is complete and the charges have been set on the downstream side of the second diversion culvert.

- Stage 1 - Spillway Gates and Power Intake:
  - The spillway gates and power intake are used to target a drawdown of 5 ft/day, and drawdown occurs over one to two days to reach the spillway crest (El. 3,785.2 ft).
- Stage 2 - Power Intake:
  - The reservoir water levels are controlled by the discharge capacity of the power intake and are dependent on the reservoir inflows.
  - Outflows through the power intake are limited to 2,850 cfs. The total outflow can be higher if the spillway is still engaged.
  - The reservoir can be lowered up to 5 ft when the power intake is initially opened in drier climatic conditions, as seen in the simulated results for 1990 and 2015.
  - The drop in reservoir water surface levels is not as large in wetter climatic conditions, and the water level may be maintained above the spillway crest, as seen in simulated results for 1984 and 1997.

  - The duration of Stage 2 is determined by the hydrologic conditions and when the downstream side of the diversion culverts can be accessed to successfully remove the stoplogs. Approximately 75% of the simulations indicate that the duration of Stage 2 is limited to less than a week under the simulated drawdown methodology. In approximately 10% of simulations, Stage 2 was limited to 2 weeks (1982, 1996, 1998, and 2002). Years with much higher than average inflows (wet years) indicate that Stage 2 can be extended for many weeks and beyond April 1 into May and June. This is observed in less than 15% of the simulated years (1983, 1984, 1985, 1997, and 2006). In case such a wet year occurs during the drawdown year, the demolition of the powerhouse and conveyance facilities at J.C. Boyle could be delayed, unless the inflows to the reservoir are managed by coordinating with the Upper Klamath River Basin (Keno Dam and/or Klamath Lake).
  - River forecasting and coordination with the Upper Klamath River Basin may be required to limit the duration of Stage 2. Reduced inflows to the reservoir will result in lower reservoir water levels, therefore, allowing for safe access to the downstream end of the diversion culverts. The steady-state inflow to the reservoir to maintain a water level at the spillway crest with the power intake is approximately 1,600 cfs for Stage 2. Alterations to the flow releases from the Upper Klamath River Basin outside of the 2019 BiOp flows were not simulated with the drawdown model.
- Stage 3 – Opening of the First Diversion Culvert:
  - A temporary drop in reservoir water surface level and an increase in outflow is observed when the diversion culvert is opened. The reservoir water surface levels will drop below 3,765 ft under most hydrological conditions when the diversion culvert is opened. Wetter hydrological conditions will result in a lesser drop in the reservoir level (e.g., 1998 drops to approximately 3,770 ft as there is an increase in reservoir inflows shortly after removing the diversion culvert stoplogs).
  - Outflows through the diversion culvert are limited to approximately 2,400 cfs prior to engaging the spillway crest. Total outflows in Stage 3 can be higher if the spillway is still engaged.
  - The reservoir water surface level is likely to increase periodically after opening the first diversion culvert. Nearly 90% of the model simulations indicate that the spillway will be reengaged at some point during Stage 3 if the initiation of Stage 4 is extended until June 10.
  - The drawdown model report (Appendix G) notes that under the drawdown operating criteria evaluated for the drawdown model, in some years both diversion culverts open on the same date (June 11). Under these hydrological conditions, coordination with the Upper Klamath River Basin would be required to permit the opening of the first diversion culvert on an earlier date, therefore initiating Stage 3 of drawdown prior to June 10.
- Stage 4 – Opening of the Second Diversion Culvert:
  - Stage 4 represents the final stage of drawdown.
  - Stage 4 is initiated on or after June 10 and when the reservoir water surface level below the spillway crest. The steady-state inflow to the reservoir to maintain a water level at the spillway crest with the first diversion culvert open is 2,210 cfs.
  - Over 90% of the drawdown model simulations indicate that the second diversion culvert is opened on June 10. Under wet hydrological conditions, such as those in simulation years 1983, 1984, and 1998, the opening on the diversion culvert is delayed – the latest date resulting from the simulations is June 29.

- The reservoir water surface levels will drop below 3,763 ft under most hydrological conditions when the second diversion culvert is opened. Wetter hydrological conditions will result in a lesser drop in the reservoir level (e.g., 1993, 1998, 1999 and 2011 drops to approximately 3,765 ft with the initial opening of the diversion culvert).
- After the diversion culvert has been opened, and after July 1, the reservoir water surface levels remain low and are within the range of 3,758 ft to 3,763.5 ft for all model simulations.

3.3.2 SCOUR POTENTIAL

The CFD study presented in Appendix B2 indicates that during the drawdown operation there will be varying levels of scour potential immediately upstream of the outlet structures. Scour potential was predicted by comparing simulations of bed shear stress from both the Flow-3D and HEC-RAS 2D models. Shear stress magnitude figures are shown in Appendix B2.

It is anticipated that the flow will have the potential to scour medium to very coarse gravels (up to 1.3 inches) during Stage 2 and Stage 3 of drawdown. The bed shear stresses in the reservoir stimulated during Stage 4 have the potential to mobilize small cobbles (2.5 inches). The scour potential is the highest at low flows when a minimal headpond is present, resulting in the ability to mobilize large cobbles (5-10 inches). Given the 9.5 ft by 10 ft opening of the diversion culverts, blockage from mobilized bed material is not anticipated.

Modelled flow paths are shown on Figure 12 and Figure 13 in Appendix B2 during low flows. It should be noted that due to current sediment deposits, water is flowing directly over the historic cofferdam rather than being diverted around it. It is anticipated that scour of the bed will alter the current sediment geometry. Mechanical evacuation of sediment may be necessary and is to be evaluated post-drawdown.

3.4 EMBANKMENT REMOVAL AND STEADY-STATE WATER SURFACE LEVELS

3.4.1 GENERAL

Design criteria and hydrology determine the projected water surface levels at the J.C. Boyle Facility. The water surface levels ultimately dictate the removal schedule for the components of the facility in direct contact with the Klamath River. The only components of the facility in direct contact with the river are at the J.C. Boyle dam – namely the intake, spillway, diversion culverts, embankment, and historic cofferdam. The steady-state water surface levels at these components are presented below.

3.4.2 INTAKE AND EMBANKMENT WATER SURFACE ELEVATIONS

Flood water surface levels at the intake and embankment are shown on Drawing C1055 for steady-state inflows. The statistical flood flows (high water) are based on peak instantaneous flows while the daily average flows are average flows over a 24-hour period. The levels are calculated using the discharge rating curves developed for the outlet structures (Drawing C1056 and Appendix B2). The levels may differ depending on the shape and volume of the flood flow hydrographs and the attenuation effects of the reservoir. It should be noted that the spillways must remain operational until June 15, during which time the 1% probable flood water surface level with freeboard is at or above the spillway invert elevation of 3,785.2 ft.

The water levels at the intake are lower compared to the embankment water levels for flows less than 3,000 cfs, and at low flows, the water surface levels at the intake and embankment will be visually distinguishable, as discussed in Section 3.3.2 above and Appendix B2.

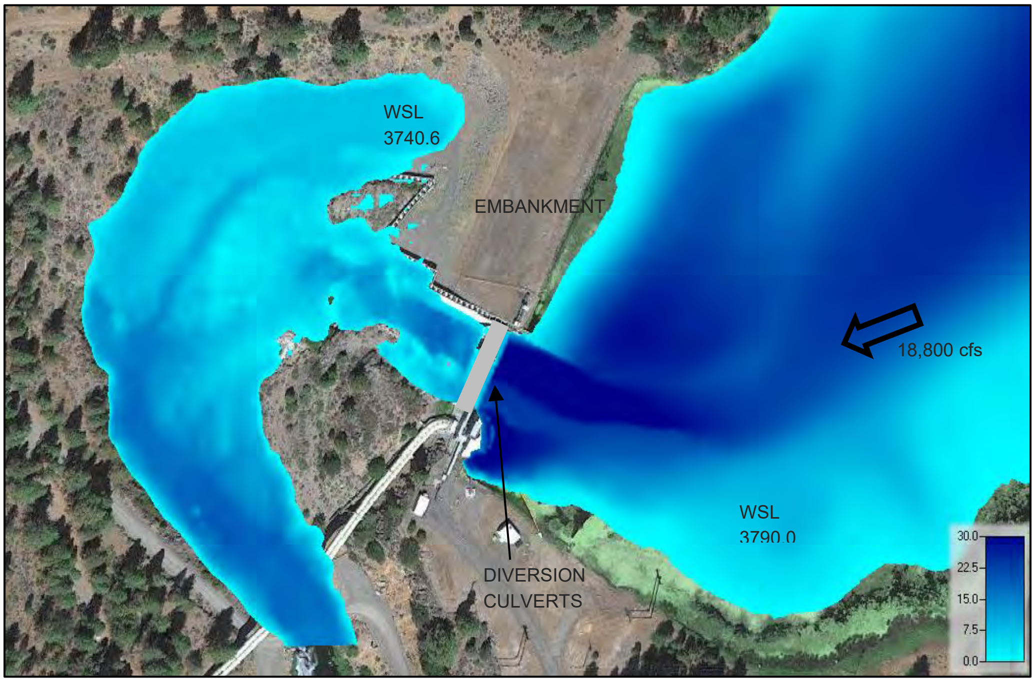
3.4.3 EMBANKMENT AND HISTORIC COFFERDAM WATER SURFACE ELEVATIONS

Flood water surface levels at the embankment and historic cofferdam are shown on Drawing C1231. The levels shown correspond to the embankment steady-state water levels as shown on C1055. The Staged crest elevations have been designed for the 1% flood with 3 or 1 ft freeboard (depending on the period). This assumes that the sediment, in its current state, remains and is not removed via scour during drawdown.

Little is known on the design or condition of the historic cofferdam. A 2018 bathymetric survey (NAVD88 datum) indicates that areas of the dam are partially eroded. Some fill placement may be required to provide a uniform crest width and slopes following cofferdam assessment post-drawdown. The staging design assumes that the crest elevation is 3,770.0 ft which is anticipated to contain all 1% flow events between July 16 and September 15 and all 5% flow events between June 15 and September 30. Mechanical evacuation of sediment may be necessary to ensure the functionality of the cofferdam.

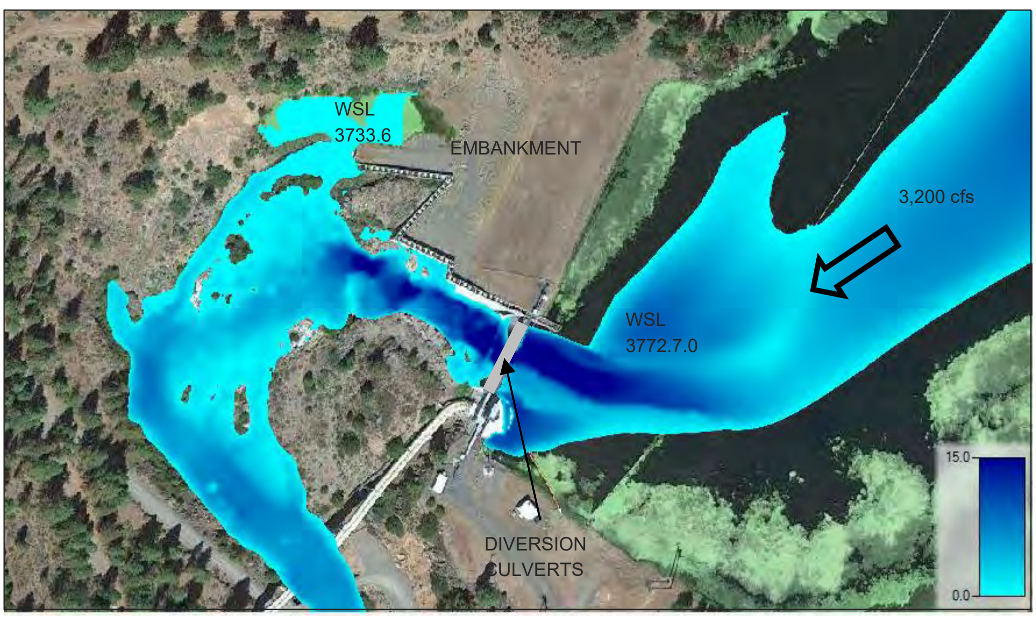
3.4.4 EMBANKMENT TAILWATER LEVELS

A hydrodynamic model was developed to investigate the tailwater surface levels downstream of the embankment once both diversion culverts are opened post-drawdown. The model was completed using HEC-RAS 2D with a Manning's n of 0.04. The model was run at a flood of 18,800 cfs, which is greater than the 1% probable annual flood, to evaluate the potential of backwatering of the downstream toe of the embankment. The model was also run at a flood of 3,200 cfs, which is the largest monthly 1% probable flood from July 1 to September 30, to evaluate the water surface levels on the downstream side of the embankment during the deconstruction period. The resulting tailwater levels indicate that the pond at the downstream toe of the dam could be hydraulically connected to the diversion culvert outflow via the original river channel during large storm events. The resulting water depths and water surface levels are shown on Figure 3.4 and Figure 3.5 for the 18,800 cfs and 3,200 cfs floods, respectively.



NOTES:
1. DEPTHS ARE IN FEET.
2. WSL: WATER SURFACE LEVEL.

Figure 3.4 J.C. Boyle Reservoir Tailwater Depths – Drawdown Stage 4



NOTES:
1. DEPTHS ARE IN FEET.
2. WSL: WATER SURFACE LEVEL.

Figure 3.5 J.C. Boyle Reservoir Tailwater Depths – Deconstruction Period

3.4.5 KEY INTAKE AND EMBANKMENT REMOVAL TIMING

Calculated water surface elevations and design criteria are used to determine the earliest removal date for key intake and embankment removal items in Table 3.2. The embankment removal work is broken up into phases that represent dates corresponding to water surface levels. Each phase has a designated removal volume and the staged elevations are shown on Drawings C1231 and C1234 to C1239.

Table 3.2 Key Intake and Embankment Elevations and Removal Timing

Removal Item	Elevation (ft)	Design Flood Event	Earliest Removal Date	Phased Volume (yd^3)	Haul Location	Comments
Spillway Gates and Trunnions	3,790.0	-	Jan 1	-	Scour Hole	Trunnions and spillway gates are not necessary for spillway operation and can be removed after drawdown.
Diversion Culvert #1 (Drawdown Stage 3)	3,755.2	-	Varies	-	-	See drawdown section (Stage 3).
Embankment Removal **Phase 1**	-	1% Probable Flood + 3 ft freeboard	Mar 15	4,400	Penstock Cover and Scour Hole	Remove erosion protection material from downstream face of the dam.
Embankment Removal **Phase 2**	3,792.1	1% Probable Flood + 3 ft freeboard	Jun 1	8,500	Penstock Cover and Scour Hole	Remove embankment to June 1 1% probable flood with 3 ft freeboard
Diversion Culvert #2 (Drawdown Stage 4)	3,755.2	-	Varies	-	-	See drawdown section (Stage 4).
Embankment Removal **Phase 3**	3,784.7	1% Probable Flood + 3 ft freeboard	Jun 15	13,200	Penstock Cover and Scour Hole	Remove embankment to June 15 1% probable flood with 3 ft freeboard
Spillway Structure	3,785.2	1% Probable Flood + 3 ft freeboard	Jul 1	-	Scour Hole	Remove spillway and intake structure to max removal elevation – maintain 15 ft width for access to left bank
Abutment Left Wall Phase 1	3,785.2	1% Probable Flood + 3 ft freeboard	Jul 1	-	Scour Hole	Match left wall elevation to spillway and elevation.
Embankment Removal **Phase 4**	3,776.7	1% Probable Flood + 3 ft freeboard	Jul 1	18,700	Left Bank Disposal Area	Remove embankment to July 1 1% probable flood with 3 ft freeboard
Embankment Removal **Phase 5**	3,770.7	1% Probable Flood + 1 ft freeboard	Jul 15	17,930	Left Bank Disposal Area	Criteria changes from 1% probable flood with 3 ft freeboard to 1% probable flood with 1 ft freeboard. Remove embankment to July 15 1% probable flood with 1 ft freeboard
Embankment Removal **Phase 6**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	120,832	Left Bank Disposal Area	Remove remaining embankment and silt. Excavate final channel to lines and grades shown on C1230. Stockpile material for eventual placement in diversion culvert channel and to bury intake concrete (Phase 9).
Evaluate/Grade Downstream Rockfill **Phase 7**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	-	-	Evaluate rockfill for use in final channel following removal Phase 6 and grade as required.
Install Erosion Protection **Phase 8**	3,770.0	1% Probable Flood + 1 ft freeboard	Aug 1	-	-	Install erosion protection and bedding material on the left bank of the final river channel.
Historic Cofferdam Breach **Phase 9**	3,755.2 (min)	-	Sep 1	4,900	Right Bank Disposal Area	To start no earlier than September 1 and be completed no later than September 30. Breaching of the historic cofferdam must take place after the final channel excavation and erosion protection installation is substantially complete.
Intake Cover **Phase 10**	-	-	Sep 1	5,000	Left Bank Disposal Area	To occur after cofferdam breach and substantial completion of the Final River Channel. Place material in diversion culvert channel and bury intake concrete.

3.5 FINAL RIVER CHANNEL WATER LEVELS

Channel characteristics and geometry of the J.C. Boyle final river channel presented in Section 2.2 and on Drawings C1230 and C1232 were used to develop a hydrodynamic model to determine the stage-discharge relationship for the post-dam removal period.

The resulting stage-discharge relationship is shown on Figure 3.6 at the location of the current dam centerline. A sensitivity of the model was completed using Manning's n roughness of 0.03 and 0.06 to account for potential variability in roughness elements added to the channel to provide localized habitat elements. The results of the sensitivity analysis are also included on Figure 3.6.

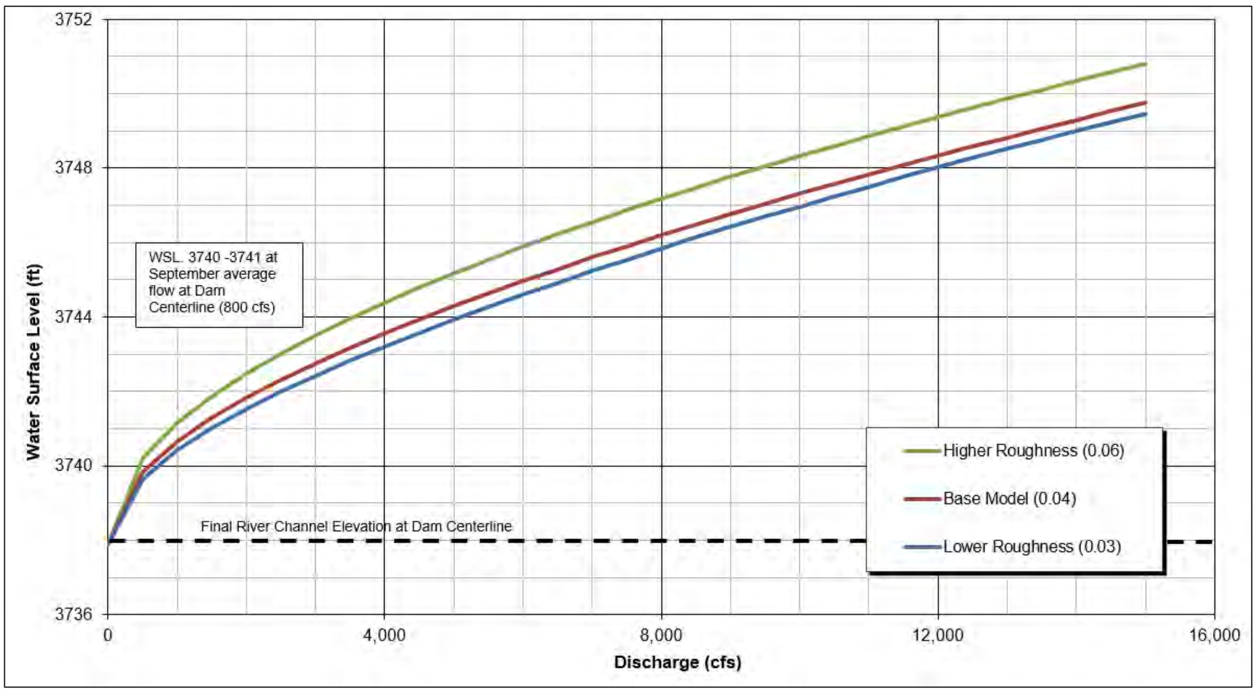


Figure 3.6 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

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

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 15 - 30
Statistical High Water (Flood Conditions)	5% Probable Flood	2,100	2,400	2,900	3,300	3,800	4,400
	20% Probable Flood	1,700	1,900	2,200	2,400	2,600	2,900
	50% Probable Flood	1,400	1,500	1,700	1,700	1,800	2,000
Mean Flow for Specified Time Period		800	790	810	890	980	950
Flow Condition		Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline[1]					
Statistical High Water (Flood Conditions)	5% Probable Flood	3,741.9	3,742.2	3,742.7	3,743.0	3,743.4	3,743.9
	20% Probable Flood	3,741.5	3,741.7	3,742.0	3,742.2	3,742.4	3,742.7
	50% Probable Flood	3,741.2	3,741.3	3,741.5	3,741.5	3,741.6	3,741.8
Mean Flow for Specified Time Period		3,740.3	3,740.3	3,740.3	3,740.5	3,740.6	3,740.6

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 3,738 ft.

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

Flow Condition		Discharge (cfs)
Statistical High Water (Flood Conditions)	1% Probable Flood	14,200
	5% Probable Flood	11,700
	20% Probable Flood	8,500
	50% Probable Flood	7,000
Annual Flow Duration 25% of Time Equaled or Exceeded		1,460
Mean Annual Flow		1,390
Annual Flow Duration 75% of Time Equaled or Exceeded		690
Flow Condition		Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline
Statistical High Water (Flood Conditions)	1% Probable Flood	3,749.4
	5% Probable Flood	3,748.2
	20% Probable Flood	3,746.5
	50% Probable Flood	3,745.6
Annual Flow Duration 25% of Time Equaled or Exceeded		3,741.2
Mean Annual Flow		3,741.1
Annual Flow Duration 75% of Time Equaled or Exceeded		3,740.1

NOTES:

1. FINAL RIVER CHANNEL BED AT DAM CENTERLINE IS AT ELEVATION 3,738 ft.

3.6 TAILRACE BACKFILL

3.6.1 STEADY-STATE WATER SURFACE LEVELS

A hydrodynamic model was developed to investigate the water surface levels at the backfilled tailrace once both diversion culverts are opened post-drawdown. The model was completed using HEC-RAS 2D with a Manning's n of 0.04. The model was run with average June and July flows, which corresponds to the anticipated period of construction, to evaluate the water surface levels on the tailrace buttress during the construction period. The resulting water surface levels indicate that the average water surface level for June and July has an approximate elevation of 3331.0 ft. This elevation is shown on Drawing C1411.

3.6.2 EROSION PROTECTION DESIGN

Using the modified Maynord method as described in Section 2.2.2, the tailrace erosion protection is designed for the post-drawdown 1% probable annual flood event. Channel characteristics and geometry was used to produce a velocity profile on the tailrace buttress and a range of expected velocities. It is a requirement that the infilled tailrace outer slope be stable, and therefore the fill slope must be protected against the long-term design flood. The 1% flood event water surface level is shown on C1411 and the analysis indicates that the outer 2 feet of the fill slope must consist of a low fines material. The maximum velocity is estimated to be 1 ft/s during the 1% flood event, assuming a Manning's n value of 0.04.

4.0 GEOTECHNICAL

4.1 DAM EMBANKMENT STABILITY DURING DRAWDOWN

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

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

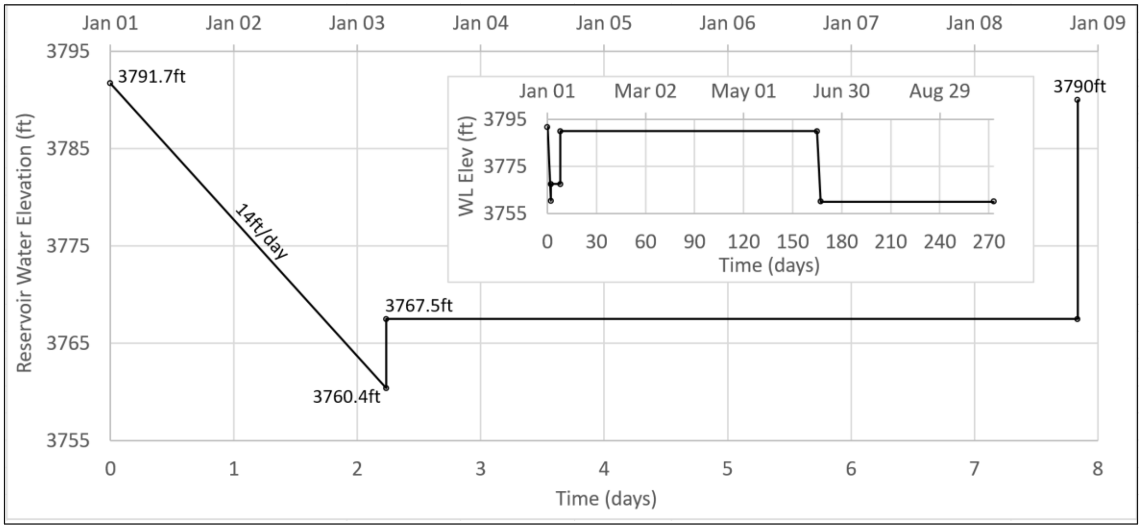


Figure 4.1 Generalized Drawdown Curve For Stability Analyses

Material properties used in the analyses are shown in Table 4.1 and include both base case and sensitivity values. A sensitivity check was completed on a second model by making changes to material properties that are shown in Table 4.1.

Table 4.1 Material Properties for Drawdown Stability Assessment

Material	Horizontal Hydraulic Conductivity (ft/s)	Vertical:Horizontal Hydraulic Conductivity Ratio	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core	5.61e-06	1 (0.1)	120	27 (19)	0
Shell	2.17e-02 (1.32e-04)	1 (0.5)	130	34	0
Upstream Riprap	3.41e-02 (1.32e-04)	1	140	34	0
Downstream Riprap	3.41e-02	1	140	34	0
Filter Blanket	3.41e-02	1	125	35	0
Waste Rock Fill	2.17e-02	1	145	40	0
Bedrock	3.28e-10	1	impenetrable		

NOTES:
1. SENSITIVITY CHECKS COMPLETED WITH VALUES IN PARENTHESES.

The analysis model geometry is shown on Figure 4.2. Three scales of slip were considered in the LEA. The first was a full-height slip, extending from the dam crest to the upstream toe. The second was a smaller slip, involving the lower slope or from the bench at elevation 3783.7 ft to the upstream toe. The third was a smaller slip that involved the upper slope, extending from the dam crest to the bench at elevation 3783.7 ft.

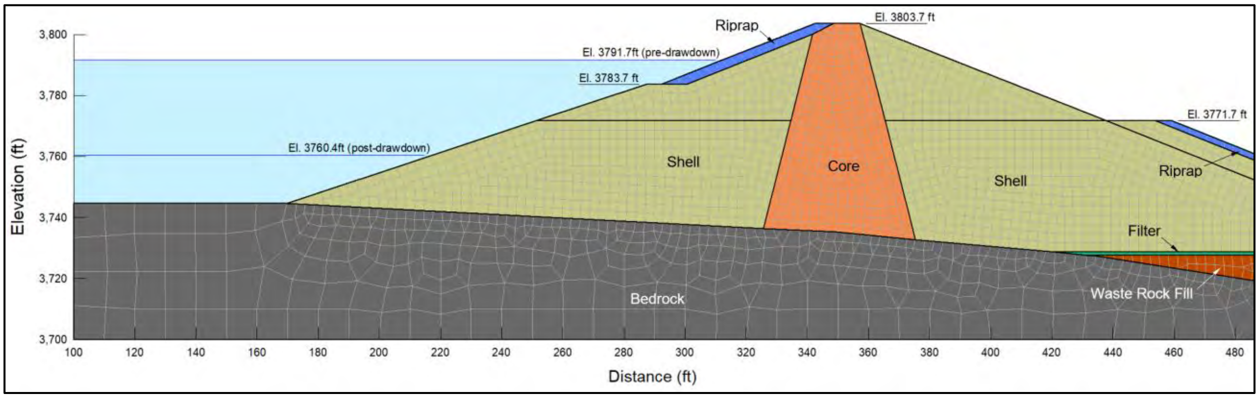
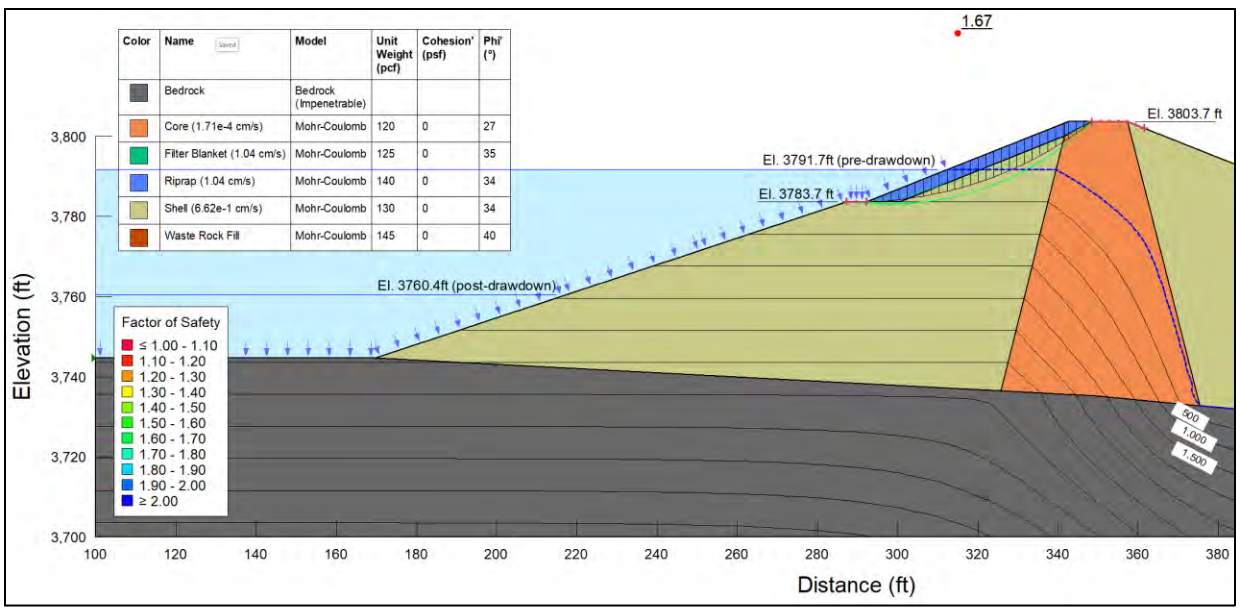


Figure 4.2 Drawdown Analysis Model Geometry

The stability results indicate the lowest FOS for the three scales of slips is 1.7 for the base case properties and 1.4 for the sensitivity check. Both values correspond to the GLE method, which produced slightly lower FOS values than the Spencer method. These results indicate the dam embankment is expected to be stable during the defined drawdown curve.

The upper slope slip governs stability for base case properties with the critical slip shown on Figure 4.3, along with pore pressure contours in psf. The timestep associated with the critical slip is at the beginning of drawdown.

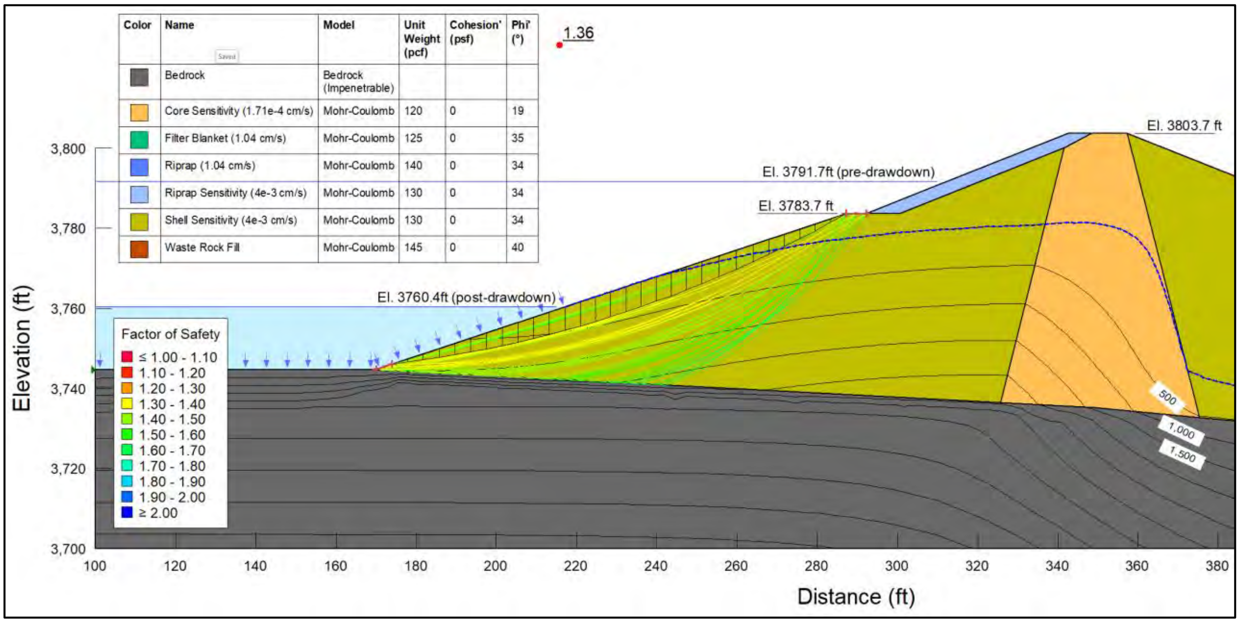


NOTES:

1. ONLY SLIPS WITH FOS < 1.7 ARE SHOWN.

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

The lower slope slip governs stability for the sensitivity check. The critical slip is shown on Figure 4.4 with pore pressure (psf) contours included. The timestep associated with the critical slip is about 2.2 days after drawdown commences, which coincides with the minimum water elevation (3,760.4 ft) used in the analyses.



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

Figure 4.4 Sensitivity Check of Drawdown Stability Results for Lower Slope Slip

Stability results are summarised in Table 4.2 for the three scales of slips. Results for the GLE method are reported since it produced the lower FOS values.

Table 4.2 Factor of Safety Results for Drawdown Stability

Slip Scale	Base Case Properties	Sensitivity Check
Full Height	1.85	1.49
Lower Slope	1.87	1.36
Upper Slope	1.67	1.58

4.2 EXCAVATION SLOPES

4.2.1 STABILITY OF THE EMBANKMENT DURING EXCAVATION

The embankment will be removed from the top down, maintaining the current slopes and toes of the embankment. Following excavation of the embankment to elevation 3,770.7 ft, the final portion of the embankment will be excavated as shown on Drawing C1238 to create the final river channel. This excavation methodology is prescribed to ensure that embankment stability is maintained through the removal process, until the embankment is substantially removed. The embankment removal sequence is shown on Drawings C1231 and C1234 to C1239.

4.2.2 RIVER CHANNEL

The left bank of the channel is excavated in embankment fill materials, while the right bank and channel bottom are excavated to bedrock. This appendix describes the geotechnical considerations of the design of the left bank excavated slopes in the embankment cut. Refer to Section 2 of this appendix for the civil design of the river channel.

The existing embankment is comprised of an exterior layer of riprap and bedding, Zone II shell zones and a Zone I core zone. The riprap and bedding materials were investigated, as summarised in the Geotechnical Data Report (VA103-640/1-2) and is expected to meet E7 erosion protection and E8 bedding material specifications but will require sorting by particle size. Zone II is expected to meet the criteria of General Fill – Type E9b, while Zone I is expected to meet the criteria of General Fill – Type E9 with no processing. Material specifications are available in the Project Technical Specifications and shown on Drawings G0050 and G0051. The following excavation slopes criteria are used in the design:

- Min. 1.5 H:1.0 V excavation slopes for all slopes located in alluvium and highly weathered rock.
- Min. 1.0 H:1.0 V excavation slopes for all slopes located in weathered rock.
- Min. 0.5 H: 1.0 V excavation slopes for all slopes located in competent rock.

Riprap and bedding materials stored from embankment excavation may be used as erosion protection for the final river channel. Dam core Zone I left in place will require final stabilization treatment as shown on Drawing C1620.

Stability of the final excavated slope of the dam embankment at the left bank was assessed by LEA for two loading conditions: static long-term and yield acceleration (ky) determination for approximating seismic displacement. The acceptance criteria require a FOS of 1.5 for static long-term stability and FOS of 1.0 for yield acceleration determination without strength reduction. In addition, STID-8 (PacifiCorp, 2015a) indicated displacements of 2 ft are acceptable according to a FERC guideline for the operating dam.

Static short-term stability was not analyzed since the design deconstruction staging entails embankment removal in horizontal layers from the right bank towards the left bank. This sequence and specific direction, in combination with expected pore pressure dissipation following reservoir drawdown, should provide reasonable confinement (buttressing) while promoting pore pressure dissipation due to controlled exposure of the core material. Pseudo-static analyses were precluded since seismic displacements were approximated from the yield acceleration determined from the LEA geometry.

The LEA was completed in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. A dry slope was assumed for the piezometric conditions. The design seismic loading was defined by STID-5 (PacifiCorp, 2015b), for a Maximum Credible Earthquake (MCE) of 0.55 g, and STID-8 (PacifiCorp, 2015a), for a magnitude 7.25 earthquake. Seismic displacements were approximated by two semi-empirical methods, developed by Makdisi and Seed (1978) and Bray and Travasarou (2007).

Material properties used in the analyses are summarised in Table 4.3. The dam embankment materials (core and shell) were adopted from STID-8 (PacifiCorp, 2015b) except for the shear strength of the shell. Previous analyses completed for the Definite Plan (KRRC, 2018) considered a reduced strength (from 37°) based on an evaluation of construction records. The reduced value was adopted for the design analysis.

Table 4.3 Material Properties for Left Bank Final Dam Excavation Stability Analysis

Material	Unit Weight (pcf)	Effective Friction Angle (°)	Effective Cohesion (psf)
Core	120	27	0
Shell	130	34	0
E4 Cap	125	35	0
E7 Erosion Protection	90	Leps (1970) Lower-bound Shear-Normal Function	

The model was simplified to assume the foundation units comprised only the dam core and shell materials. This simplification did not affect the results as the critical slips were located wholly within the remaining dam embankment.

The GLE method provided more conservative results and was used for assessing the design of the permanent slope. The yield acceleration slip search considered two scales. The smaller-scale shallow slip resulted in the lowest ky (0.17) and extends approximately two-thirds of the embankment height. The larger-scale slip (full height) resulted in a larger ky of 0.22. Displacement estimates indicate the shallow slip will likely displace greater than 2 ft. However, the predicted slip is small in volume (500 yd^3) and shallow in depth (5 ft at its maximum). The consequence of this size of failure is not expected to dam the river. The larger-scale slip, approximately 15 ft deep with a volume of 4,200 yd^3, is predicted to displace less than 2 ft.

The static FOS for a full-height slip is 1.8 and is associated with a maximum depth of approximately 14 ft and a predicted volume of 4,500 yd^3. The analyses indicate the design final slope of 3H:1V satisfies the requirements of the acceptance criteria. Smaller-scale slips are possible under long-term static conditions; however, such occurrences are expected to be localised and not sizeable enough to dam the river.

4.3 SCOUR HOLE DESIGN

The material used to infill the scour hole will be a mix of E9/E9a - General Fill and Type CR1/CR2 - Concrete Rubble. Cover material will be a minimum of 6' of Type E9/E9b – General Fill and be comprised of scour hole cut material and/or forebay grading materials. Due to the anticipated method of material placement, the scour hole fill has been designed to have minimal compaction requirements. Placement requirements are detailed in Technical Specification 31 23 00 – Excavation and Fill Placement.

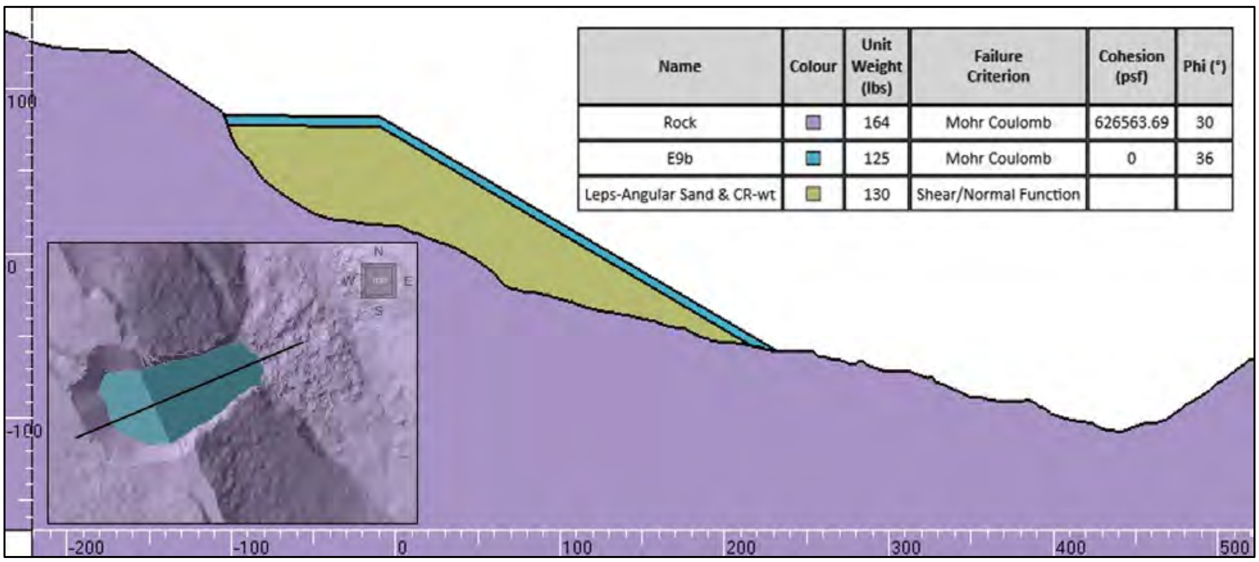
The scour hole design is shown on Drawings C1340 and C1341 and has the following design parameters:

- Maximum Slope: 1.7H:1V
- Crest Elevation: 3,728 ft
- Maximum Height: 140 ft

The stability of the scour hole design fill slope was assessed in a similar manner as the final excavated slope of the dam embankment. The same acceptance criteria and approach were used. The target FOS for the long-term static condition is 1.5 and 1.0 for determining the yield acceleration. Seismic displacements were estimated with semi-empirical methods (Makdisi and Seed, 1978; Bray and Travasarou, 2007) and a target of 2 ft. Pseudo-static analysis was precluded by estimating the seismic displacement. The short-term end-of-construction condition was not assessed since excess pore pressure generation and dissipation are not expected, given the free-draining nature of the design fills and the understanding of the foundation conditions. The creation of the scour hole and high-velocity nature of facility operations suggest fines would be mostly washed out of the accumulated coarse-grained erosion debris at the base of the hole.

The LEA was completed in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. A dry slope was assumed and the design seismic loading was defined by the STIDs (PacifiCorp, 2015a and 2015b).

The model geometry and material properties are shown on Figure 4.7. Material strengths were assumed based on the construction method and sequence. The Leps (1970) shear-normal function developed for the Angular Sand dataset was assumed for the fill strength but the equation was extrapolated to provide zero cohesion at zero normal stress. As a result, the analyses indicated the mobilized base friction angle was 38°. A constant value (36°) was assumed for the frictional strength of the 6-ft E9b cap based on gradation limits and compaction effort. Cohesive strength for both units was zero.



Name	Colour	Unit Weight (lbs)	Failure Criterion	Cohesion (psf)	Phi (°)
Rock		164	Mohr Coulomb	626563.69	30
E9b		125	Mohr Coulomb	0	36
Leps-Angular Sand & CR-wt		130	Shear/Normal Function		

NOTES:

1. SECTIONAL VIEW (LOOKING UPSTREAM) OF BLACK LINE IN INSET PLAN VIEW.

Figure 4.5 Scour Hole Model Geometry and Material Properties

The GLE method provided more conservative ky results and were again used for assessing the design. The yield acceleration slip search produced localized small-scale shallow slips with ky less than 0.17. A sizeable slip, extending from the fill crest to about mid-height, produced a ky of 0.17 and a displacement estimate greater than 2 ft. The maximum depth of the associated slip is about 17 ft and involves a volume of approximately 4,500 yd^3. The estimated volume of such a slip is not expected to dam the river, given it is less than 10% of the overall volume of the design fill and erosion of the scour hole was not known to have dammed the river or impeded its flow in the past. A full-height (30-ft deep) slip with a ky of 0.18 was estimated to displace less than 2 ft and involves a volume of approximately 11,500 yd^3. The results of these two slips are shown on Figure 4.8.

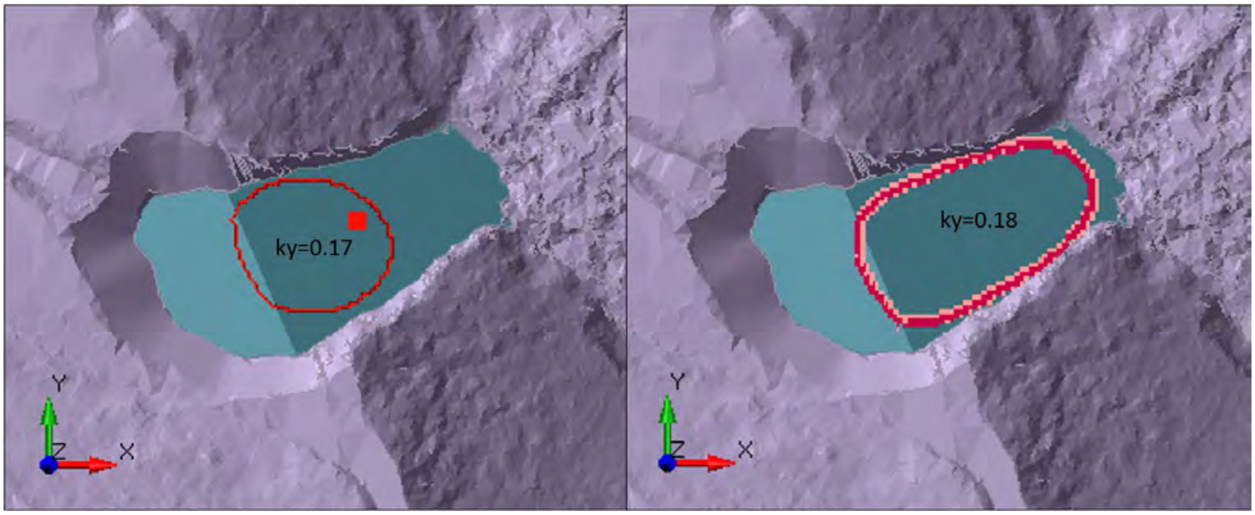
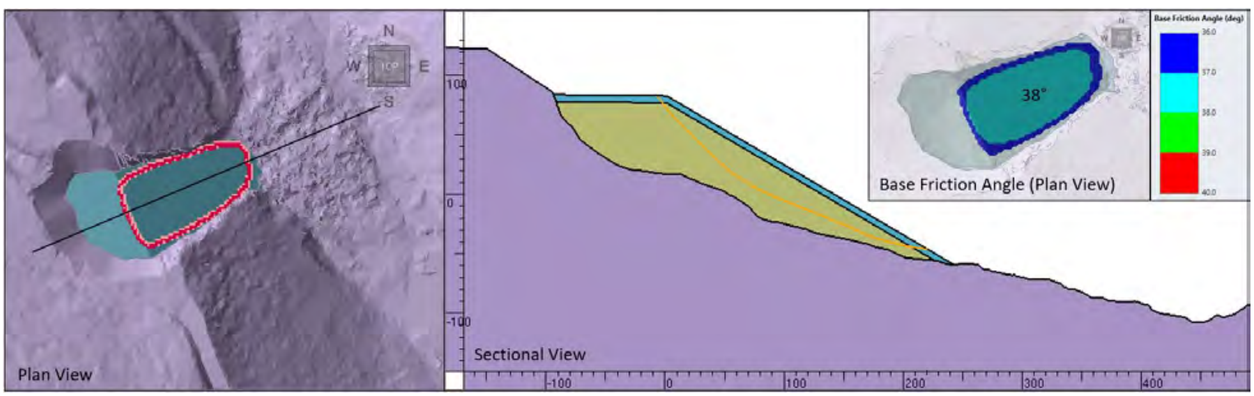


Figure 4.6 Scour Hole Yield Acceleration Results

The static result for a full-height slip marginally satisfies the target FOS of 1.5 for dry conditions, with both Spencer and GLE methods. The critical slip is predicted to involve a volume of approximately 13,500 yd^3 with a maximum depth of roughly 35 ft. The critical slip of the static analysis is shown on Figure 4.9. Search results indicate localized smaller-scale slips could occur but are found within the 6-ft E9b cap material.



NOTES:

1. SECTIONAL VIEW (LOOKING UPSTREAM) OF CRITICAL SLIP (YELLOW LINE) SHOWN IN THE RIGHT IMAGE. BLACK LINE IN PLAN VIEW (LEFT IMAGE) SHOWS LOCATION OF SECTION LINE.
2. BASE FRICTION ANGLE CONTOURS SHOWN IN INSET. ZERO COHESIVE STRENGTH WAS ASSIGNED.

Figure 4.7 Scour Hole Static Stability Results

4.4 POWER CANAL AND PENSTOCK ACCESS ROAD SLOPE HAZARDS

The power canal slope terrain hazard assessment identifies hazards along the alignment. It is included as part of the Geotechnical Data Report (VA103-640/1-2). The existing power canal plan and typical sections are shown on Drawings C1320, C1321, and C1323.

The penstock access road slope terrain hazard assessment has also been completed. The extent of anticipated access road stabilization is limited to the lower penstock access roads shown on Drawing

C1512. The access road slope terrain hazard assessment is included as part of the Geotechnical Data Report (VA103-640/1-2).

4.5 BORROW AREAS

Borrow areas may be required at the intake and the forebay area to provide backfilling material. The intake borrow areas are to be evaluated during construction and may be used as erosion protection materials for the Final River Channel as shown on Drawings C1210 and C1230. The forebay borrow source proposed grading is required to cover the forebay concrete structure left in place and provide scour hole fill materials and is shown on Drawings C1334 and C1335. The grading is designed to convey direct precipitation to the scour hole drainage swale.

4.6 DISPOSAL SITES

4.6.1 FOUNDATION PREPARATION

Geotechnical data and investigations are presented in the Geotechnical Data Report (VA103-640/1-2). On site investigations have been completed for the original upland disposal site. Foundation conditions for the river bank disposal sites require inspection after drawdown, under dry conditions.

Foundation preparation of staging and disposal sites will consist of removing and stockpiling topsoil and organic and soft materials away from the disposal site. Topsoil and organic materials may be redistributed on the disposal site slopes following embankment and historic cofferdam excavation.

4.6.2 DISPOSAL SITE DESIGN

Two concepts were considered for siting the permanent disposal areas. The preferred concept is along the left river bank. An additional disposal site on the right river bank has been designed for use if the left bank disposal site reaches capacity.

4.6.2.1 RIVER BANK DISPOSAL SITES

The proposed disposal areas are shown on Drawings C1240 and C1241. Disposal areas are designed with stable permanent slopes and suitable drainage requirements.

The disposal site fill materials will be comprised of dam embankment, historic cofferdam, and remaining sediment excavation materials, expected to meet the, E9/E9a – General Fill and E10 - Random Fill material specifications. The disposal sites that are considered for this excavation are described as follows:

- The Left Bank Disposal Site which will hold the majority of the excavated materials and include an area of cover fill on the remainder of the spillway and intake structure concrete left in place.
- The Alternative Right Bank Disposal Site which, if used, will be primarily comprised of excavated material from the final cofferdam breach.

The material placed in the disposal sites will be track packed and graded with a bulldozer to meet the requirements of the technical specifications. The disposal sites will be graded to promote surface drainage towards the Klamath River. Parameters for the Left Bank Disposal Area are as follows:

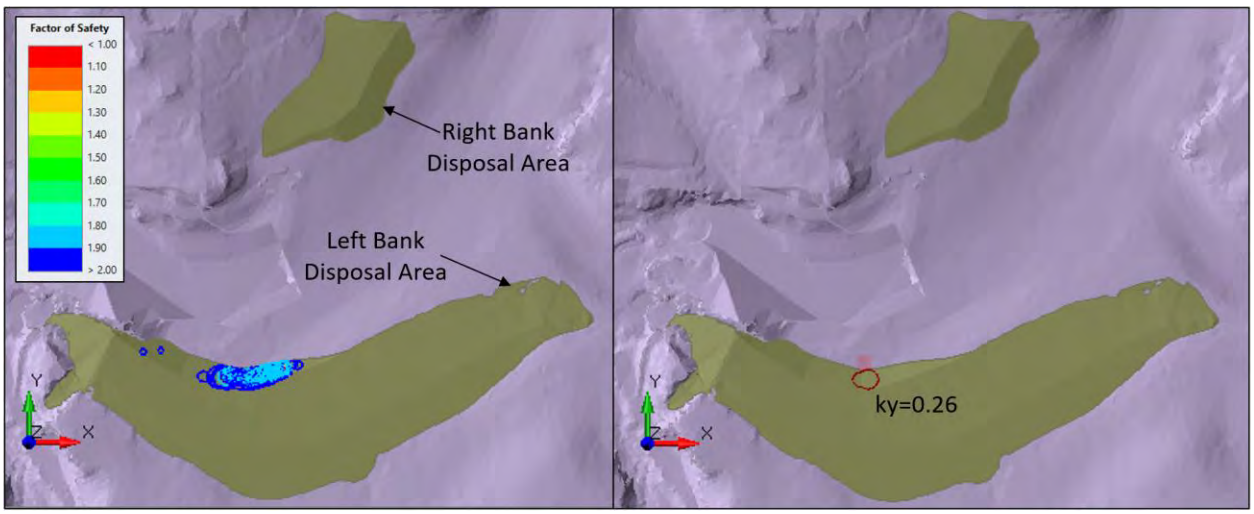
- Maximum Slope: 3.5H:1V
- Crest Elevation: 3,800 ft

- Maximum Height: 35 ft

Parameters for the Alternative Right Bank Disposal Area are as follows:

- Maximum Slope: 5H:1V
- Crest Elevation: 3,794 ft
- Maximum Height: 18 ft

Stability of the two river bank disposal areas were assessed by LEA, in three dimensions with Slide3 (Rocscience, 2020) and using both Spencer and GLE methods. The target FOS for the long-term static condition is 1.5 and 1.0 for determining the yield acceleration. Seismic displacements were estimated with semi-empirical methods (Makdisi and Seed, 1978; Bray and Travasarou, 2007) and a target of 2 ft. Pseudo-static analyses was completed with a 20% strength reduction for 50% MCE. The short-term end-of-construction condition was not assessed since excess pore pressure generation and dissipation are not expected, given the free-draining nature of the design fills and the understanding of the foundation conditions. The design seismic loading was defined by the STIDs (PacifiCorp, 2015a and 2015b). The analysis model and results of the disposal areas are shown on Figure 4.10.



NOTES:

1. STATIC SEARCH RESULTS SHOWN IN LEFT, ONLY SLIPS WITH FOS < 2.0 ARE SHOWN.
2. YIELD ACCELERATION RESULT SHOWN IN RIGHT.

Figure 4.8 Disposal Sites Stability Results

The disposal fill is E9 and the frictional strength was assigned 32° based on design gradation limits and compaction requirements. Zero cohesive strength and dry piezometric conditions were assumed. The right bank disposal area satisfies the requirements for static and pseudo-static stability. The left bank disposal area satisfies static stability but not pseudo-static stability. Seismic displacements were estimated for a yield acceleration of 0.26 and less than 2 ft of movement was predicted.

4.6.2.2 ALTERNATIVE UPLAND DISPOSAL SITE

An alternative to the river bank disposal sites is the original borrow source area that was developed for dam construction and the location of the disposal site considered during preliminary design. Preliminary analyses indicate slope stability achieves a factor of safety of 1.5 for dry static conditions.

Attachments:

1 – J.C. Boyle Facility Simulated Drawdown

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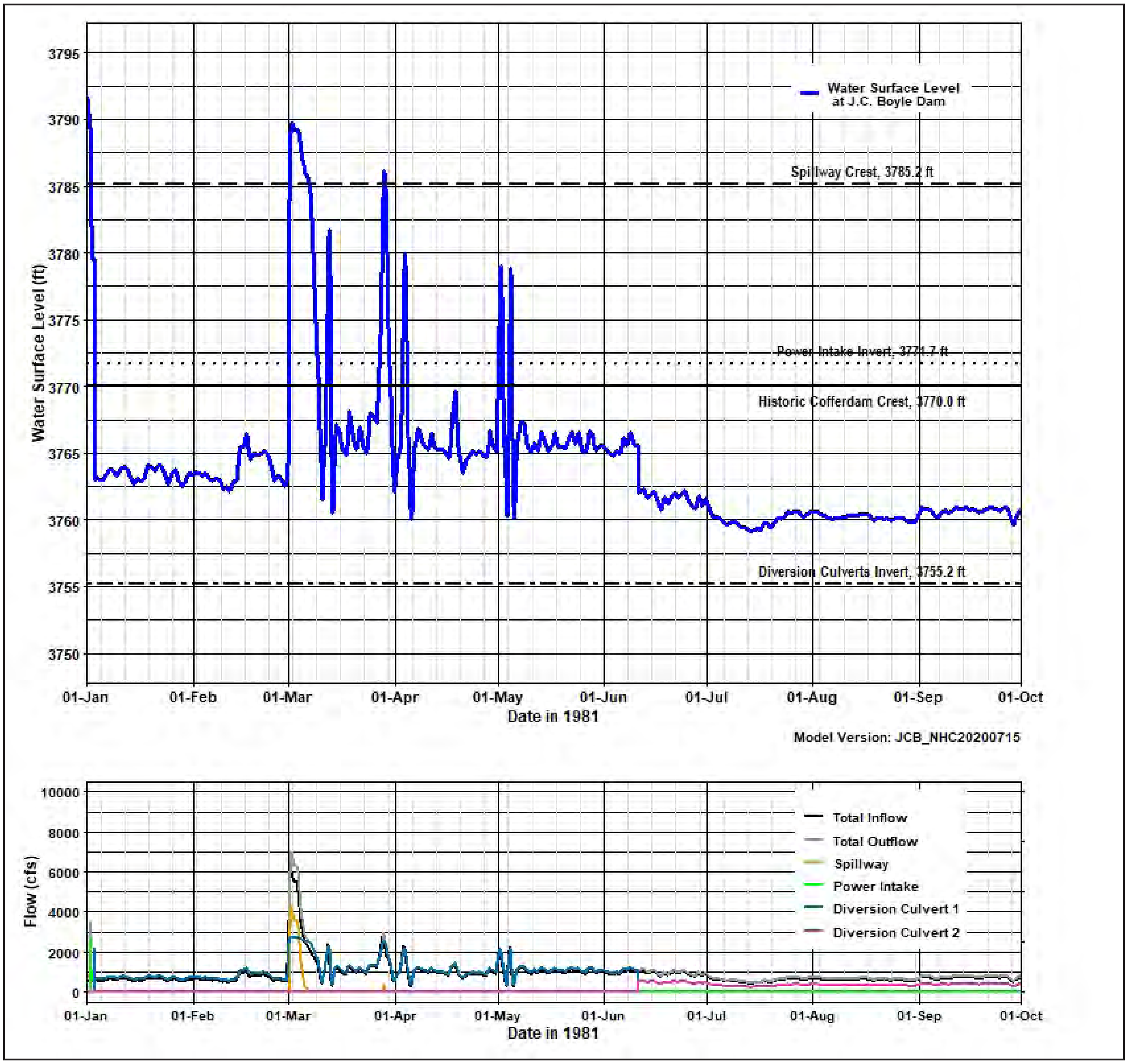

Figure 1 - J.C. Boyle Facility Simulated Drawdown - Year 1981

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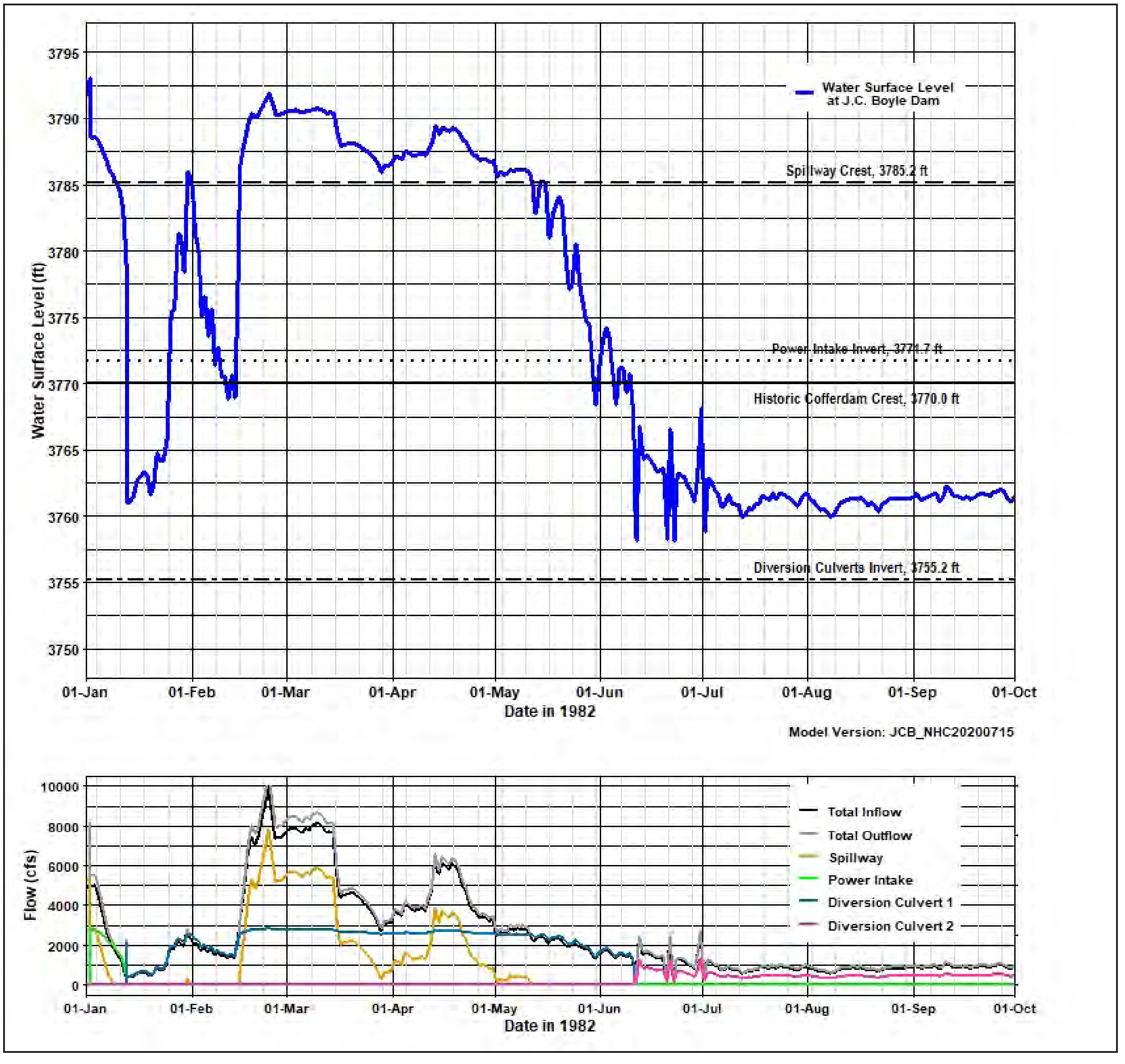

Figure 2 - J.C. Boyle Facility Simulated Drawdown - Year 1982

July 29, 2020

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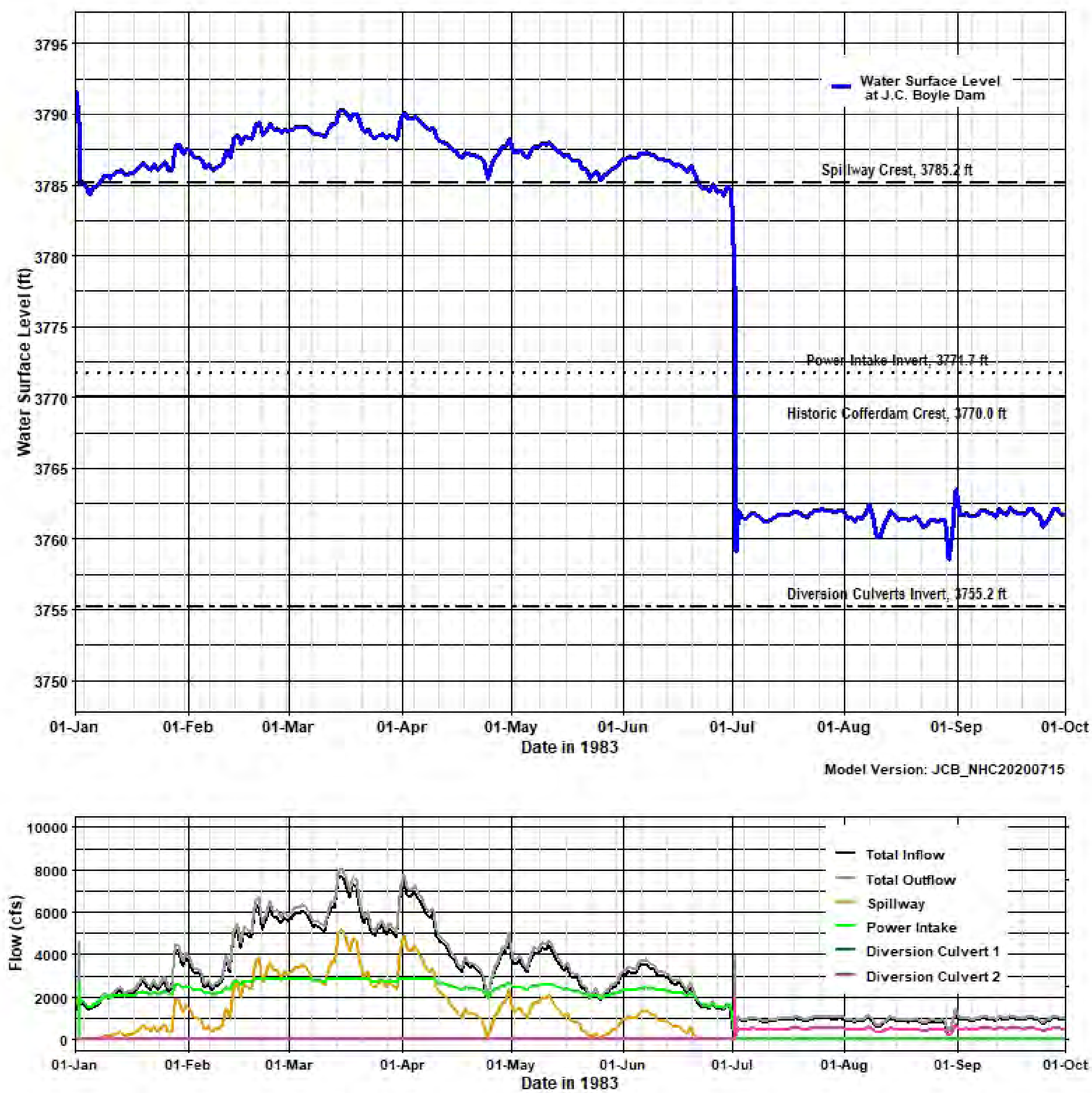

Figure 3 - J.C. Boyle Facility Simulated Drawdown - Year 1983

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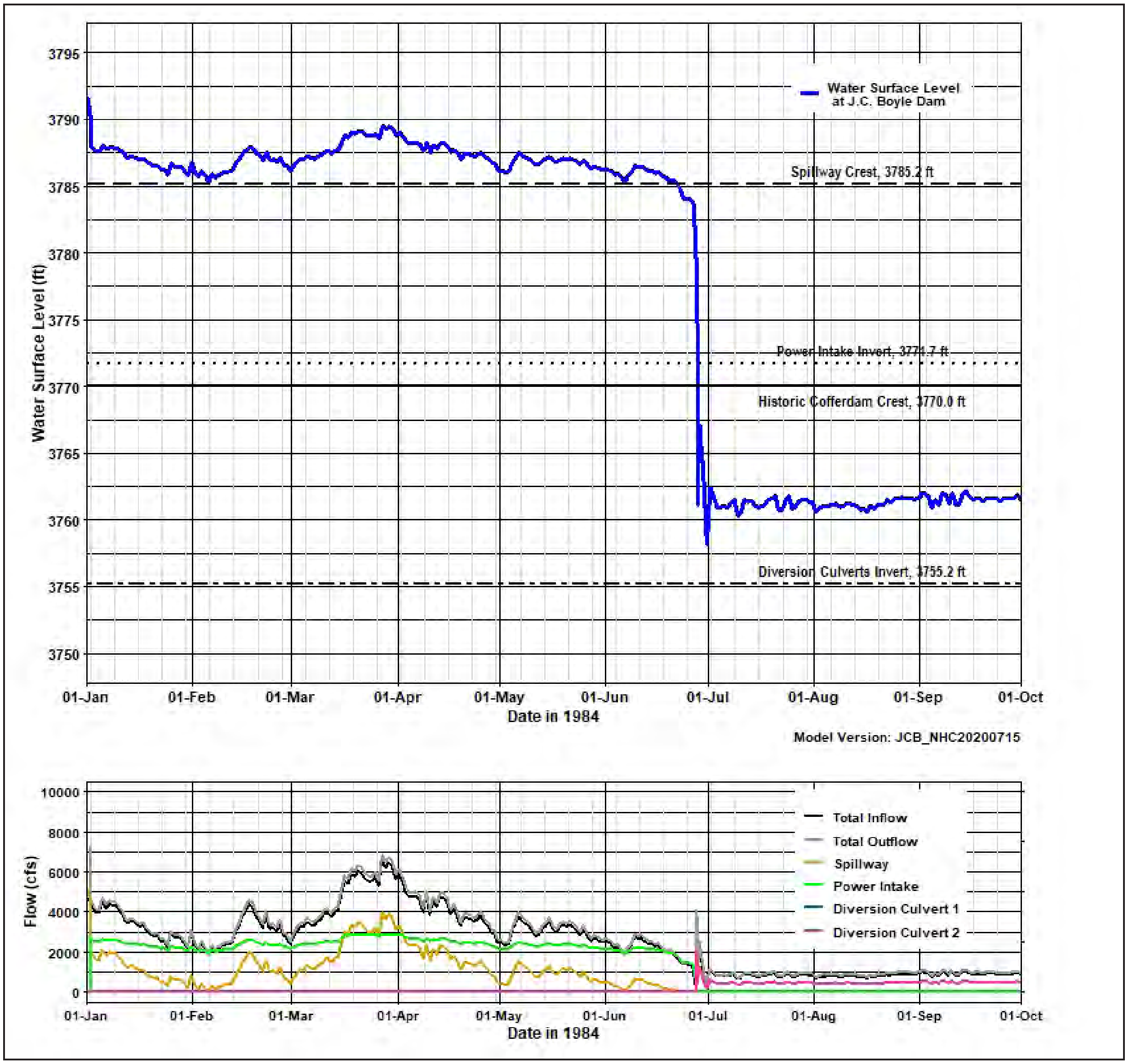

Figure 4 - J.C. Boyle Facility Simulated Drawdown - Year 1984

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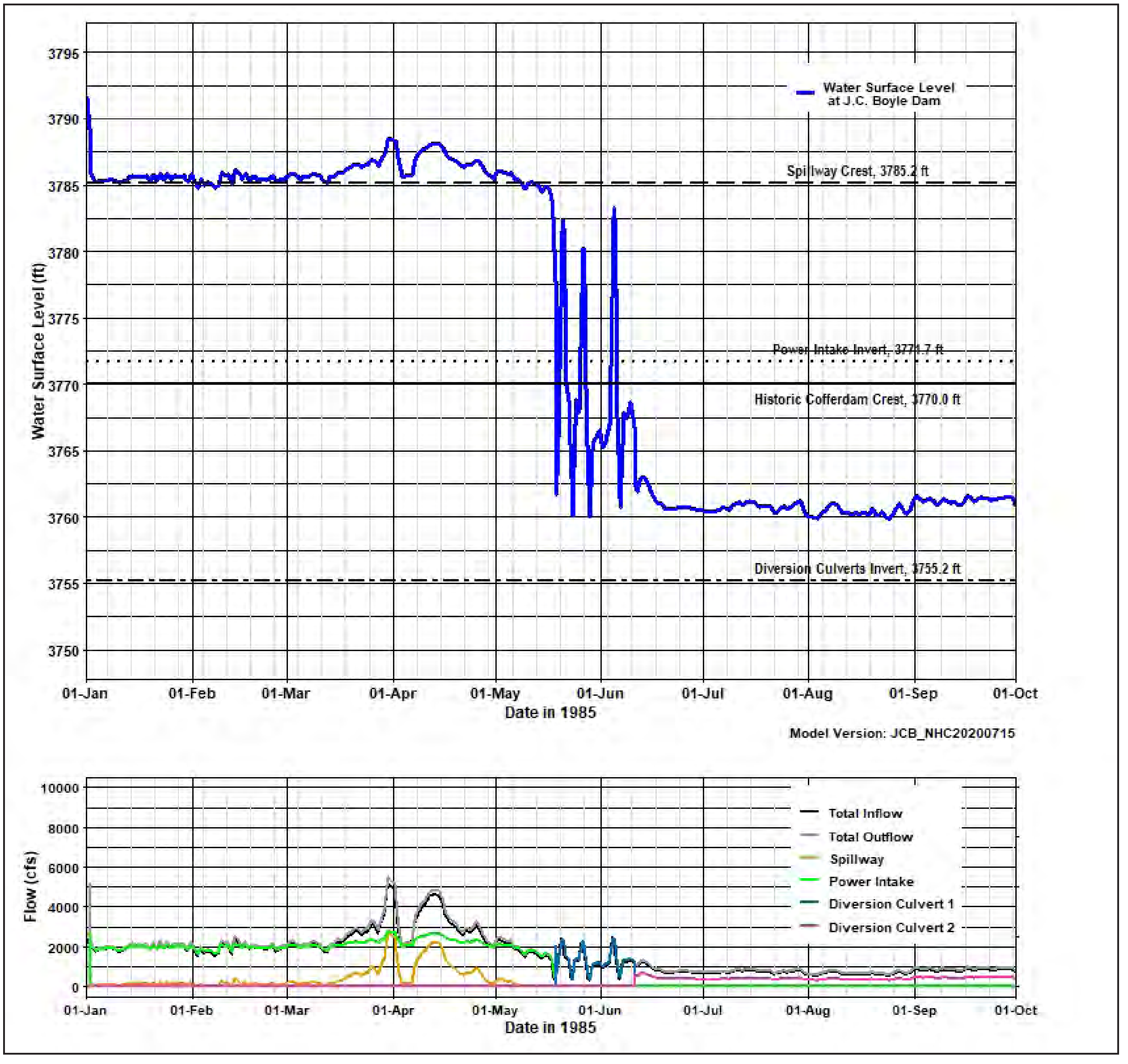

Figure 5 - J.C. Boyle Facility Simulated Drawdown - Year 1985

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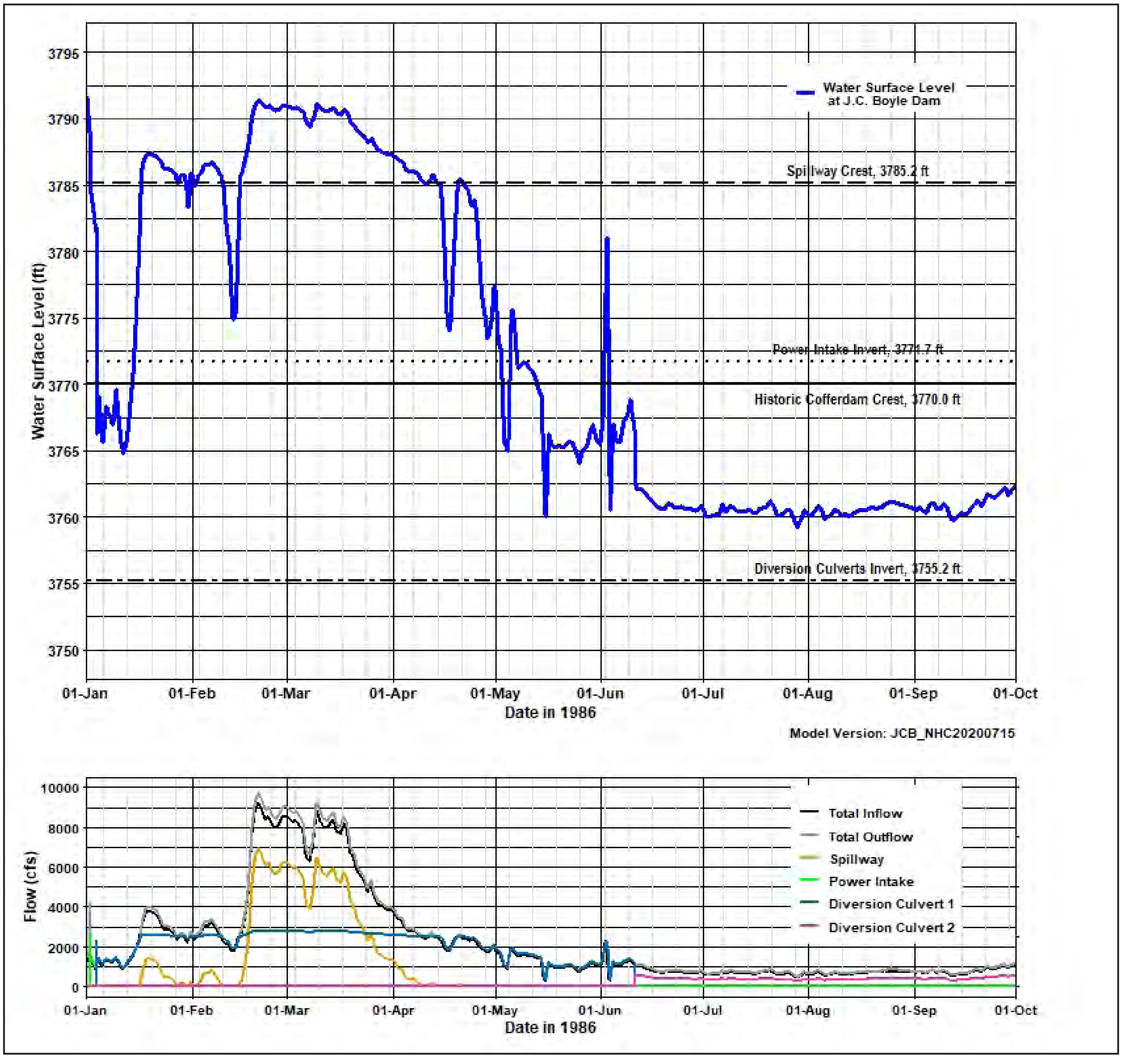

Figure 6 - J.C. Boyle Facility Simulated Drawdown - Year 1986

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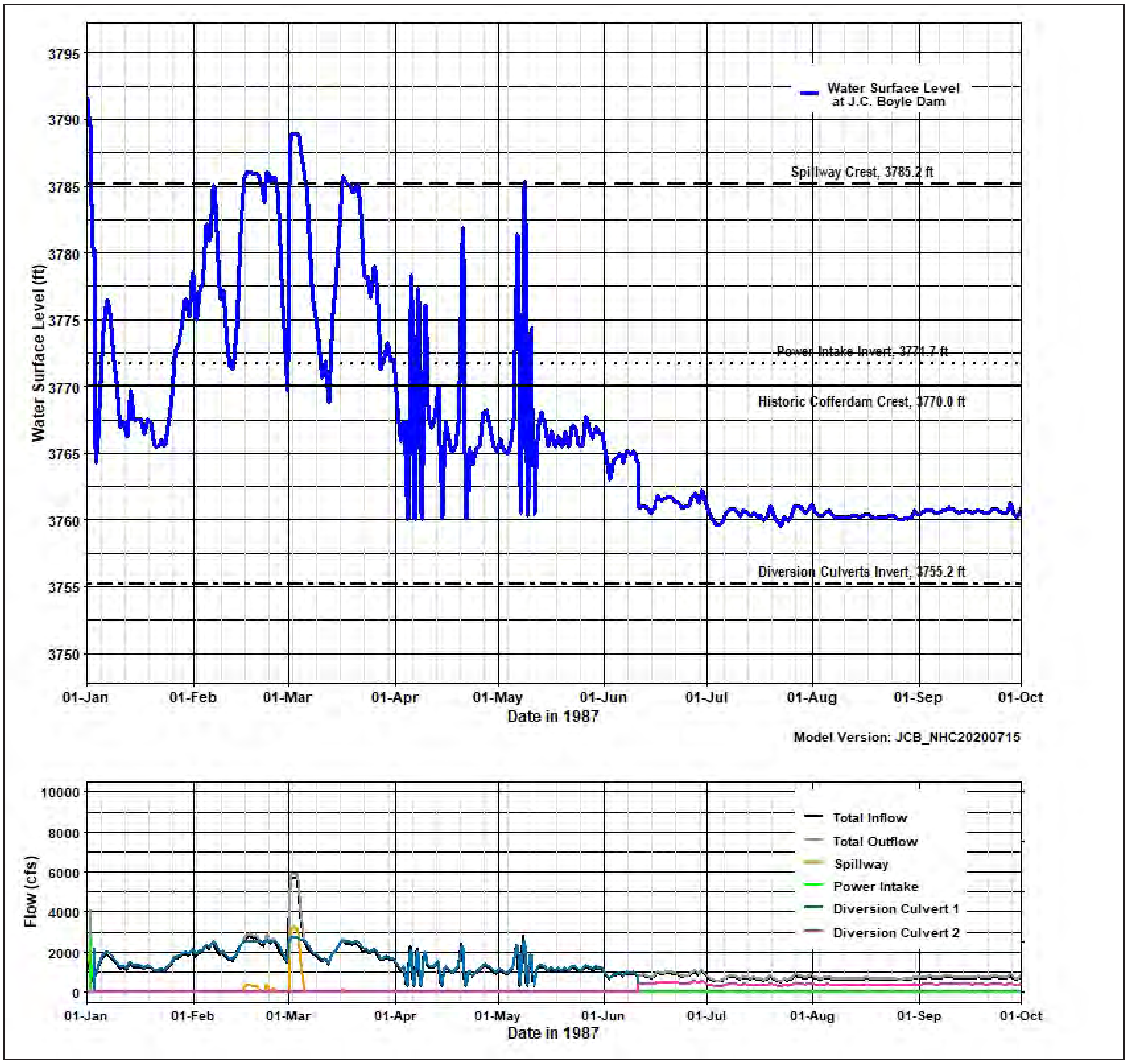

Figure 7 - J.C. Boyle Facility Simulated Drawdown - Year 1987

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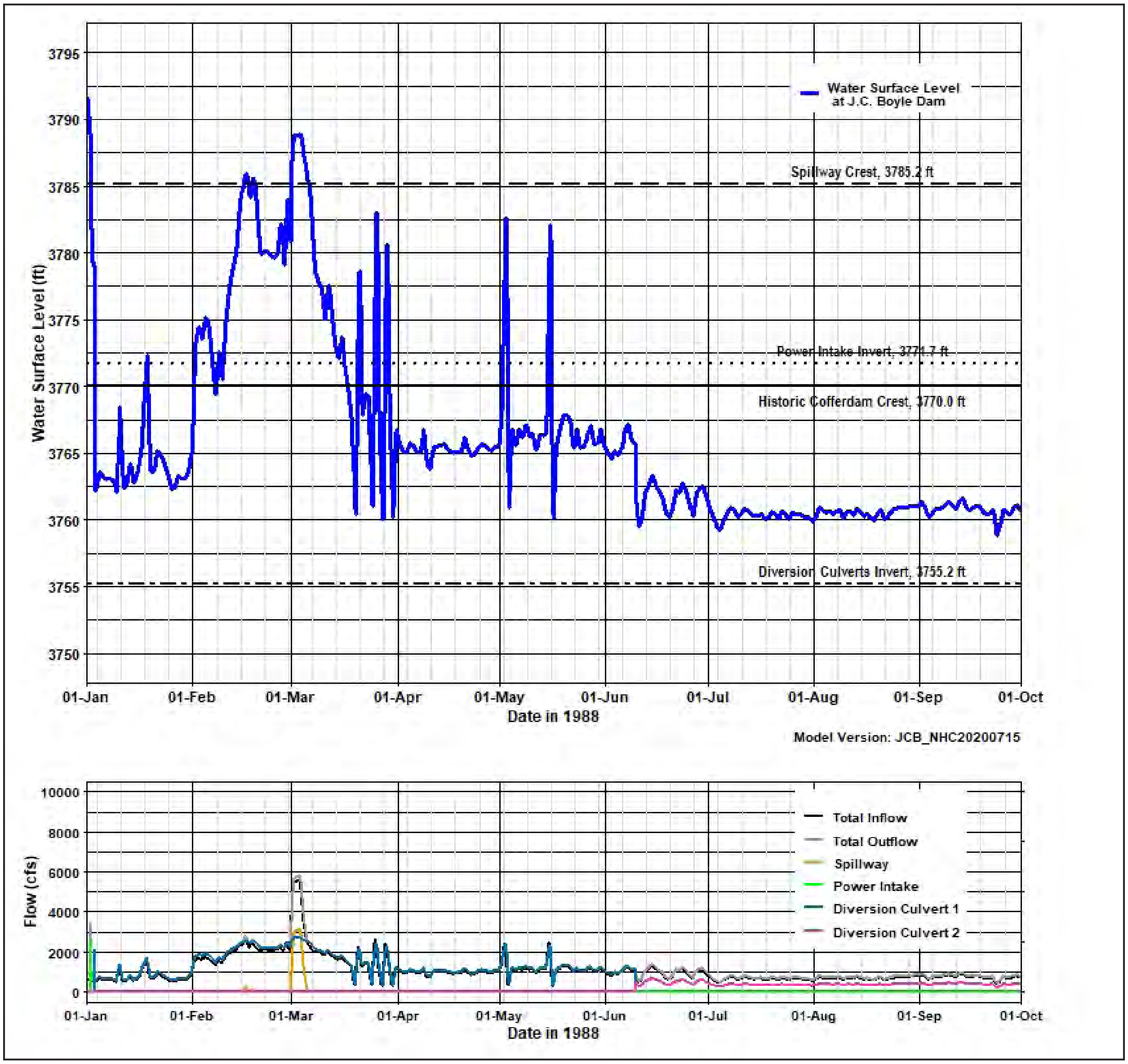

Figure 8 - J.C. Boyle Facility Simulated Drawdown - Year 1988

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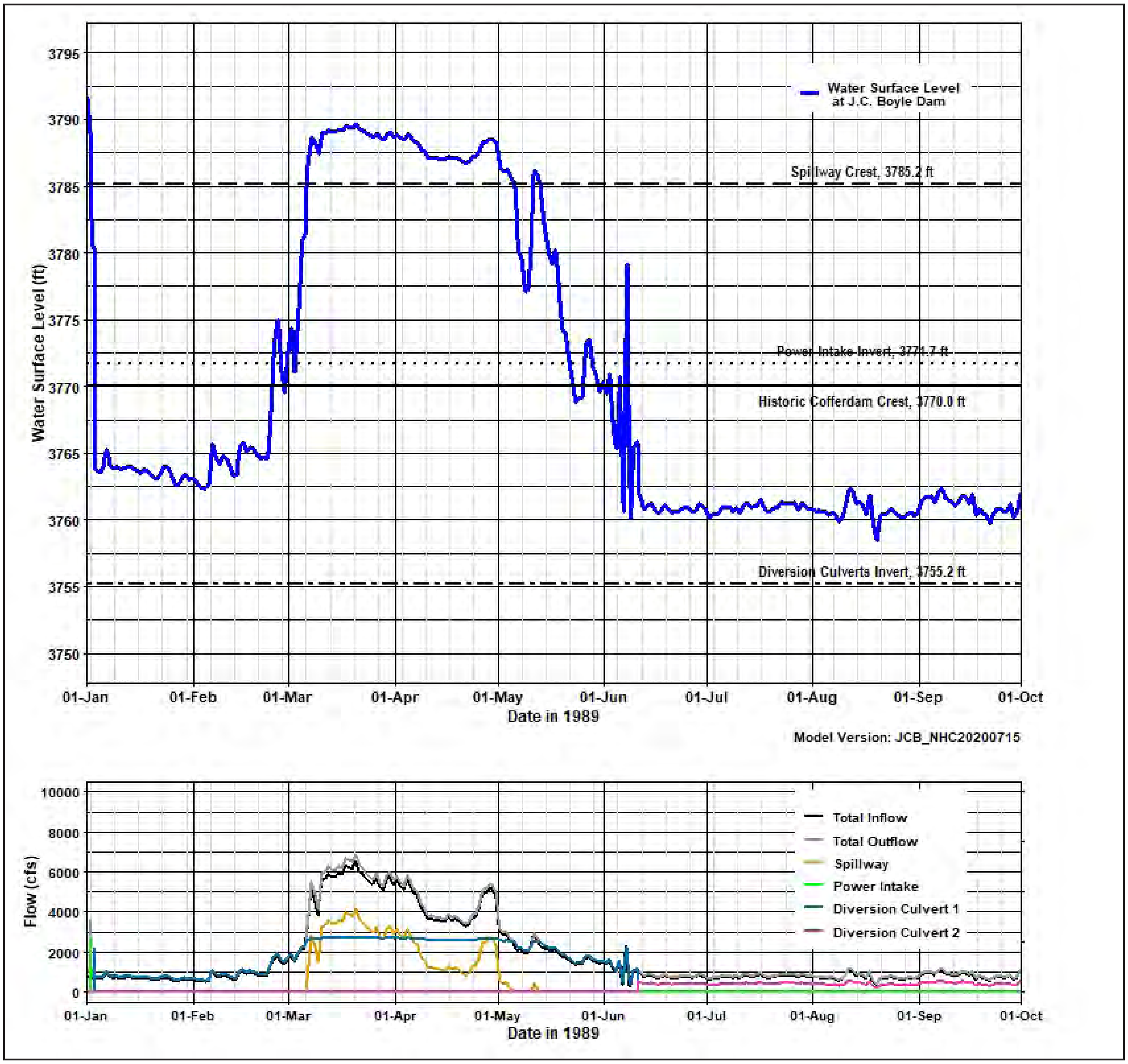

Figure 9 - J.C. Boyle Facility Simulated Drawdown - Year 1989

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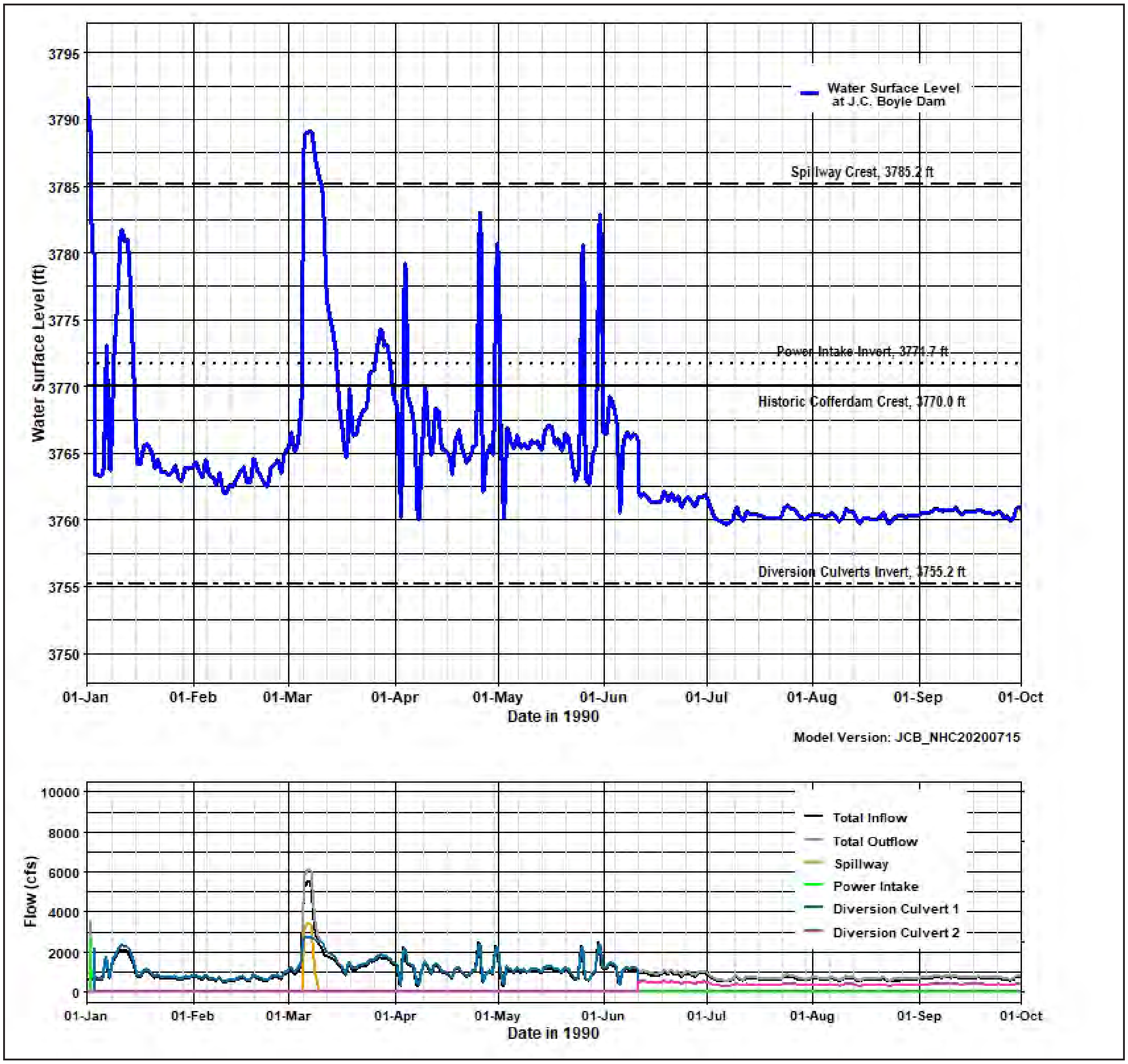

Figure 10 - J.C. Boyle Facility Simulated Drawdown - Year 1990

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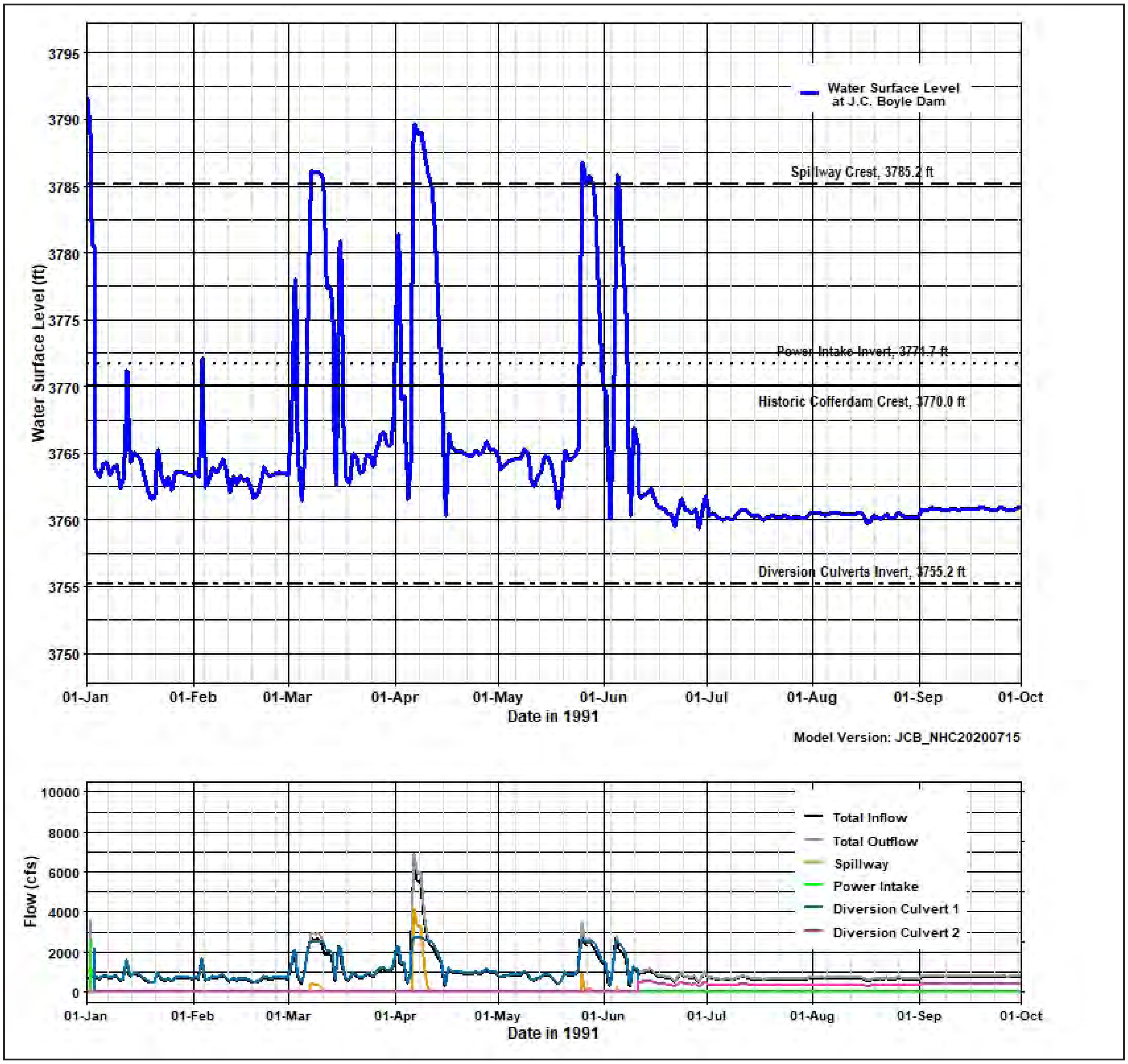

Figure 11 - J.C. Boyle Facility Simulated Drawdown - Year 1991

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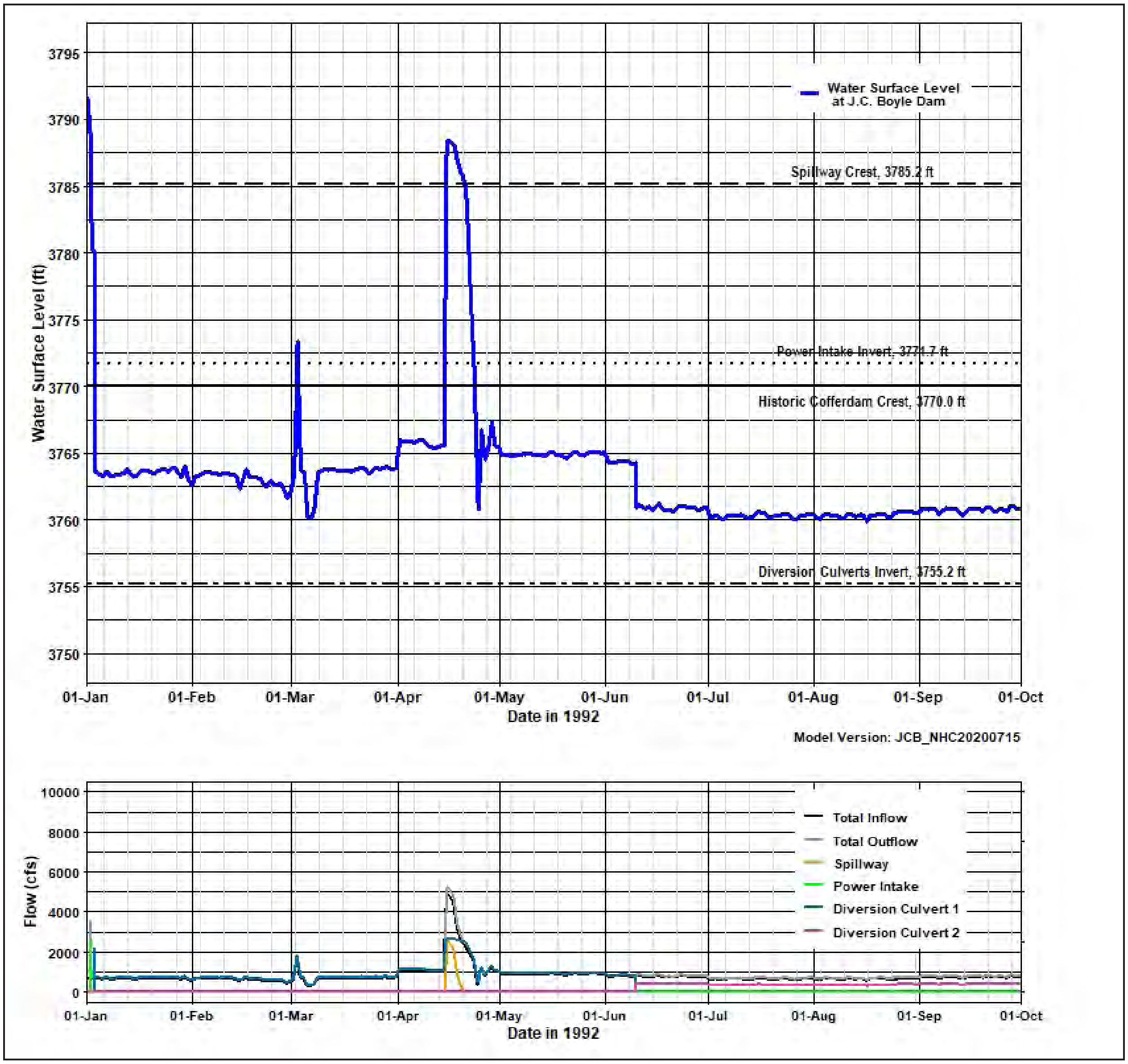

Figure 12 - J.C. Boyle Facility Simulated Drawdown - Year 1992

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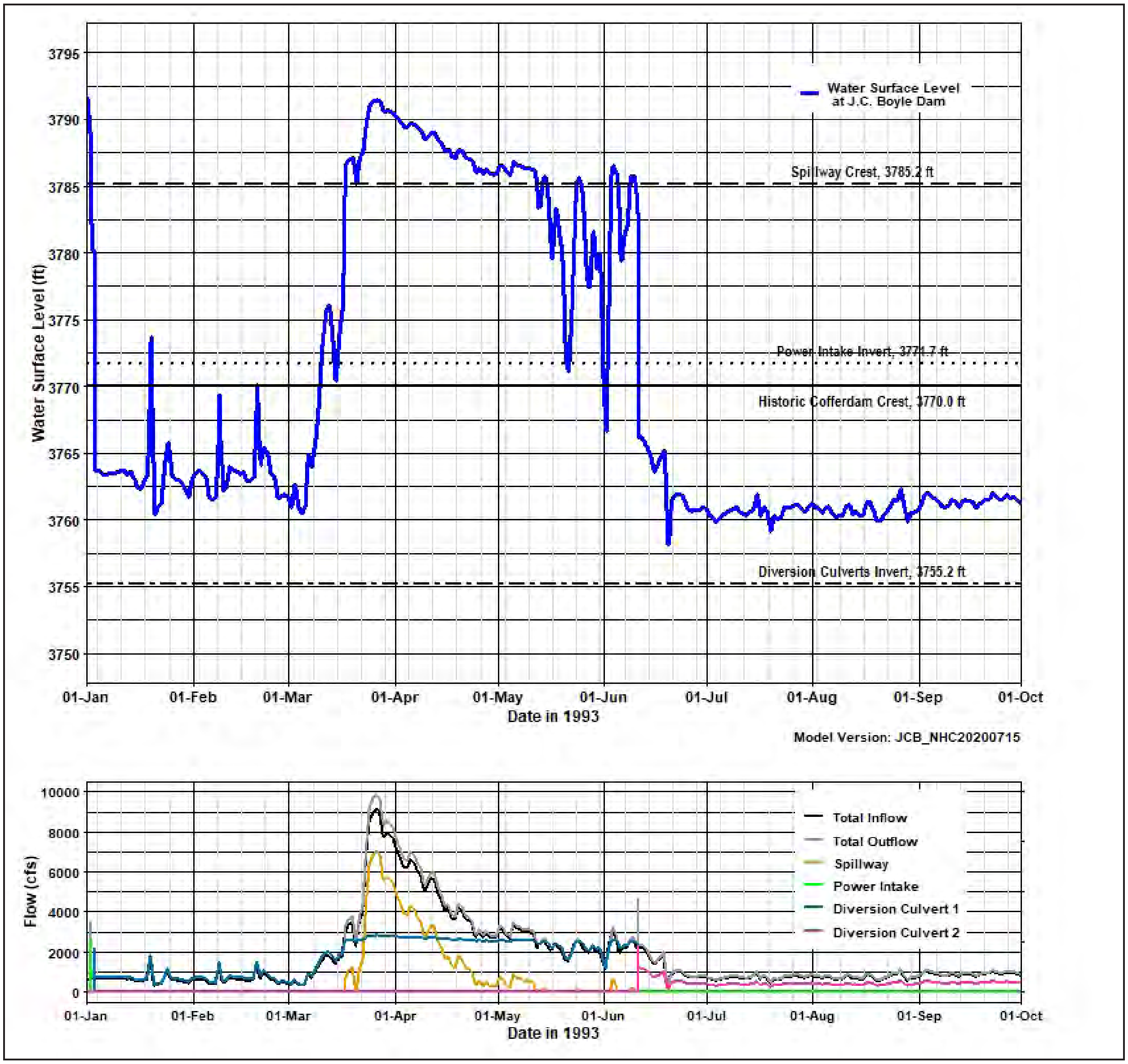

Figure 13 - J.C. Boyle Facility Simulated Drawdown - Year 1993

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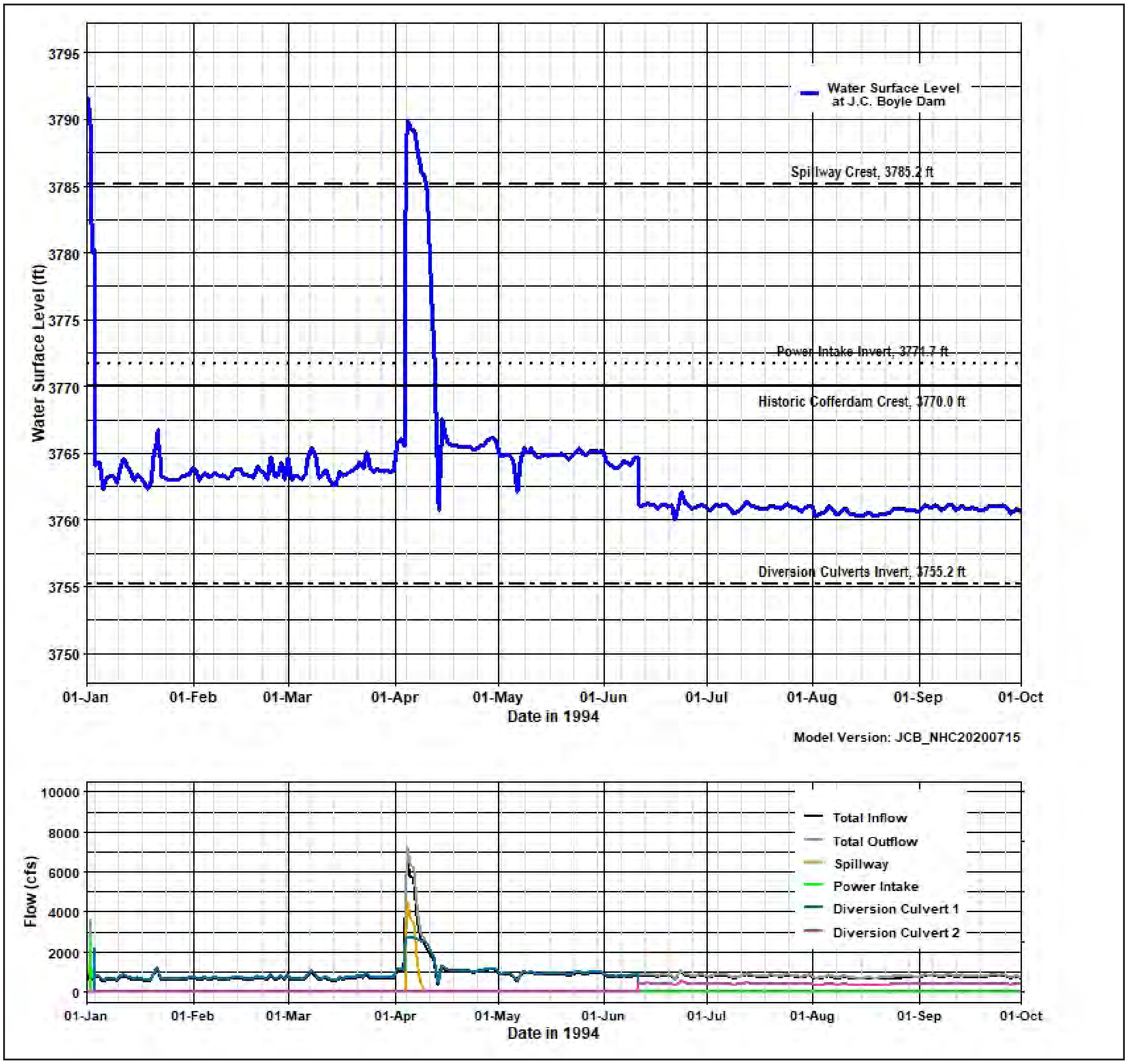

Figure 14 - J.C. Boyle Facility Simulated Drawdown - Year 1994

July 29, 2020

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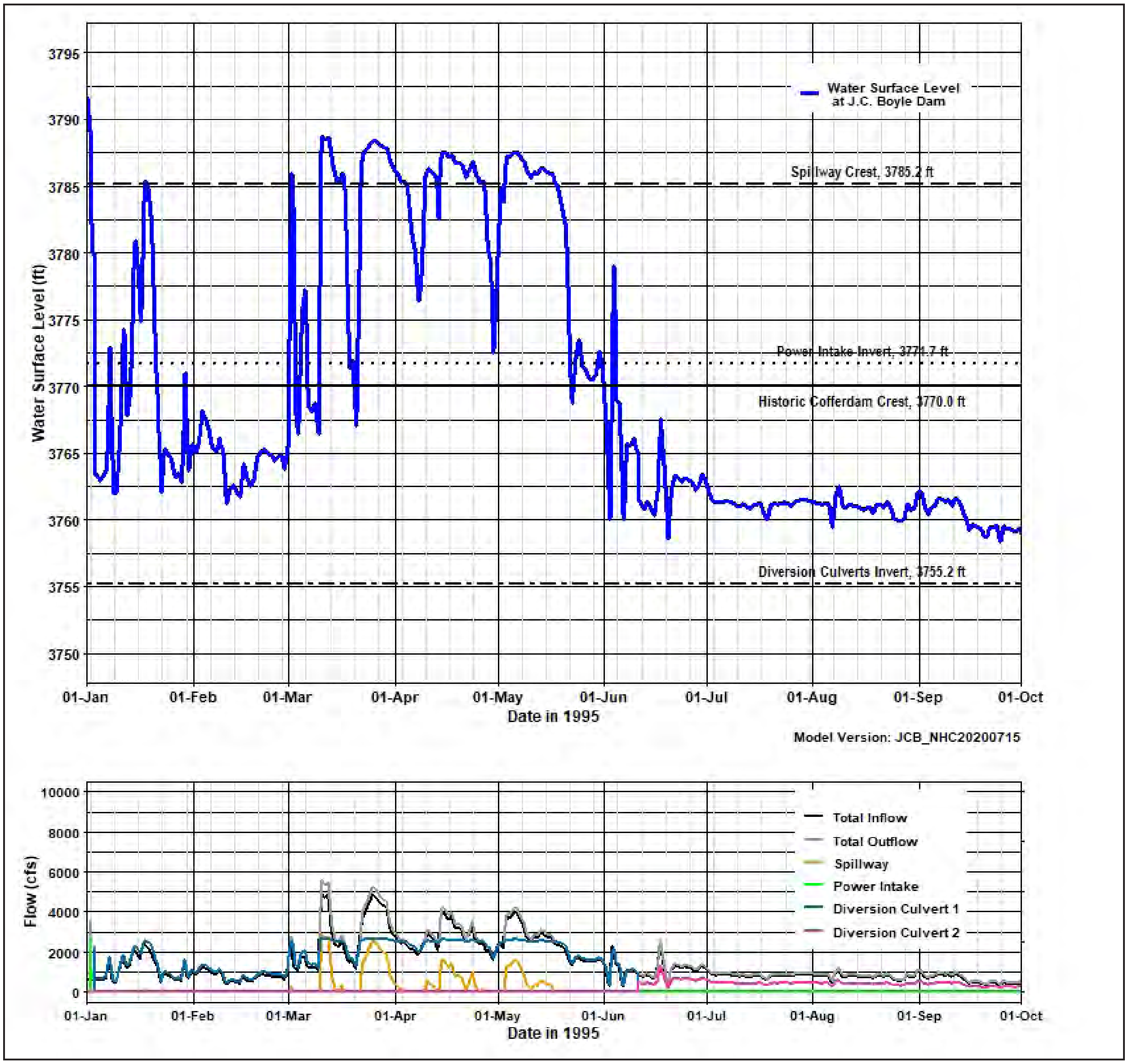

Figure 15 - J.C. Boyle Facility Simulated Drawdown - Year 1995

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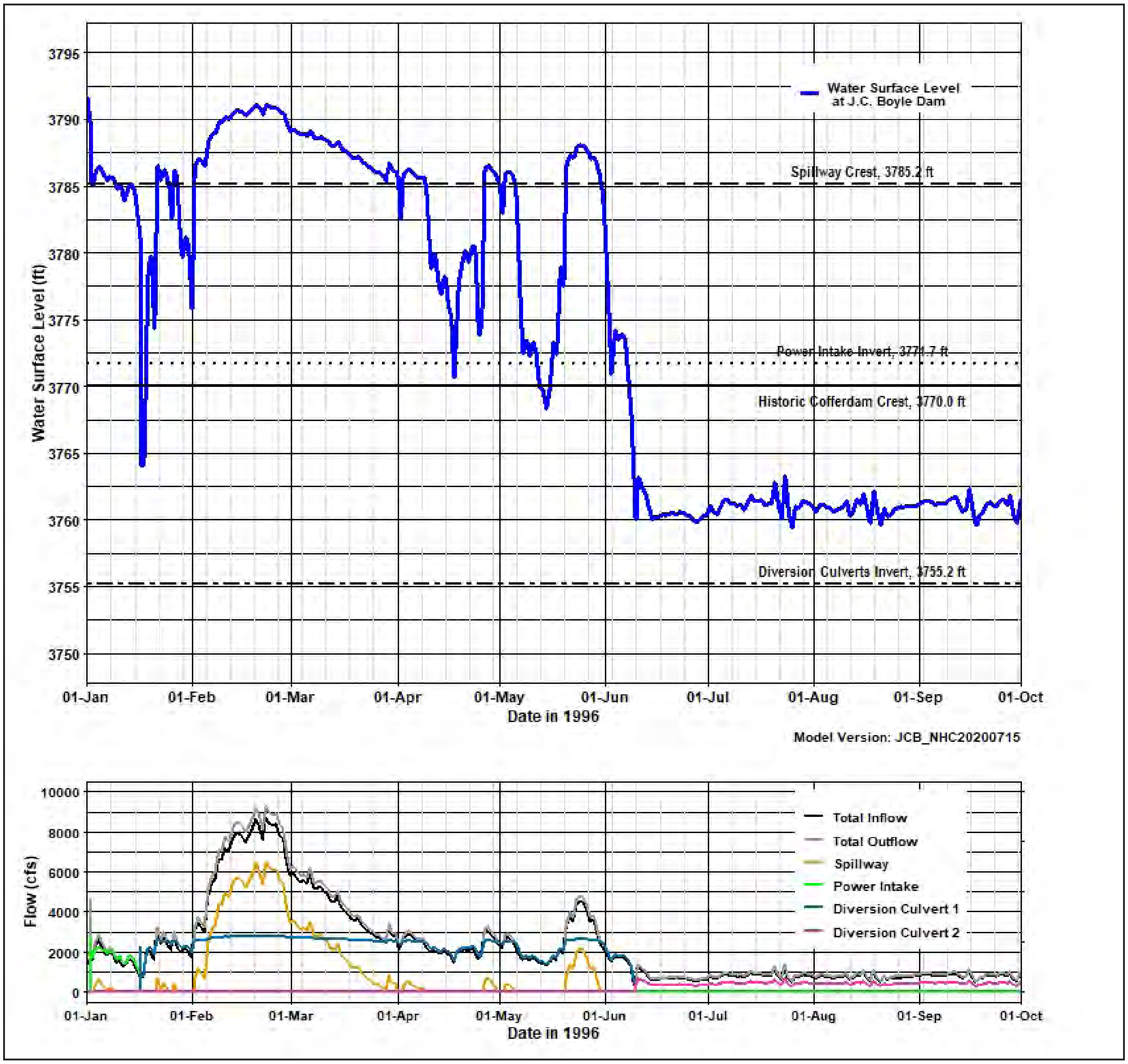

Figure 16 - J.C. Boyle Facility Simulated Drawdown - Year 1996

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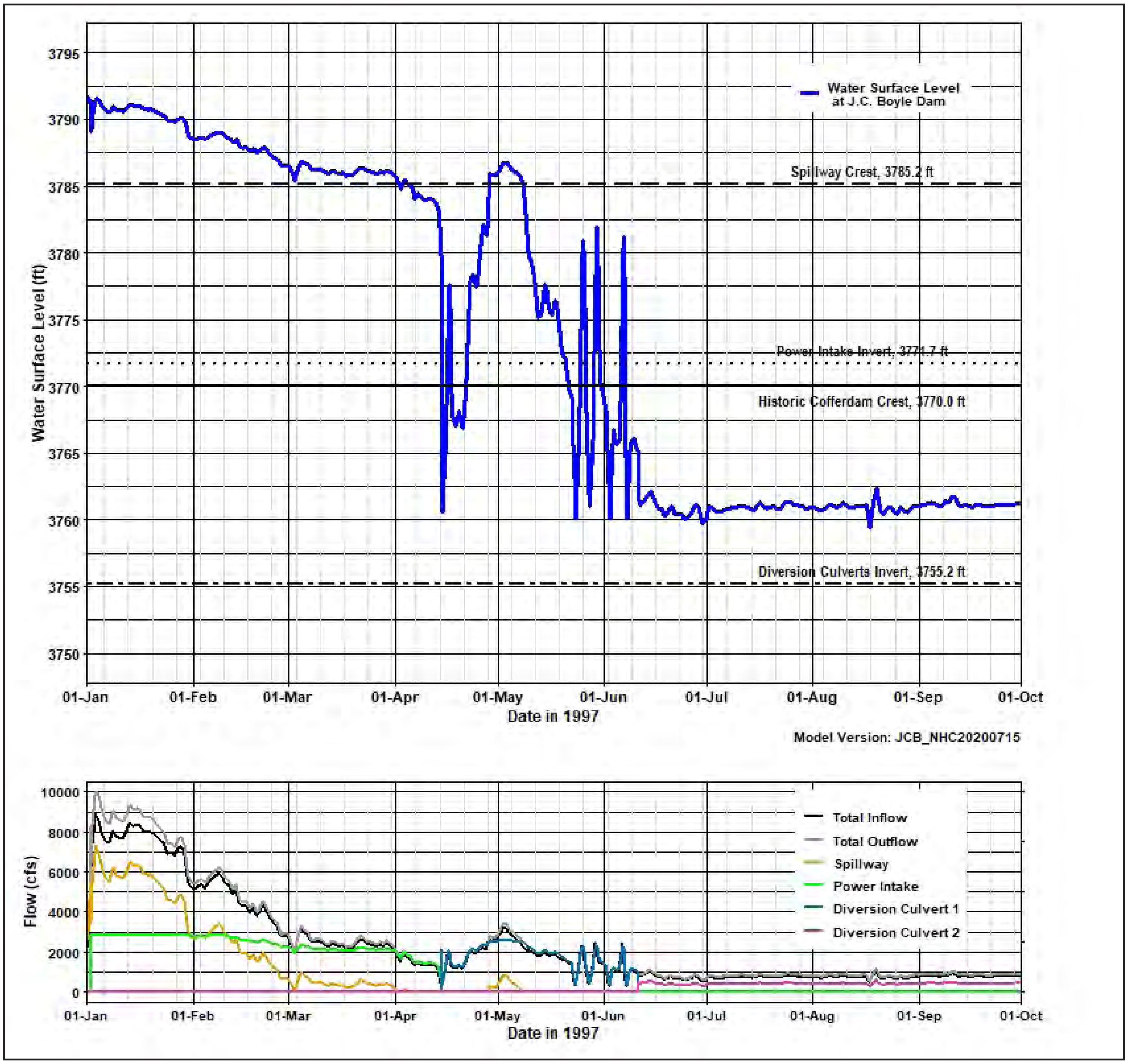

Figure 17 - J.C. Boyle Facility Simulated Drawdown - Year 1997

July 29, 2020

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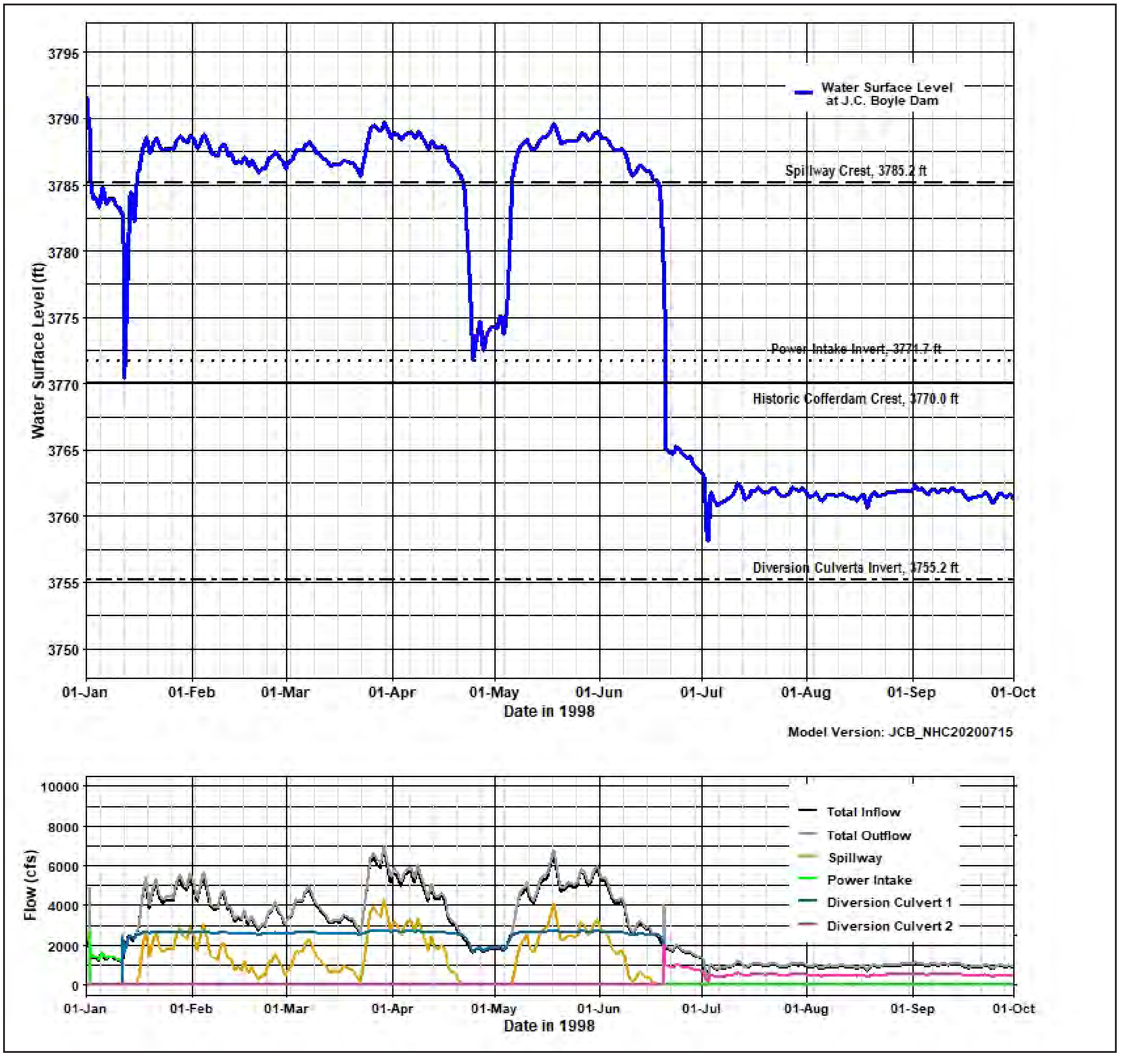

Figure 18 - J.C. Boyle Facility Simulated Drawdown - Year 1998

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

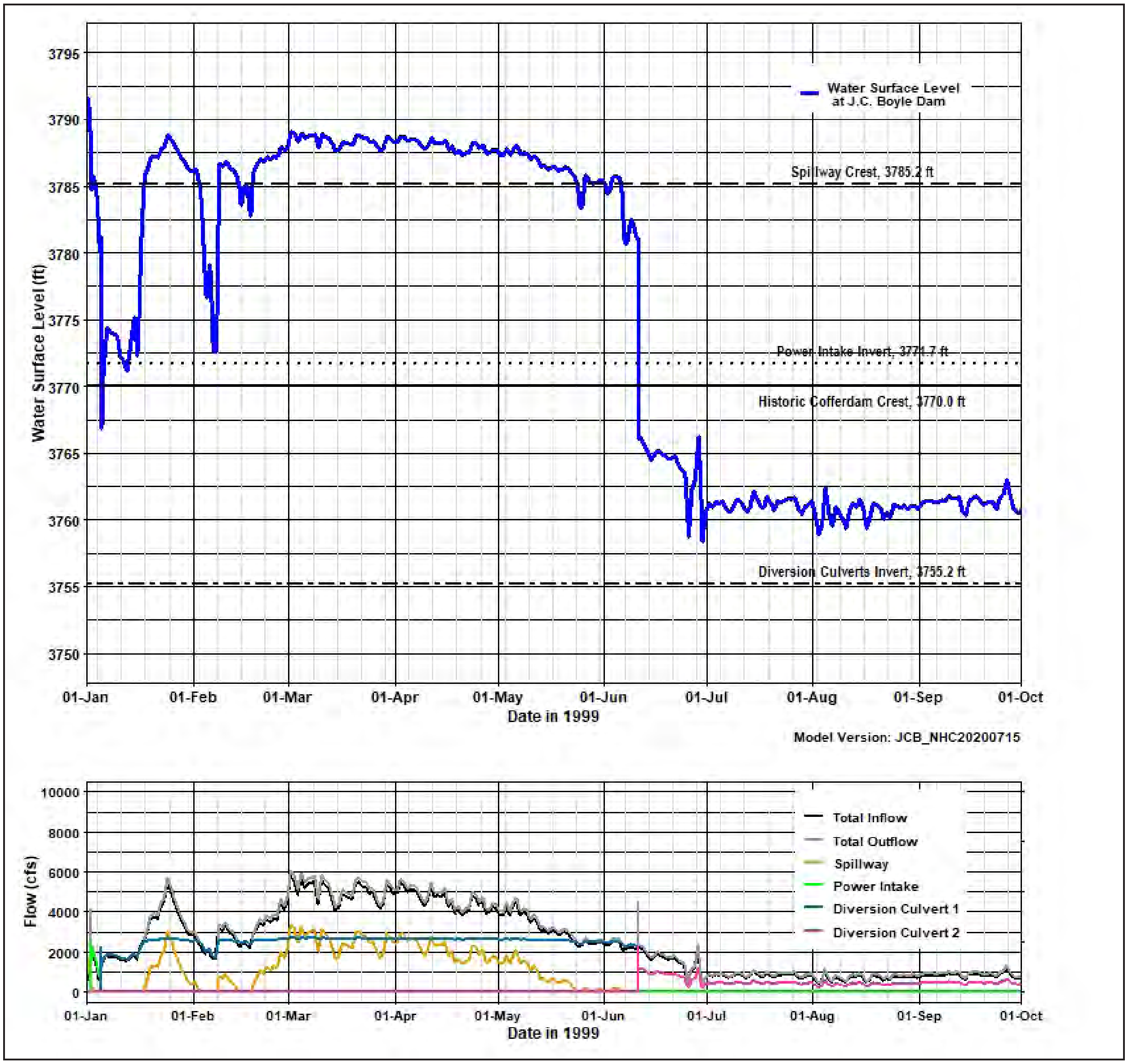

Figure 19 - J.C. Boyle Facility Simulated Drawdown - Year 1999

July 29, 2020

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

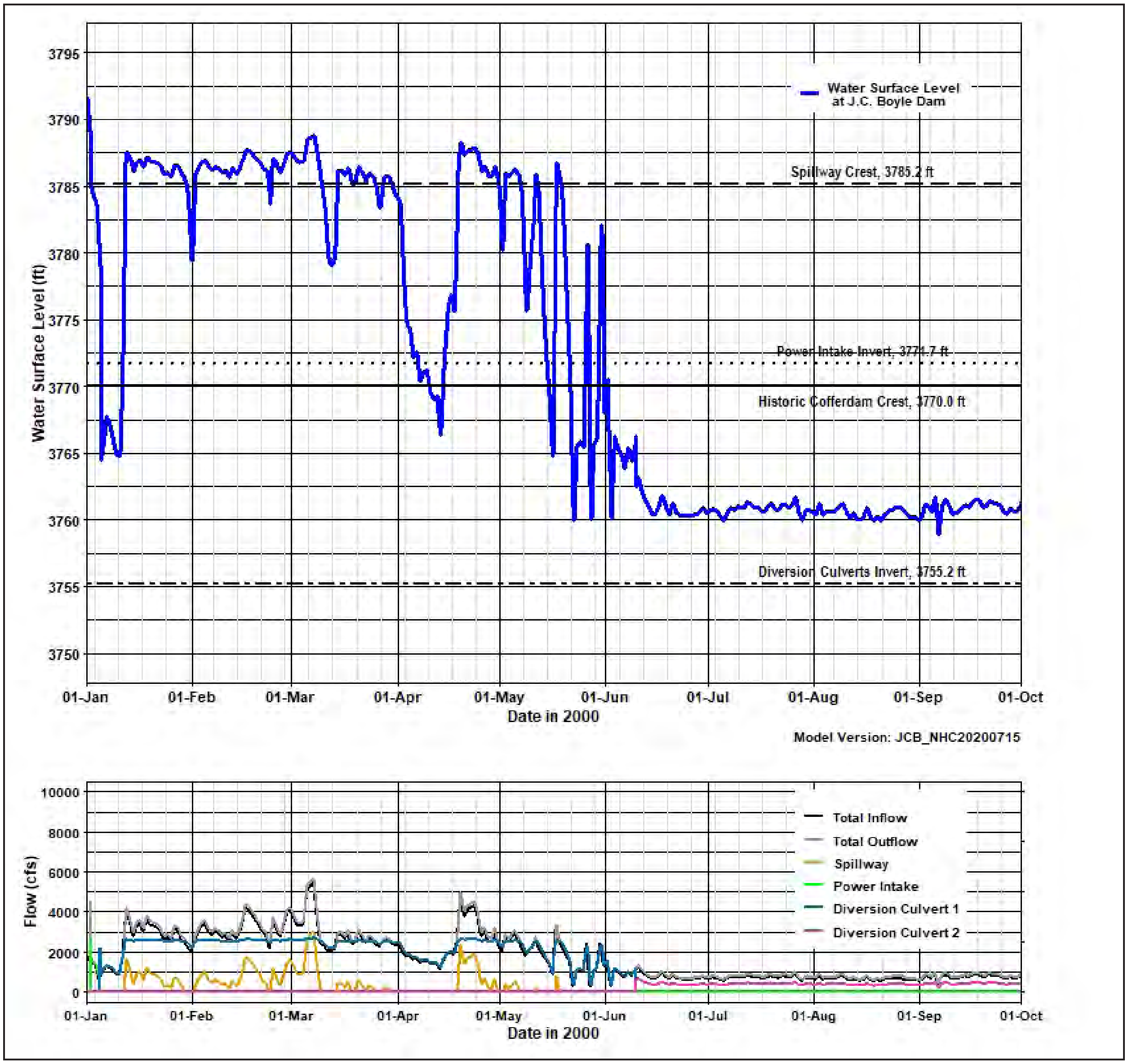

Figure 20 - J.C. Boyle Facility Simulated Drawdown - Year 2000

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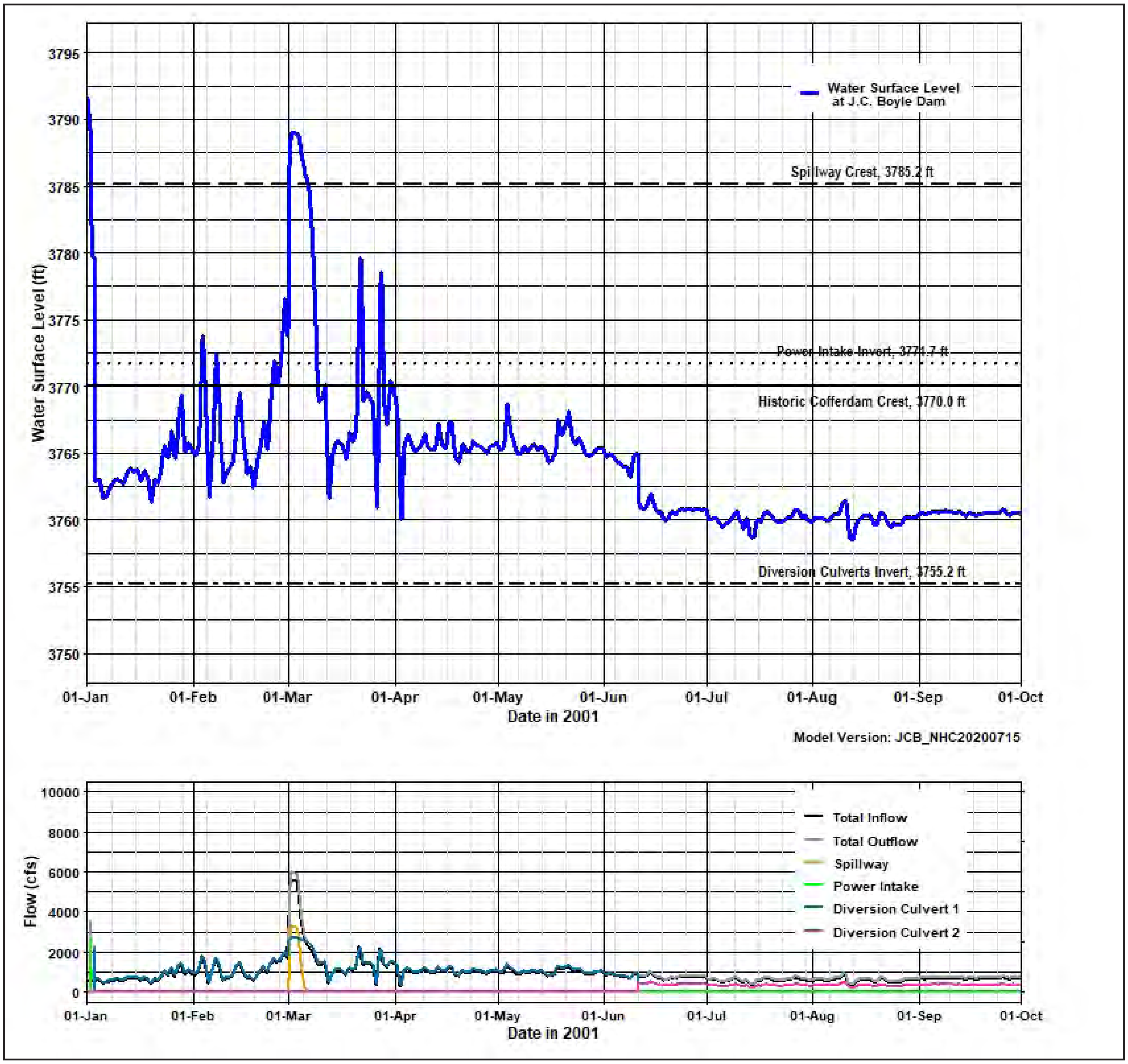

Figure 21 - J.C. Boyle Facility Simulated Drawdown - Year 2001

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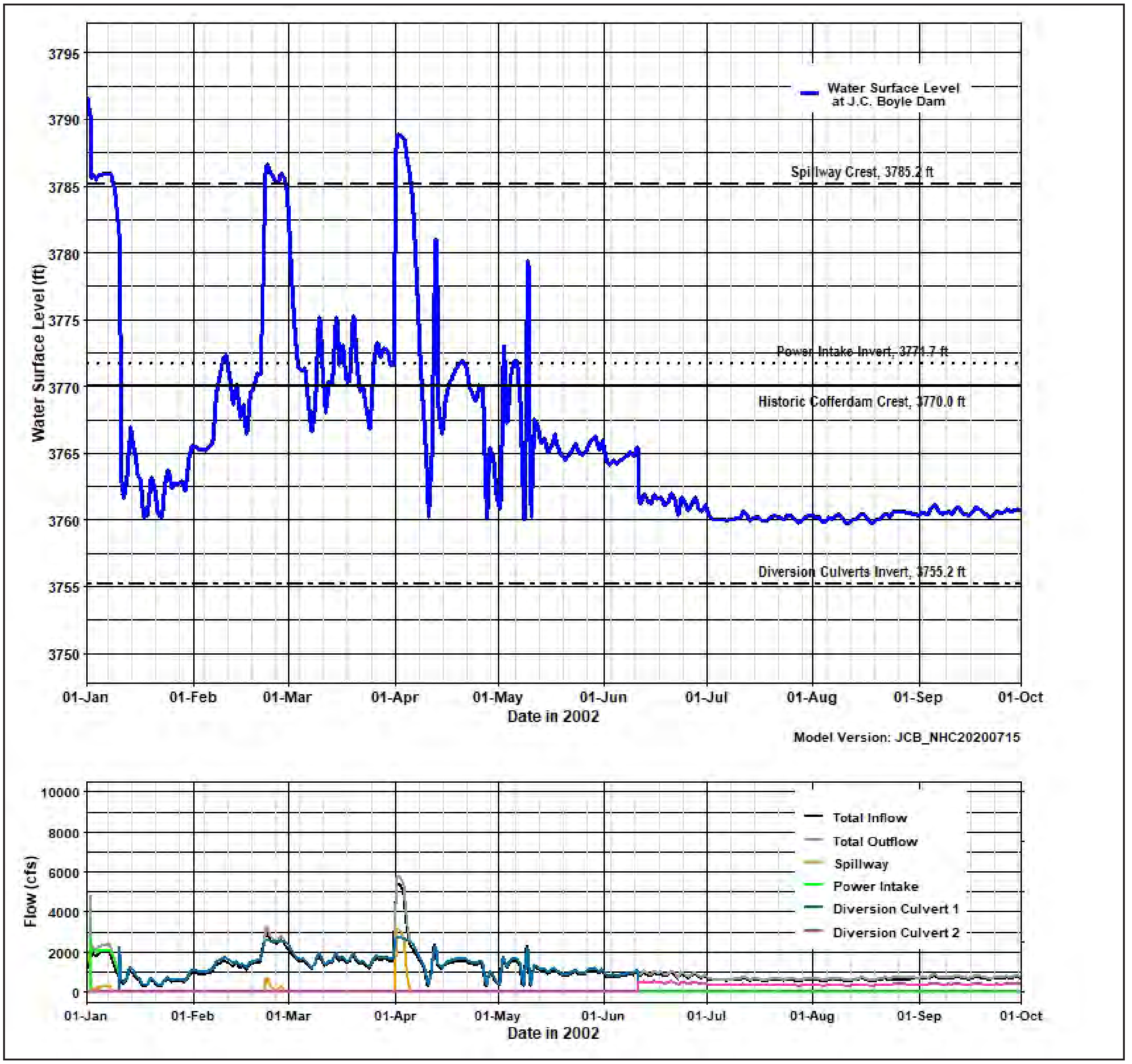

Figure 22 - J.C. Boyle Facility Simulated Drawdown - Year 2002

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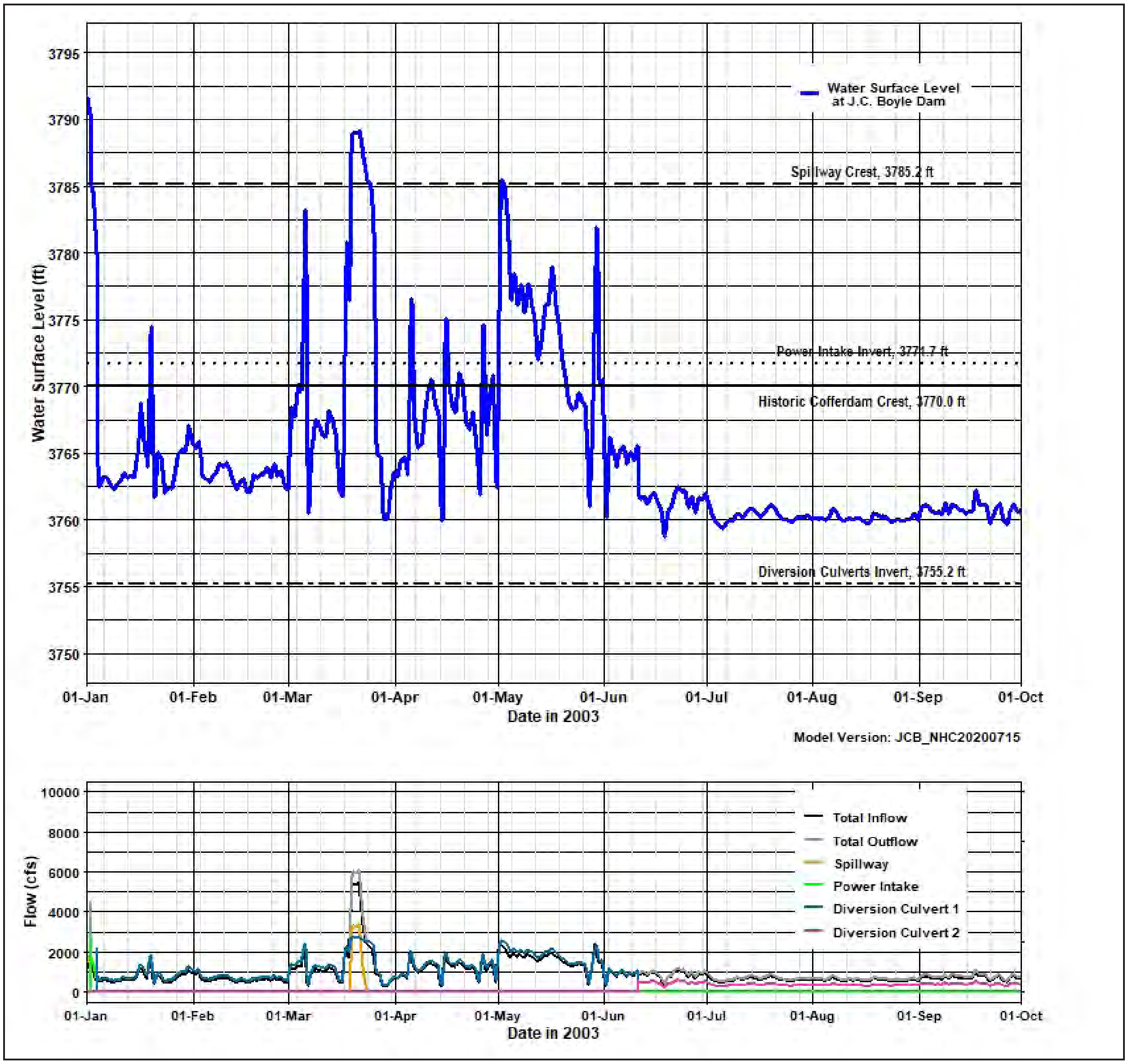

Figure 23 - J.C. Boyle Facility Simulated Drawdown - Year 2003

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

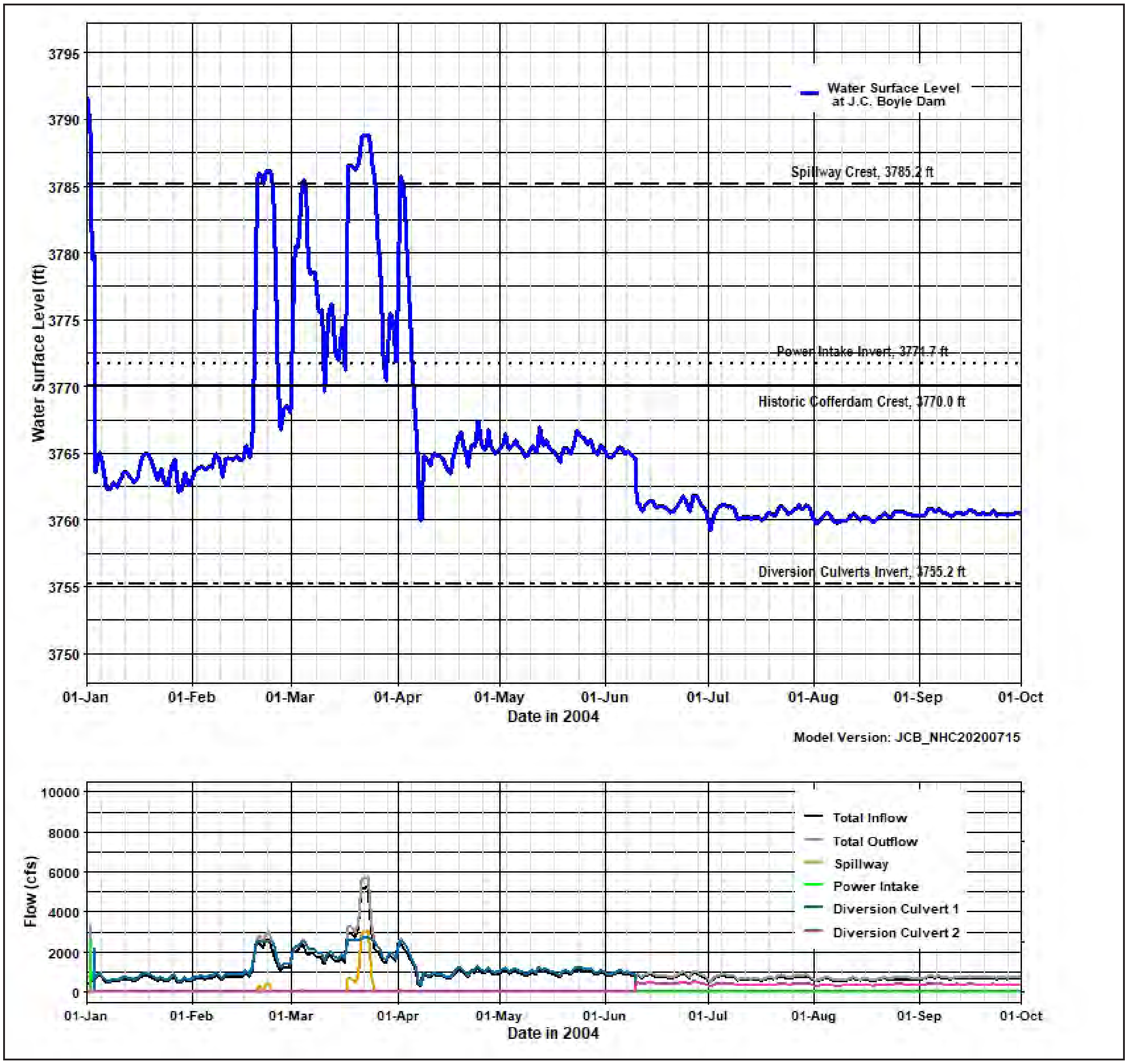

Figure 24 - J.C. Boyle Facility Simulated Drawdown - Year 2004

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

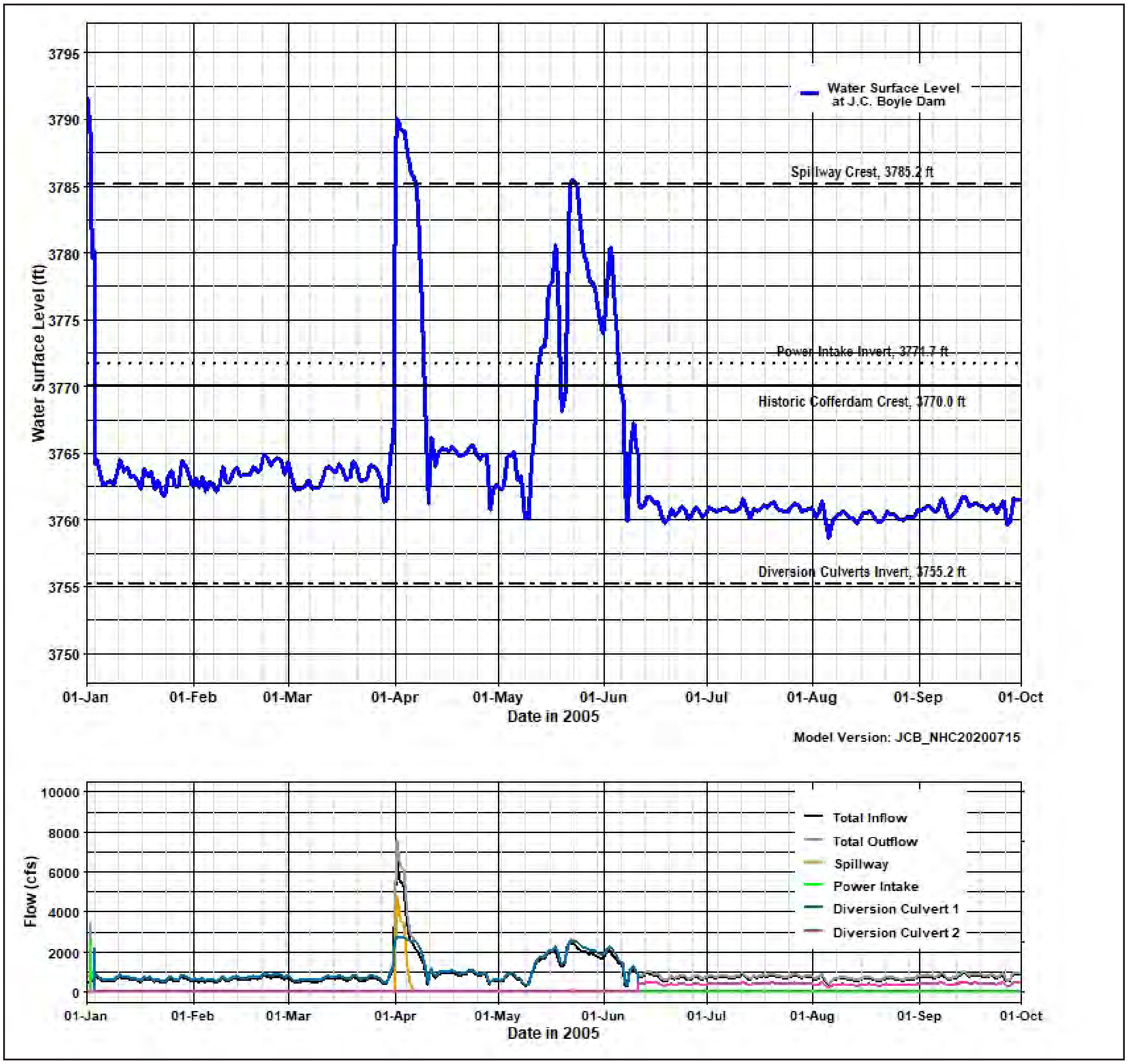

Figure 25 - J.C. Boyle Facility Simulated Drawdown - Year 2005

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

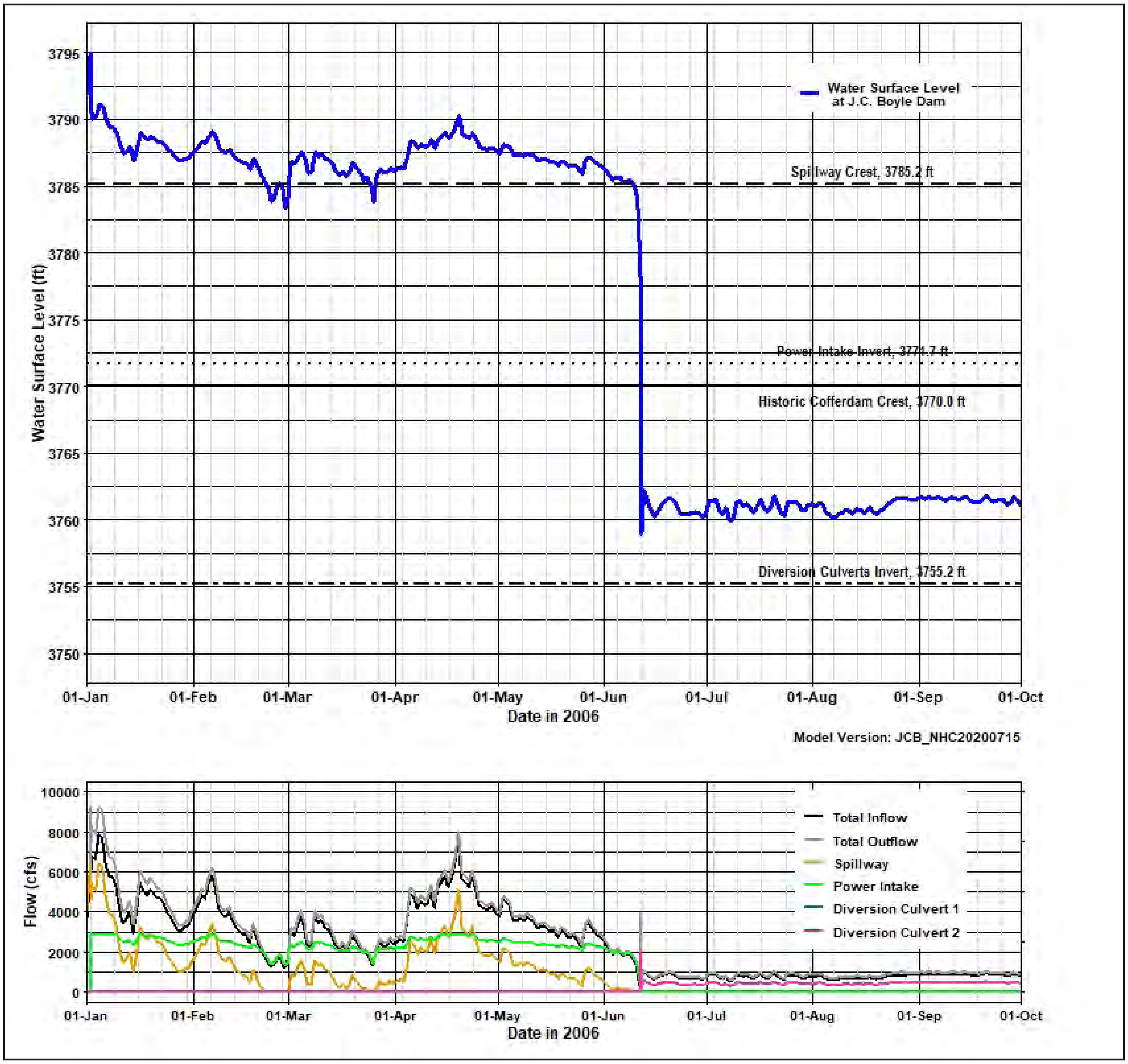

Figure 26 - J.C. Boyle Facility Simulated Drawdown - Year 2006

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

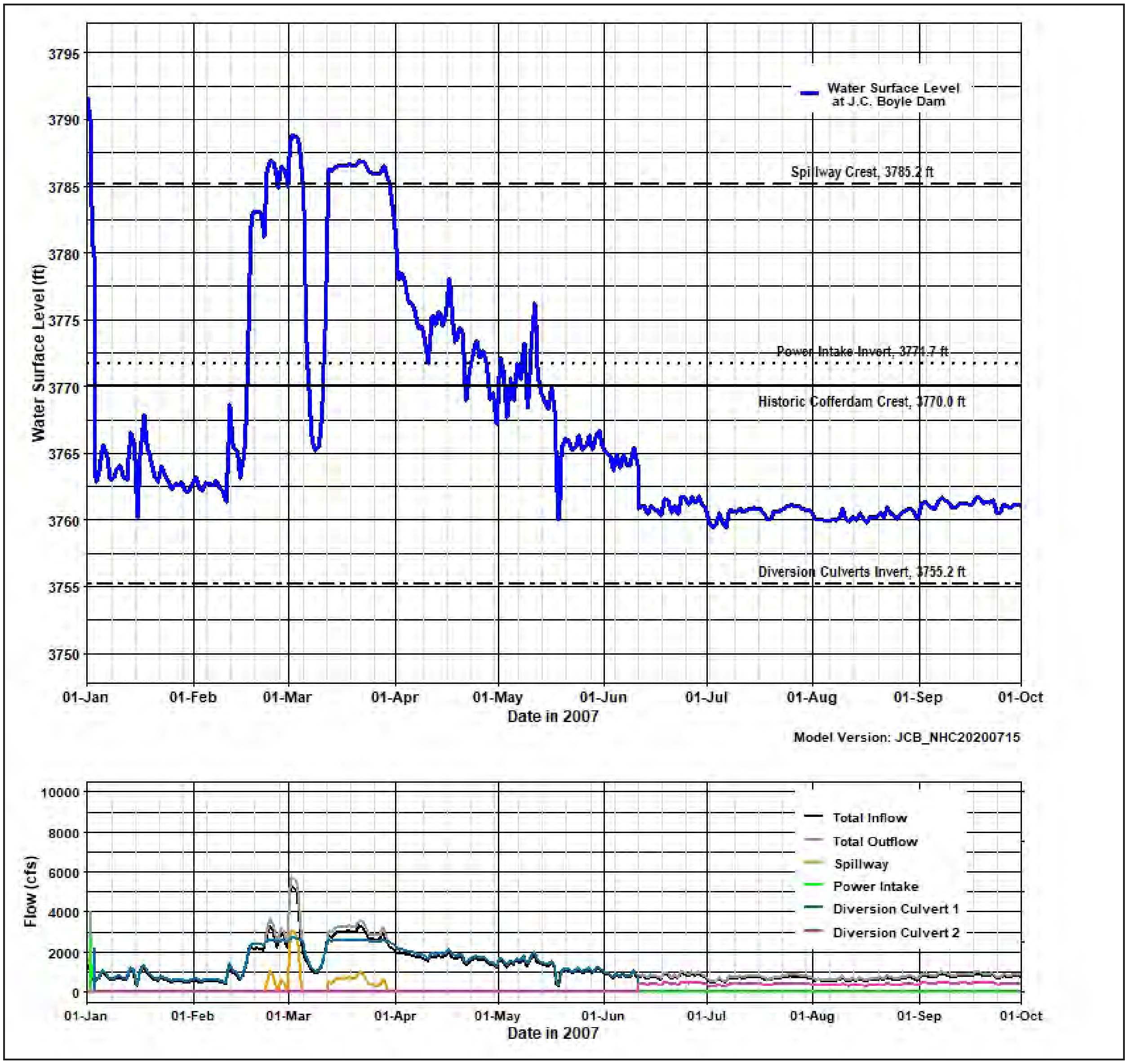

Figure 27 - J.C. Boyle Facility Simulated Drawdown - Year 2007

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

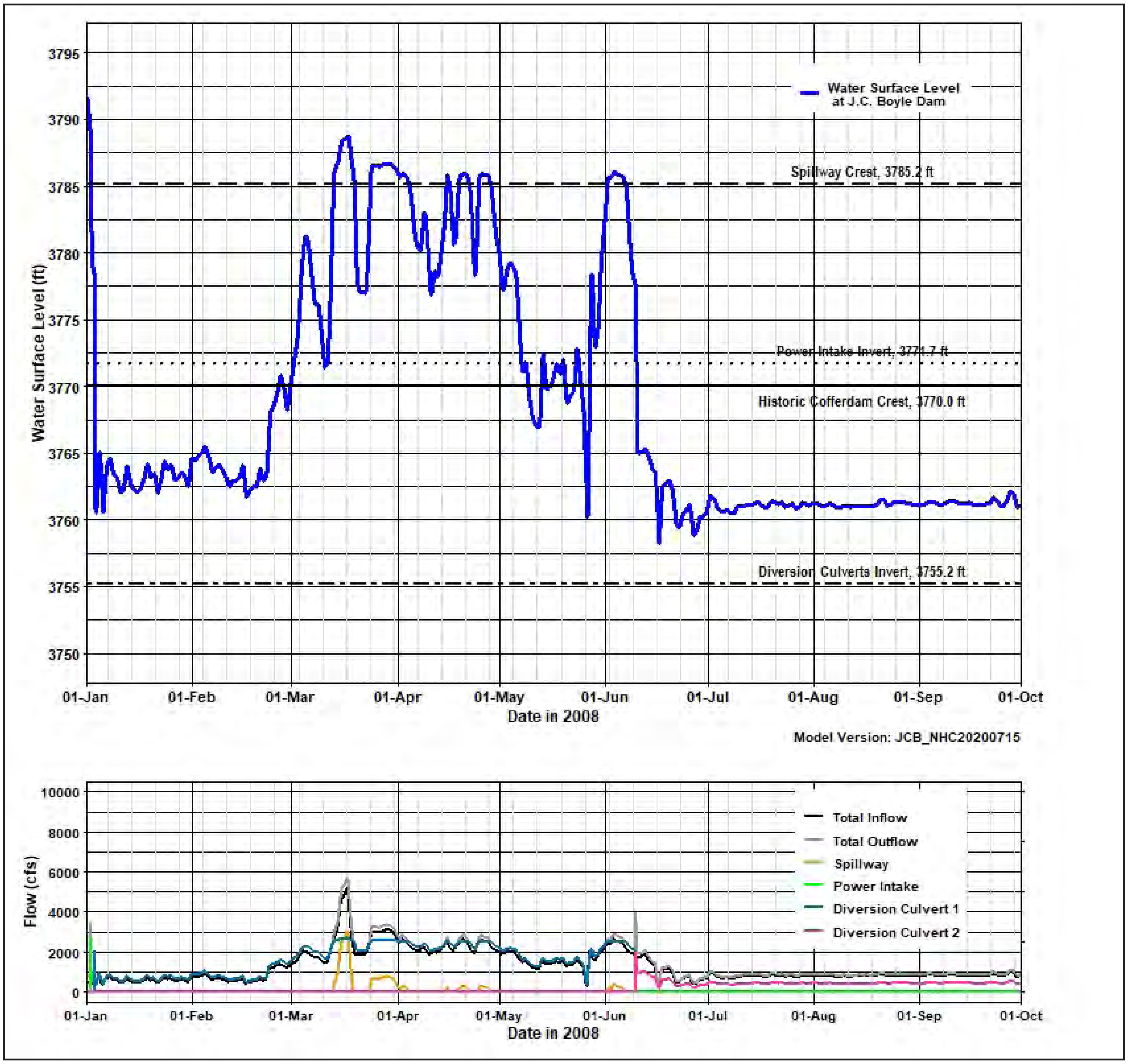

Figure 28 - J.C. Boyle Facility Simulated Drawdown - Year 2008

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

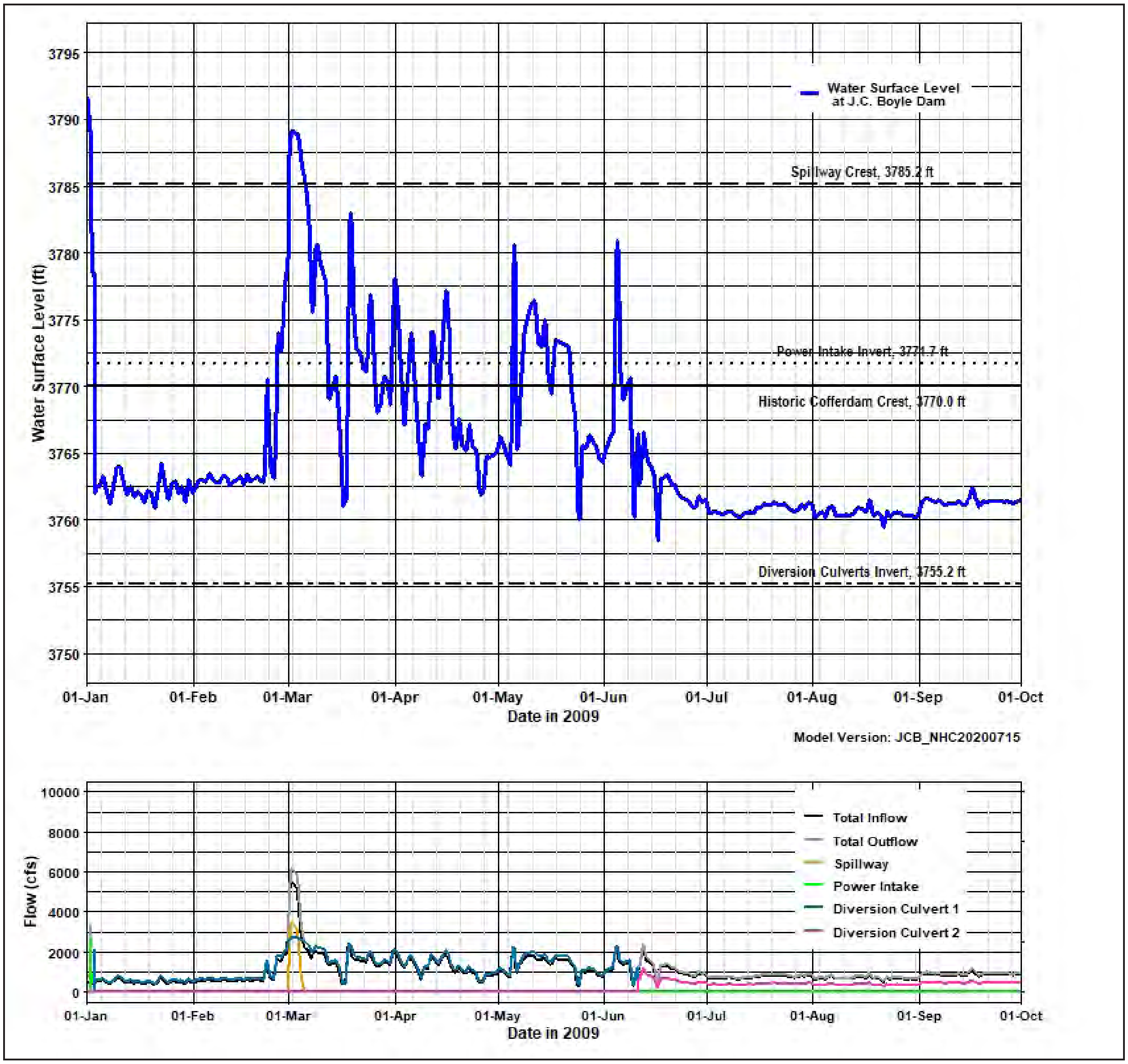

Figure 29 - J.C. Boyle Facility Simulated Drawdown - Year 2009

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

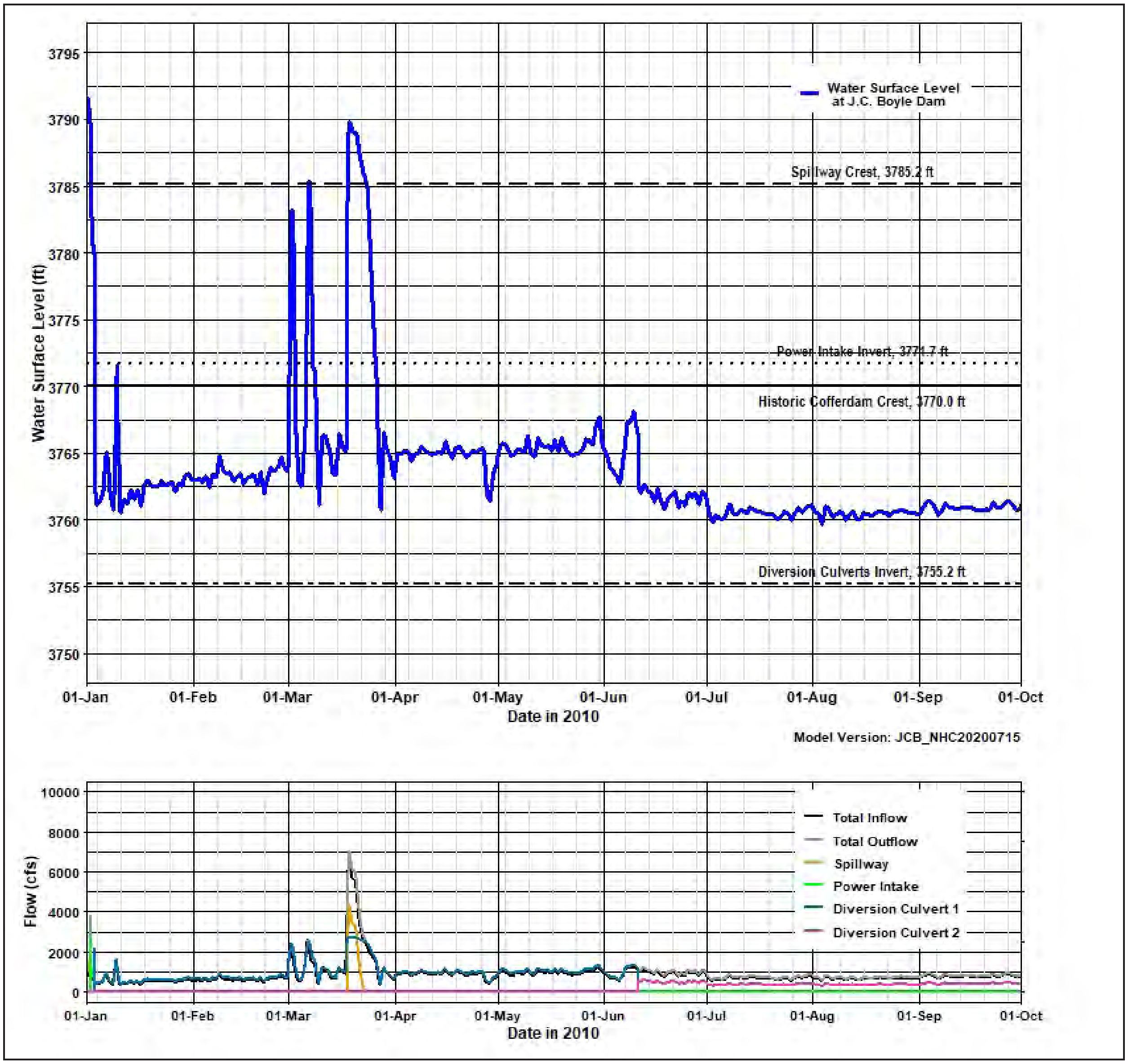

Figure 30 - J.C. Boyle Facility Simulated Drawdown - Year 2010

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

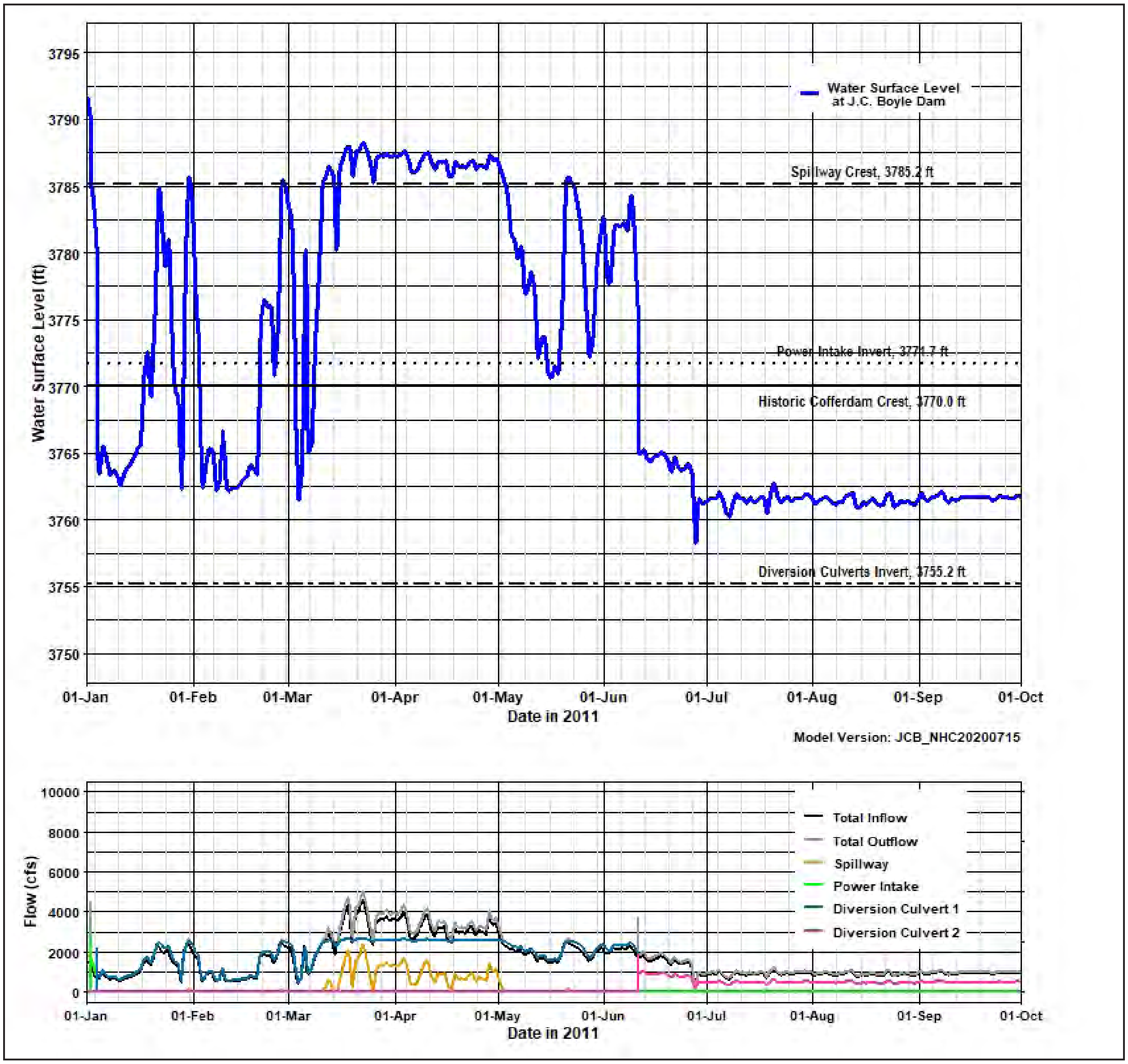

Figure 31 - J.C. Boyle Facility Simulated Drawdown - Year 2011

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

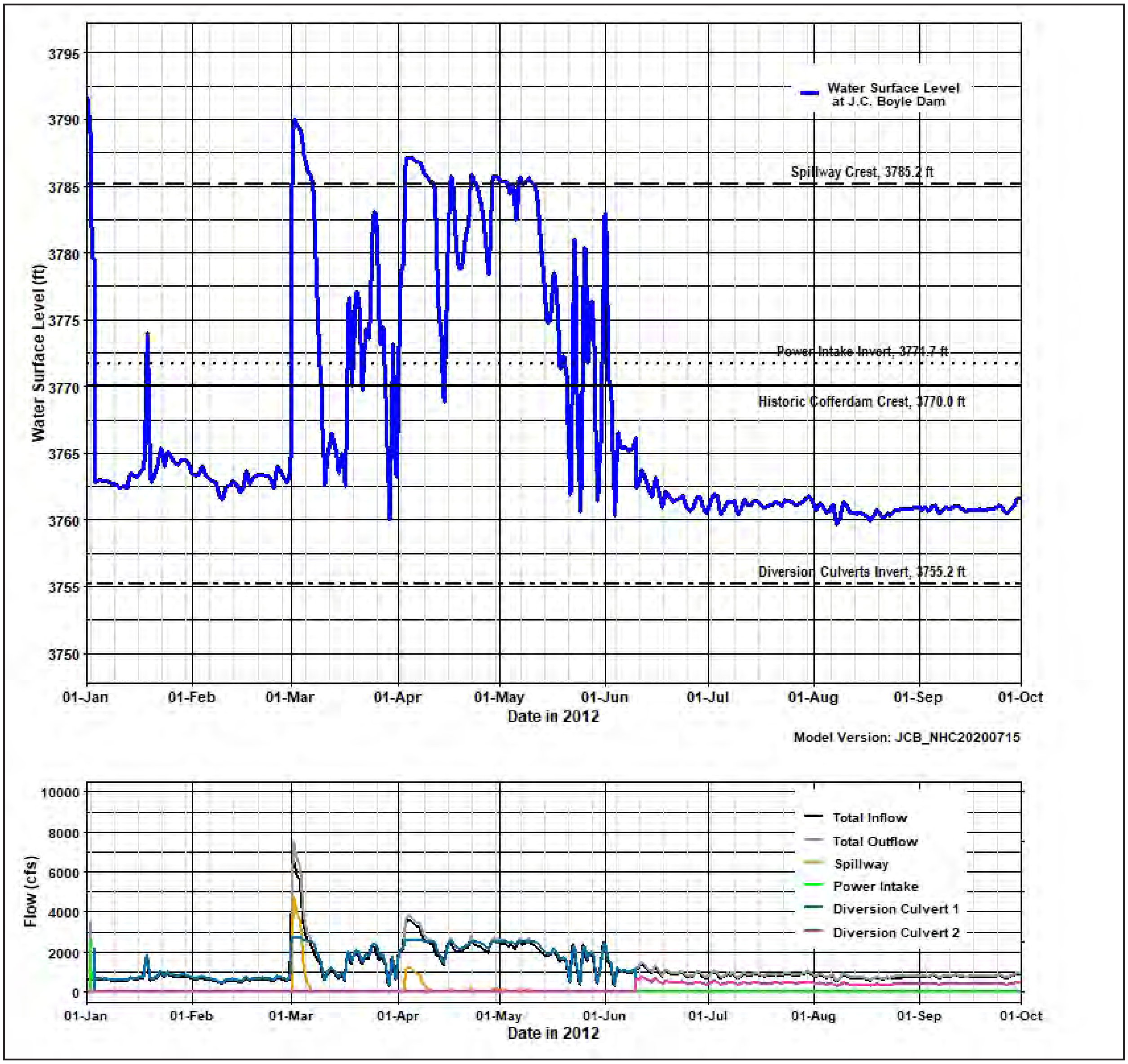

Figure 32 - J.C. Boyle Facility Simulated Drawdown - Year 2012

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\JCB_NHC20200715\individual_years

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

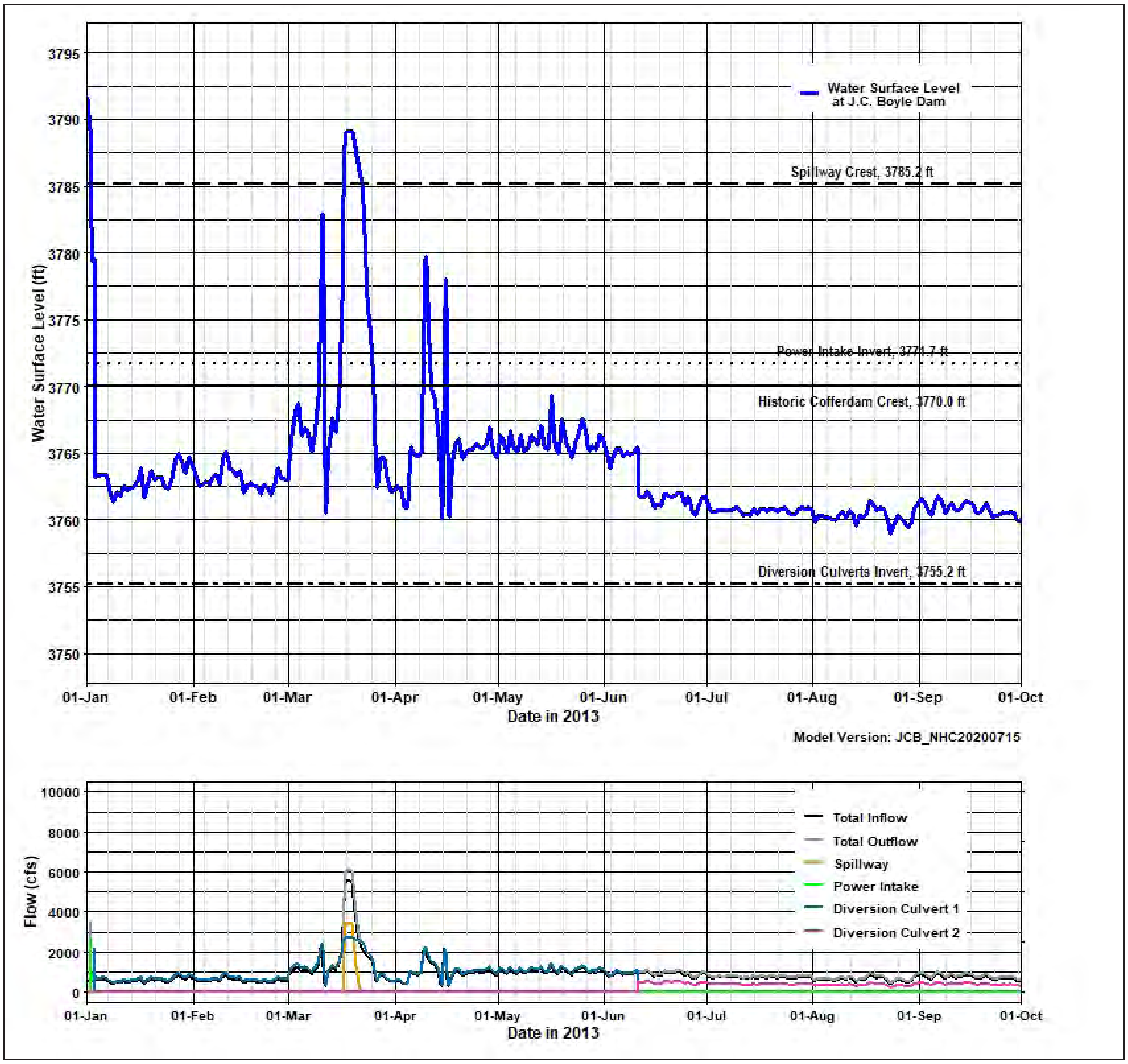

Figure 33 - J.C. Boyle Facility Simulated Drawdown - Year 2013

July 29, 2020

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

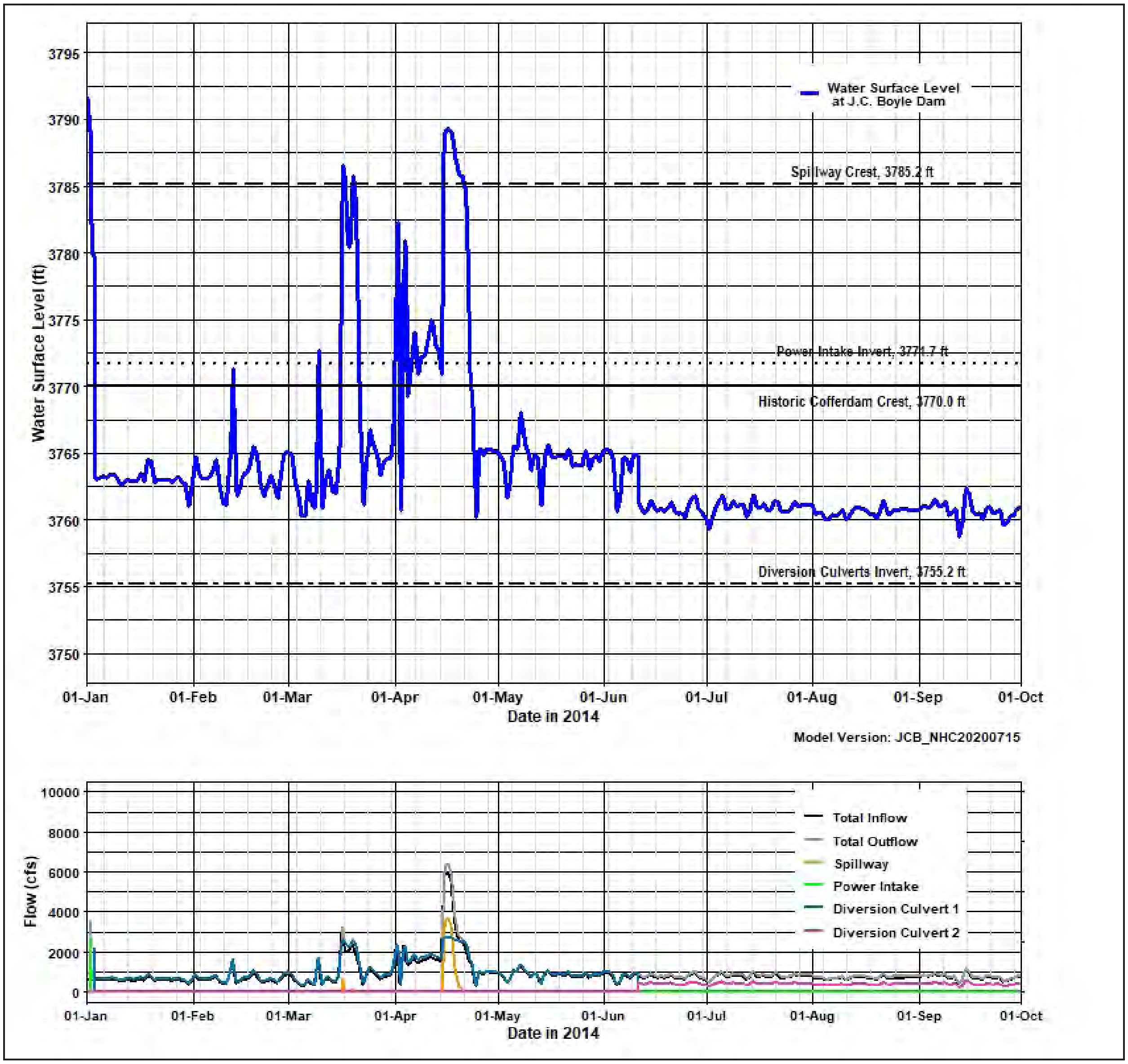

Figure 34 - J.C. Boyle Facility Simulated Drawdown - Year 2014

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J.C. BOYLE FACILITY SIMULATED DRAWDOWN

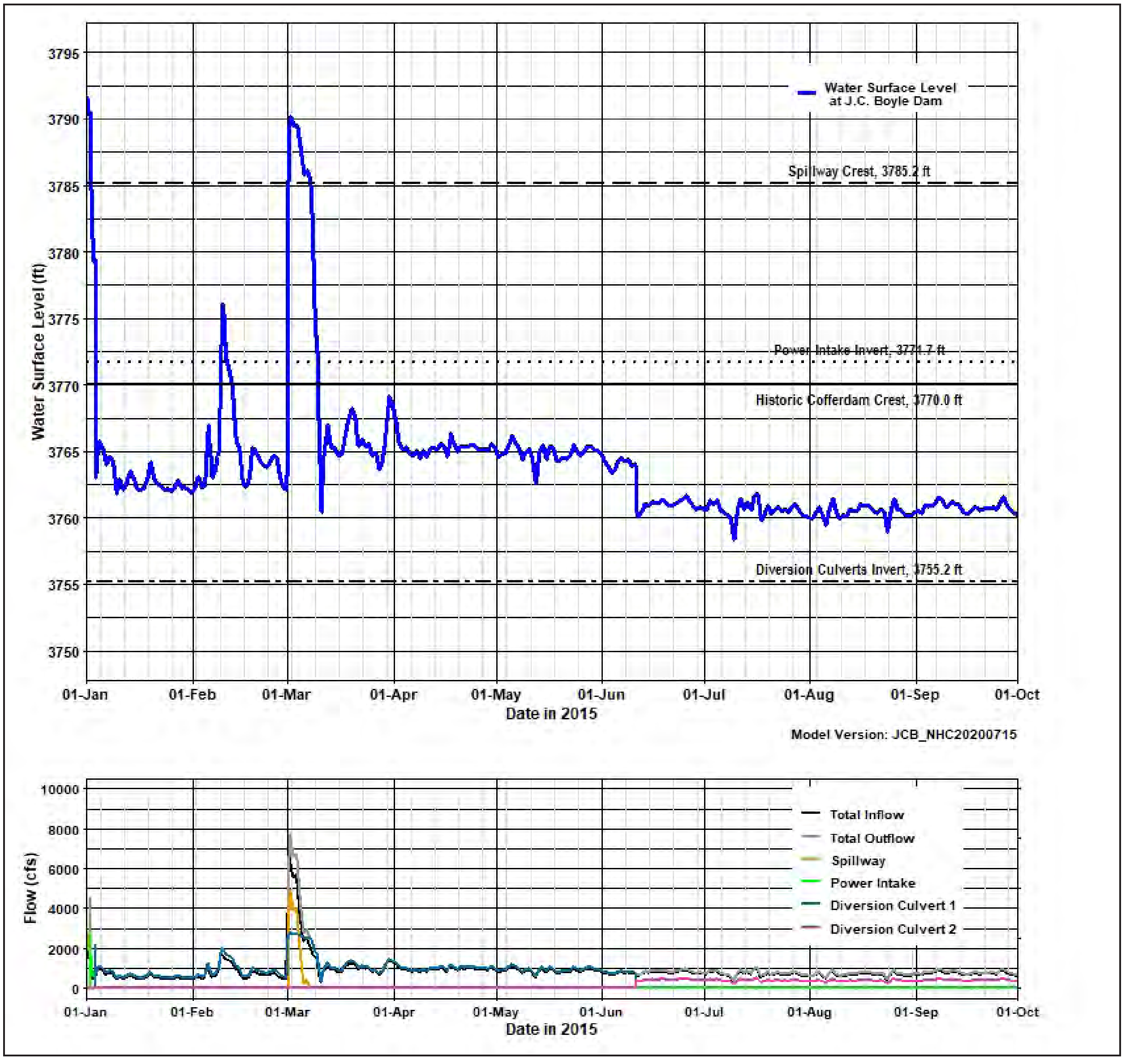

Figure 35 - J.C. Boyle Facility Simulated Drawdown - Year 2015

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J.C. BOYLE FACILITY
SIMULATED DRAWDOWN

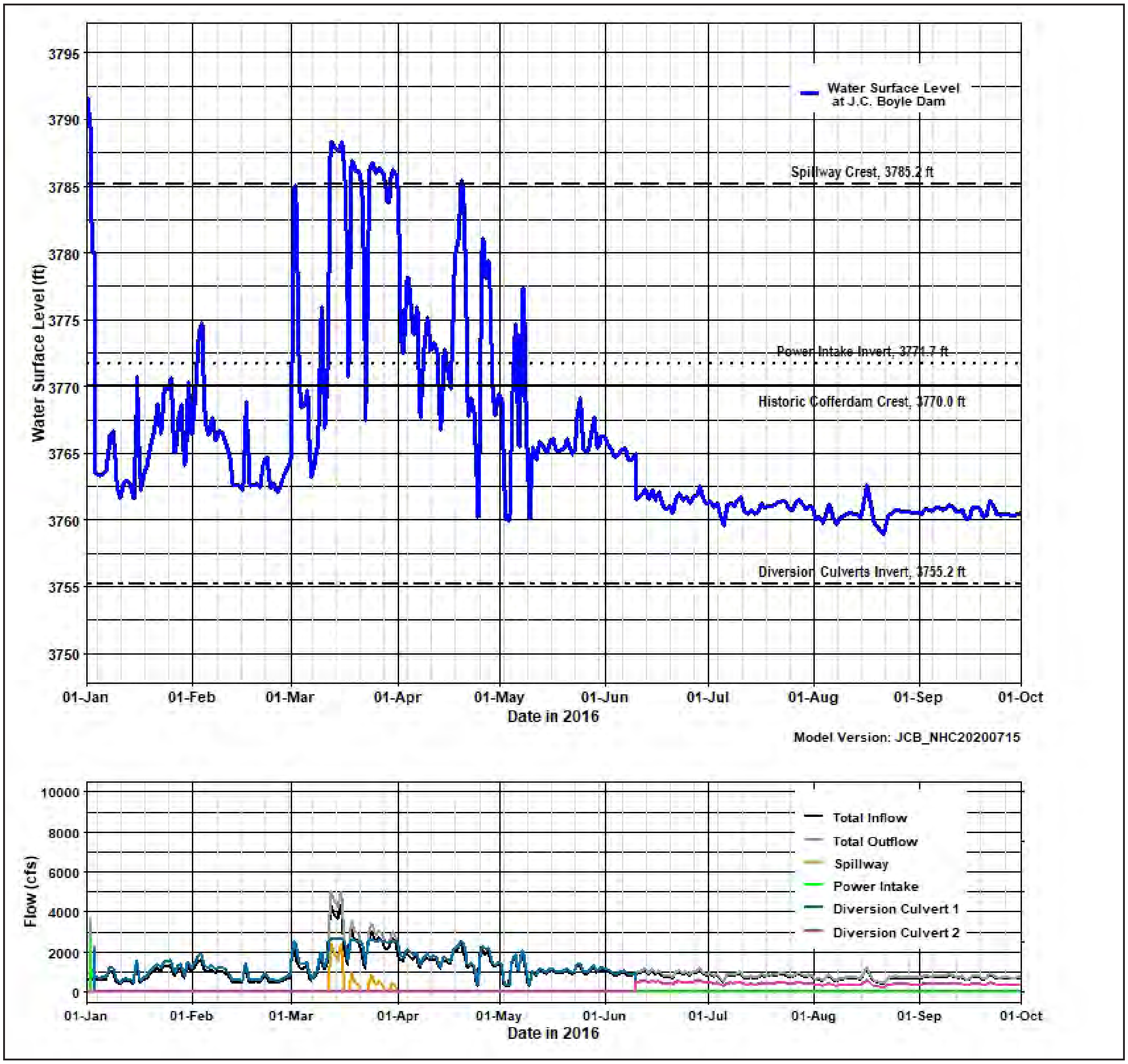

Figure 36 - J.C. Boyle Facility Simulated Drawdown - Year 2016

July 29, 2020

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nhc
northwest hydraulic consultants

NHC Ref. No. 2004947.11.5

21 September 2020

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

Attention: **Norm Bishop**

Re: **CFD Modeling of J.C. Boyle – 100% Design**

DRAWDOWN OPERATIONS – 100% DESIGN

Knight Piésold Ltd. (KP) proposed rules for outlet operations during drawdown of the J.C. Boyle Reservoir (KP, 2020a). NHC conducted computational fluid dynamics (CFD) model simulations to develop rating curves to be applied to the drawdown modeling for the following conditions:

- Spillways and power intake are open;
- Spillways and Diversion Culvert #1 are open and power intake is closed; and,
- Spillways, Diversion Culvert #1 and Diversion Culvert #2 are open and power intake is closed.

COMPUTATIONAL FLUID DYNAMICS MODELING

NHC conducted CFD modeling of the J.C. Boyle Dam outlet structures to verify spillway and culvert capacities at various reservoir water surface elevations (RWS). This information will be used to assist the design work for drawdown and be used to adjust rules for the J.C. Boyle one-dimensional drawdown modeling. The following conditions were simulated to determine capacities at the various stages KP identified for drawdown (Table 1):

1. Reservoir at normal operating water level of RWS = El. 3,796.7 ft; all three spillway gates fully open and power intake (invert El. 3,771.7 ft) diverting 2,800 cfs. This simulation is intended to verify the spillway can pass 15,400 cfs. Note, the power intake may be operated to divert up to 2,850 cfs; however, this is considered within the tolerance of the model and model results.
2. RWS = El. 3,780.6 ft, which is below the crest of the spillway (El. 3,785.2 ft). Power intake assumed closed. This simulation is intended to verify the capacity of Culvert #1.
3. RWS = El. 3,766.0 ft. This simulation is intended to verify the combined flow capacity of Culvert #1 and Culvert #2.

Table 1. Simulated Conditions

Simulation	RWS (ft., NAVD88)	Spillway	Intake	Culvert 1	Culvert 2
1	3,796.7	Fully open	2,800 cfs	Closed	Closed
2	3,780.6	0 cfs	0 cfs	Open	Closed
3	3,766.0	0 cfs	0 cfs	Open	Open

GEOMETRY AND ROUGHNESS

The model terrain includes topo-bathymetric data (GMA, 2018) of the J.C. Boyle Reservoir and outlet channel. Also included are the spillway, outlet culverts, and intake structure per the 1956 plans (elevations in project datum, Ref. Drawings G-7585, G-8215, G- 8337, AA-78084-A, AA-78085A, AA-78087A, AA-78114A). All project elevations were converted from the historical project datum to NAVD88 using a +3.7 ft conversion. All elevations mentioned in this memo are in NAVD88 project datum.

The J.C. Boyle ogee spillway comprises three 36-ft wide bays separated by 4.5-ft wide piers. The crest of the spillway is at El. 3,785.2 ft and the top of the spillway deck at El. 3,803.7 ft. The normal operating level is RWS 3,796.7 ft. Below the central and right (north) bays are located the two historical diversion culverts that were used during construction and are currently plugged. The culverts are 9.5 ft wide and 10 ft high with invert inlet elevations at El. 3,755.2 ft. The intake tower to the 14-ft diameter steel pipe is located on the left (south) side. The pipe invert inlet elevation is El. 3,771.7 ft. Spillway gates were not incorporated in the CFD geometry. For the first simulation (Table 1), the two culverts remained plugged. For second simulation, the plug in Culvert # 1 was removed; while for the remaining simulations, the plugs from both culverts were removed.

The CFD model uses roughness height to evaluate friction losses. The roughness heights for the ground, spillway, and power intake are assumed to be 1 ft, 0.002 ft, and 0.001 ft, respectively. Model sensitivity to roughness height was not tested, as flow upstream is controlled by the spillway's weir and culvert orifice, with friction losses playing a minor role. Downstream from the dam, the river channel goes over a very steep reach and tailwater levels and roughness do not play a major influence on spillway capacity.

MESH INDEPENDENCE

The 3D mesh was developed to balance model accuracy and computation time. Decreasing the mesh cell size increases model accuracy and computation time, though there is a mesh resolution for which additional refinement does not yield significant changes in the solution. Table 2 summarizes the results of spillway capacity sensitivity to mesh cell size. The solution using the 0.25-ft cell size at the spillway is used to determine the spillway discharge coefficient, and the 1-ft cell size around the spillway is considered optimal for further simulations. The 3D mesh used for subsequent simulations includes 8-ft elements through the reservoir with a 1-ft refinement region around the spillway.

Table 2. Mesh Independence Comparison - Spillway Capacity

Cell Size (ft)		# cells	CPU	Discharge	Relative
Dam	Spillway	(million)	Time (h)	(cfs)	difference
1.00	1.00	7.8	6	18,284	-
1.00	0.50	12.5	16	17,455	-4.5%
1.00	0.25	15.5	6	17,848	2.1%

SIMULATION 1 RESULTS (SPILLWAY OPEN)

Since spillway rating curves were not available, this simulation was intended to verify the spillway capacity. The reservoir was set at normal operating water level of El. 3,796.7 ft, with the all three spillway gates removed and the power intake diverting 2,800 cfs. Under these conditions, average spillway capacity is approximately 17,850 cfs.

Figure 1 below illustrates velocity variations over the spillway. The spillway outlet channel is relatively steep with the spillway high above the ground, so backwater or a submerged tailwater condition are not expected. The water jet reaches a maximum velocity of approximately 50 ft/s at the toe of the spillway.

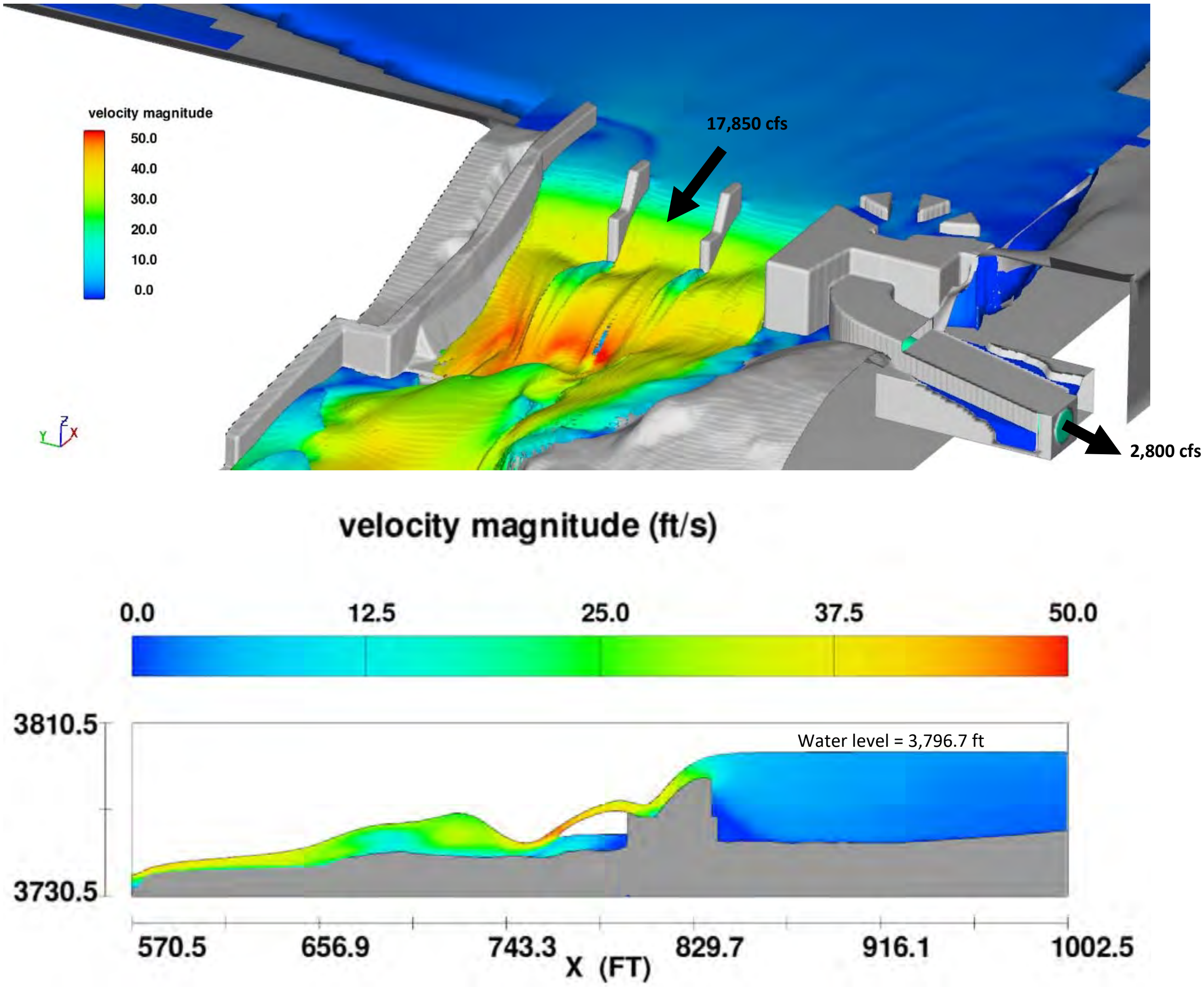


Figure 1. Results for Simulation 1 – Spillway with reservoir water level El. 3,796.7 ft.

SIMULATION 2 RESULTS (CULVERT #1 OPEN)

This simulation was conducted to provide a datapoint to verify the left (southernmost) culvert capacity (Culvert #1). The reservoir is operating at El. 3,780.6 ft (4.6 ft below the crest of the spillway) with no intake diversion (i.e. power tunnel discharge of 0 cfs). The culvert opening is assumed per the design drawings (Ref. Drawing AA-78085A, AA-78087A), rectangular with 9.5-ft span and 10-ft rise. Model results for this condition show that the culvert is not flowing full and there is no tailwater submerging the outlet (note that tailwater effects are not expected for total river flows up to at least 9,500 cfs); therefore, at this water level, the culvert capacity is inlet-controlled and flow capacity is approximately 2,200 cfs, with an orifice discharge coefficient of approximately 0.64, which is reasonable for a square edge orifice. In this scenario, flow reaches a maximum velocity of approximately 45 ft/s at the culvert outlet. Figure 2 below illustrates velocity variations at the culvert outlet.

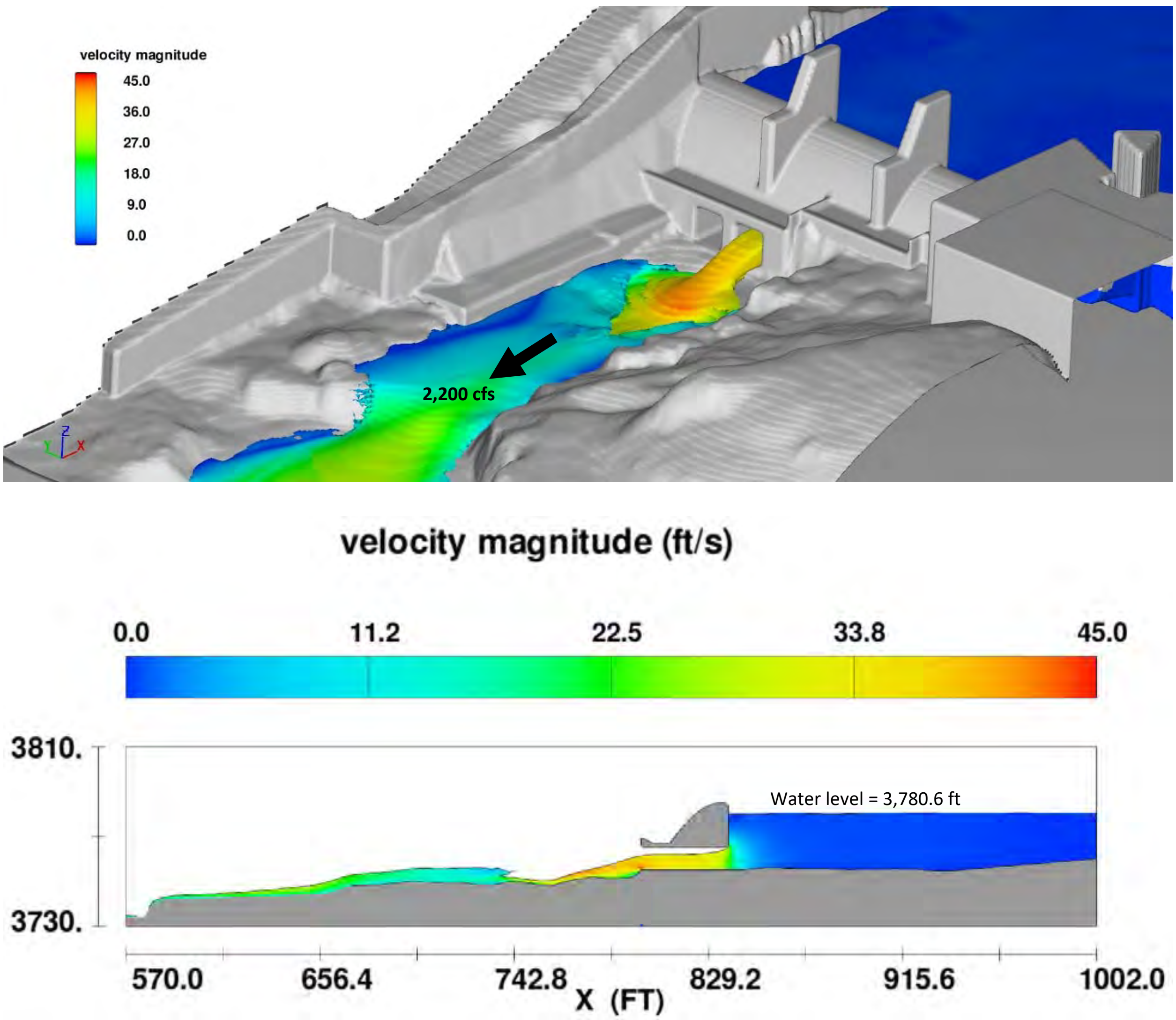
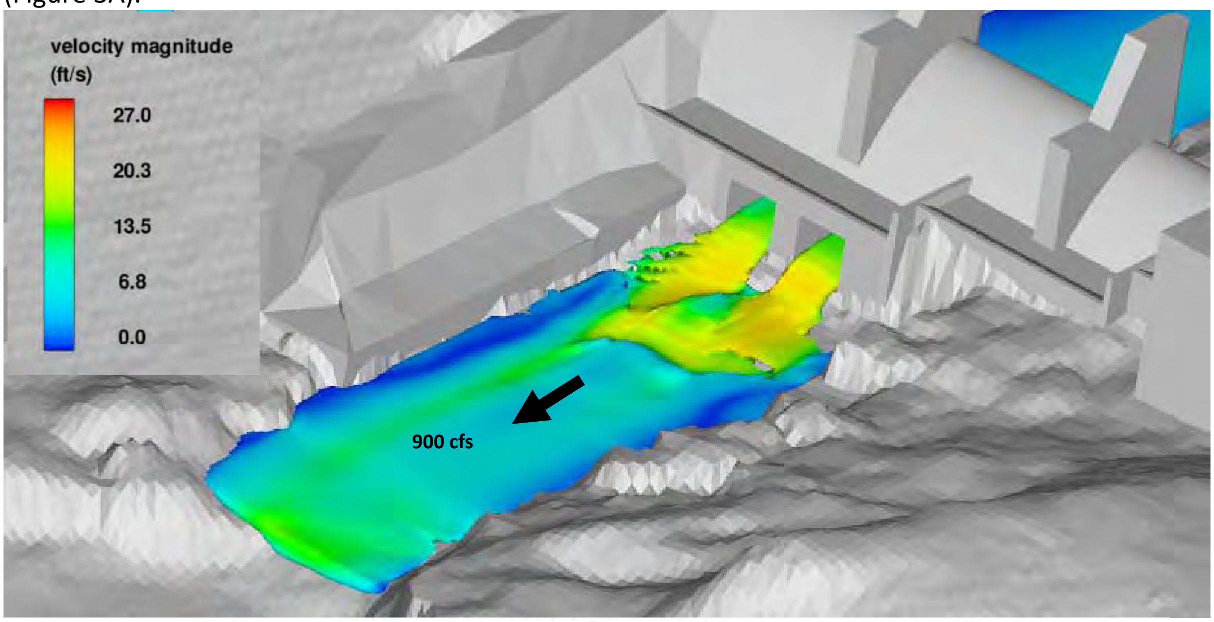


Figure 2. Results for Simulation 2 – Culvert # 1 open with reservoir water level El. 3,780.6 ft.

SIMULATION 3 RESULTS (TWO CULVERTS OPEN)

For both Simulations 1 and 2, the water surface in the reservoir remains relatively flat and it is easy to determine a unique RWS value. However, for Simulation 3, there was a strong gradient in the water surface as the water concentrated and accelerated along the steep historically excavated approach channel upstream of the diversion culverts. Because of this, two CFD simulations with two different waters levels were conducted for Simulation 3.

The first, Simulation 3A, provided capacity verification of both culverts when the reservoir was operating at El. 3,766 ft with no power intake diversion. The selected water level of El. 3,766 ft corresponded to a location in the approach channel approximately 300 feet upstream of the dam, near the historical cofferdam, where the slope of the approach channel is approximately 5 percent. When the approach channel water surface is at El. 3,766 ft, this corresponded to a water surface of approximately El. 3,761.3 ft just upstream of the dam, and a flow of approximately 900 cfs through the culverts (Figure 3A).



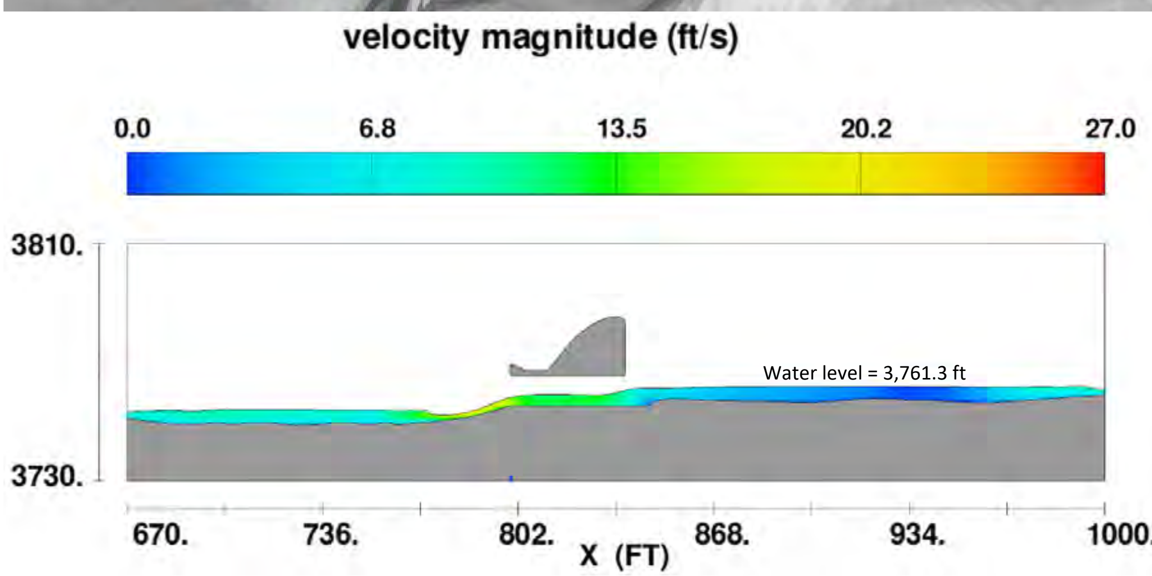
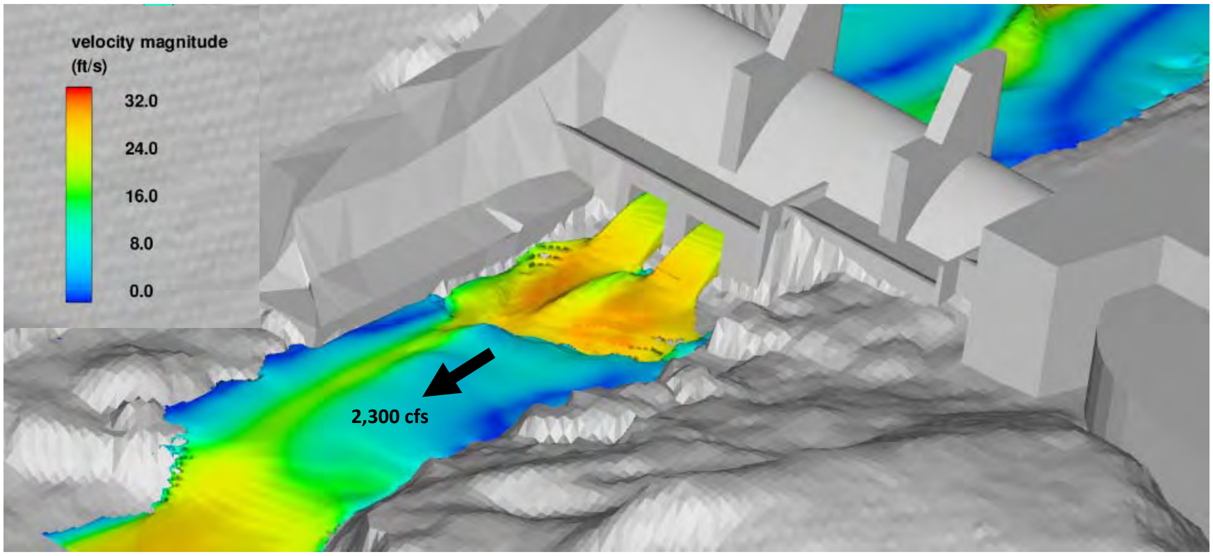


Figure 3A. Results for Simulation 3A (Two culverts - 900 cfs)

A second Simulation 3B was conducted to determine the outlet capacity when the water surface just upstream of the dam was at El. 3,766 ft. A combined flow of approximately 2,300 cfs can be conveyed by both culverts when the water surface just upstream of the dam is at El. 3,766 feet. The orifice discharge coefficients become 0.59 for Culvert # 1 and 0.62 for Culvert # 2. Figure 3B below illustrates velocity variations at the outlet of the culverts for this condition.



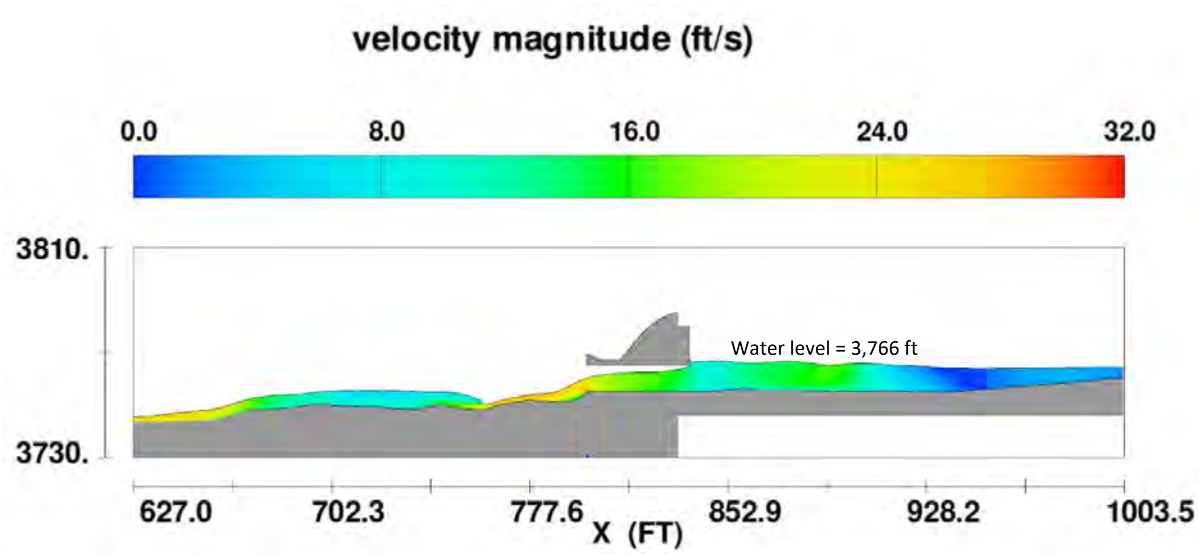


Figure 3B. Results for Simulation 3B (Two culverts - 2,300 cfs)

RATING CURVES

Figure 4 shows the discharges modeled in each CFD simulation as described above and the rating curves for various combinations of outlets. Rating curves reference a range of discharge coefficients computed from the various modelled scenarios. The spillway discharge coefficient varies from 4.0 to 4.2 $ft^{0.5}/s$, and culvert orifice discharge coefficients vary from 0.56 to 0.77. Appendix A provides tables of the data used to produce the curves shown in Figure 4 and for integration into the 1D drawdown modeling effort.

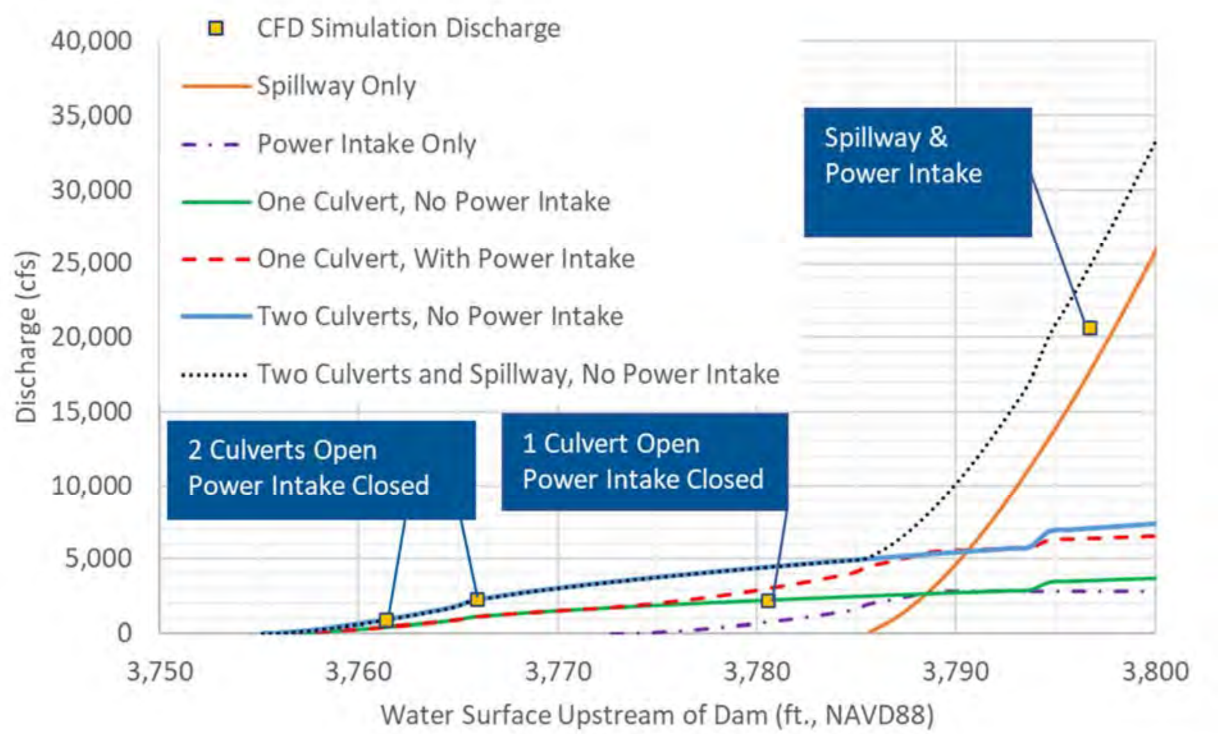


Figure 4. Rating Curves for J.C. Boyle Dam (Note: Power intake rating curve provided by KP, 2020b)

DAM EMBANKMENT WATER SURFACE ELEVATIONS

Reservoir water surface elevations at the center of the dam embankment, upstream of the dam, were requested for a range of annual probable flows when all outlets are open except the power intake. The definition of the flows was defined in the 60% Design Report (KP and RES, 2020). Computation time for this number of simulations would be significant using a CFD model, so NHC developed a HEC-RAS 2D model to assist with providing a wide range of results upstream of J.C. Boyle in a timely manner. Results include an estimate of water surface elevations at the center of the upstream embankment and shear stresses at the headpond.

The HEC-RAS 2D model uses the post-drawdown outlet rating curve for the spillway and two culverts developed from the CFD model results as a 2D flow area connection to simulate the dam outlet works. Notice that water levels at the embankment are higher than at the concrete structure (spillway and culverts), especially during low flows which generate a strong water surface gradient. For example, for a discharge of 900 cfs, the water level difference is 6 ft.

Figure 5 shows the estimated reservoir water surface elevations at the embankment. Appendix A provides tables of the data used to produce Figure 5. Figures 6 through 9 illustrate the FLOW 3D and 2D model simulations of localized drawdown at the outlet structure and resulting variability in water surface elevations between the outlet structure and the embankment.

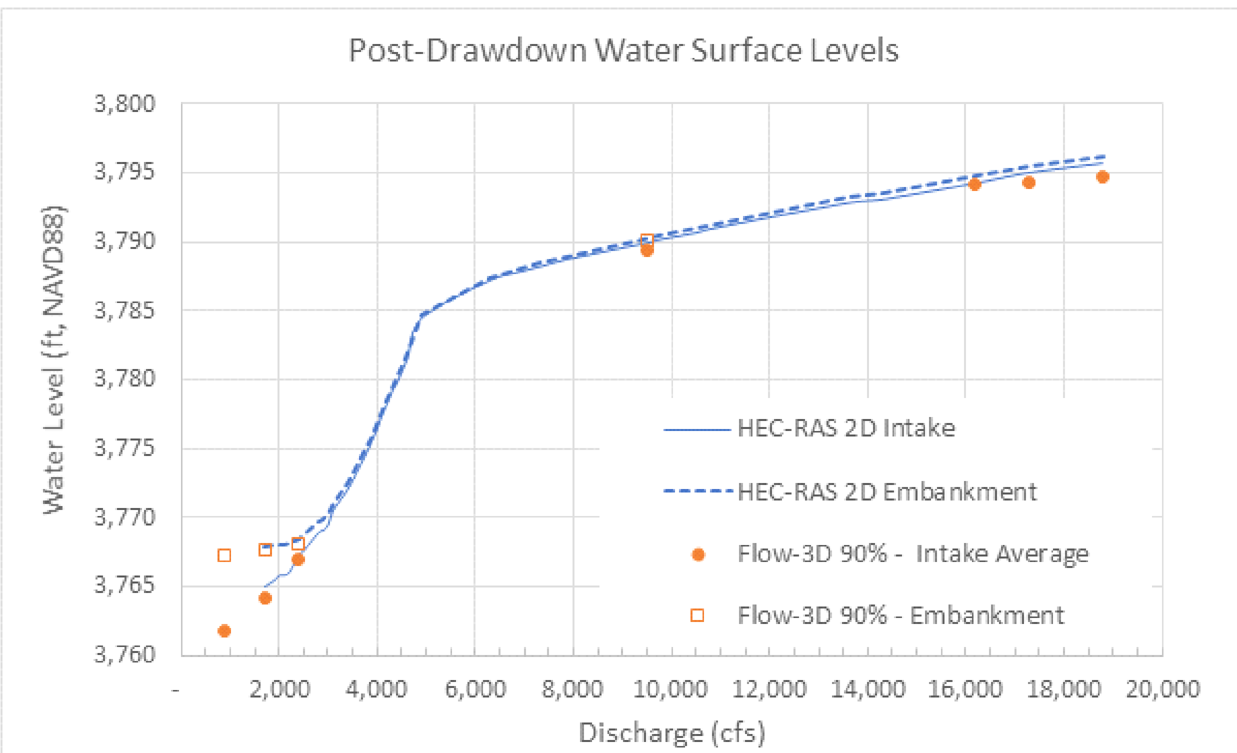


Figure 5. Reservoir Water Surface Elevations at the Intake and Embankment predicted by 2D and 3D models

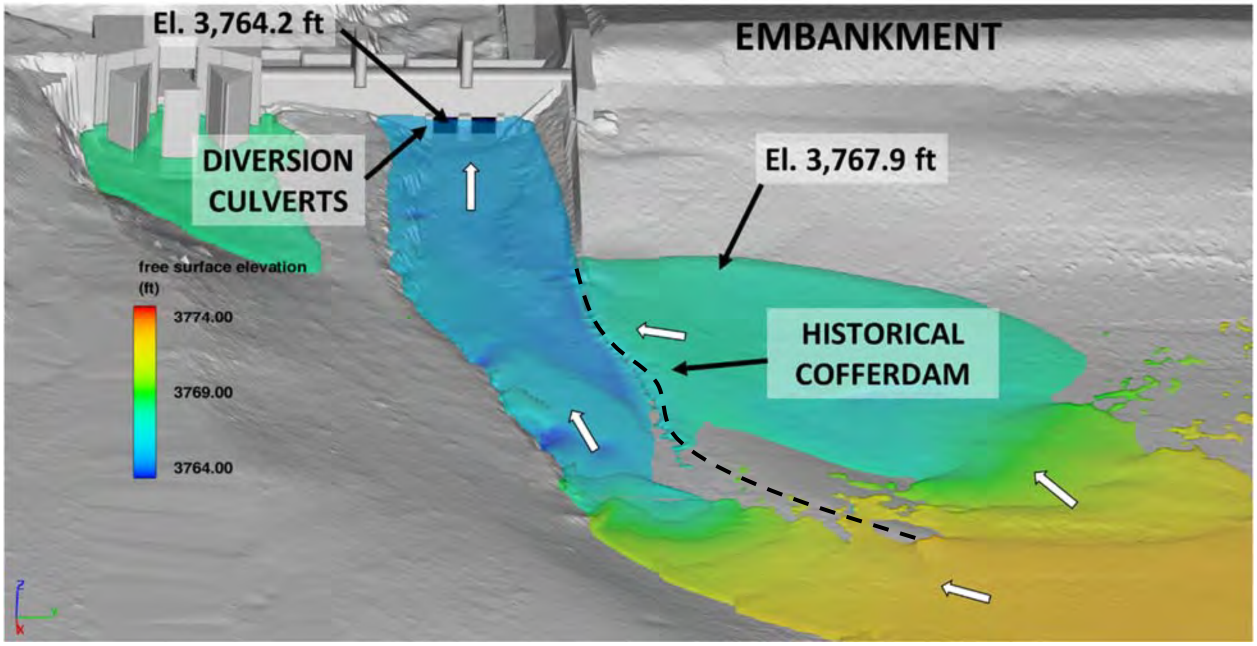


Figure 6. Reservoir Water Surface Elevations (ft) at the Embankment at 1,700 cfs (Using FLOW-3D)

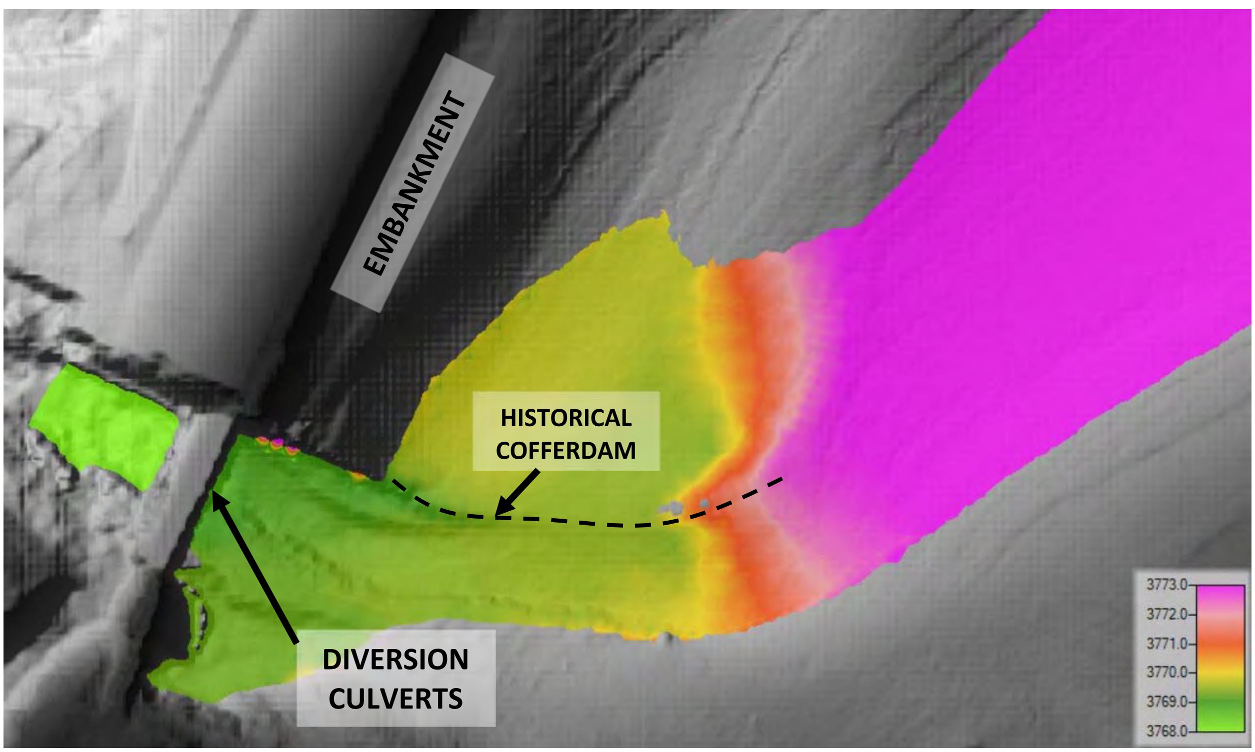


Figure 7. Reservoir Water Surface Elevations (ft) at the Embankment at 2,800 cfs

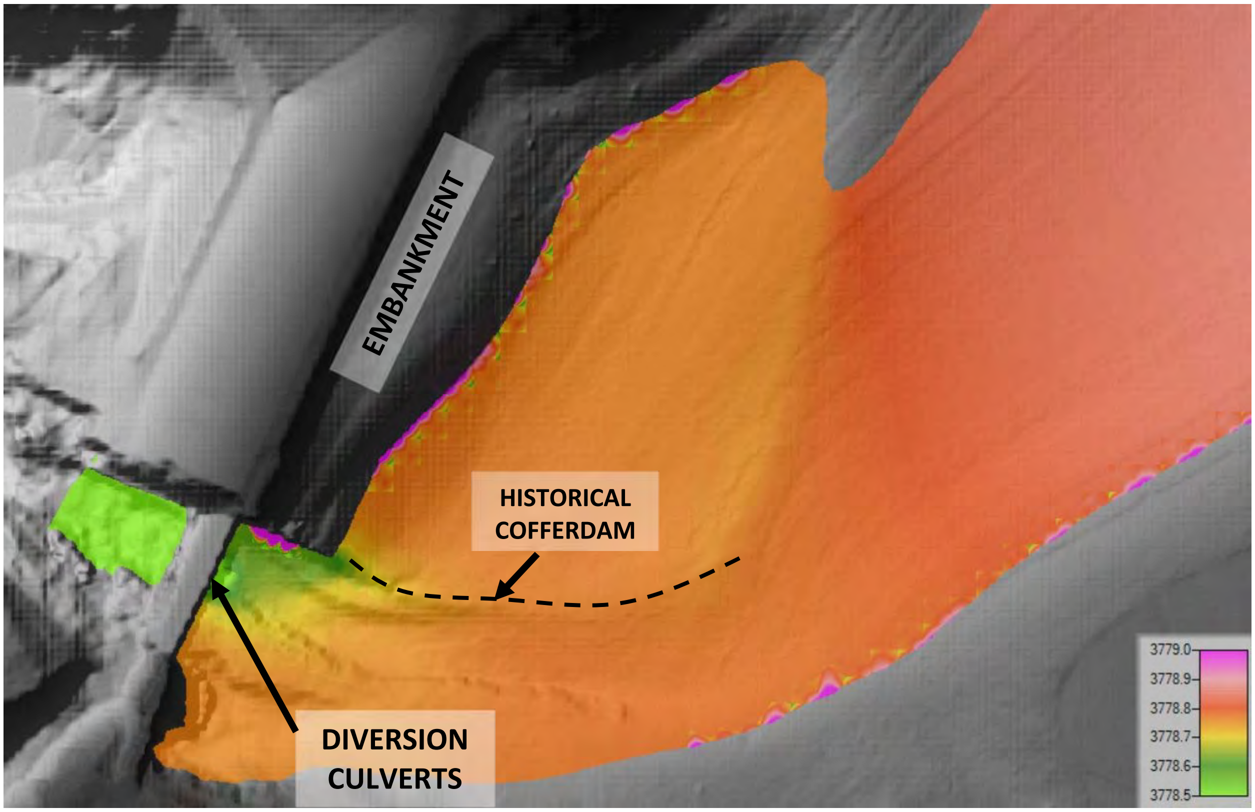


Figure 8. Reservoir Water Surface Elevations (ft) at the Embankment at 4,200 cfs

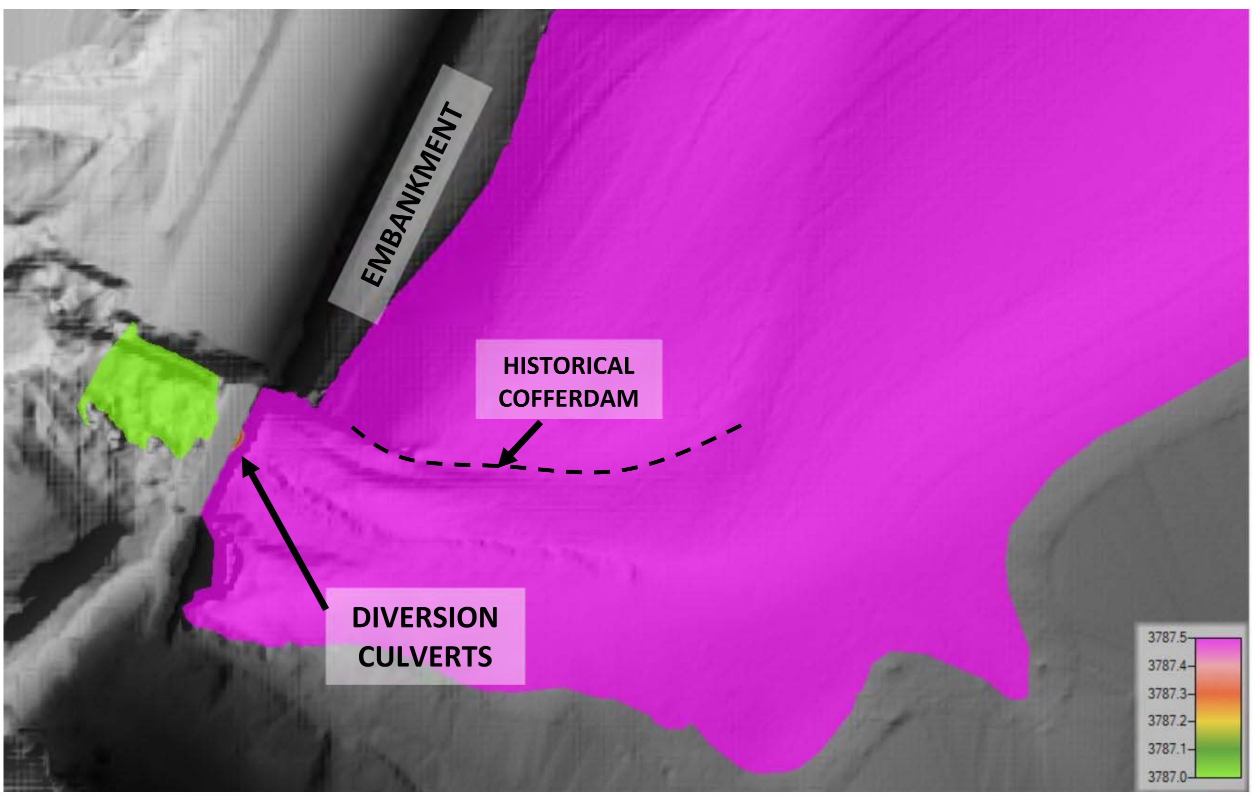


Figure 9. Reservoir Water Surface Elevations (ft) at the Embankment at 6,700 cfs

SEDIMENT MOBILITY

The potential of flow to mobilize sediment at the upstream headpond was assessed by comparing the bed shear stress of the flow predicted by FLOW-3D and HEC-RAS 2D 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.

Table 3. Critical shear stress to initiate sediment motion on flat bed. Adapted from Julien (2002).

Sediment class name	Particle size		Critical shear stress	
	(mm)	(ft)	(Pa)	(lbf/ft^2)
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

UPSTREAM RESERVOIR AND HEADPOND

Figure 10 to 13 illustrate shear stresses near the headpond during Simulations 1, 2, and 3. Table 4 below compares the shear stresses, and largest particle size anticipated to be mobilized, at the headpond during each event. During Simulations 1 and 2, velocities in the approach channel generate shear stresses capable of mobilizing medium to very coarse gravels. Once drawdown is complete and both culverts are opened, a larger flow event during post-drawdown operations could generate local scour at the headpond outlet as shear stress near the headpond is great enough to mobilize large cobbles and small boulders. The duration for which these culverts will operate at these velocities will be determined during drawdown and is dependent on reservoir water levels and inflow rates. Drawdown modeling results may be reviewed in a subsequent design phase to determine possible ranges of time that culverts and low-level outlets will be operated under abrasive conditions.

Table 4. Anticipated Sediment Mobility

Simulation	Reservoir WS (ft NAVD88)	Flow (cfs)	Shear Stress at Headpond (lbf/ft^2)	Mobilized Sediment Class	Mobilize Particle Size (in)
1	3,796.7	17,850	0.5	Very coarse gravel	1.3
2	3,780.6	2,200	0.1	Medium gravel	0.3
3B	3,766.0	2,300	1	Small Cobble	2.5
3A	3,761.3	900	3	Large Cobble	6.5

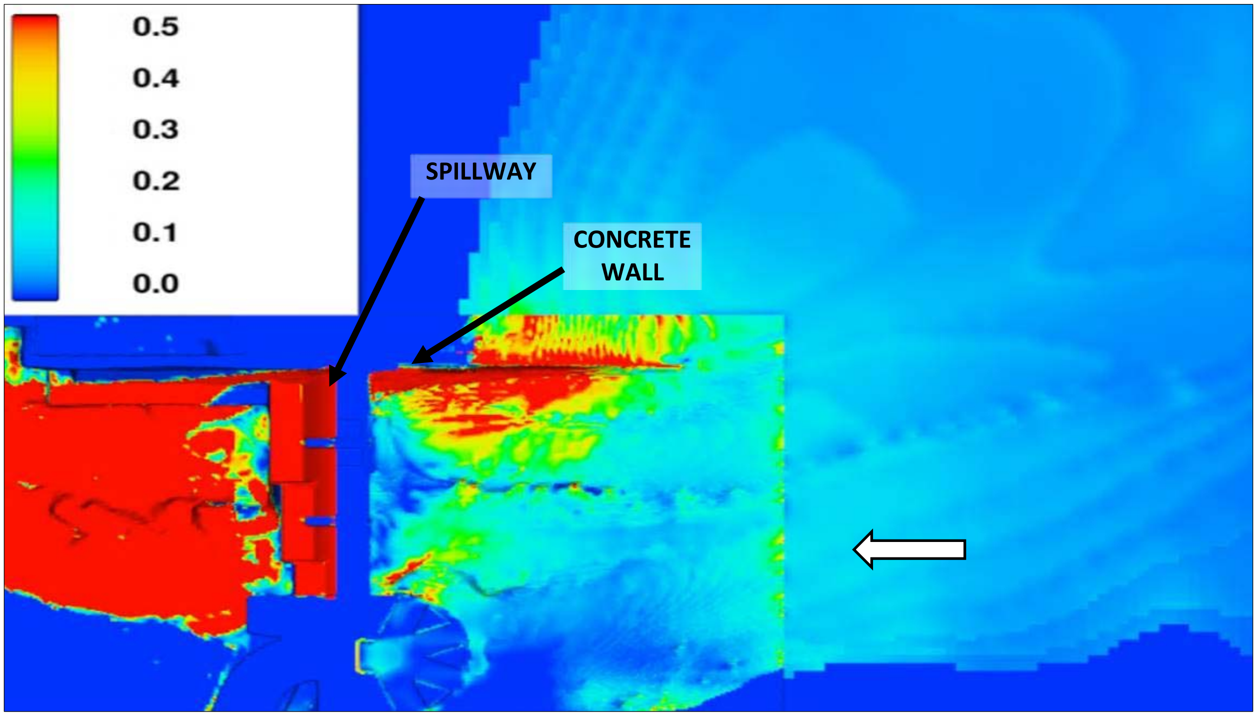


Figure 10. Simulation 1, Shear Stress Magnitude (lbf/ft^2), maximum shear stresses < 0.5 lbf/ft^2

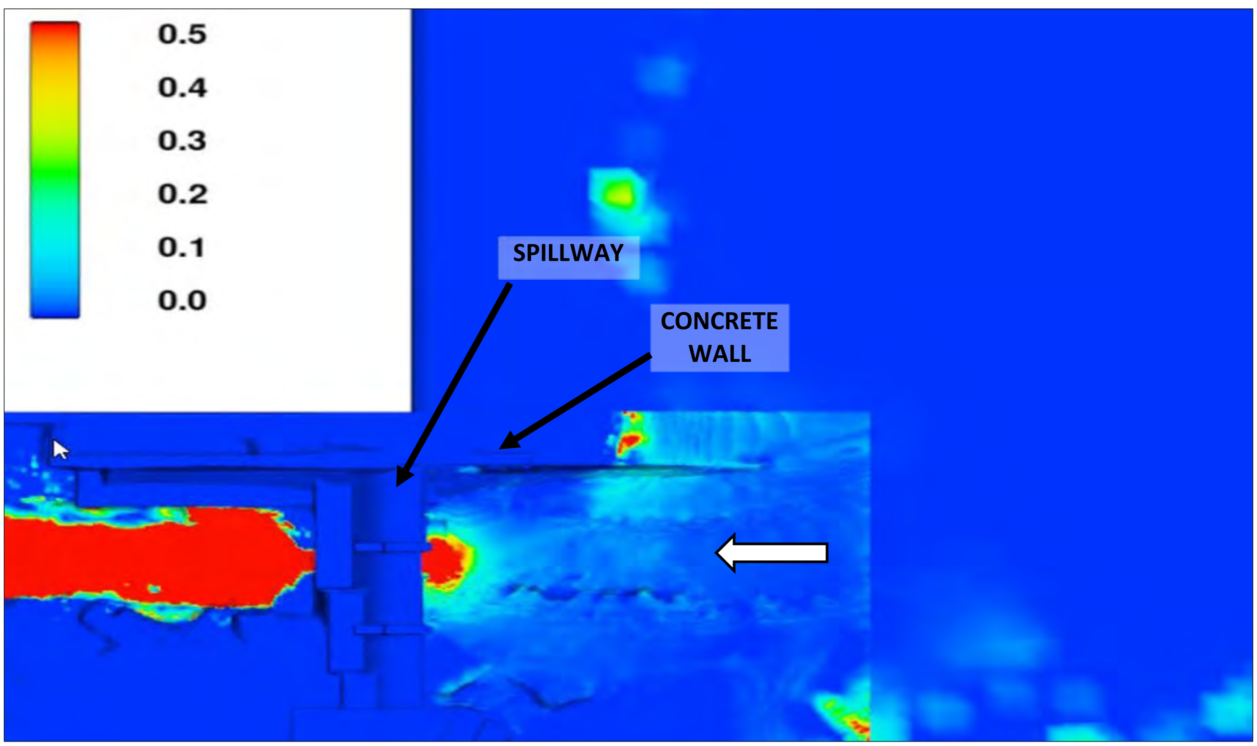


Figure 11. Simulation 2, Shear Stress Magnitude (lbf/ft^2), maximum shear stresses < 0.1 lbf/ft^2

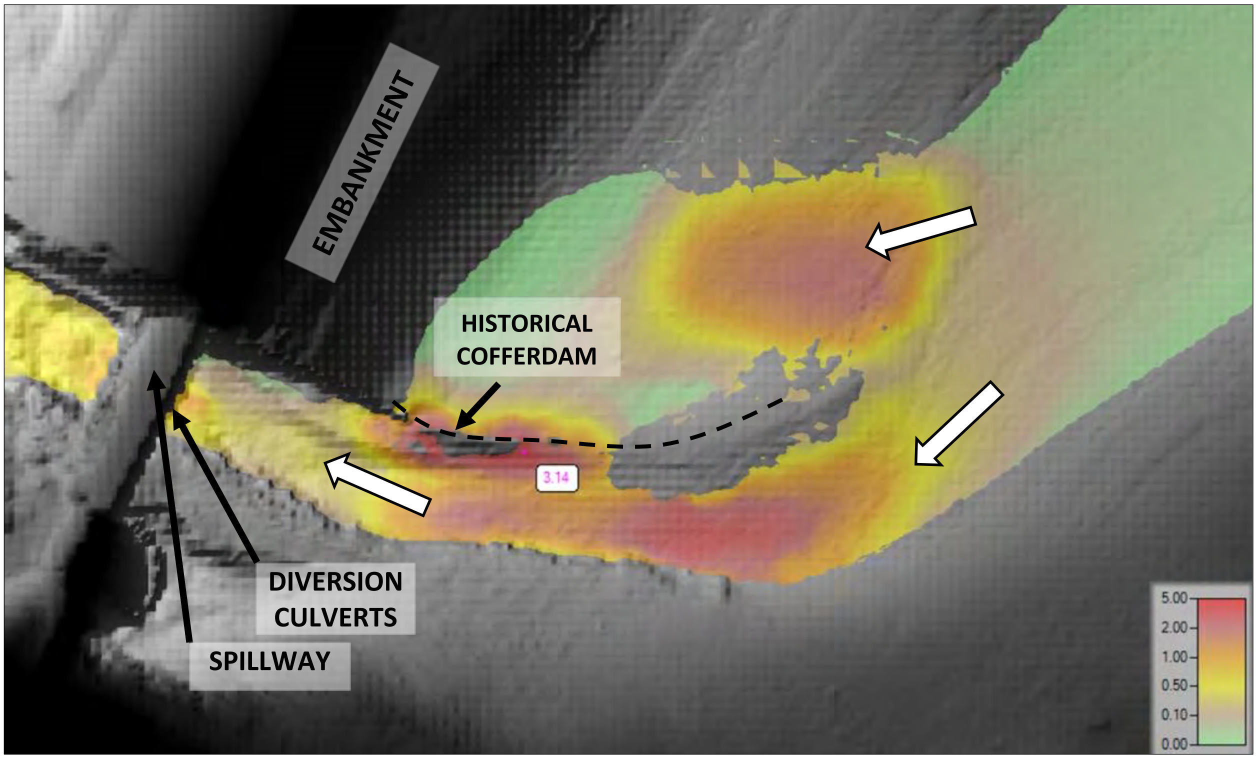


Figure 12. Simulation 3A, Shear Stress Magnitude (lbf/ft^2), Q = 900 cfs, maximum shear stresses ~ 3 lbf/ft^2

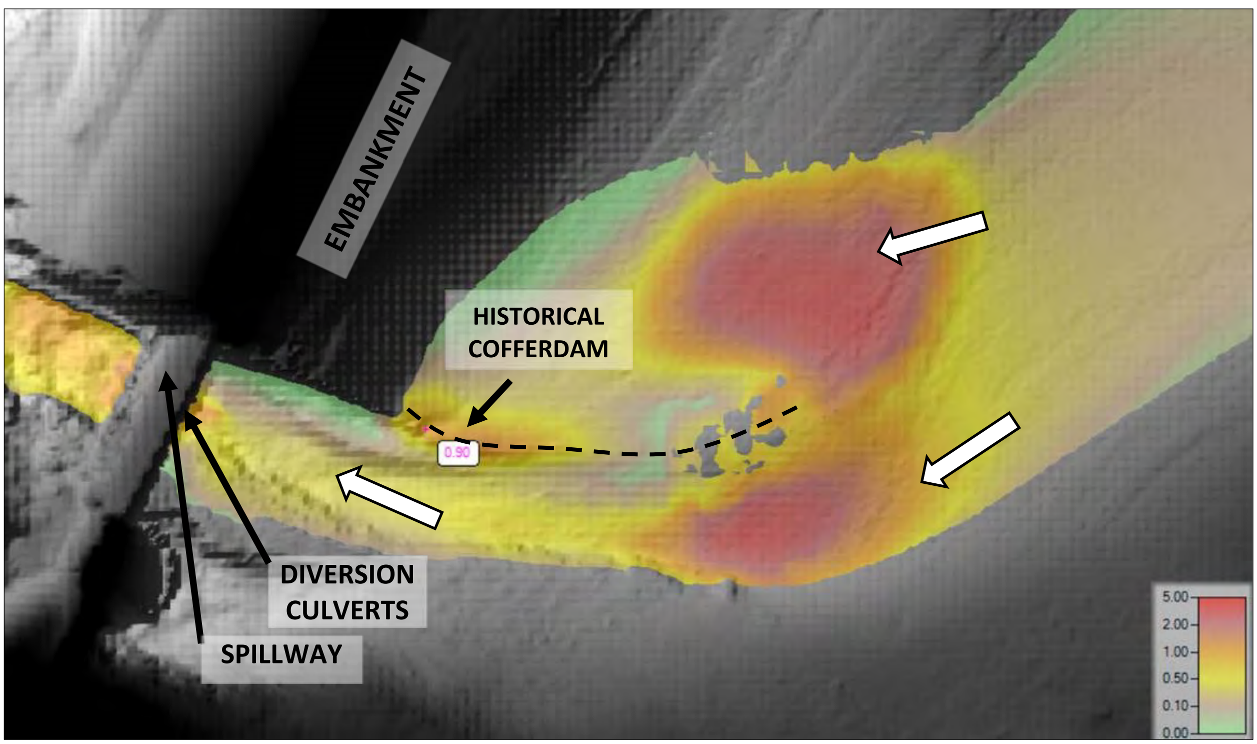


Figure 13. Simulation 3B Shear Stress Magnitude(lbf/ft^2), Q = 2,300 cfs, maximum shear stresses < 1 lbf/ft^2

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Report Prepared by:

Jessica Lammers, P.E.
Hydraulic Engineer

Pepe Vasquez, Ph.D., P.Eng.
Principal

Nancy Sims

Nancy Sims, P.Eng.
Principal

Report Reviewed by:

Erik Rowland, P.E. (State of Oregon)
Principal

REGISTERED PROFESSIONAL ENGINEER
93771PE
OREGON
JULY 10, 2018
ERIK R. ROWLAND
EXPIRES 12-31-22

APPENDIX A

J.C. BOYLE TOTAL DISCHARGE CAPACITY AND DRAWDOWN OPERATIONS PLAN CURVES

RWS (ft, NAVD88)	Total Capacity (per NHC CFD model, 2020)					
	Spillway Only cfs	Power Intake Only cfs	One culvert - no power cfs	One culvert with power cfs	Two culverts cfs	Two culverts + spillway cfs
3801.7	30,402	2,850	3,786	6,636	7,572	37,974
3800.7	27,680	2,850	3,740	6,590	7,480	35,160
3799.7	25,045	2,850	3,694	6,544	7,388	32,433
3798.7	22,500	2,850	3,647	6,497	7,294	29,794
3797.2	20,046	2,850	3,599	6,449	7,198	27,244
3796.7	17,690	2,850	3,550	6,400	7,100	24,790
3795.7	15,433	2,850	3,501	6,351	7,002	22,435
3794.7	13,282	2,850	3,451	6,301	6,902	20,184
3793.7	11,241	2,850	2,915	5,765	5,830	17,071
3792.7	9,265	2,850	2,868	5,718	5,736	15,001
3791.7	7,433	2,850	2,820	5,670	5,640	13,073
3790.7	5,752	2,850	2,772	5,622	5,544	11,296
3789.7	4,233	2,850	2,723	5,573	5,446	9,679
3788.7	2,887	2,805	2,674	5,479	5,348	8,235
3787.7	1,733	2,531	2,623	5,154	5,246	6,979
3786.7	801	2,269	2,572	4,841	5,144	5,945
3785.7	153	2,020	2,520	4,540	5,040	5,193
3785.2	-	1,784	2,494	4,278	4,988	4,988
3784.7	-	1,561	2,467	4,028	4,934	4,934
3783.7	-	1,351	2,414	3,765	4,828	4,828
3782.7	-	1,155	2,359	3,514	4,718	4,718
3781.7	-	973	2,303	3,276	4,606	4,606
3780.7	-	805	2,240	3,045	4,480	4,480
3779.7	-	651	2,187	2,838	4,374	4,374
3778.7	-	512	2,128	2,640	4,256	4,256
3777.7	-	388	2,066	2,454	4,132	4,132
3776.7	-	279	2,003	2,282	4,006	4,006
3775.7	-	187	1,939	2,126	3,878	3,878
3774.7	-	111	1,872	1,983	3,744	3,744
3773.7	-	54	1,803	1,857	3,606	3,606
3772.7	-	15	1,731	1,746	3,462	3,462
3771.7	-	-	1,657	1,657	3,314	3,314
3770.7	-	-	1,578	1,578	3,156	3,156
3769.7	-	-	1,496	1,496	2,992	2,992
3768.7	-	-	1,409	1,409	2,818	2,818
3767.7	-	-	1,316	1,316	2,632	2,632
3766.7	-	-	1,135	1,135	2,270	2,270
3765.7	-	-	1,098	1,098	2,196	2,196
3764.7	-	-	868	868	1,736	1,736
3763.7	-	-	735	735	1,470	1,470
3762.7	-	-	609	609	1,218	1,218
3761.7	-	-	491	491	982	982
3760.7	-	-	382	382	764	764
3760.2	-	-	331	331	662	662
3760.0	-	-	312	312	624	624
3759.7	-	-	283	283	566	566
3758.7	-	-	194	194	388	388
3757.7	-	-	117	117	234	234
3756.7	-	-	54	54	108	108
3755.7	-	-	10	10	20	20
3755.2	-	-	-	-	-	-

Notes: *Power intake rating curve provided by KP, Ref. VA103-640-01

J.C. BOYLE POST-DRAWDOWN RESERVOIR WATER SURFACE ELEVATIONS AT THE EMBANKMENT

Discharge (cfs)	Embankment Reservoir Water Surface Elevation (ft., NAVD88)	Modelling Method
1,700	3767.9	Hec-RAS 2D
1,900	3768.0	
2,000	3768.1	
2,100	3768.1	
2,200	3768.2	
2,300	3768.4	
2,400	3768.7	
2,500	3769.7	
2,800	3769.8	
2,900	3770.2	
3,000	3771.1	
3,100	3772.5	
3,400	3776.0	
3,900	3777.8	
4,100	3778.5	
4,200	3779.4	
4,300	3780.1	
4,400	3781.7	
4,600	3784.6	
4,900	3787.4	
6,300	3788.0	
6,800	3788.4	
7,300	3789.1	
8,000	3790.1	
9,400	3790.4	
9,700	3790.6	
9,900	3791.0	
10,500	3791.4	
10,900	3793.2	
13,600	3793.5	
14,300	3793.7	
16,200	3794.1	Flow 3D
17,300	3794.4	
18,800	3794.7	

APPENDIX C1
COPCO NO. 1 HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

TABLE OF CONTENTS

1.0 INTRODUCTION

This appendix includes a summary of data, design methodology, and other information used in the civil, hydrotechnical, 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.

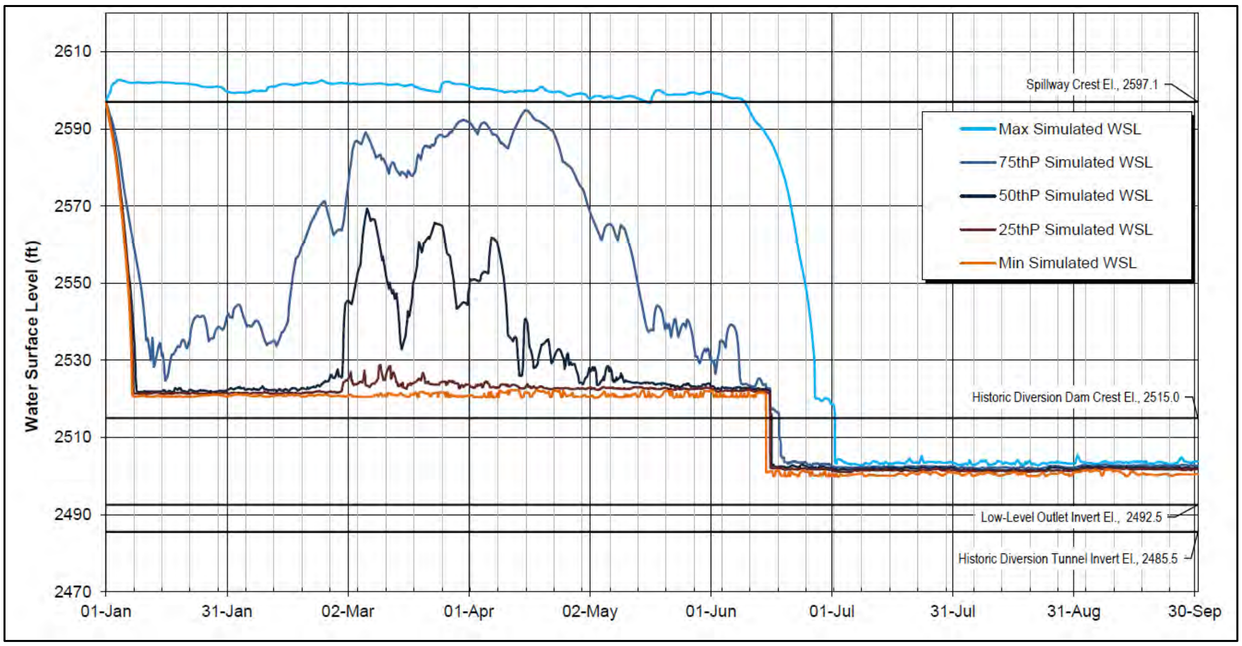

Figure 3.1 Copco No. 1 Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

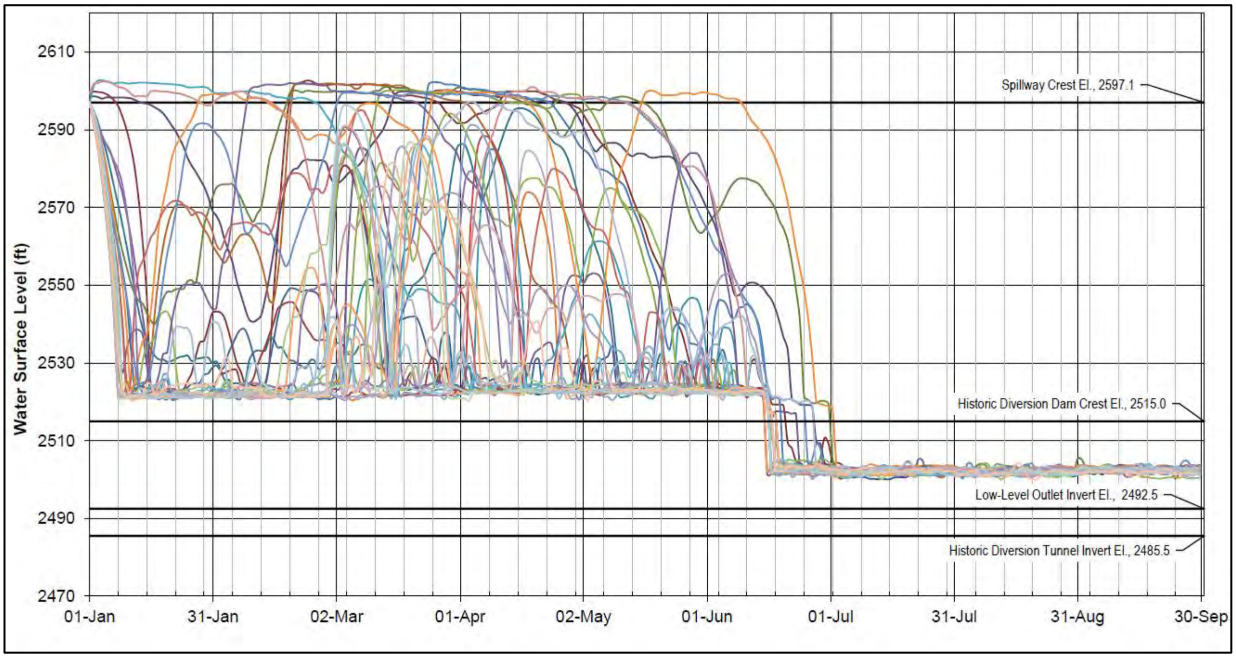

Figure 3.2 Copco No. 1 Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

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.

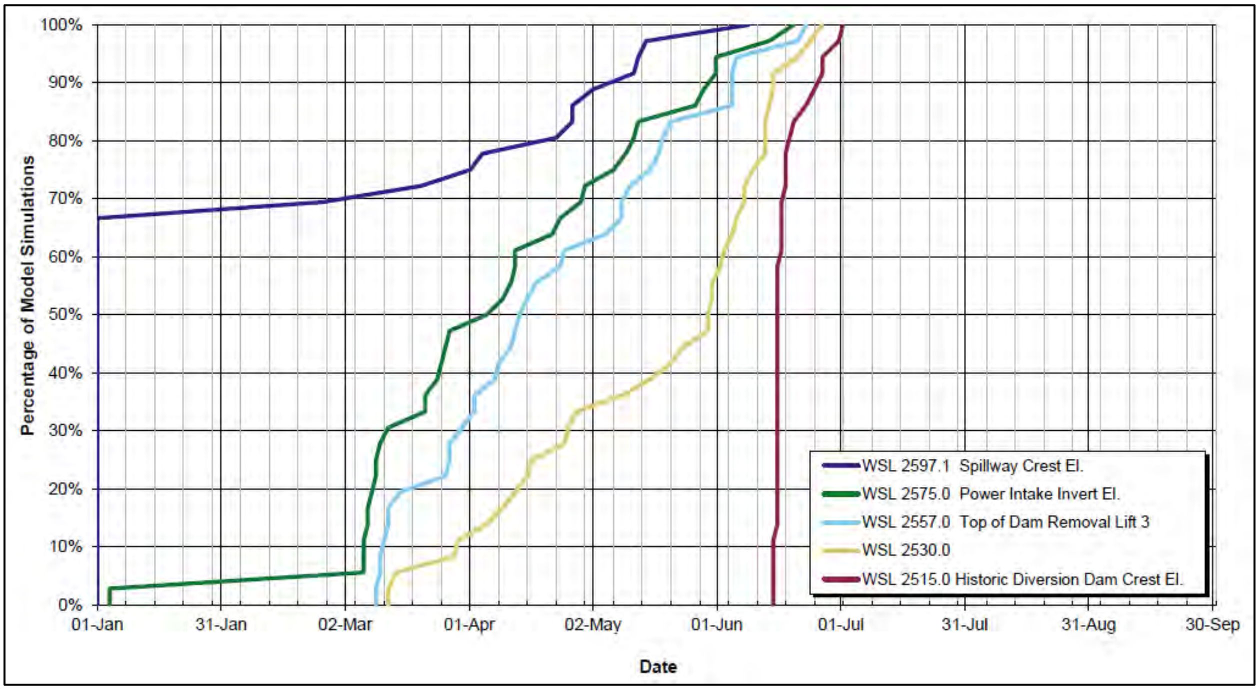

Figure 3.3 Copco No. 1 Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below Various Relevant Elevations

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.

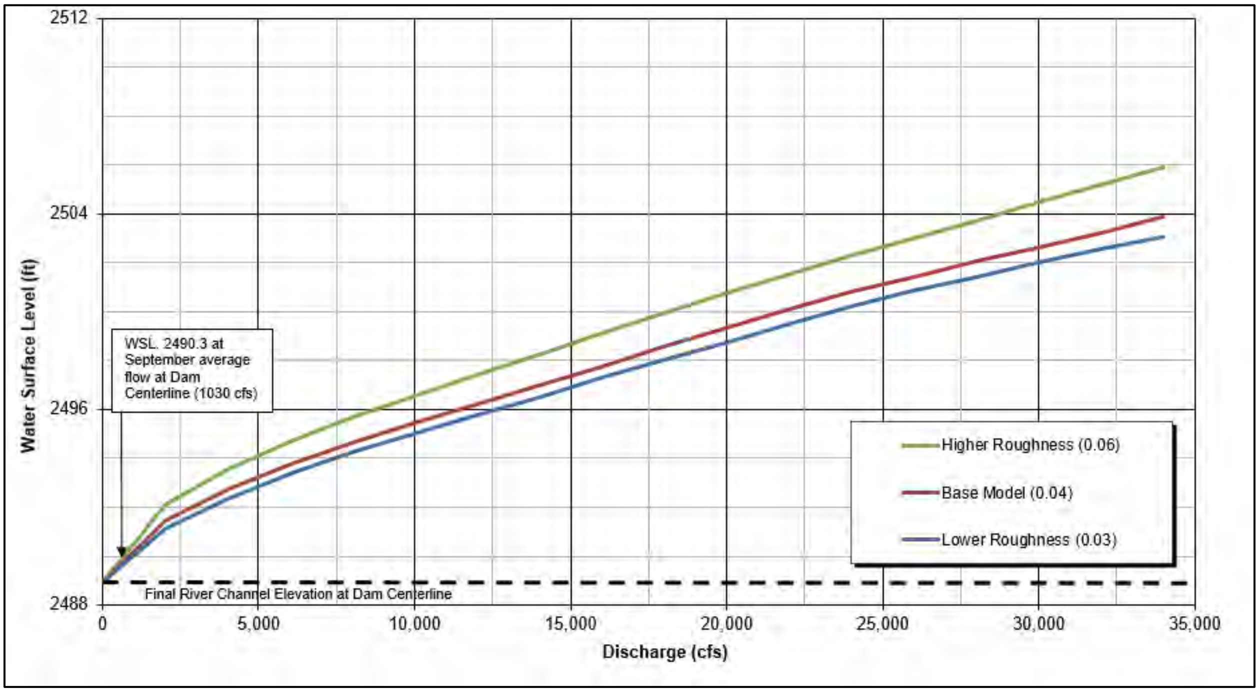

Figure 3.4 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

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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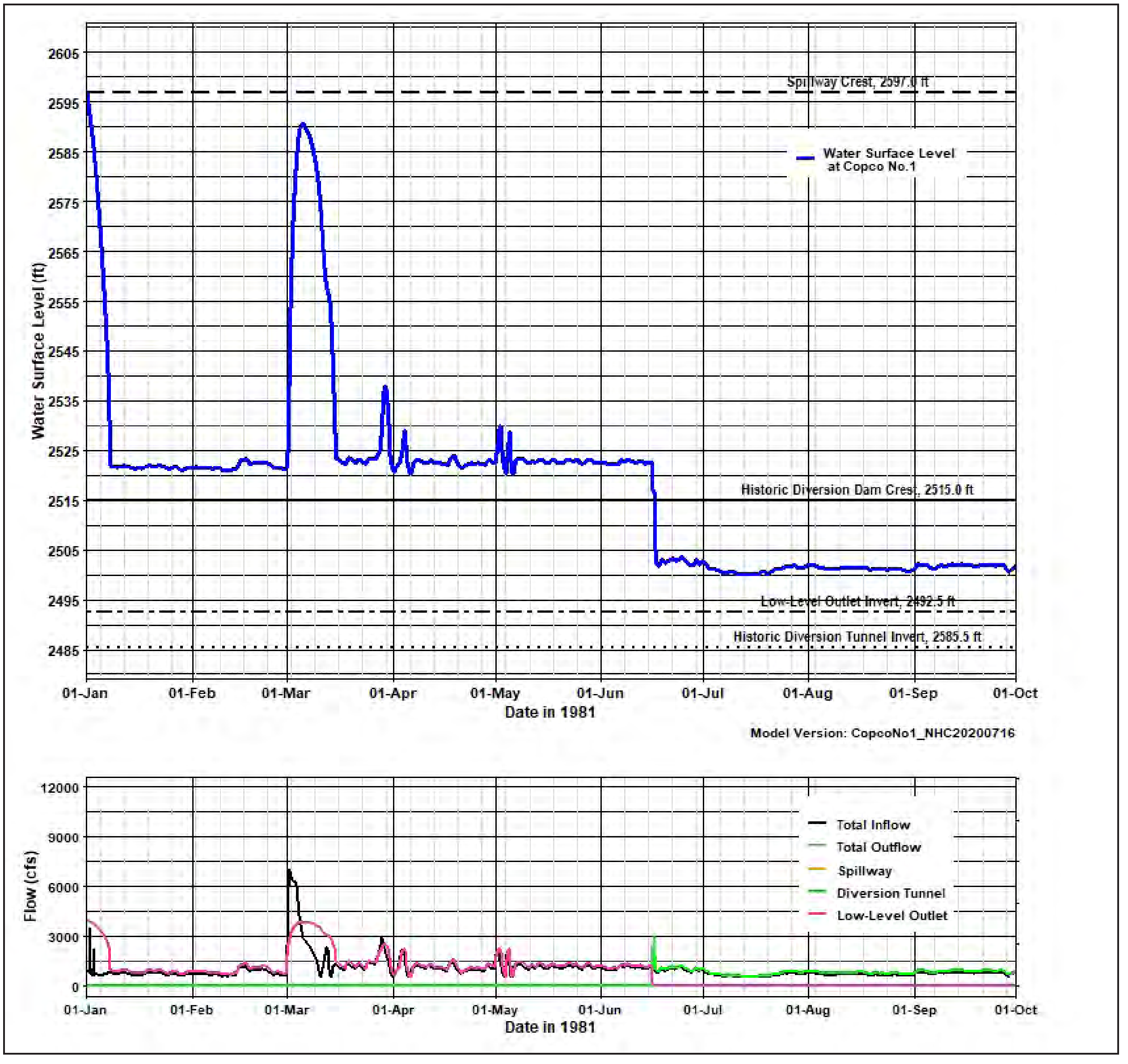

Figure 1 - COPCO No.1 Facility Simulated Drawdown - Year 1981

July 29, 2020

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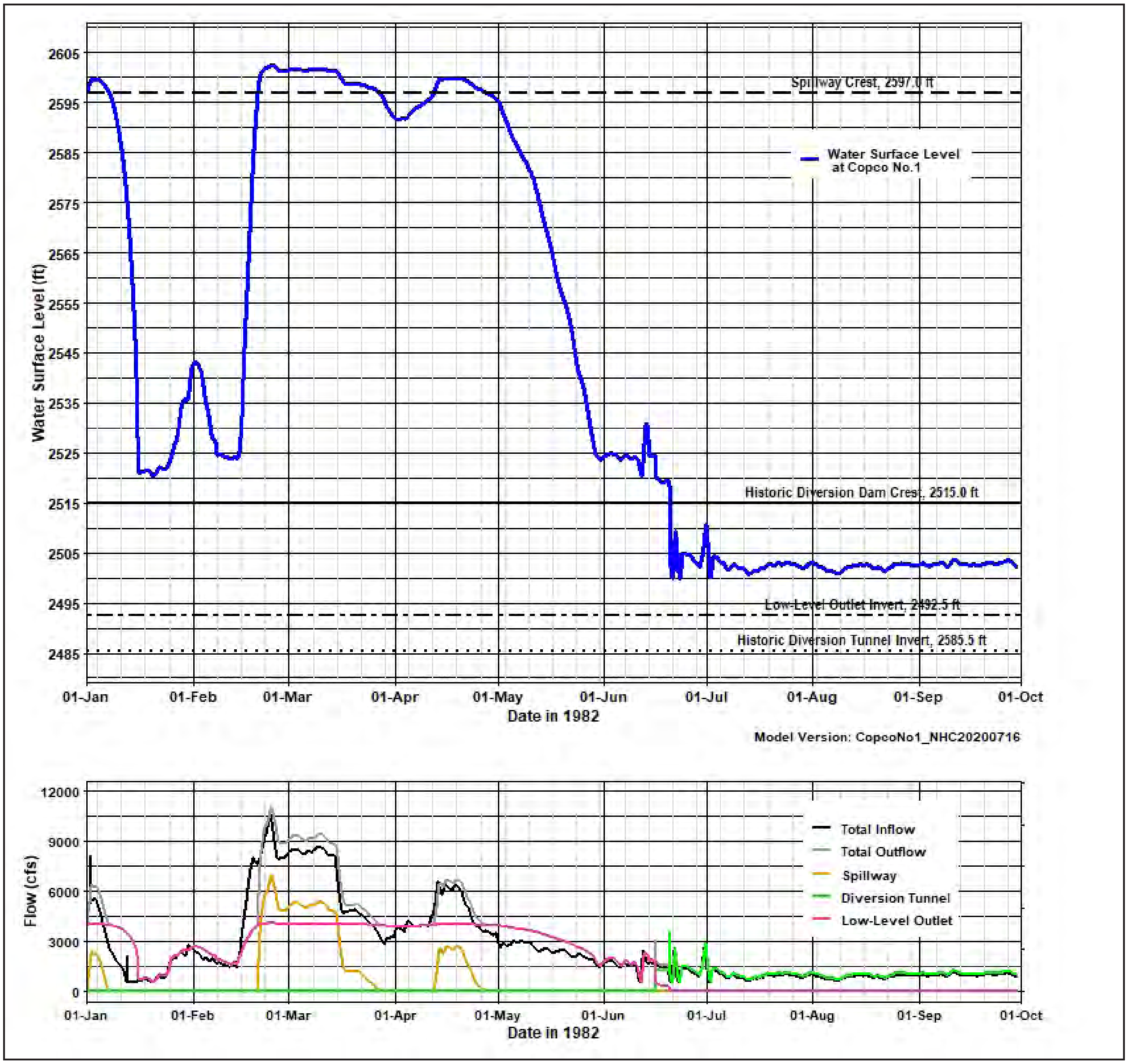

Figure 2 - COPCO No.1 Facility Simulated Drawdown - Year 1982

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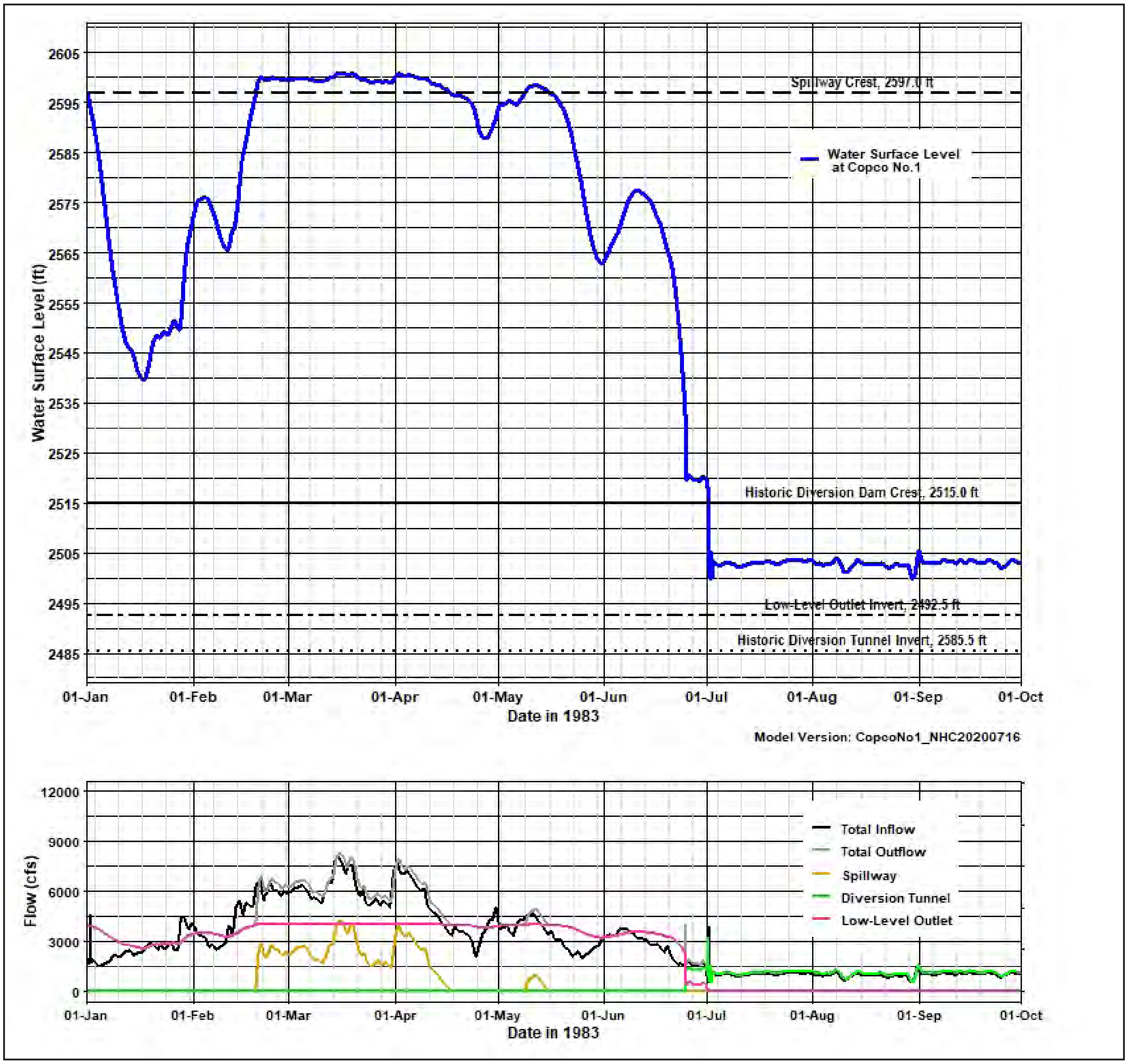

Figure 3 - COPCO No.1 Facility Simulated Drawdown - Year 1983

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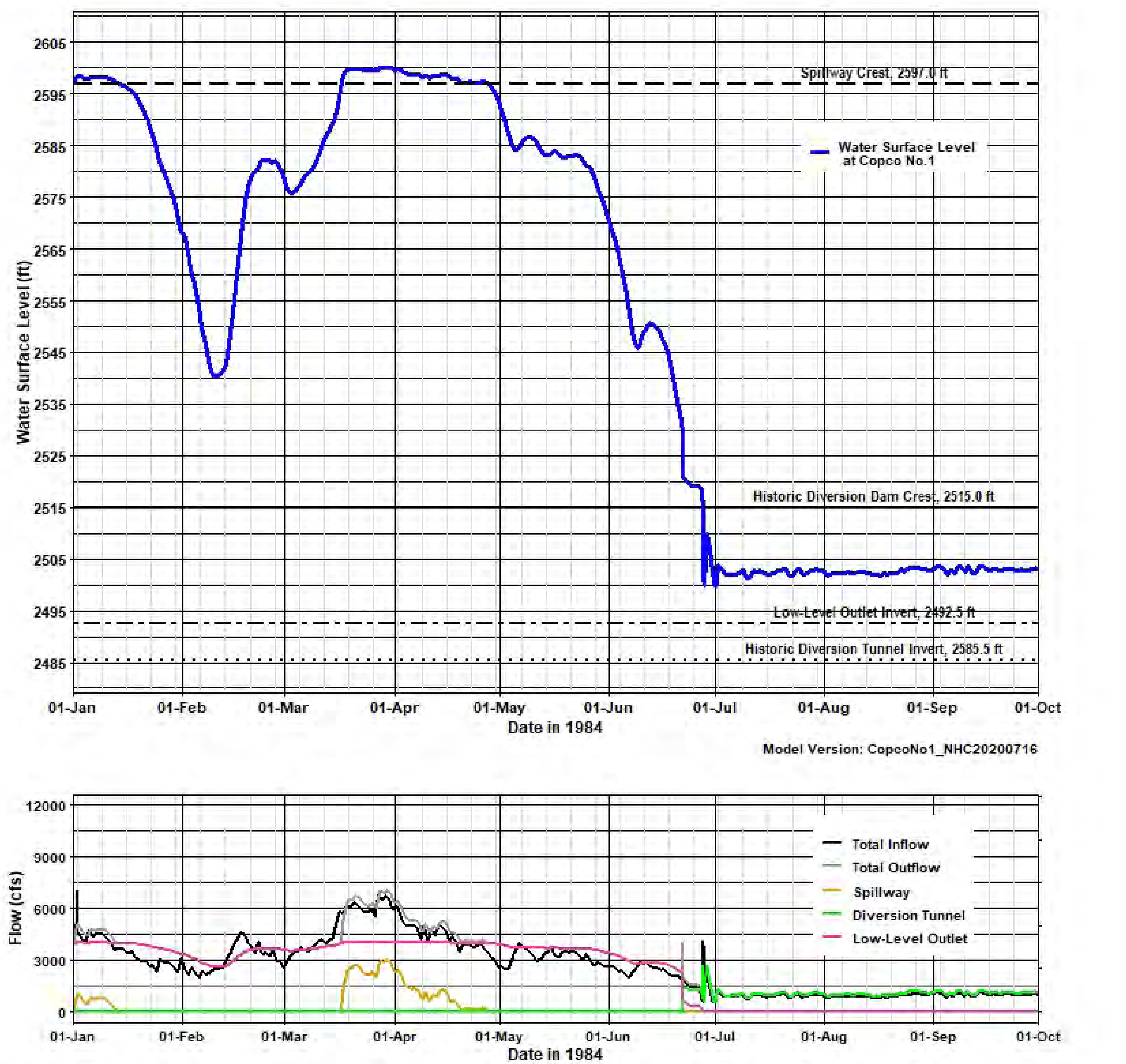

Figure 4 - COPCO No.1 Facility Simulated Drawdown - Year 1984

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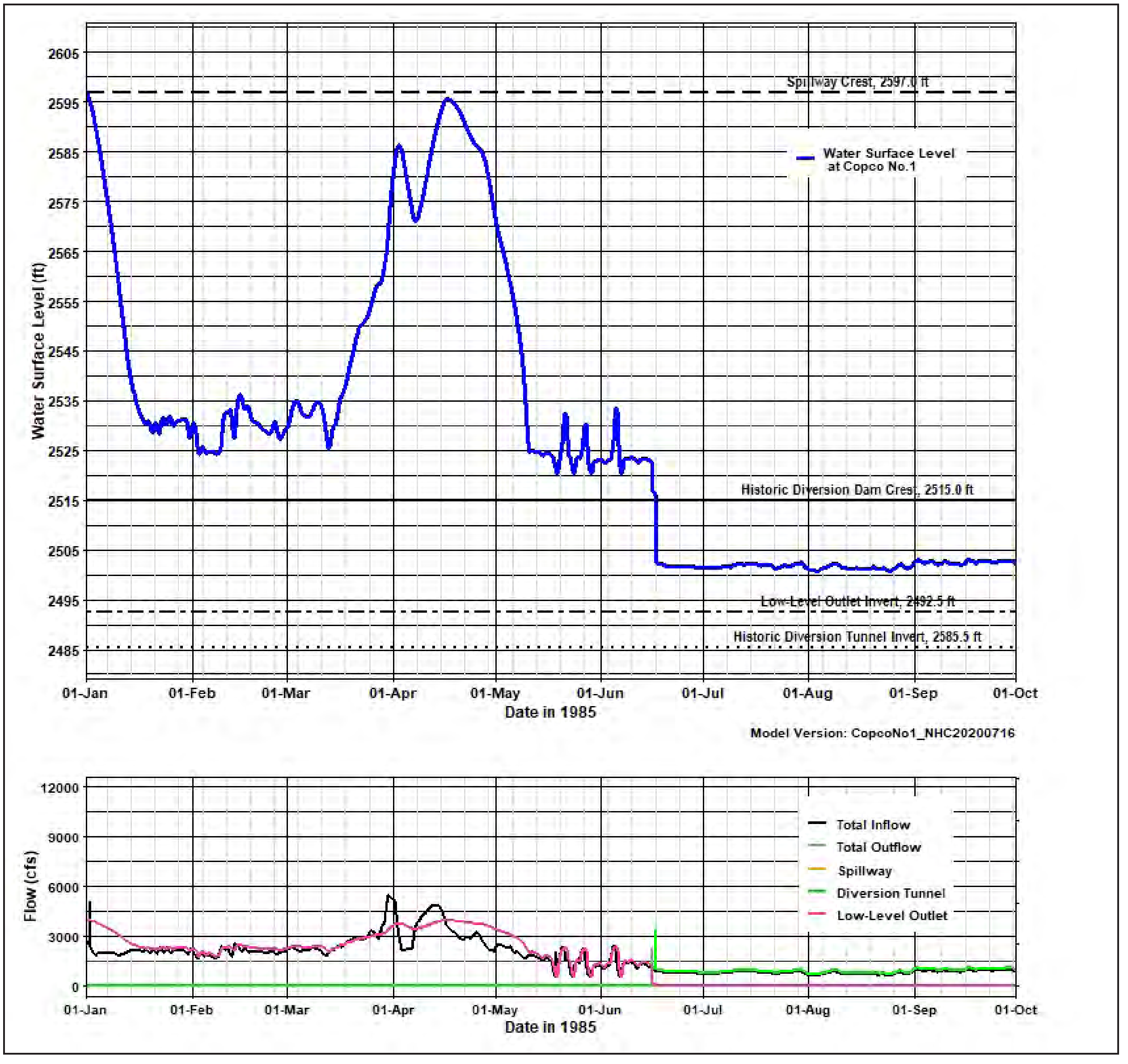

Figure 5 - COPCO No.1 Facility Simulated Drawdown - Year 1985

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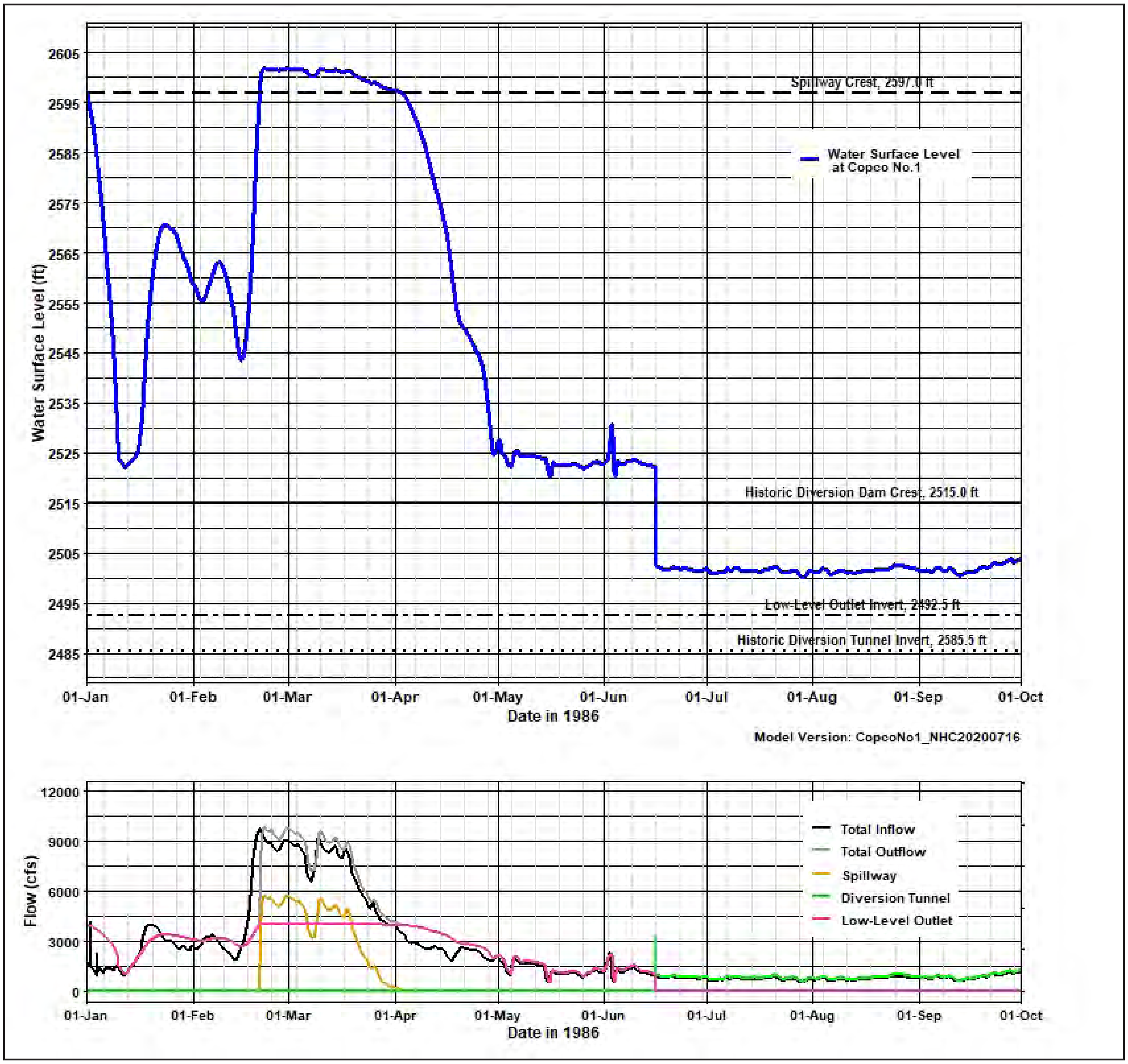

Figure 6 - COPCO No.1 Facility Simulated Drawdown - Year 1986

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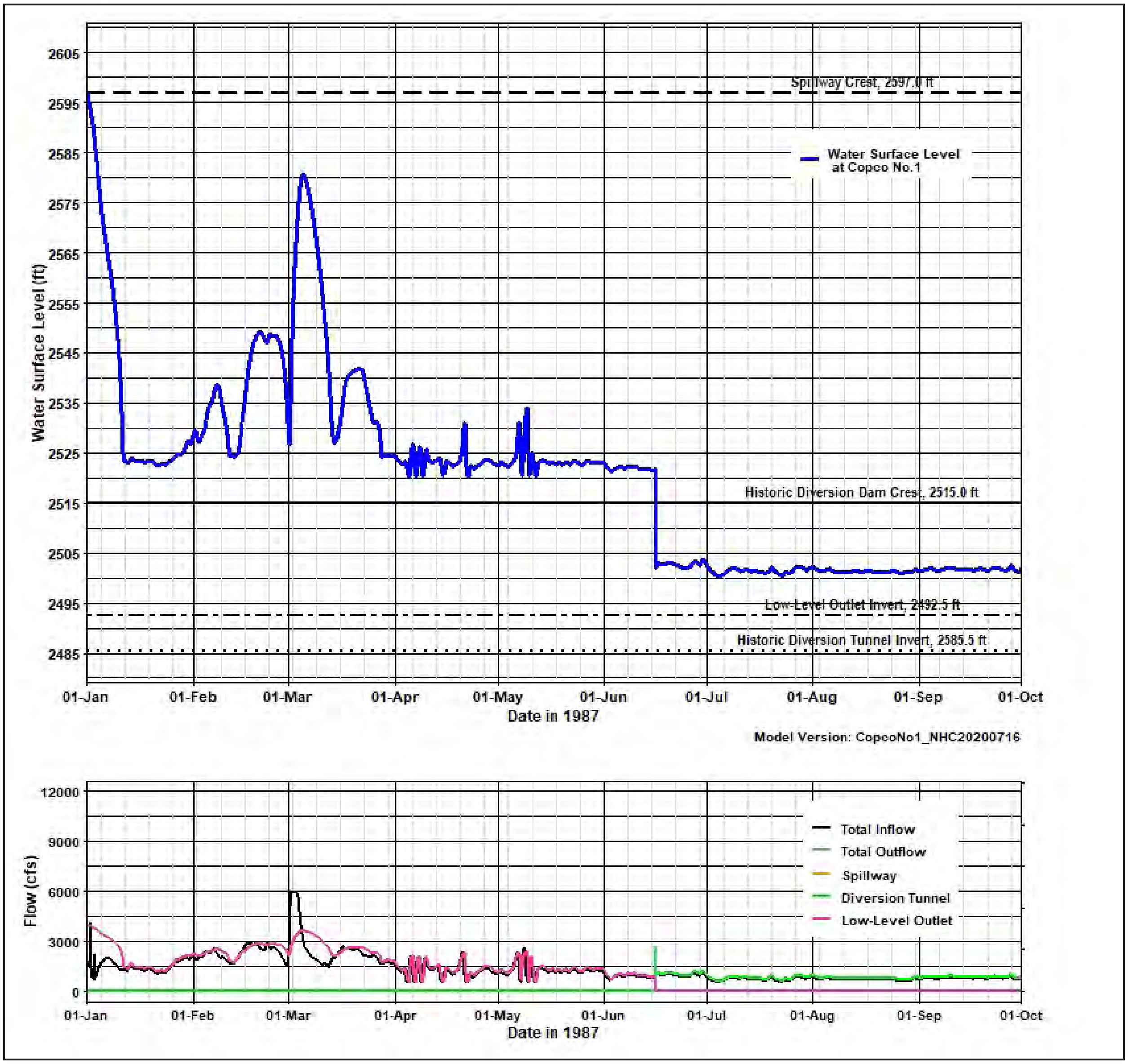

Figure 7 - COPCO No.1 Facility Simulated Drawdown - Year 1987

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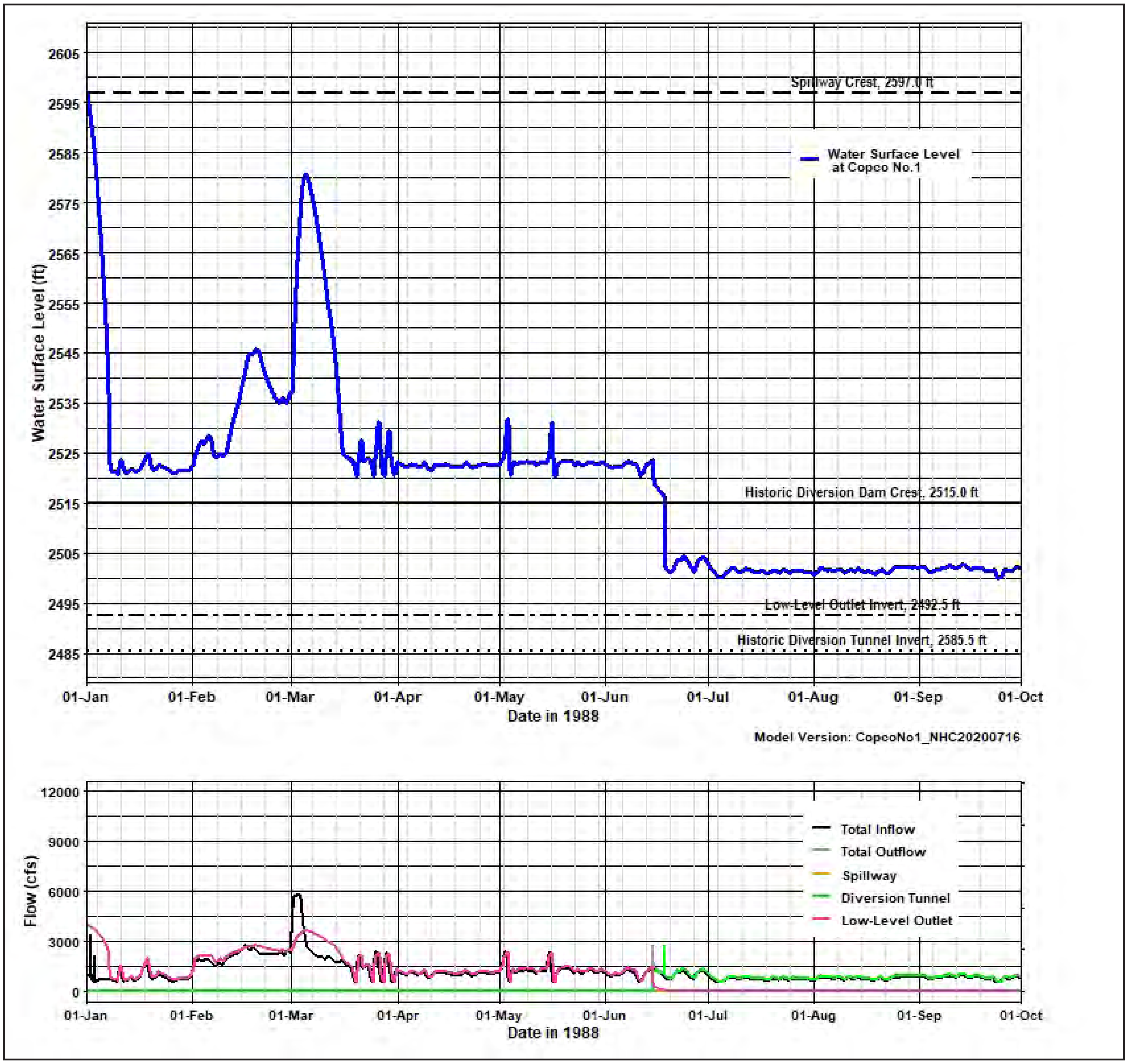

Figure 8 - COPCO No.1 Facility Simulated Drawdown - Year 1988

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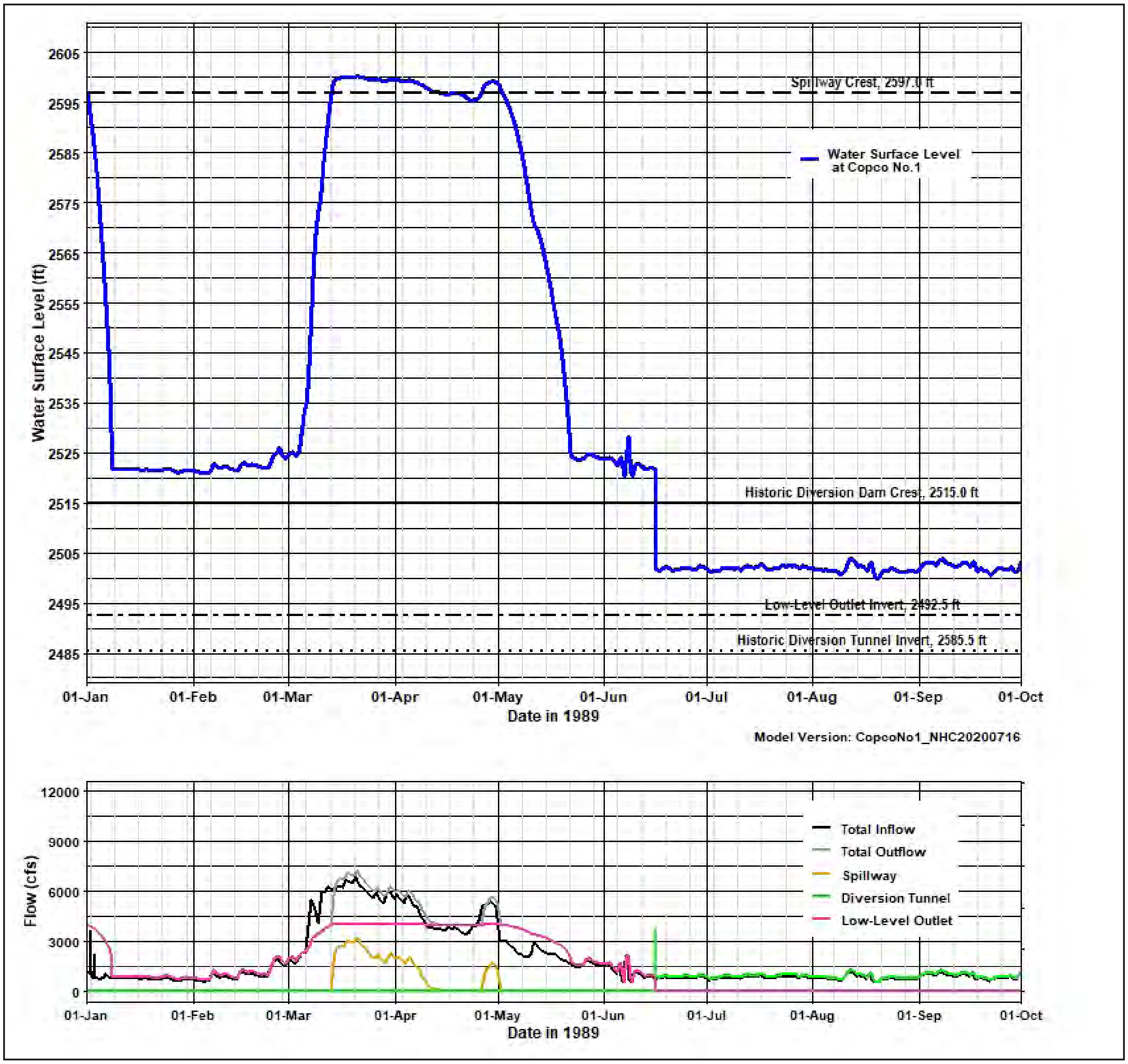

Figure 9 - COPCO No.1 Facility Simulated Drawdown - Year 1989

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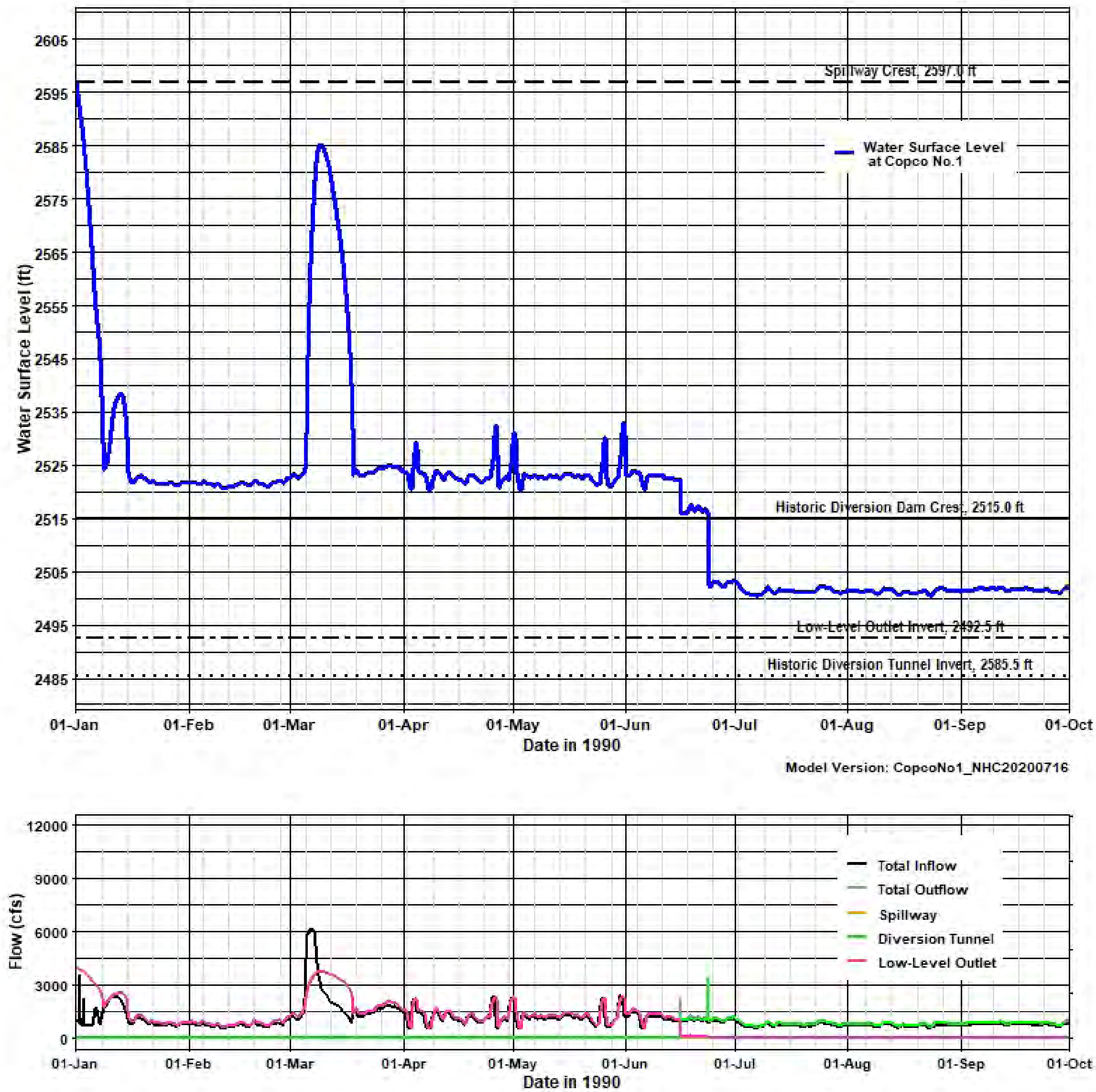

Figure 10 - COPCO No.1 Facility Simulated Drawdown - Year 1990

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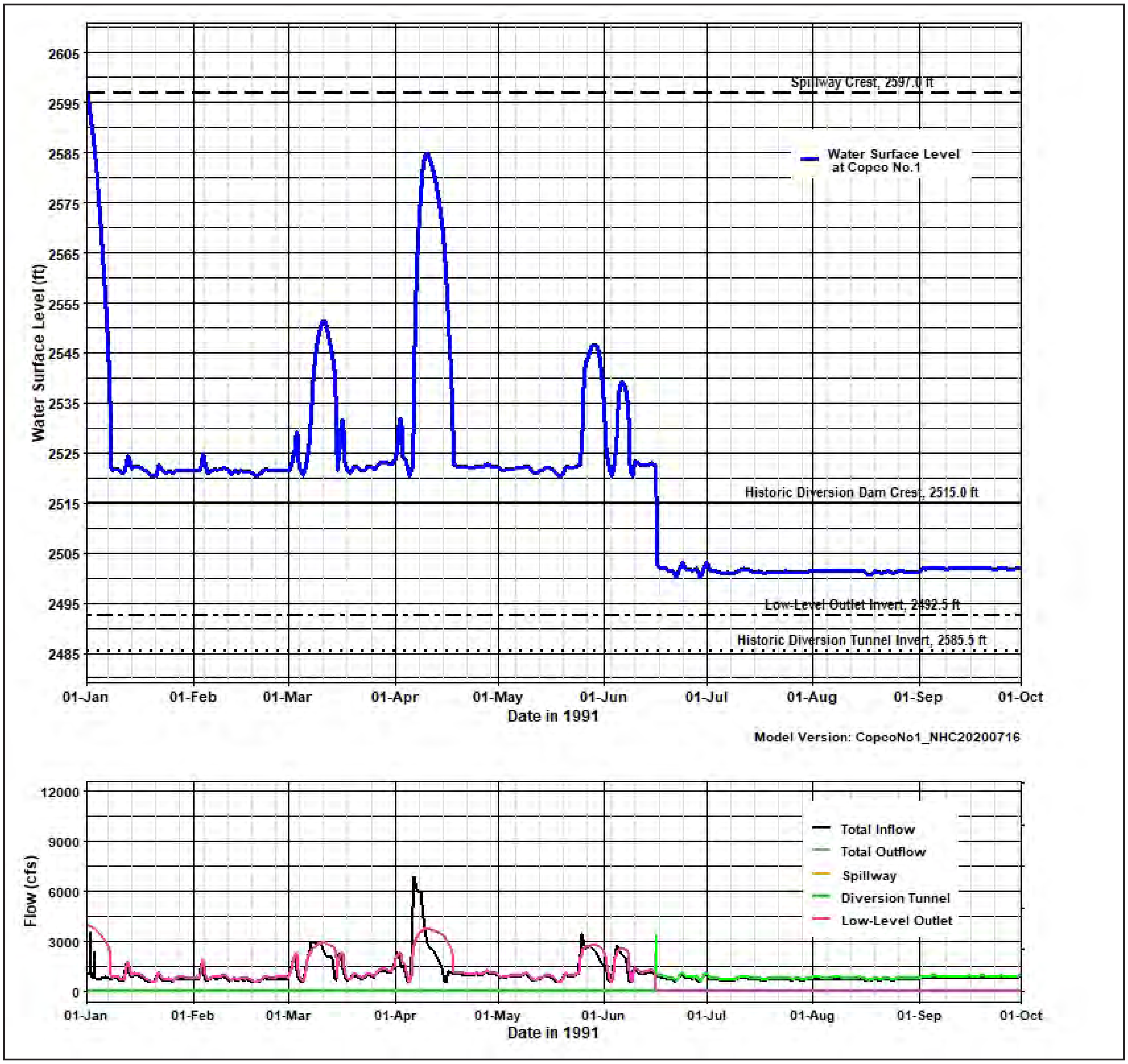

Figure 11 - COPCO No.1 Facility Simulated Drawdown - Year 1991

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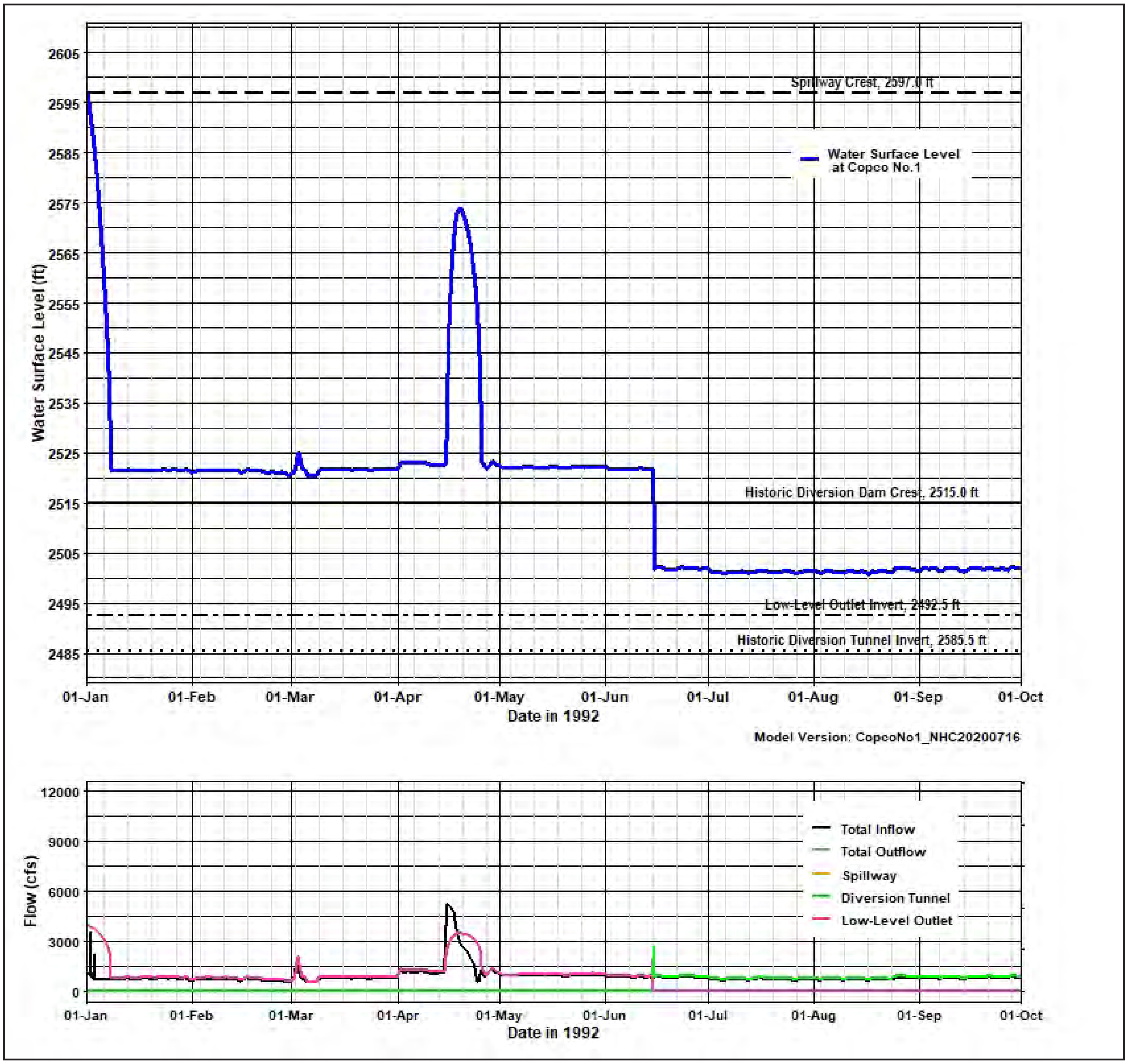

Figure 12 - COPCO No.1 Facility Simulated Drawdown - Year 1992

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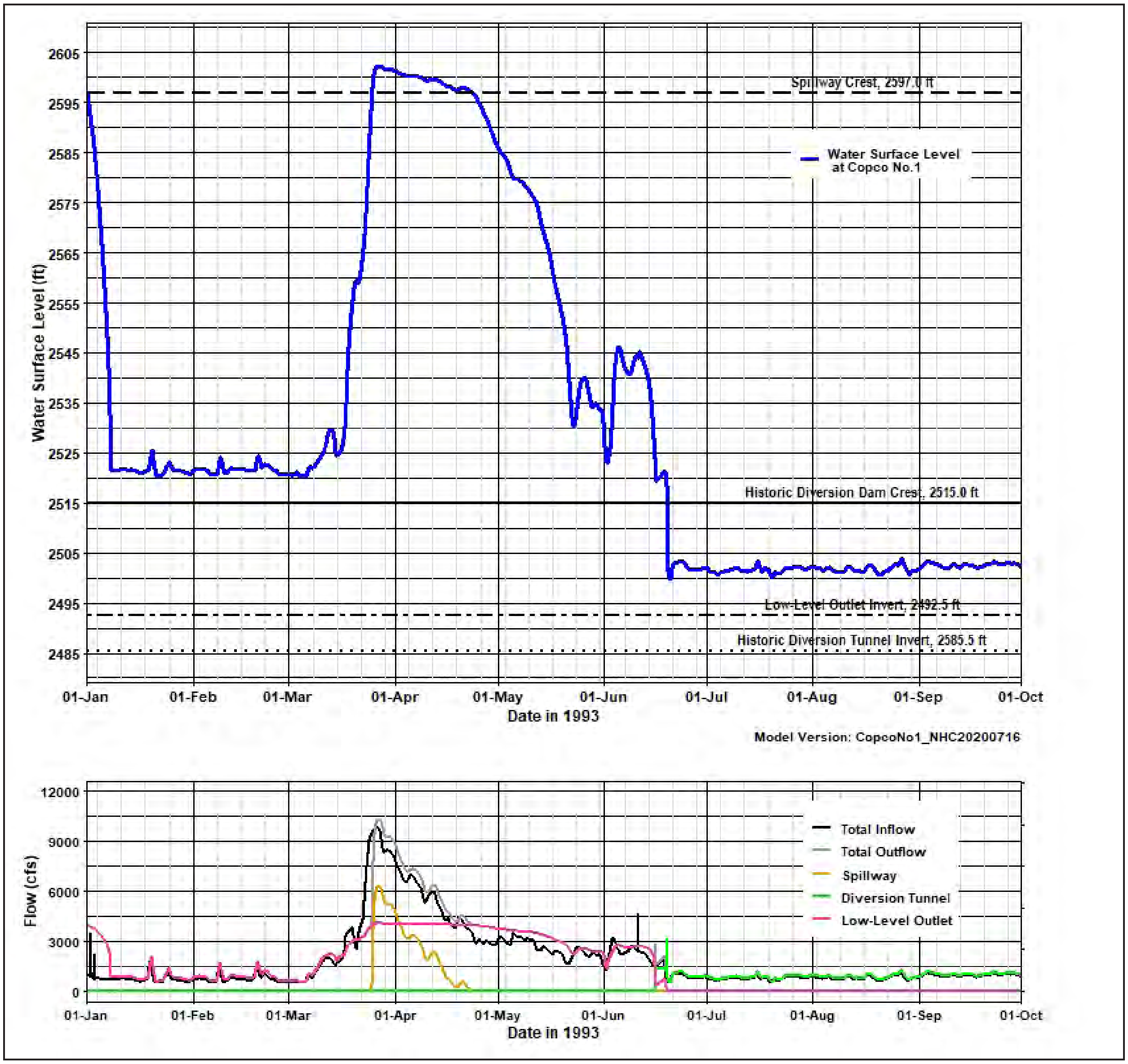

Figure 13 - COPCO No.1 Facility Simulated Drawdown - Year 1993

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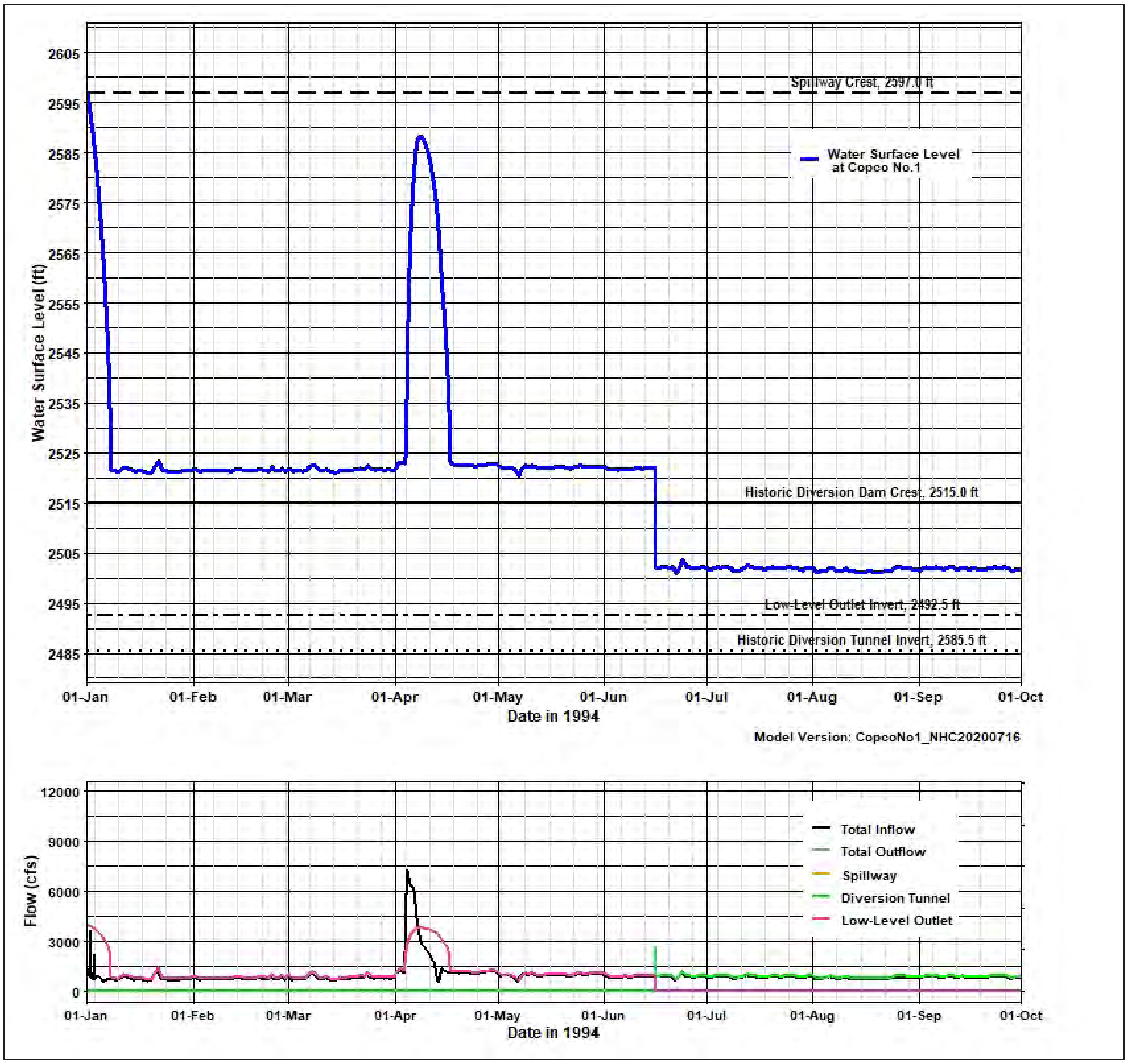

Figure 14 - COPCO No.1 Facility Simulated Drawdown - Year 1994

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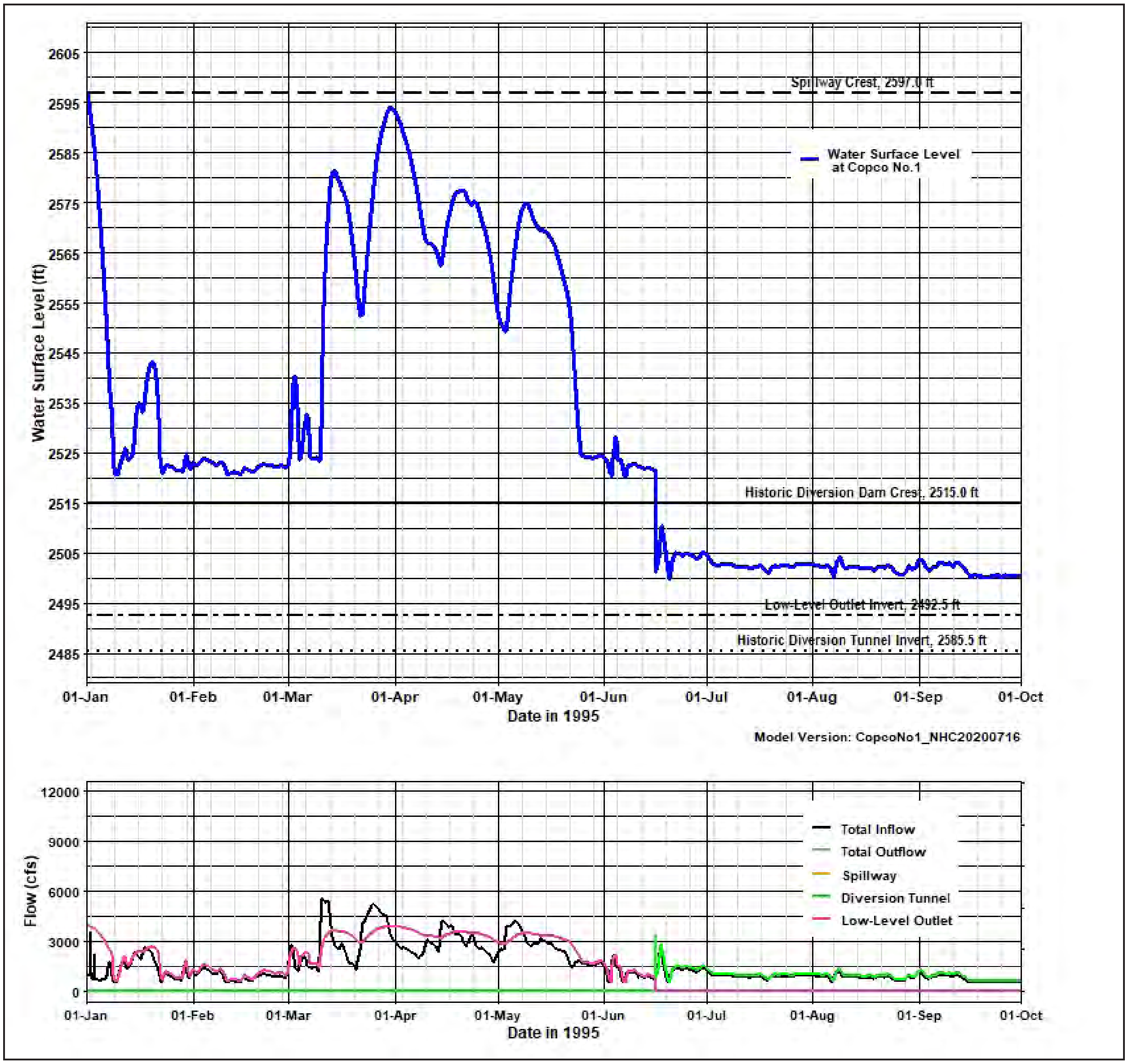

Figure 15 - COPCO No.1 Facility Simulated Drawdown - Year 1995

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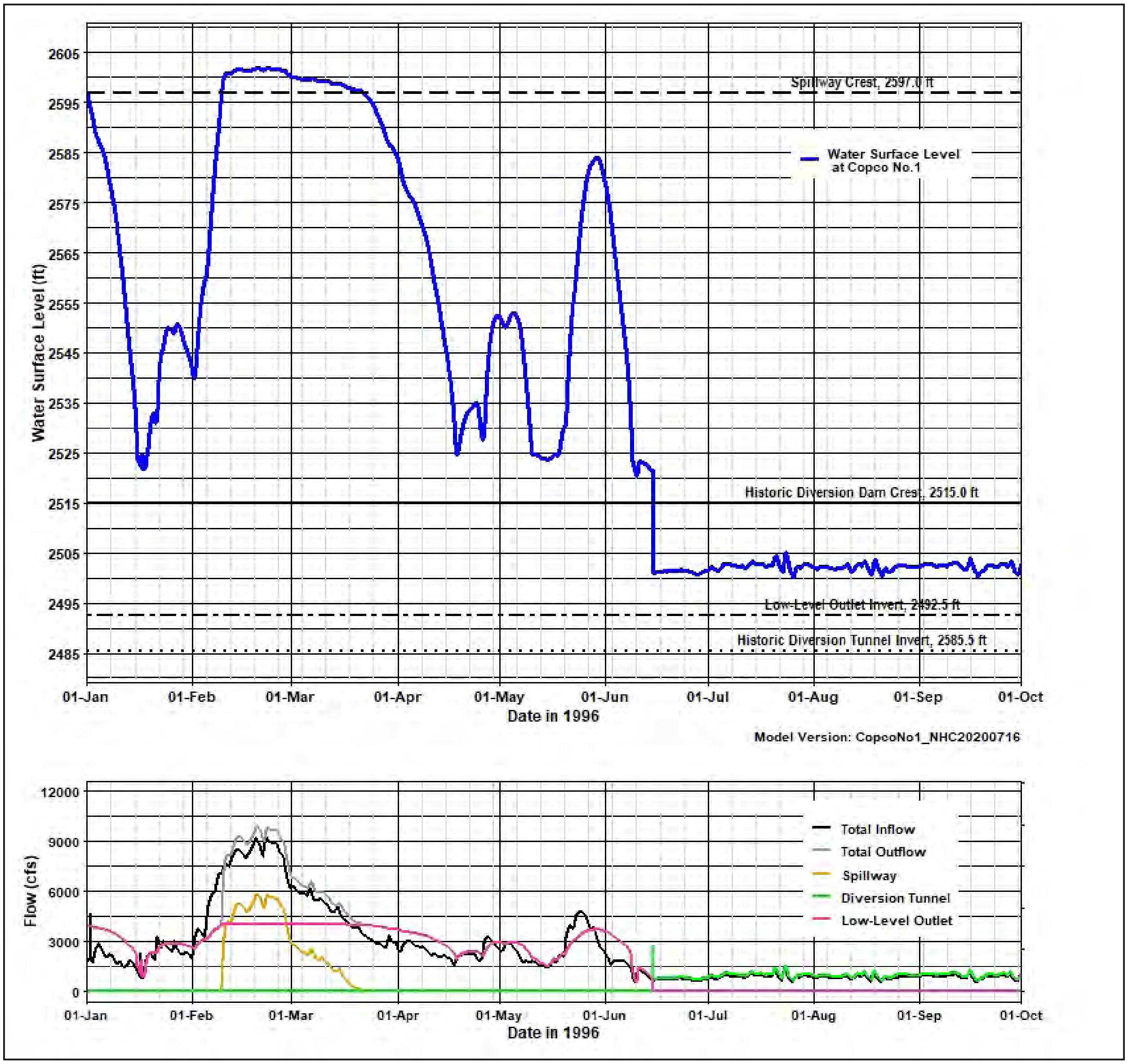

Figure 16 - COPCO No.1 Facility Simulated Drawdown - Year 1996

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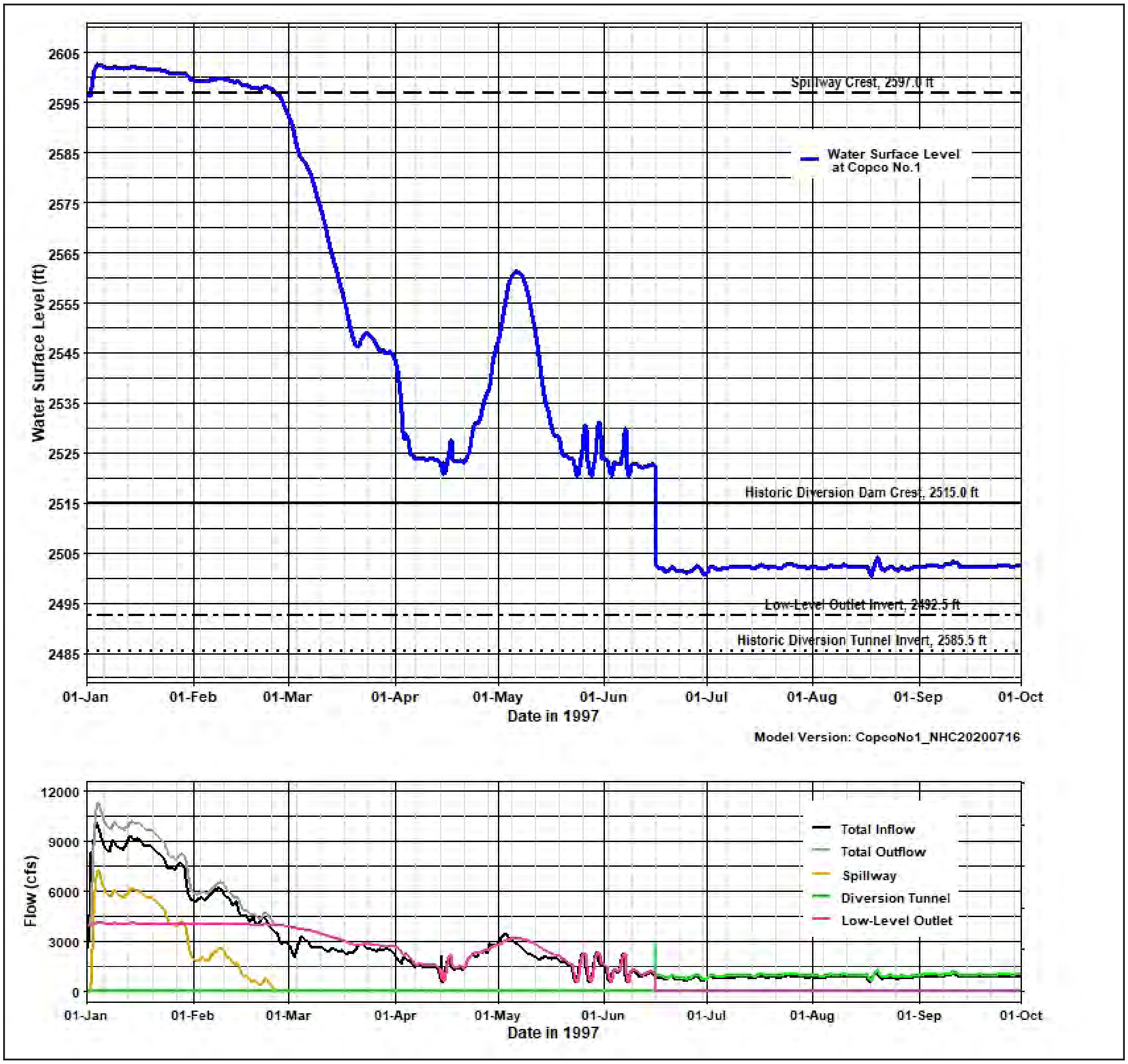

Figure 17 - COPCO No.1 Facility Simulated Drawdown - Year 1997

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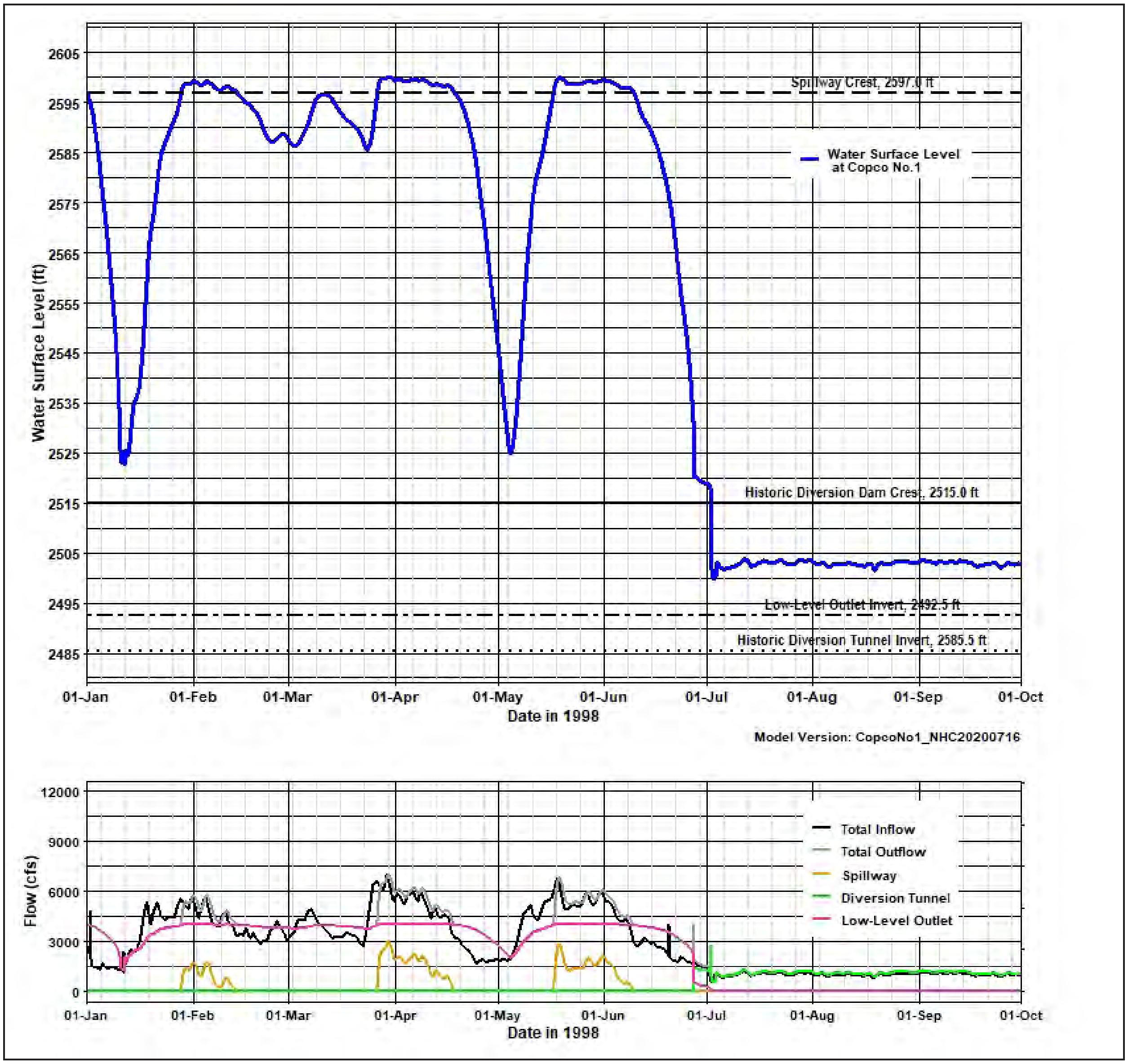

Figure 18 - COPCO No.1 Facility Simulated Drawdown - Year 1998

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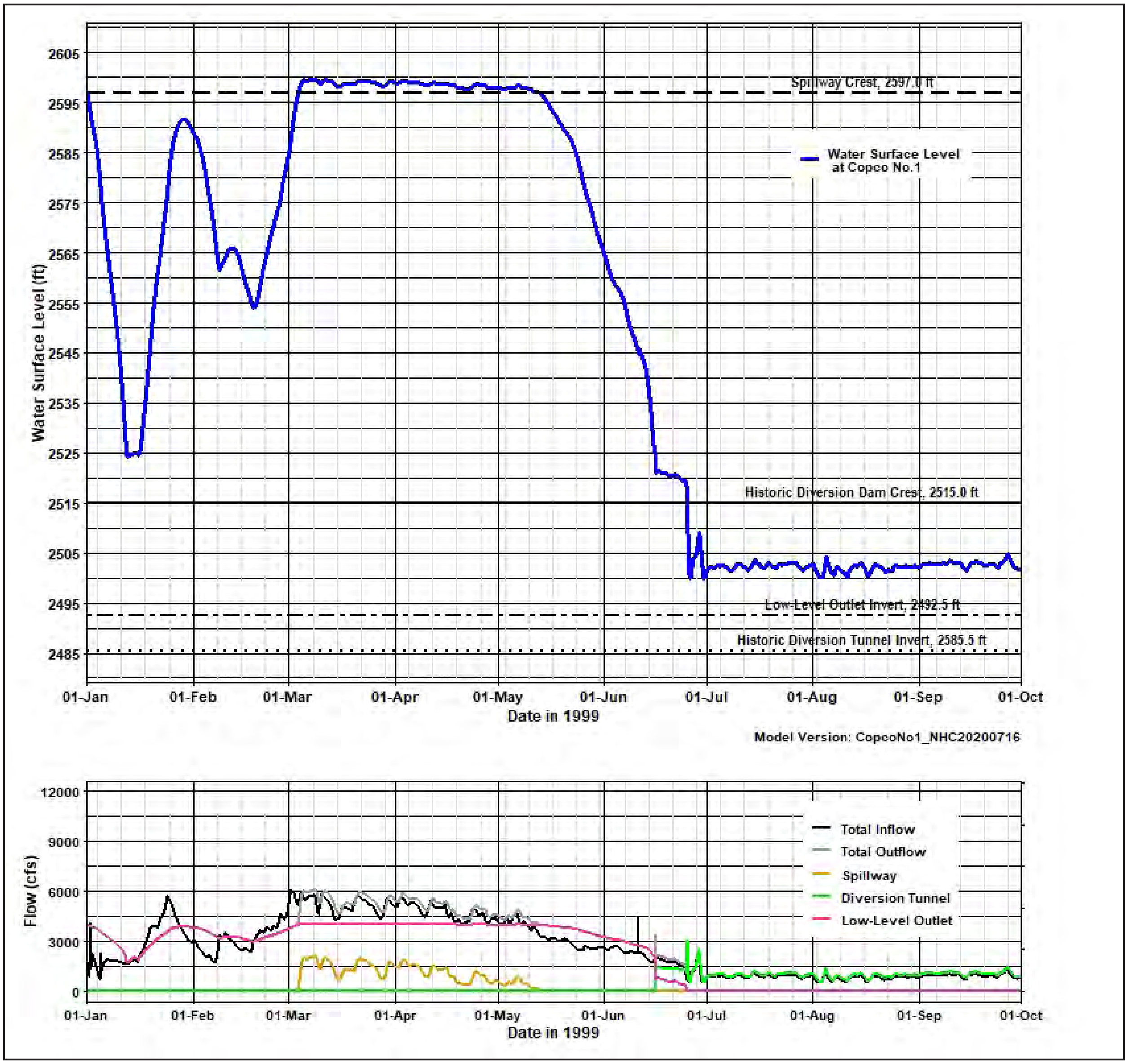

Figure 19 - COPCO No.1 Facility Simulated Drawdown - Year 1999

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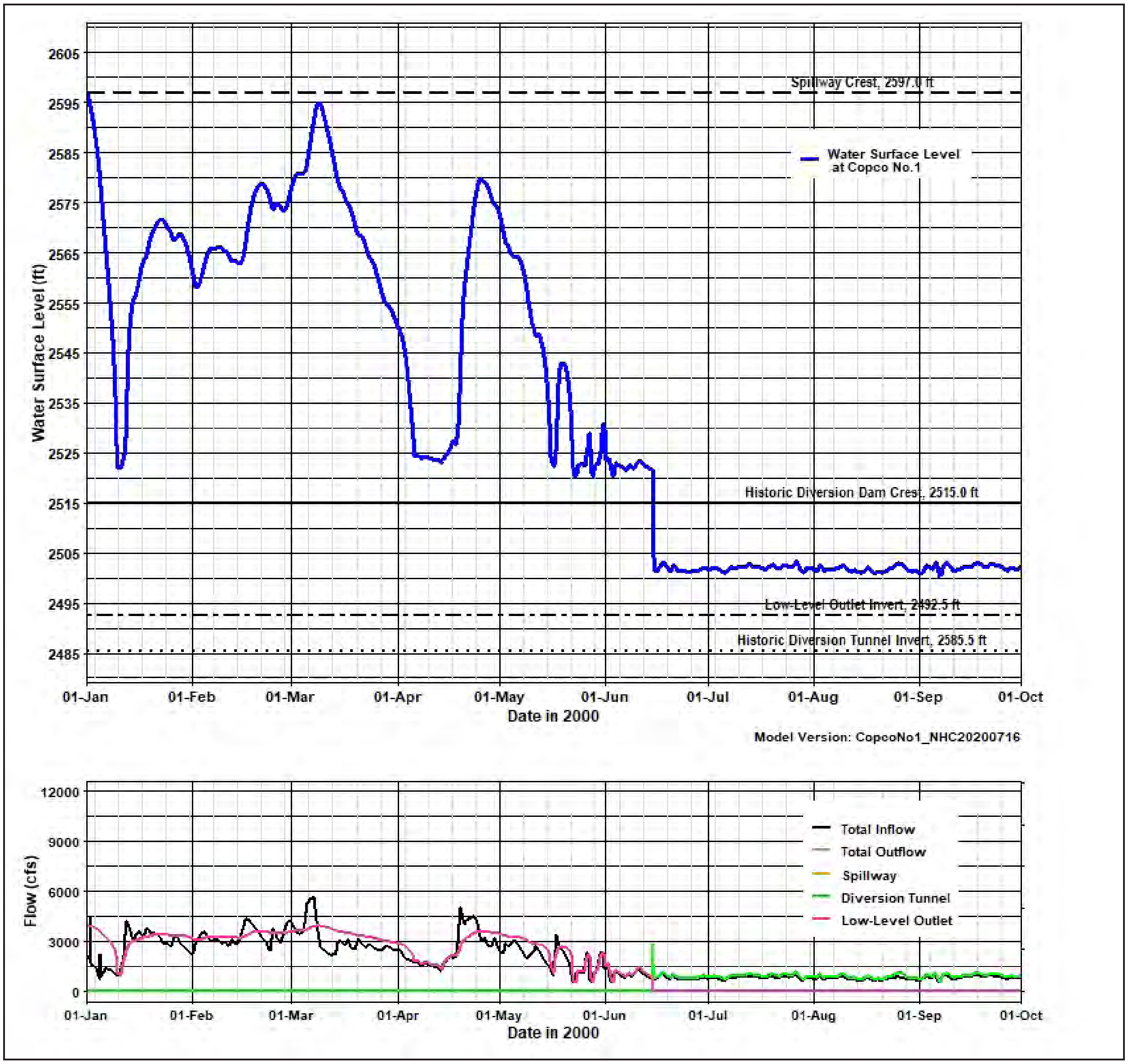

Figure 20 - COPCO No.1 Facility Simulated Drawdown - Year 2000

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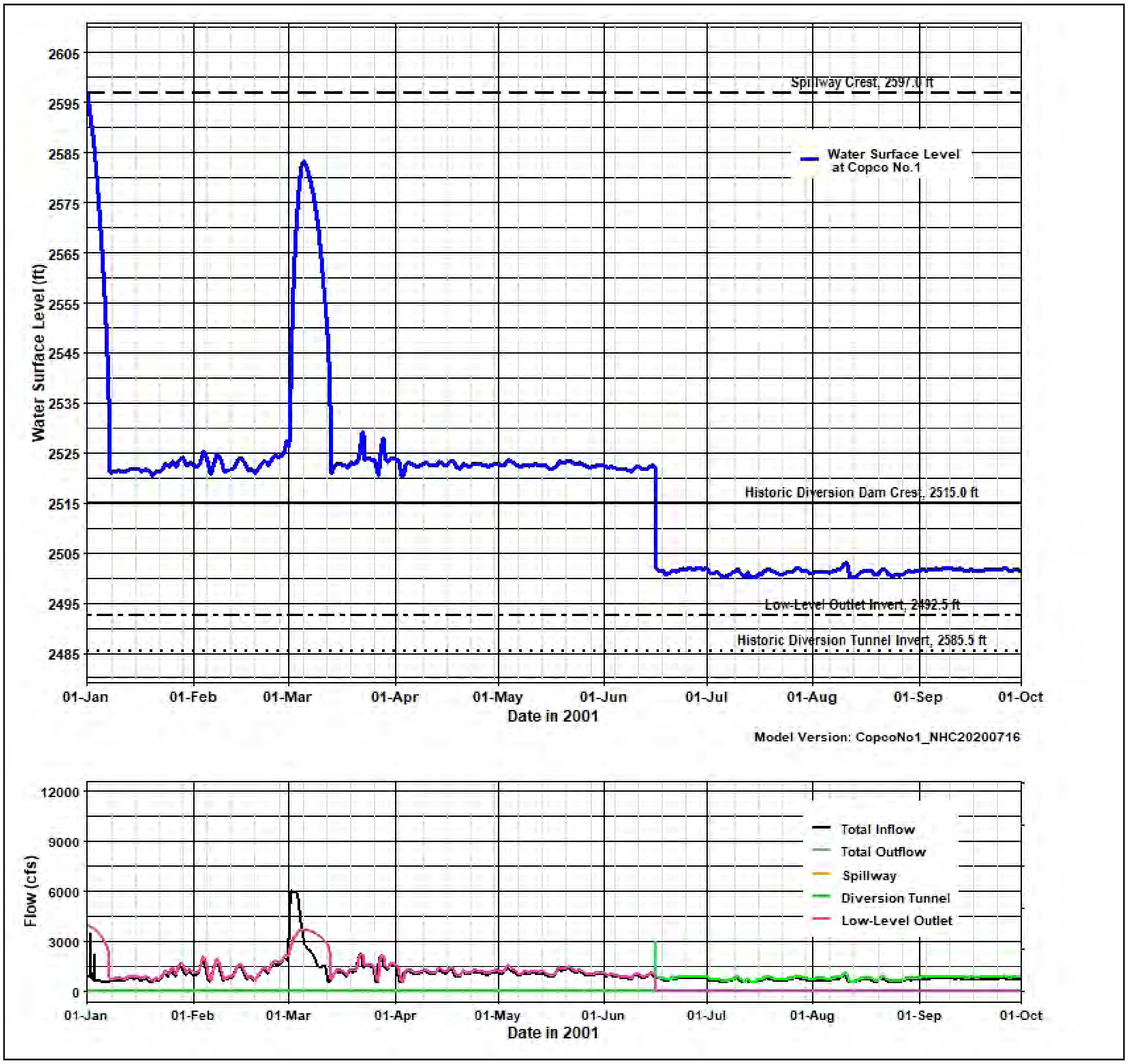

Figure 21 - COPCO No.1 Facility Simulated Drawdown - Year 2001

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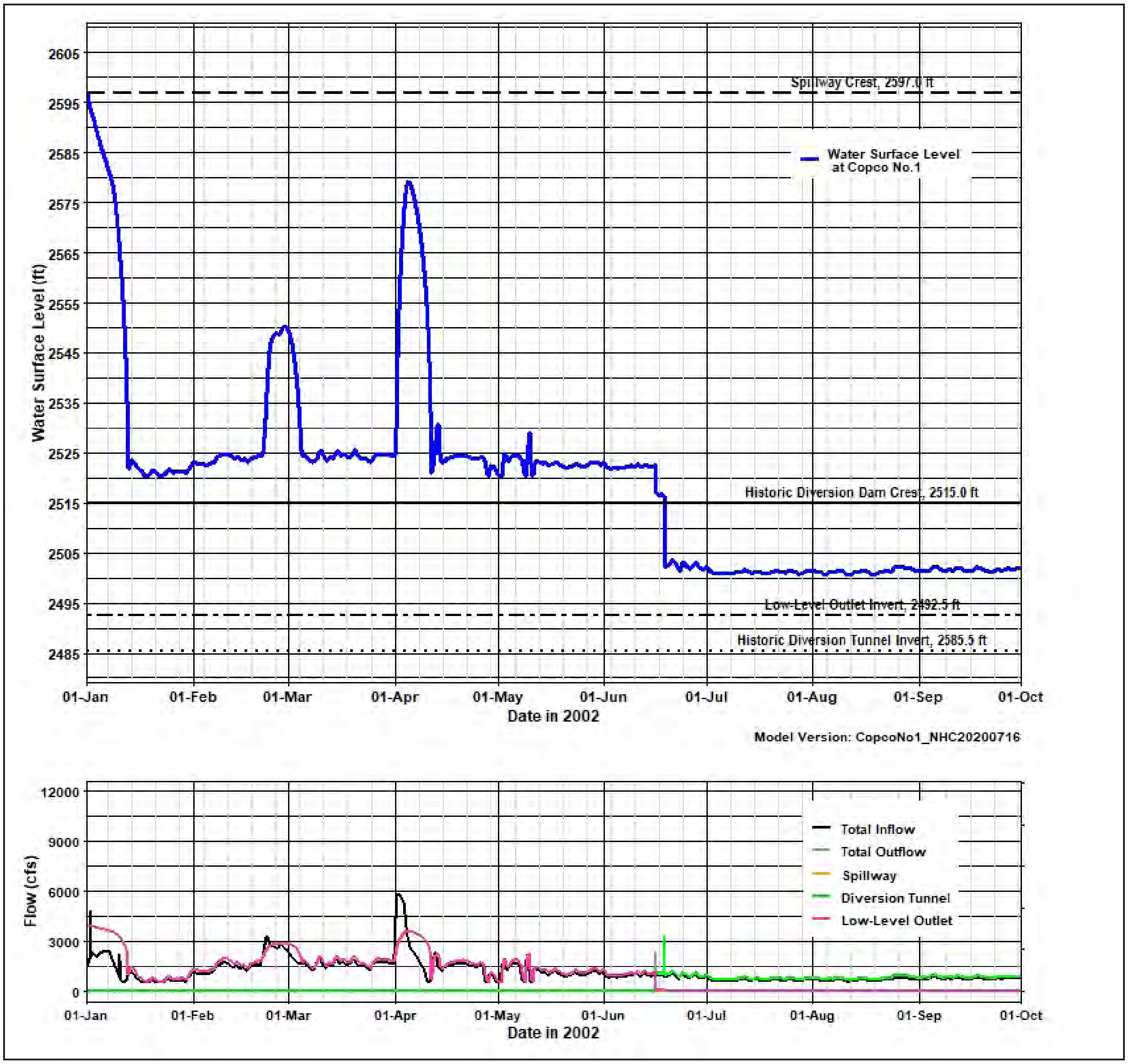

Figure 22 - COPCO No.1 Facility Simulated Drawdown - Year 2002

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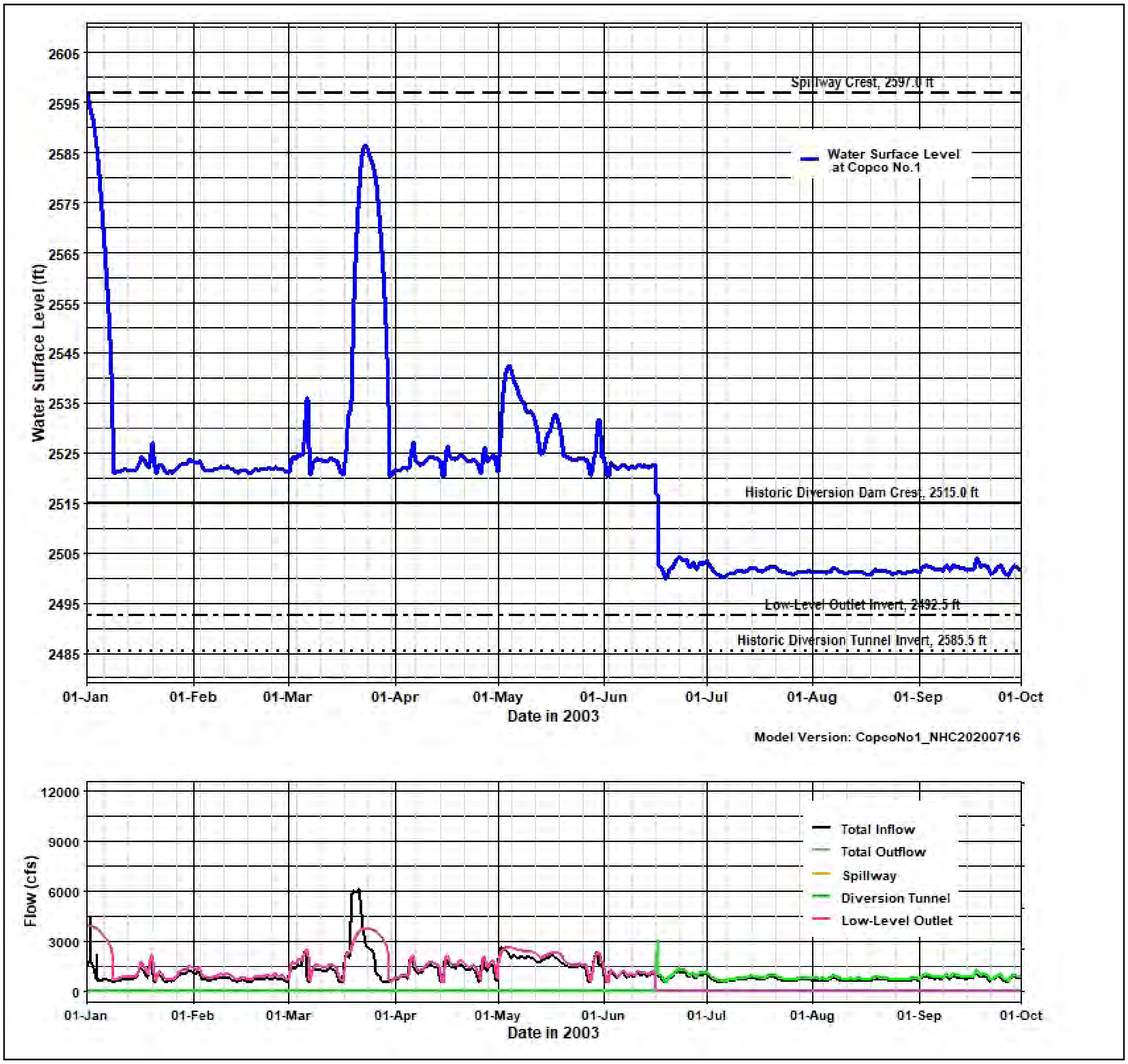

Figure 23 - COPCO No.1 Facility Simulated Drawdown - Year 2003

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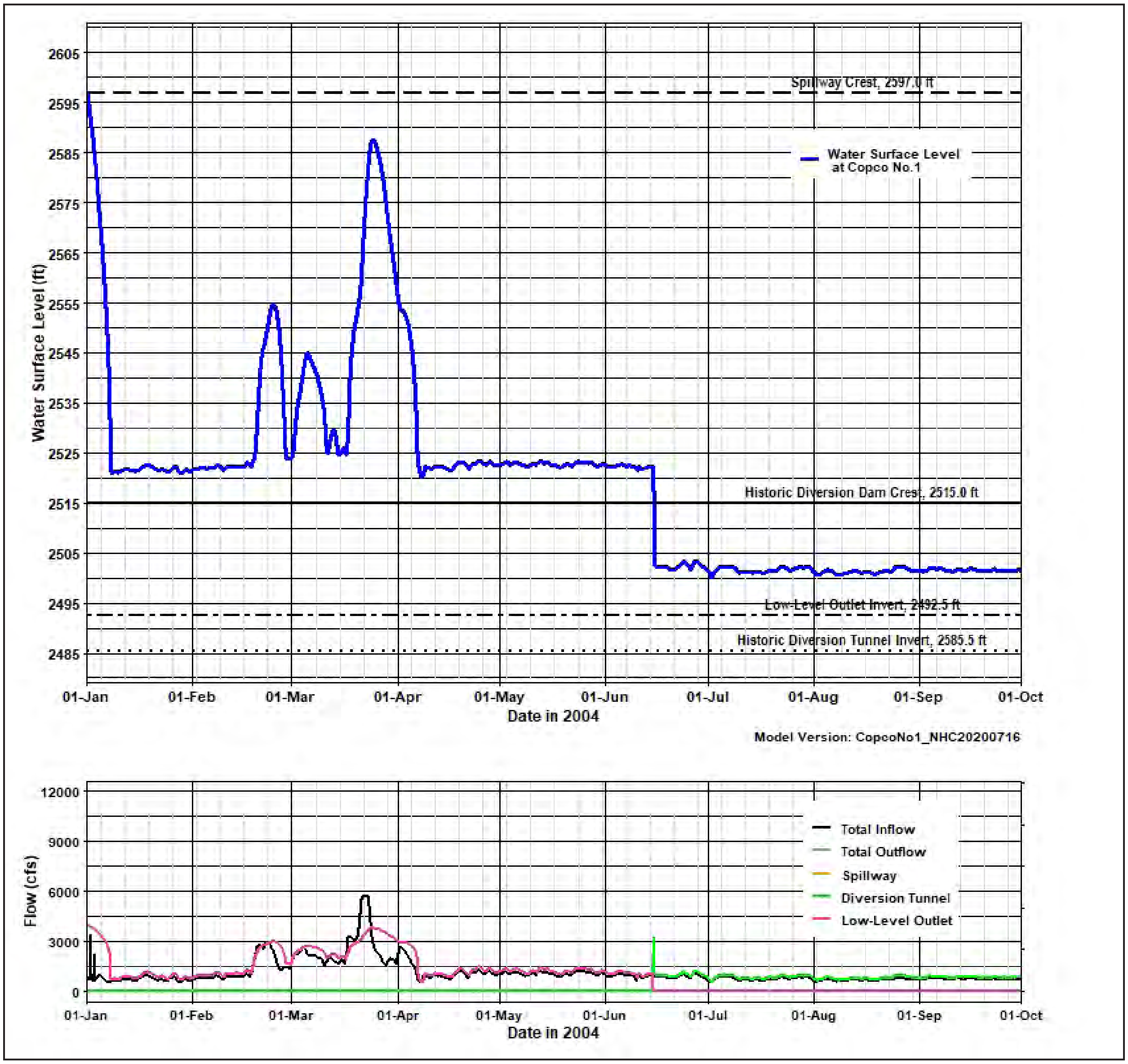

Figure 24 - COPCO No.1 Facility Simulated Drawdown - Year 2004

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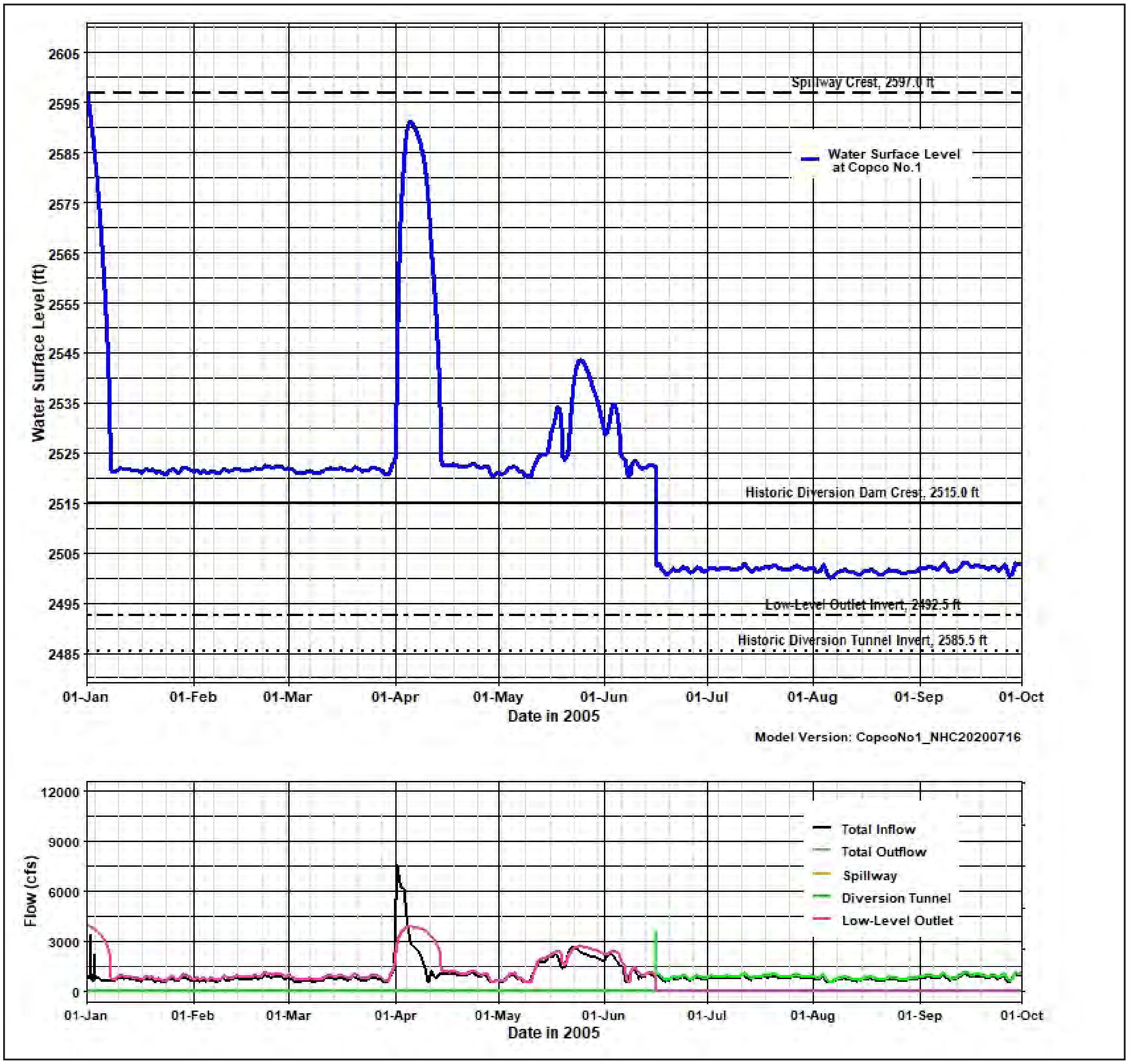

Figure 25 - COPCO No.1 Facility Simulated Drawdown - Year 2005

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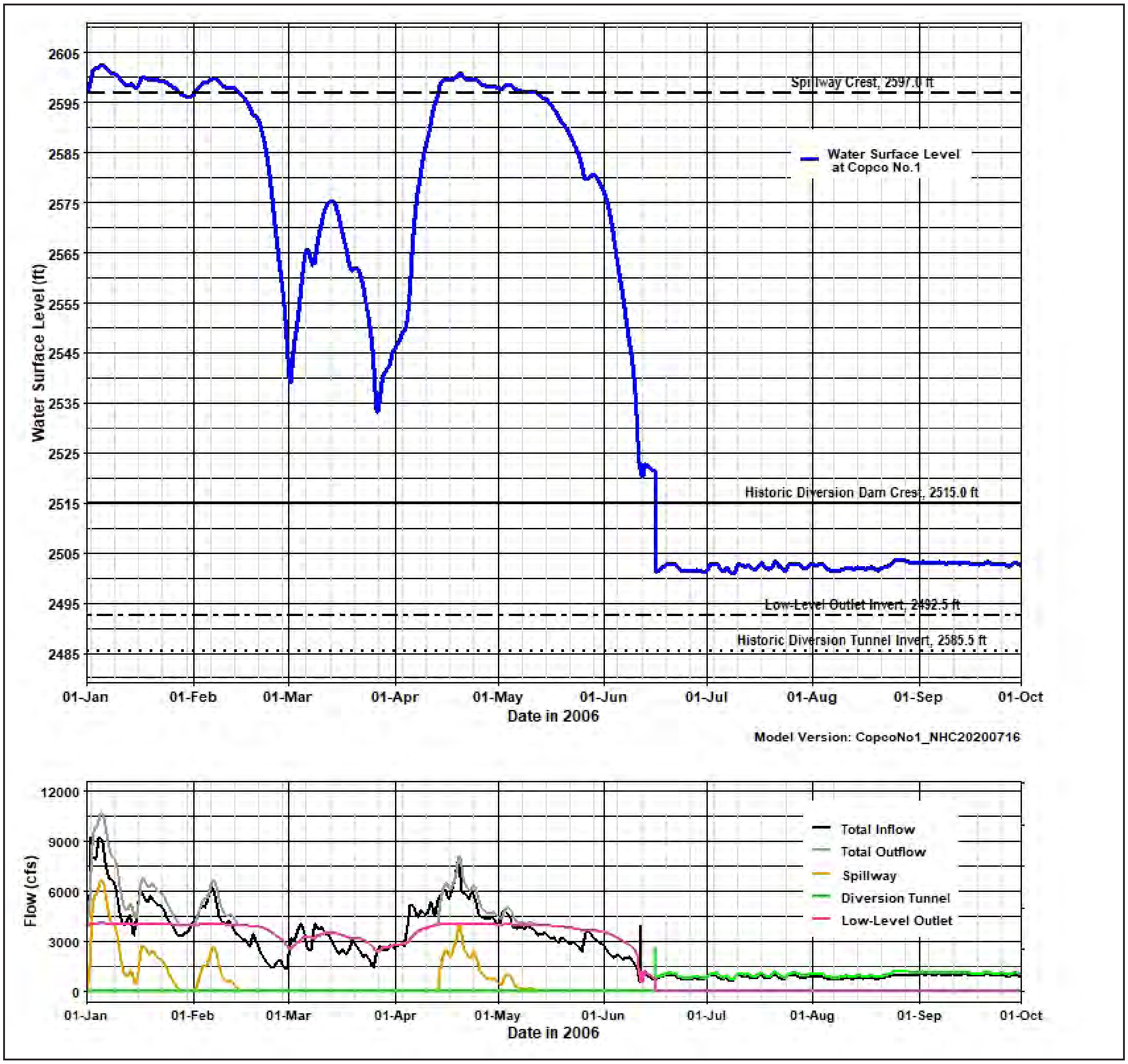

Figure 26 - COPCO No.1 Facility Simulated Drawdown - Year 2006

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

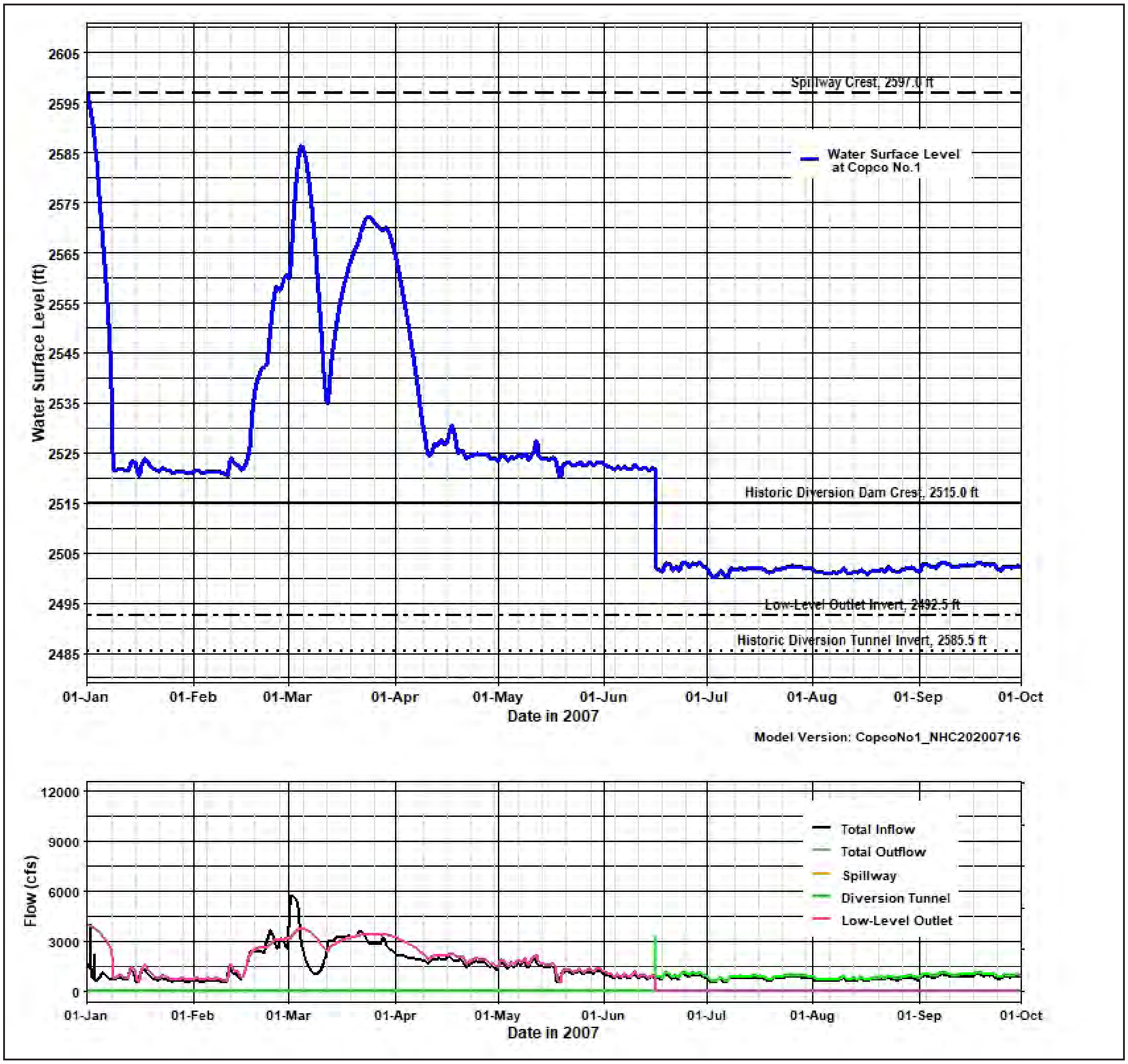

Figure 27 - COPCO No.1 Facility Simulated Drawdown - Year 2007

Knight Piésold CONSULTING

COPCO NO.1 FACILITY SIMULATED DRAWDOWN

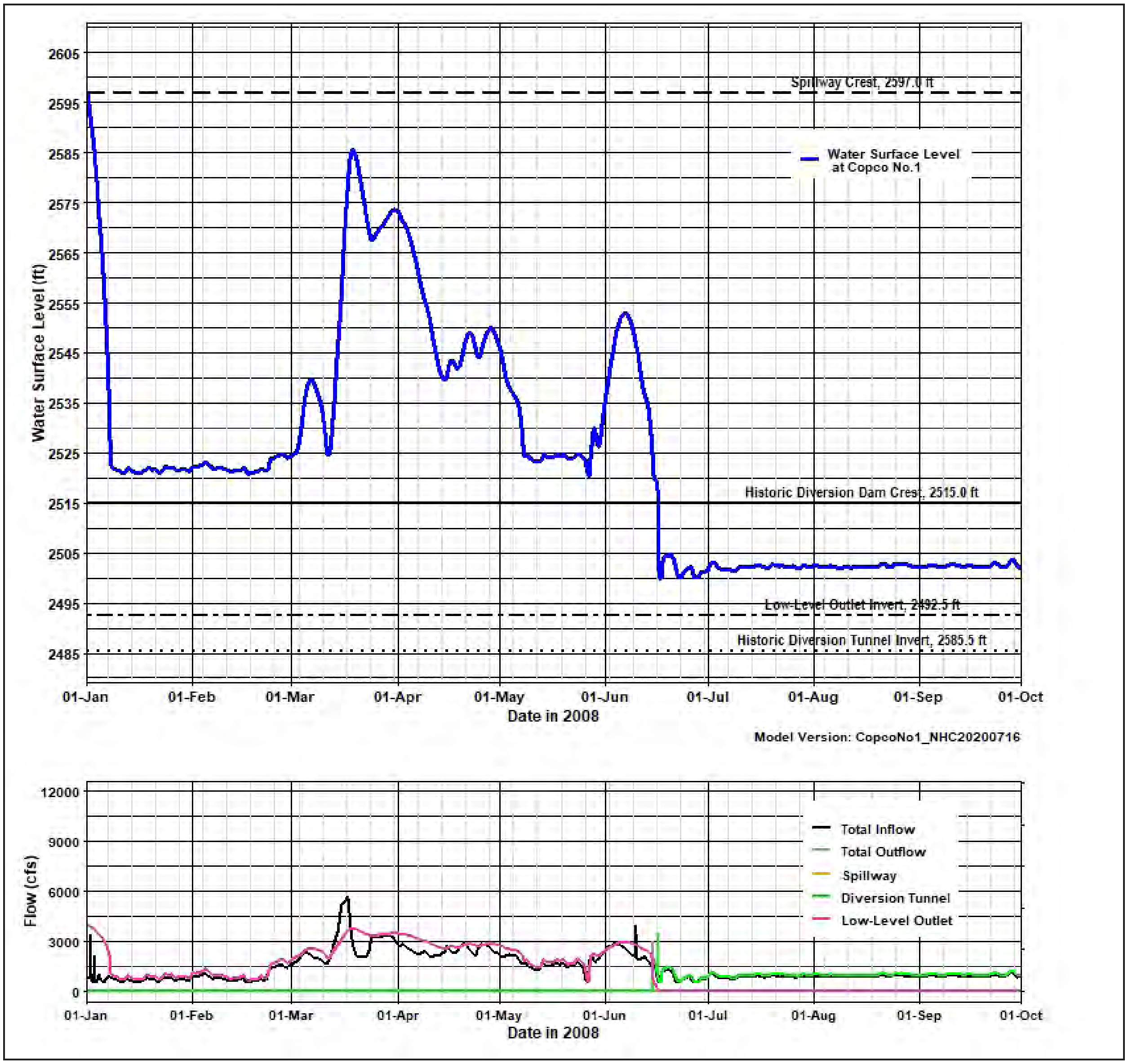

Figure 28 - COPCO No.1 Facility Simulated Drawdown - Year 2008

Knight Piésold CONSULTING

COPCO NO.1 FACILITY SIMULATED DRAWDOWN

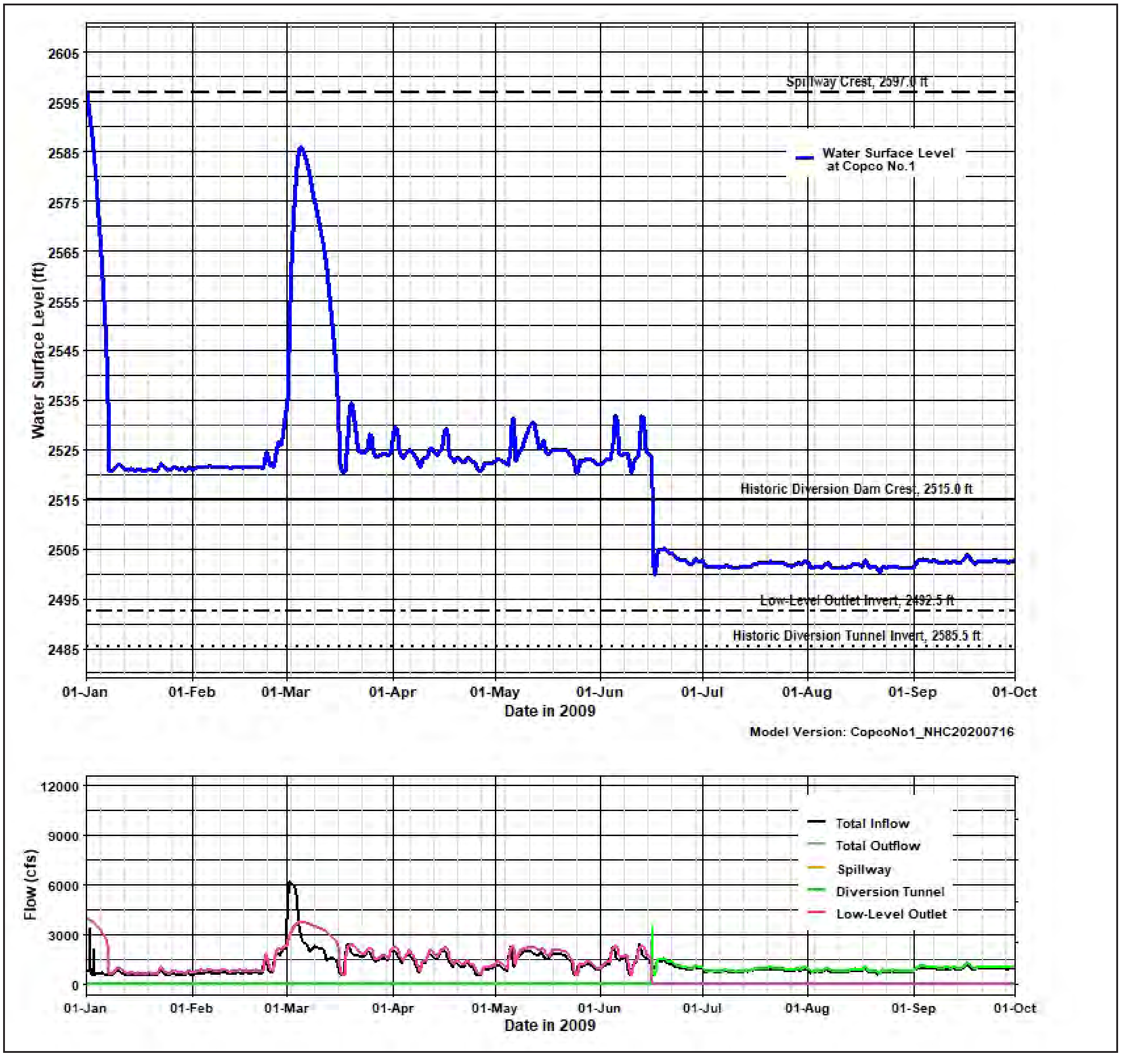

Figure 29 - COPCO No.1 Facility Simulated Drawdown - Year 2009

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo1_NHC20200716\individual_years

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

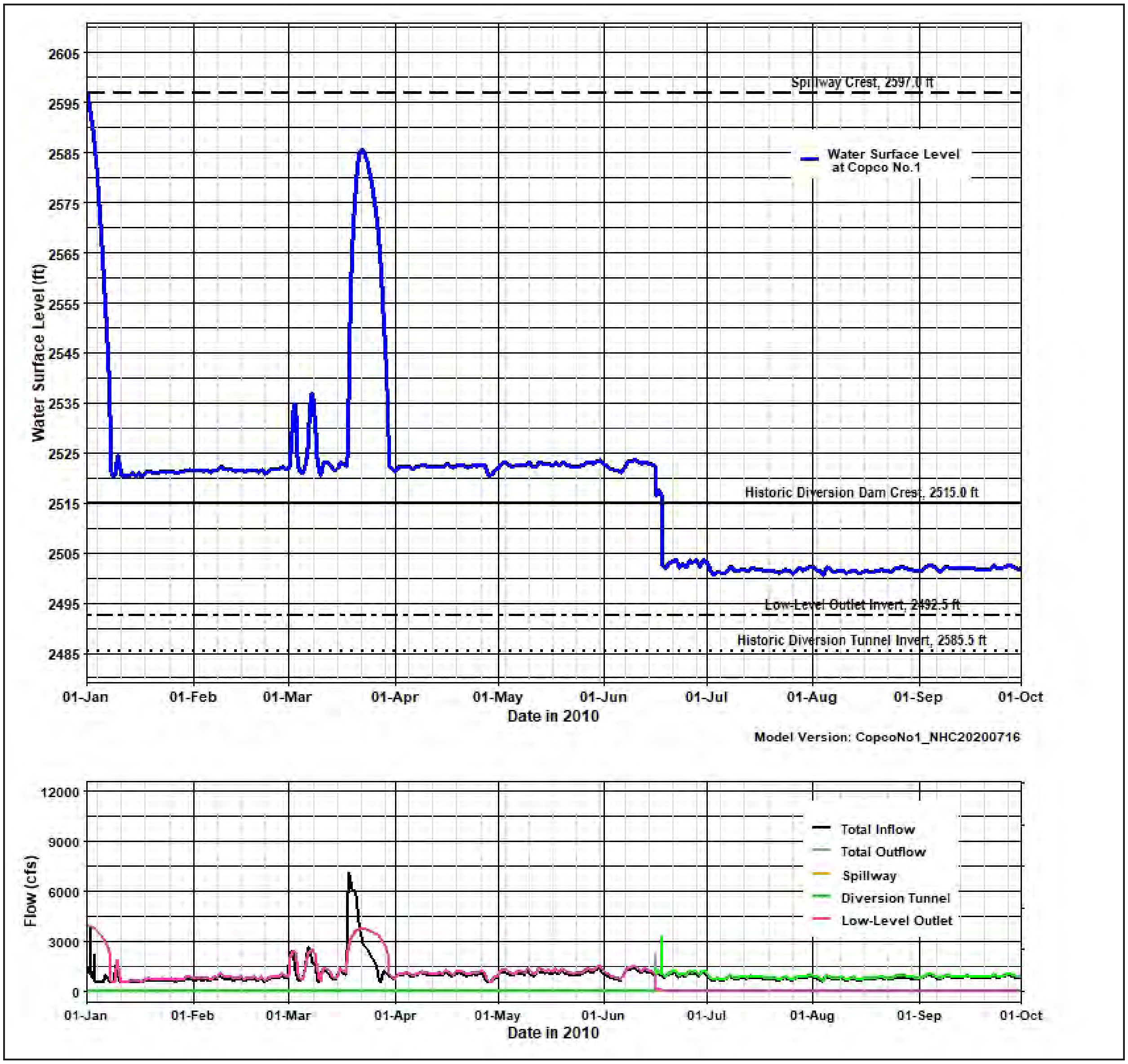

Figure 30 - COPCO No.1 Facility Simulated Drawdown - Year 2010

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

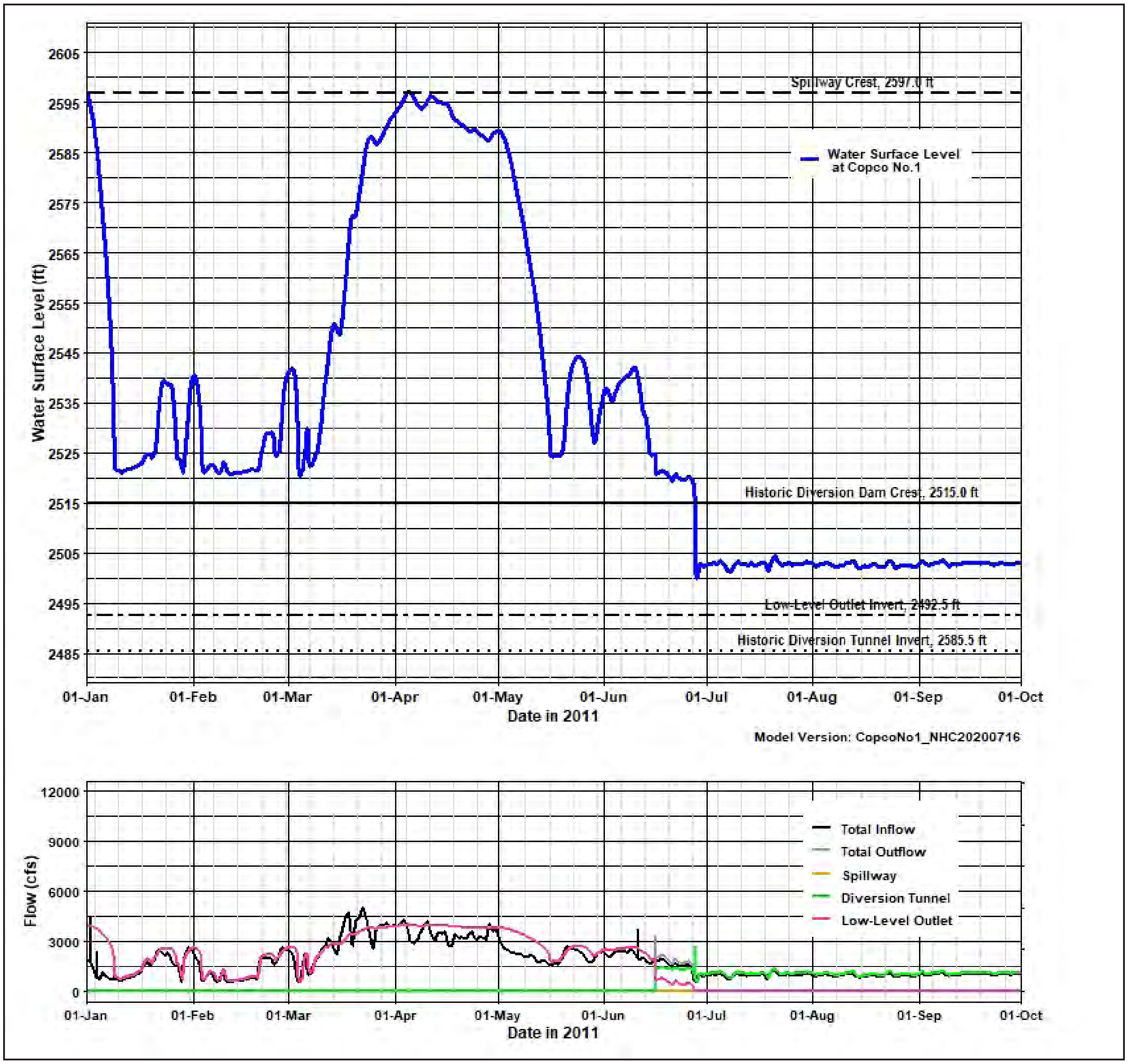

Figure 31 - COPCO No.1 Facility Simulated Drawdown - Year 2011

July 29, 2020

\\KPL\VA-Prj$\1\03\00640\01\A\Data\Task 0900 - 90% Design\08 - Hydrology\4_Drawdown Assessment\8_NHC Drawdown Model Results\CopcoNo1_NHC20200716\individual_years

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

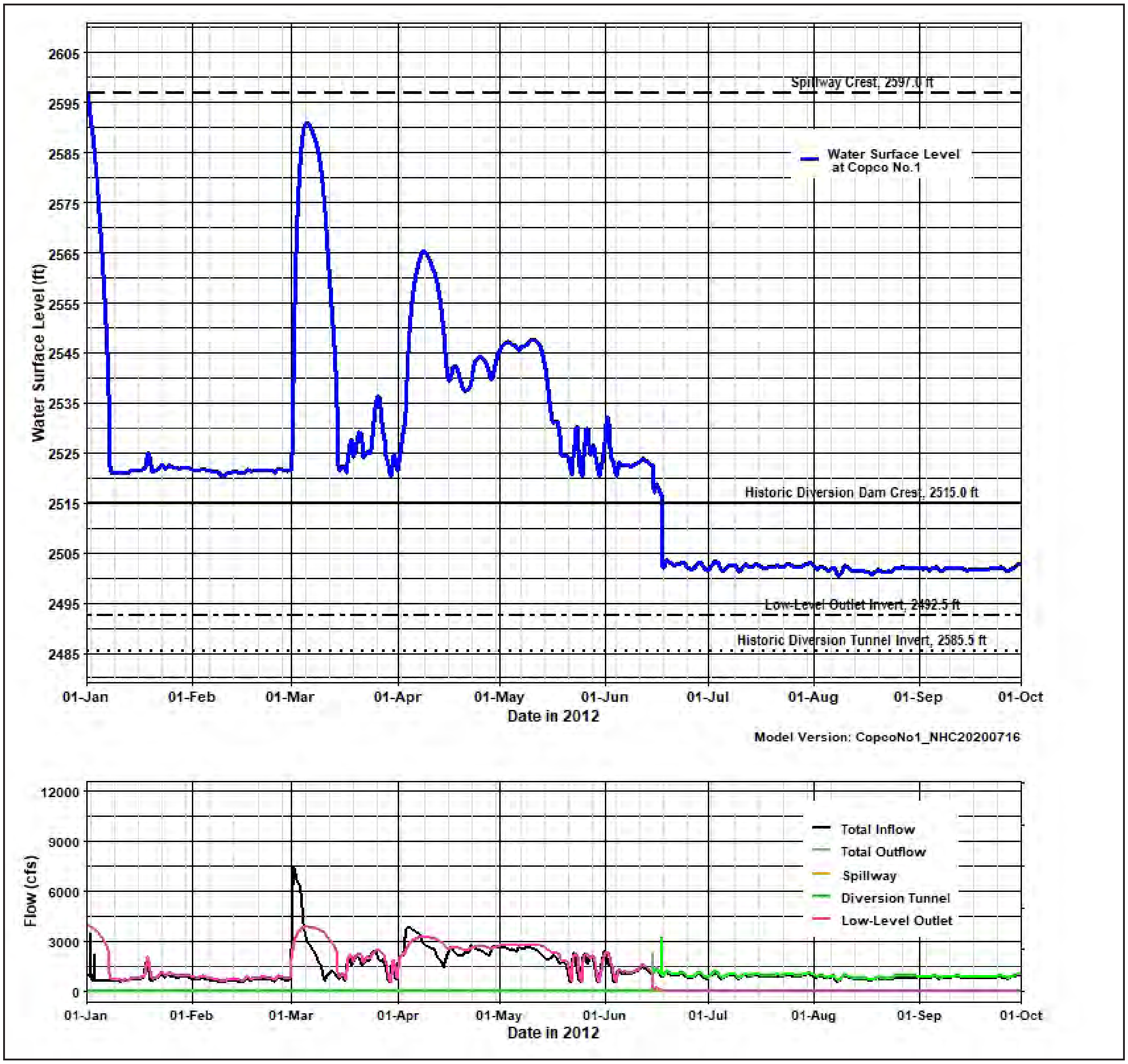

Figure 32 - COPCO No.1 Facility Simulated Drawdown - Year 2012

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

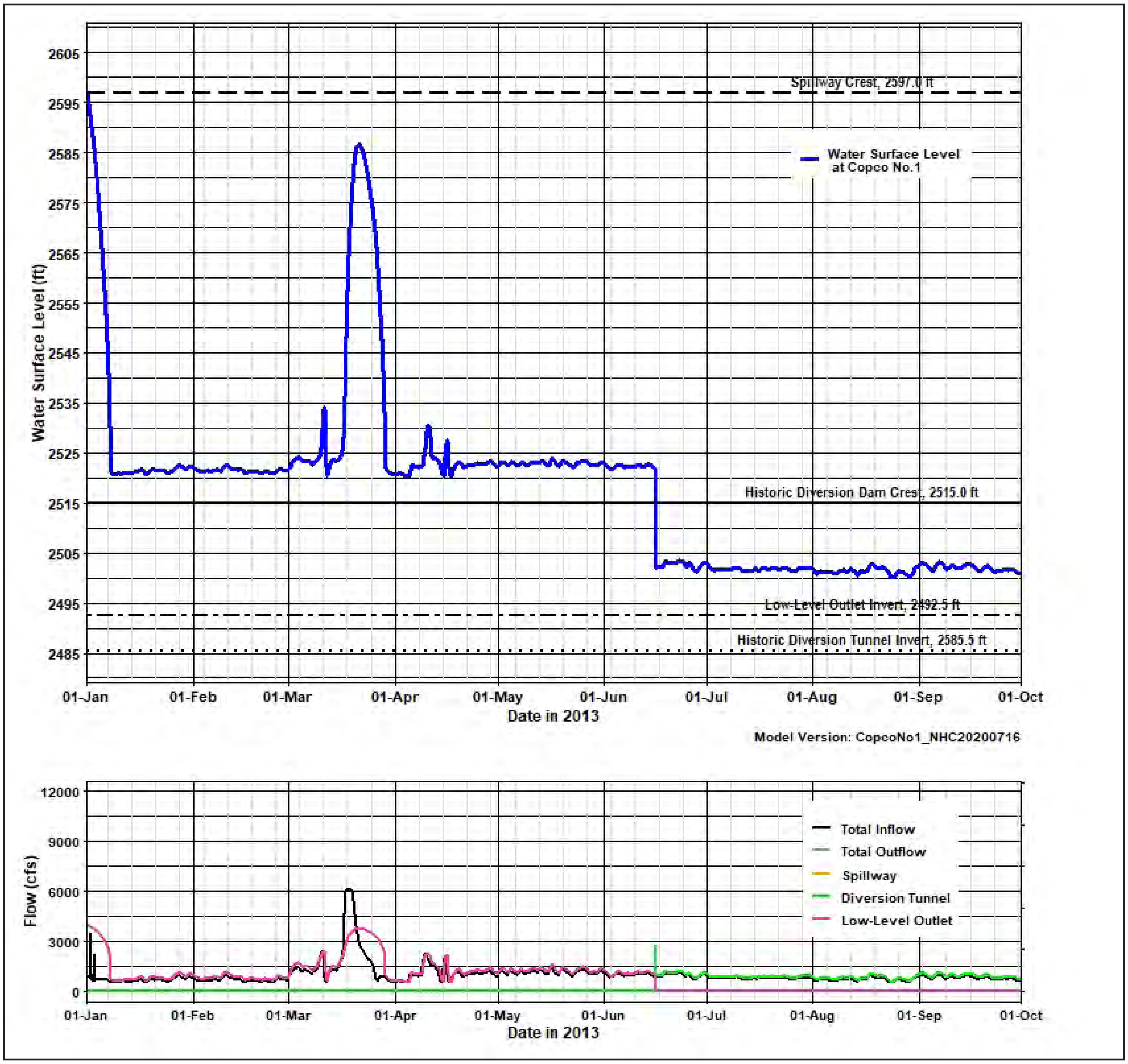

Figure 33 - COPCO No.1 Facility Simulated Drawdown - Year 2013

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

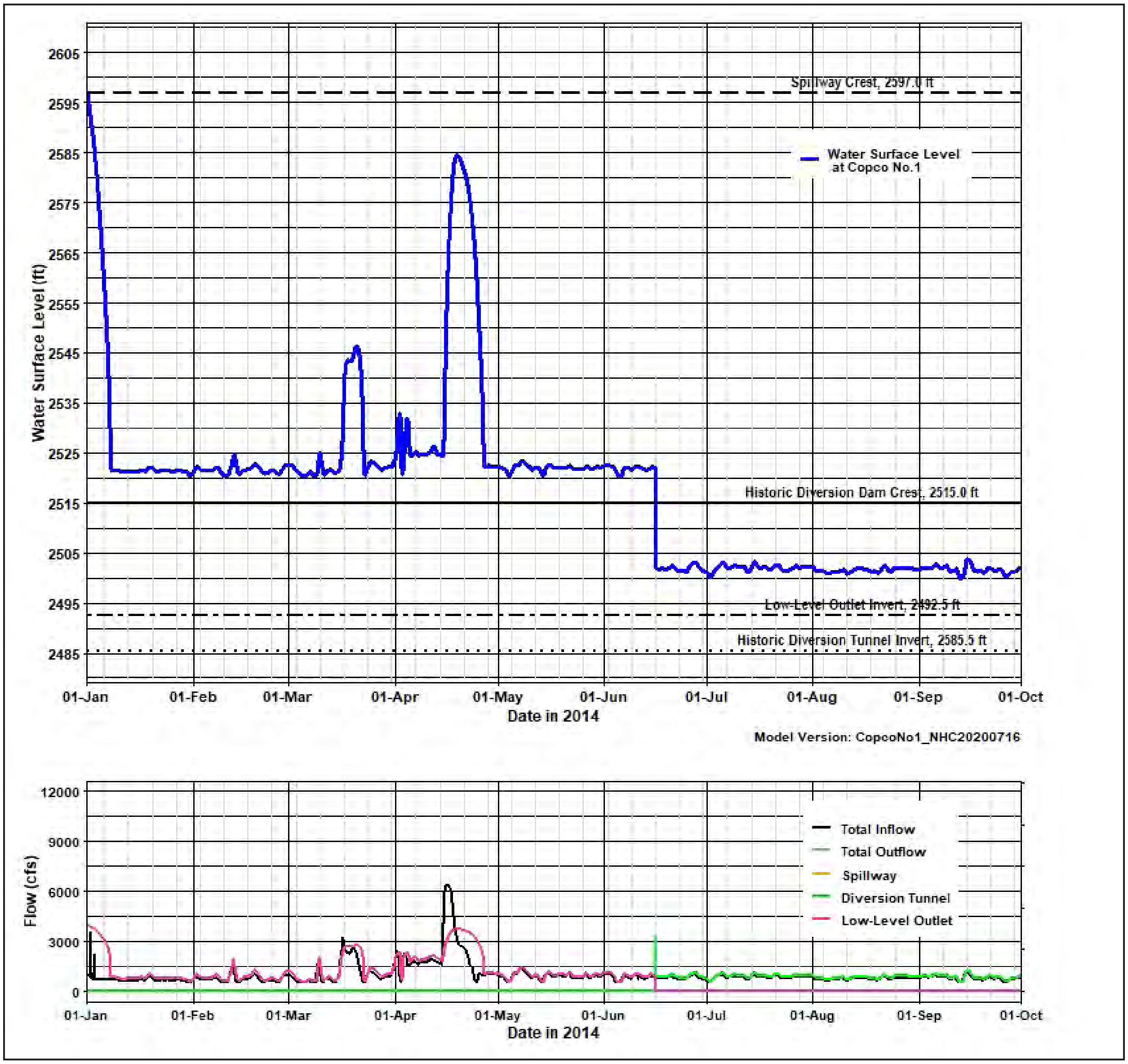

Figure 34 - COPCO No.1 Facility Simulated Drawdown - Year 2014

July 29, 2020

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

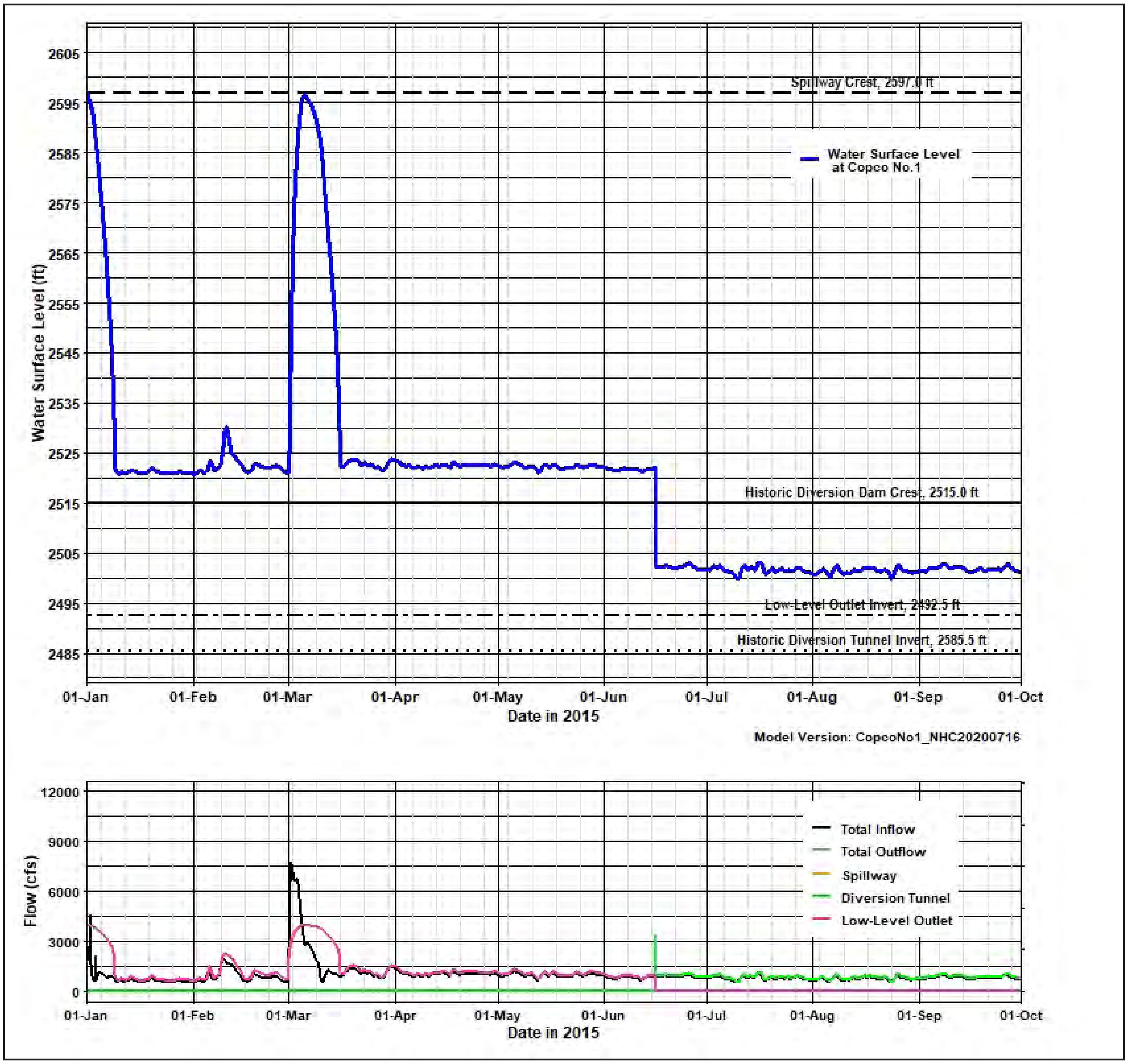

Figure 35 - COPCO No.1 Facility Simulated Drawdown - Year 2015

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COPCO NO.1 FACILITY SIMULATED DRAWDOWN

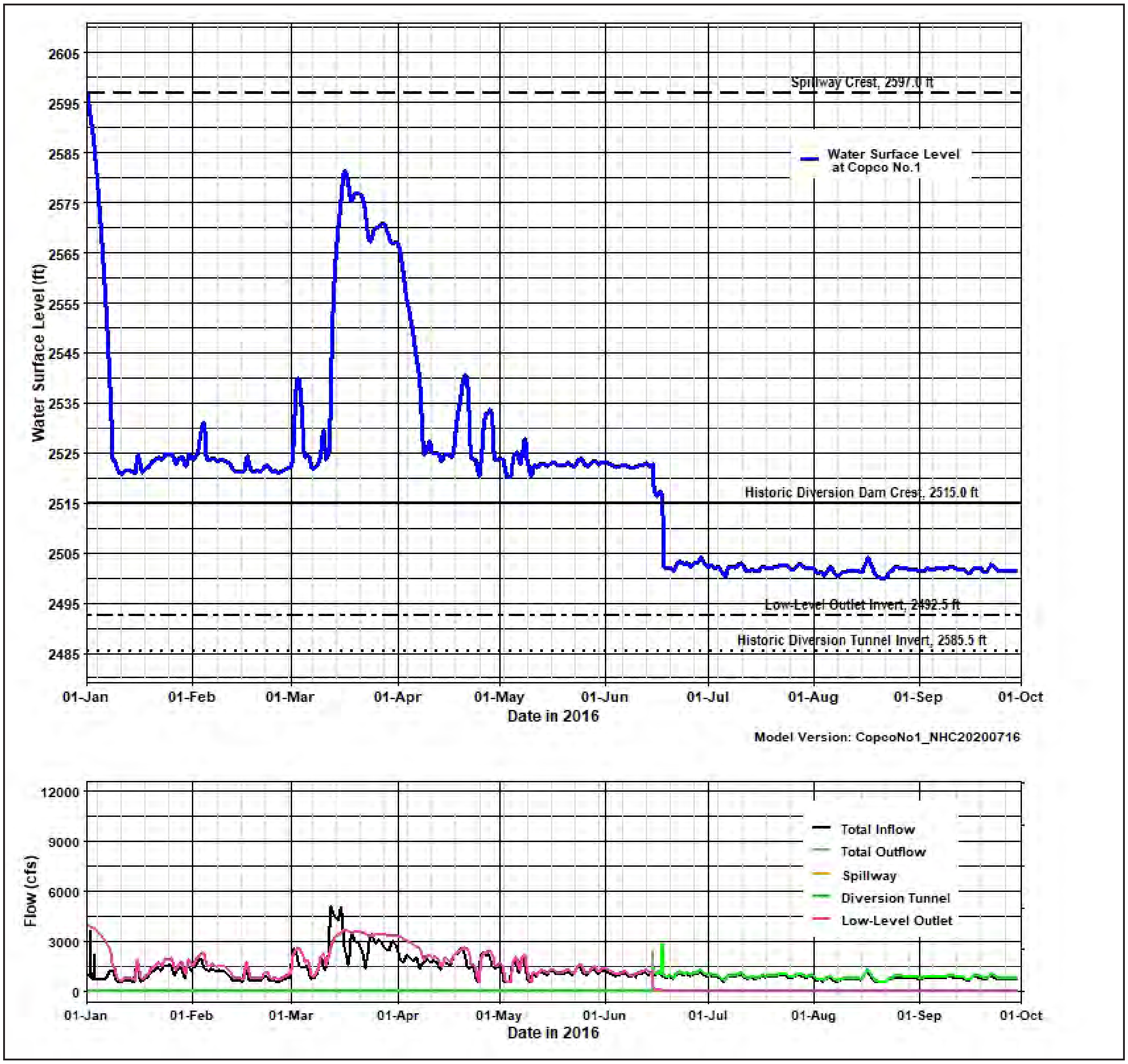

Figure 36 - COPCO No.1 Facility Simulated Drawdown - Year 2016

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.

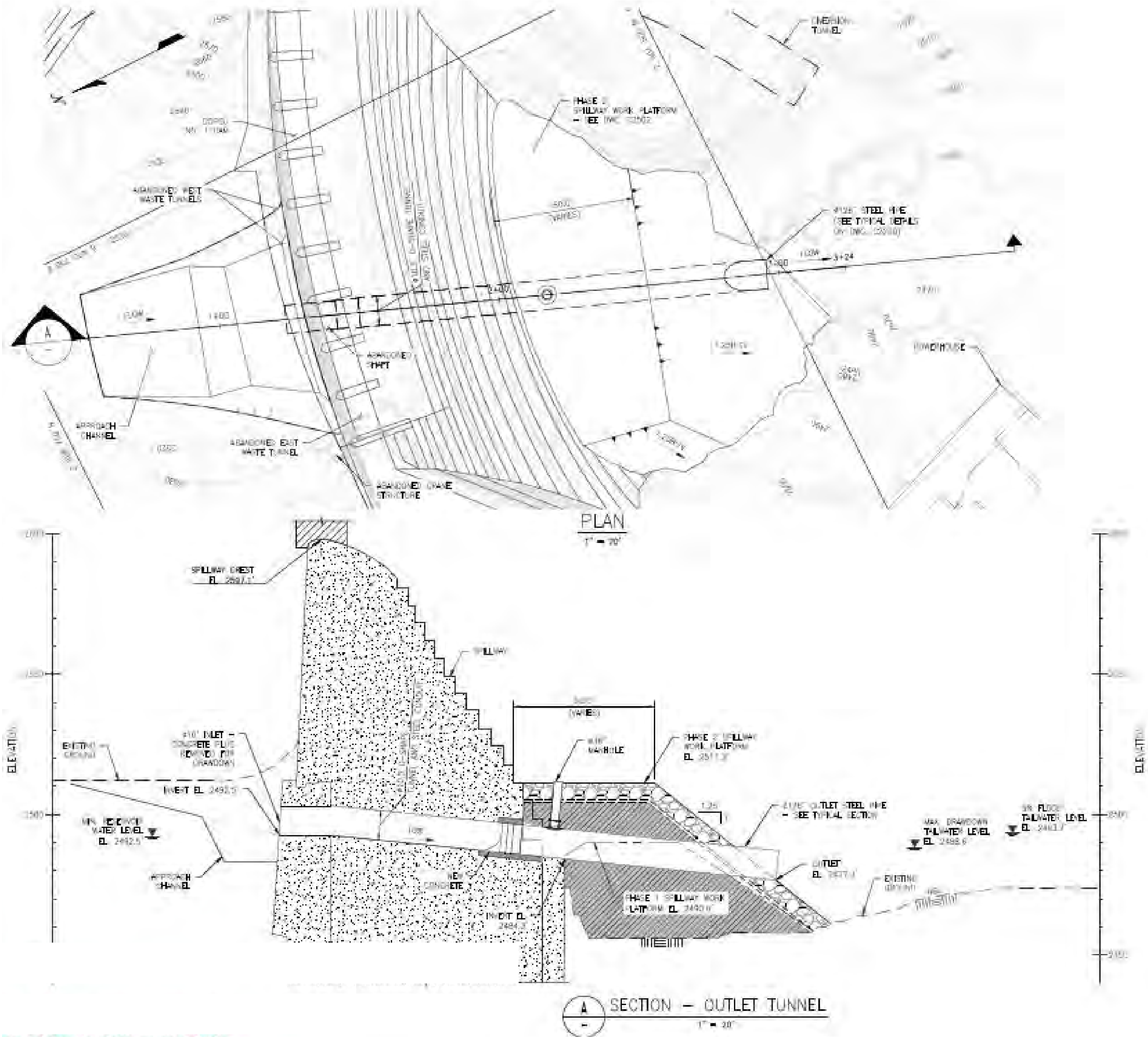

Figure 1. Plan view and section of proposed Copco No. 1 drawdown low-level outlet, KP Drawing C2205 (07/22/2020)

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.

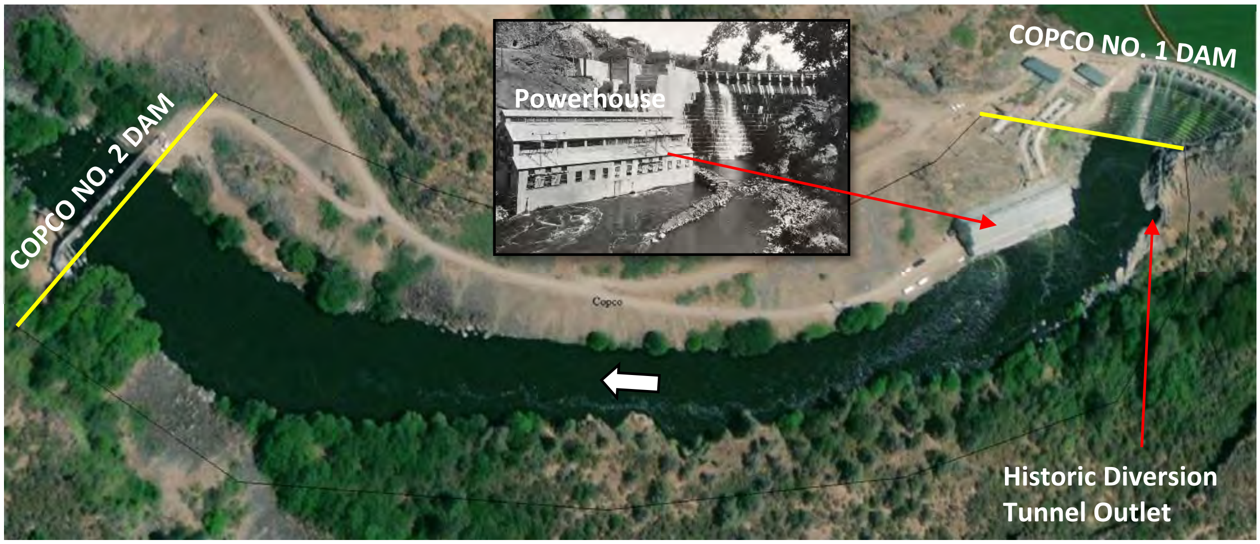

Figure 2. Extent of 2D hydrodynamic model between Copco No. 1 and Copco No. 2 Dams. Insert depicts historical photograph of Copco No. 1 powerhouse and cofferdam. Yellow lines show inflow and outflow sections of the model.

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.

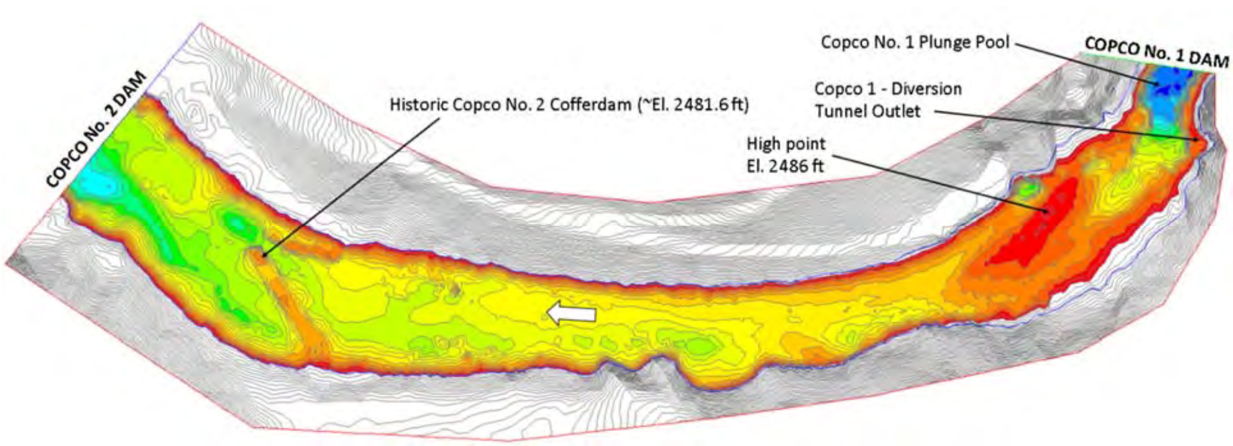

Figure 3. Existing pre-deconstruction bathymetry of Klamath River between Copco No. 1 and Copco No. 2 Dams (from *Topo_2018DEM_3ft.tif*)

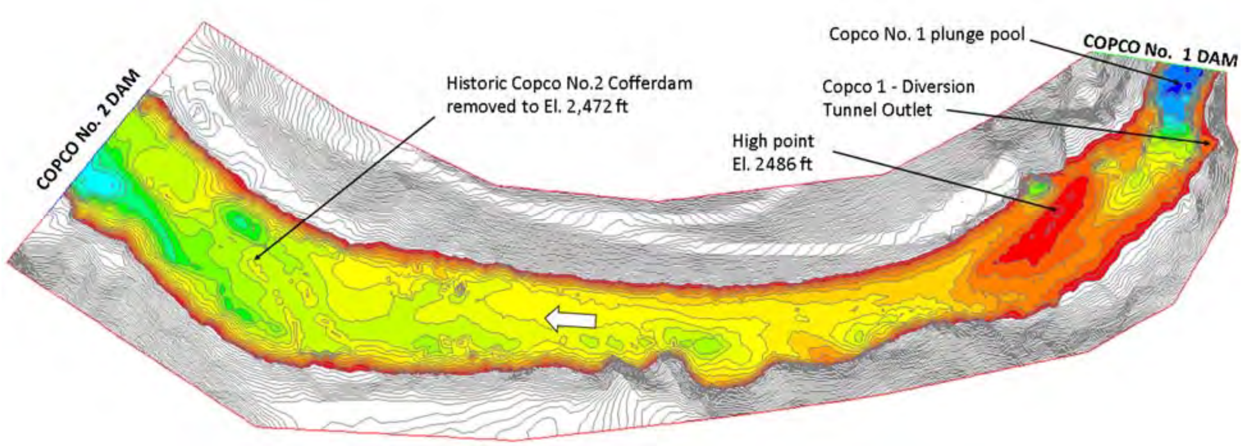

Figure 4. Modified post-deconstruction bathymetry of Klamath River between Copco No. 1 and Copco No. 2 Dams with historic Copco No. 2 cofferdam removed down to El. 2,472 ft.

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.

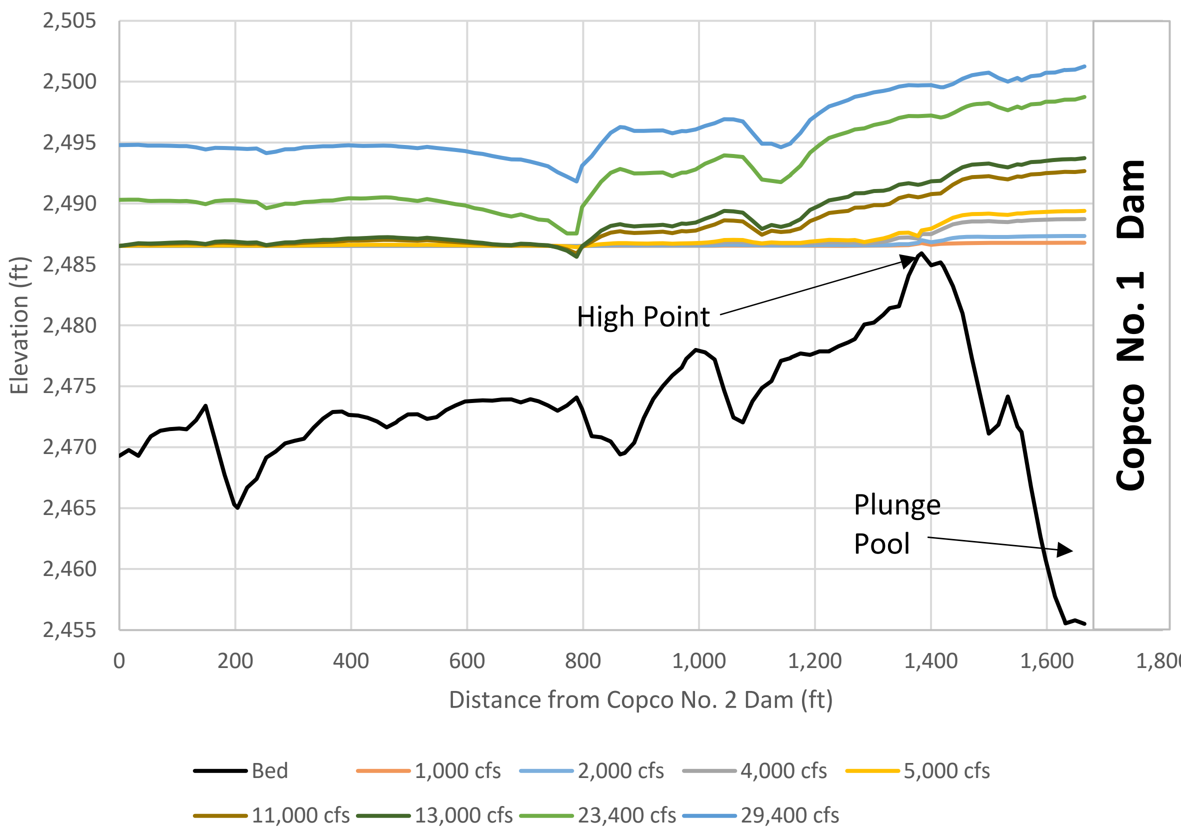

Figure 5. Pre-deconstruction water surface profile along Klamath River between Copco No. 1 and Copco No. 2 Dams

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.

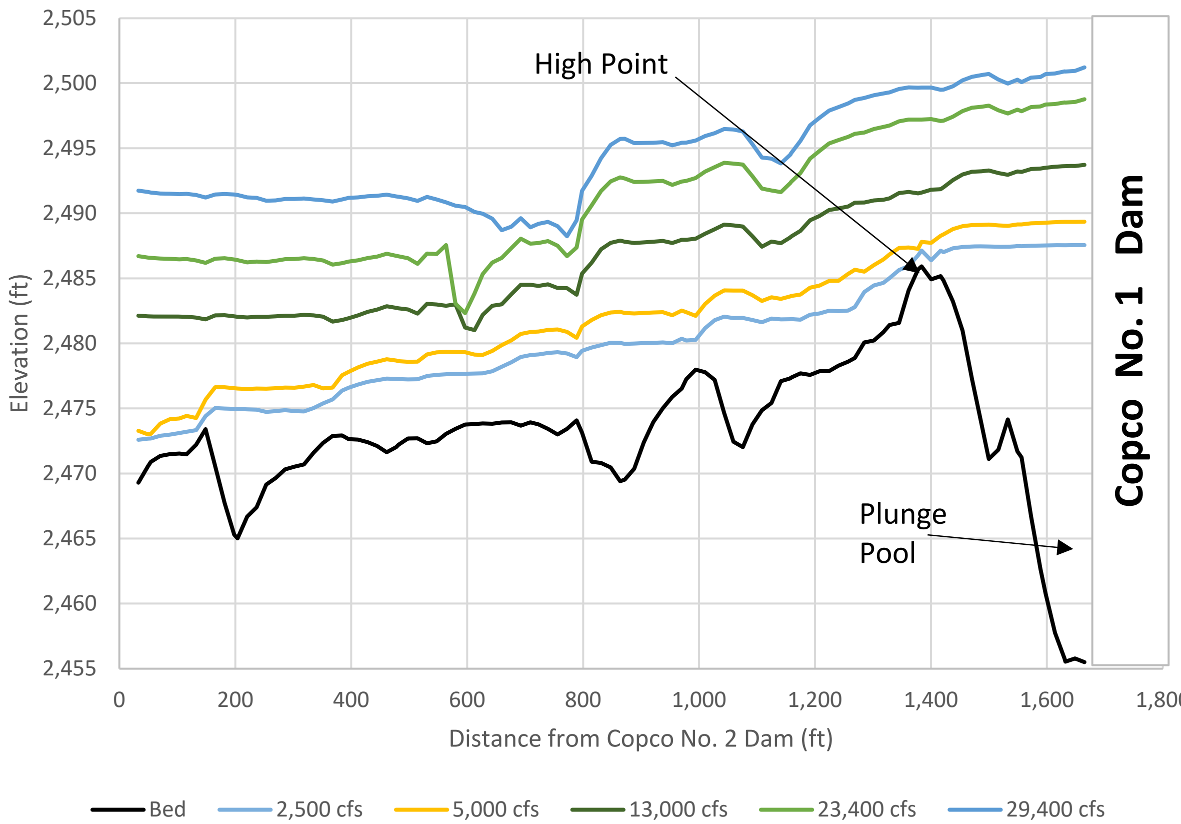

Figure 6. Post-deconstruction water surface profile along Klamath River between Copco No. 1 and Copco No. 2 Dams

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.

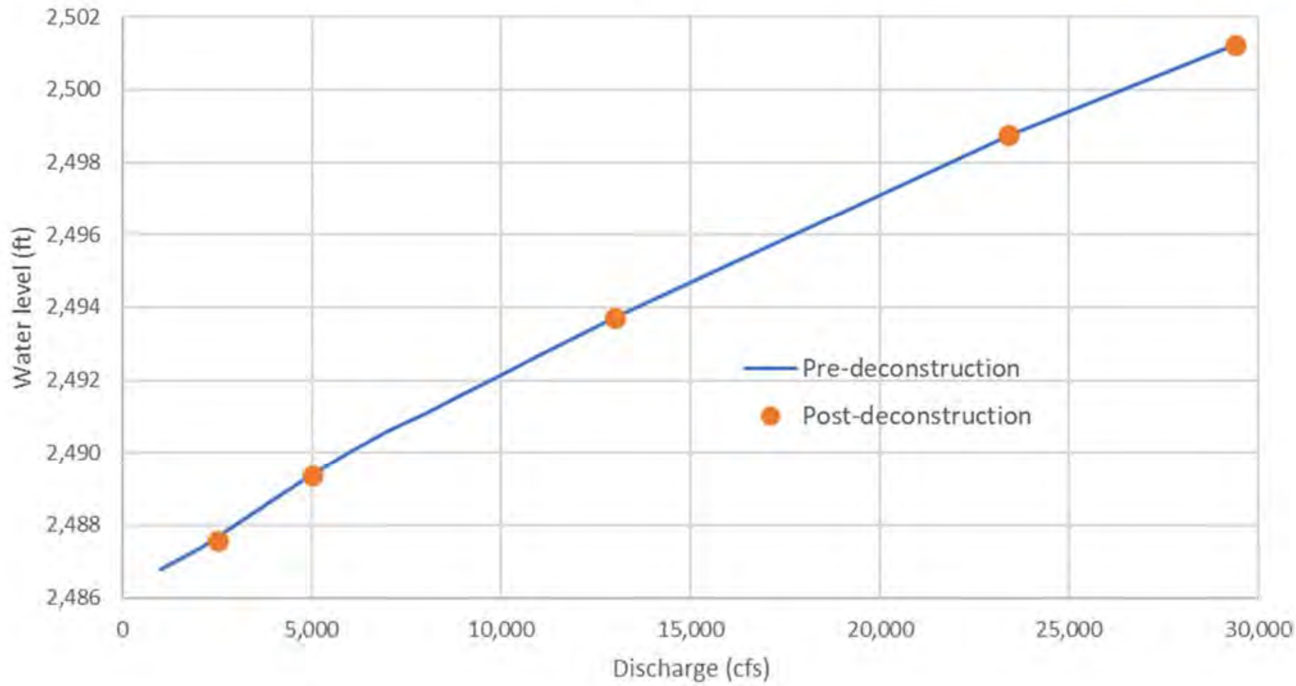

Figure 7. Copco No. 1 Dam tailwater rating curve

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.

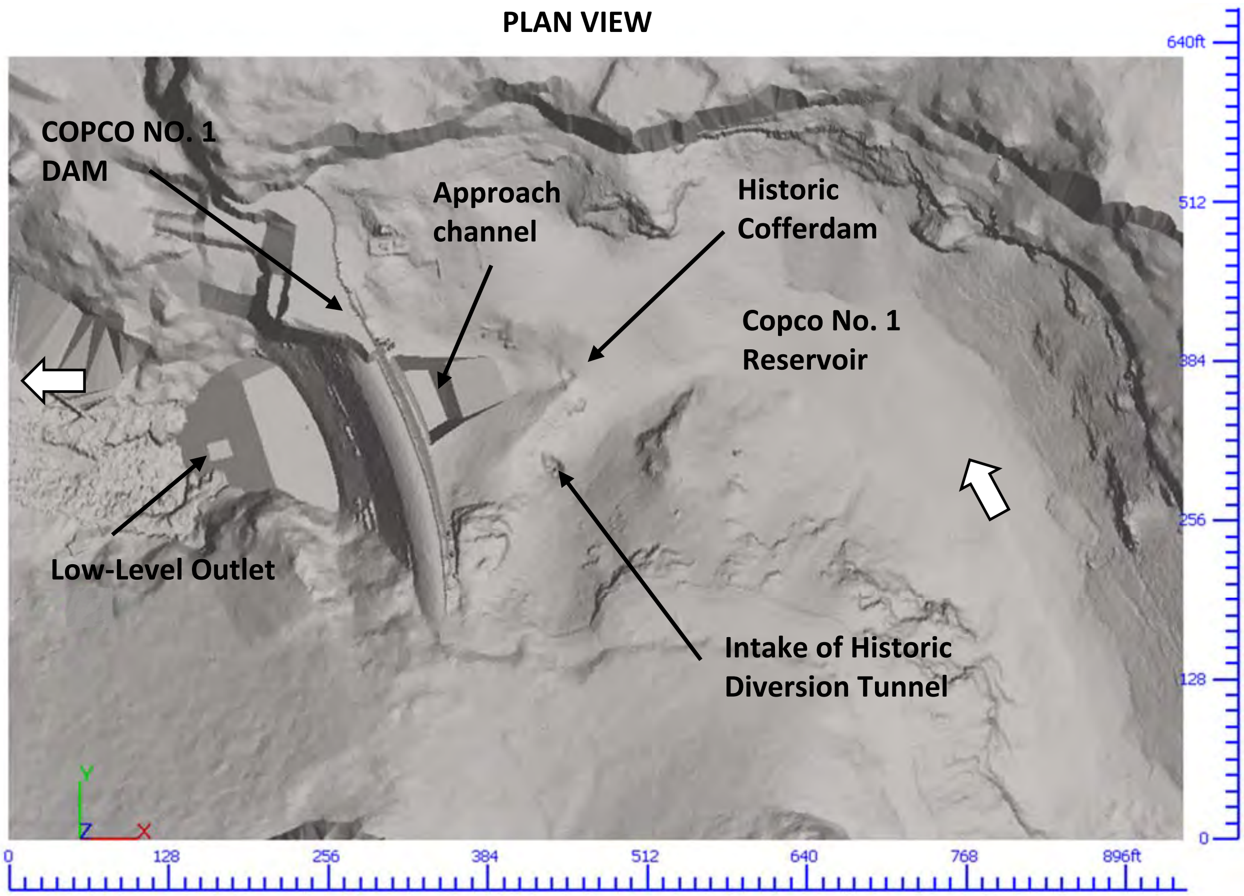

Figure 8. Geometry of 100% Design Copco No. 1 Dam CFD model

VIEW LOOKING UPSTREAM

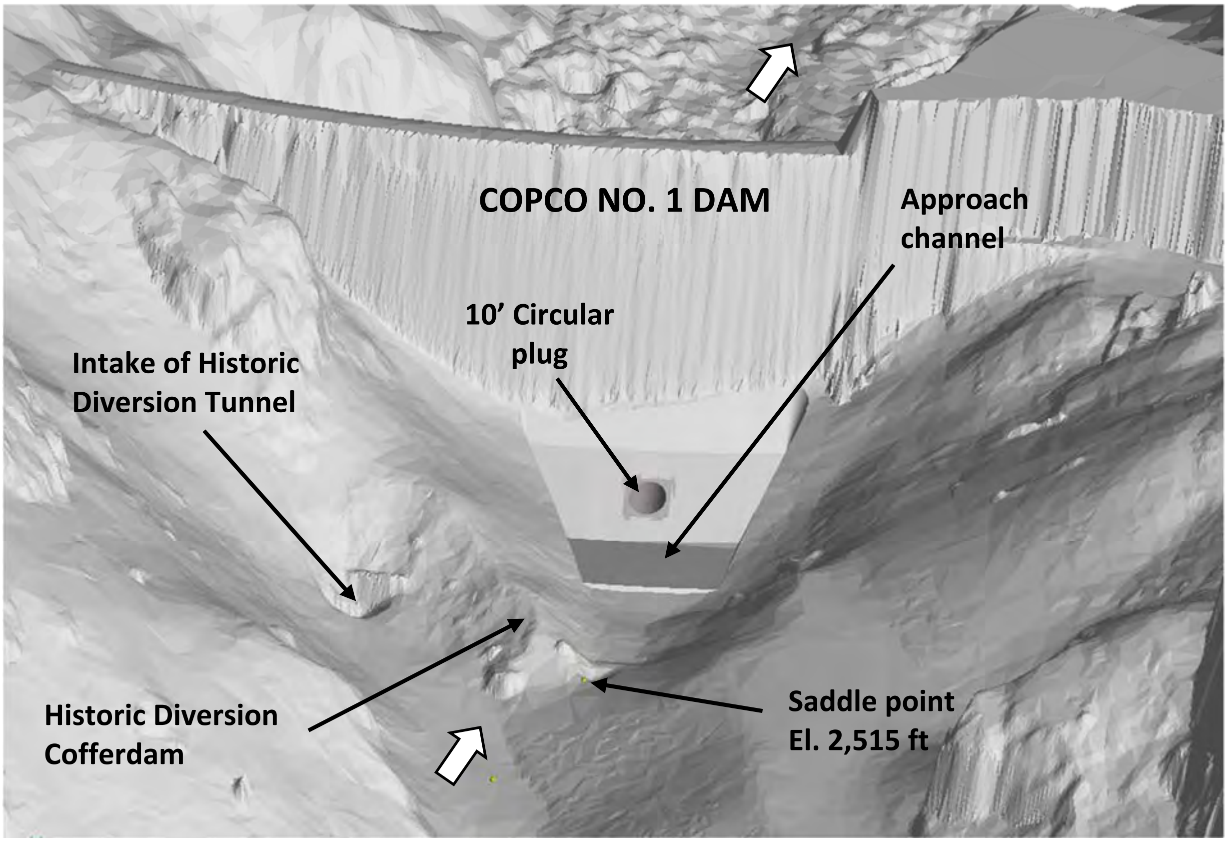

Figure 9. Historic Diversion Cofferdam located upstream of Copco No. 1 Dam (image looking downstream)

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.

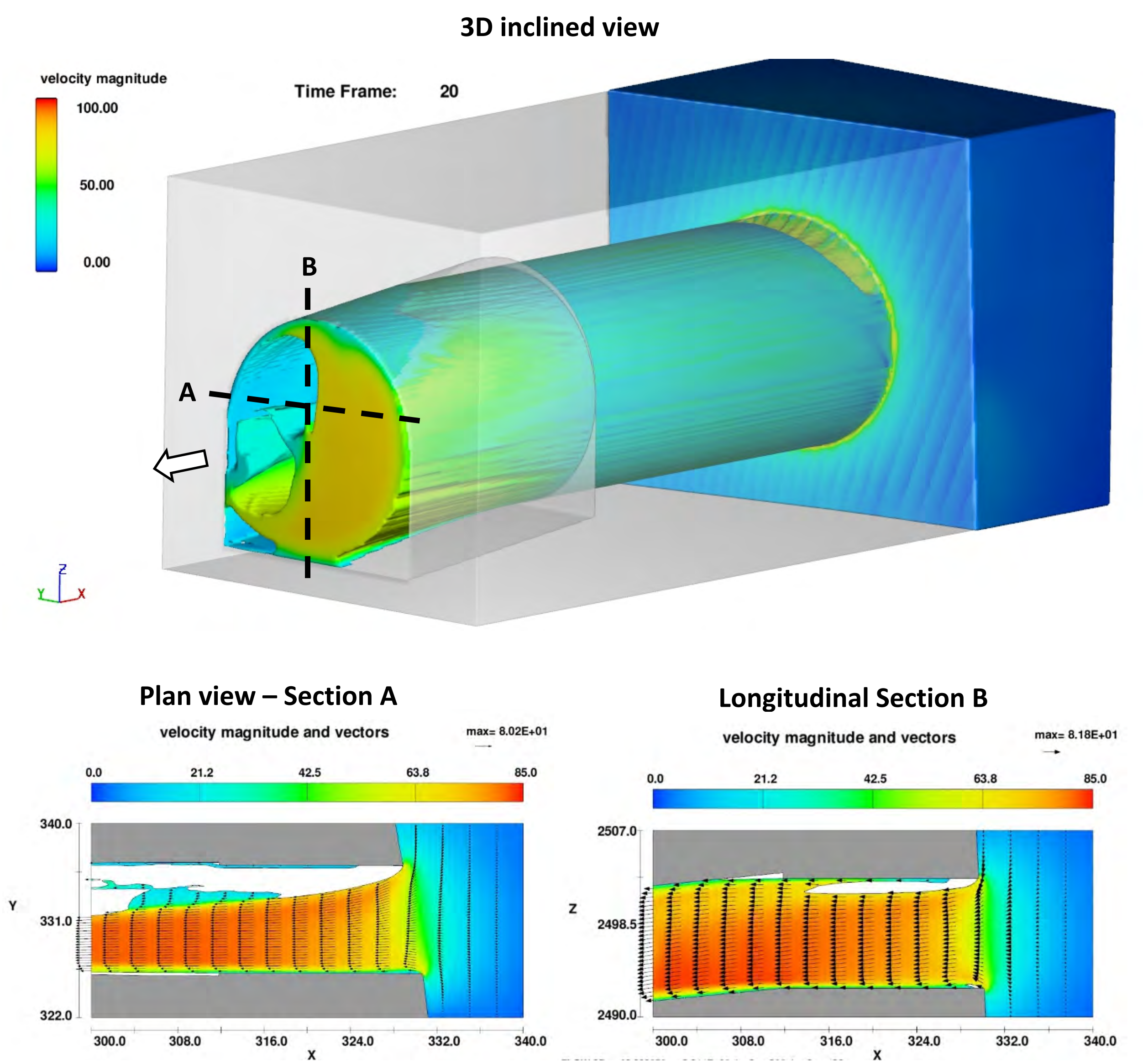

Figure 10. Close view of flow in the LLO inlet for RWS 2,597.1 ft. 3D inclined view and two sections cutting through the center of LLO's inlet orifice

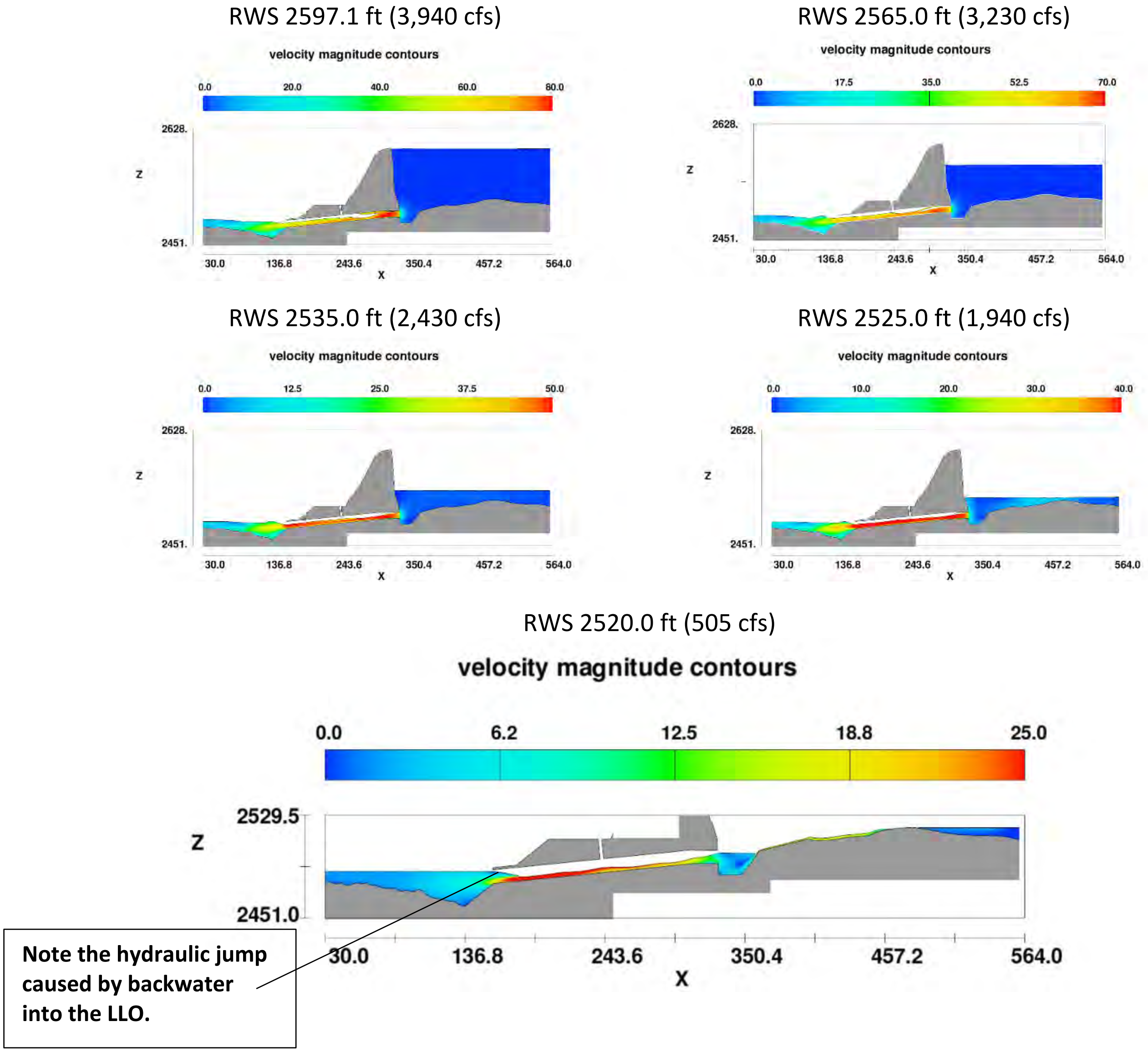

Figure 11. Longitudinal profiles along Copco No. 1 Low-Level Outlet for various reservoir water surface (RWS) elevations. Velocities presented in ft/s

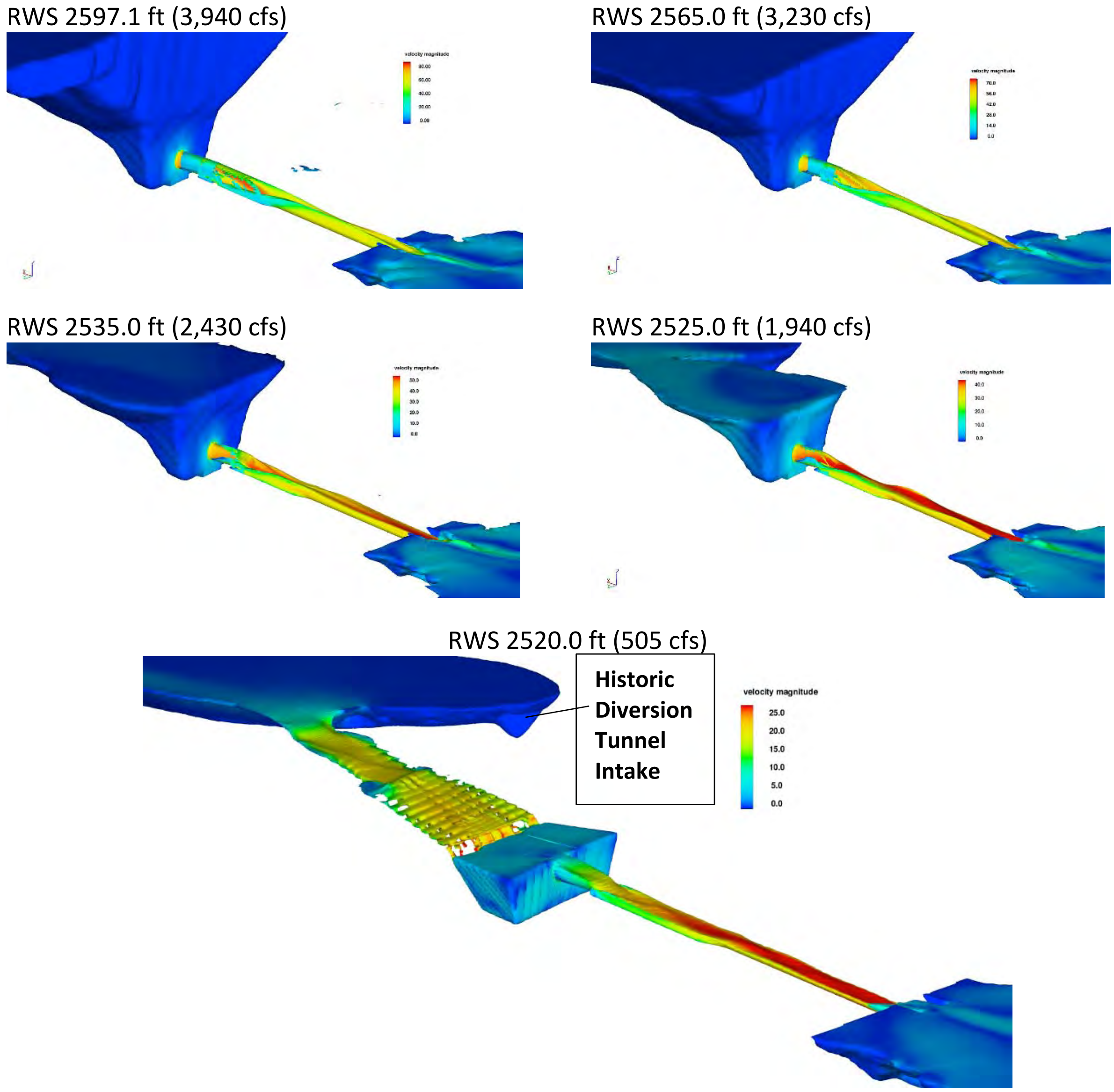

Figure 12. Three-dimensional views of Copco No. 1 Low-Level Outlet for various reservoir water surface (RWS) elevations.

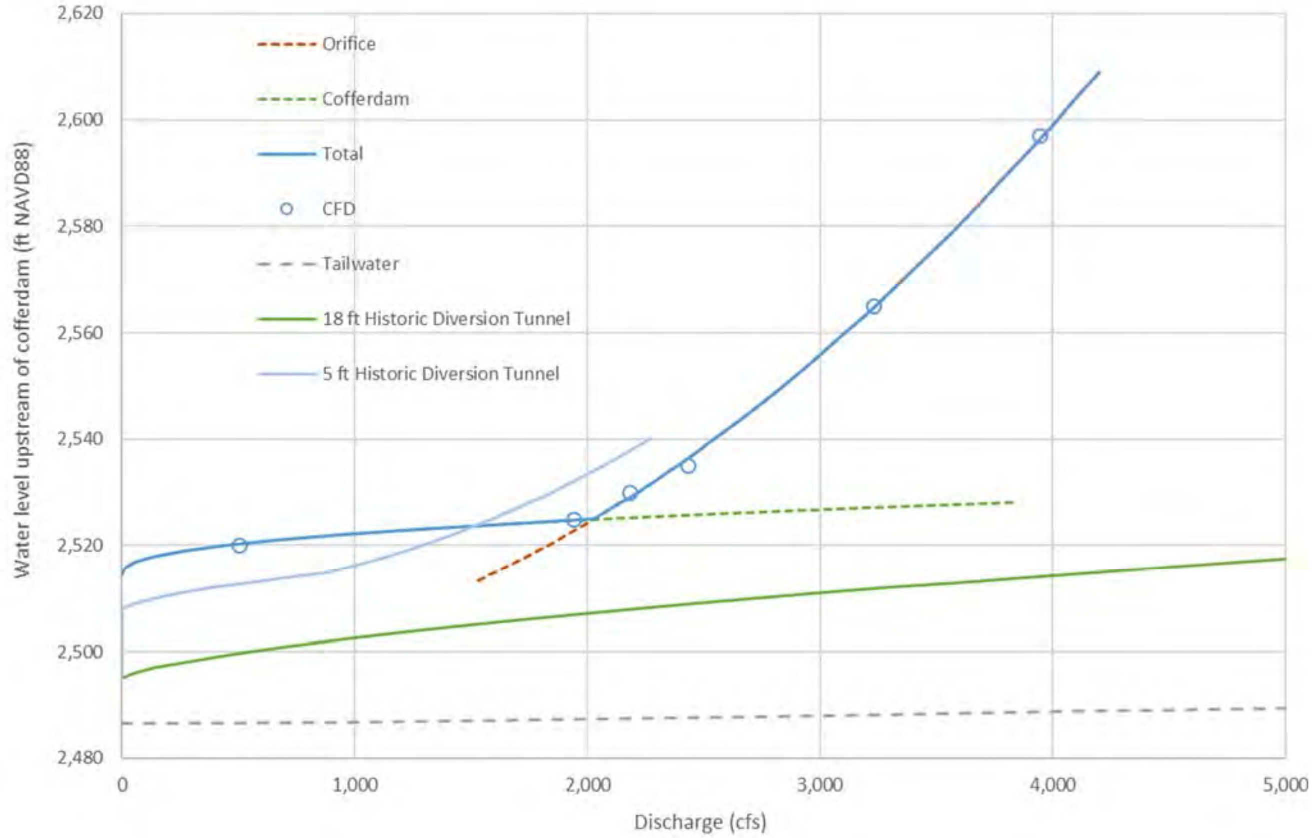

Figure 13. Rating curve of Copco No. 1 Low-Level Outlet (showing tailwater rating curve and Historic Diversion Tunnel Rating Curves for reference).

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.

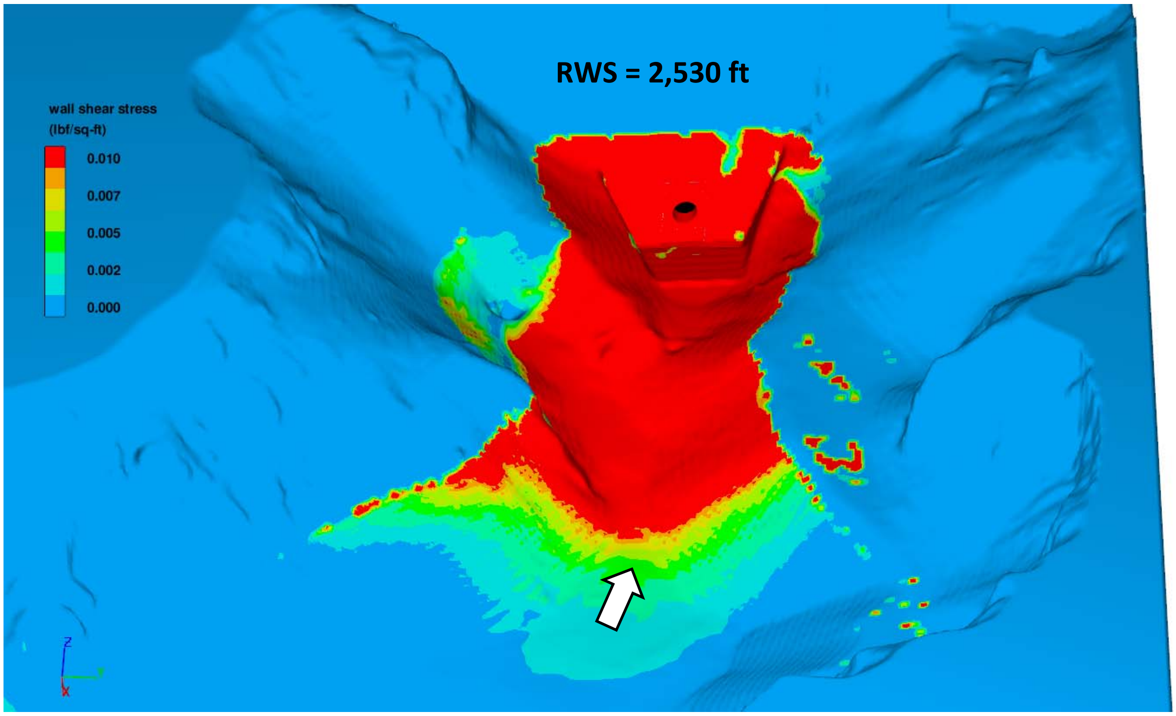

Figure 14. Bed shear stress in Copco No. 1 reservoir for RWS 2,530 ft with the LLO only (red color shading indicates areas where sand could be mobile)

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.

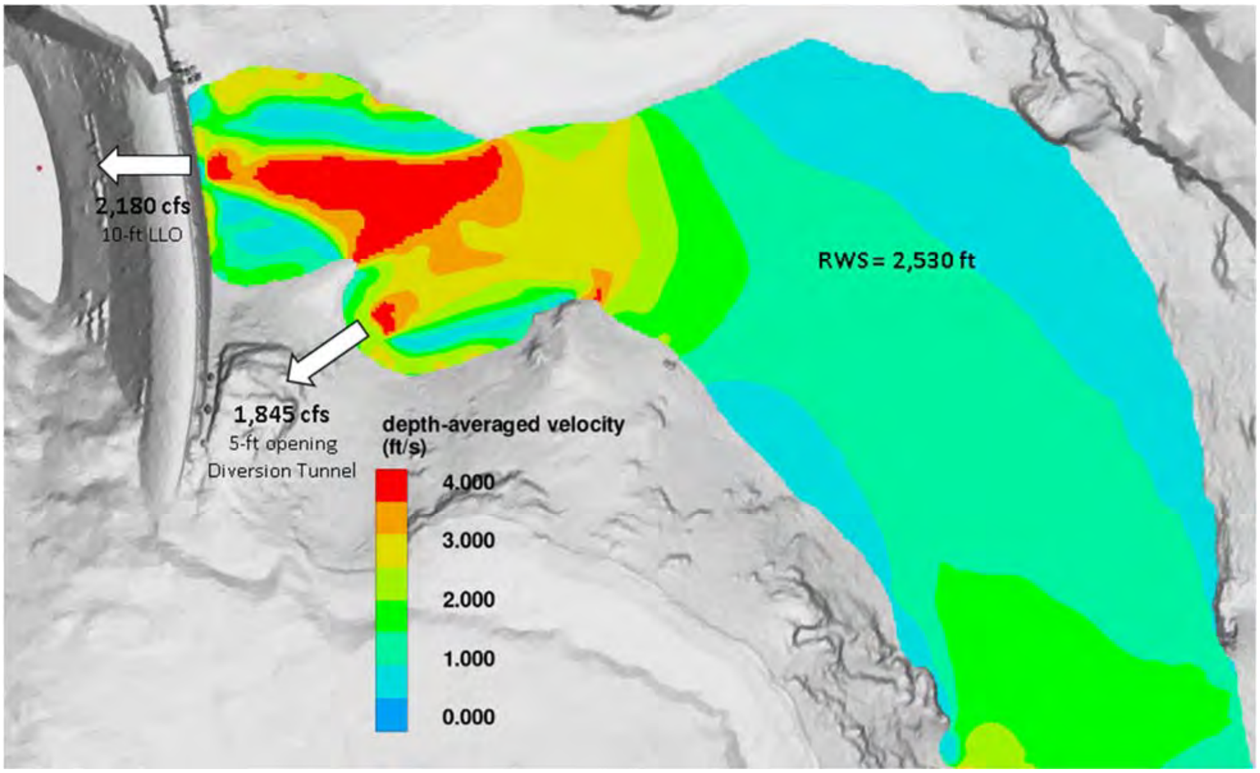

Figure 15. Depth-averaged velocity in Copco No. 1 reservoir for RWS 2,530 ft with the LLO and 5-ft opening Historic Diversion Tunnel operating.

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).

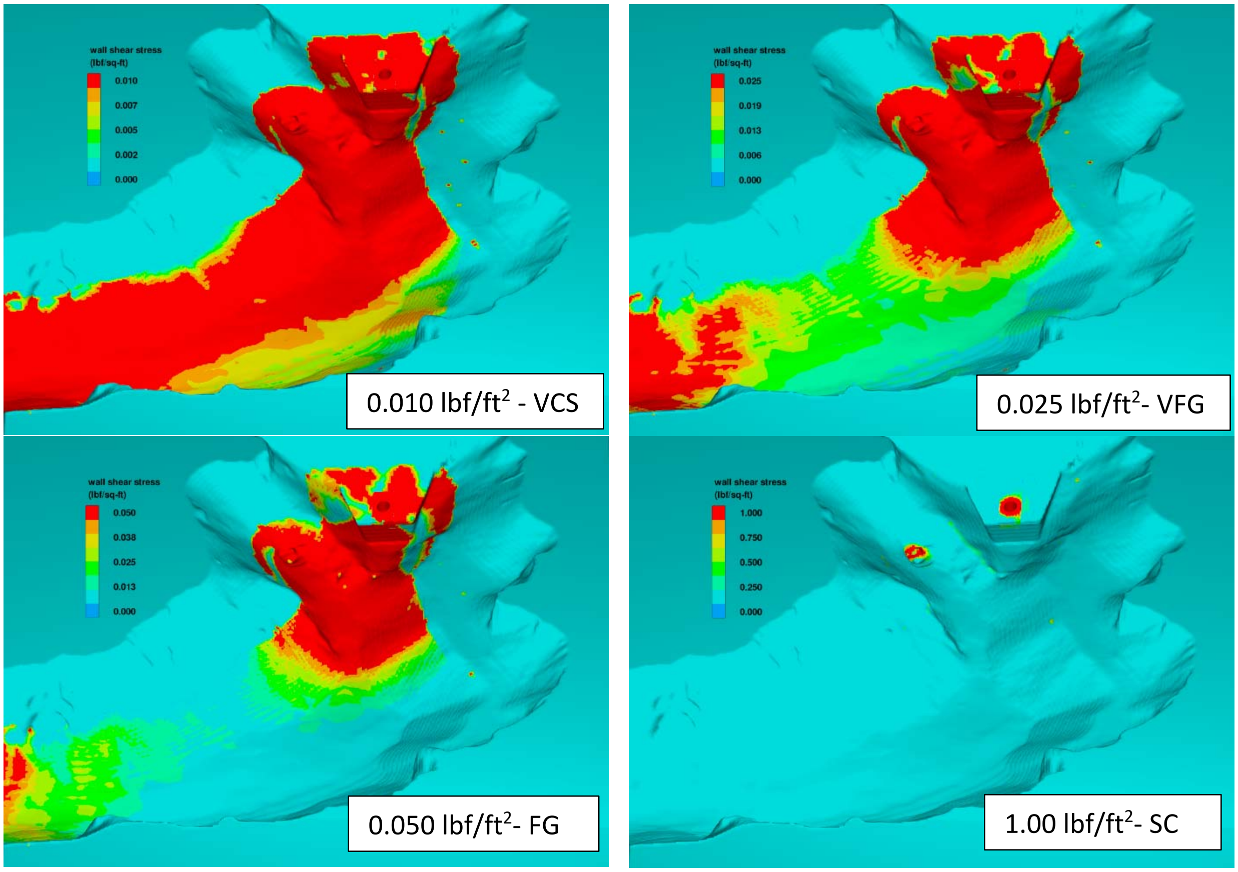

Figure 16. Bed shear stress in Copco No. 1 reservoir for RWS 2,530 ft with the LLO and 5-ft opening Historic Diversion Tunnel operating. Maximum bed shear stress in each plot was truncated at the values shown in text box to better visualize mobility of given size (see Table 4).

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

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DISCLAIMER

Report Prepared by:

Jose Vasquez, Ph.D., P.Eng.
Principal

Nancy Sims, P.Eng.
Principal

Report Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
HENRY M. FEHLMAN
42088
Exp. 3/2023
CIVIL
STATE OF CALIFORNIA

Henry Fehlman, P.E. (CA)
Principal

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	5×10^{-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.

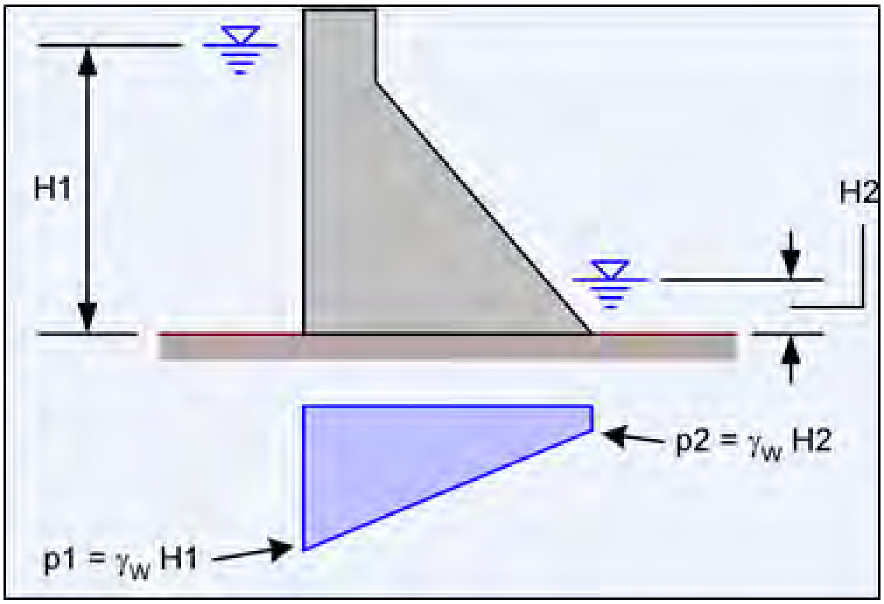


Figure 6.1 Uplift Pressure Distribution Diagram

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

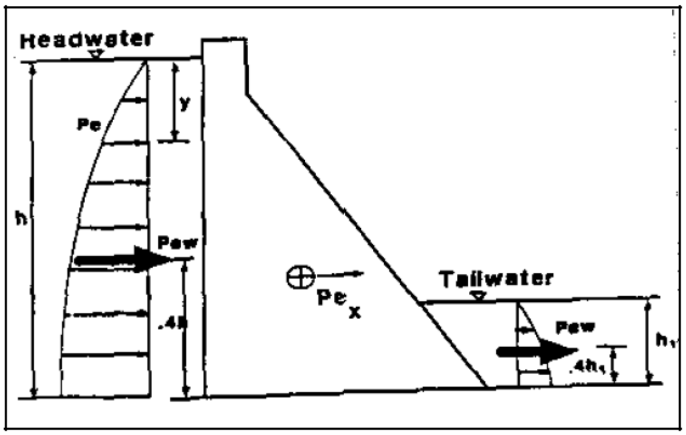
A pseudo static analysis has been completed using the traditional approach (sometimes called the seismic coefficient method of analysis) for the purpose of evaluating overall stability of the structure subject to earthquake loading.

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.

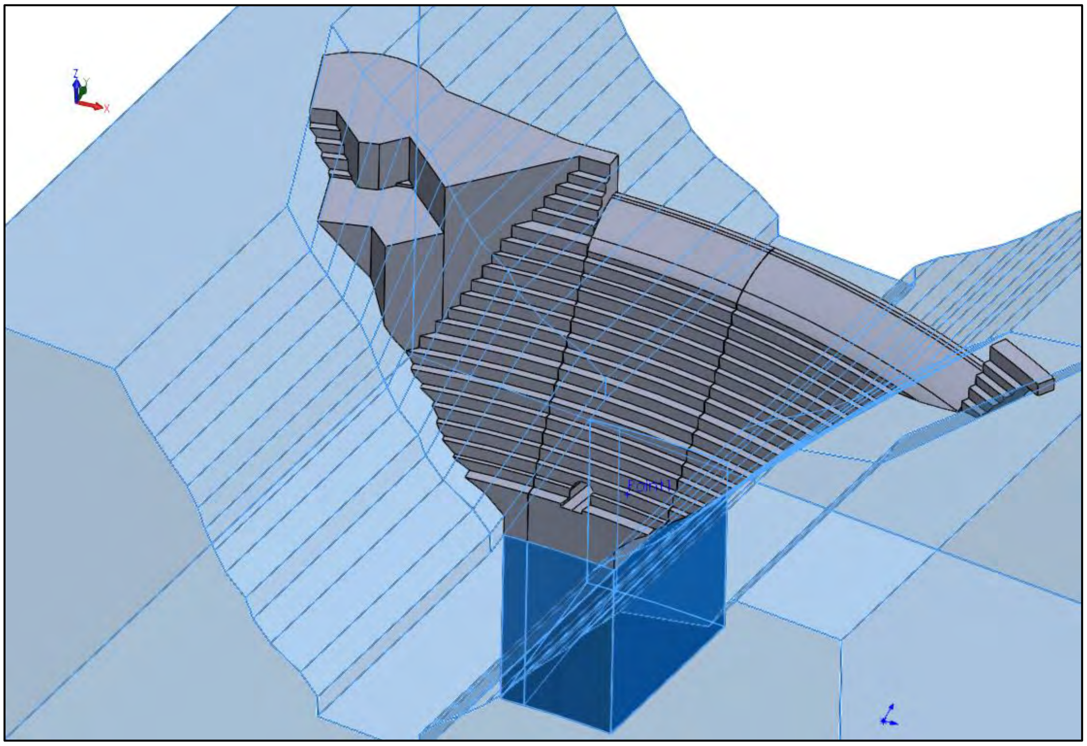


Figure 8.1 3D Model of Structure

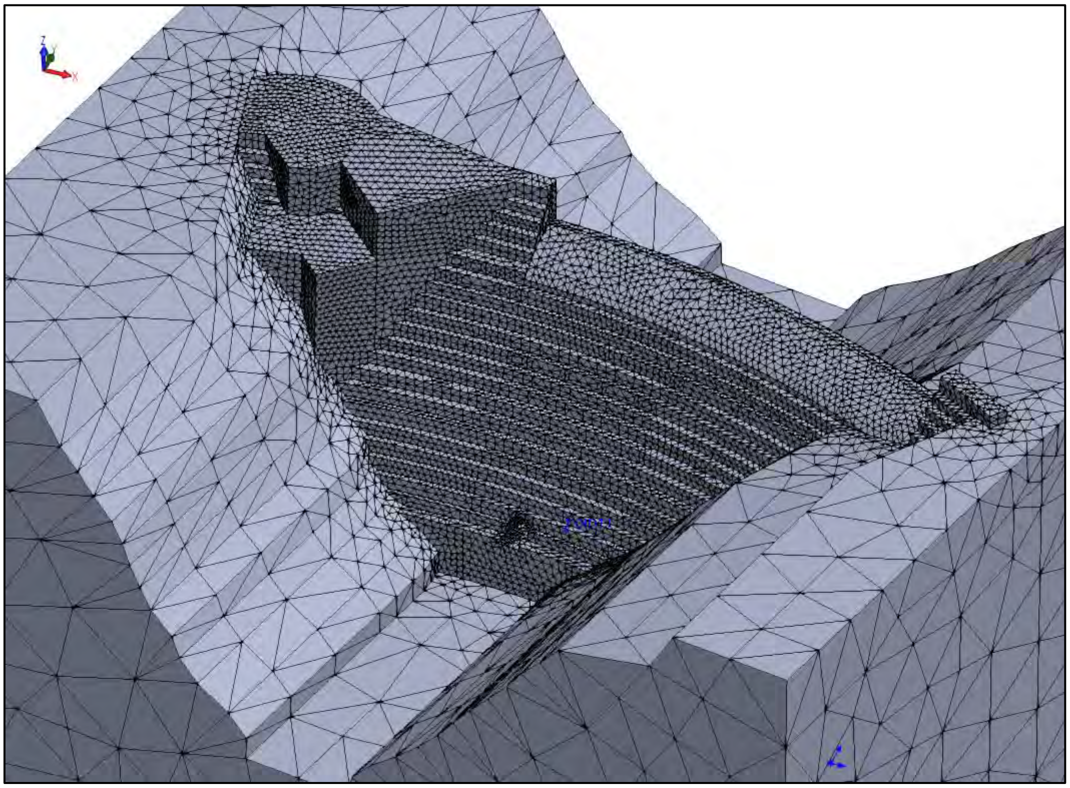


Figure 8.2 Tetrahedral Meshing of Structure

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:

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

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.

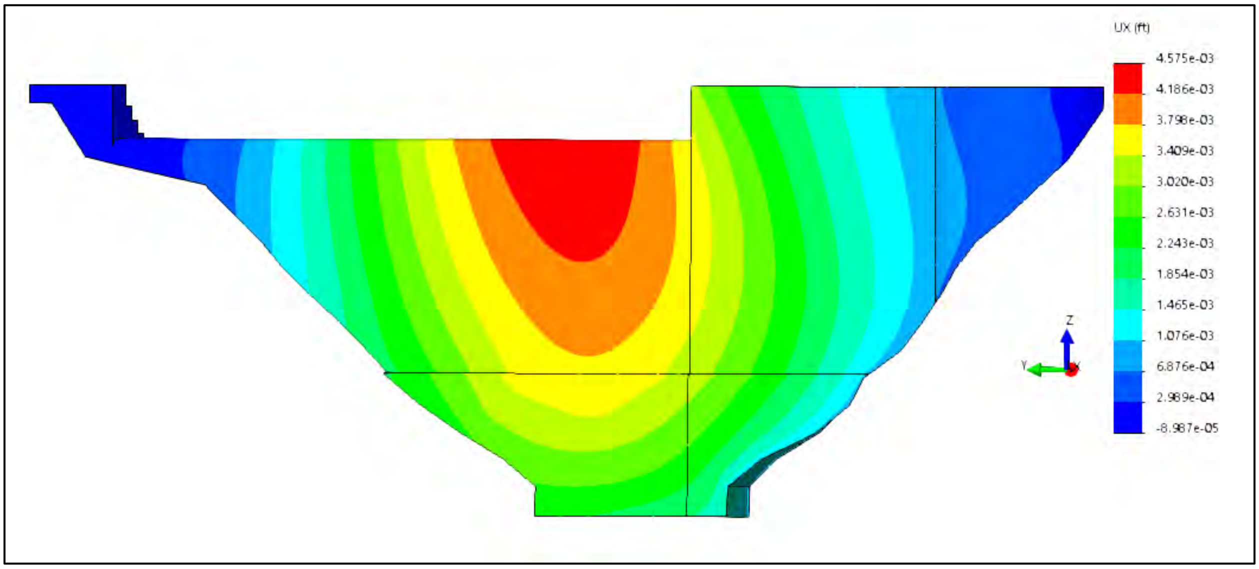


Figure 8.3 Displacement Contours, Case USLC-2

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.

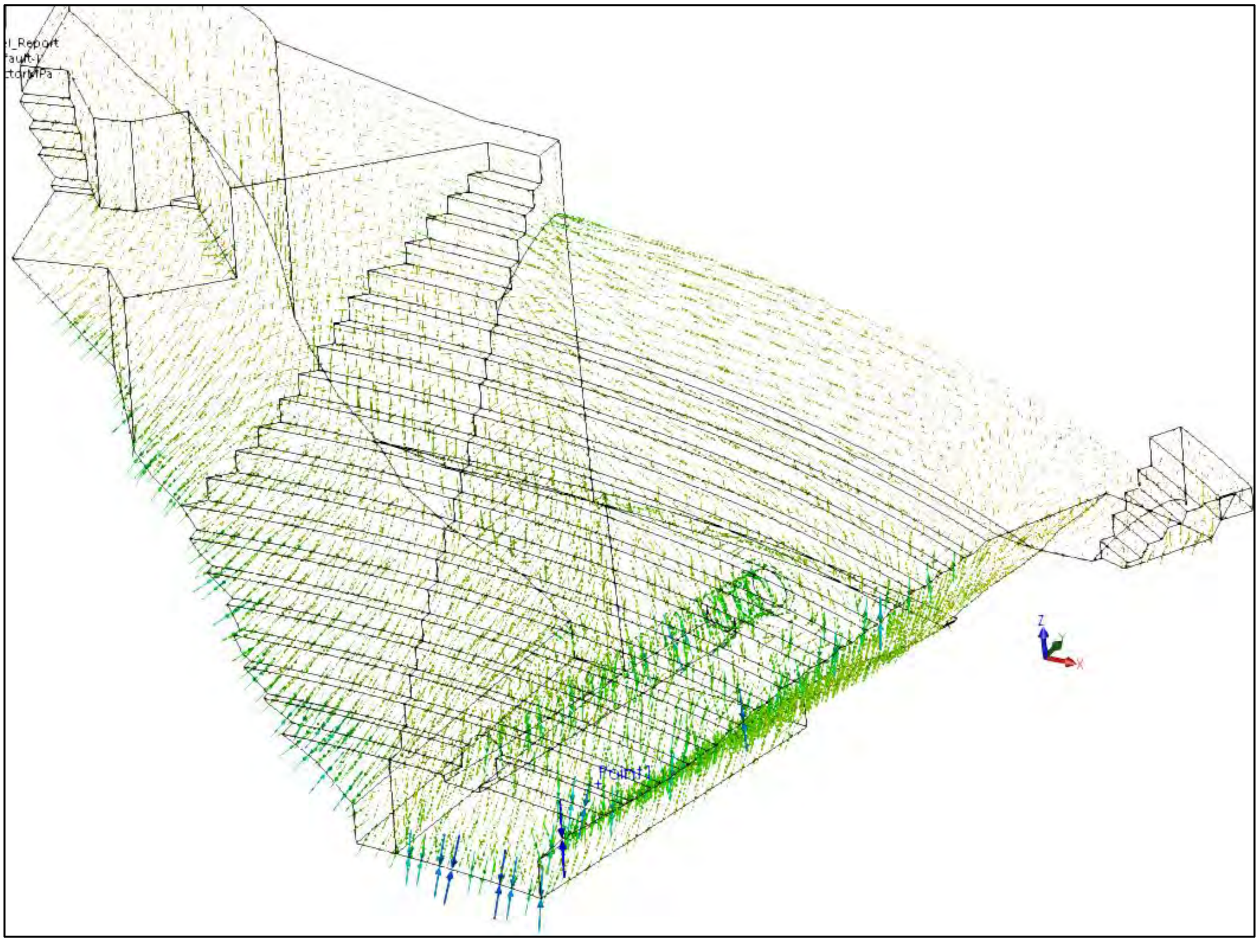


Figure 8.4 P3 Compressive Stresses, USLC-2b

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.

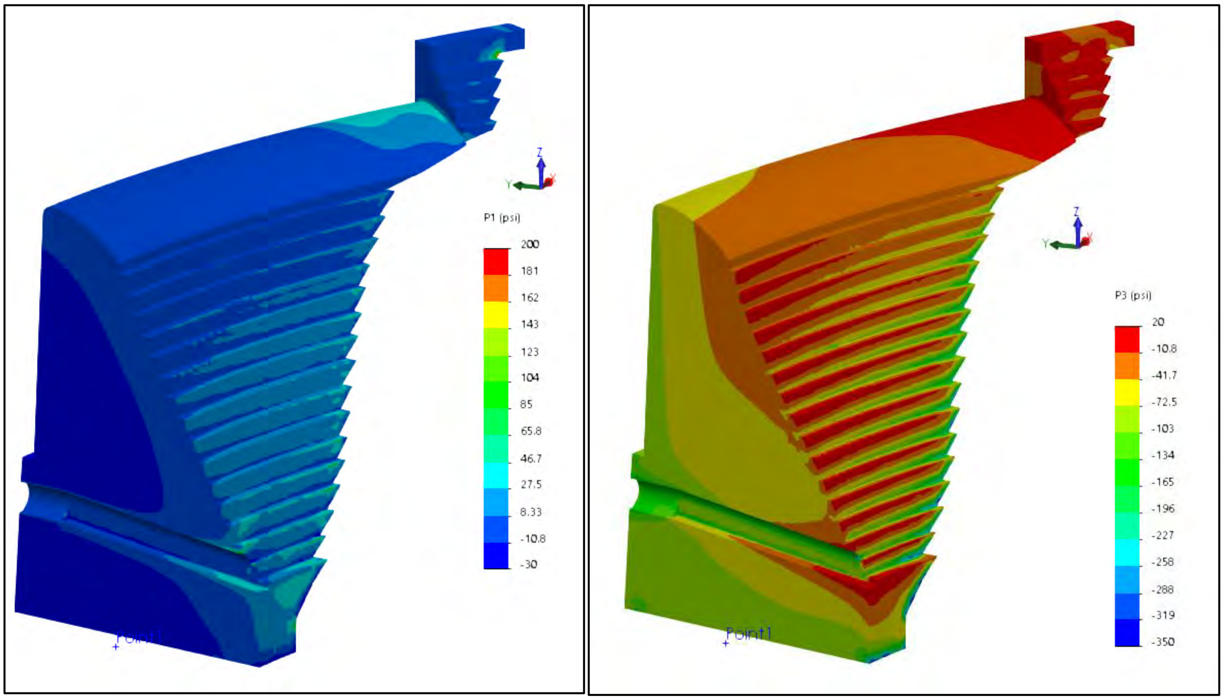


Figure 8.5 Stress Contours, Low-Level Outlet, Centerline, USLC-2b

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.

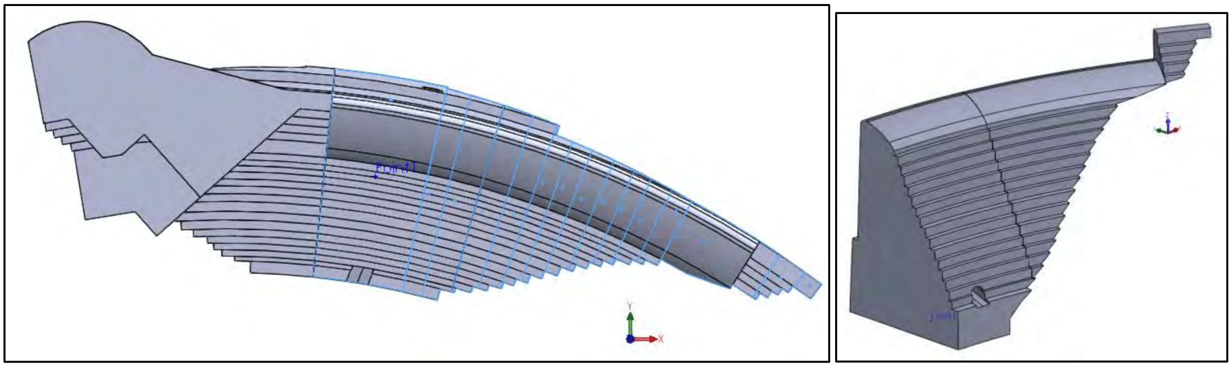


Figure 8.6 Single-Wedge Sliding Model

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

TABLE OF CONTENTS

1.0 INTRODUCTION

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

Appendix D2 provides a summary of the computational fluid dynamic (CFD) modeling, and drawdown modeling results are included as Appendix G.

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

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

2.2 DIVERSION DAM

2.2.1 RIVER CHANNEL

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

2.2.2 BACKFILL DESIGN

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

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

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

2.3 PRE-DRAWDOWN DIVERSION DAM WORKS AND CONTINGENCY REMOVAL METHOD

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

2.4 CONCRETE DIVERSION DAM REMOVAL STABILITY

2.4.1 CURRENT CONDITIONS OF UPSTREAM FACE

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

2.4.2 PRE-DRAWDOWN SPILLWAY CONCRETE PLUG

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

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

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

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

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

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

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

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

2.4.3 LEFT BANK WING WALL STABILITY

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

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

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

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

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

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

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

2.4.4 INTAKE DOWNSTREAM WALL STABILITY

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

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

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

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

2.4.5 INTAKE CONCRETE PLUG

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

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

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

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

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

2.5 SPILLWAY APRON TEMPORARY WORK PLATFORM

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

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

2.6 EROSION PROTECTION

2.6.1 SPILLWAY APRON

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

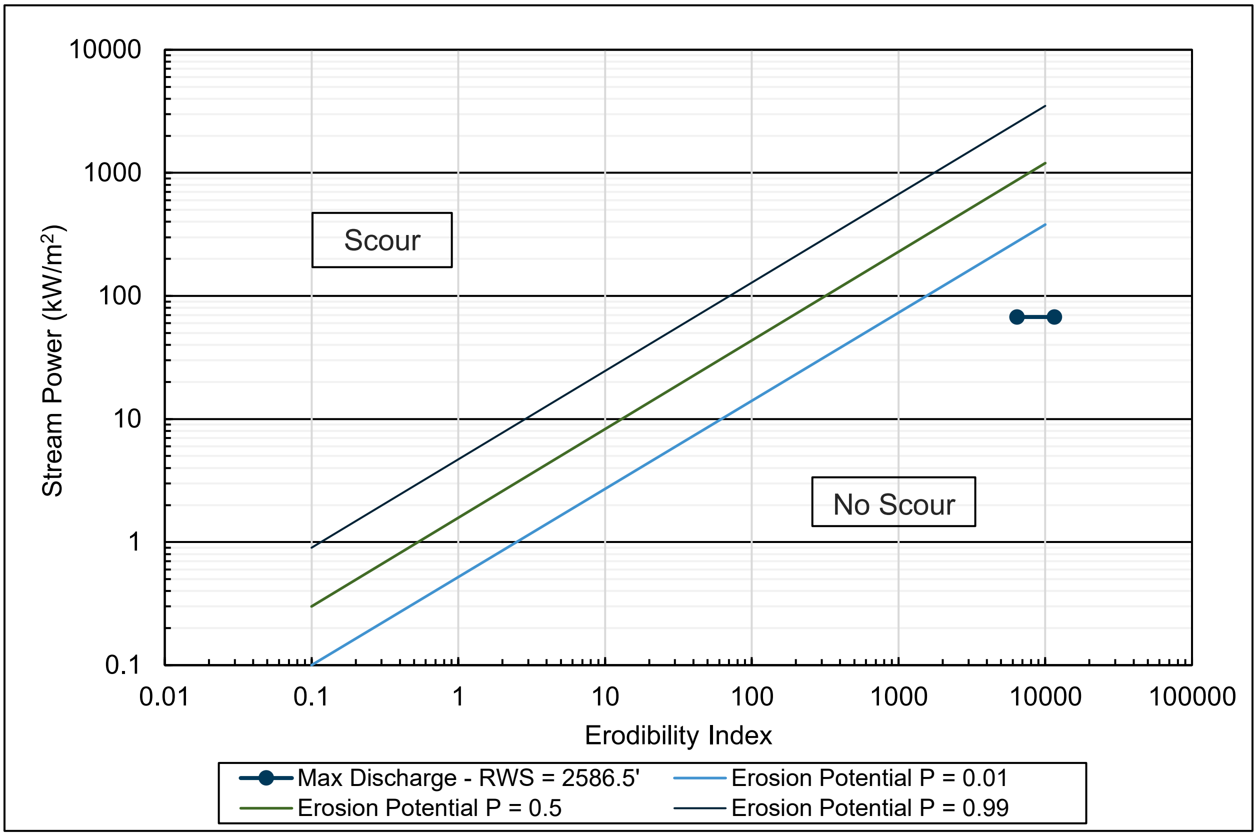

Figure 2.1 Probability of Erosion of the Spillway Apron

2.6.2 INTAKE STRUCTURE BACKFILL

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

2.6.3 FINAL CHANNEL SLOPE EROSION PROTECTION

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

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

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

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

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

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

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

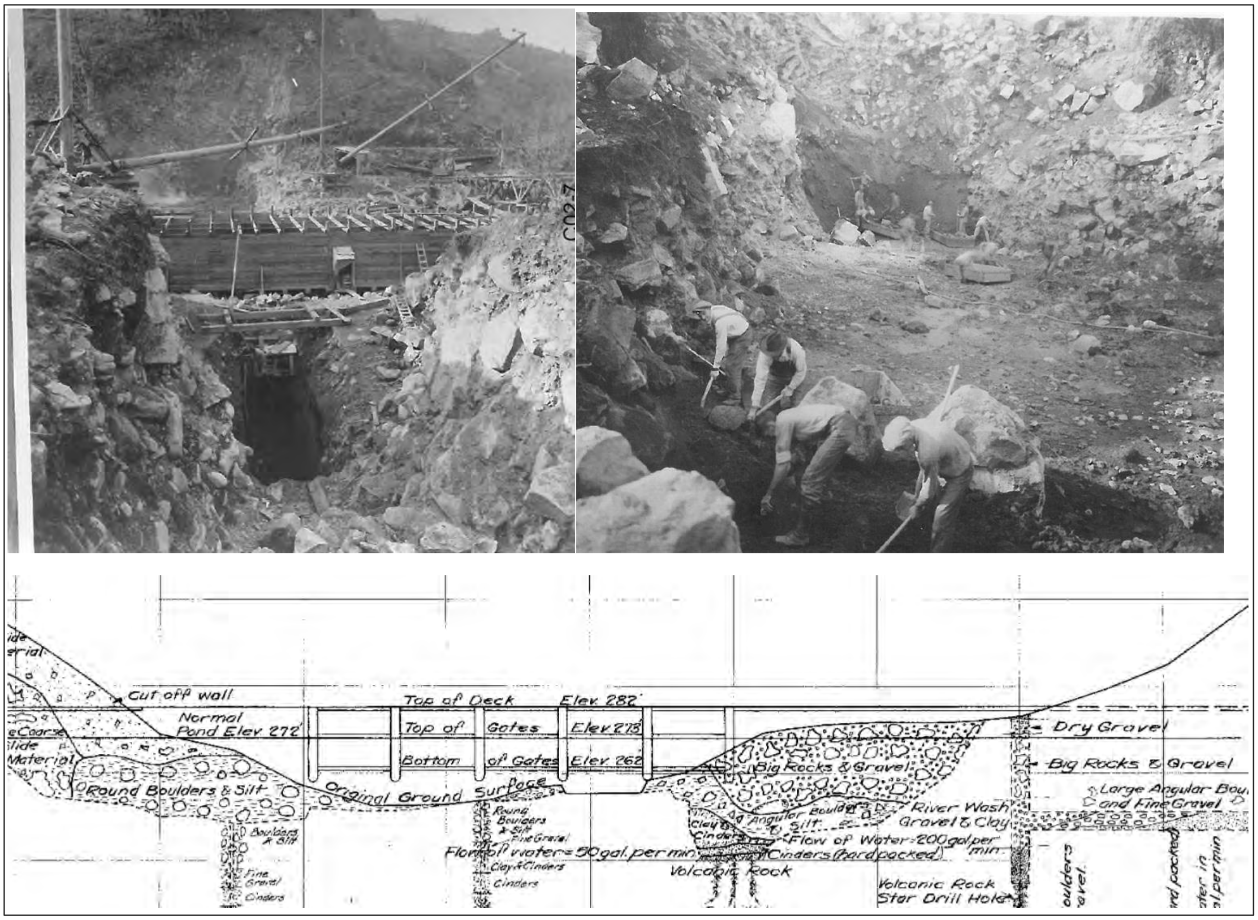

Figure 2.2 Diversion Dam Excavation (Top Left and Right), Historic Geological Interpretation along Centerline of Dam

2.6.4 POWERHOUSE TAILRACE BACKFILL

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

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

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

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

The tailrace backfill is shown on Drawing C3420.

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

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

Table 3.1 Reservoir Storage Capacity for Various Key Elevations

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

3.2 OUTLET STRUCTURE RATING CURVES

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

3.3 RESERVOIR SEQUENCING

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

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

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

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

Reservoir water surface levels are simulated in the drawdown model (Appendix G) for the full record of inflows available for the 2019 Biological Opinion (2019 BiOp) dataset. The 2019 BiOp flows are available for 36 years, from October 1980 through September 2016. The drawdown model only considers the diversion dam contingency removal method configuration where Spillway Bay No. 1 is removed to the spillway apron. The results of the drawdown model are summarized in three ways:

- Individual year simulations are provided in the attached Copco No. 2 Simulated Drawdown Figures 1 through 36. These plots indicate the following:
  - Reservoir water surface levels.
  - Daily average inflows, total outflows, and outflows for each outlet structure (i.e., spillway and power intake).
- Maximum daily reservoir water surface level daily non-exceedance percentiles (percentiles) are shown on Figure 3.1, and on Drawing C3057. This figure represents the results from all 36 model simulations as non-exceedance percentiles of reservoir water surface levels to summarize the distribution of the results on any given day of the simulations. These results to not represent a single simulation and are based on all model simulations.
- Ensemble figures with each line representing a single model simulation for a different year, (also referred to as spaghetti figures) are shown on Figure 3.2. This figure overlaps the simulated reservoir water surface levels on a common x-axis that spans January 1 to September 30. Each line represents a single model simulation.

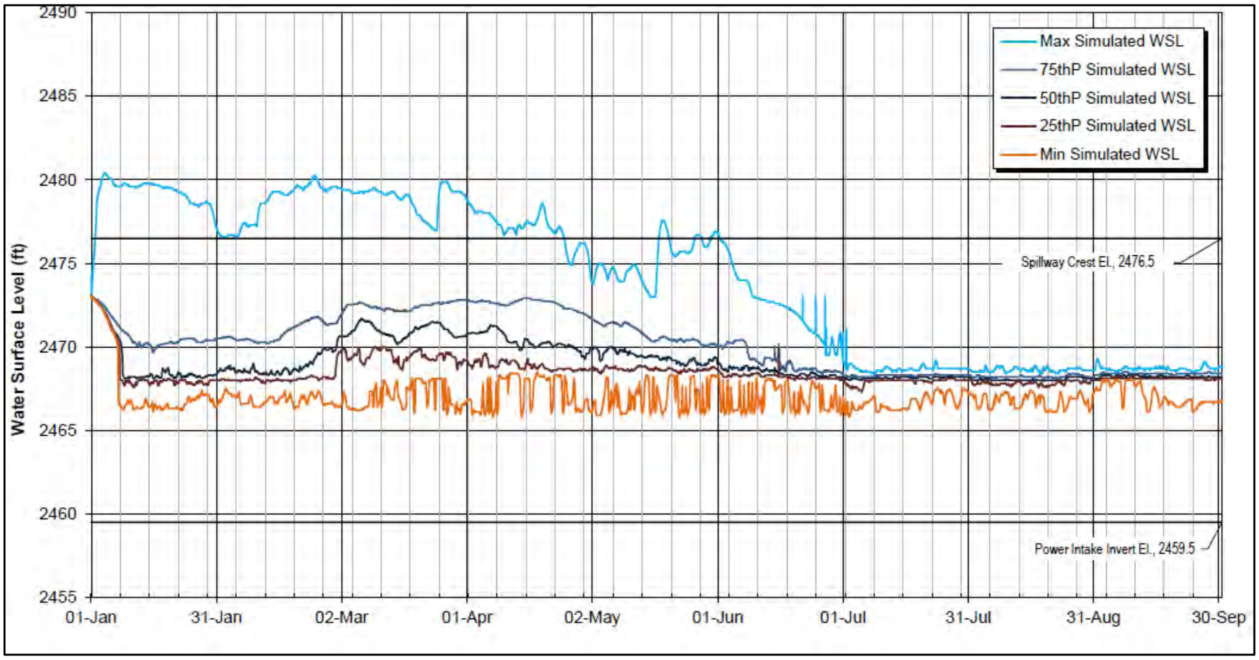

Figure 3.1 Copco No. 2 Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

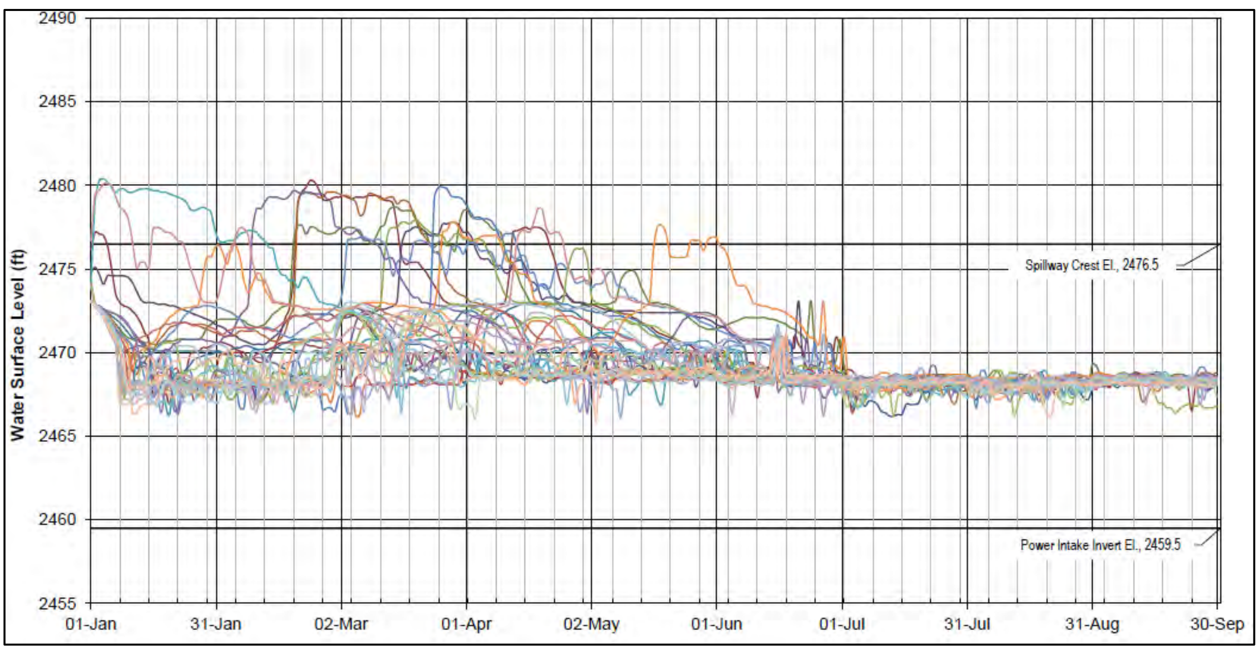

Figure 3.2 Copco No. 2 Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

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

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

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

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

- Elevation 2,476.5 ft – represents the spillway crest. Approximately 70% of the drawdown simulations have reservoir water levels sustained below the spillway crest by January 2, 80% of the simulations by April 1, 98% of the simulations by May 1, and 100% of the simulations by June 1. Spillway overtopping and flooding of the removal work area along the work platform is not likely to occur following June 1 based on average daily conditions for the 36 simulated model years.

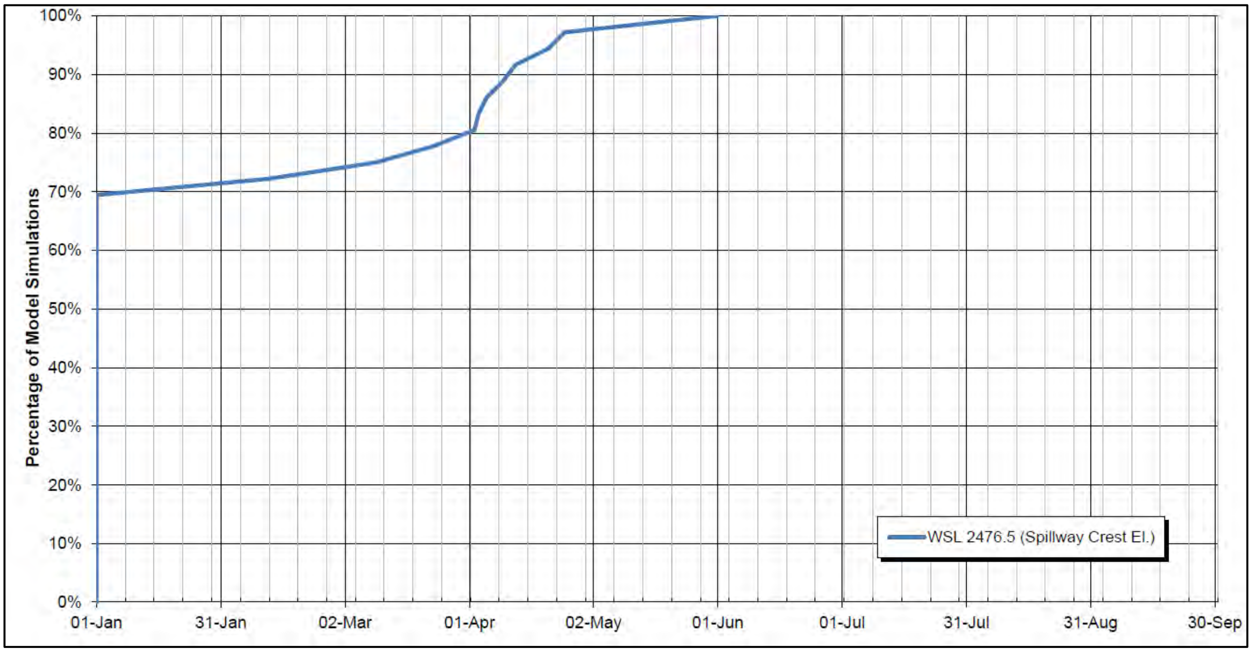

Figure 3.3 Copco No. 2 Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below the Spillway Crest Elevation

3.4 SCOUR POTENTIAL FOR CONTINGENCY REMOVAL METHOD

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

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

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

3.5 STEADY-STATE WATER SURFACE LEVELS

3.5.1 CONTINGENCY REMOVAL METHOD WORK PLATFORM WATER SURFACE LEVELS

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

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

Flood water surface levels at the dam and tailwater levels are shown on Drawing C3057 for steady-state inflows. The statistical flood flows (high water) are based on peak instantaneous flows outlined in Appendix A6, while the daily average flows are average flows over a 24-hour period. The flood flows assume that the J.C. Boyle and Copco No. 1 facilities provide flow attenuation from January through June 15 of the drawdown year. Once river diversion has been achieved at these facilities, the flood flows will no longer be attenuated, therefore, no flow attenuation is assumed for the flood flows between June 16 and December of the drawdown year, as discussed in Appendix A6. The levels are calculated using the discharge rating curves developed for the outlet structures and the average tailwater levels downstream of Spillway Bays No. 1 through 5 as detailed in Appendix D2 and shown on Drawing C3057.

3.5.3 HISTORIC DIVERSION DAM WATER LEVELS

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

3.5.4 TUNNEL NO. 1 STAGE-DISCHARGE CURVE

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

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

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

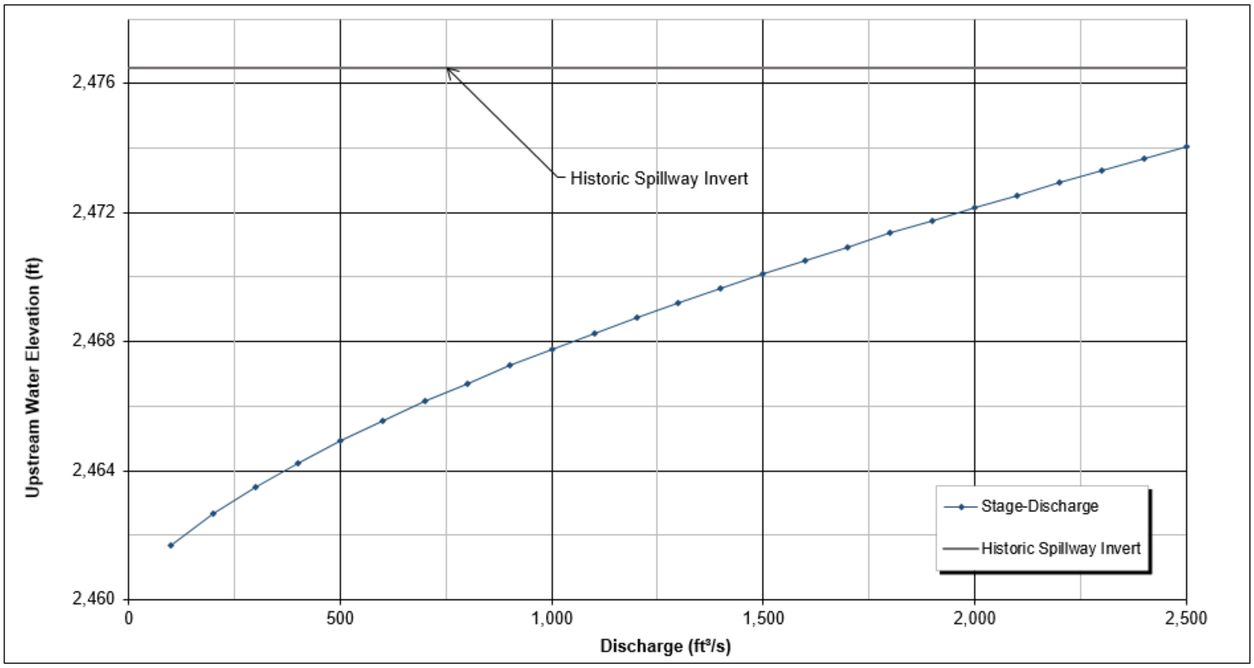

Figure 3.4 Tunnel No. 1 Stage-Discharge Curve at Spillway Bay No. 3

3.6 FINISHED GRADE HYDRAULIC CONDITIONS

3.6.1 FINAL RIVER CHANNEL WATER LEVELS

Channel characteristics and geometry of the Copco No. 2 final river channel presented on Drawing C3234 are used to develop a hydrodynamic model to determine the discharge-stage relationship post-dam removal. The model uses HEC-RAS 2D with a Manning's n of 0.04.

The resulting stage-discharge relationship is shown on Figure 3.5 at the location of the current dam centerline. A sensitivity of the model uses Manning's n roughness of 0.03 and 0.06 to account for potential variability in roughness elements added to the channel to provide localized habitat. The results of the sensitivity analysis are included on Figure 3.5.

Dam removal construction activities in the vicinity of the final river channel may continue to occur into the fall. Steady-state water surface levels for probable floods and mean monthly flows for specified periods in September through November, are provided for reference in Table 3.2 using the base model Manning's n value of 0.04.

In addition, steady-state water surface levels for the final river channel for the annual probable floods, the mean annual flow, and the annual 25% and 75% flow durations are provided in Table 3.3 using the base model Manning's n value of 0.04.

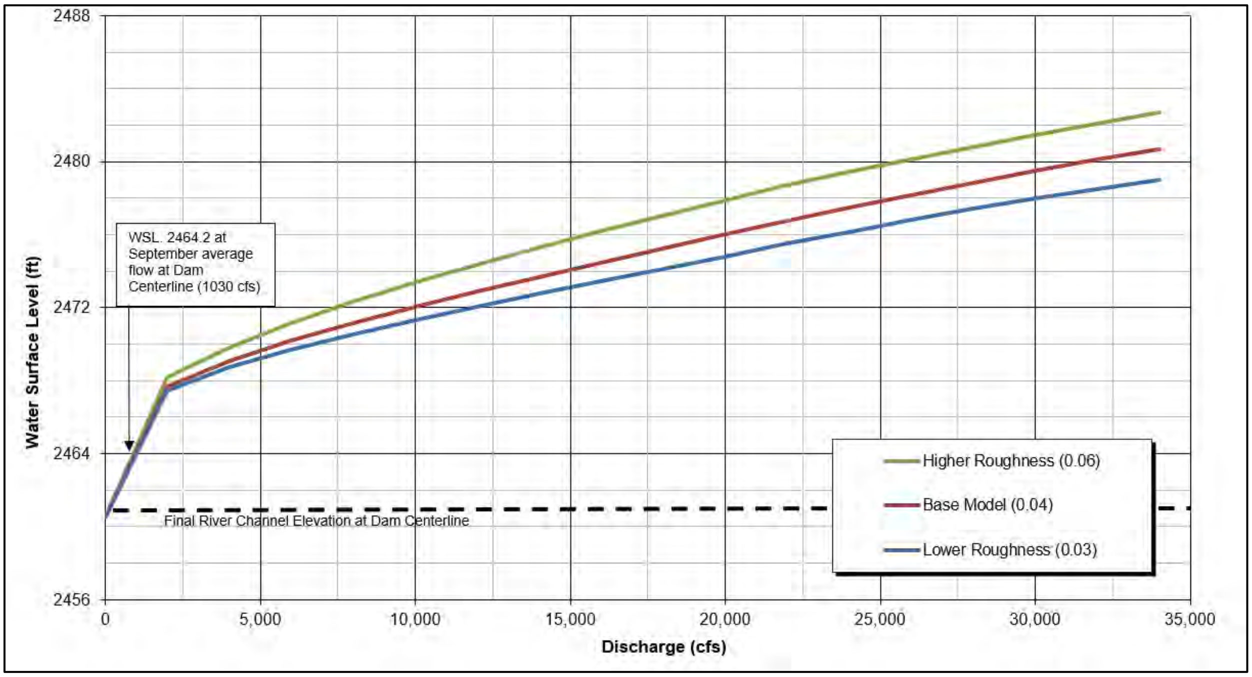

Figure 3.5 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

Table 3.2 Final River Channel Monthly Steady-State Water Surface Levels at Existing Dam Centerline

Flow Condition		Discharge (cfs)					
		Time Period					
		Sep 1 – 15	Sep 16 – 30	Oct 1 – 15	Oct 16 – 31	Nov 1 -15	Nov 16 -30
Statistical High Water (Flood Conditions)	5% Probable Flood	1,900	2,100	5,700	6,300	7,400	8,400
	20% Probable Flood	1,600	1,600	4,400	4,700	5,100	5,800
	50% Probable Flood	1,300	1,300	3,600	3,600	3,900	4,200
Mean Monthly Flow for Time Period		1,030	1,030	1,050	1,140	1,230	1,240
Flow Condition		**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline**					
Statistical High Water (Flood Conditions)	5% Probable Flood	2,467.3	2,467.8	2,470.0	2,470.3	2,470.9	2,471.3
	20% Probable Flood	2,466.3	2,466.3	2,469.3	2,469.5	2,469.7	2,470.1
	50% Probable Flood	2,465.2	2,465.2	2,468.8	2,468.8	2,469.0	2,469.2
Mean Monthly Flow for Time Period		2,464.2	2,464.2	2,464.2	2,464.3	2,464.6	2,465.0

NOTES:

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

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

Flow Condition		Discharge with Attenuation from Upstream Facilities (cfs)	Discharge with No Attenuation from Upstream Facilities (cfs)
Statistical High Water (Flood Conditions)	1% Probable Flood	29,400	32,700
	5% Probable Flood	18,200	24,300
	20% Probable Flood	10,300	15,400
	50% Probable Flood	7,100	11,200
Annual Flow Duration 25% of Time Equaled or Exceeded		1,780	1,780
Mean Annual Flow		1,710	1,710
Annual Flow Duration 75% of Time Equaled or Exceeded		940	940
Flow Condition		**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline**	**Water Surface Levels (ft) - Post-Dam Removal at Dam Centerline**
Statistical High Water (Flood Conditions)	1% Probable Flood	2,479.3	2,480.3
	5% Probable Flood	2,475.3	2,477.6
	20% Probable Flood	2,472.2	2,474.2
	50% Probable Flood	2,470.7	2,472.6
Annual Flow Duration 25% of Time Equaled or Exceeded		2,466.7	2,466.9
Mean Annual Flow		2,466.4	2,466.7
Annual Flow Duration 75% of Time Equaled or Exceeded		2,463.8	2,463.9

NOTES:

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

3.6.2 BACKFILLED POWERHOUSE AND TAILRACE CROSS-SECTION HYDRAULICS

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

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

Table 3.4 Water Surface Elevation and Velocity at Backfilled Powerhouse Location

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

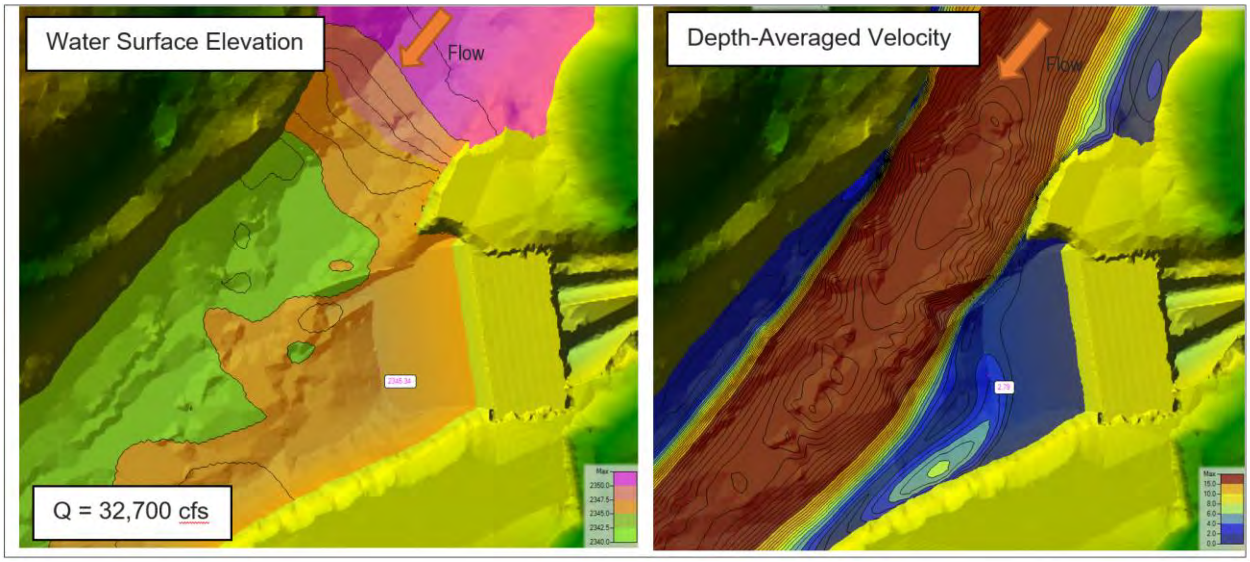

Figure 3.6 Water Surface Elevation and Velocity at Backfilled Powerhouse Location for the 1% Probable Flood Flow Condition

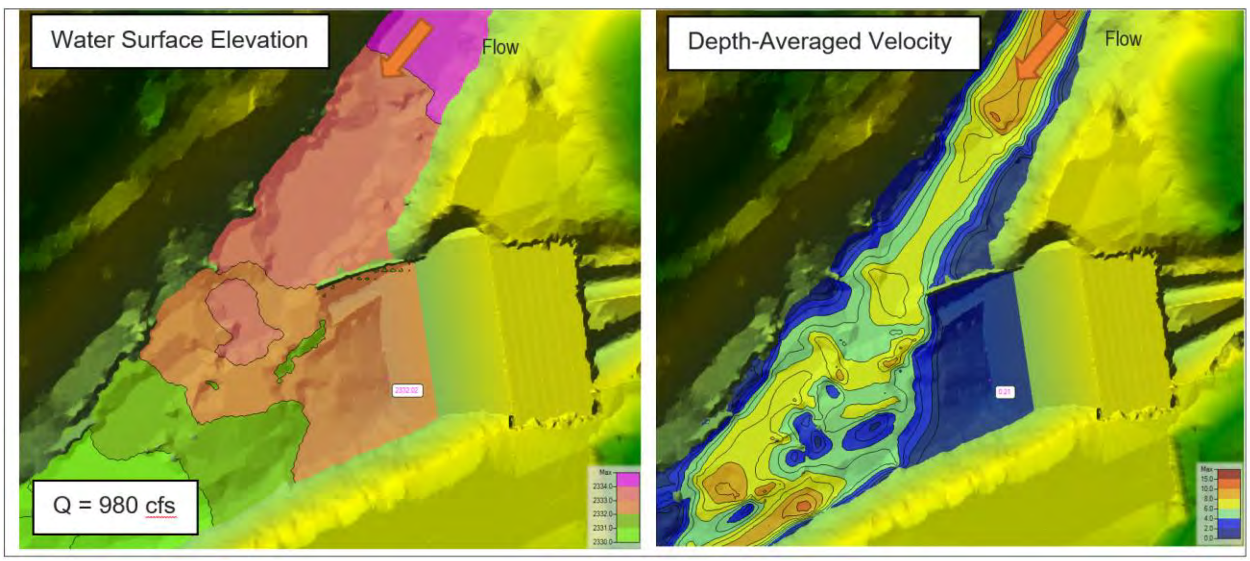

Figure 3.7 Water Surface Elevation and Velocity at Backfilled Powerhouse Location for the Average August Flow Condition

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

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

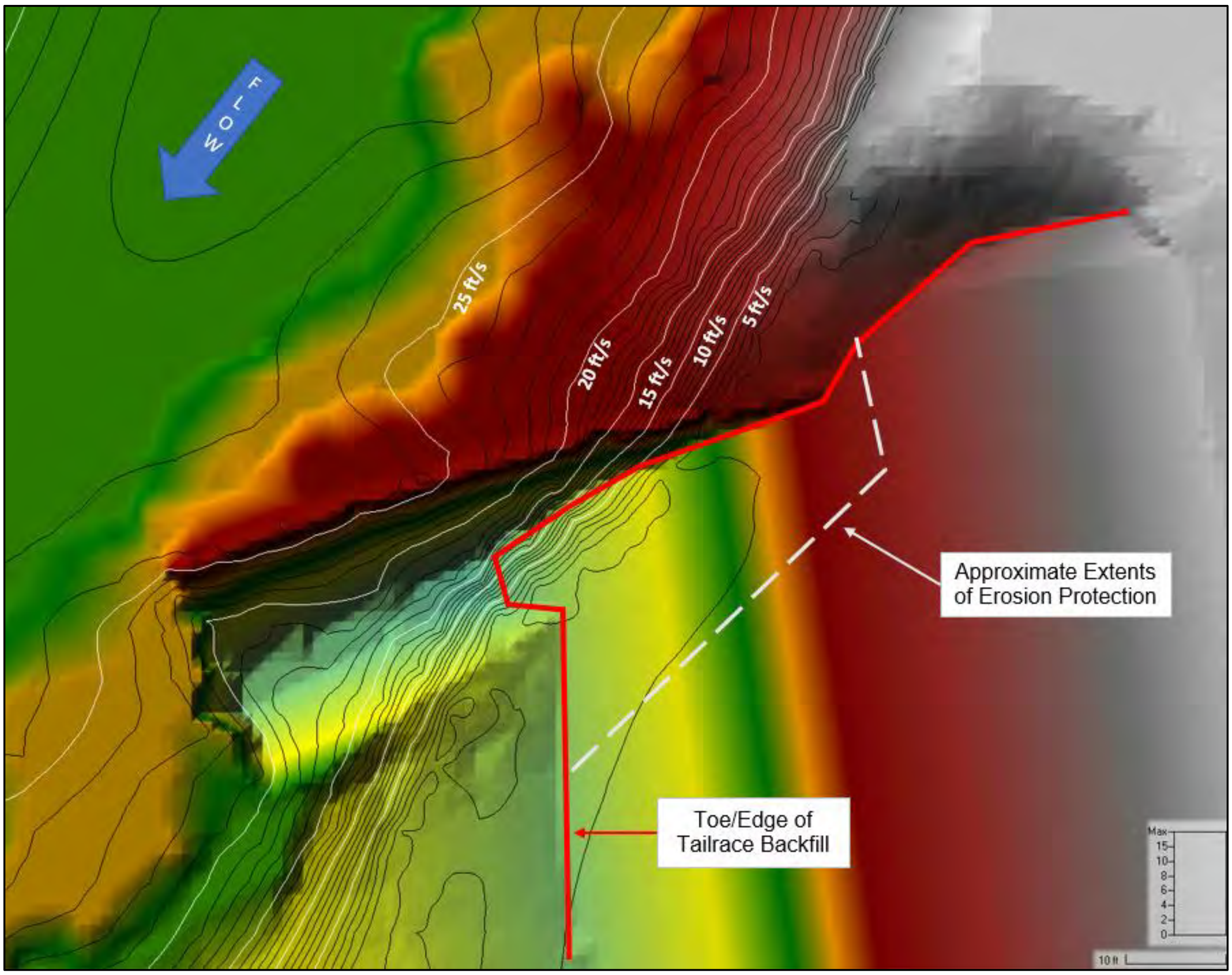

Figure 3.8 Velocity Contours for the 1% Probable Flood Flow Condition at the Tailrace Backfill Area

4.0 GEOTECHNICAL

4.1 DAM REMOVAL

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

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

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

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

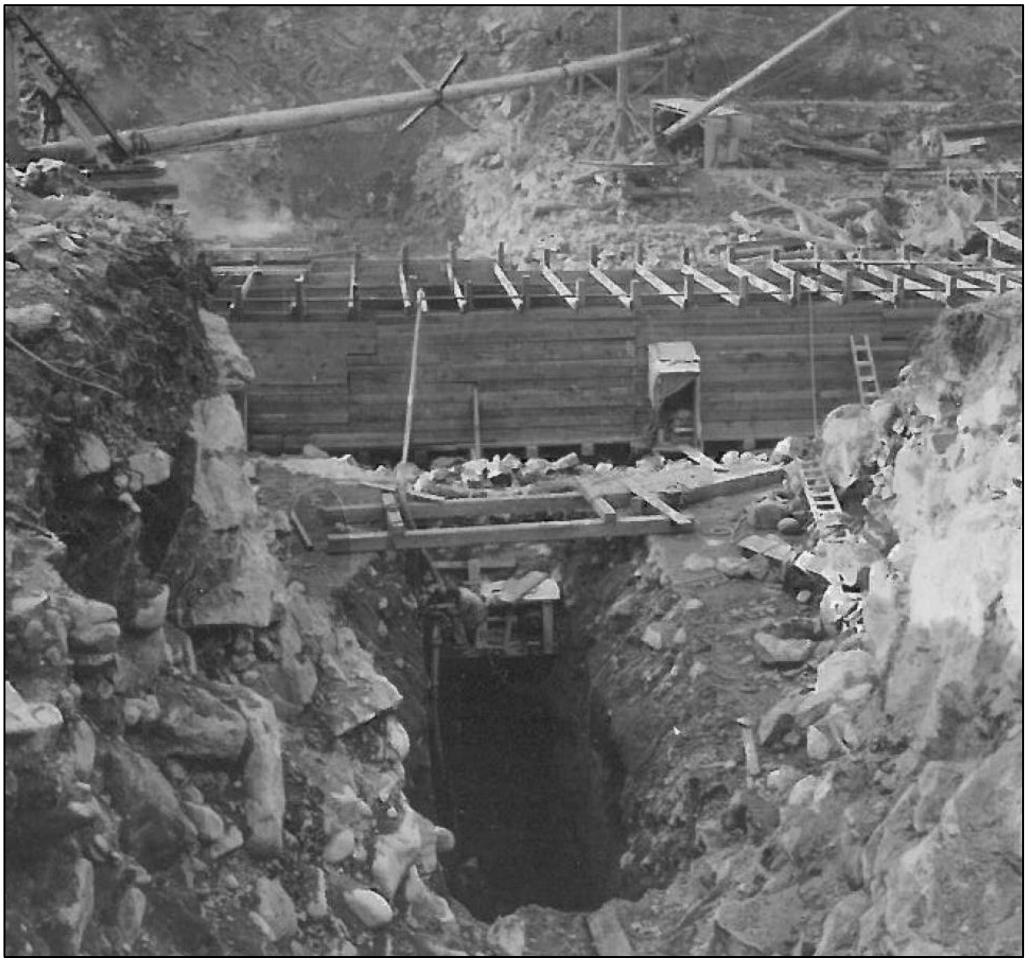

Figure 4.1 Diversion Dam Excavation Looking South from Right Bank

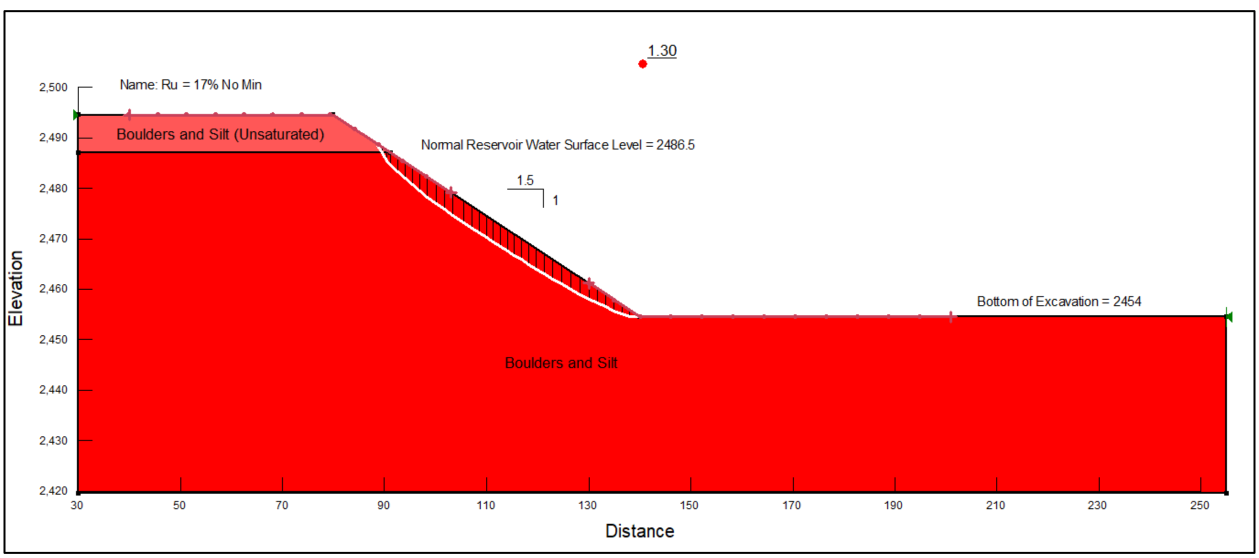

Figure 4.2 Diversion Dam Excavation Slope Stability Results

4.2 INTAKE STRUCTURE BACKFILL

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

4.3 BORROW SITES

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

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

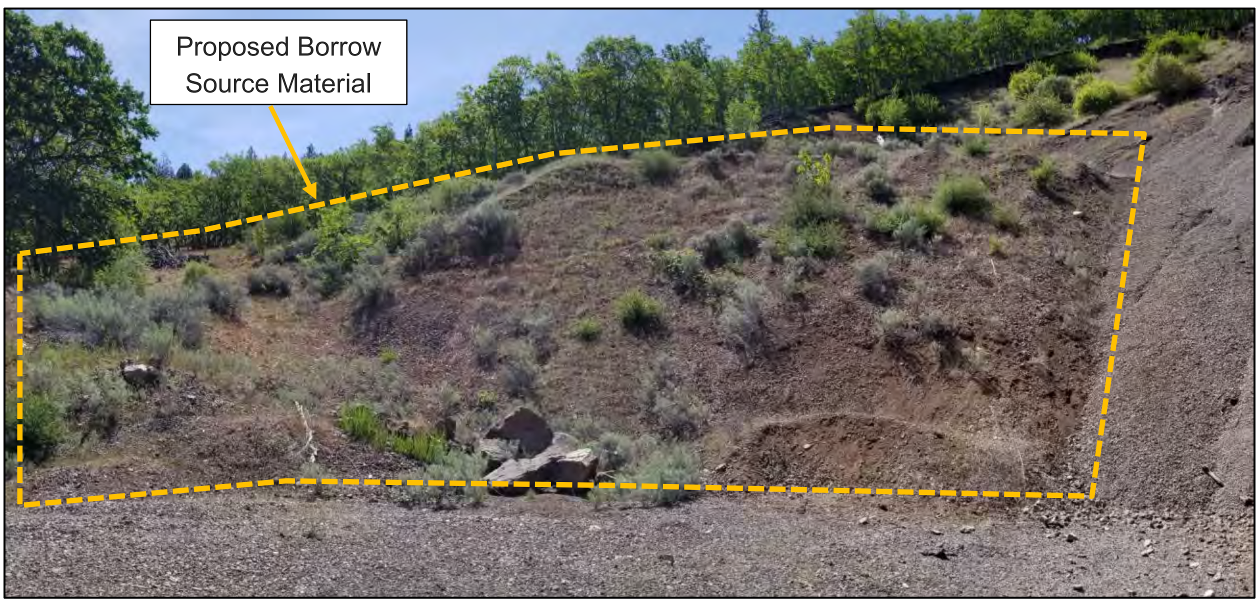

Figure 4.3 Wood-stave Borrow Site

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

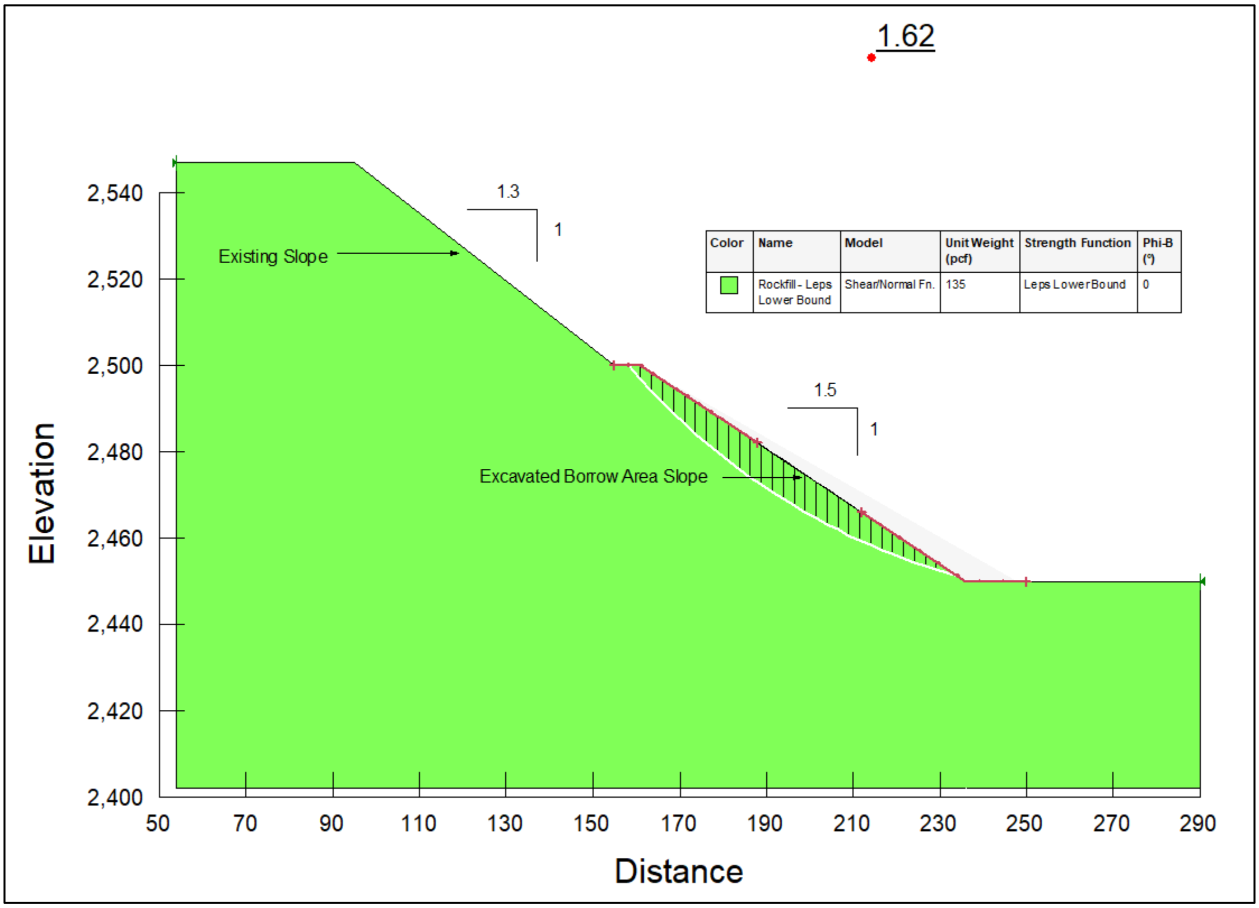

Figure 4.4 Wood-stave Borrow Area Slope Stability Analysis Results

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

4.4 TUNNEL #2 OUTLET PORTAL BACKFILL

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

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

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

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

The slope stability LEA assess two loading conditions: static long-term and yield acceleration (ky) determination for approximating seismic displacement. The acceptance criteria require a FOS of 1.5 for static long-term stability and FOS of 1.0 for yield acceleration determination without strength reduction. The design seismic loading is taken from STID-5 (PacifiCorp, 2015b), and is equal to the Copco No. 1 MCE of 0.26 g, and a maximum magnitude 7.5 earthquake is assumed based on the maximum estimated earthquake magnitudes in faults nearby as presented in the STID-5 (PacifiCorp, 2015b). Seismic displacements are approximated by the semi-empirical method, developed by Makdisi and Seed (1978).

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

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

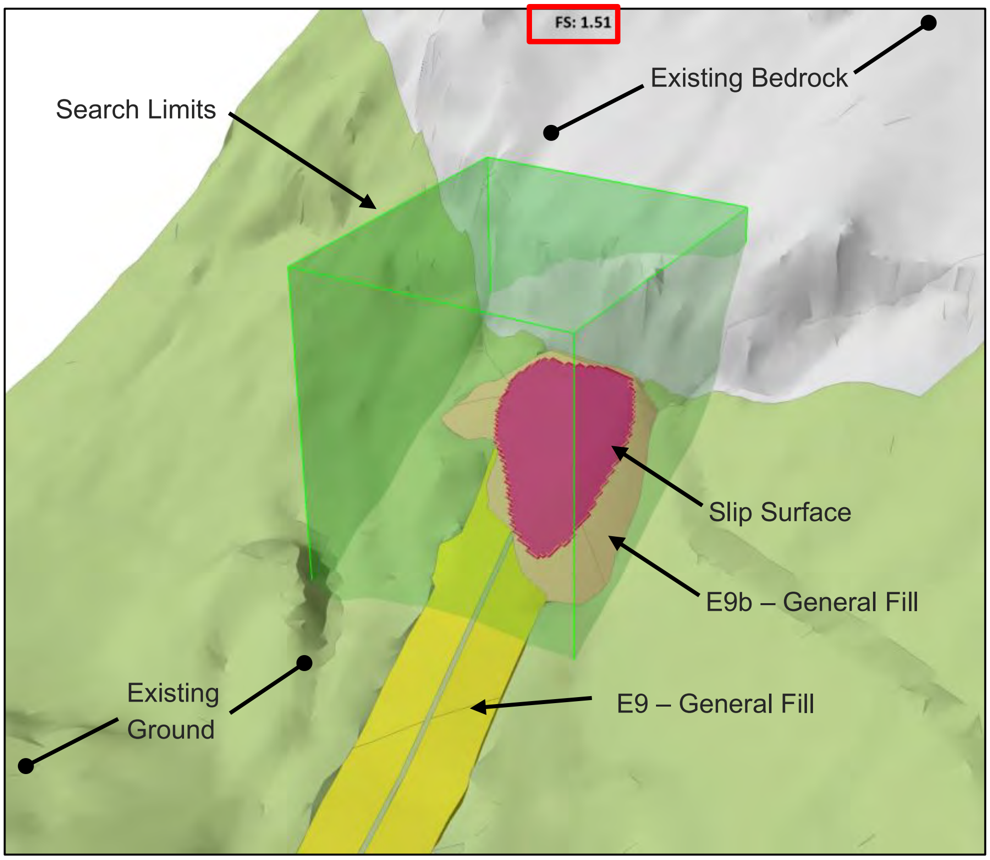

Figure 4.5 Tunnel #2 Portal Backfill Slope Stability LEA Results

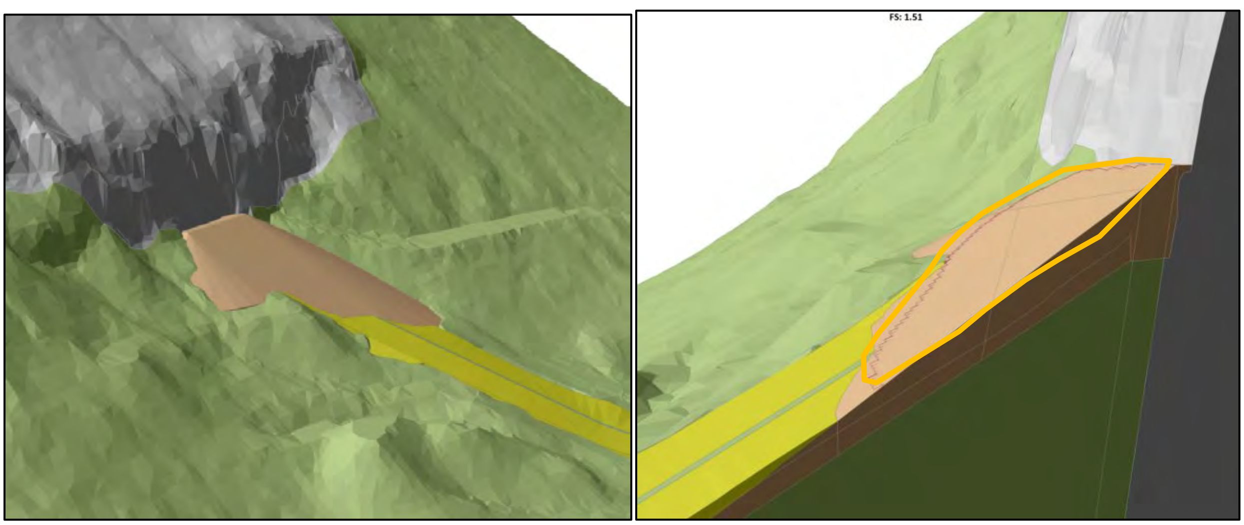

Figure 4.6 Oblique View of Backfill with no Slip Surface (Right) Cross Section of Backfill with Slip Surface Highlighted (Left)

4.5 POWERHOUSE TAILRACE BACKFILL

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

Table 4.2 Material Parameters for Tailrace Backfill Slope Stability LEA

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

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

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

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

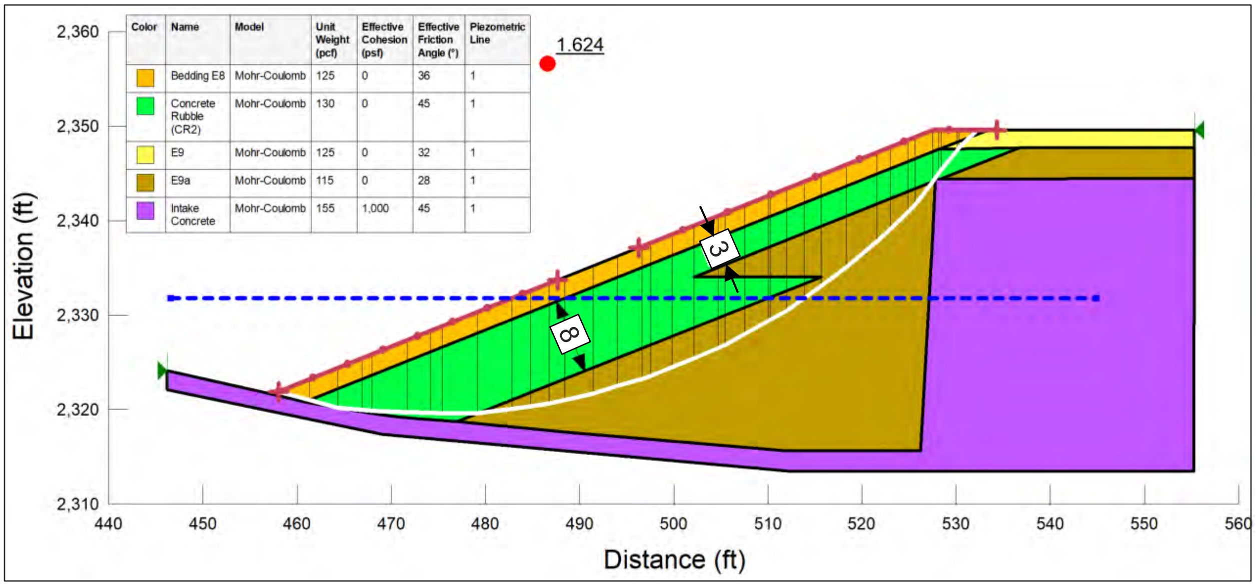

Figure 4.7 Tailrace Backfill LEA Stability Results

4.6 LEFT BANK ACCESS ROAD

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

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

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

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

4.7 LEFT ABUTMENT CLIFF BAND

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

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

Figure 4.8 Cliff Band and Timber Crib above Intake Structure

Figure 4.9 Upper Cliff Basalt Columns

Attachments:

1 – Copco No. 2 Facility Simulated Drawdown

2 – Copco No. 2 Geohazard Terrain Maps

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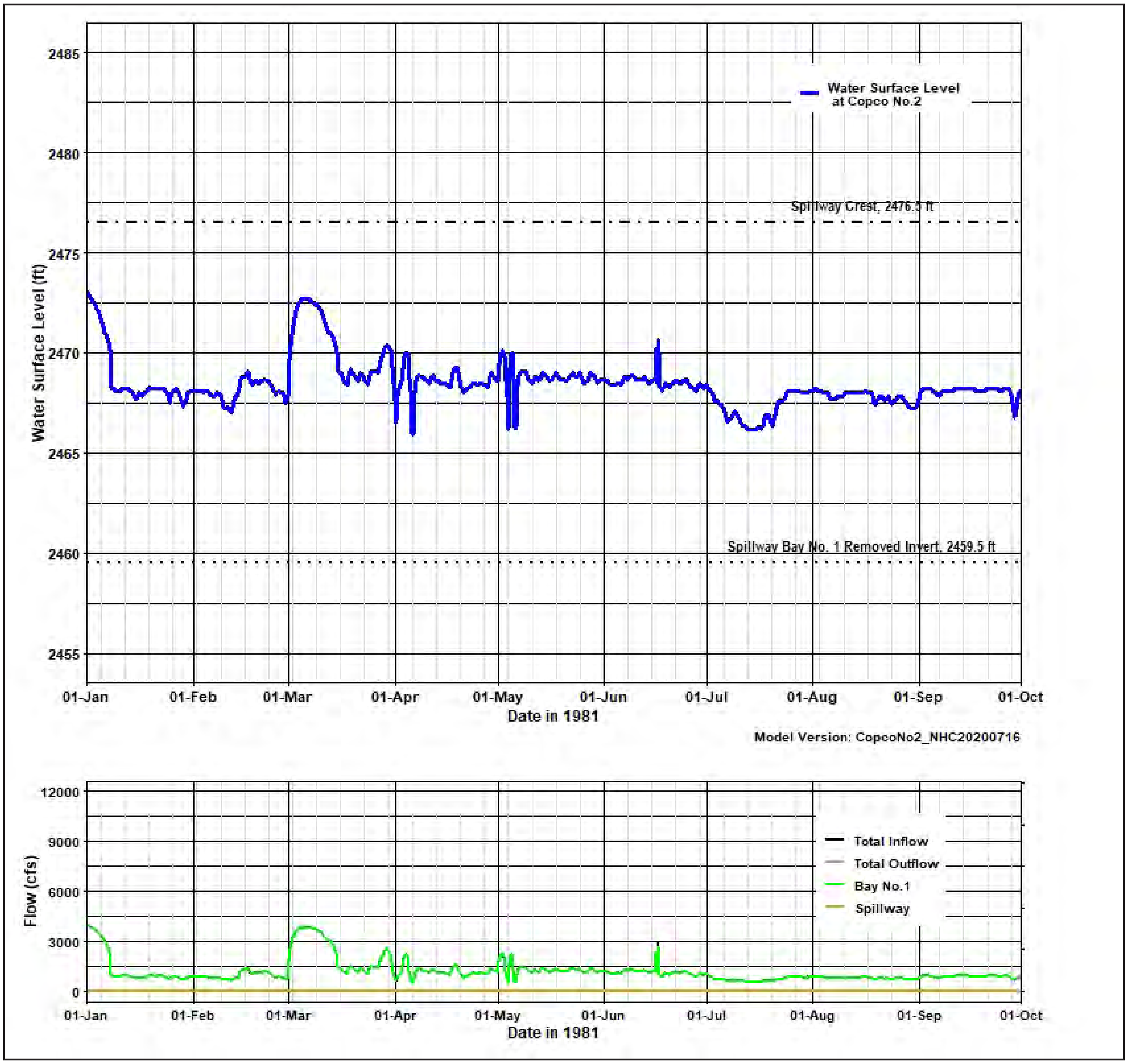

Figure 1 - COPCO No.2 Facility Simulated Drawdown - Year 1981

July 29, 2020

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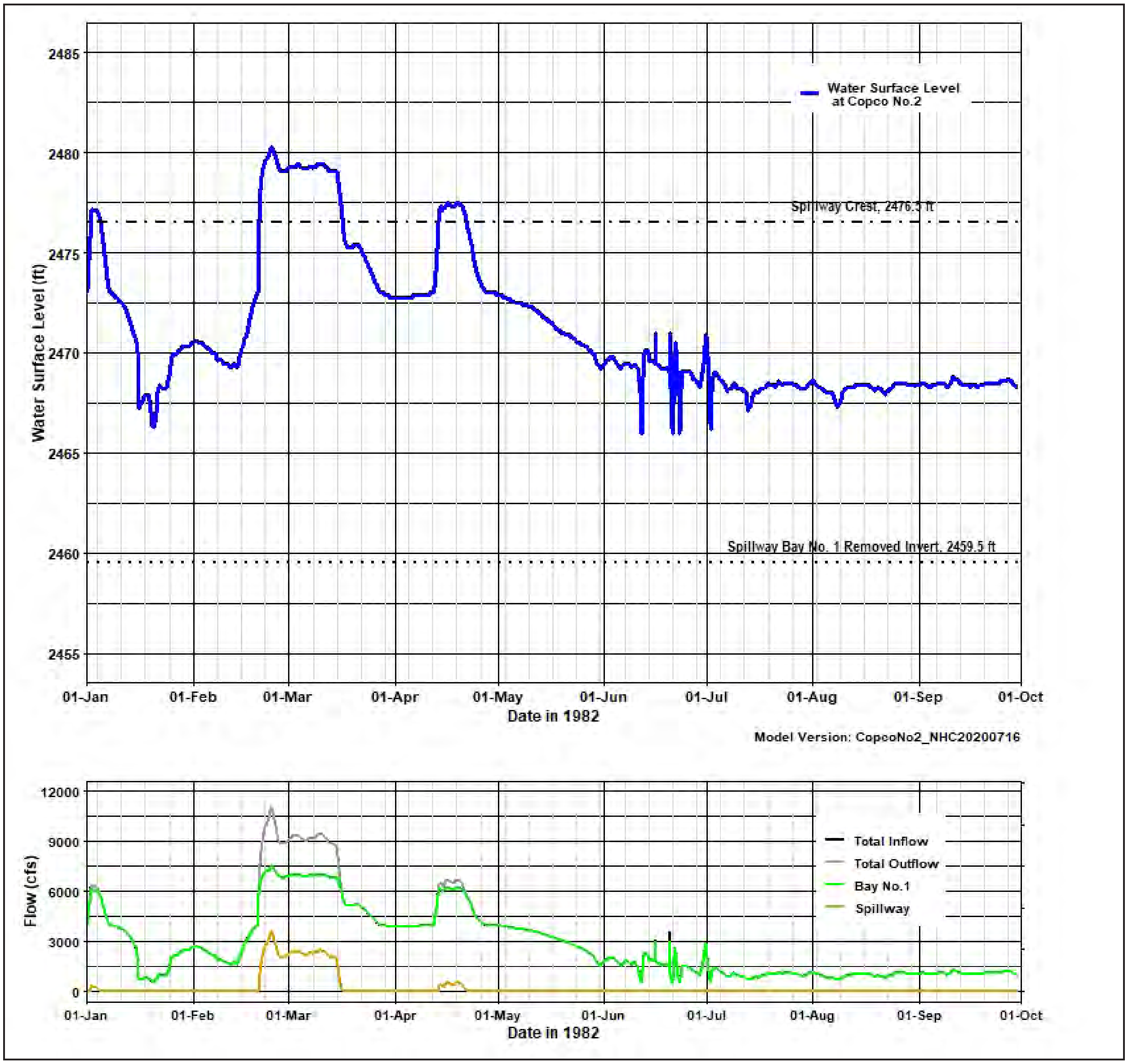

Figure 2 - COPCO No.2 Facility Simulated Drawdown - Year 1982

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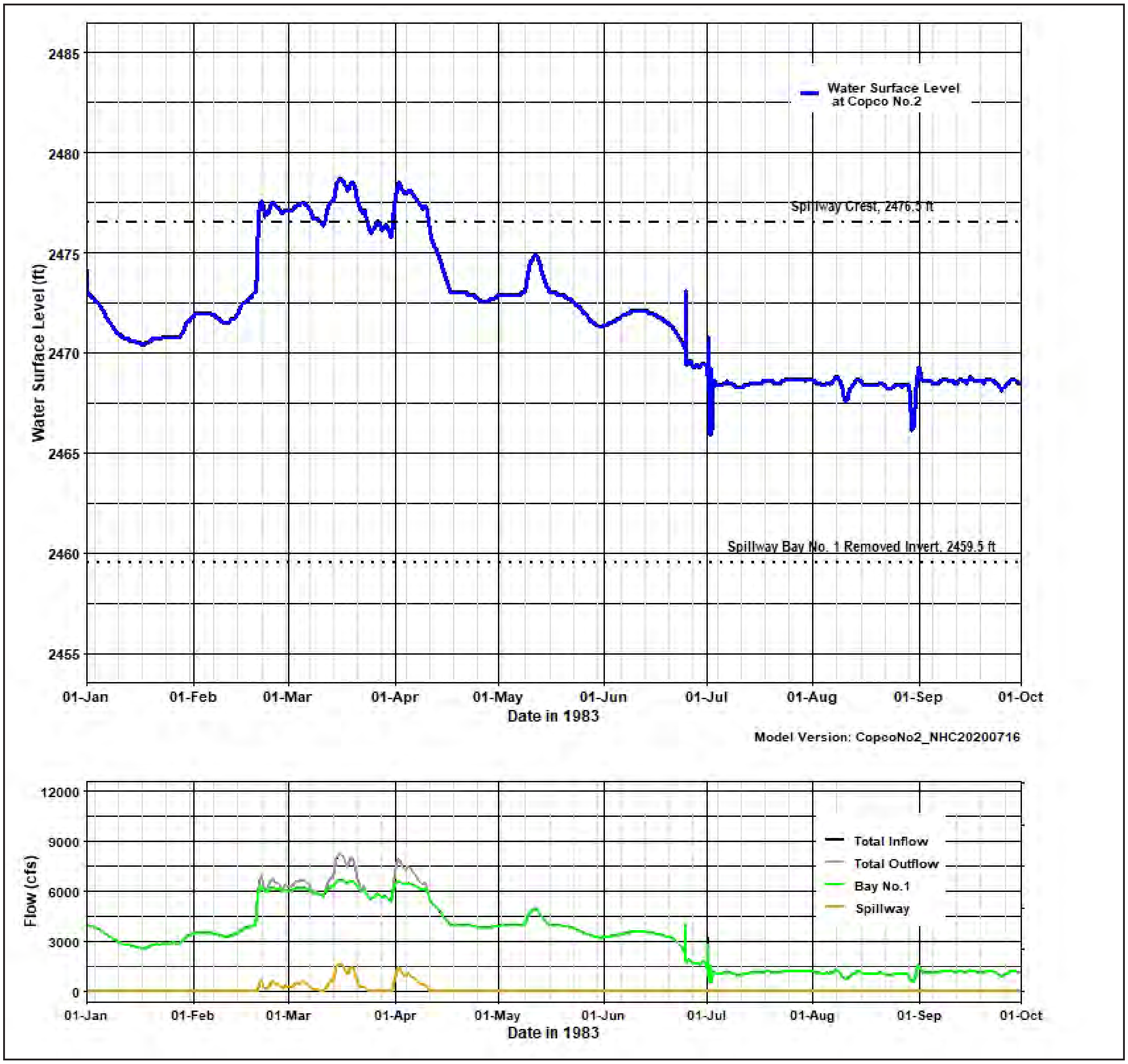

Figure 3 - COPCO No.2 Facility Simulated Drawdown - Year 1983

July 29, 2020

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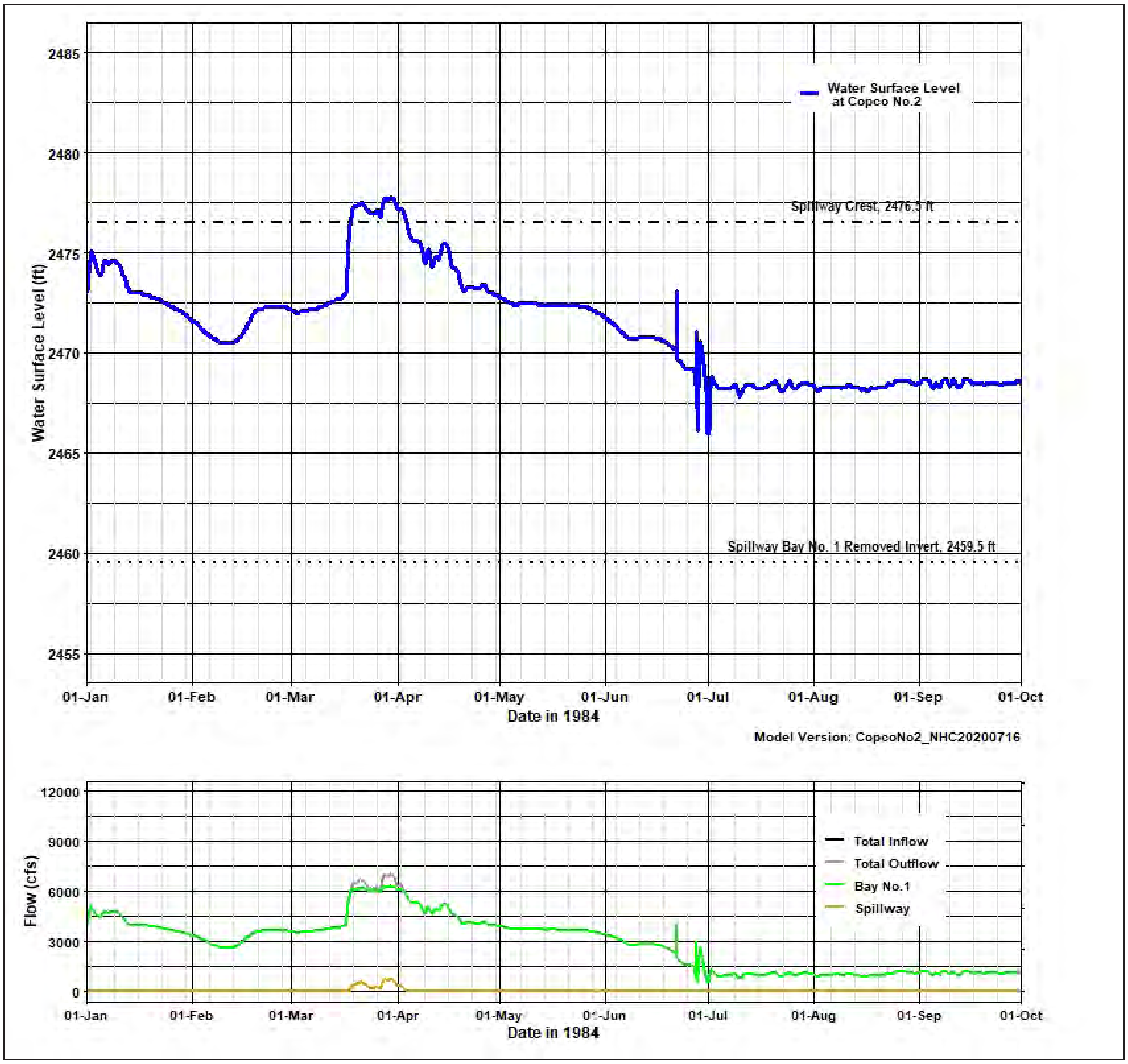

Figure 4 - COPCO No.2 Facility Simulated Drawdown - Year 1984

July 29, 2020

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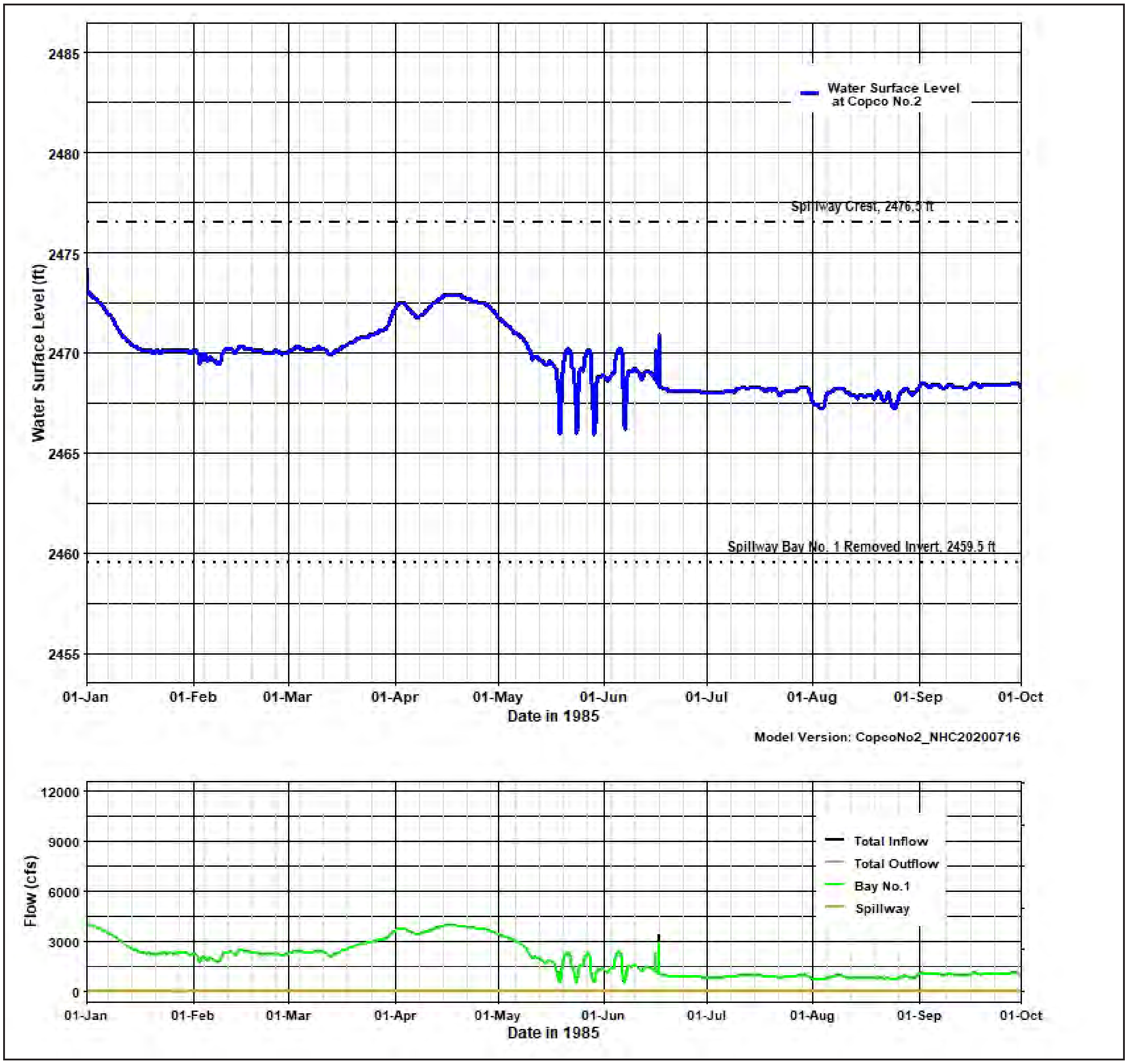

Figure 5 - COPCO No.2 Facility Simulated Drawdown - Year 1985

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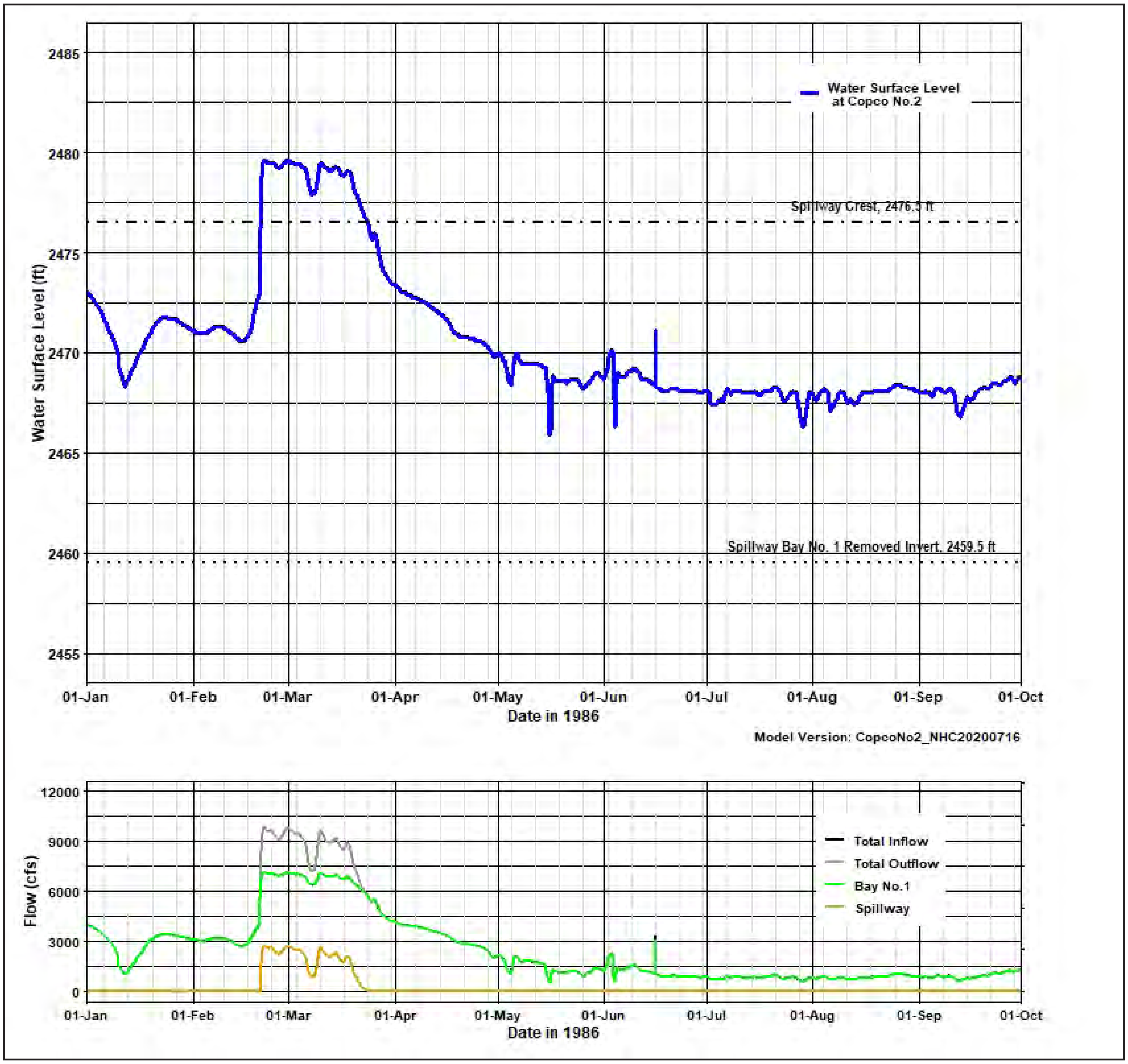

Figure 6 - COPCO No.2 Facility Simulated Drawdown - Year 1986

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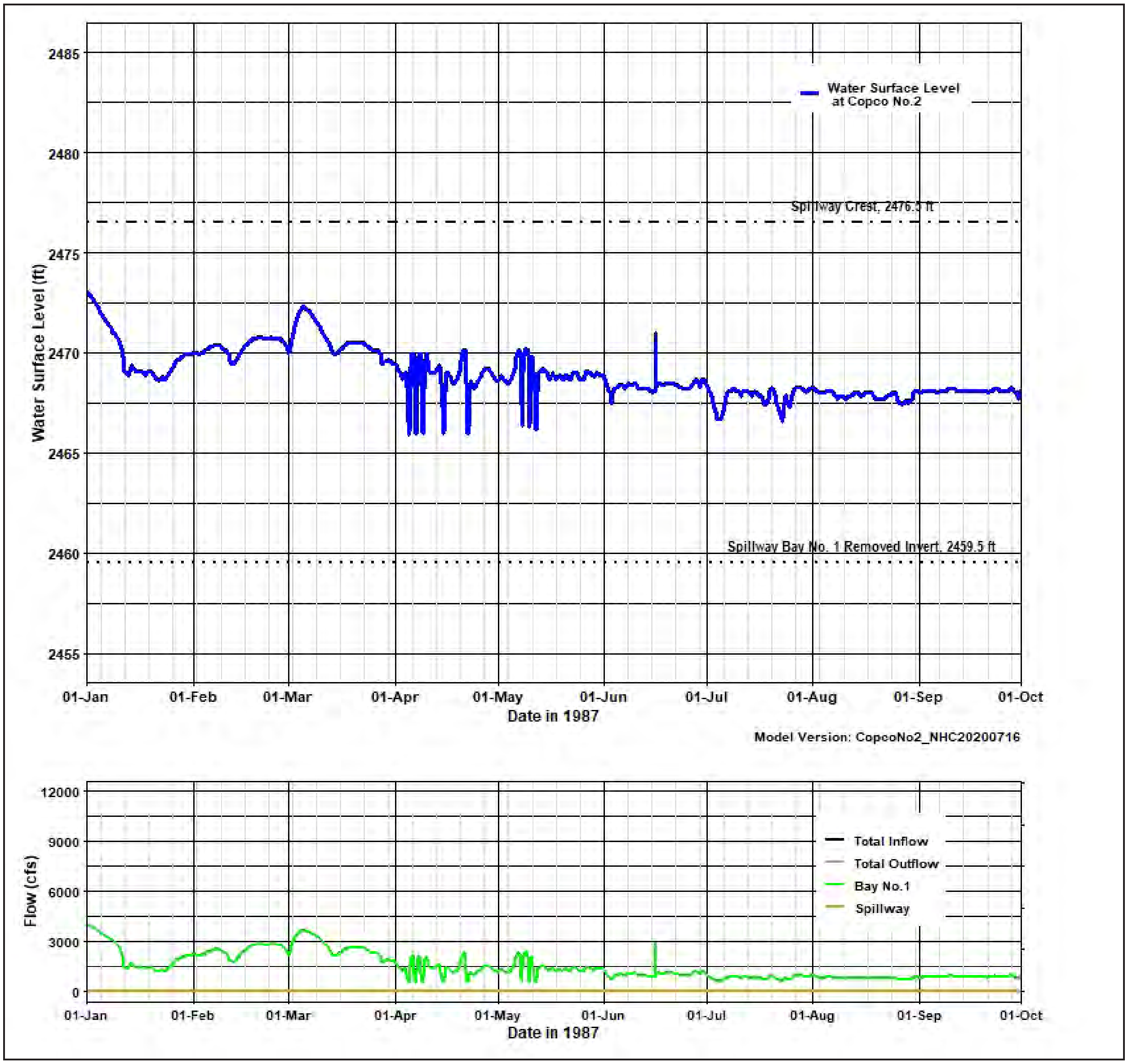

Figure 7 - COPCO No.2 Facility Simulated Drawdown - Year 1987

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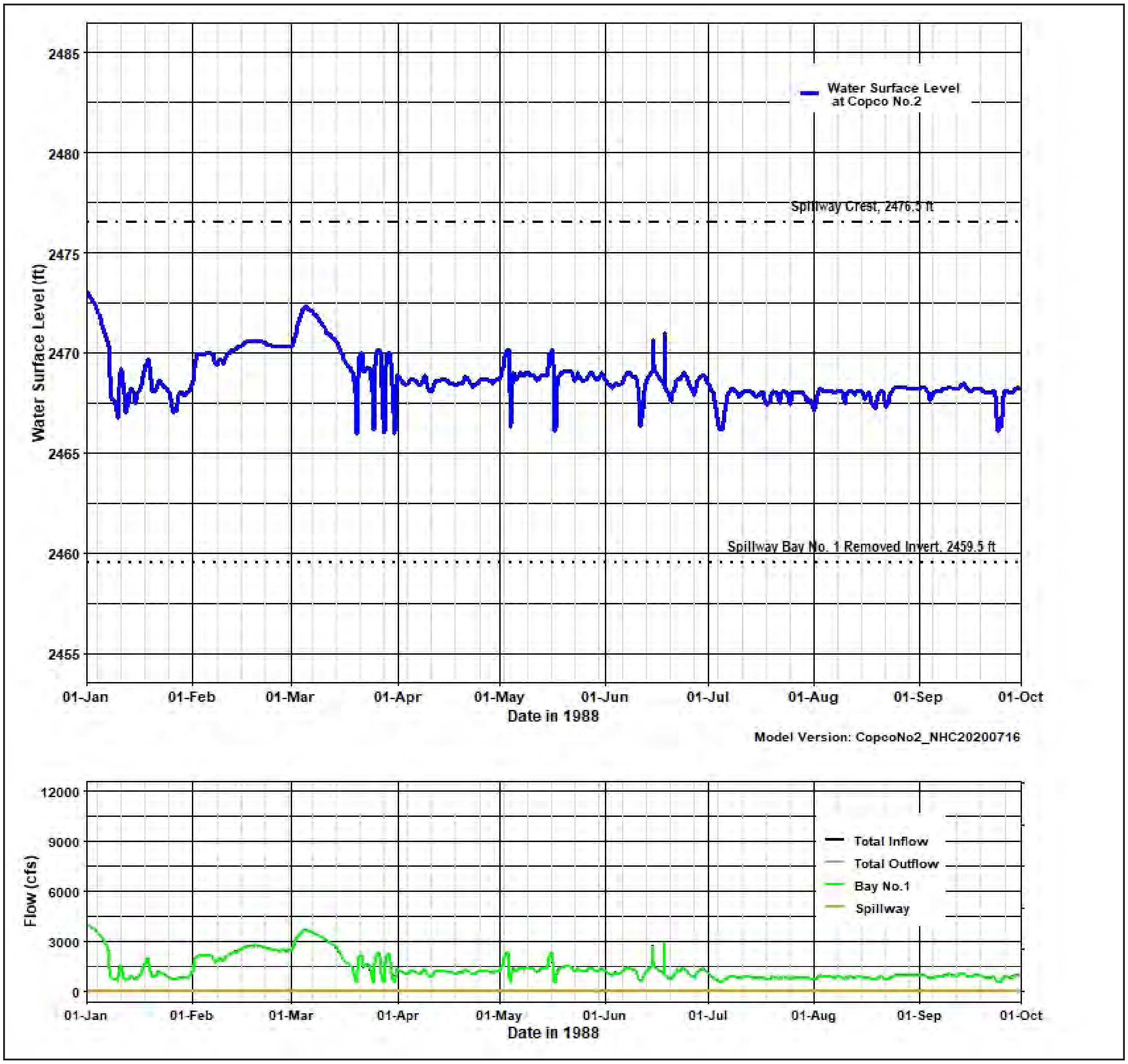

Figure 8 - COPCO No.2 Facility Simulated Drawdown - Year 1988

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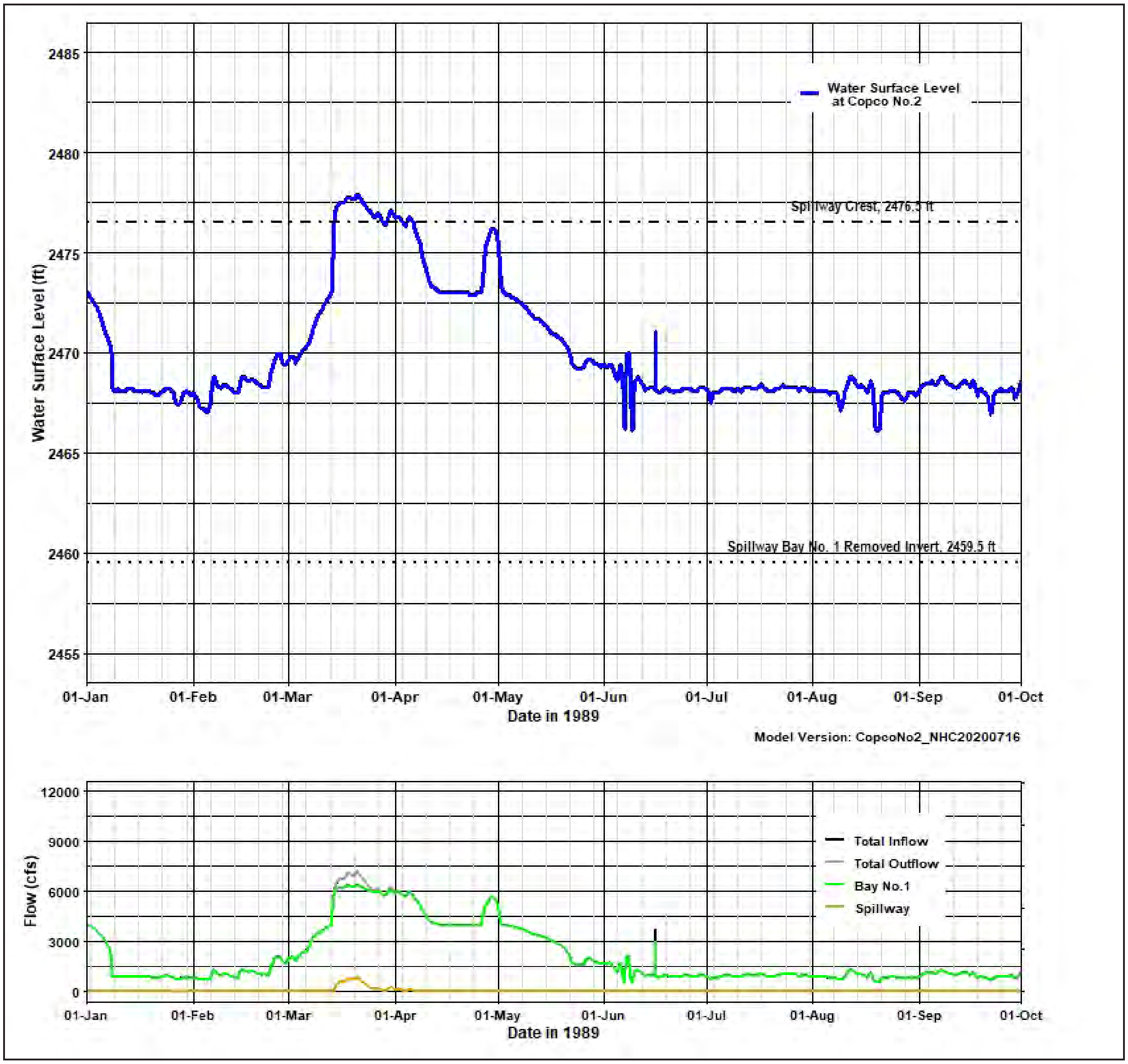

Figure 9 - COPCO No.2 Facility Simulated Drawdown - Year 1989

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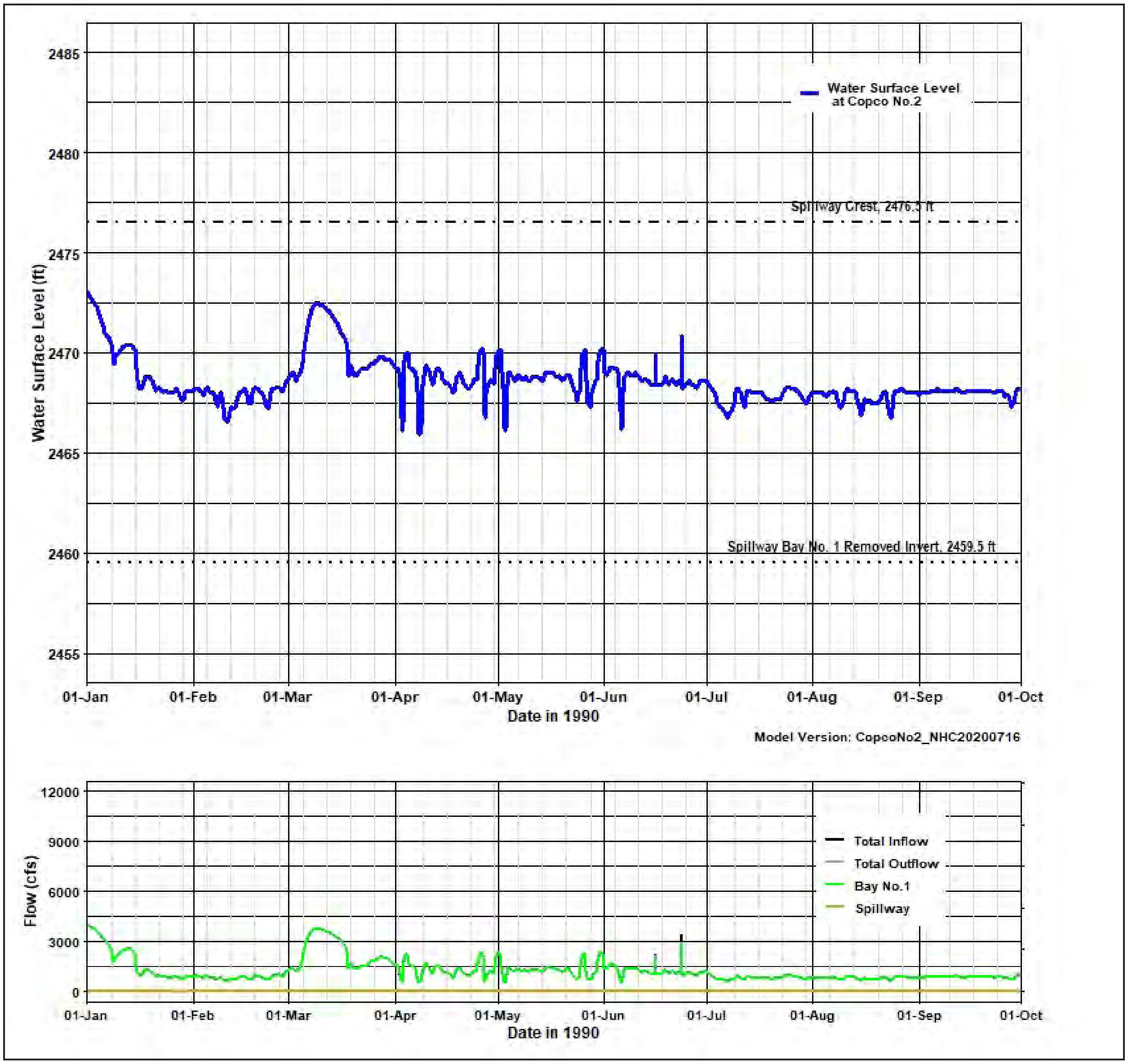

Figure 10 - COPCO No.2 Facility Simulated Drawdown - Year 1990

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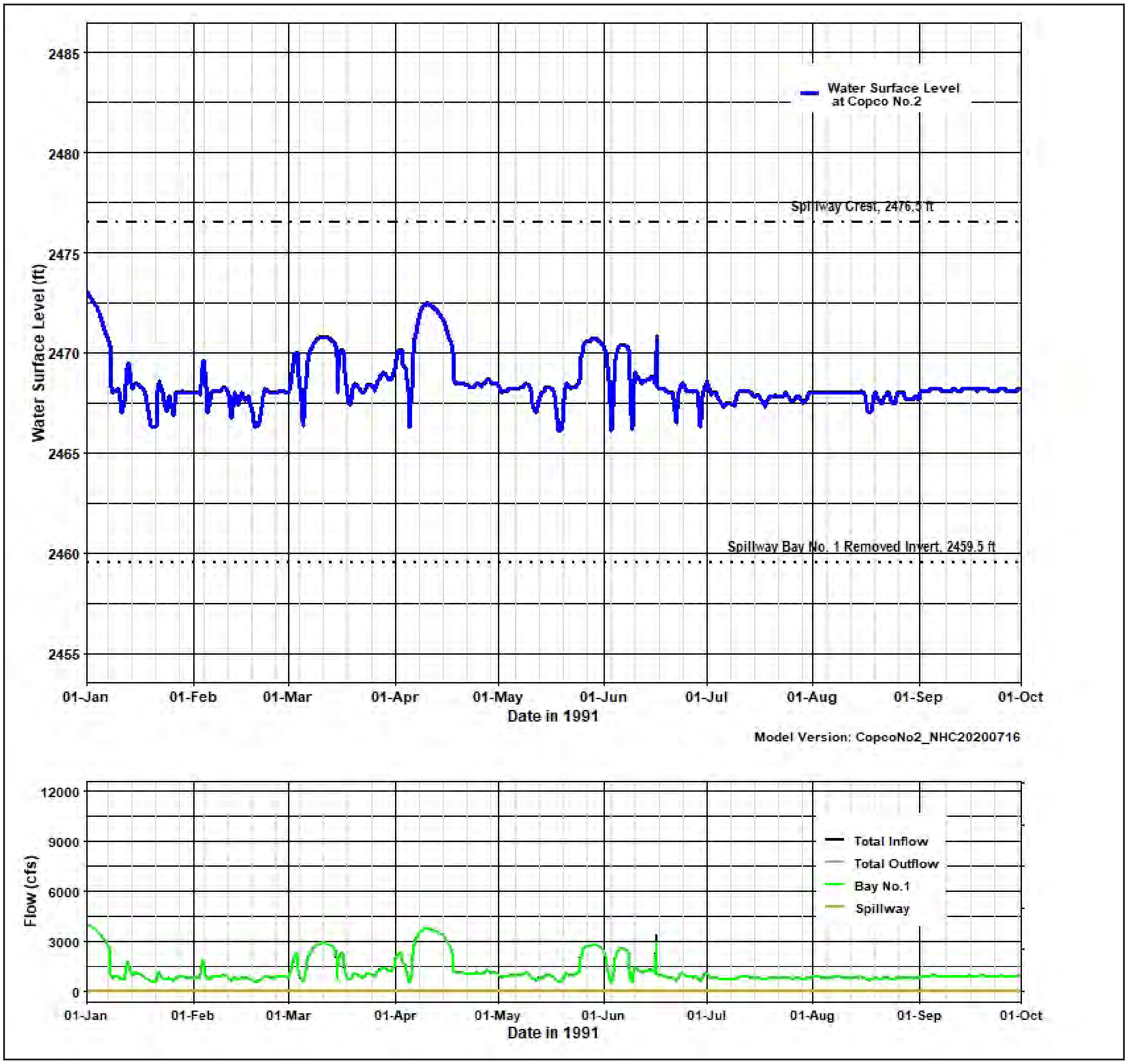

Figure 11 - COPCO No.2 Facility Simulated Drawdown - Year 1991

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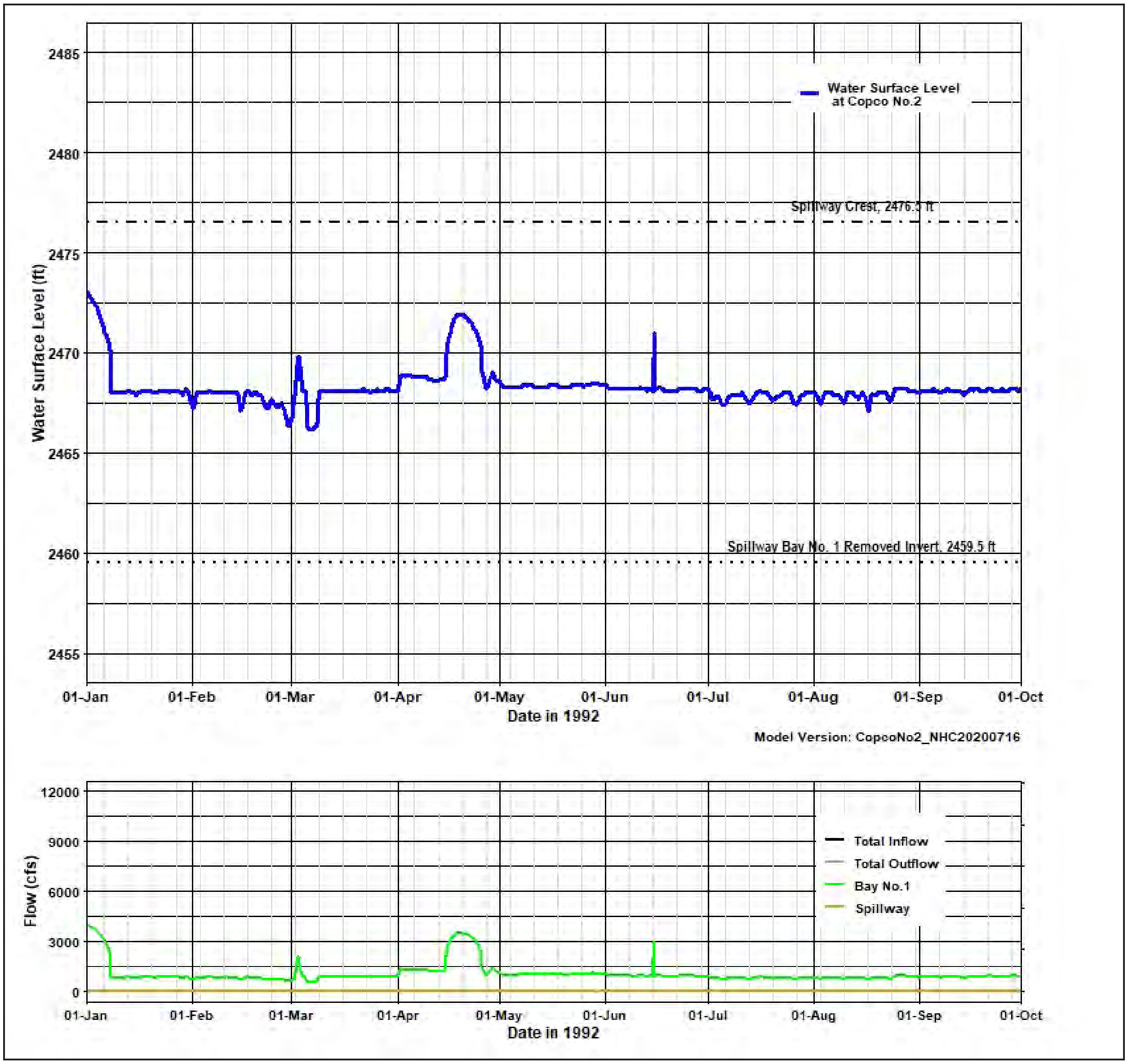

Figure 12 - COPCO No.2 Facility Simulated Drawdown - Year 1992

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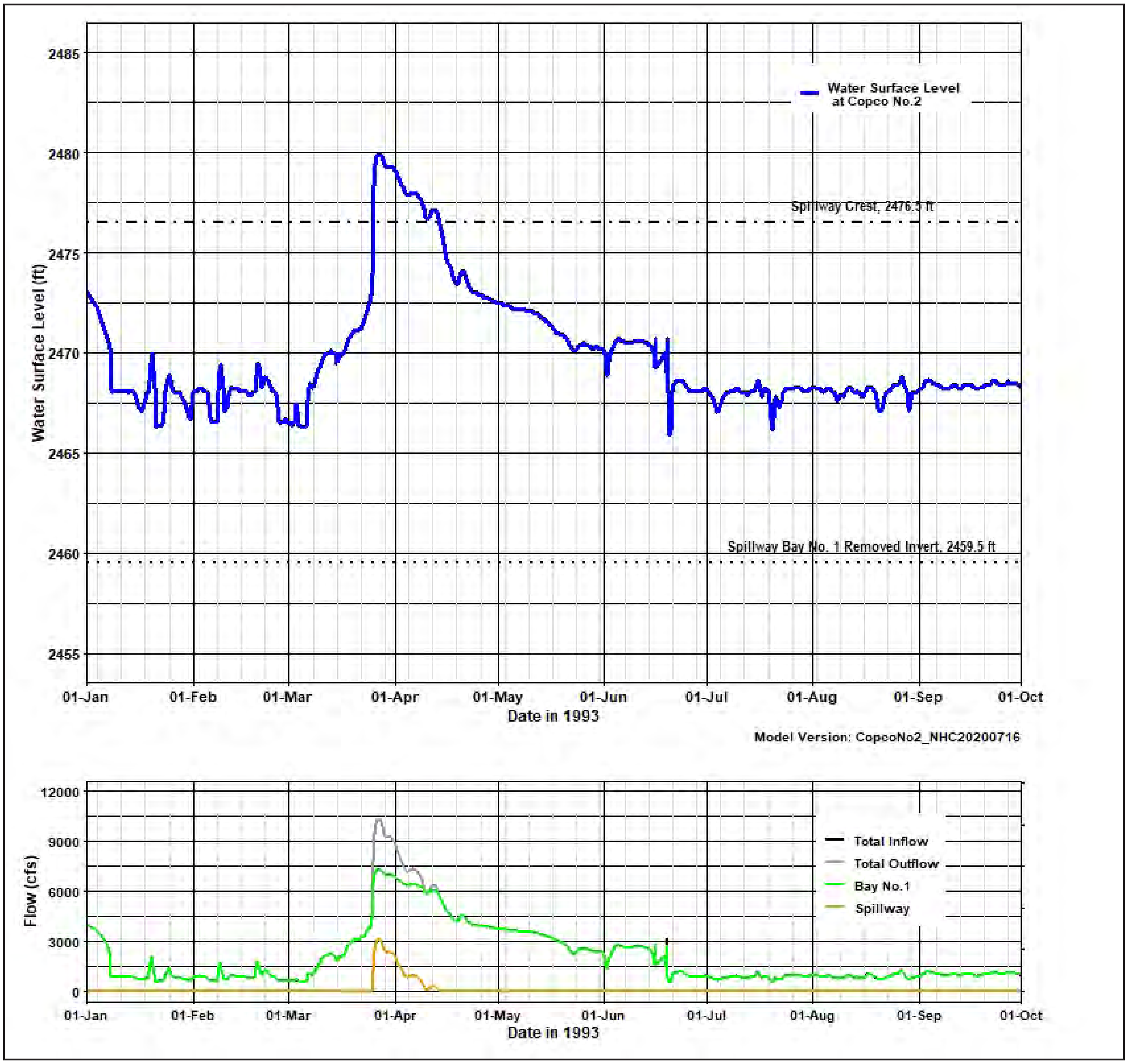

Figure 13 - COPCO No.2 Facility Simulated Drawdown - Year 1993

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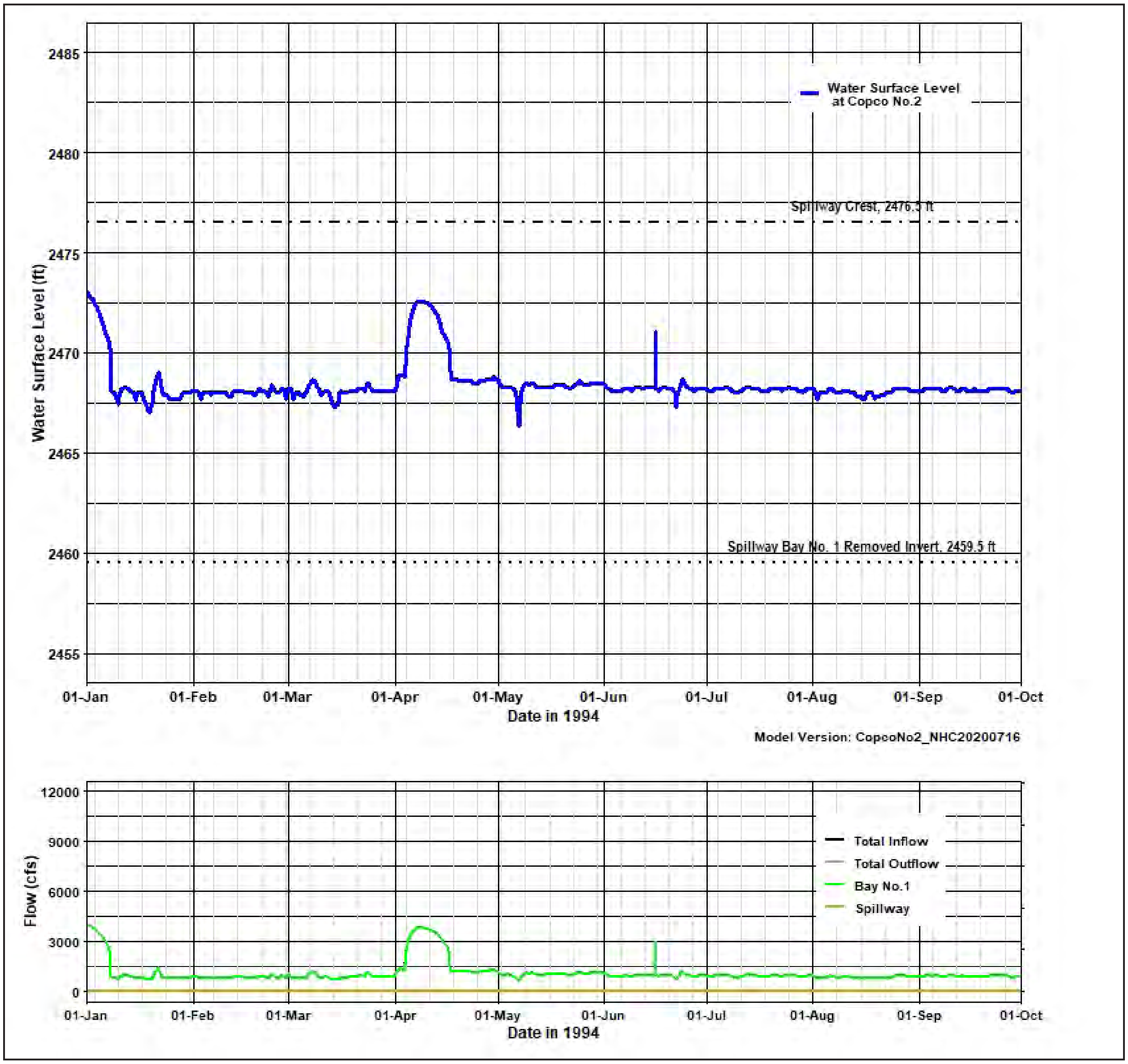

Figure 14 - COPCO No.2 Facility Simulated Drawdown - Year 1994

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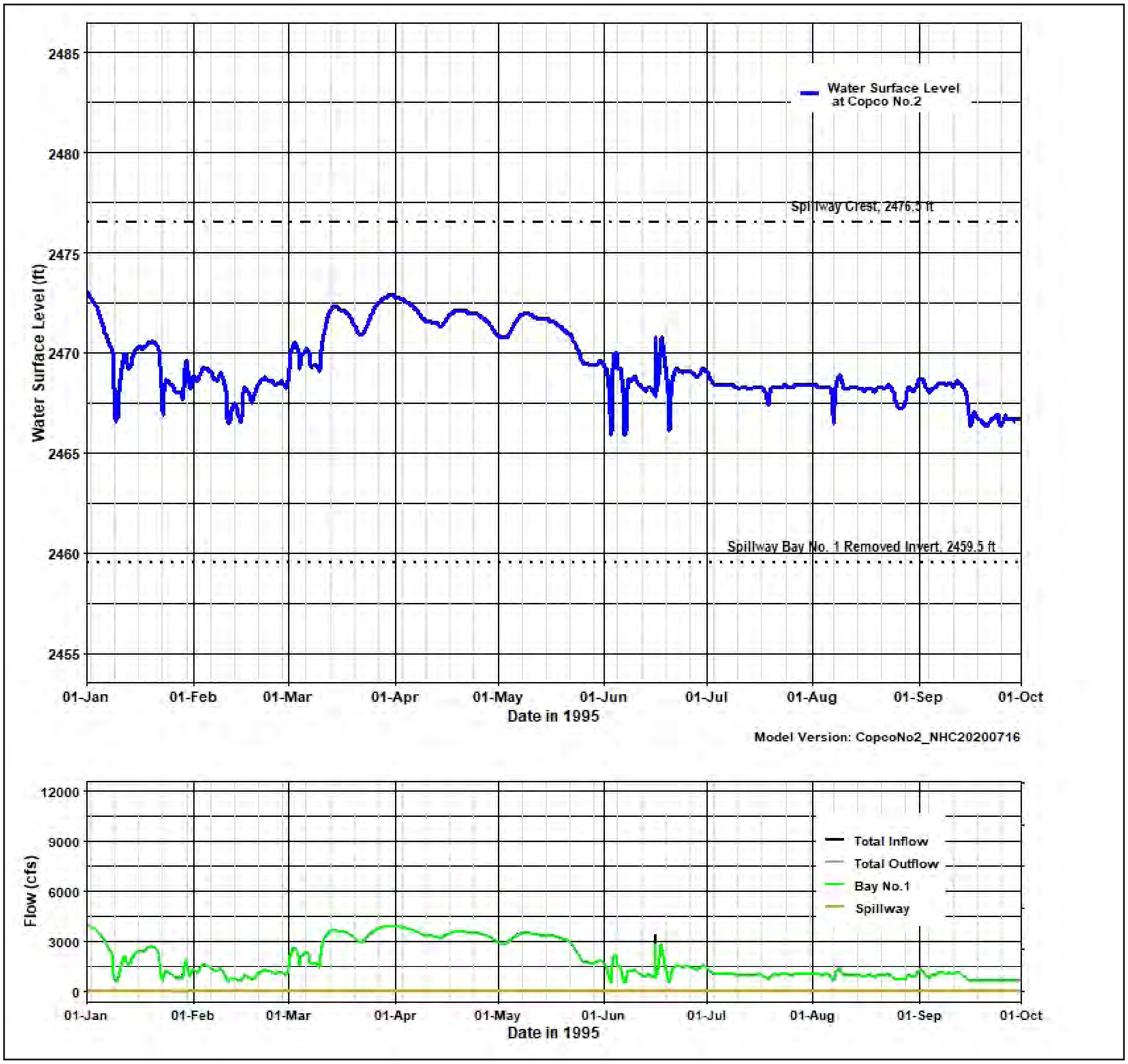

Figure 15 - COPCO No.2 Facility Simulated Drawdown - Year 1995

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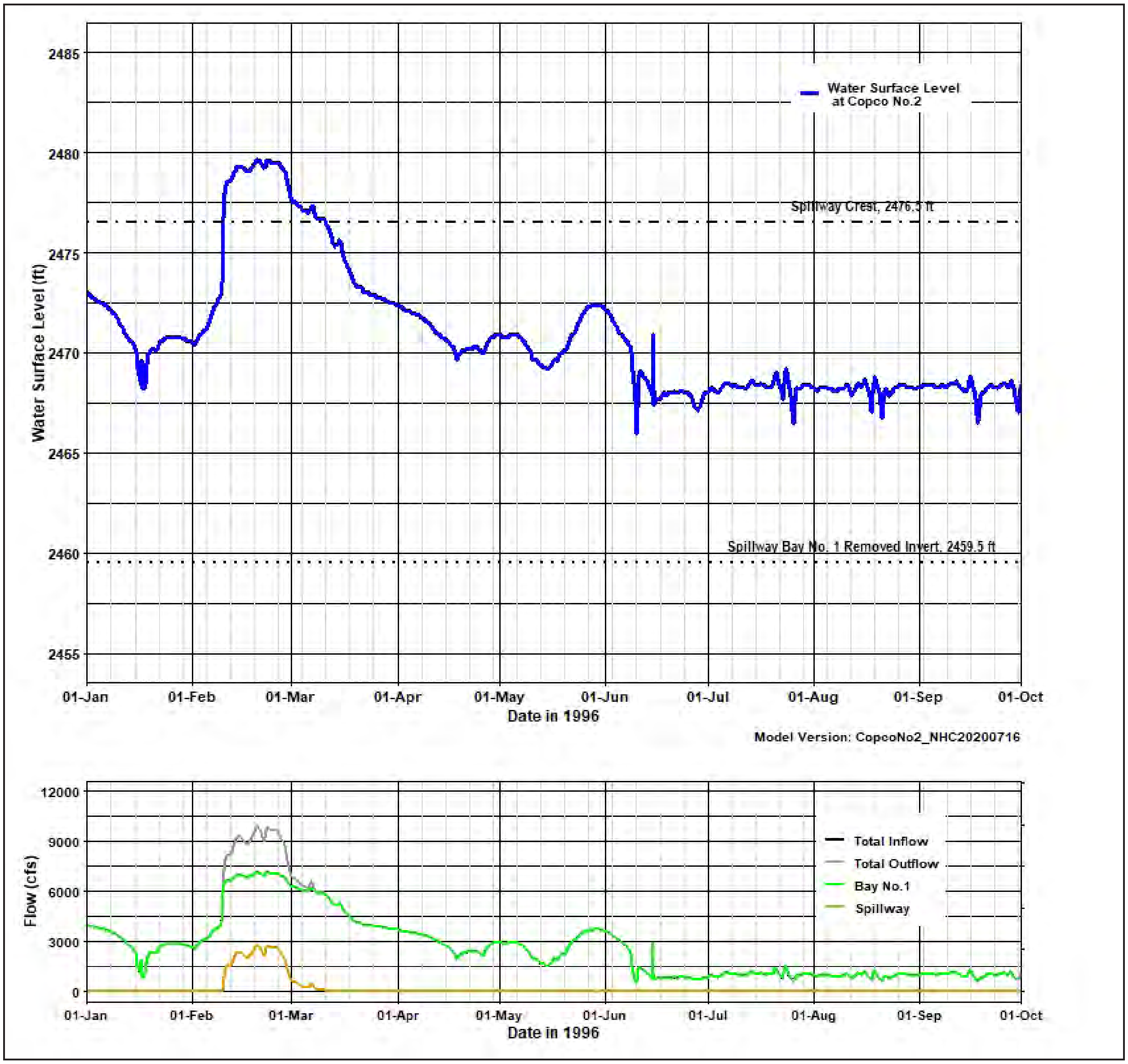

Figure 16 - COPCO No.2 Facility Simulated Drawdown - Year 1996

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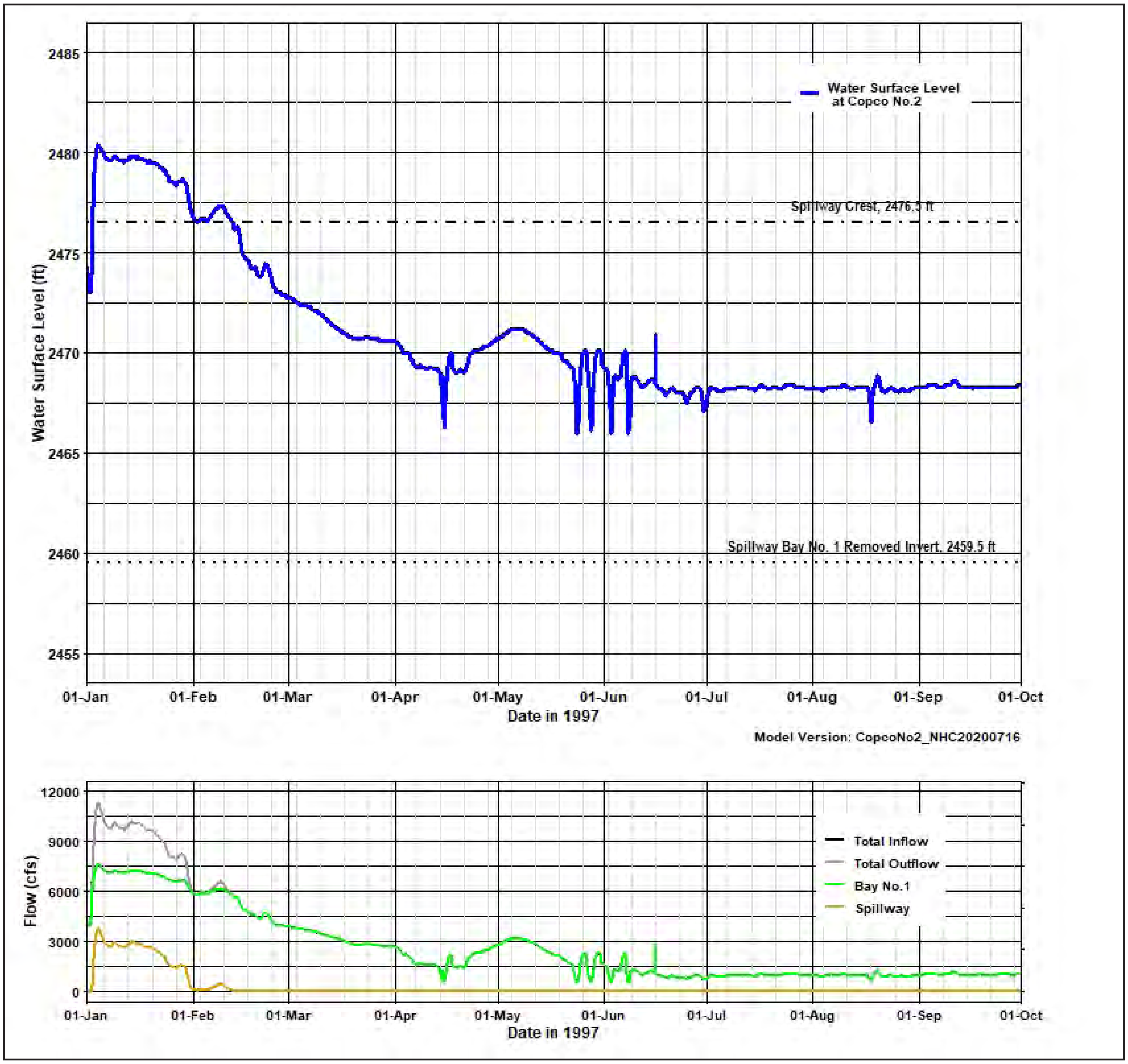

Figure 17 - COPCO No.2 Facility Simulated Drawdown - Year 1997

July 29, 2020

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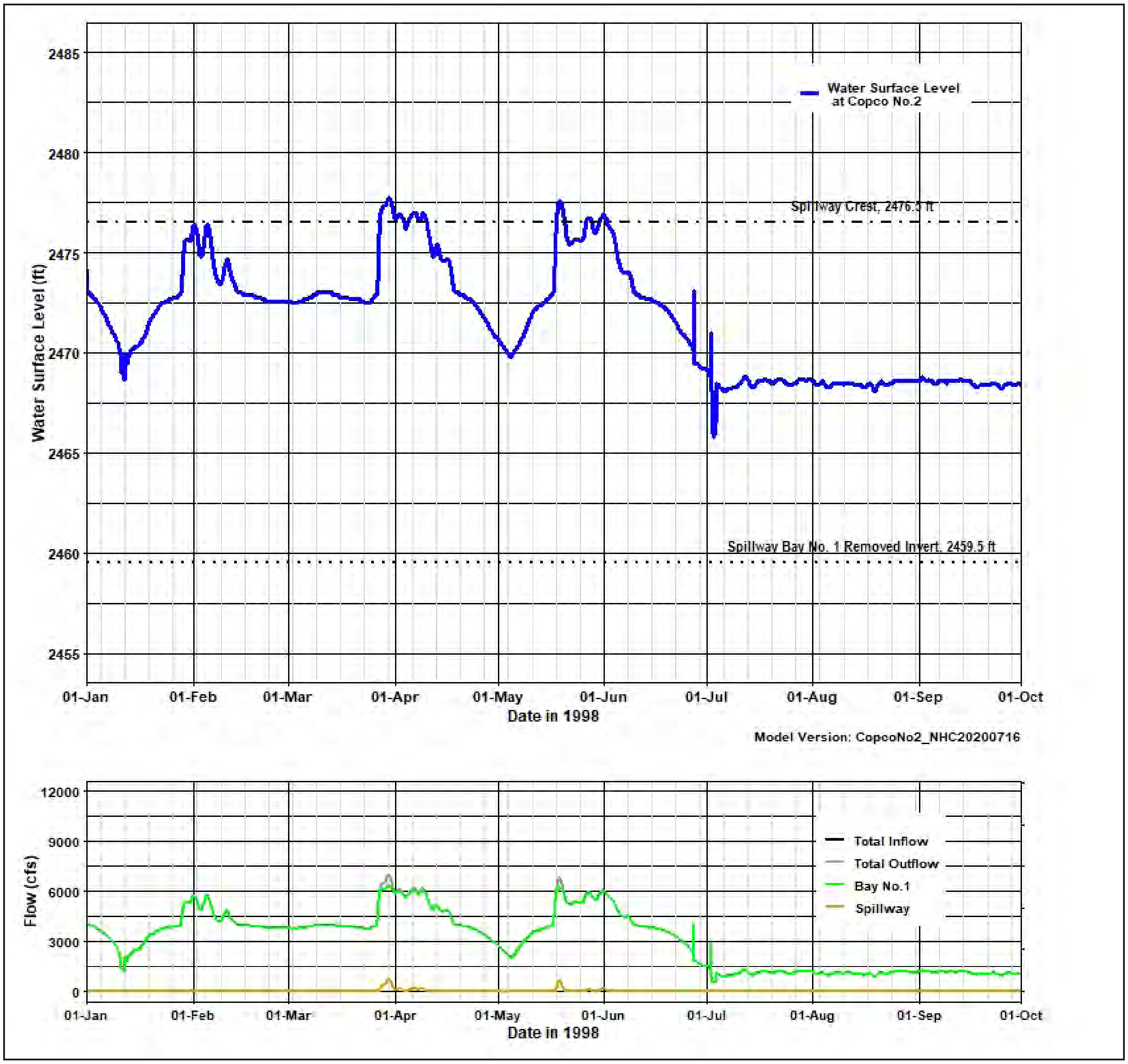

Figure 18 - COPCO No.2 Facility Simulated Drawdown - Year 1998

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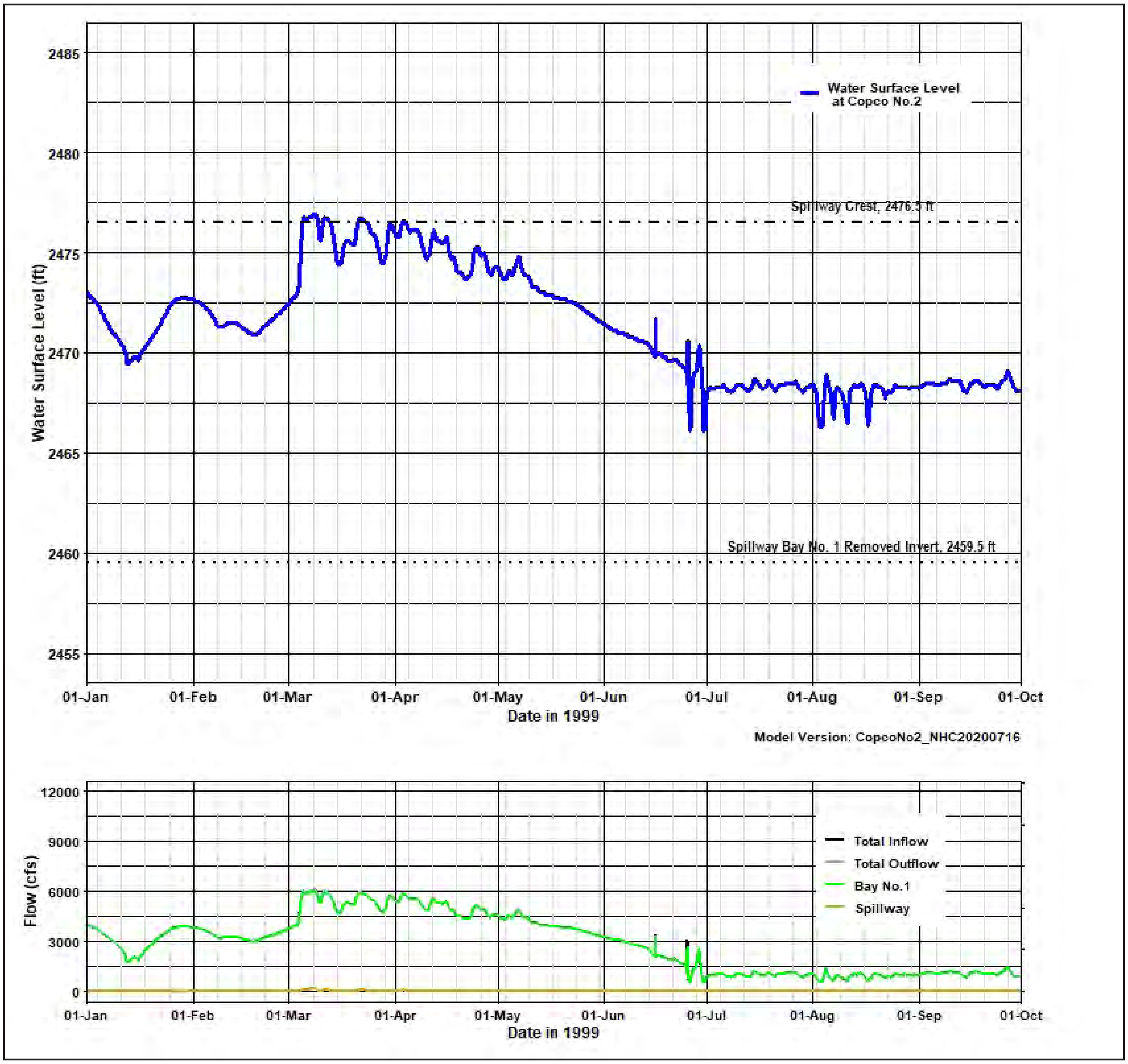

Figure 19 - COPCO No.2 Facility Simulated Drawdown - Year 1999

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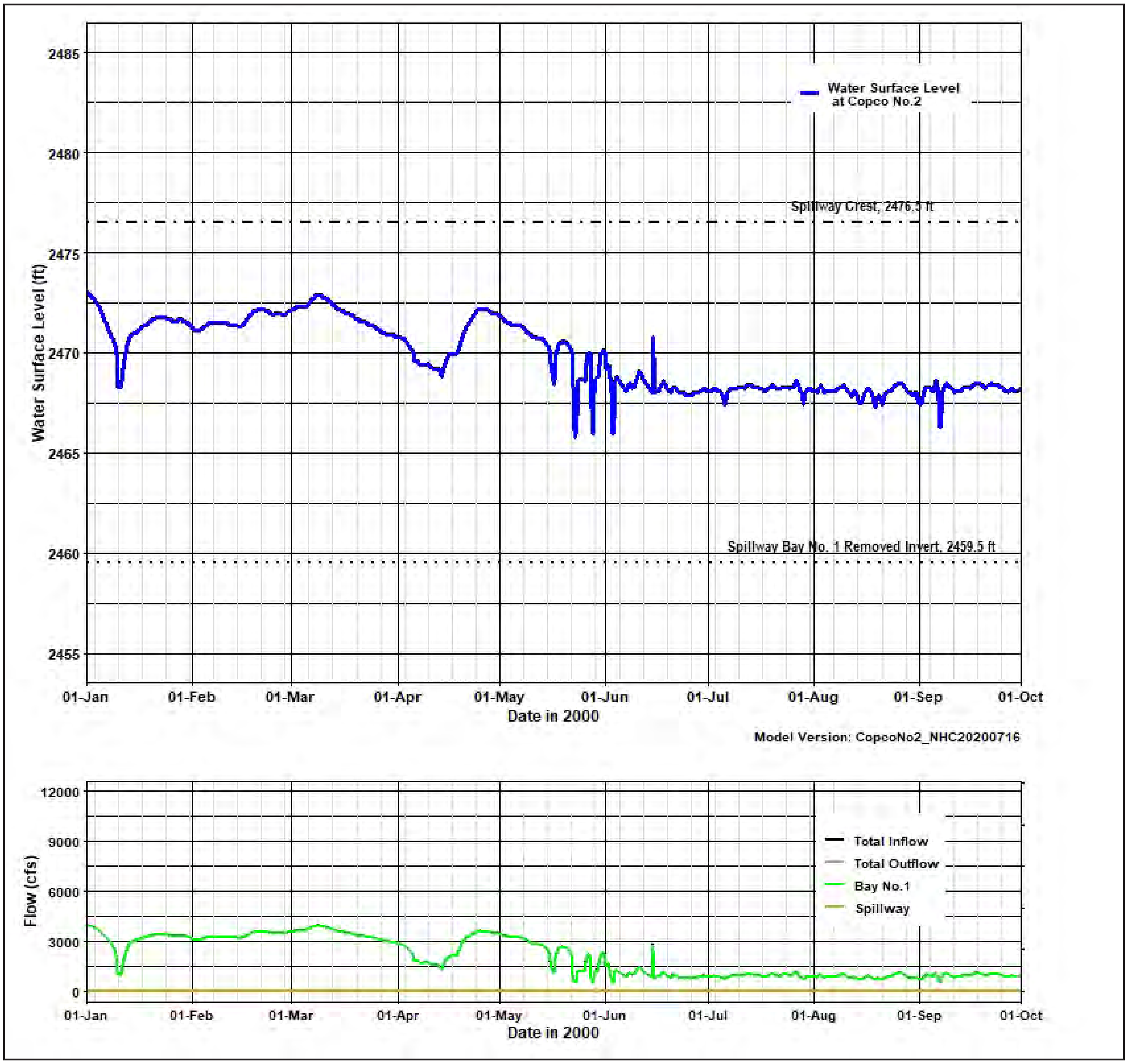

Figure 20 - COPCO No.2 Facility Simulated Drawdown - Year 2000

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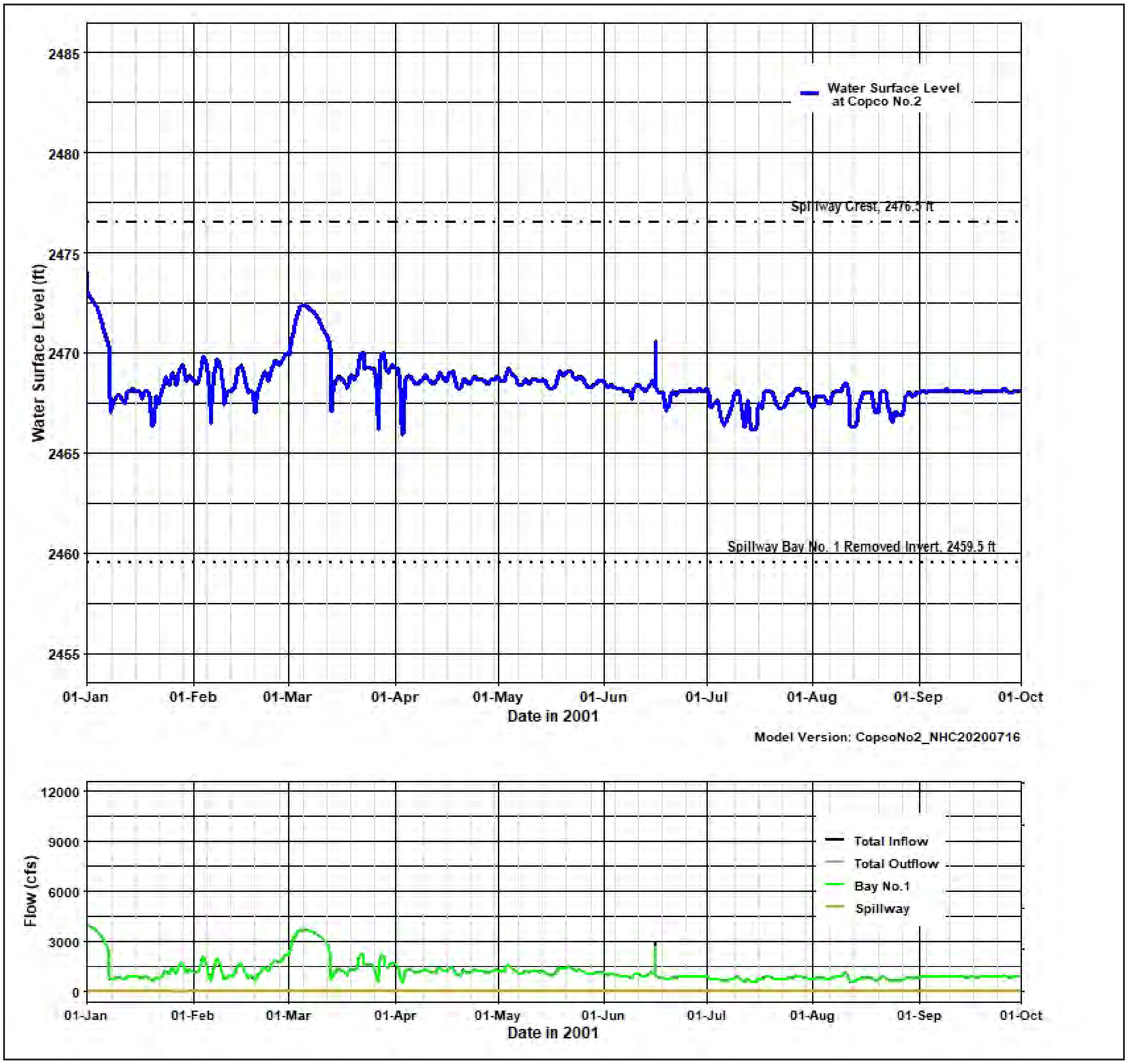

Figure 21 - COPCO No.2 Facility Simulated Drawdown - Year 2001

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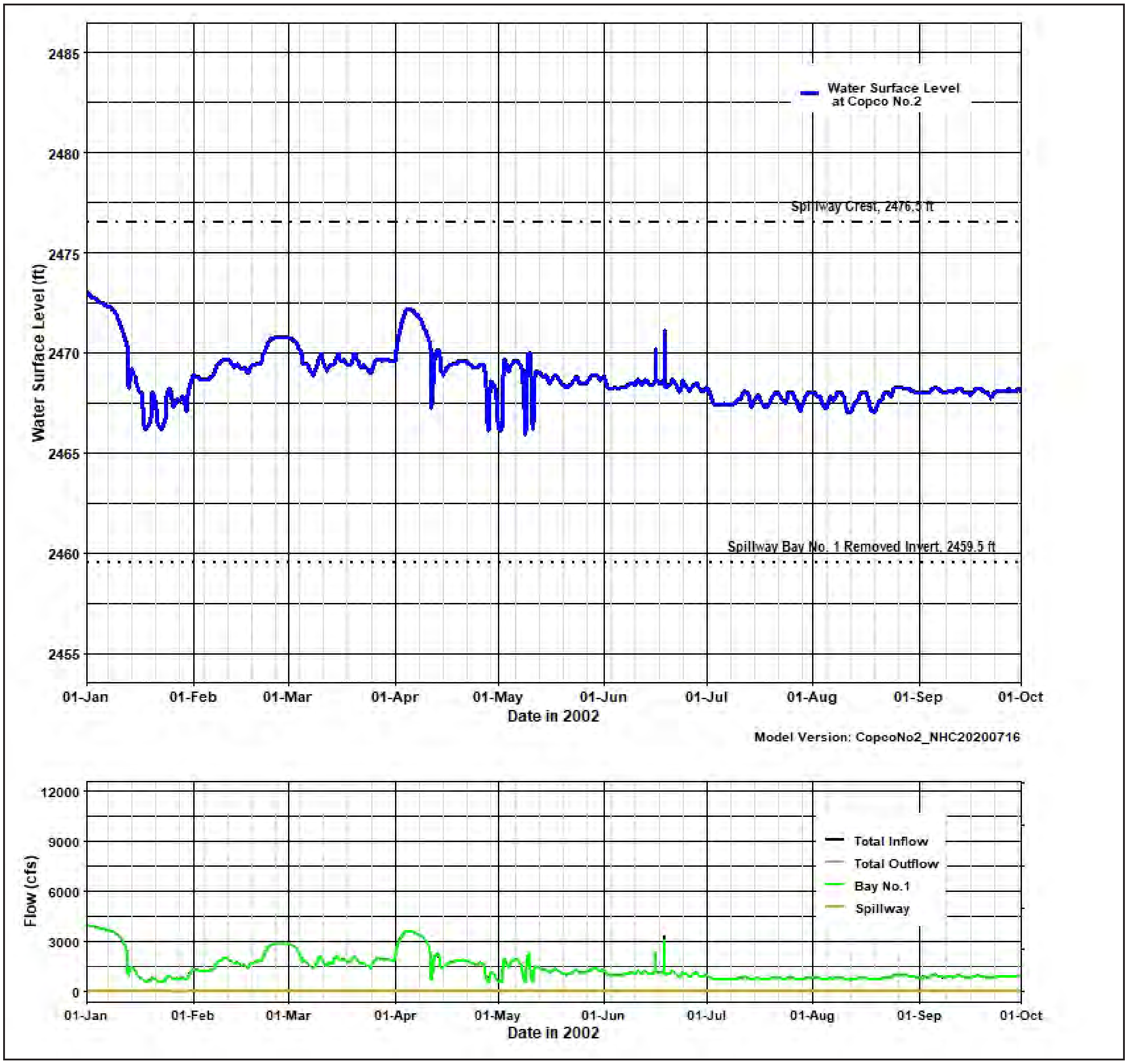

Figure 22 - COPCO No.2 Facility Simulated Drawdown - Year 2002

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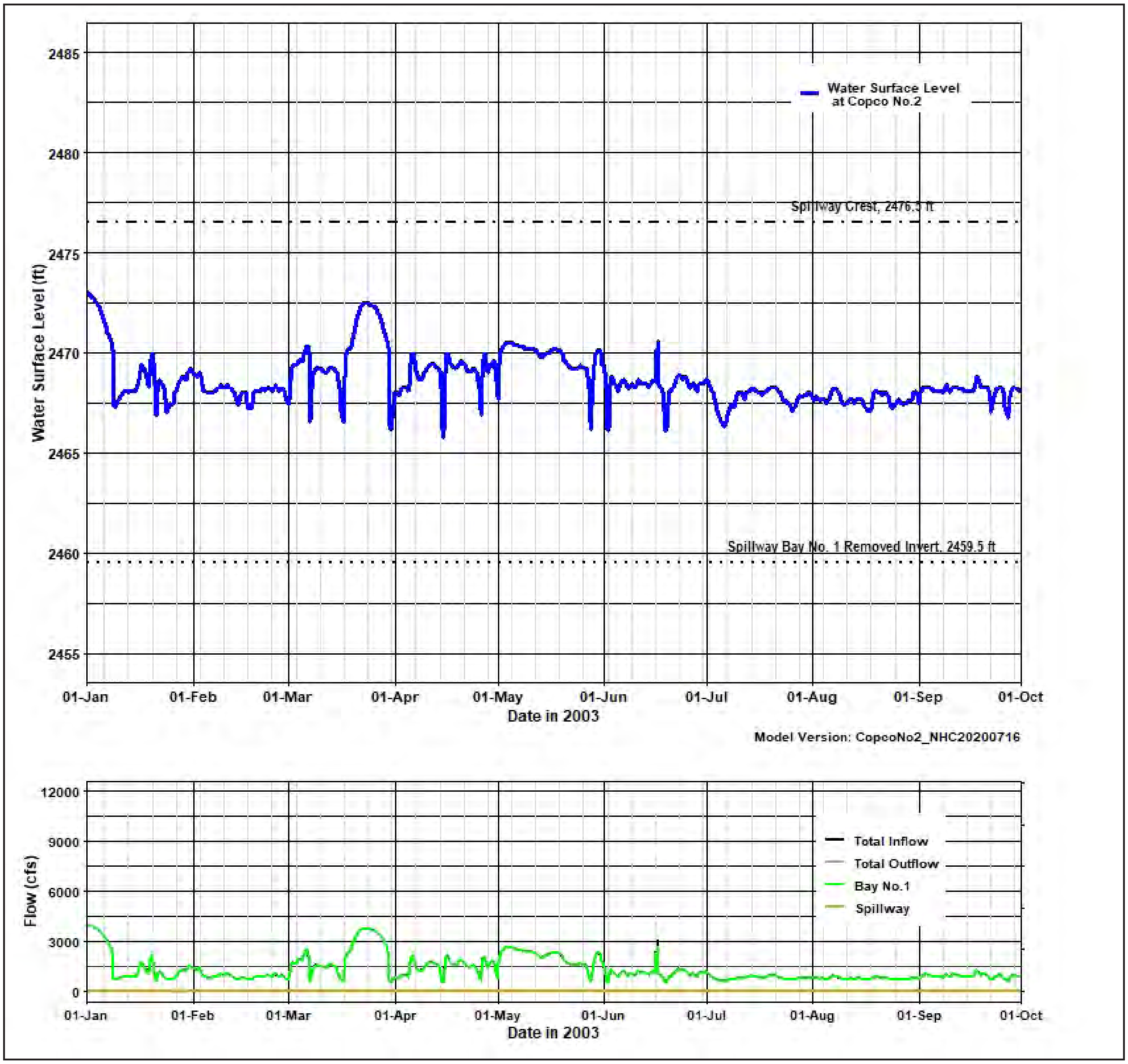

Figure 23 - COPCO No.2 Facility Simulated Drawdown - Year 2003

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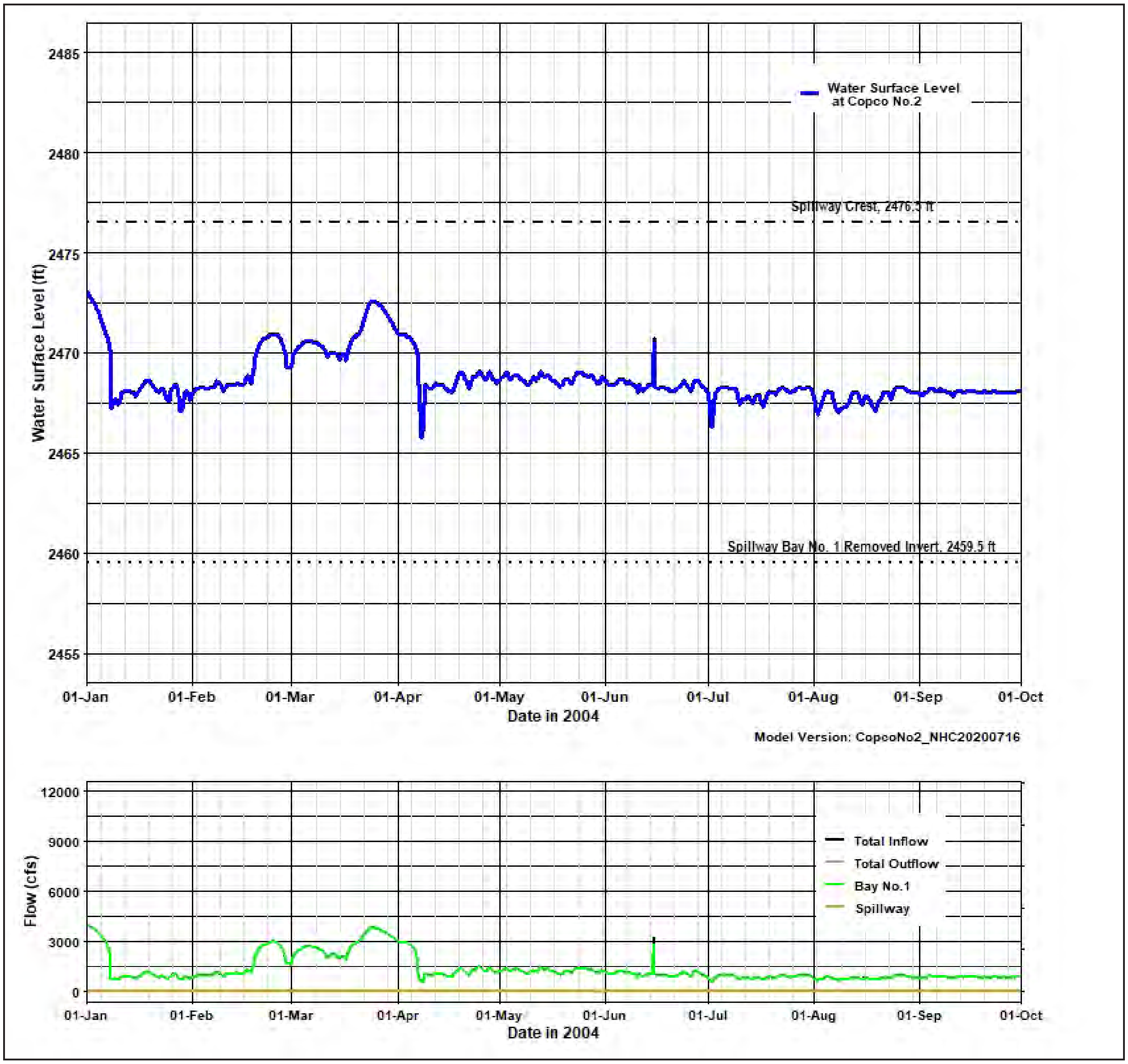

Figure 24 - COPCO No.2 Facility Simulated Drawdown - Year 2004

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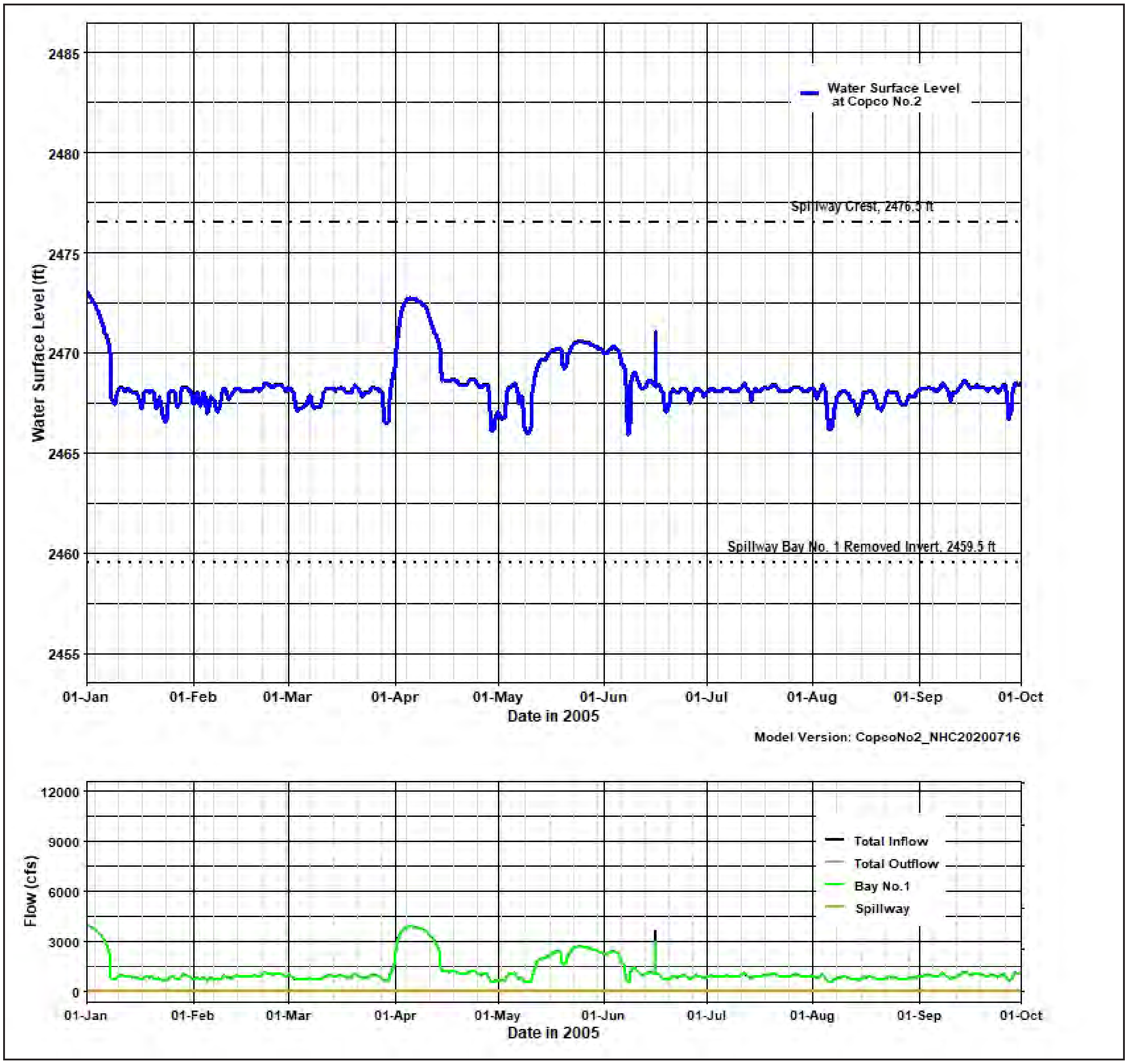

Figure 25 - COPCO No.2 Facility Simulated Drawdown - Year 2005

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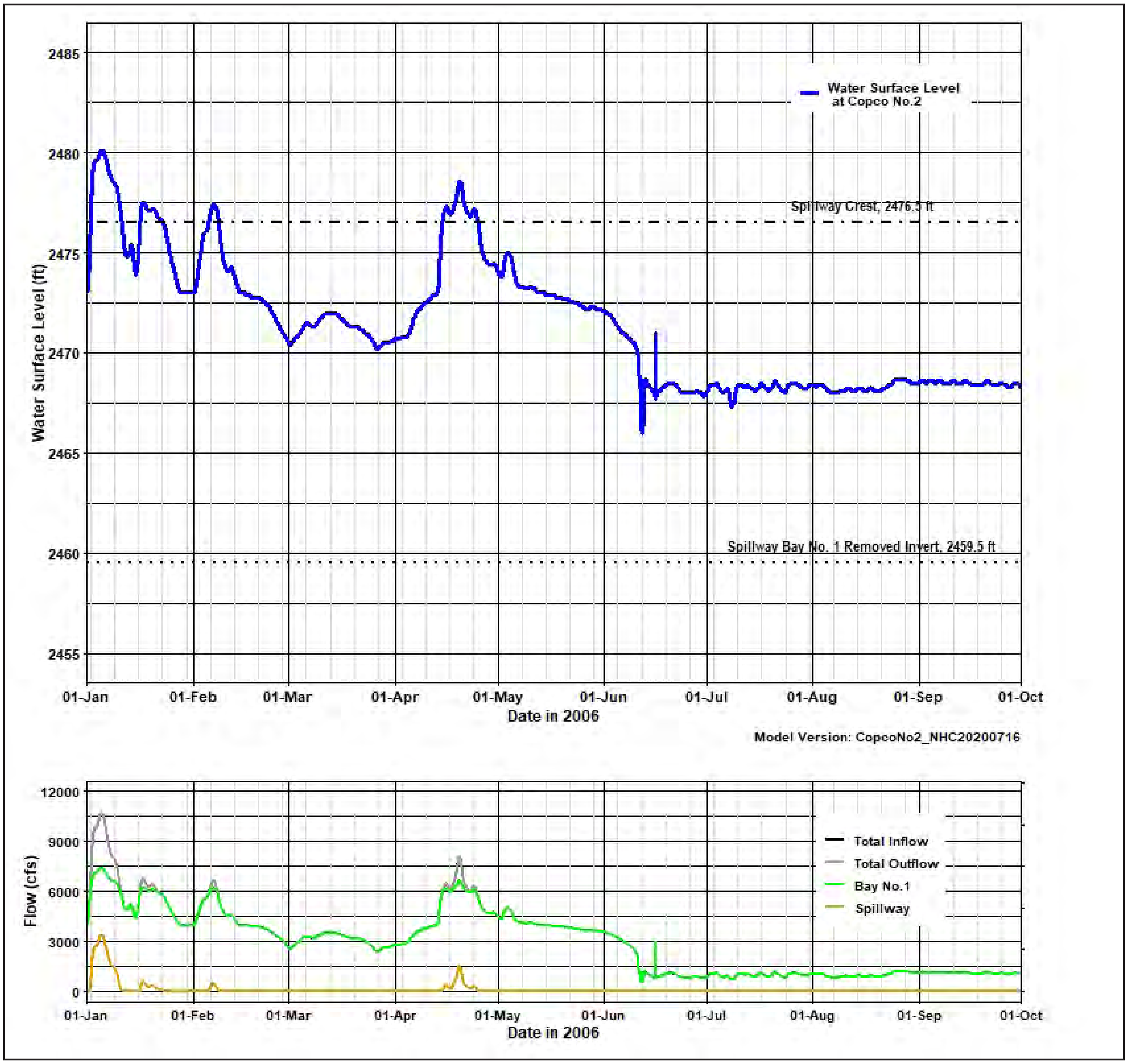

Figure 26 - COPCO No.2 Facility Simulated Drawdown - Year 2006

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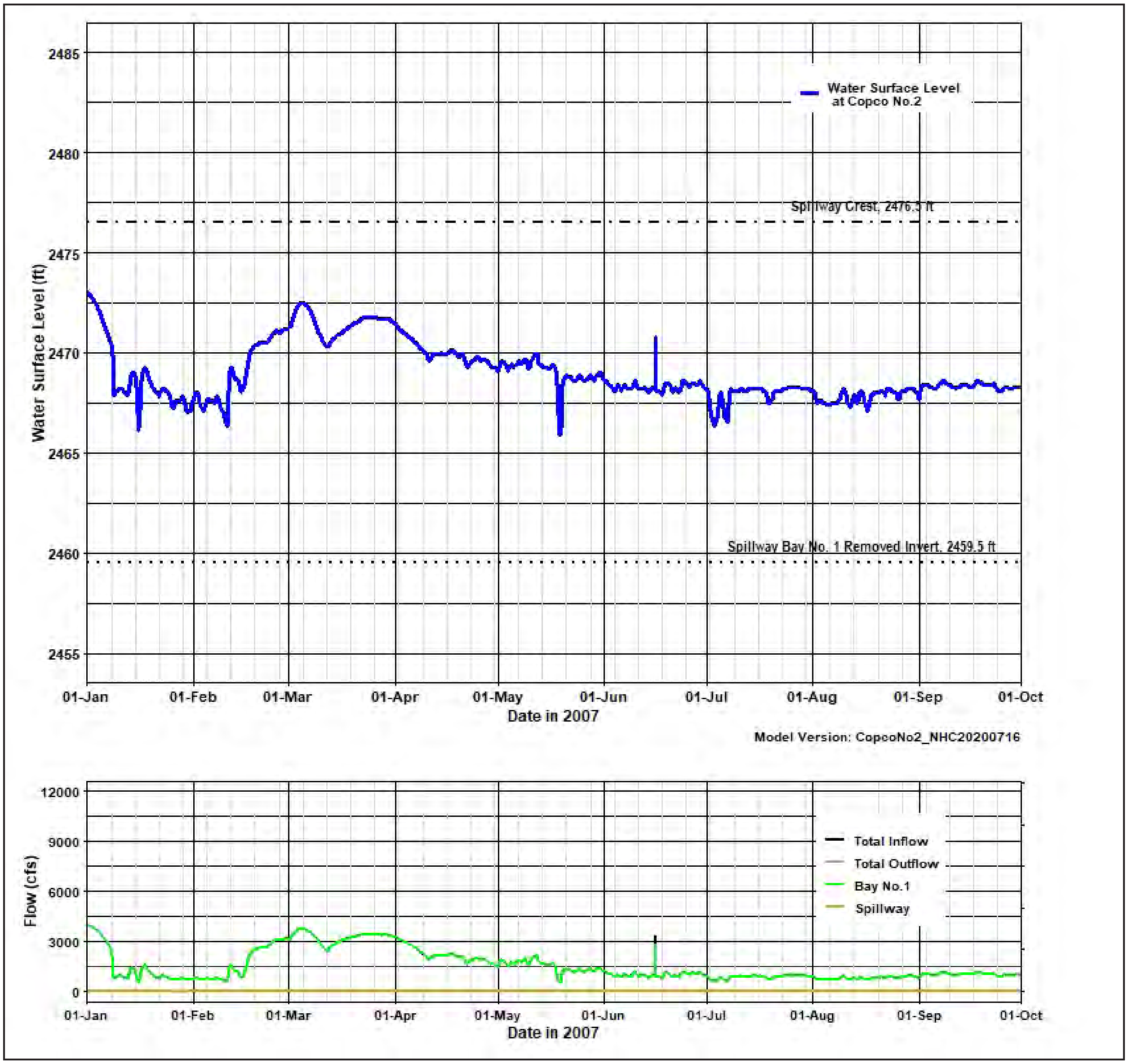

Figure 27 - COPCO No.2 Facility Simulated Drawdown - Year 2007

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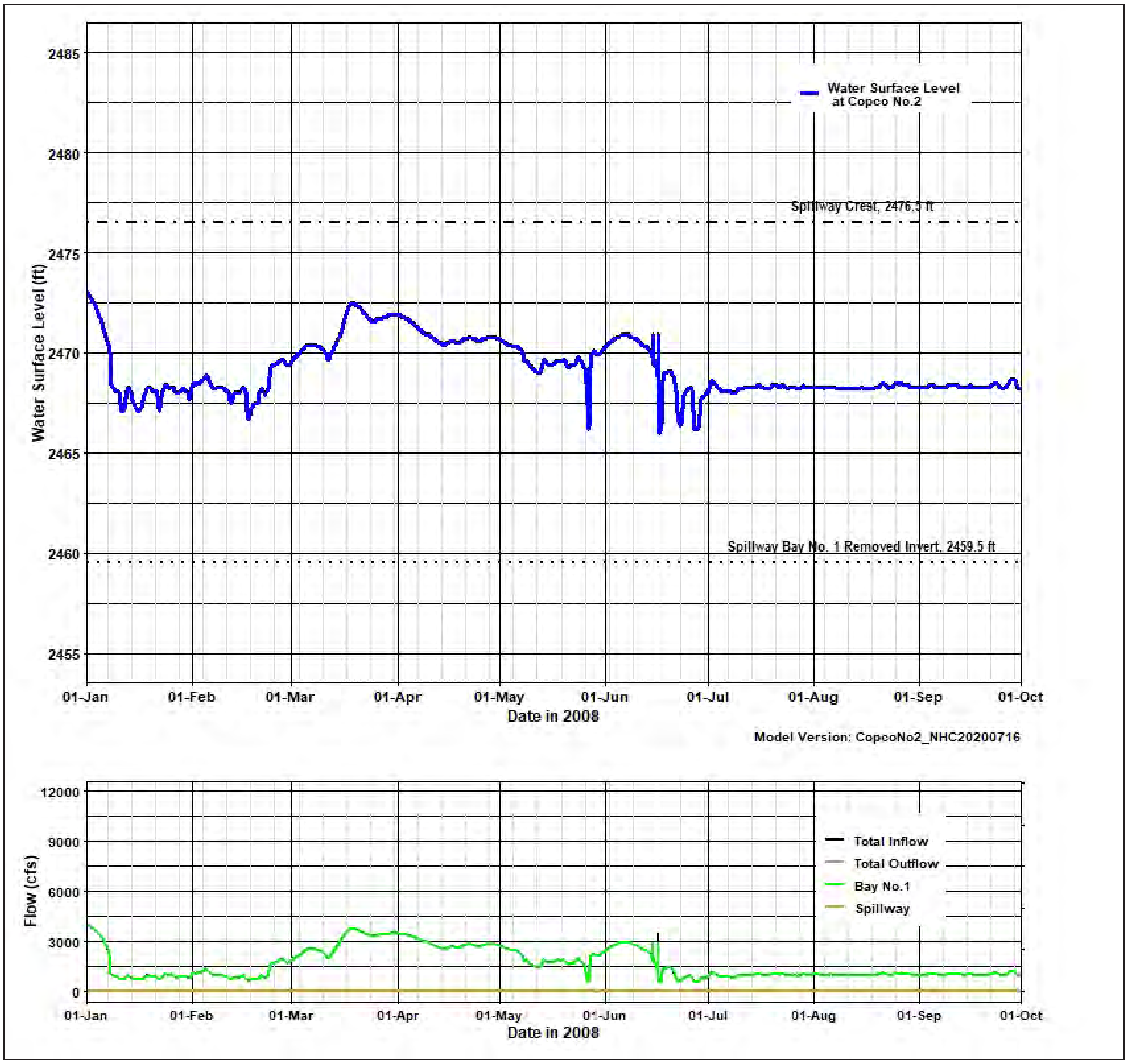

Figure 28 - COPCO No.2 Facility Simulated Drawdown - Year 2008

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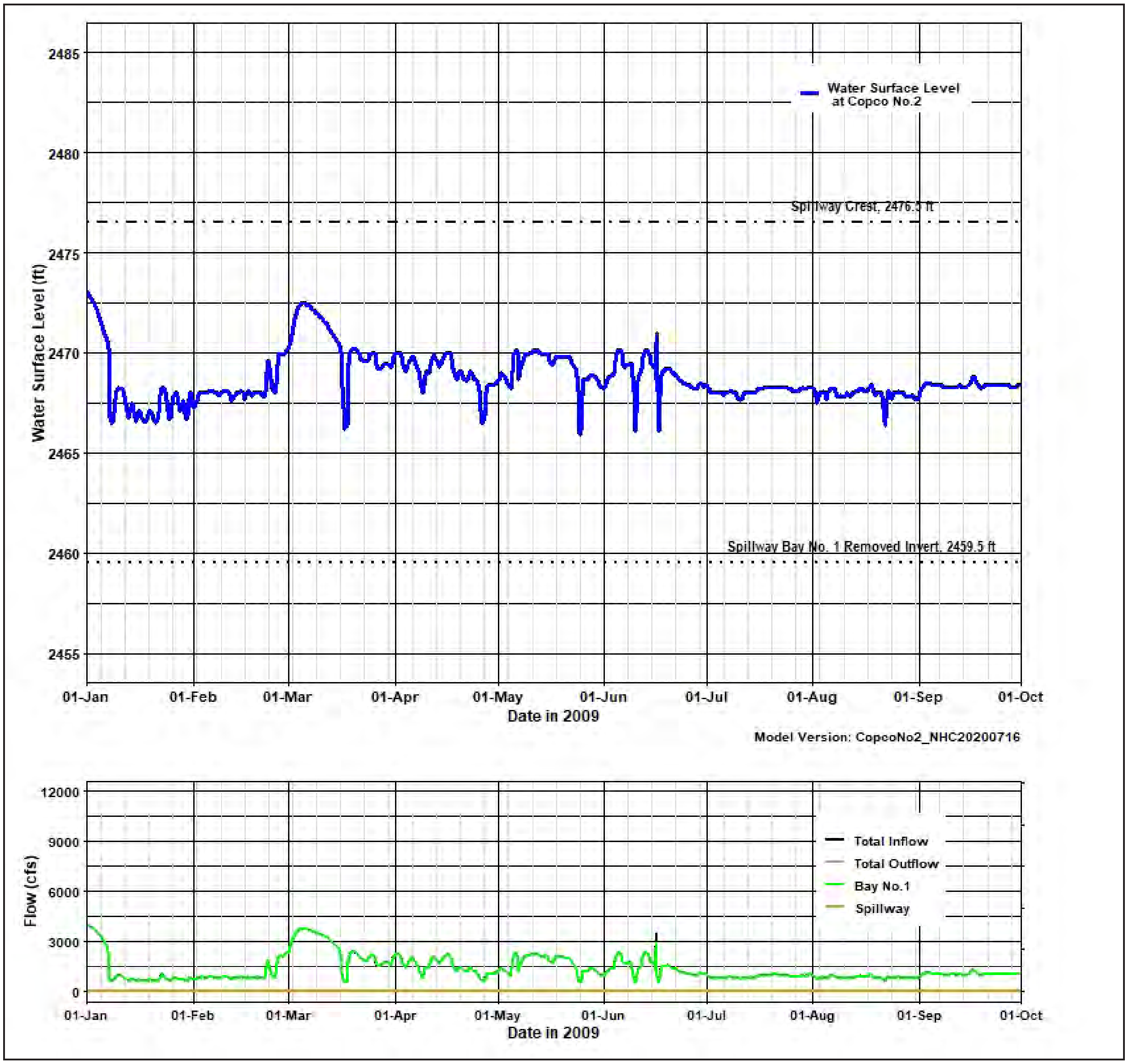

Figure 29 - COPCO No.2 Facility Simulated Drawdown - Year 2009

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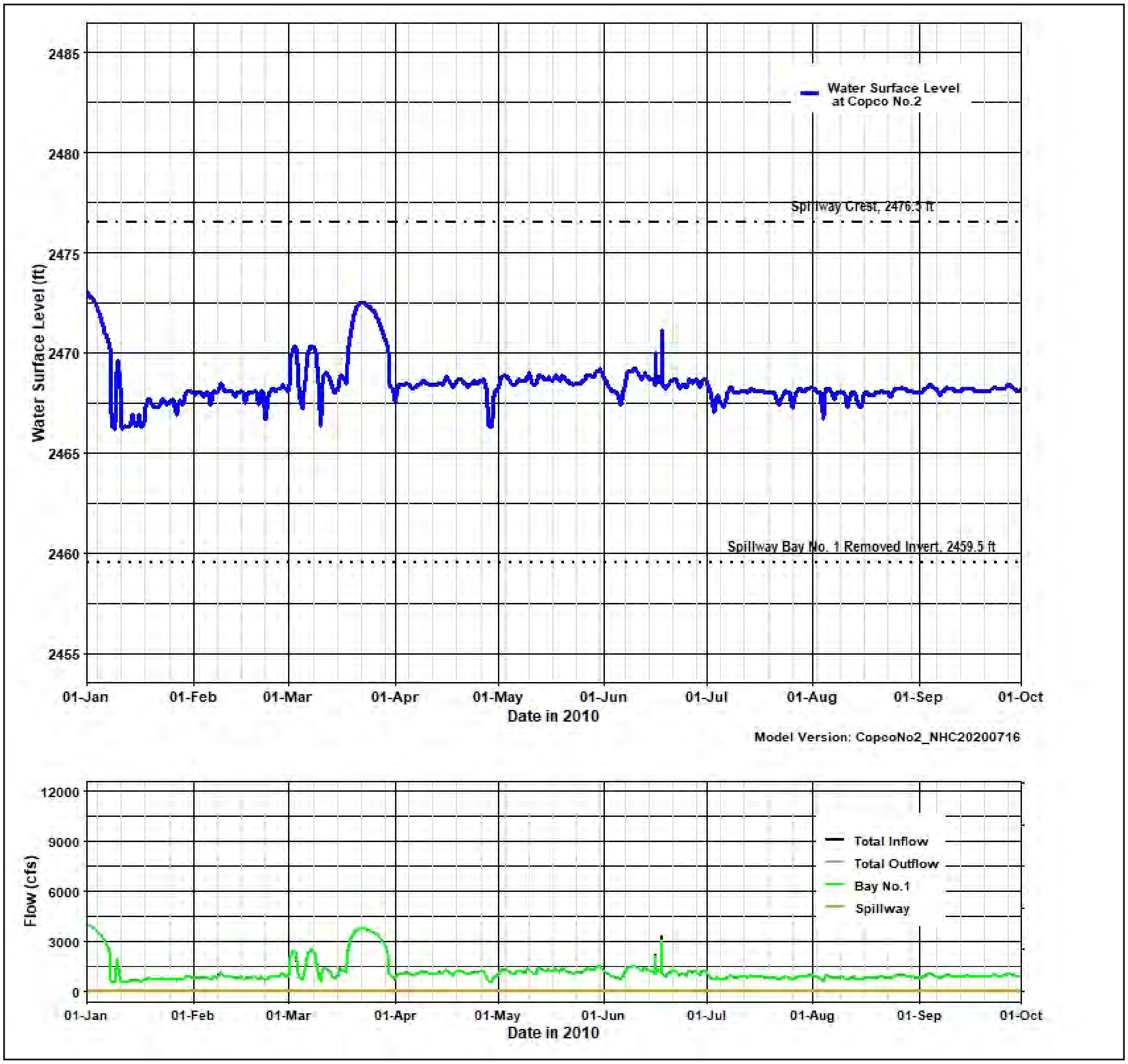

Figure 30 - COPCO No.2 Facility Simulated Drawdown - Year 2010

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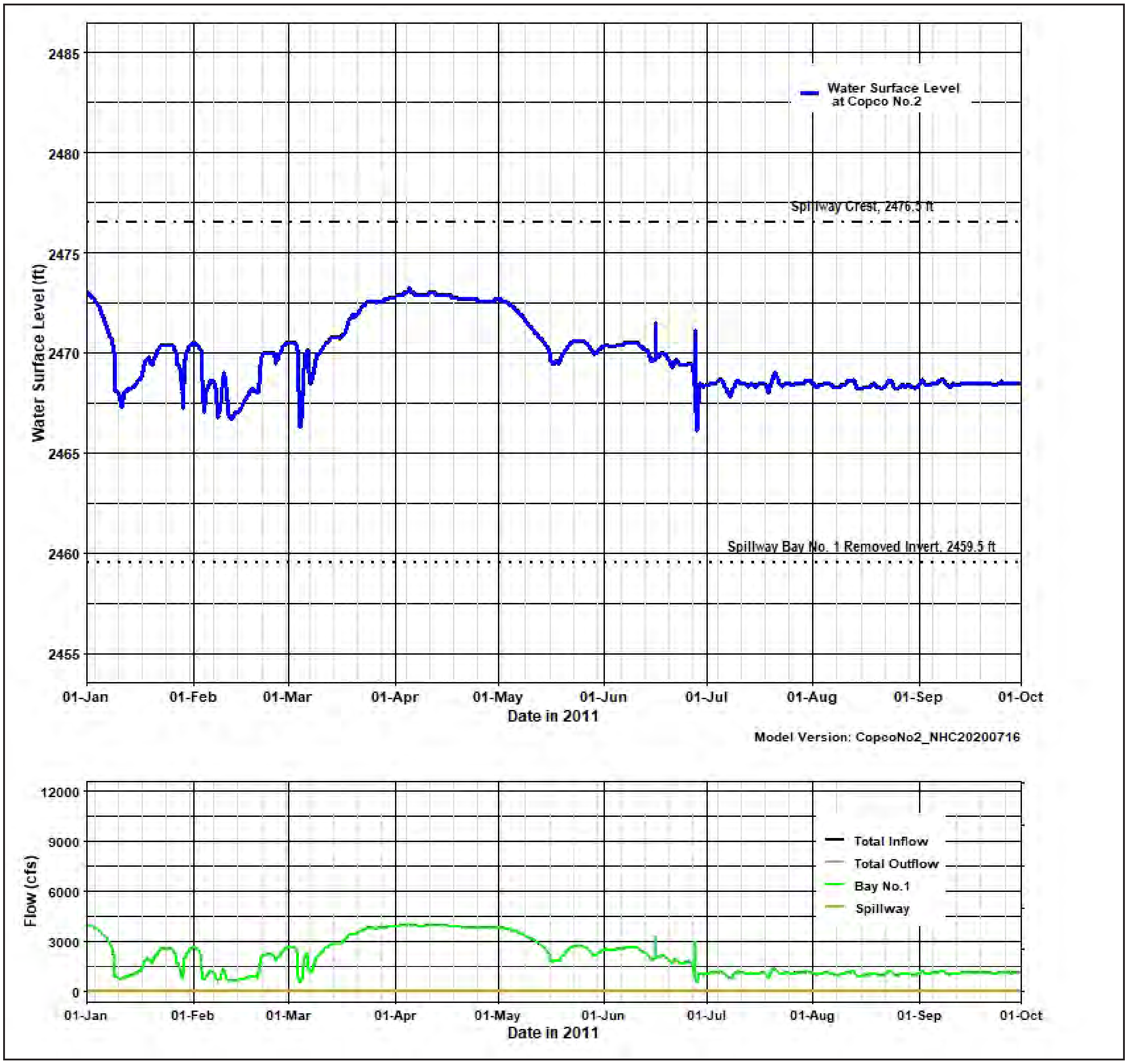

Figure 31 - COPCO No.2 Facility Simulated Drawdown - Year 2011

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COPCO NO.2 FACILITY SIMULATED DRAWDOWN

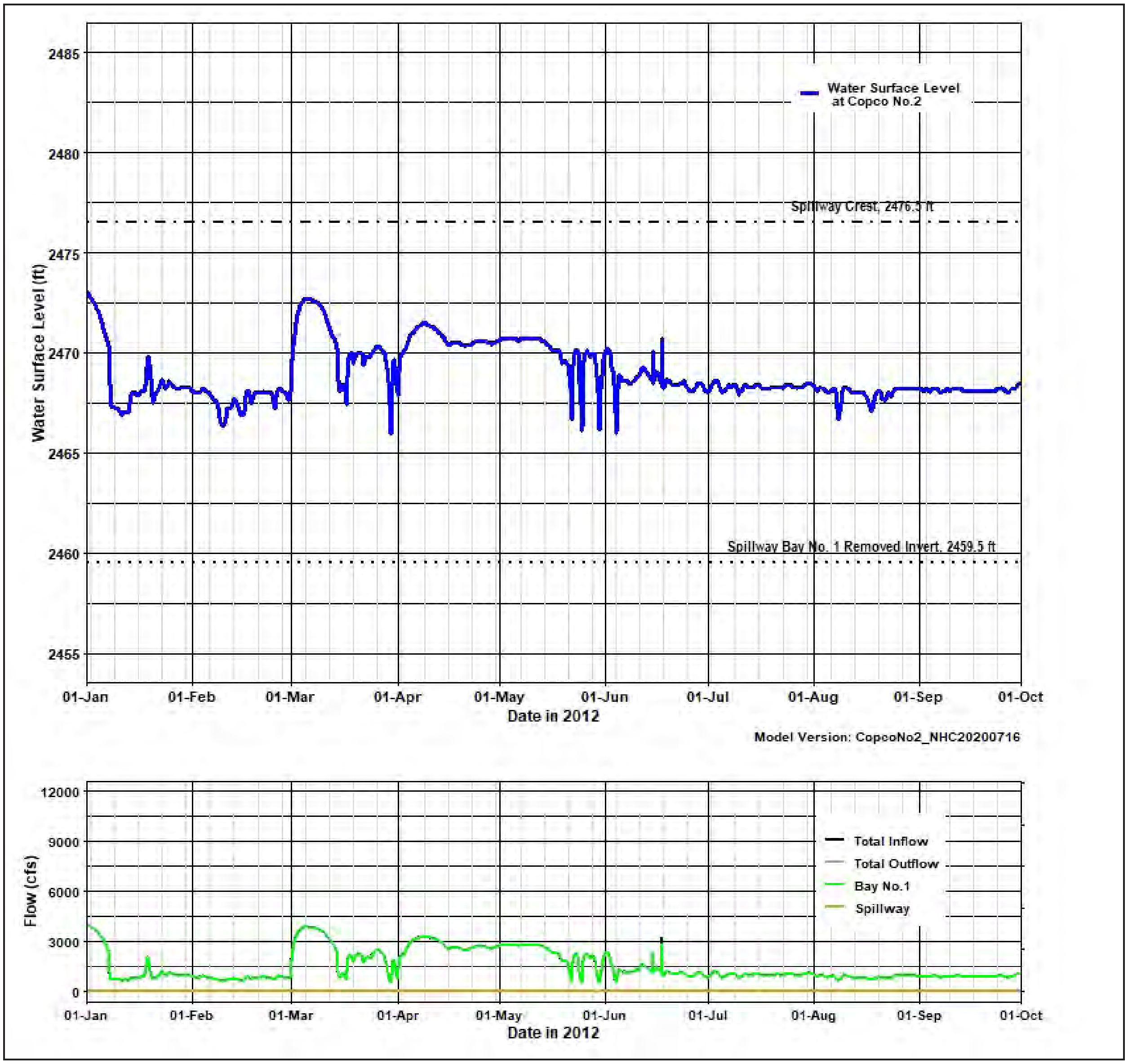

Figure 32 - COPCO No.2 Facility Simulated Drawdown - Year 2012

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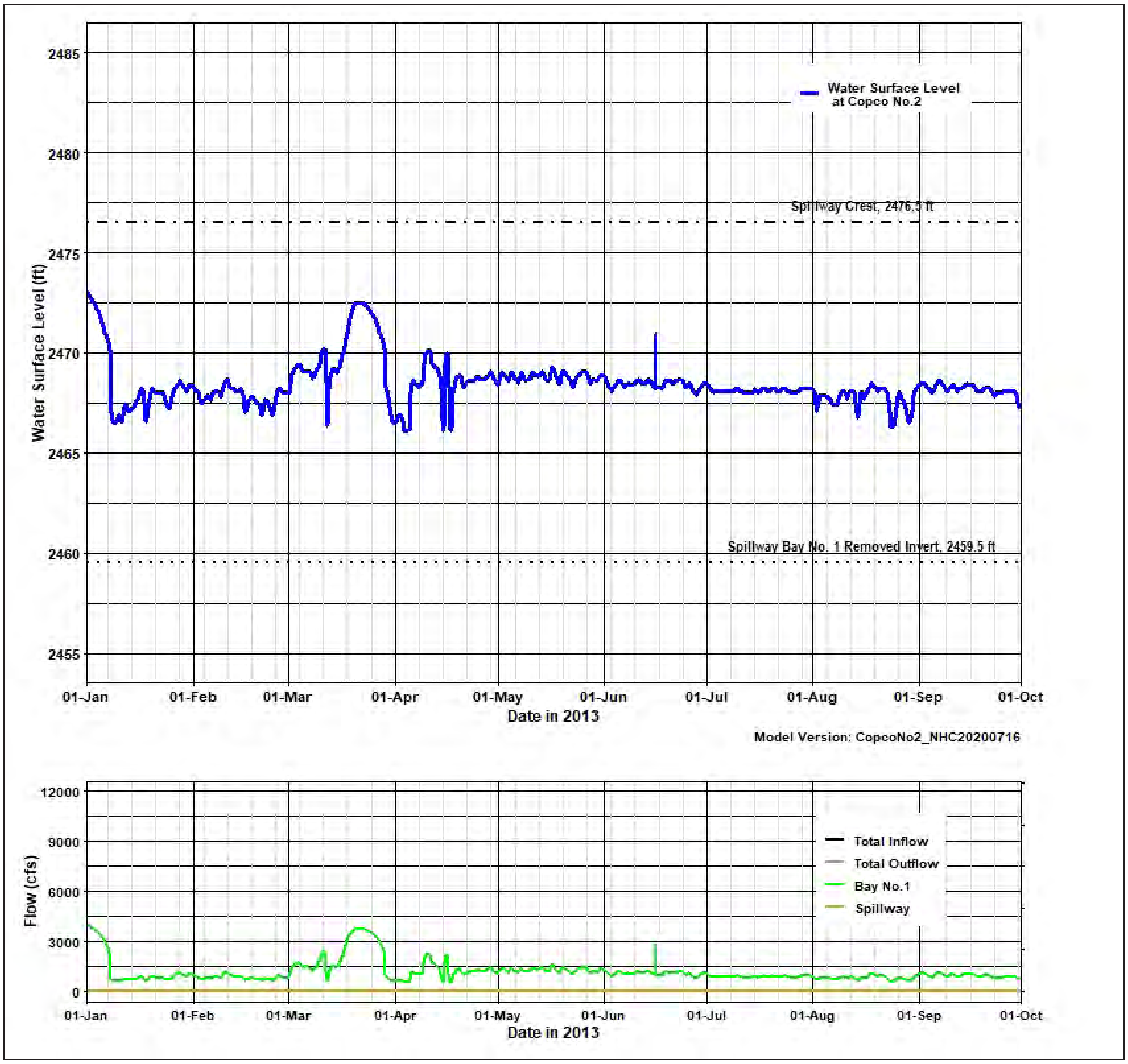

Figure 33 - COPCO No.2 Facility Simulated Drawdown - Year 2013

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COPCO NO.2 FACILITY SIMULATED DRAWDOWN

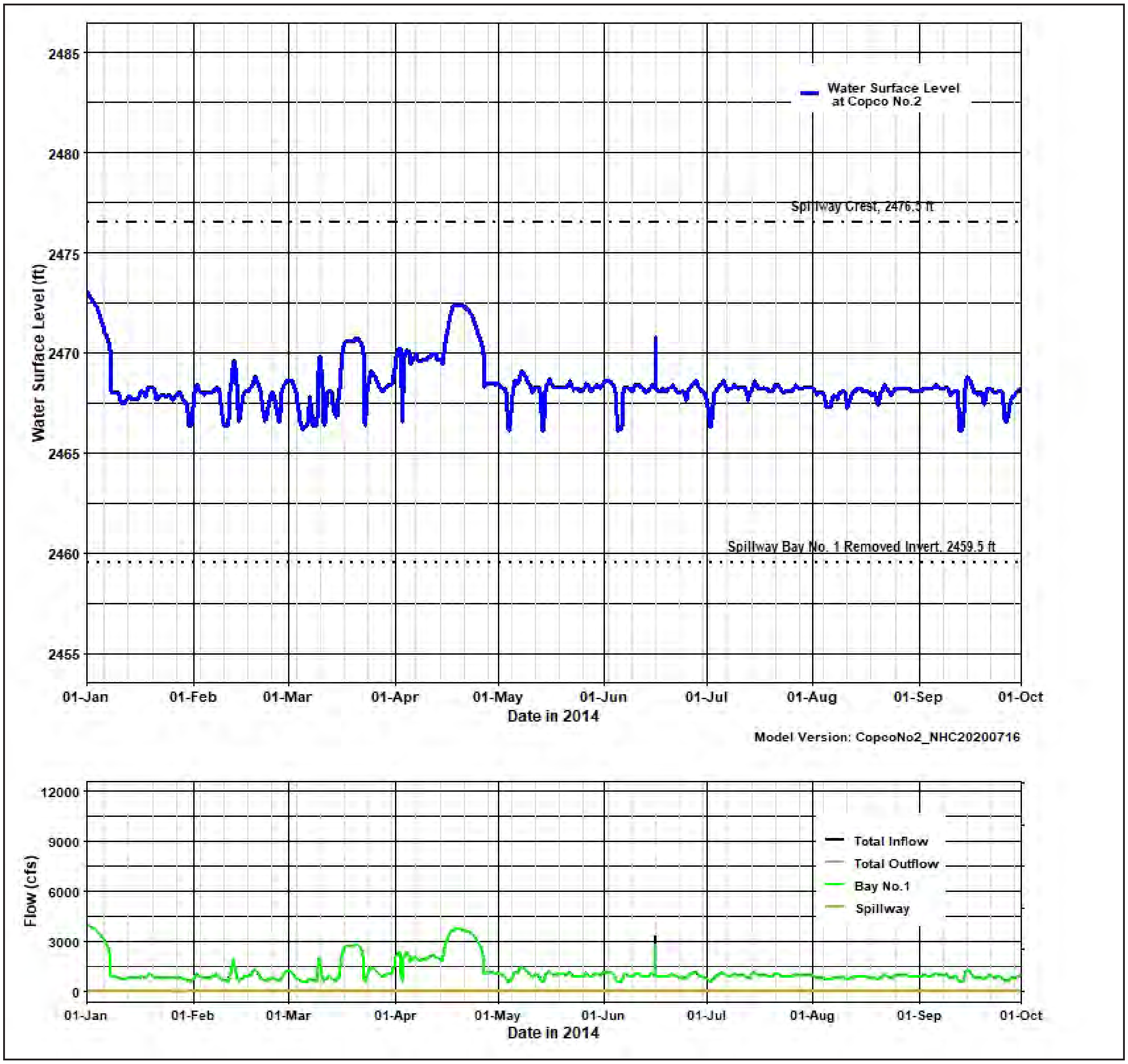

Figure 34 - COPCO No.2 Facility Simulated Drawdown - Year 2014

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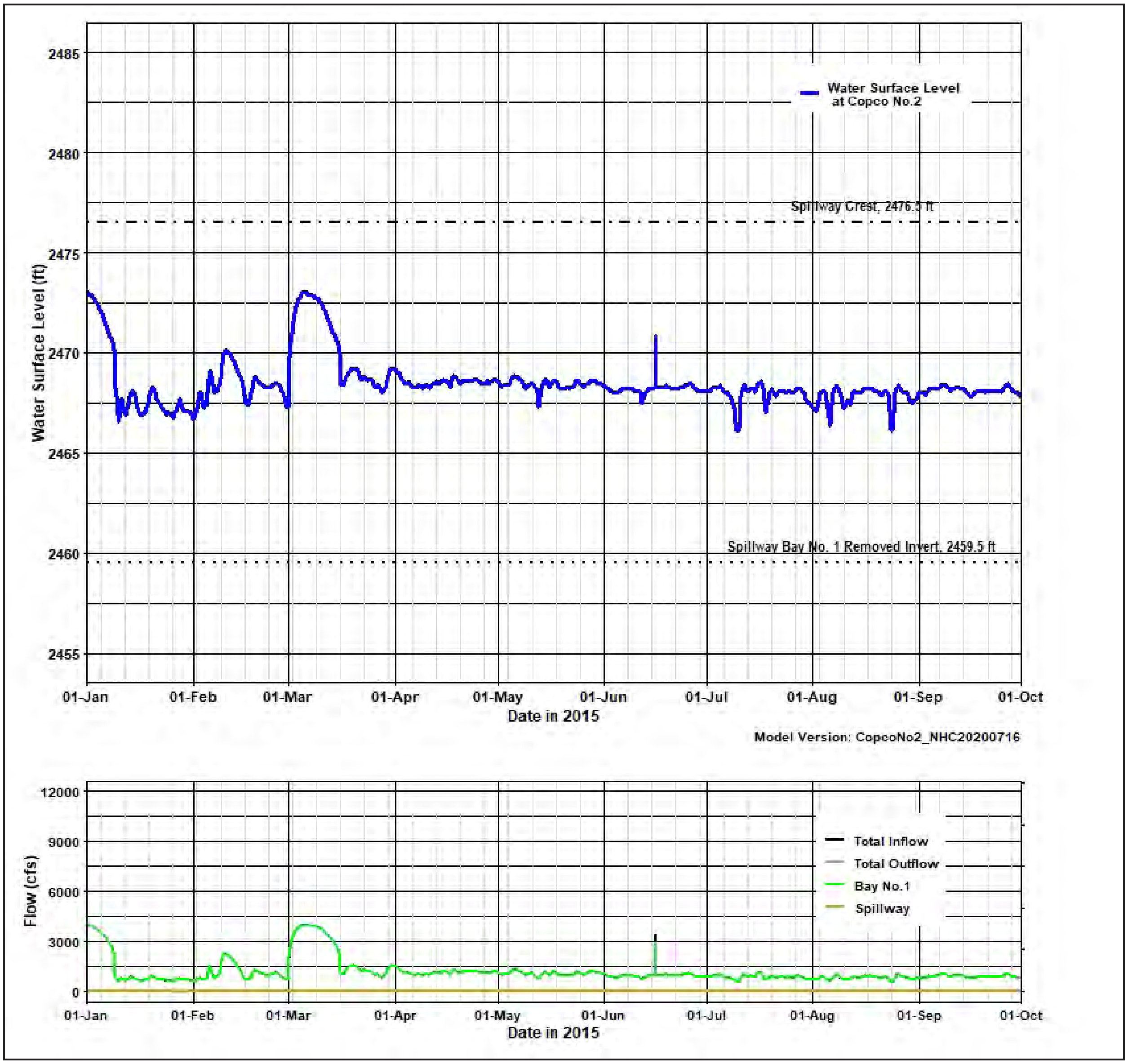

Figure 35 - COPCO No.2 Facility Simulated Drawdown - Year 2015

July 29, 2020

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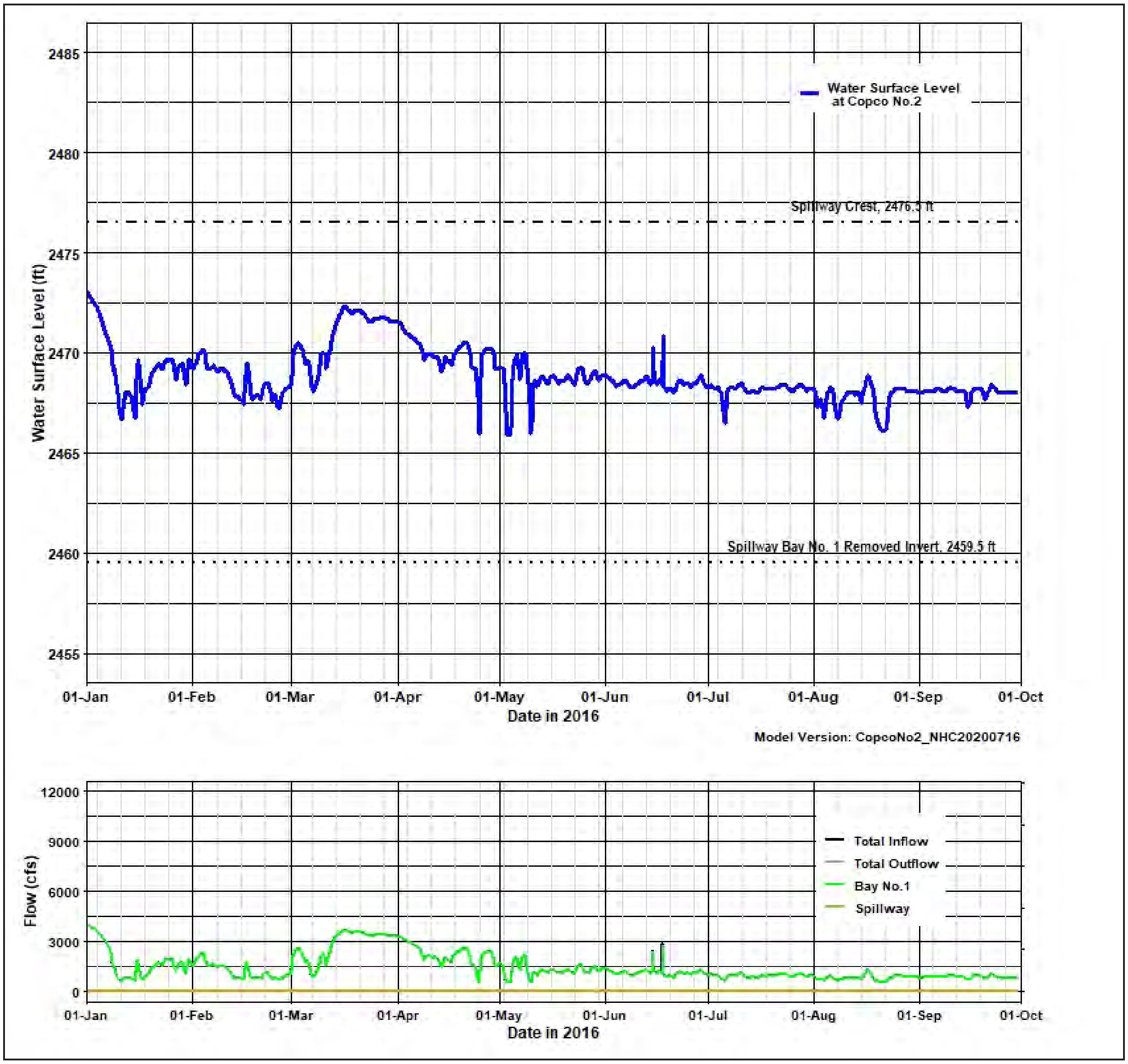

Figure 36 - COPCO No.2 Facility Simulated Drawdown - Year 2016

July 29, 2020

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EXHIBIT A

100% FINAL Design Report_Appendix D1(June2022) (CEII)
(pages 73 & 74 of 74)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

PAGES REDACTED IN ENTIRETY

The redacted material qualifies as CEII pursuant to the Commission's rules because it contains sensitive dam safety and construction information that (a) relates details about the production, generation, transmission, or distribution of energy, (b) could be useful to a person planning an attack on critical infrastructure, (c) is exempt from mandatory disclosure under the Freedom of Information Act, and (d) gives strategic information beyond the location of the critical infrastructure. Accordingly, the Renewal Corporation has requested confidential treatment of this material pursuant to 18 C.F.R. § 388.113.

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

nhc
northwest hydraulic consultants

NHC Ref. No. 2004947.11.6

21 September 2020

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

Attention: Norm Bishop

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

DRAWDOWN OPERATIONS – 100% DESIGN – CONTINGENCY REMOVAL METHOD

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

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

COMPUTATIONAL FLUID DYNAMICS MODELING

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

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

GEOMETRY AND ROUGHNESS

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

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

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

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

MESH INDEPENDENCE

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

Table 1. Mesh Independence Comparison - Spillway Capacity

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

PRE-DRAWDOWN RESULTS

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

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

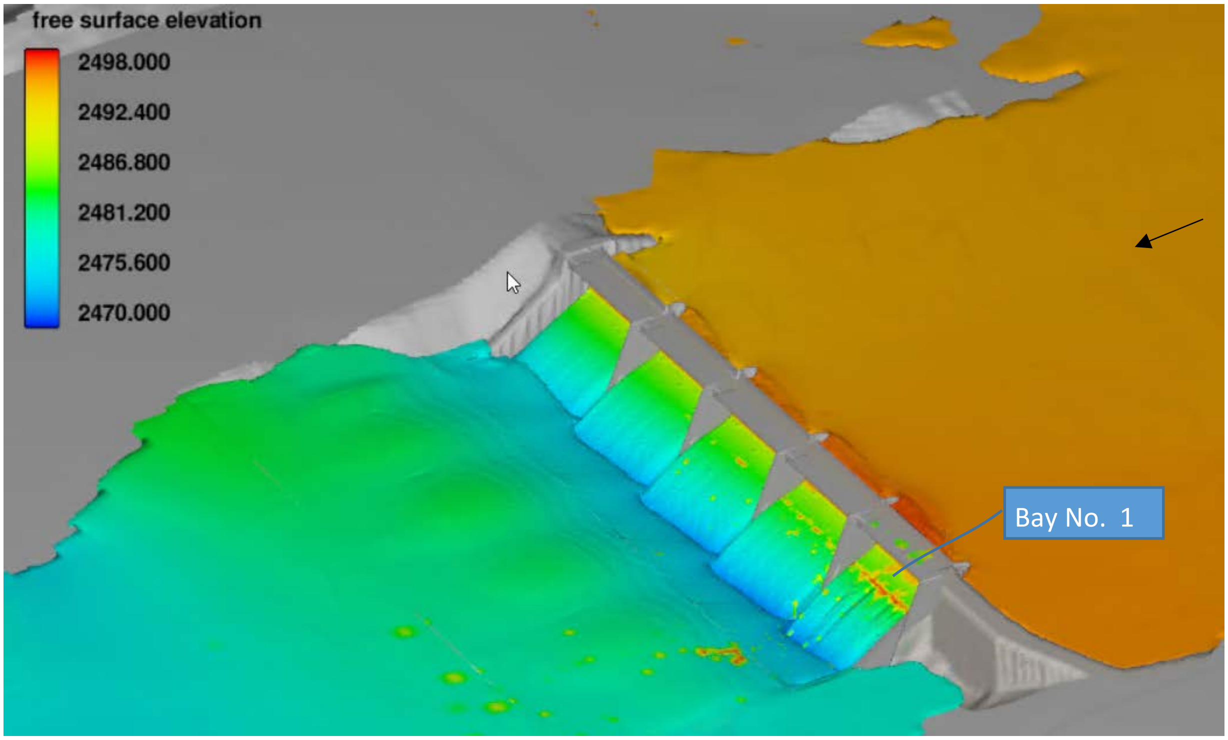


Figure 1. Water Surface Elevation (ft) for a discharge of 29,400 cfs with five gates fully open.

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

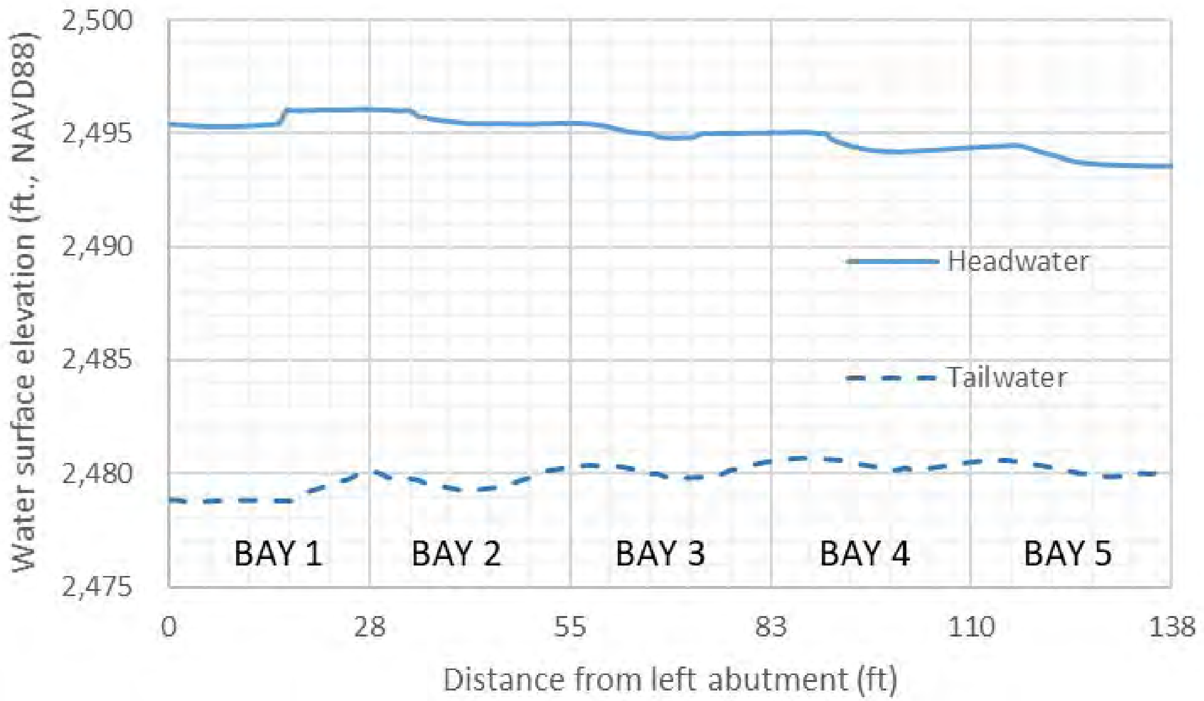


Figure 2. Pre-Drawdown headwater and tailwater elevations for a discharge of 29,400 cfs with five gates fully open (view looking downstream).

SPILLWAY BAY NO. 1 REMOVAL RESULTS

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

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

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

Table 2. Simulated Drawdown Conditions

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

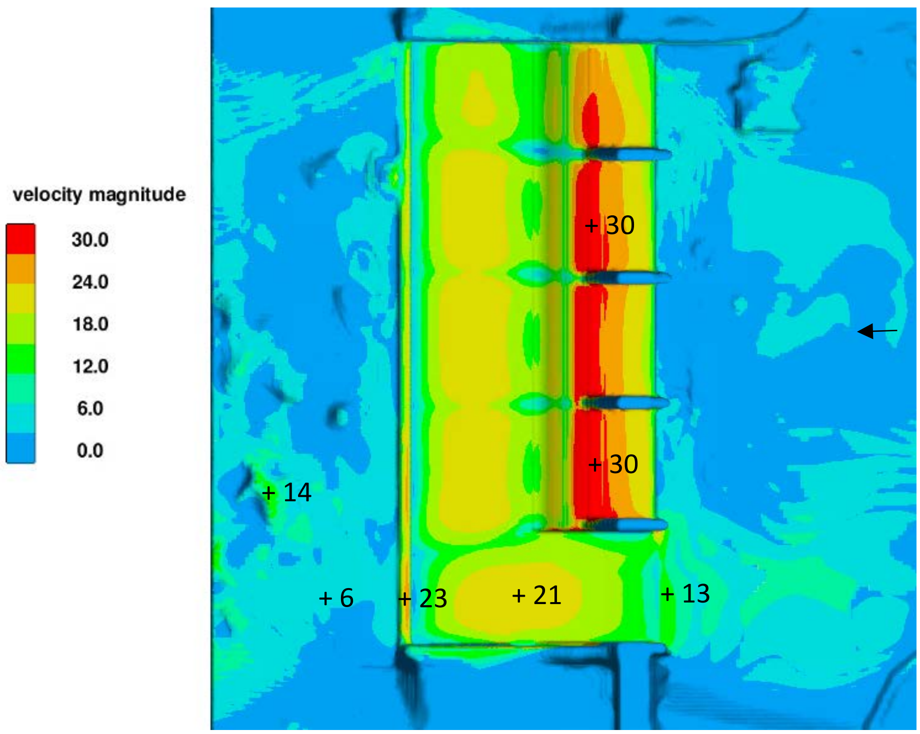


Figure 3. Near-bed maximum velocity during drawdown for 29,400 cfs. Crosses indicate point velocity in ft/s.

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

DIVERSION DAM REMOVAL RESULTS

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

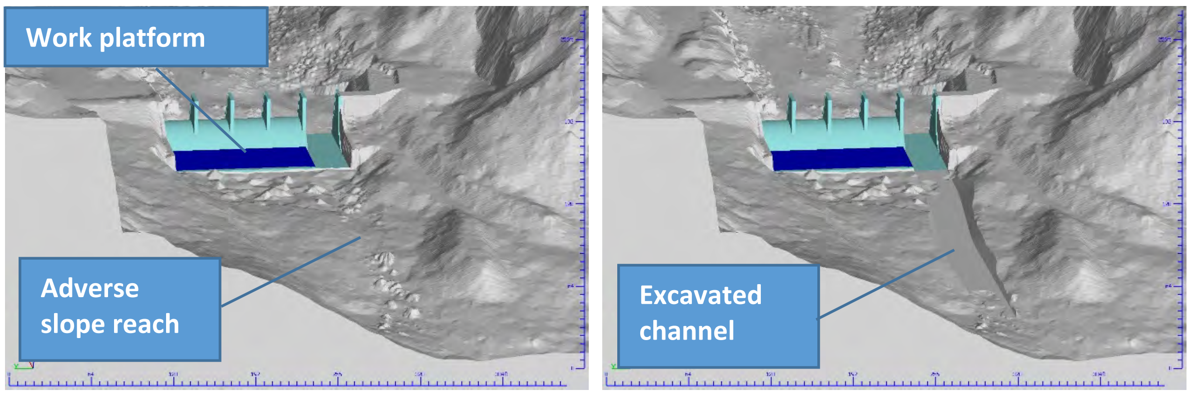


Figure 4. Geometry comparison (left, existing conditions downstream; right, excavated downstream channel)

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

Table 3. Simulated Diversion Dam Removal Conditions

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

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

WORK PLATFORM WATER LEVEL ANALYSIS

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

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

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

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

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

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

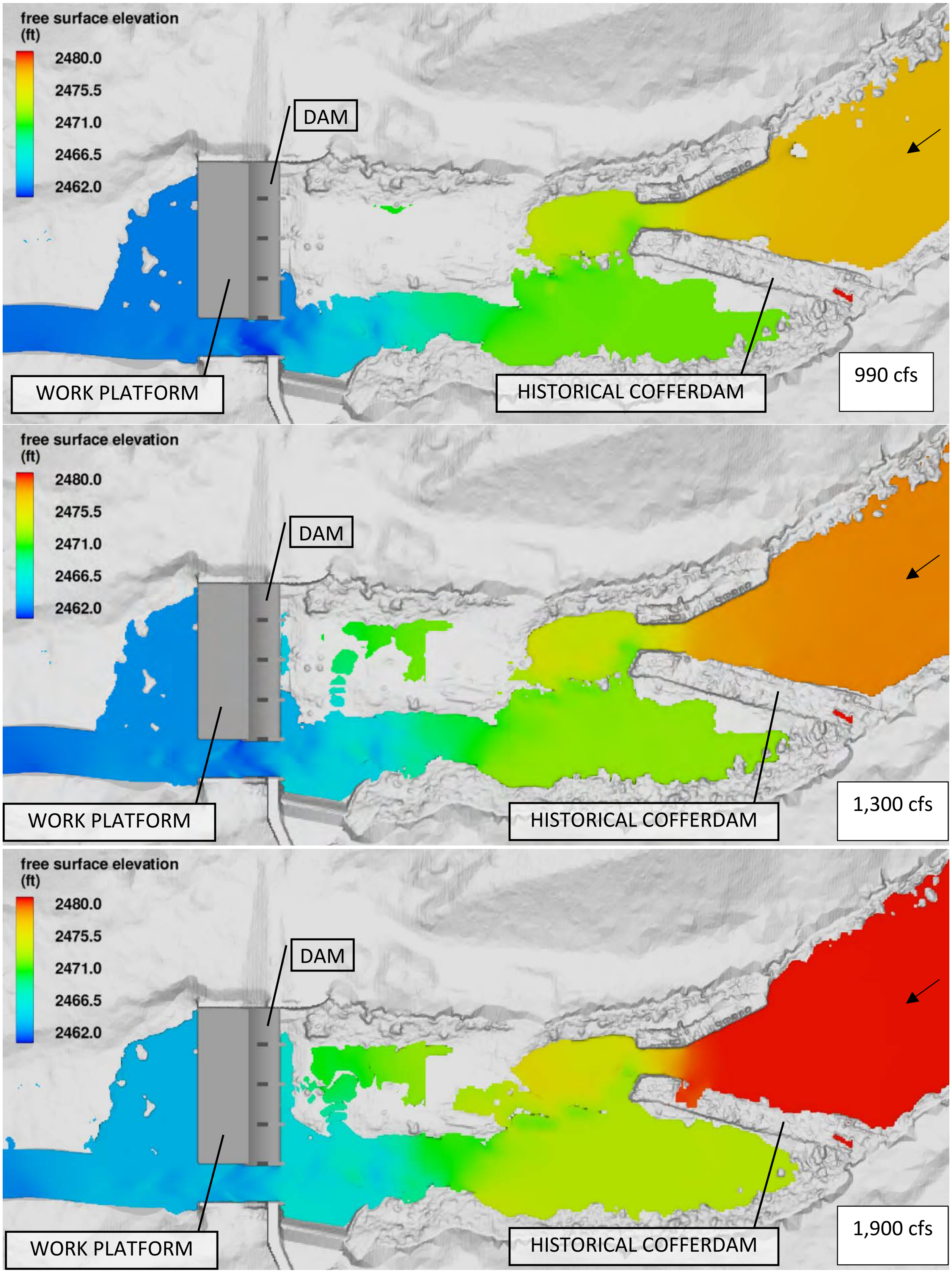


Figure 5. Water surface elevation for the excavated channel condition and work platform at El. 2,466 ft

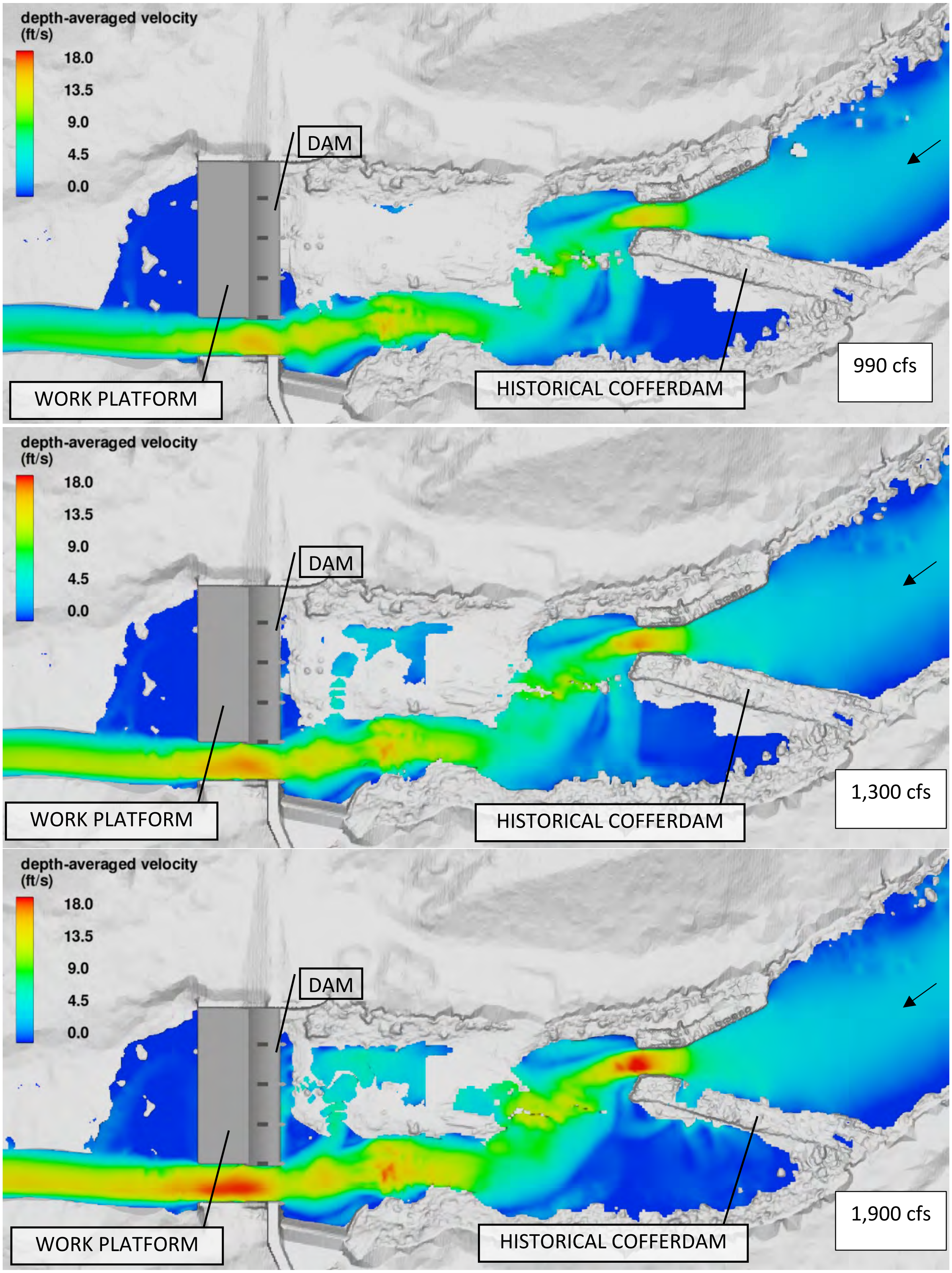


Figure 6. Depth-averaged velocity for the excavated channel condition and work platform at El. 2,466 ft

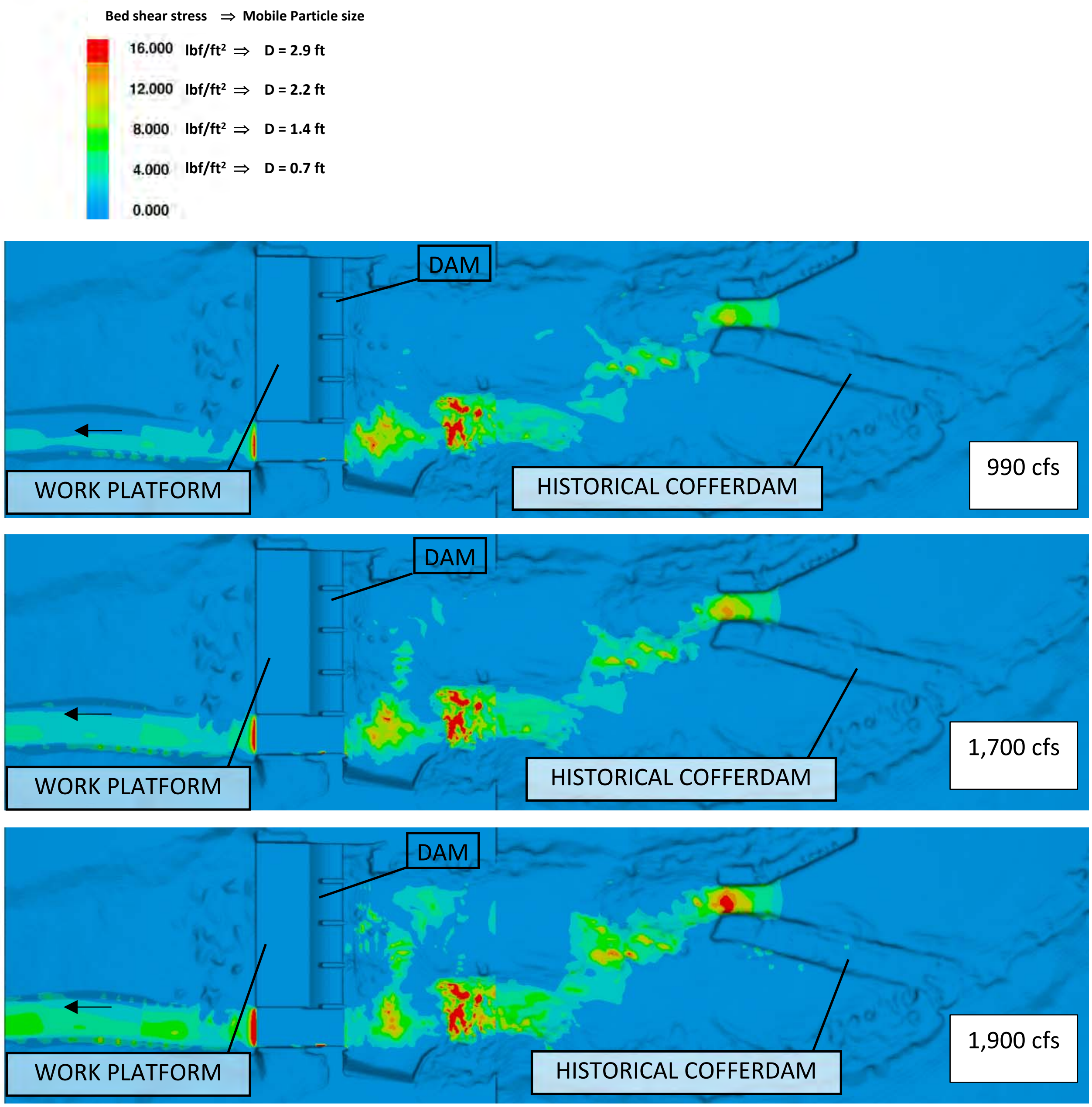


Figure 7. Bed shear stress for the excavated channel conditions

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

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

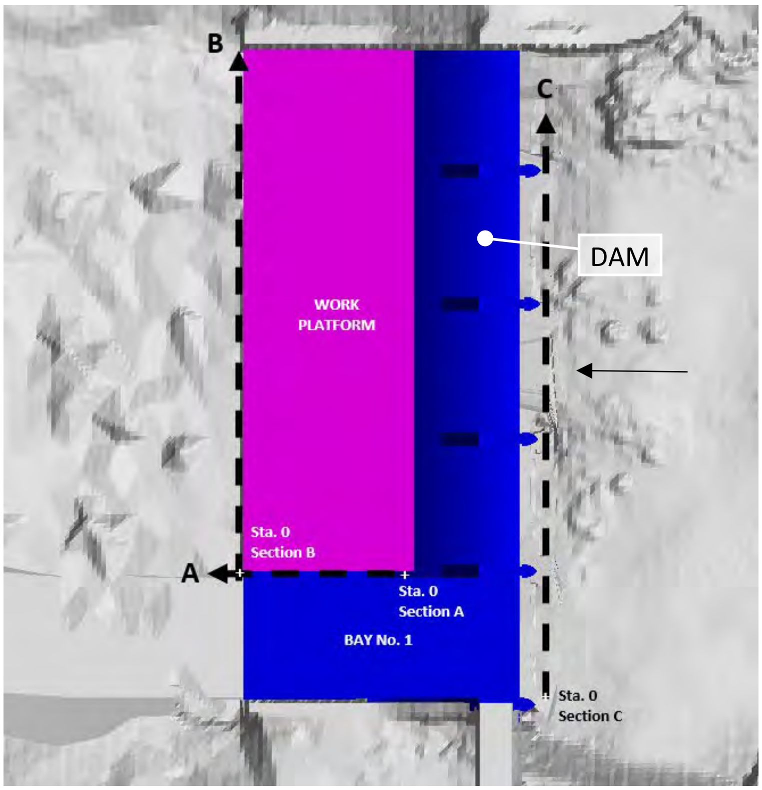
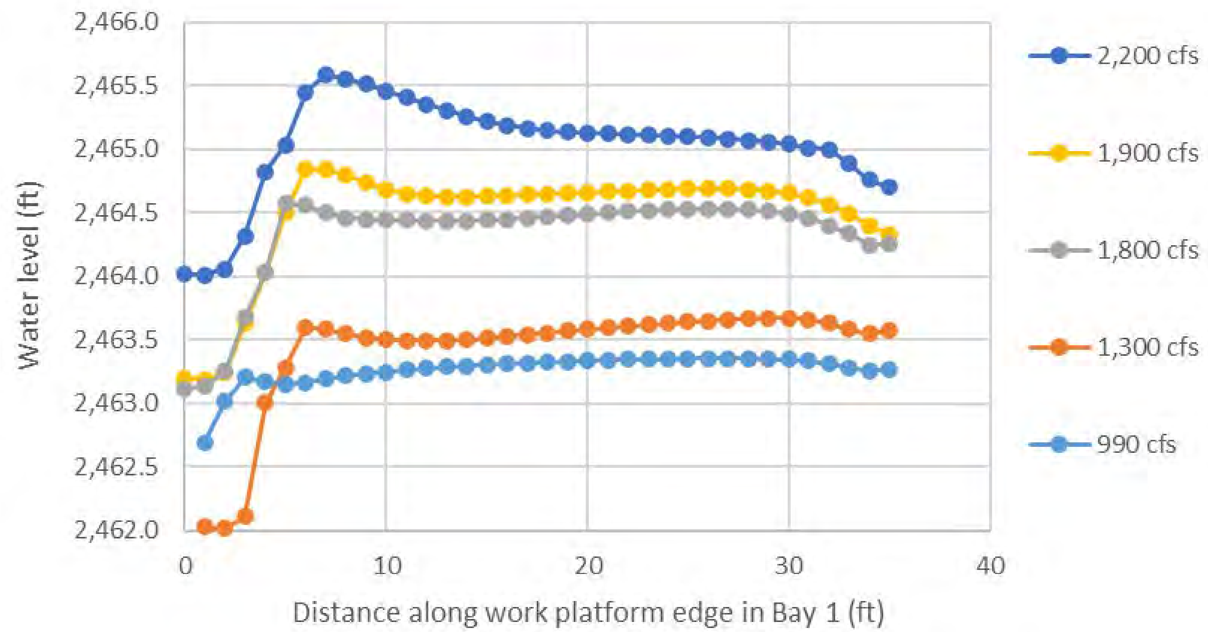


Figure 8. Sections A, B, and C along perimeter of work platform.

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



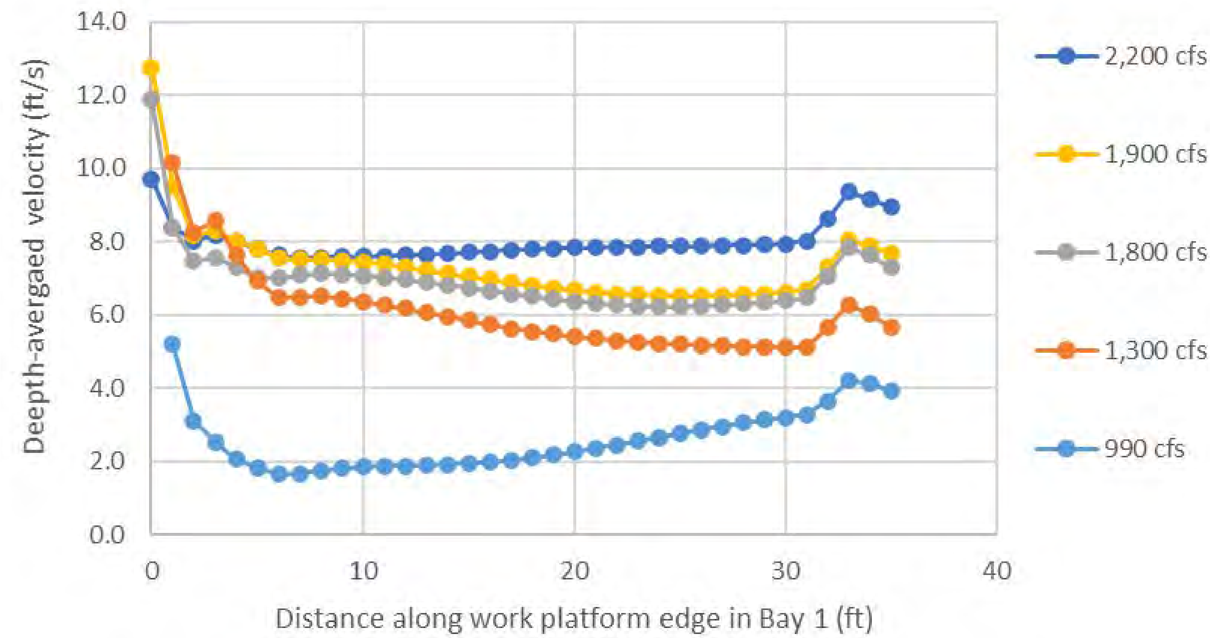


Figure 9. Water levels and depth-averaged velocity along Section A.

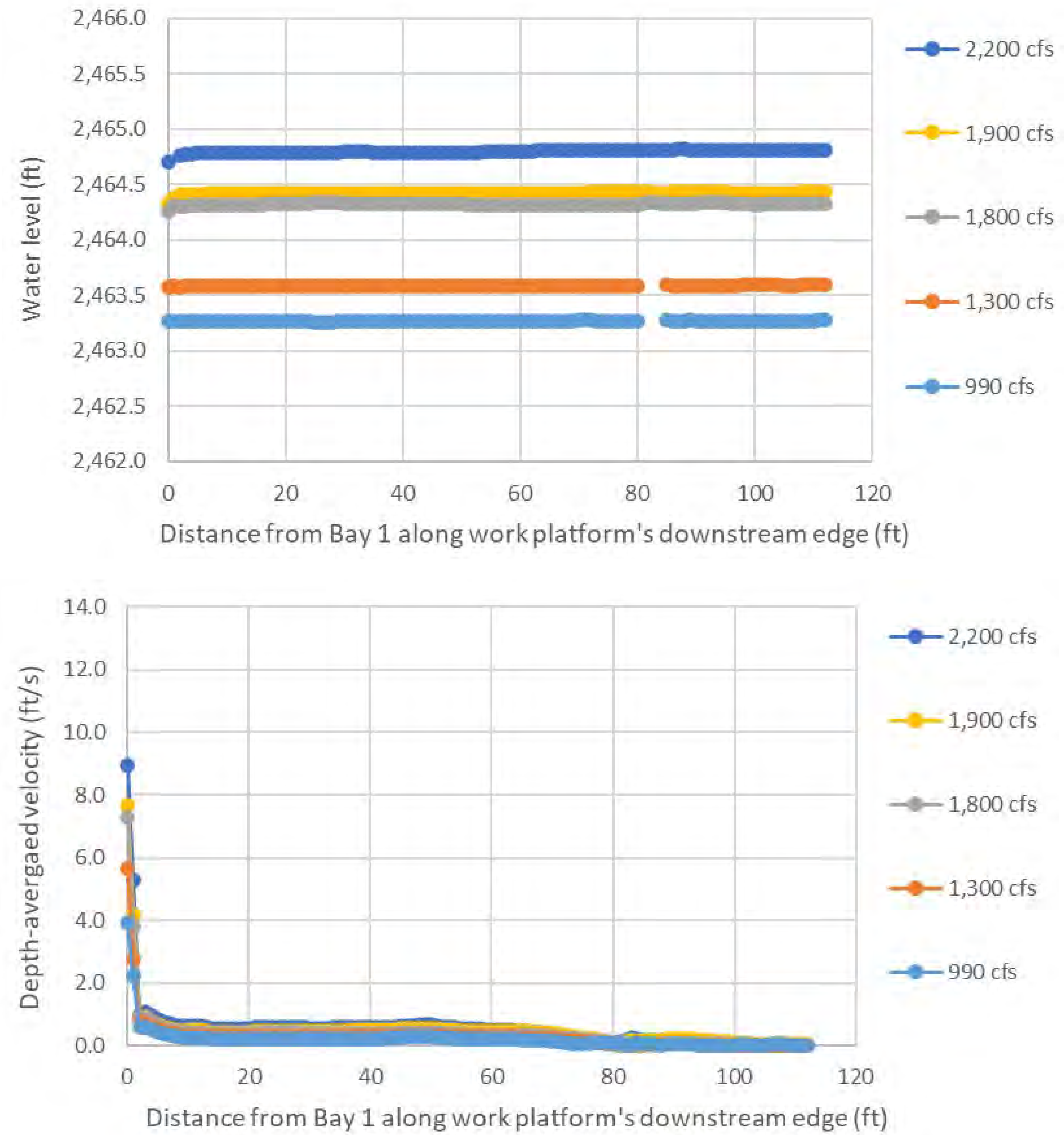
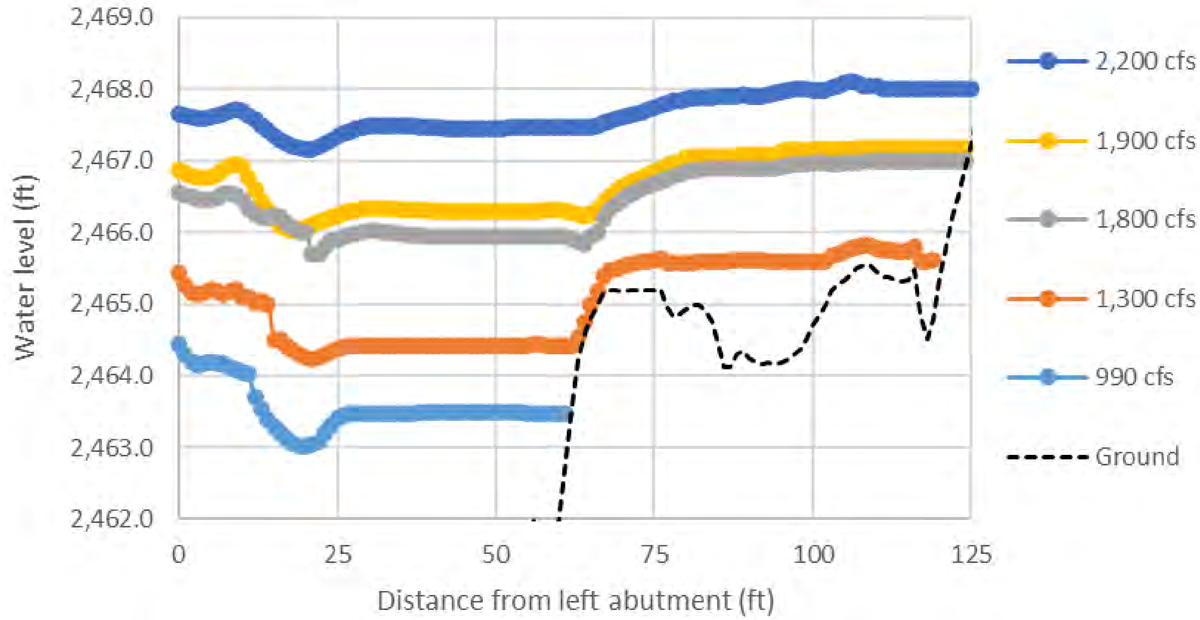


Figure 10. Water levels and depth-averaged velocity along Section B.



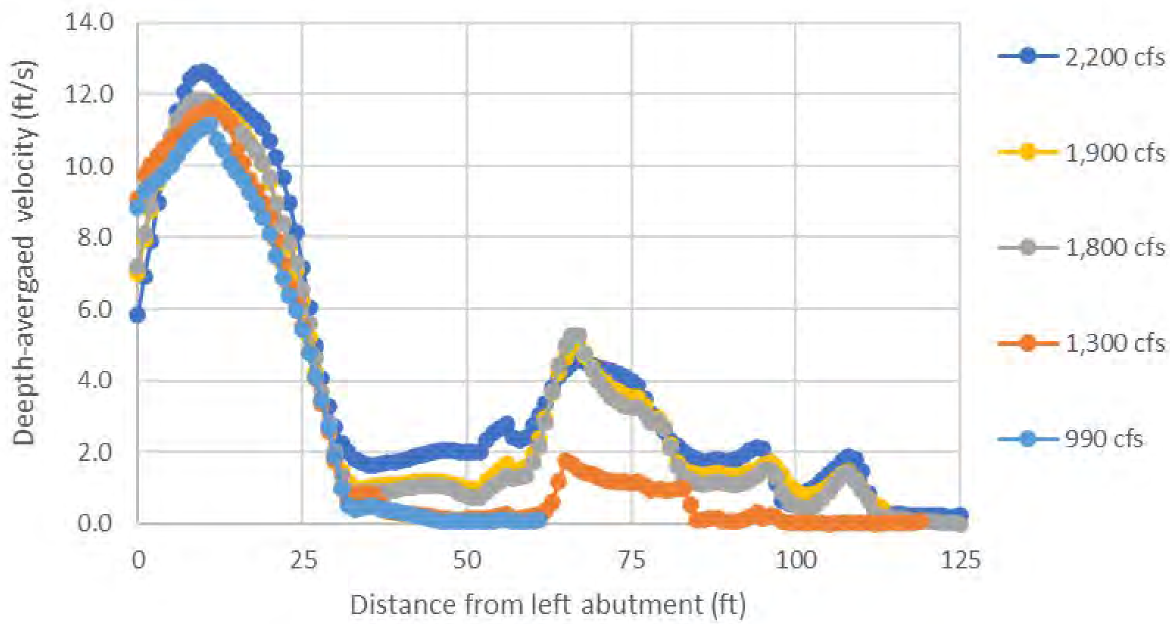


Figure 11. Water levels and depth-averaged velocity along Section C.

Table 4. Maximum water levels around work platform

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

RATING CURVE

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

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

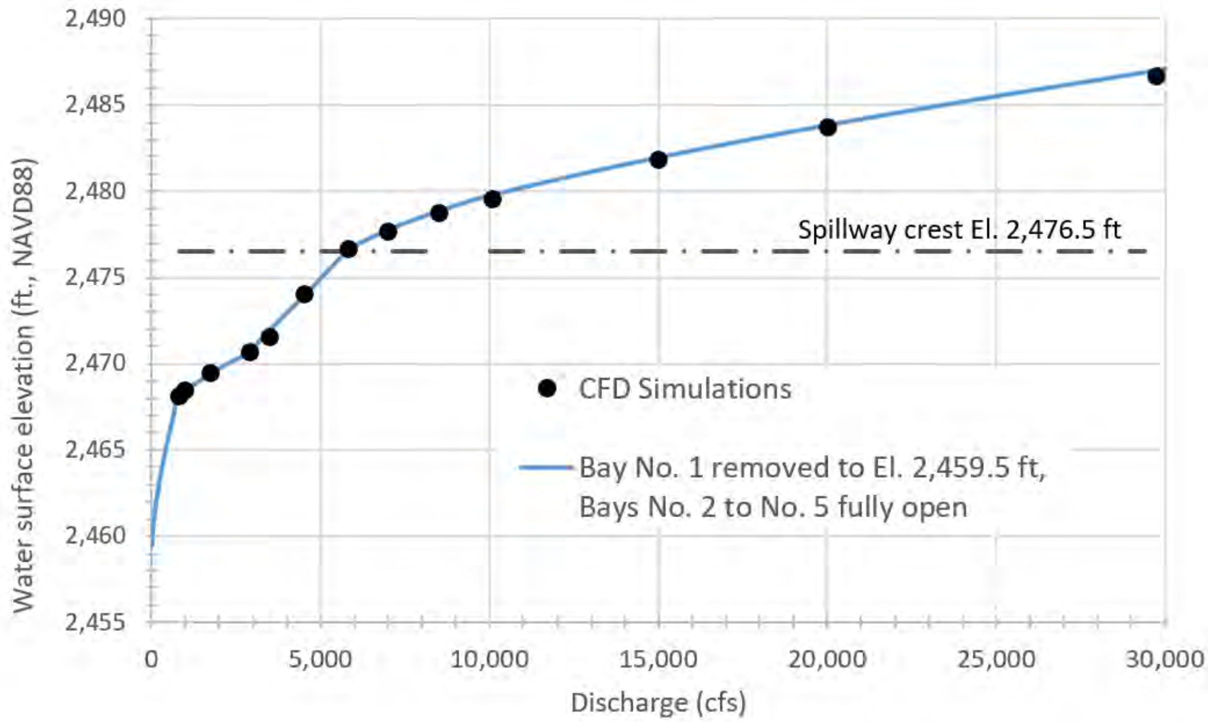


Figure 12. Copco No.2 Dam - Headwater Rating Curve

REFERENCES

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

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Report Prepared by:

Jessica Lammers, P.E.
Hydraulic Engineer

Pepe Vasquez, Ph.D., P.Eng.
Principal

Nancy Sims, P.Eng.
Principal

Report Reviewed by:

Henry Fehlman, P.E. (CA)
Principal

APPENDIX A

Copco No. 2 Post-Drawdown Rating Curve

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

APPENDIX E1
IRON GATE HYDROPOWER FACILITY DAM REMOVAL DESIGN DETAILS

TABLE OF CONTENTS

1.0 INTRODUCTION

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

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

2.0 CIVIL / STRUCTURAL

2.1 MATERIAL PROPERTIES

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

2.2 DIVERSION TUNNEL EXISTING CONDITIONS

2.2.1 GATE SHAFT AND TUNNEL UPSTREAM REACH

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

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

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

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

2.2.2 TUNNEL DOWNSTREAM REACH

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

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

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

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

2.3 TUNNEL MODIFICATIONS

2.3.1 GENERAL

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

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

Table 2.1 Iron Gate Facility Drawings – Tunnel Modifications

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

2.3.2 PRE-DRAWDOWN WORKS

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

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

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

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

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

2.3.2.1 BEST FIT LINER OPTION – HYDRAULIC BEHAVIOR MAXIMUM HEAD

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

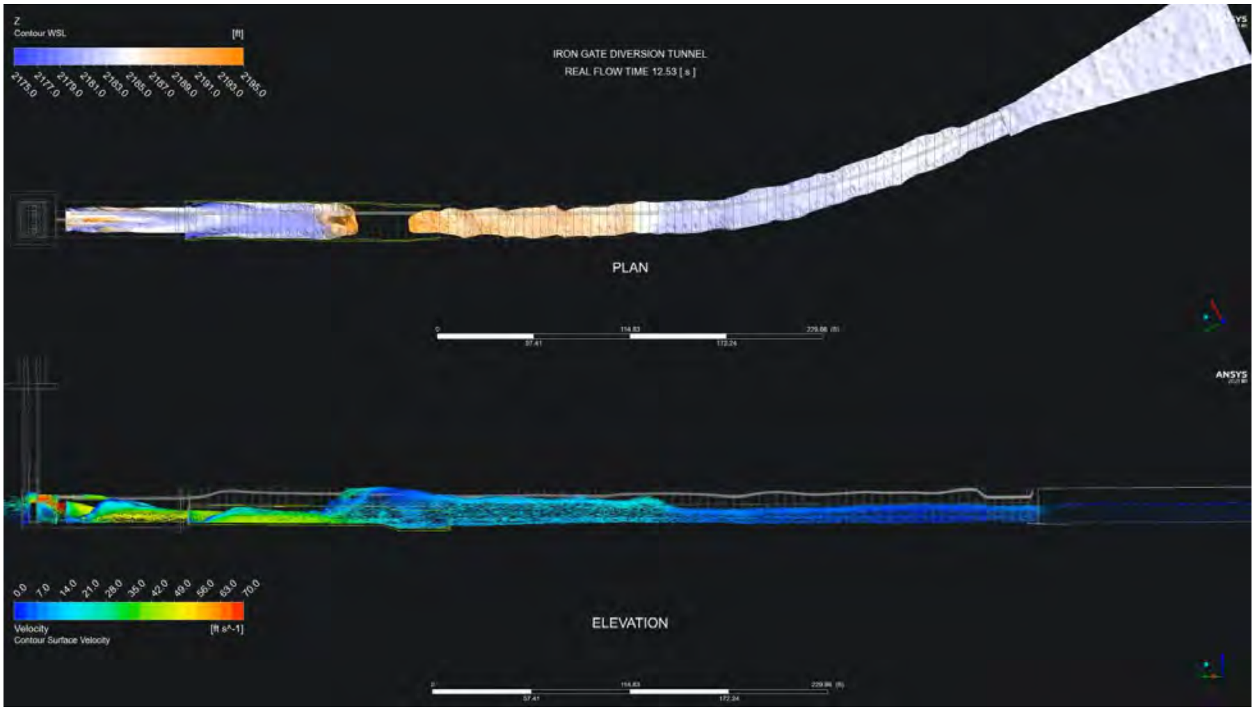


Figure 2.1 Iron Gate Diversion Tunnel CFD – Best Fit Liner Option - Water Surface Profile and Flow Velocity Contours – Initial Stages after Full Gate Opening

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

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

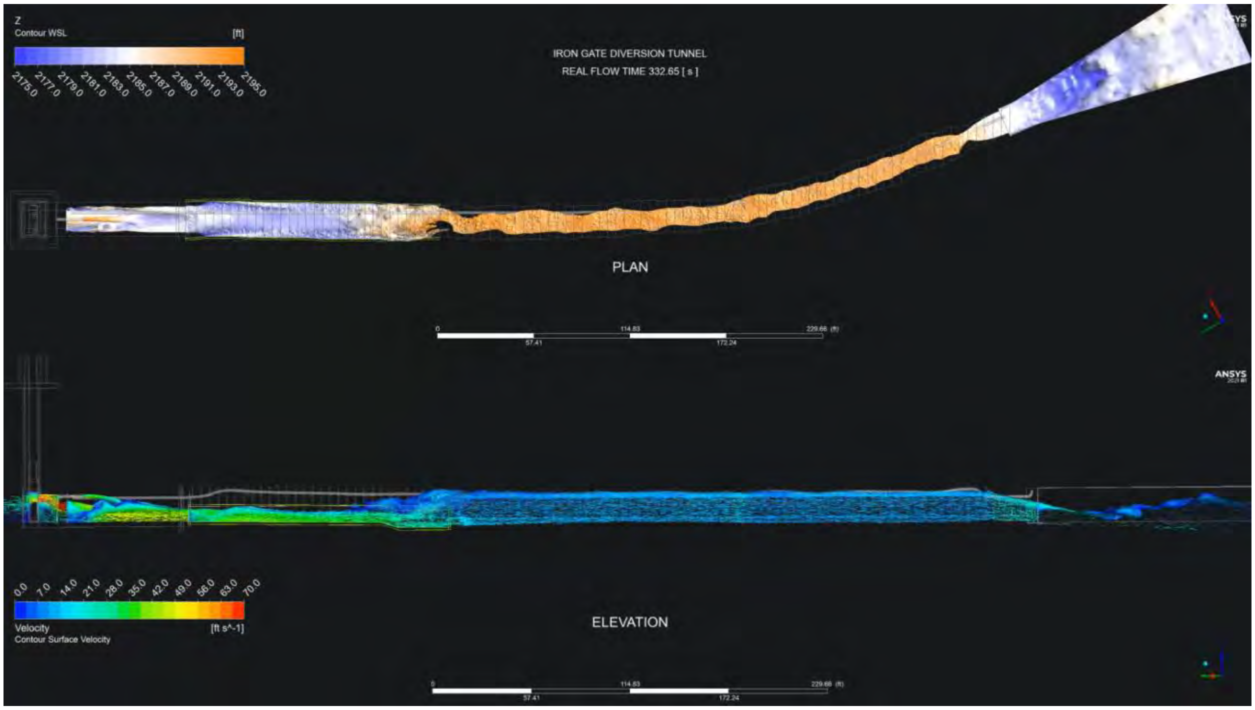


Figure 2.2 Iron Gate Diversion Tunnel CFD – Best Fit Liner Option – Water Surface Profile and Flow Velocity Contours –at Maximum Reservoir Level 2,331.3 ft

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

2.3.2.2 BEST FIT LINER OPTION - CONCRETE LINER DESIGN

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

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

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

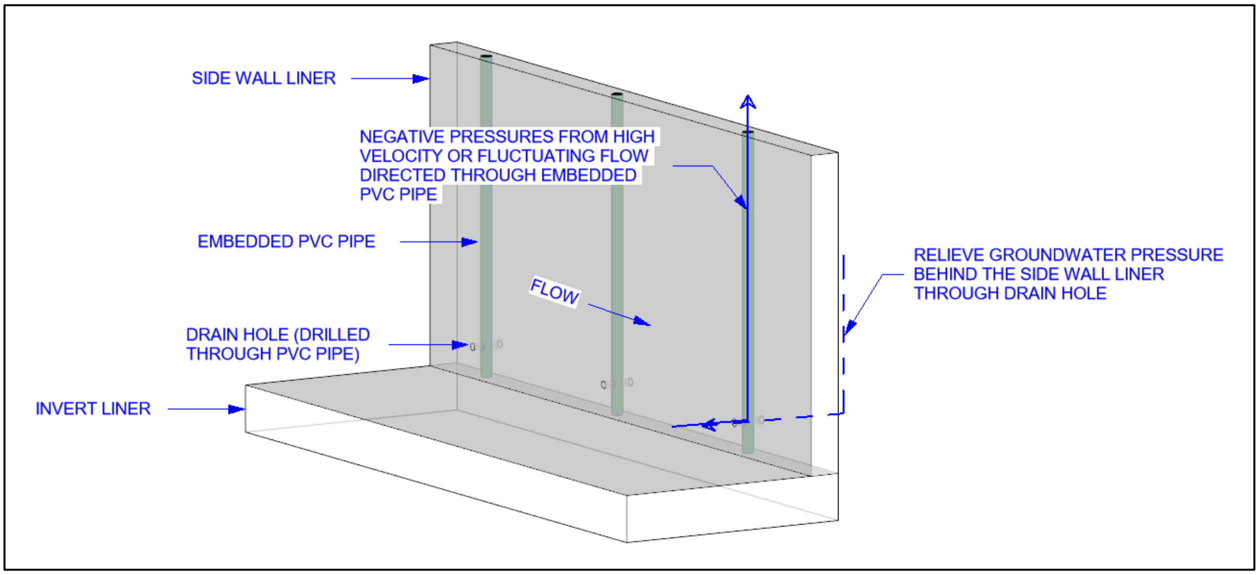


Figure 2.3 Side Wall Liner – Drain Hole and Embedded PVC Pipe Arrangement

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

Table 2.2 Concrete Liner Design – Summary of Loads

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

2.3.2.3 BAFFLED OPTION – HYDRAULIC BEHAVIOR AT MAXIMUM HEAD

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

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

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

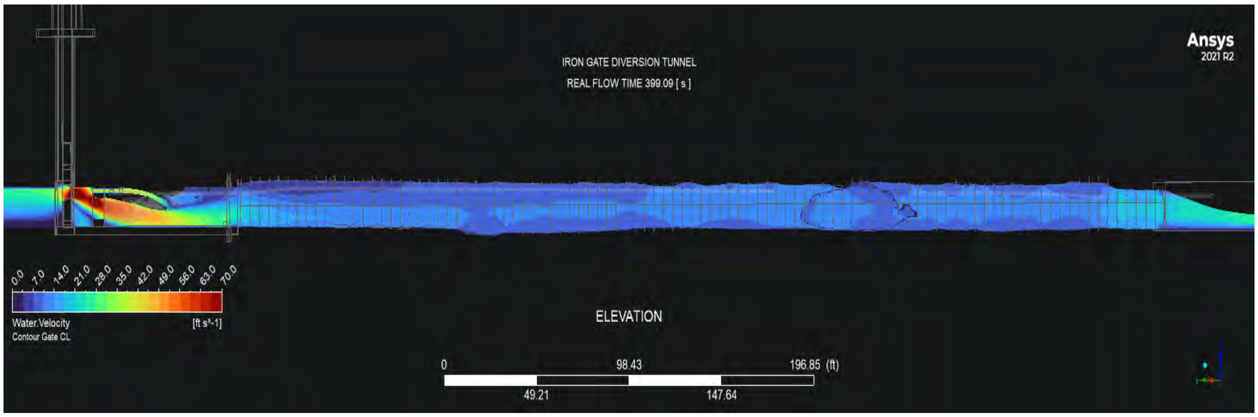


Figure 2.4 Iron Gate Diversion Tunnel CFD – Baffled Option – Water Surface Profile - Maximum Reservoir Level 2,331.3 ft

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

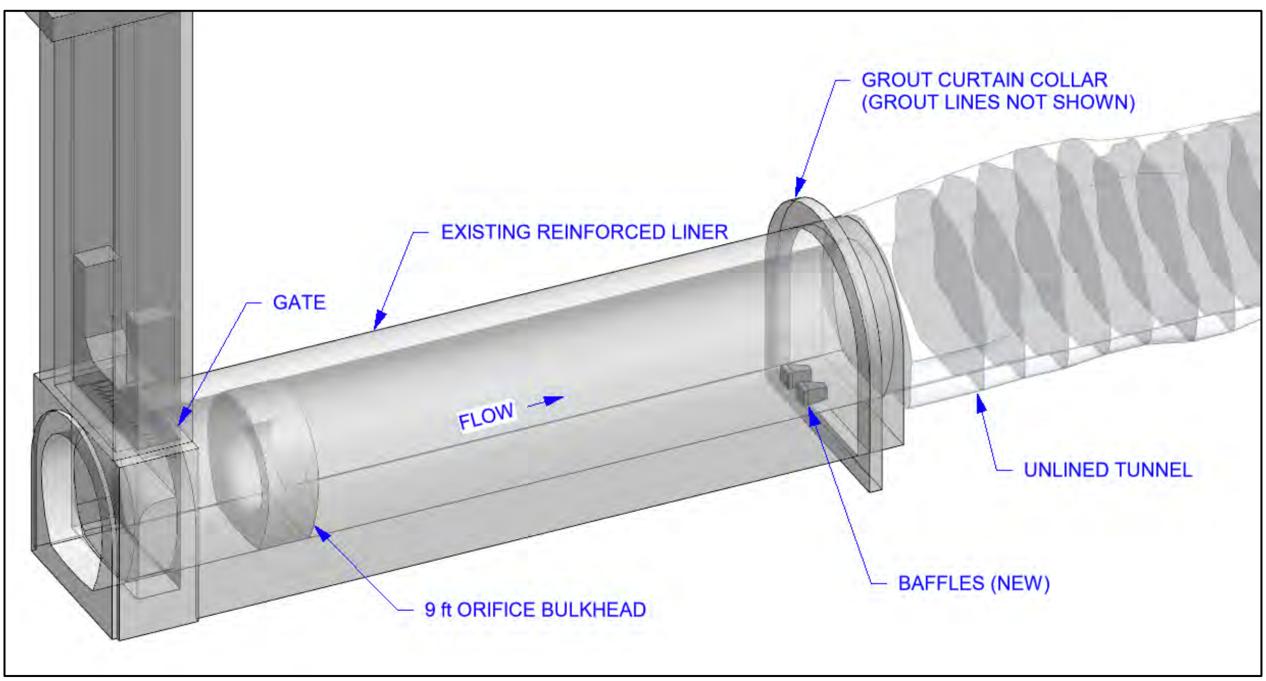


Figure 2.5 Modelled Baffle Location

2.3.2.4 BAFFLED OPTION – BAFFLE DESIGN

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

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

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

2.3.3 PRE-DRAWDOWN WORKS - VENTILATION DESIGN

2.3.3.1 GENERAL

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

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

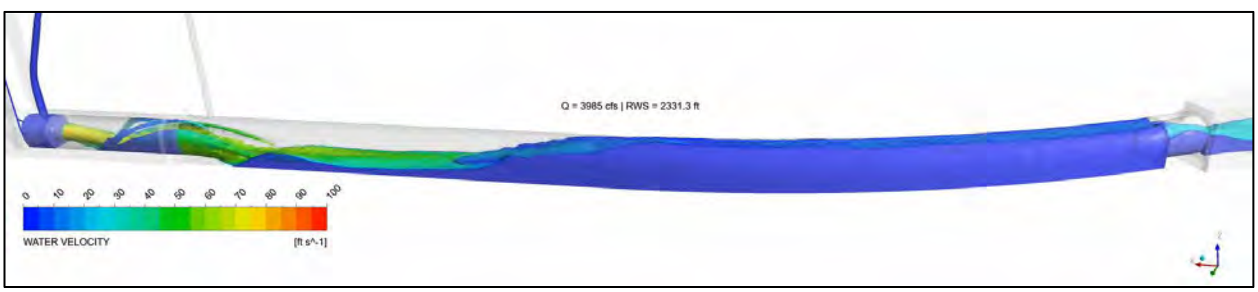


Figure 2.6 Hydraulic Jump Forming Inside Tunnel

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

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

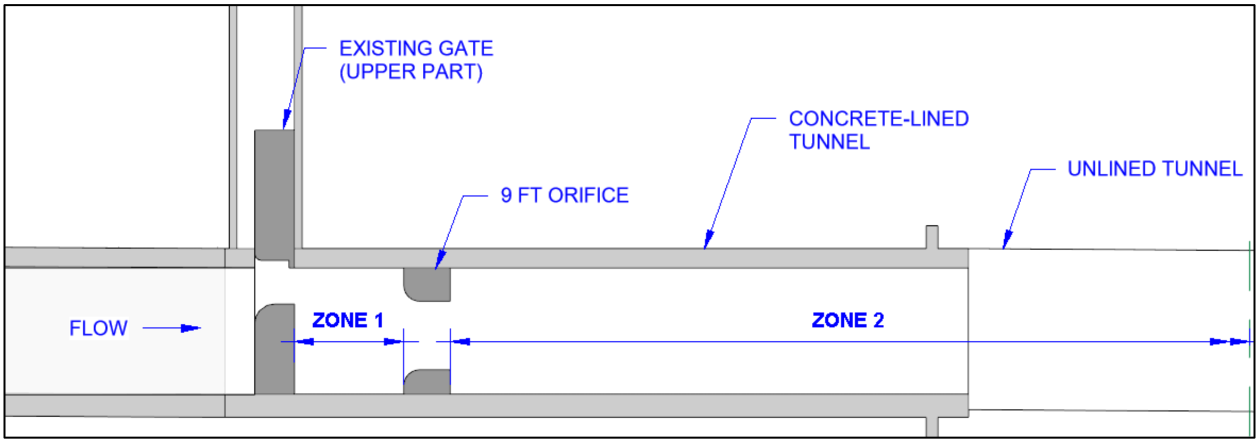


Figure 2.7 Areas Inside Tunnel Targeted by the Venting – Tunnel Elevation View

Two options for providing the additional venting were considered:

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

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

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

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

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

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

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

2.3.3.2 VENT PIPE ALIGNMENT

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

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

2.3.3.3 ANALYSIS AND DESIGN OF DOWNSTREAM VENT PIPE SUPPORTS

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

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

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

Table 2.3 Structural Mass System

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

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

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

Table 2.4 Downstream Vent Pipe – Vibration Analysis Results

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

NOTES:

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

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

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

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

Table 2.5 Downstream Vent Pipe Support – LRFD Loading Condition

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

NOTES:

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

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

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

2.4 CHANNEL PROTECTION

2.4.1 DRAWDOWN

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

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

2.4.2 POST DRAWDOWN CHANNEL FINAL GRADING

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

Table 2.6 Iron Gate Facility Drawings – Channel Final Grading

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

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

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

2.4.2.1 EROSION PROTECTION DESIGN

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

3.0 HYDROTECHNICAL

3.1 RESERVOIR DEPTH-AREA-CAPACITY

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

Table 3.1 Reservoir Storage Capacity for Various Elevations

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

3.2 OUTLET STRUCTURE RATING CURVES

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

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

3.3 RESERVOIR CONDITIONS DURING DRAWDOWN AND POST-DRAWDOWN

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

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

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

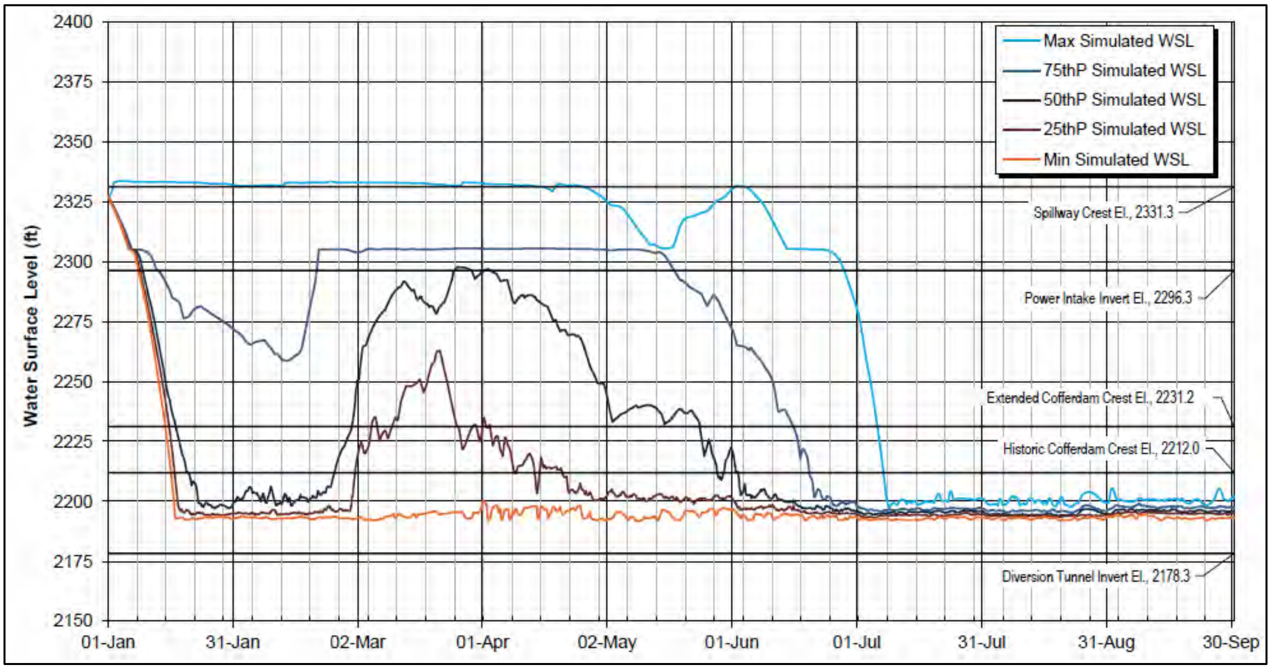


Figure 3.1 Iron Gate Reservoir Drawdown Simulated Water Surface Levels Non-Exceedance Percentiles

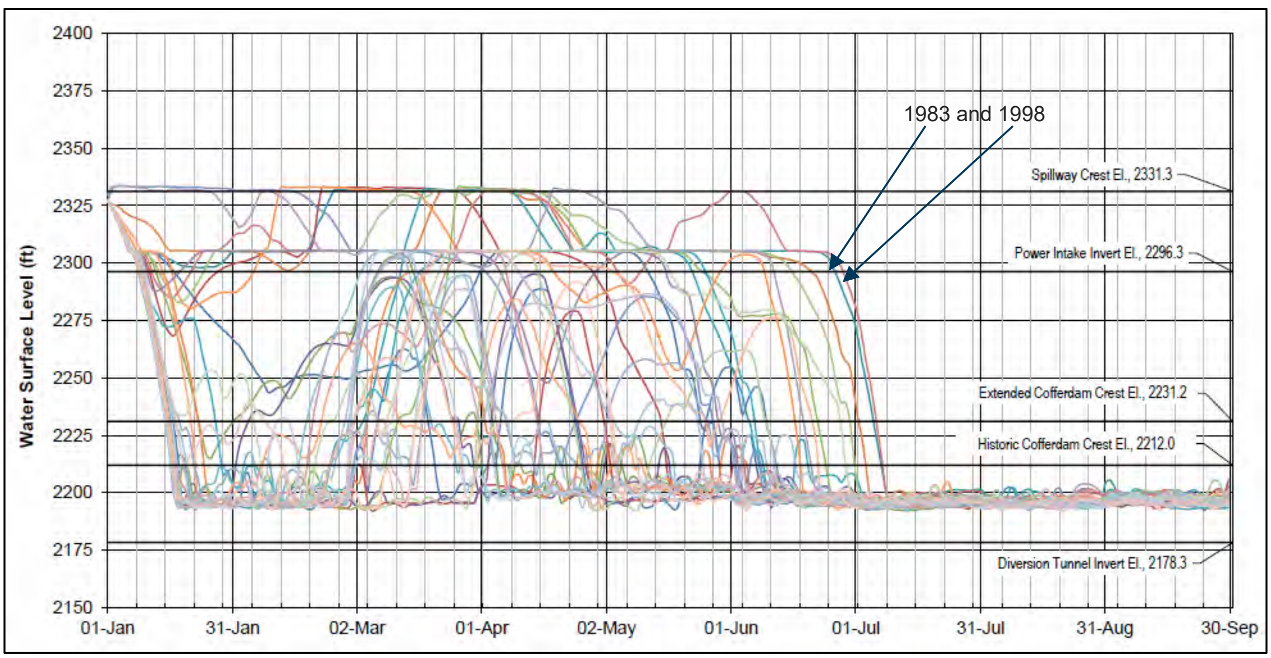


Figure 3.2 Iron Gate Reservoir Drawdown Simulated Water Surface Levels Ensemble Plot

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

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

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

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

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

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

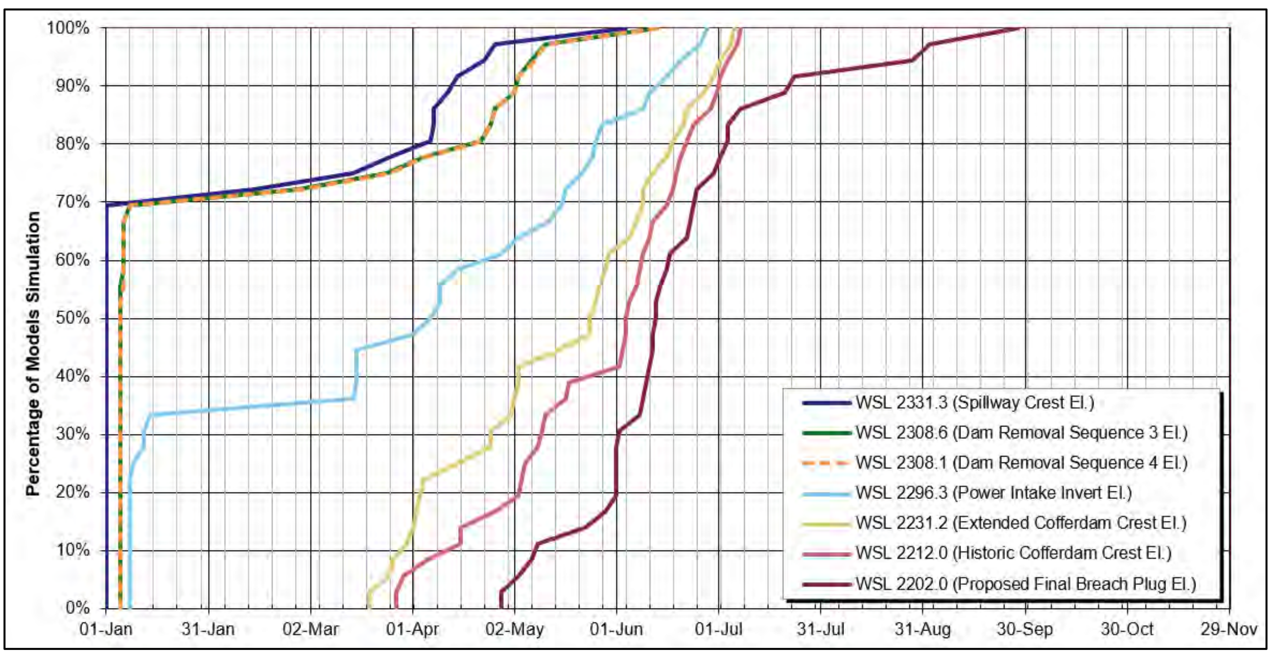


Figure 3.3 Iron Gate Reservoir Drawdown Cumulative Model Simulation Dates to Achieve and Sustain Reservoir Water Surface Levels below the Crest of the Historic Cofferdam

3.3.1.1 POST-DRAWDOWN RIVER DIVERSION

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

3.4 STEADY-STATE WATER SURFACE ELEVATIONS

3.4.1 PRE-DRAWDOWN TAILWATER LEVELS

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

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

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

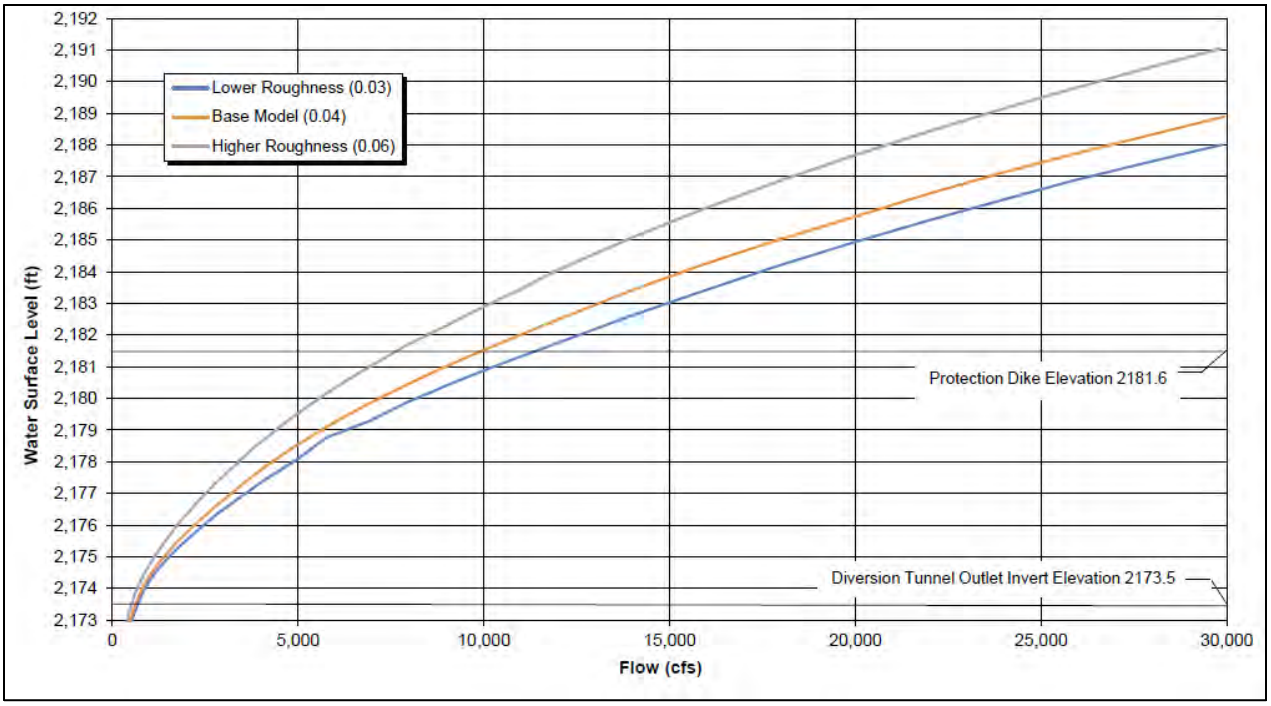


Figure 3.5 Iron Gate Dam Pre-Drawdown Tailwater Level Rating Curve Sensitivity Results

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

3.4.2 EMBANKMENT AND RESERVOIR LEVELS

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

3.4.3 POST-DRAWDOWN TAILWATER LEVELS

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

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

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

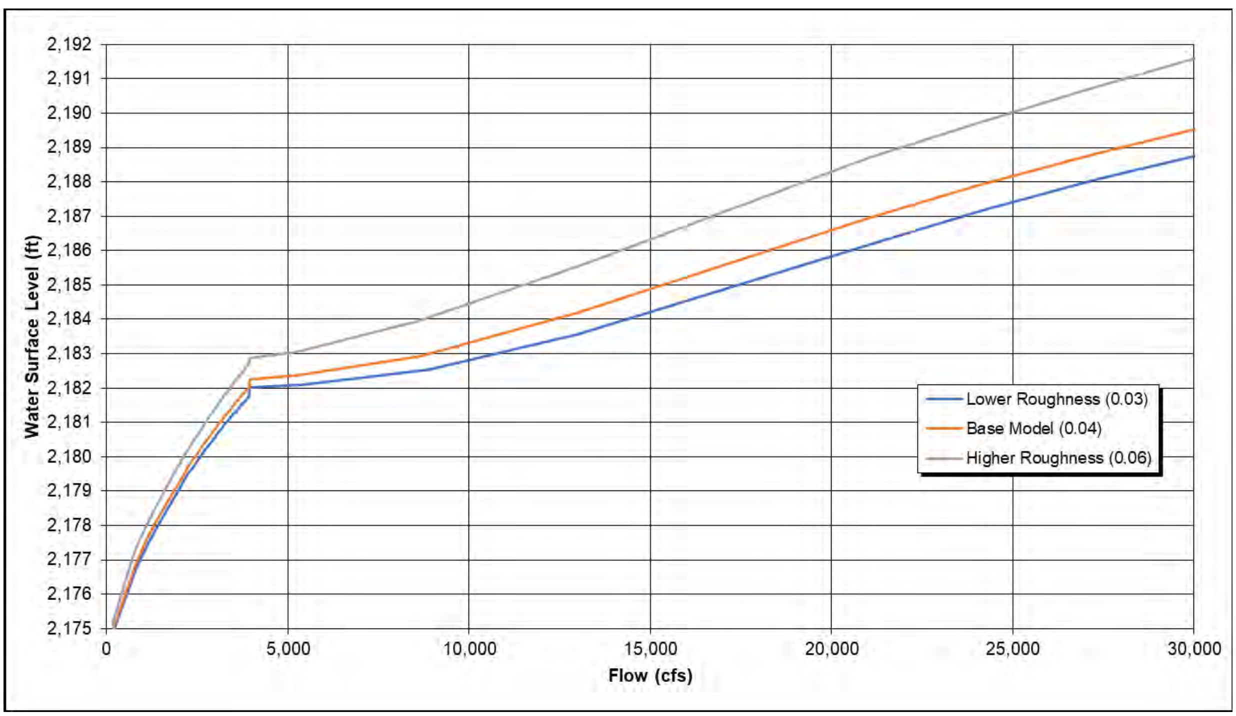


Figure 3.7 Iron Gate Dam Post-Drawdown Tailwater Level Rating Curve Sensitivity Results

3.5 FINAL COFFERDAM BREACH

3.5.1 DAM BREACH OVERVIEW

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

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

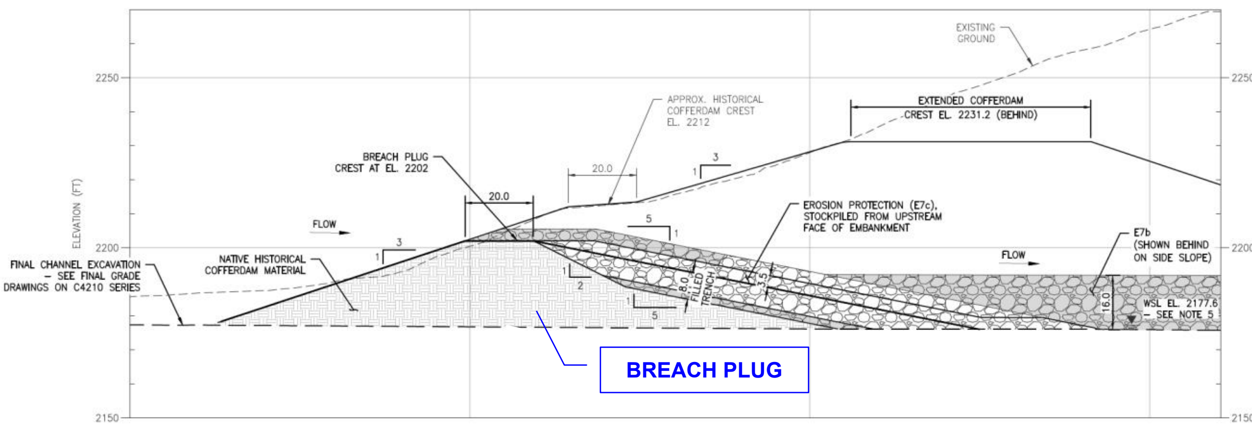


Figure 3.8 Extended Cofferdam Configuration Prior to Final Dam Breach

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

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

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

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

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

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

Table 3.2 Iron Gate Facility Drawings – Final Dam Breach

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

3.5.2 DAM BREACH ANALYSIS

3.5.2.1 GENERAL

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

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

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

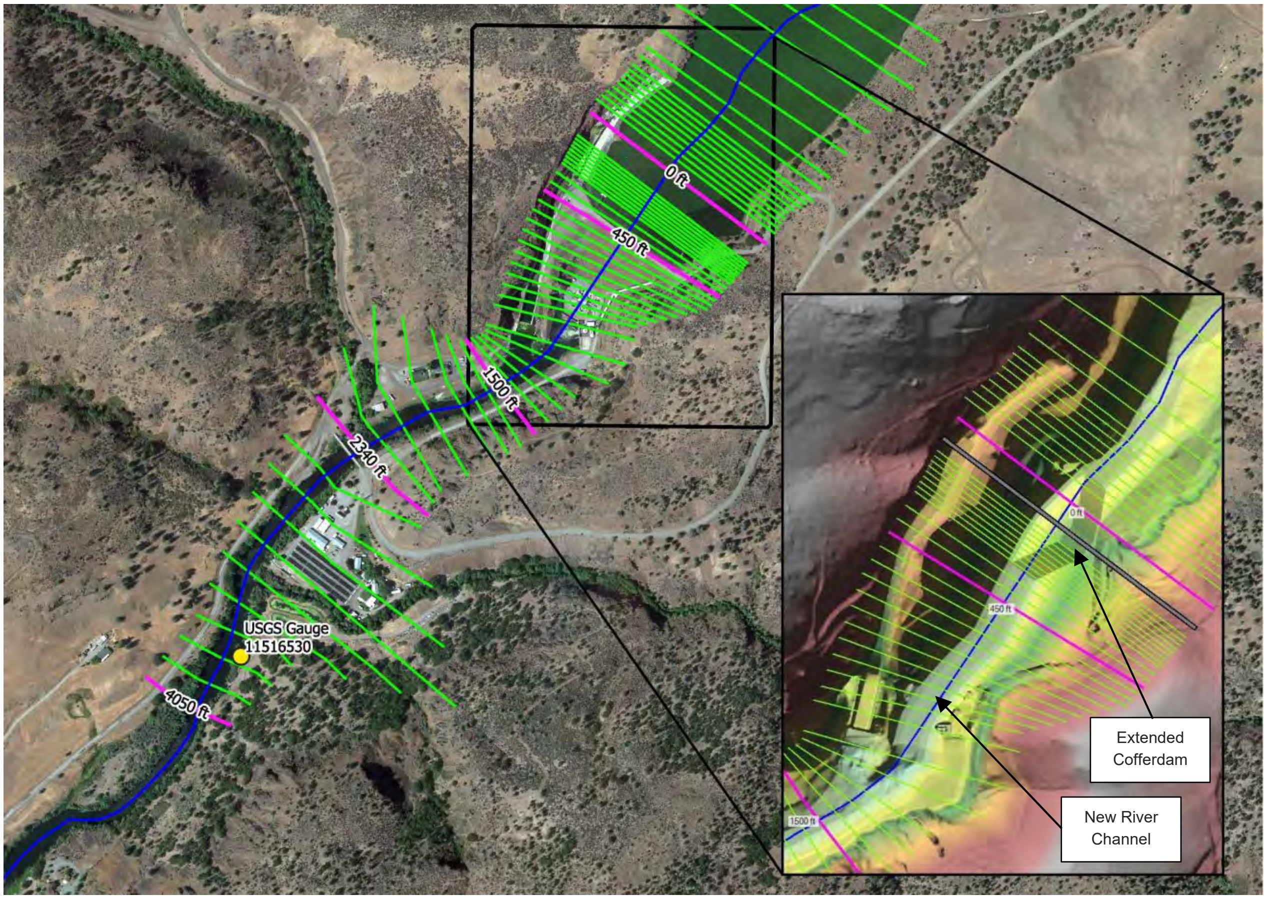


Figure 3.9 HEC-RAS Model Setup and Cross Sections for Dam Breach Analysis

3.5.2.2 DAM BREACH PARAMETERS

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

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

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

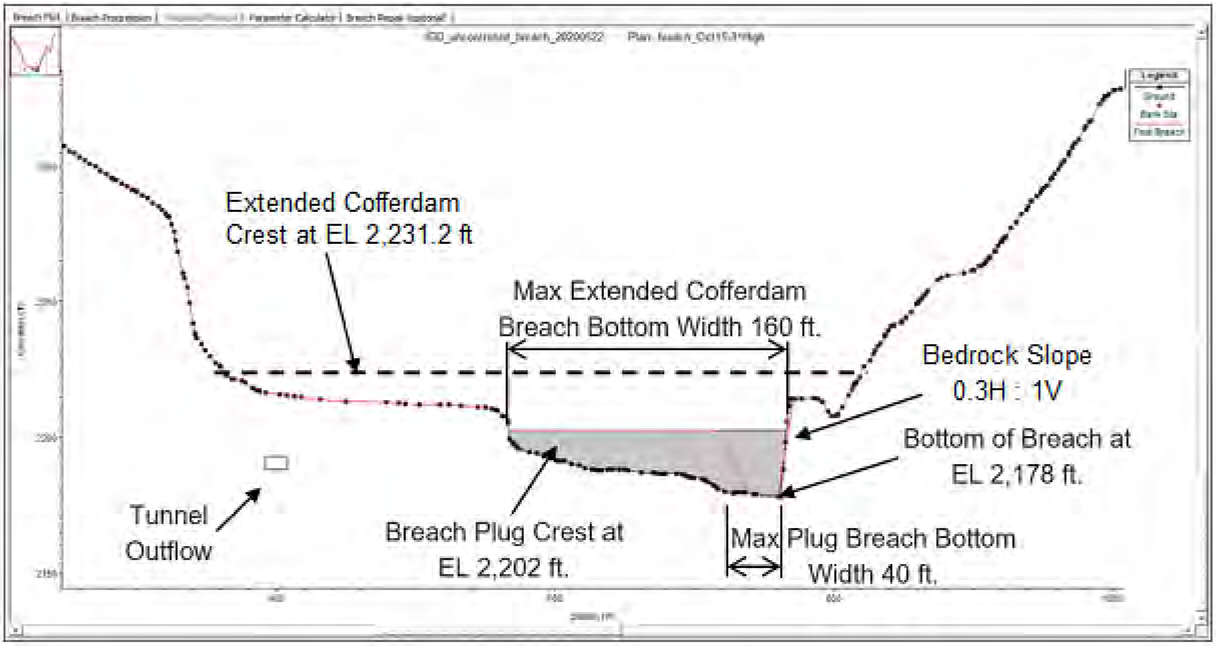


Figure 3.10 Cross-Section Used in HEC-RAS for Dam Breach Analysis

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

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

NOTES:

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

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

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

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

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

3.5.2.3 BREACH OUTFLOW HYDROGRAPHS

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

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

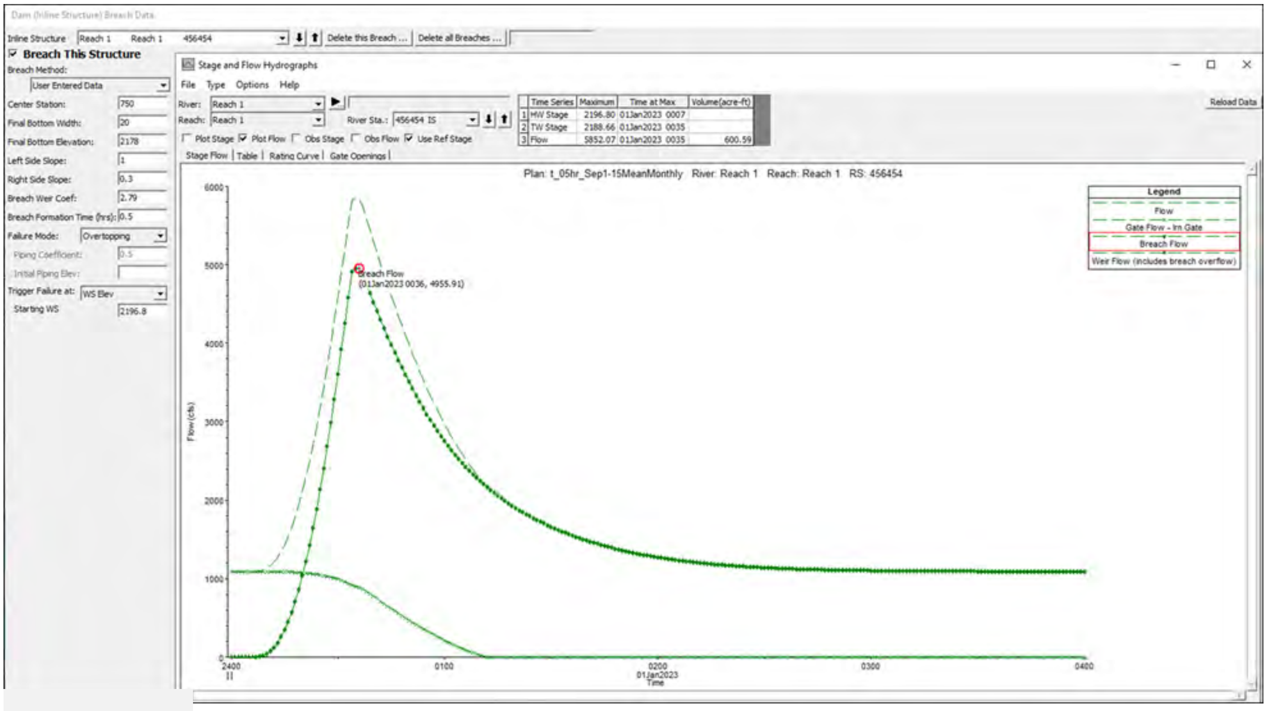


Figure 3.11 Example HEC-RAS Output for the Breach, the Tunnel and Total Outflows

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

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

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

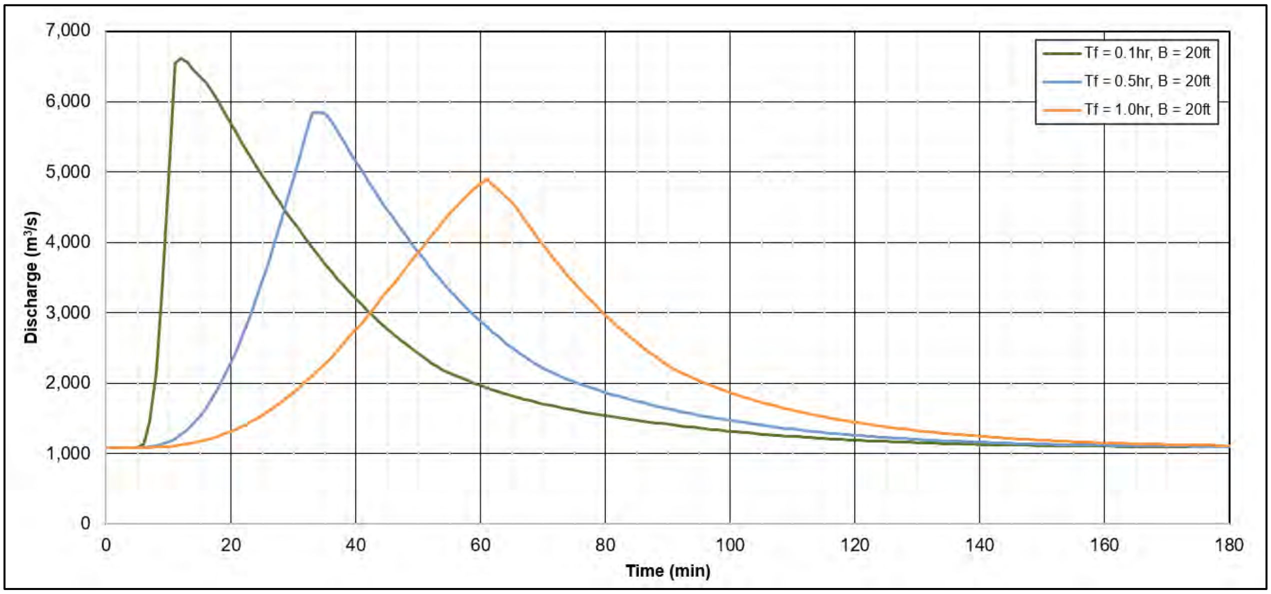


Figure 3.12 Total Outflow Hydrographs (Breach Plug and Tunnel) for Different Breach Formation Times

3.5.2.4 DOWNSTREAM ATTENUATION

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

Table 3.4 Downstream Peak Flow Attenuation

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

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

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

3.5.3 RIPRAP DESIGN

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

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

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

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

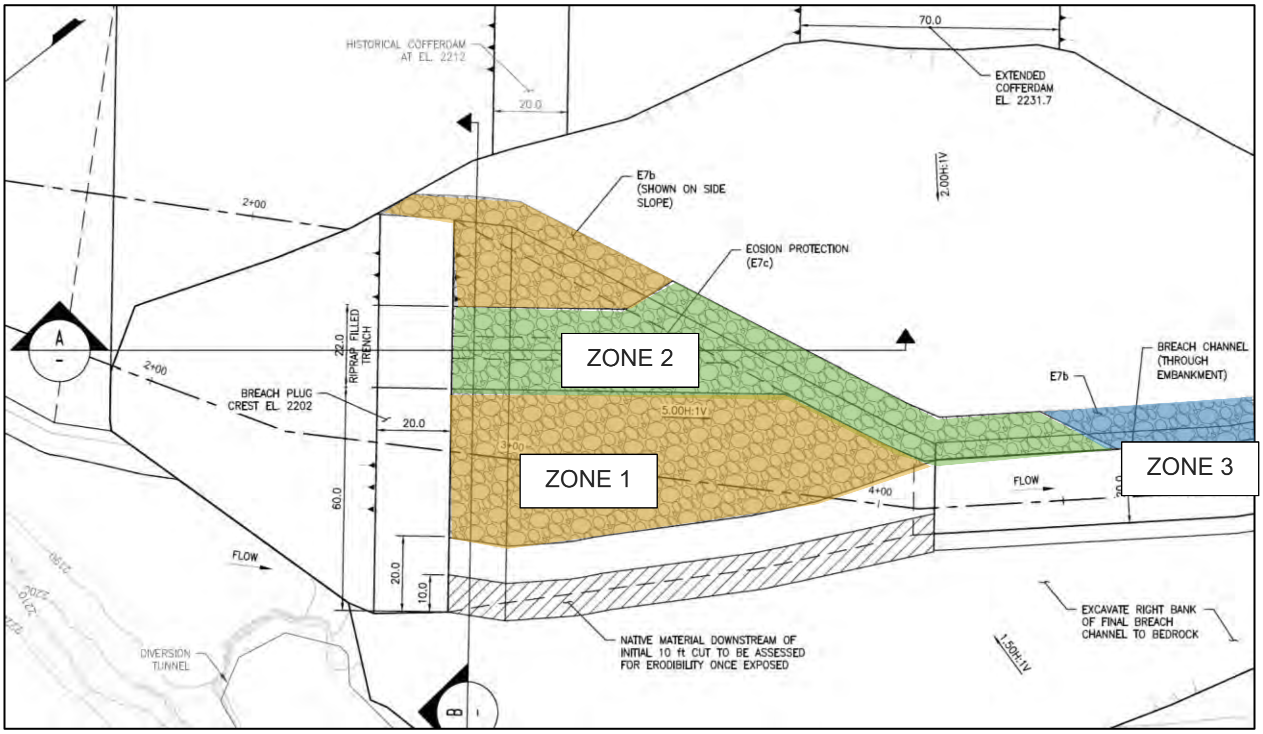


Figure 3.13 Breach Plug and Extended Cofferdam Channel – Riprap Zones – Plan

Table 3.5 Riprap Design Summary

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

3.6 FINAL GRADING HYDRAULIC CONDITIONS

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

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

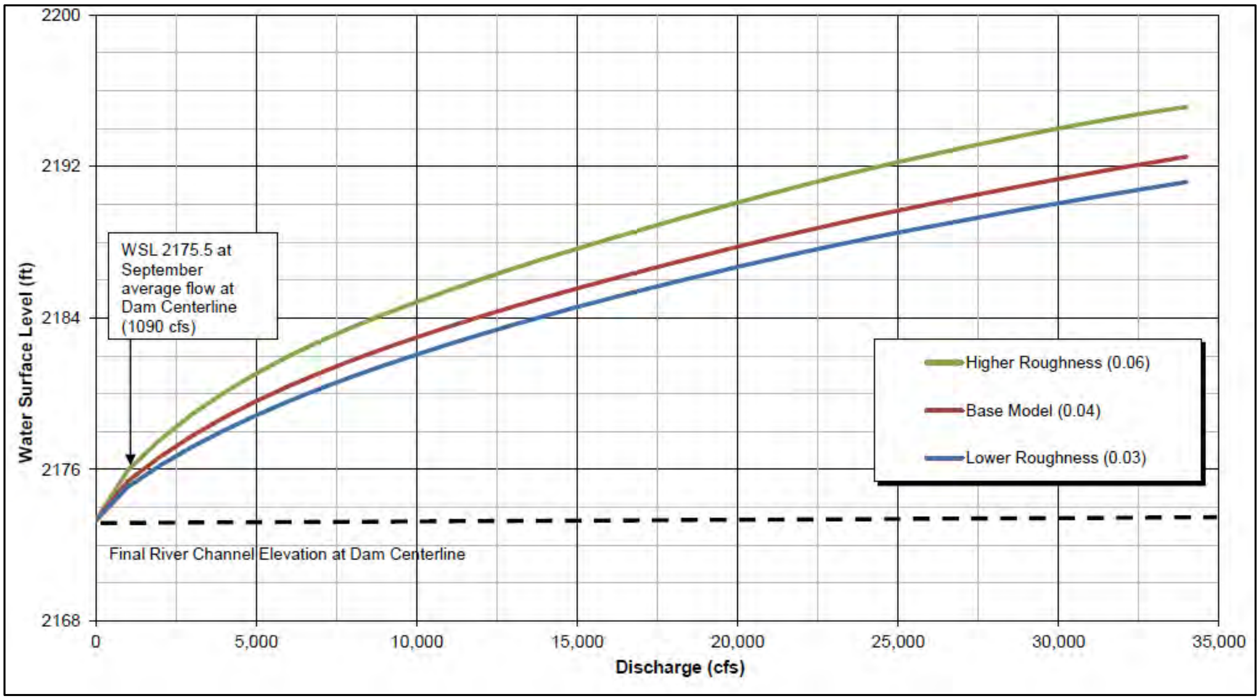


Figure 3.14 Final River Channel Stage-Discharge Relationship at Existing Dam Centerline

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

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

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

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

NOTES:

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

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

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

NOTES:

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

4.0 GEOTECHNICAL

4.1 DIVERSION TUNNEL

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

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

4.2 DAM EMBANKMENT STABILITY DURING DRAWDOWN

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

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

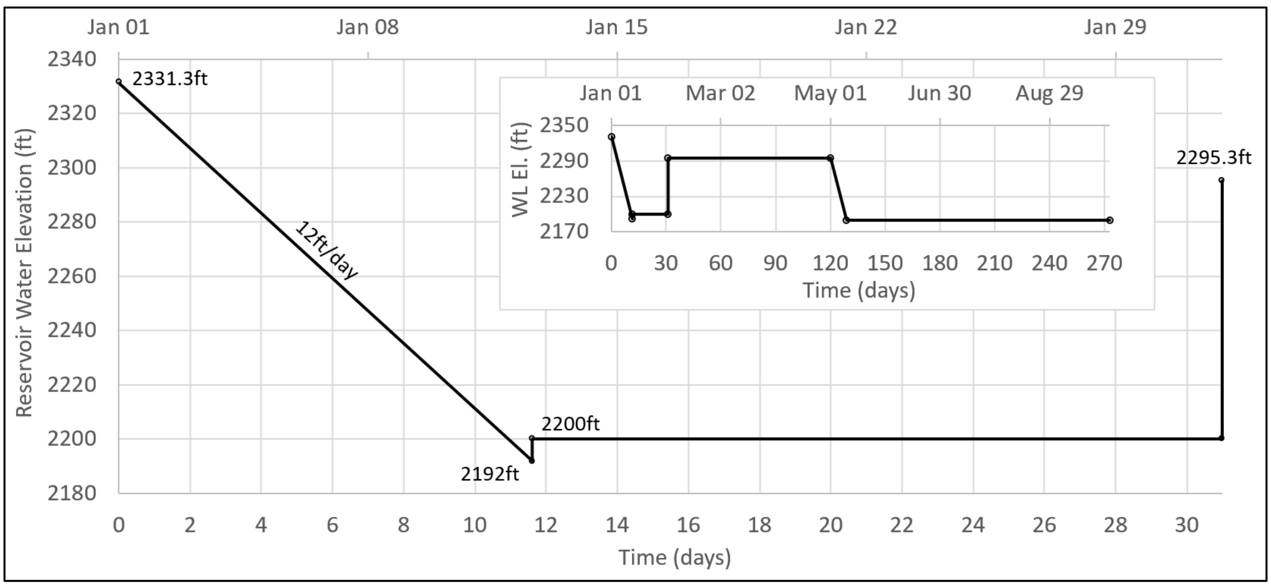


Figure 4.1 Generalized Drawdown Curve For Stability Analyses

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

Table 4.1 Material Properties for Drawdown Stability Assessment

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

NOTES:

1. SENSITIVITY CHECKS COMPLETED WITH VALUES IN PARENTHESES.

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

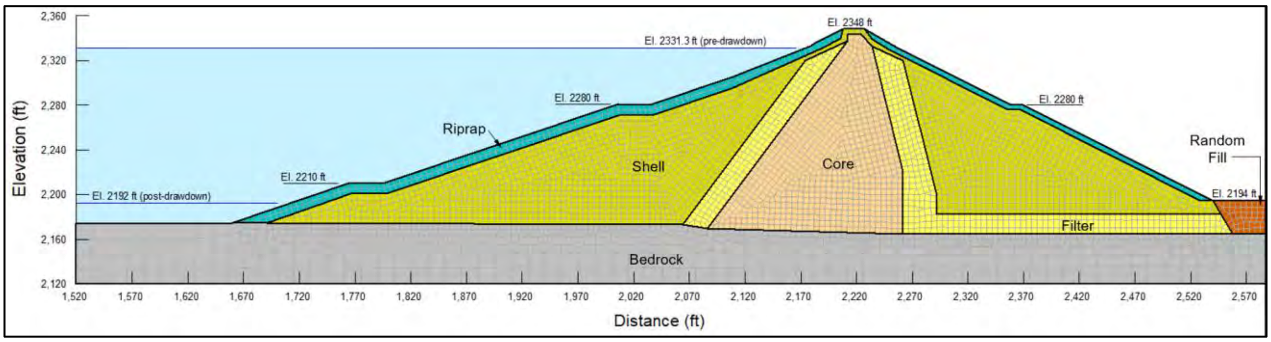
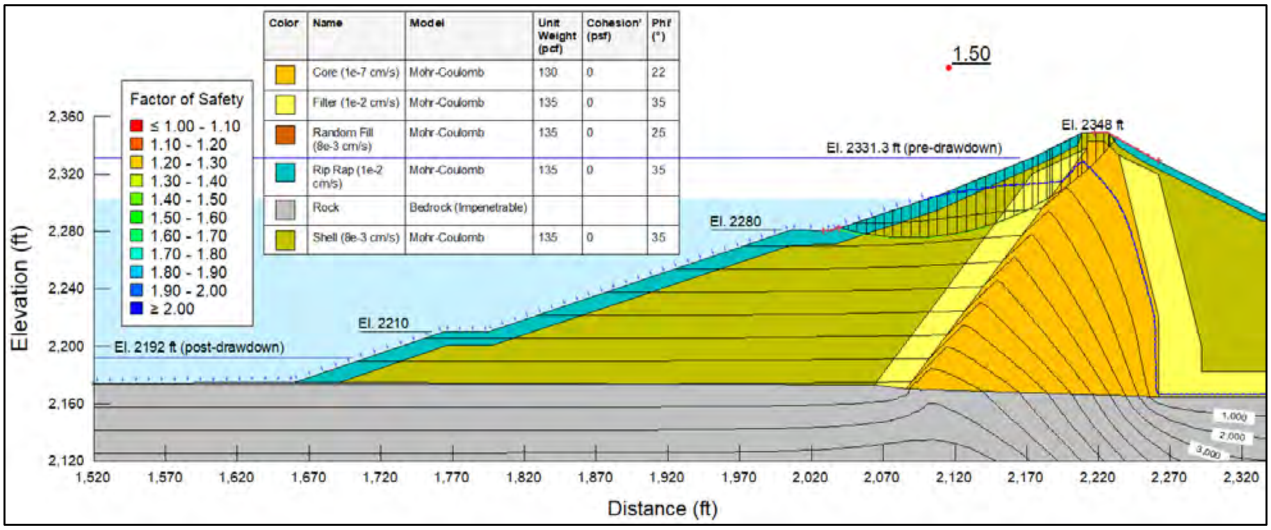


Figure 4.2 Drawdown Analysis Model Geometry

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

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

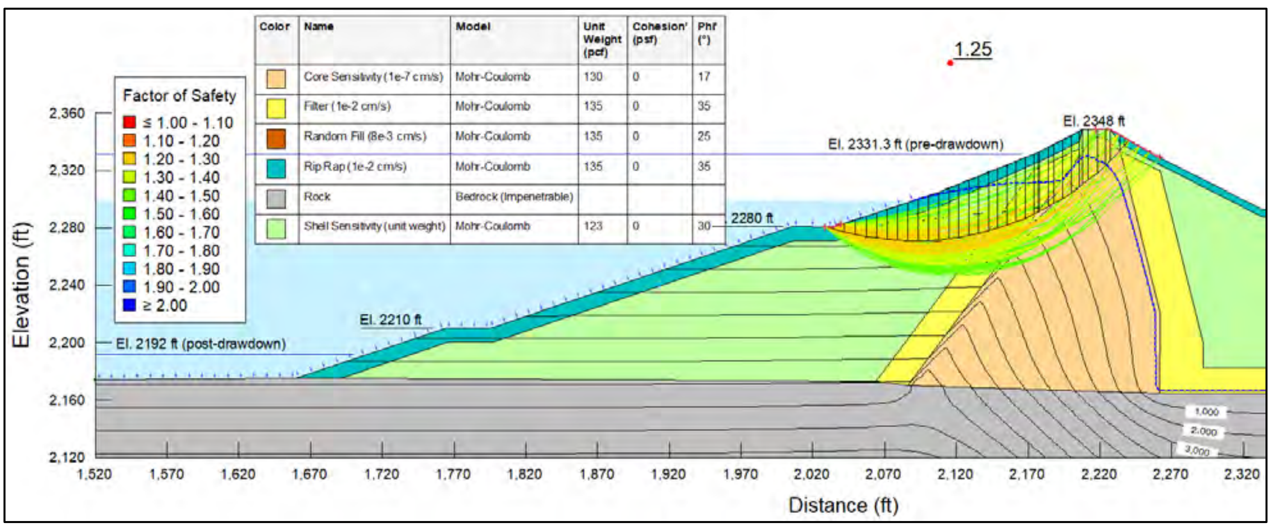
Stability results are summarised in Table 4.2 for the three scales of slips. Results for the GLE and Spencer methods are similar; the lower FOS is reported.



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

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



NOTES:

1. ONLY SLIPS WITH FOS < 1.5 ARE SHOWN.

Figure 4.4 Sensitivity Check Drawdown Stability Results for Upper Slope Slip

Table 4.2 Factor of Safety Results for Drawdown Stability

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

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

4.3 STABILITY DURING DAM REMOVAL

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

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

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

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

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

Table 4.3 Embankment Cut Volumes by Sequence Number

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

Table 4.4 Embankment Removal Mass Balance

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

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

4.4 EXCAVATION SLOPES

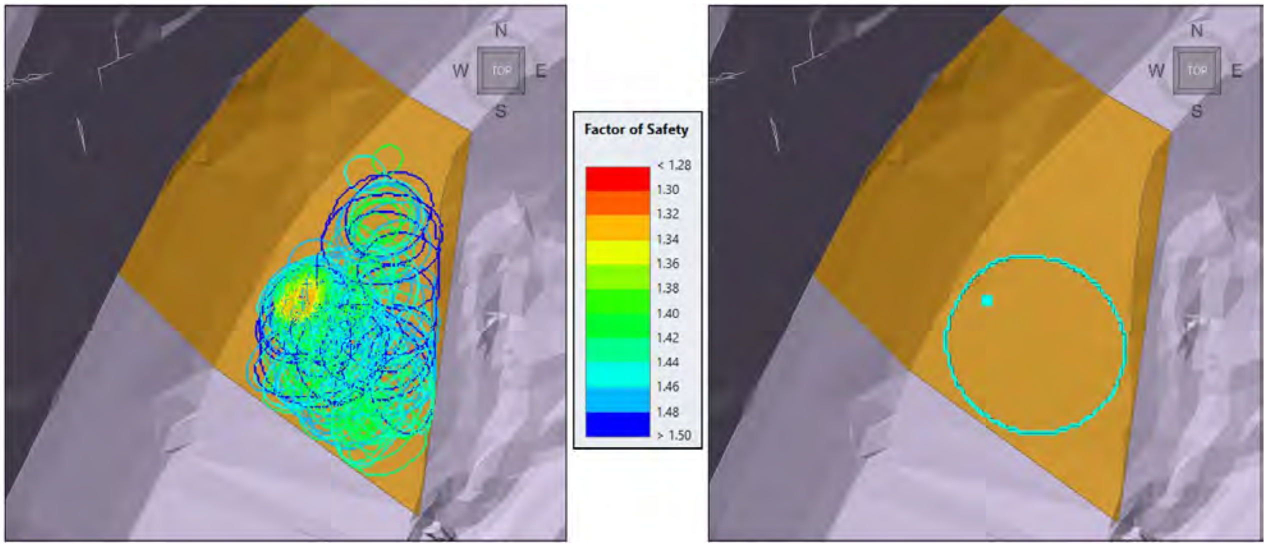
The dam embankment will be removed in stages and the river channel will be restored by excavating the existing dam embankment to form temporary and permanent slopes. Excavated slopes are designed to be stable with suitable drainage. Areas where excavated slopes within the dam footprint cannot be considered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

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

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

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

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



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

Figure 4.5 Static Stability Results for Left Bank Dam Final Grade

4.5 DISPOSAL SITES

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

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

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

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

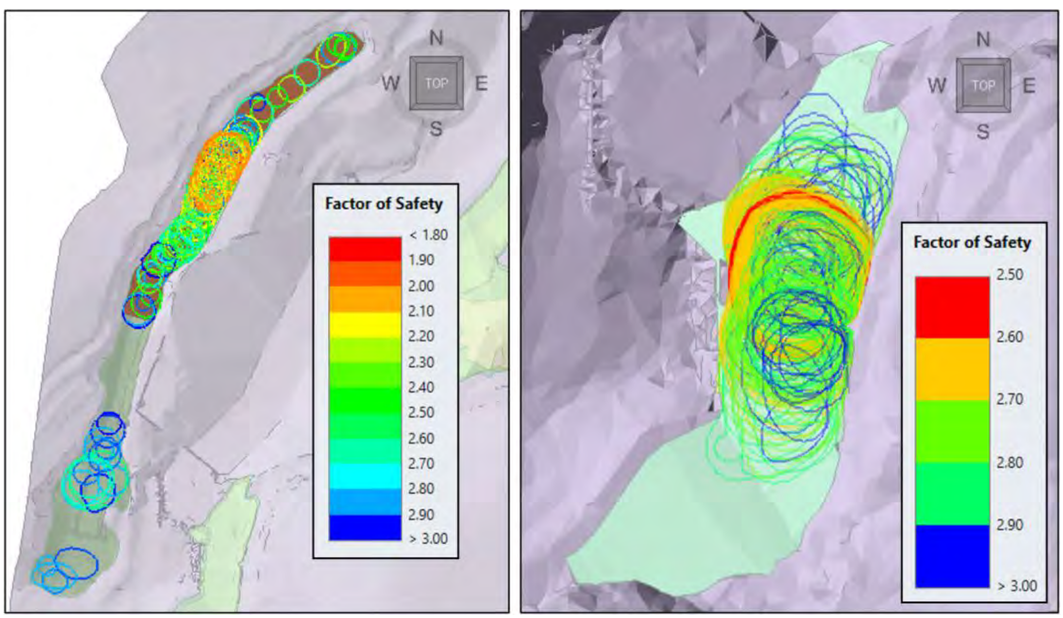
Table 4.5 Disposal Site Material Properties

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

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

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

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



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

Figure 4.6 Static Stability Result for Spillway and Powerhouse Disposal Sites

4.5.1 FOUNDATION PREPARATION

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

5.0 MECHANICAL

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

5.1 GENERAL

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

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

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

5.2 SITE VISIT OBJECTIVE AND SYSTEM INSPECTION

5.2.1 OVERVIEW

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

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

The objective of this site visit was to:

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

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

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

5.2.2 SYSTEM INSPECTION

5.2.2.1 HYDRAULIC POWER UNIT

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

5.2.2.2 HYDRAULIC CYLINDER AND HOIST RODS

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

Figure 5.1 Hoist Stem Pin Location

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

Figure 5.2 Historical Flow Rate Recordings on Hoist Stems

5.2.2.3 HYDRAULIC ACCUMULATORS

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

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

Figure 5.3 Accumulator No. 3 Leak Location

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

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

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

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

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

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

5.2.2.5 DIVERSION GATEHOUSE ELECTRICAL SYSTEMS

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

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

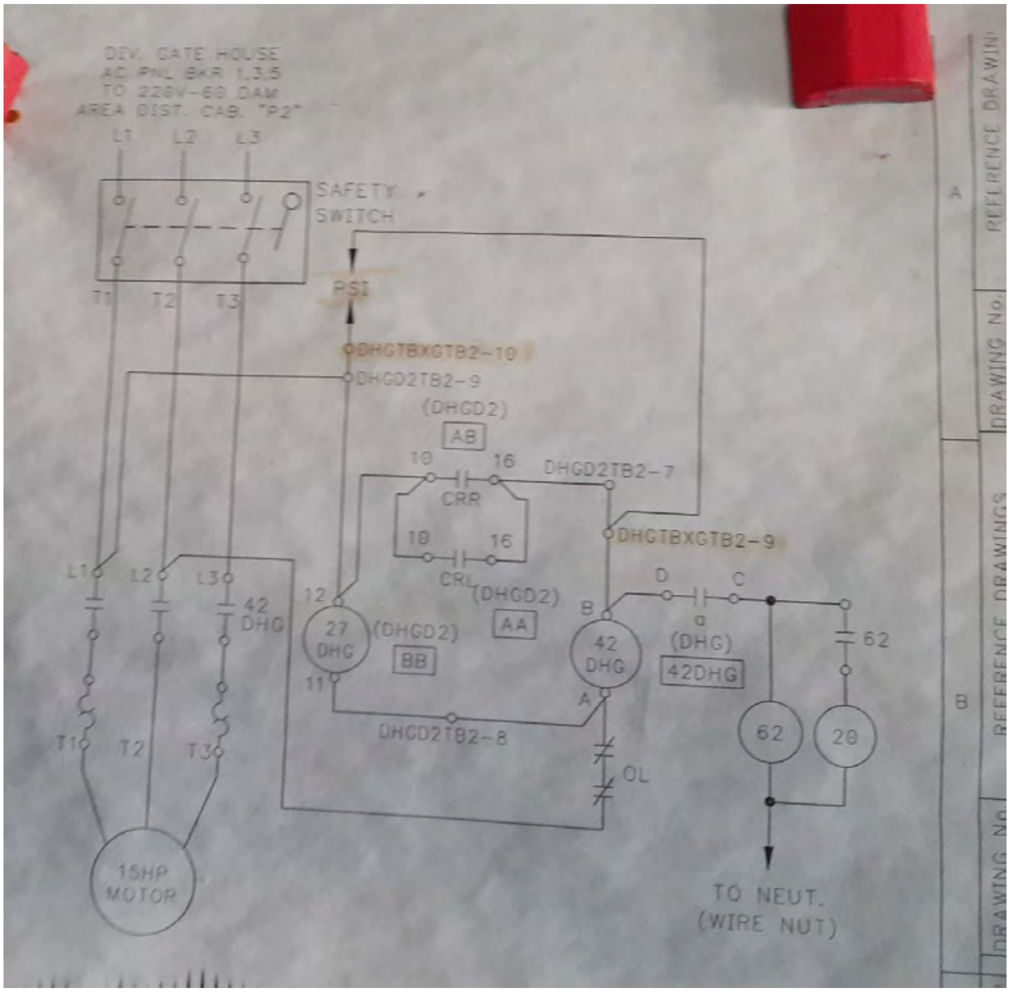


Figure 5.4 Electrical Schematic

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

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

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

5.2.3 COMMUNICATION WITH PACIFICORP ON SYSTEM OPERATION

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

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

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

5.2.4 OPEN ITEMS AND NEXT STEPS

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

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

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

5.3 OPERATIONAL ANALYSIS OF GATE AND HOIST MECHANISM

5.3.1 PAST INSPECTION DATA

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

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

Table 5.1 Diversion Tunnel Gate Testing and Inspection Summary

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

5.3.2 COMMISSIONING DATA

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

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

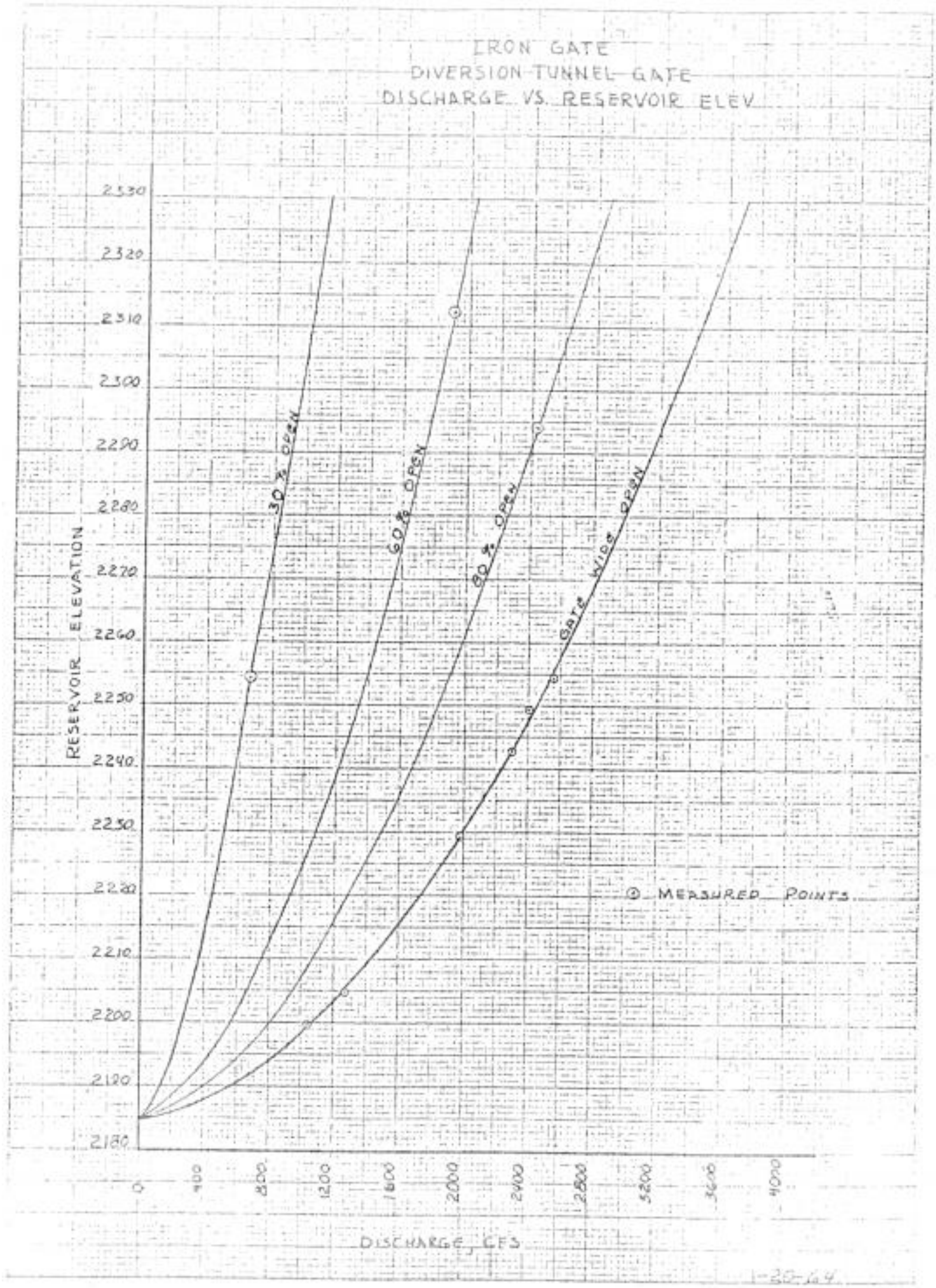


Figure 5.5 Original Iron Gate Rating Curve

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

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

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

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

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

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

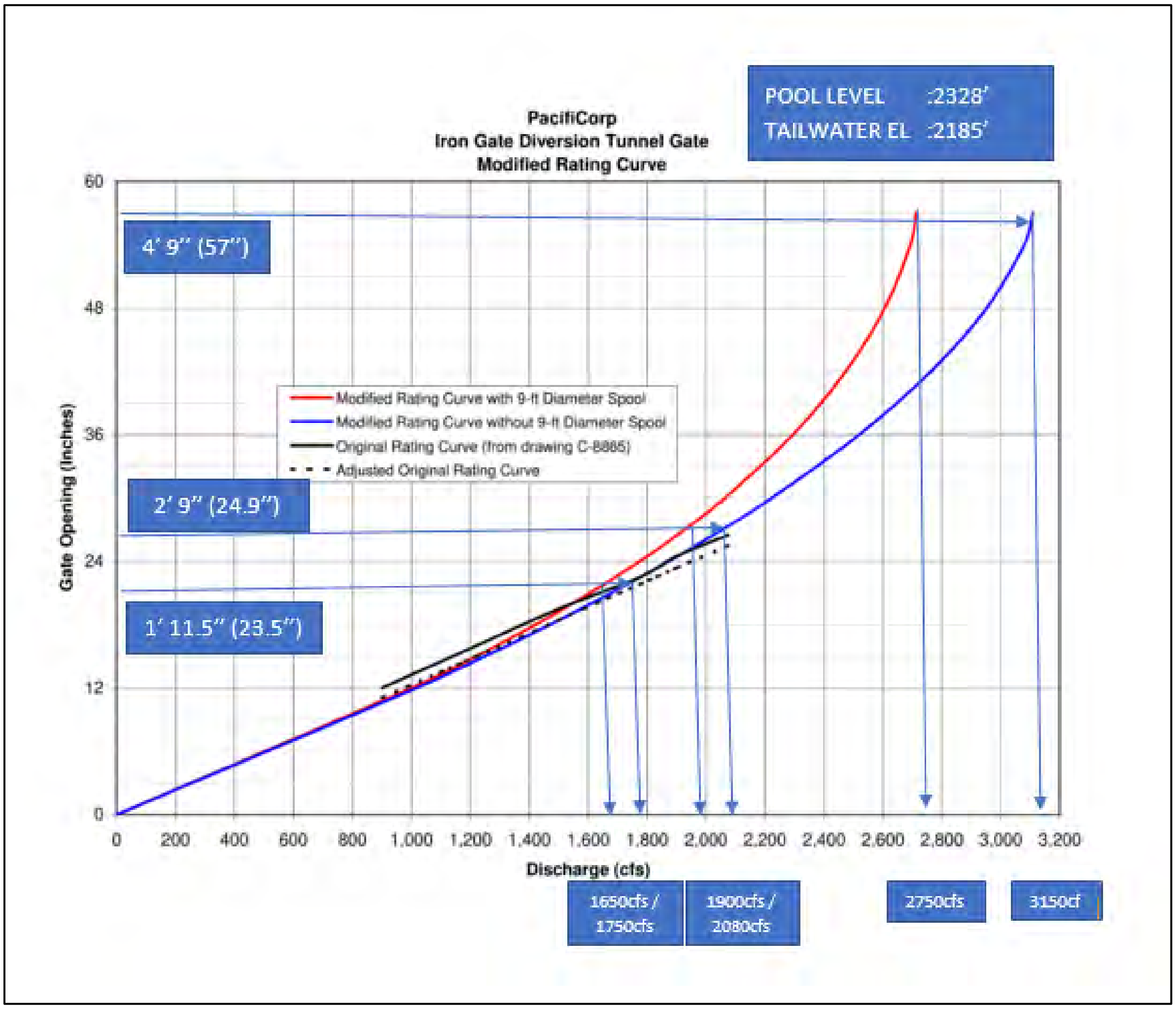
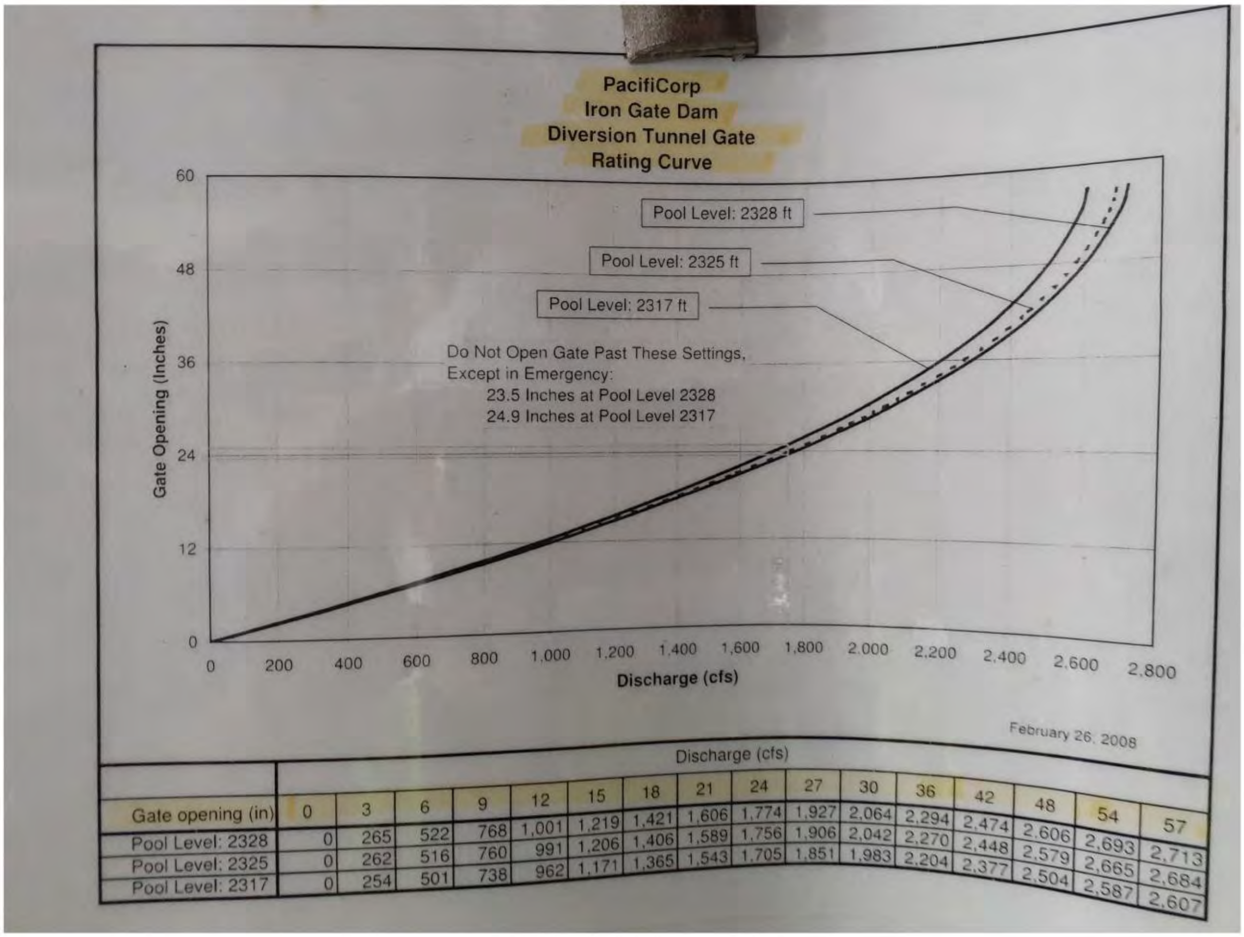


Figure 5.6 Modified Operational Rating Curve After Installation of Blind Flange



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

Figure 5.7 PacifiCorp Operational Tunnel Rating Curve

Attachments:

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

2 – Iron Gate Dam Facility Simulated Drawdown

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YURO K TRIBE

190 Klamath Boulevard • Post Office Box 1027 • Klamath, CA 95548

EXTERNAL TECHNICAL MEMO

Date: December 11th, 2020

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

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

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

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

Introduction

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

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

Acquisition Details

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

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

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

November 17th

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

November 18th

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

November 19th

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

November 20th

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

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

Table 1: Data Acquisition Dates and Daily Work Activity

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

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

Geodetic Control

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

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

Low-Level Outlet Reconnaissance

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

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

Conventional Total Station Survey

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

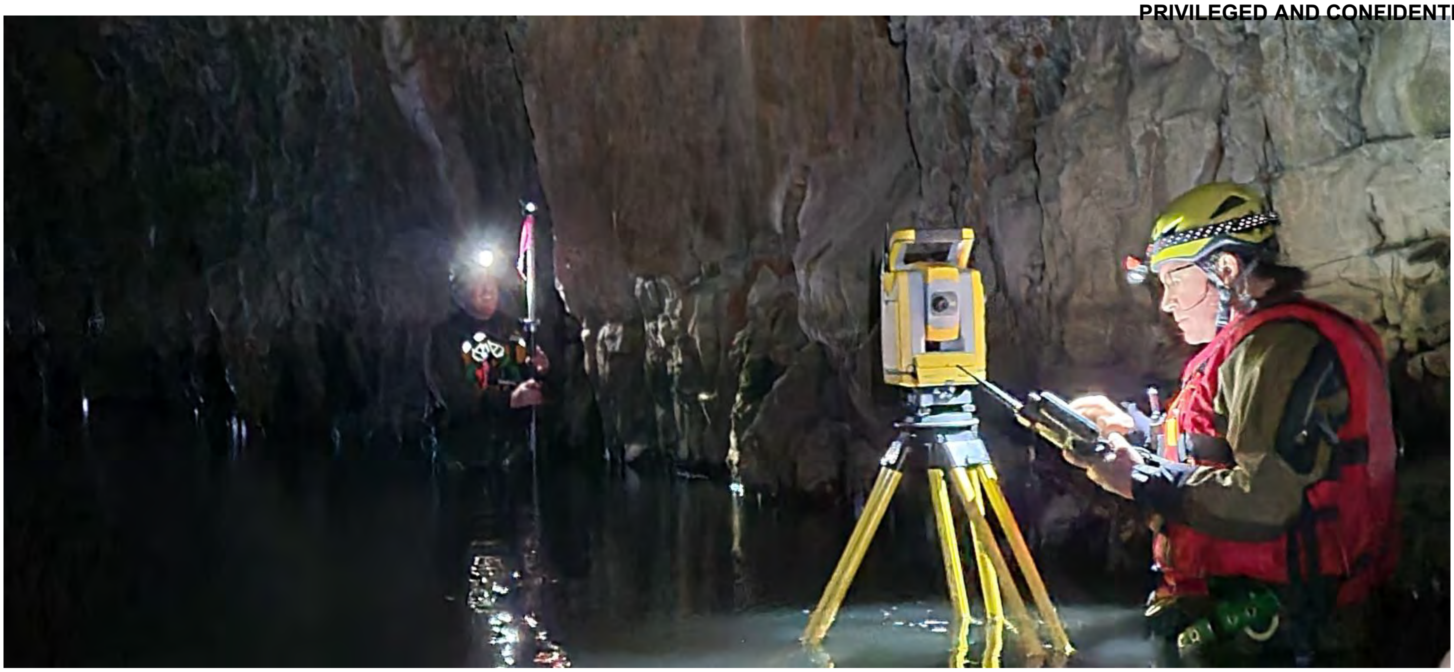

Photo 2: Bathymetric Data Collection Using Trimble S-5 Robotic Total Station

Mobile Ground-Based LiDAR Survey

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

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

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

DATA PROCESSING

Geodetic Control

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

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

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

Conventional Surveys

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

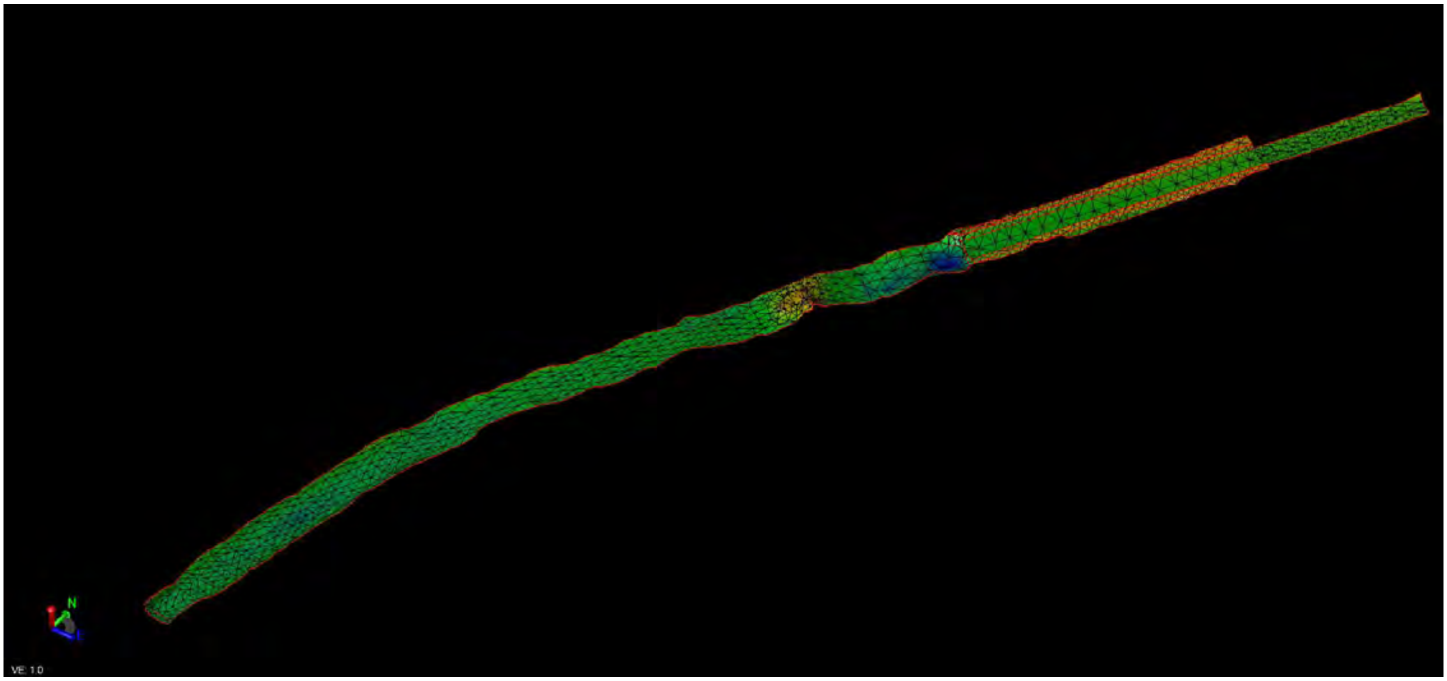

Figure 2: Isometric View of the TIN Surface Created from the Bathymetric Survey Points

Mobile Ground-Based LiDAR Survey

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

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

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

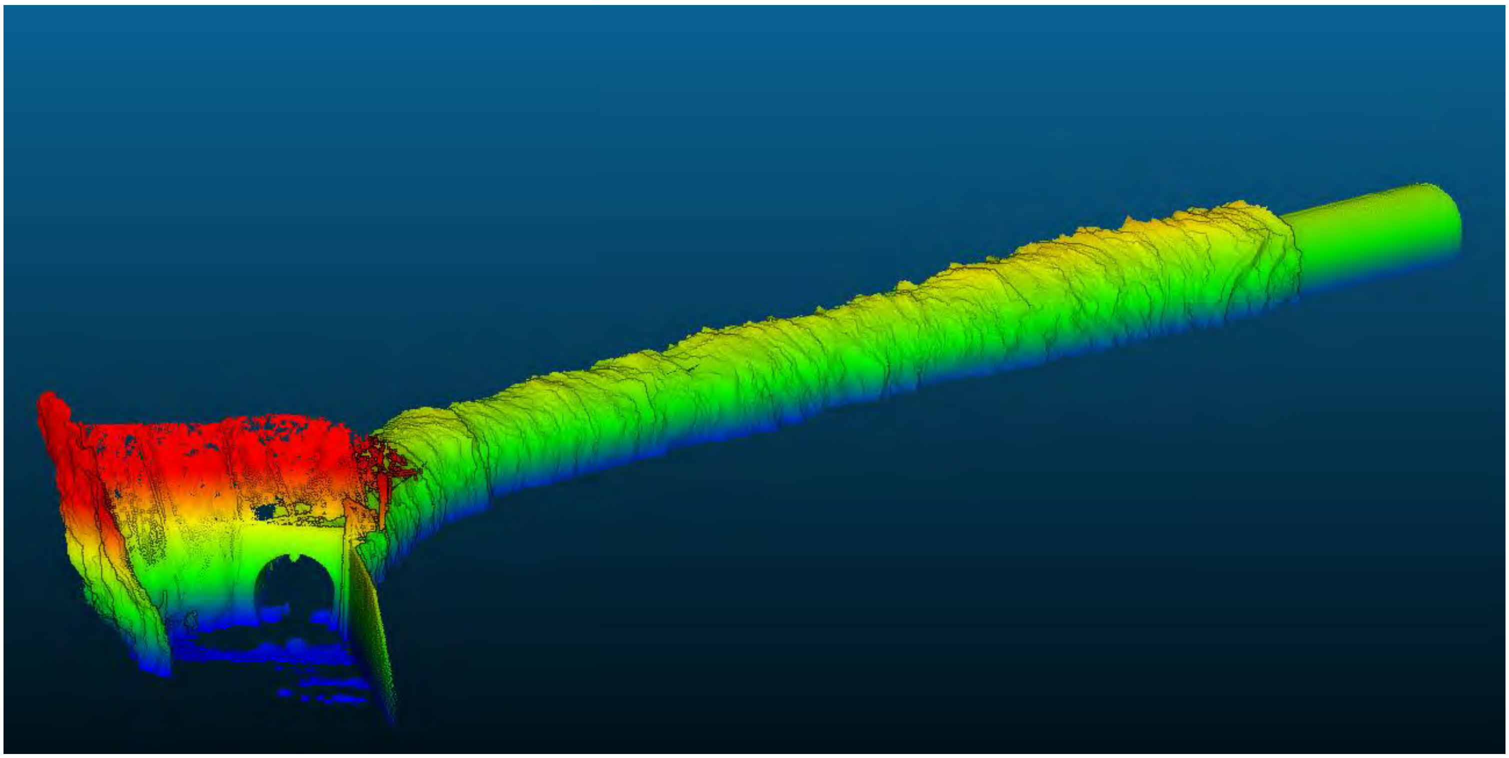

Figure 3: Isometric View of the Low-Level Outlet Tunnel Structure

Data Integration and Product Development

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

<u>Trimble Business Center Advanced</u>

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

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

CloudCompare

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

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

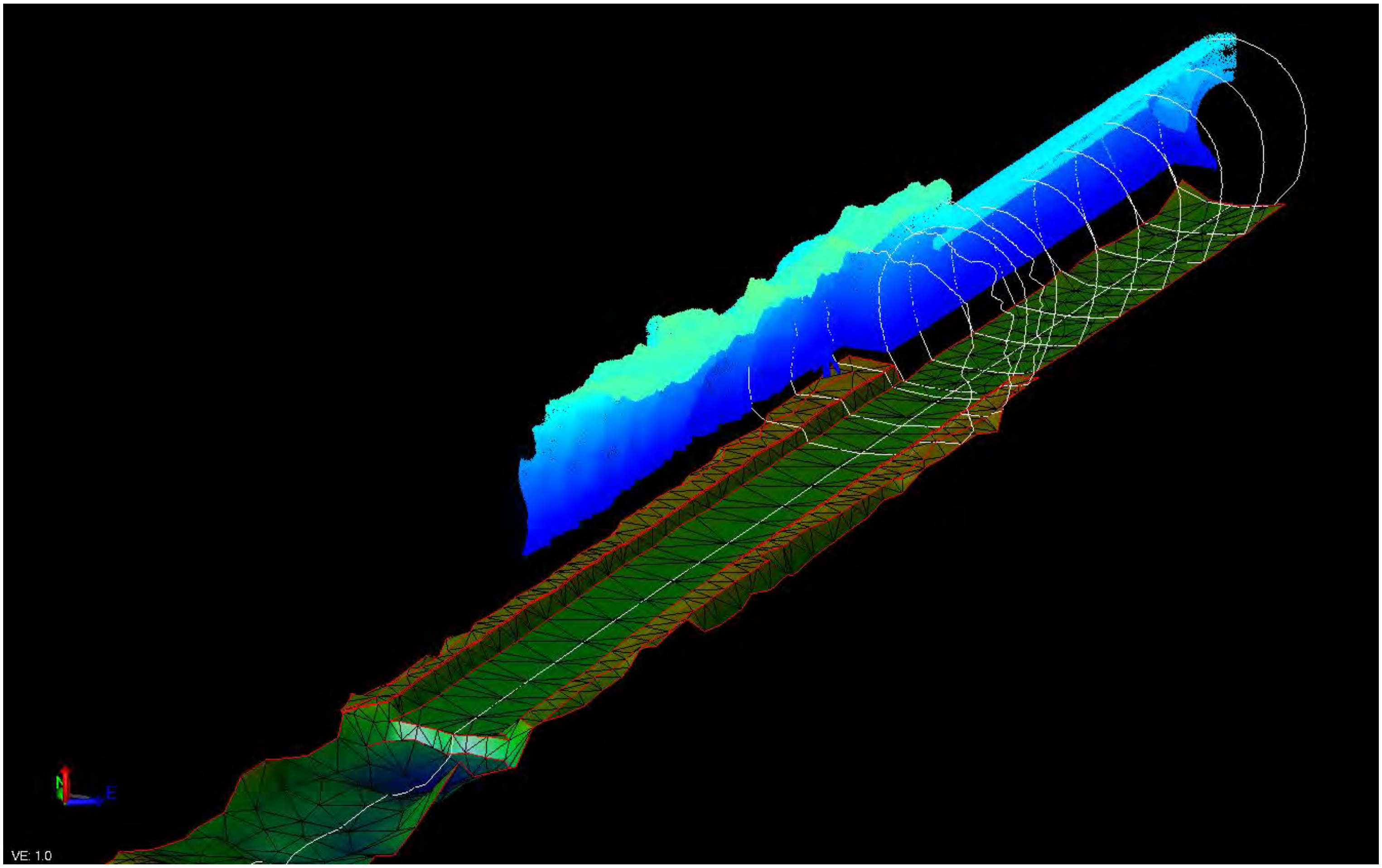

Figure 4: Isometric View of Bathymetric TIN, Outlet Structure Point Cloud and Extracted Features

Deliverables

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

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

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

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

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

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

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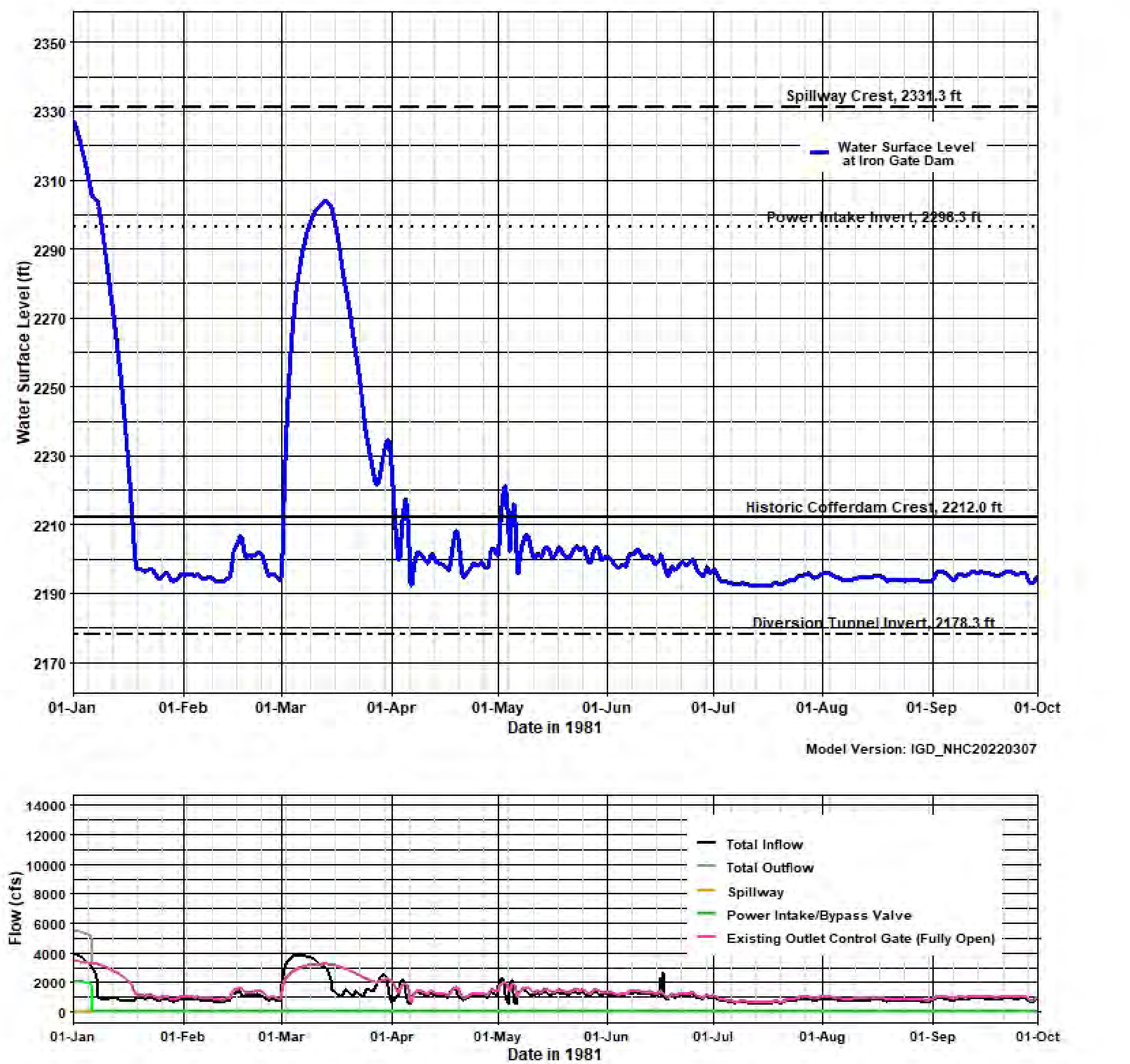

Figure 1 - Iron Gate Dam Facility Simulated Drawdown - Year 1981

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

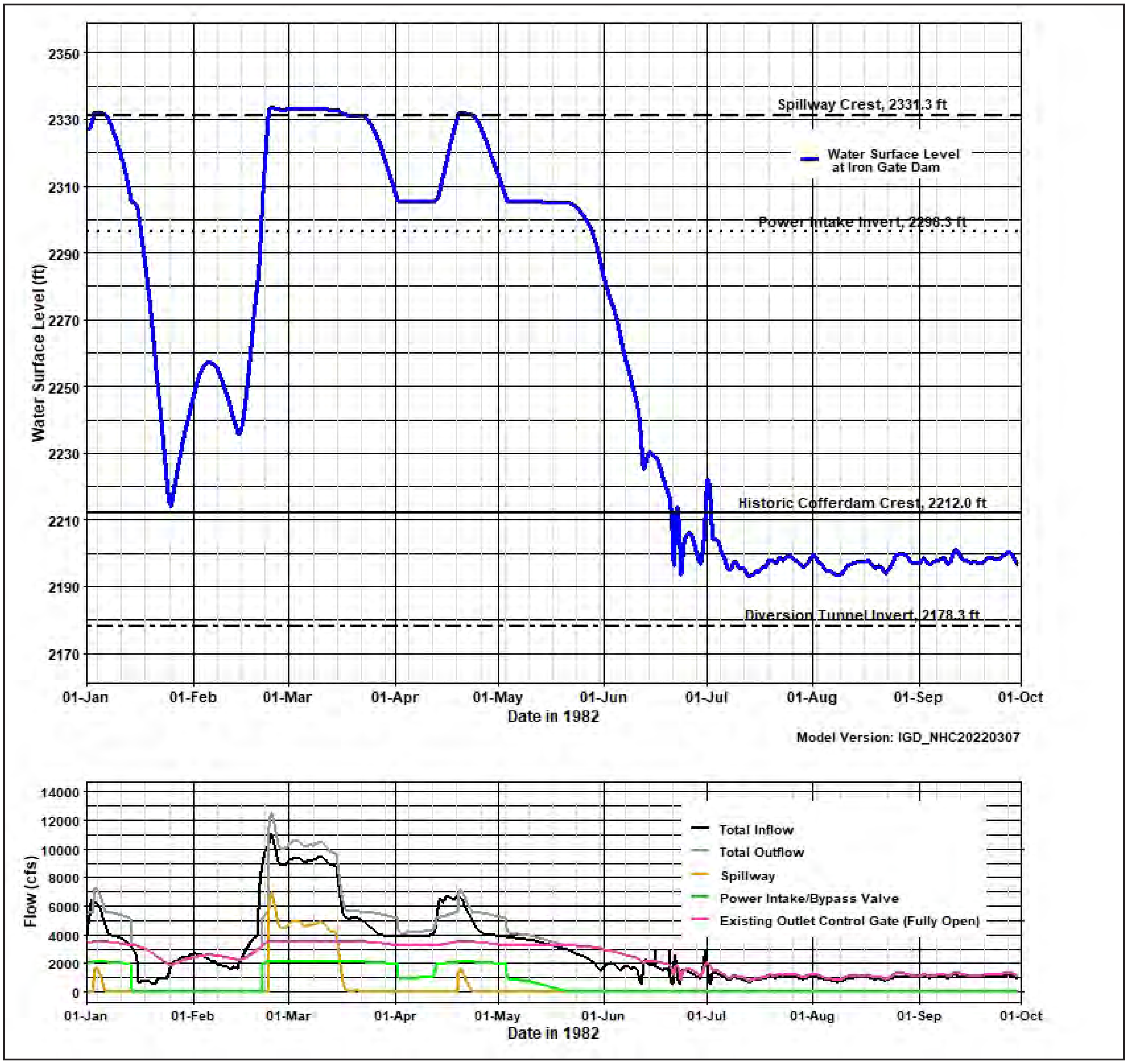

Figure 2 - Iron Gate Dam Facility Simulated Drawdown - Year 1982

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SIMULATED DRAWDOWN

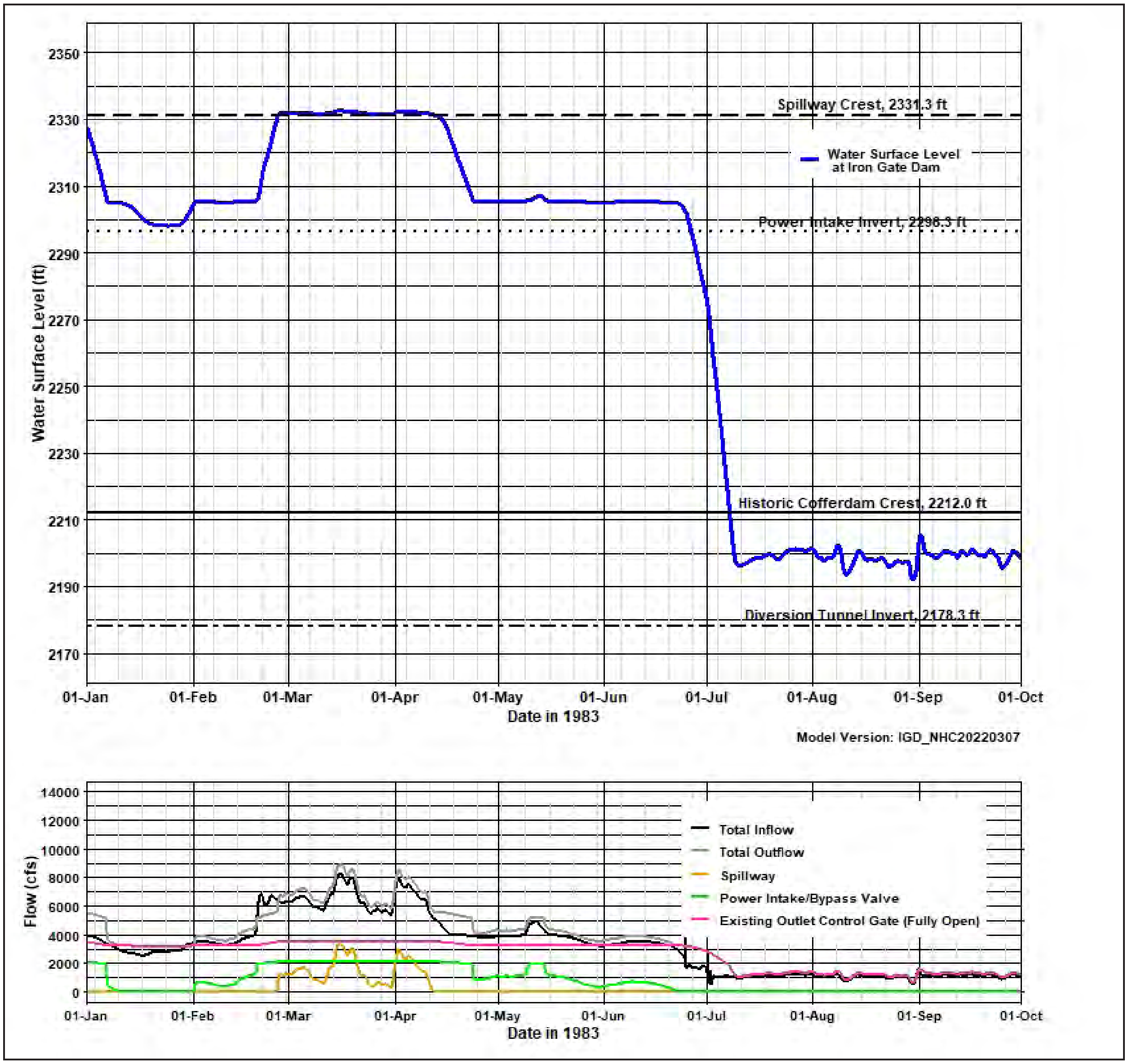

Figure 3 - Iron Gate Dam Facility Simulated Drawdown - Year 1983

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

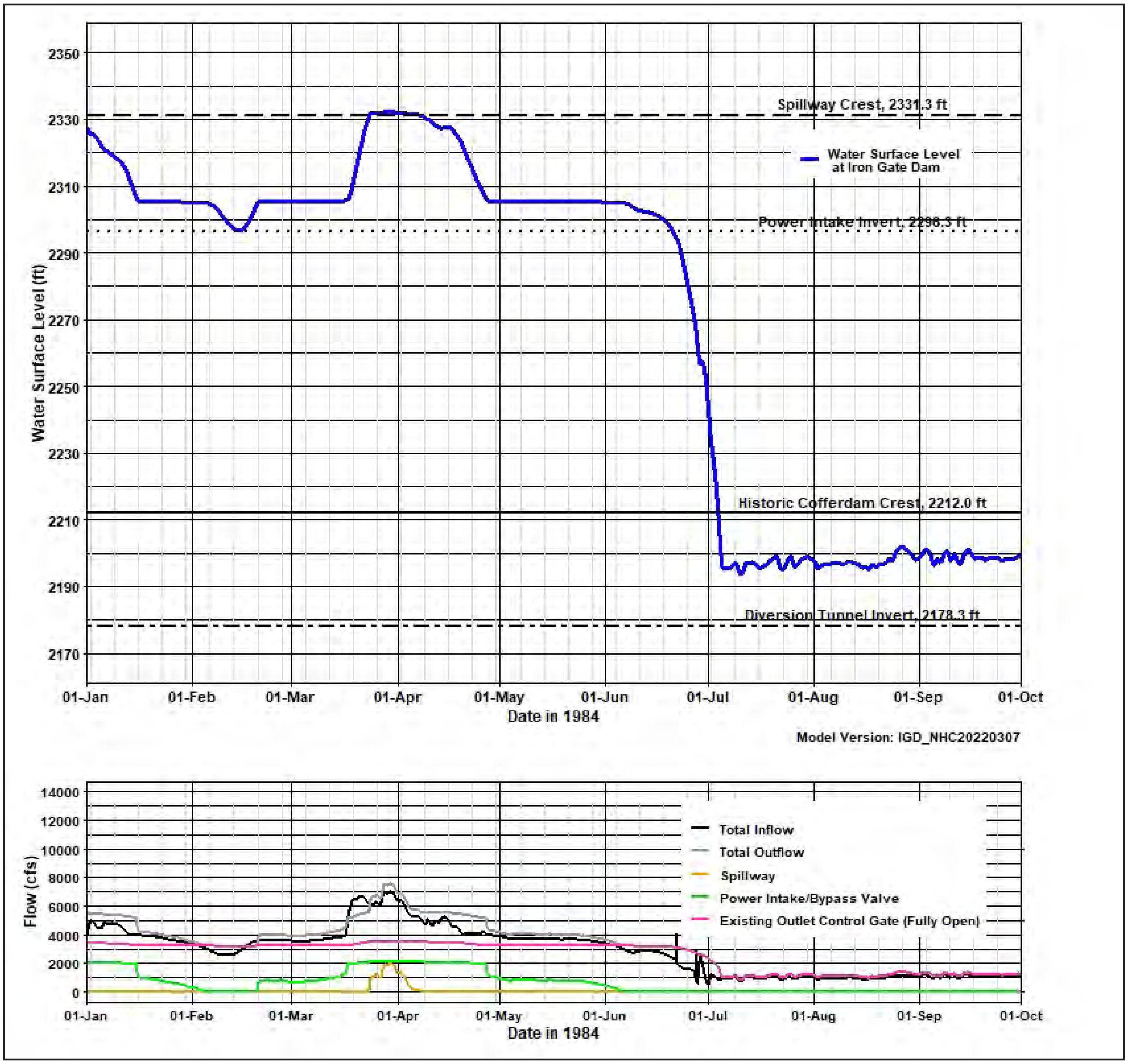

Figure 4 - Iron Gate Dam Facility Simulated Drawdown - Year 1984

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

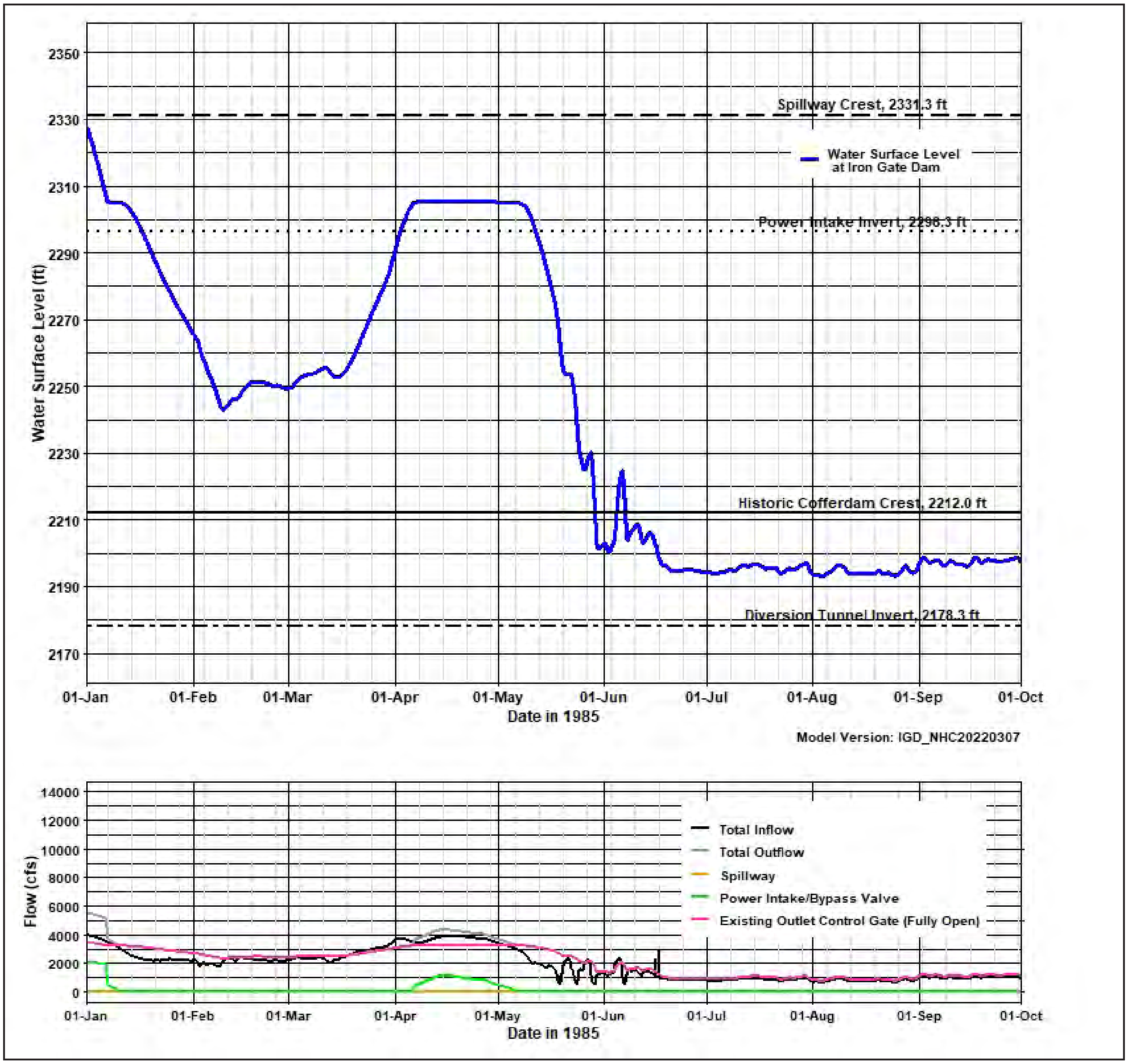

Figure 5 - Iron Gate Dam Facility Simulated Drawdown - Year 1985

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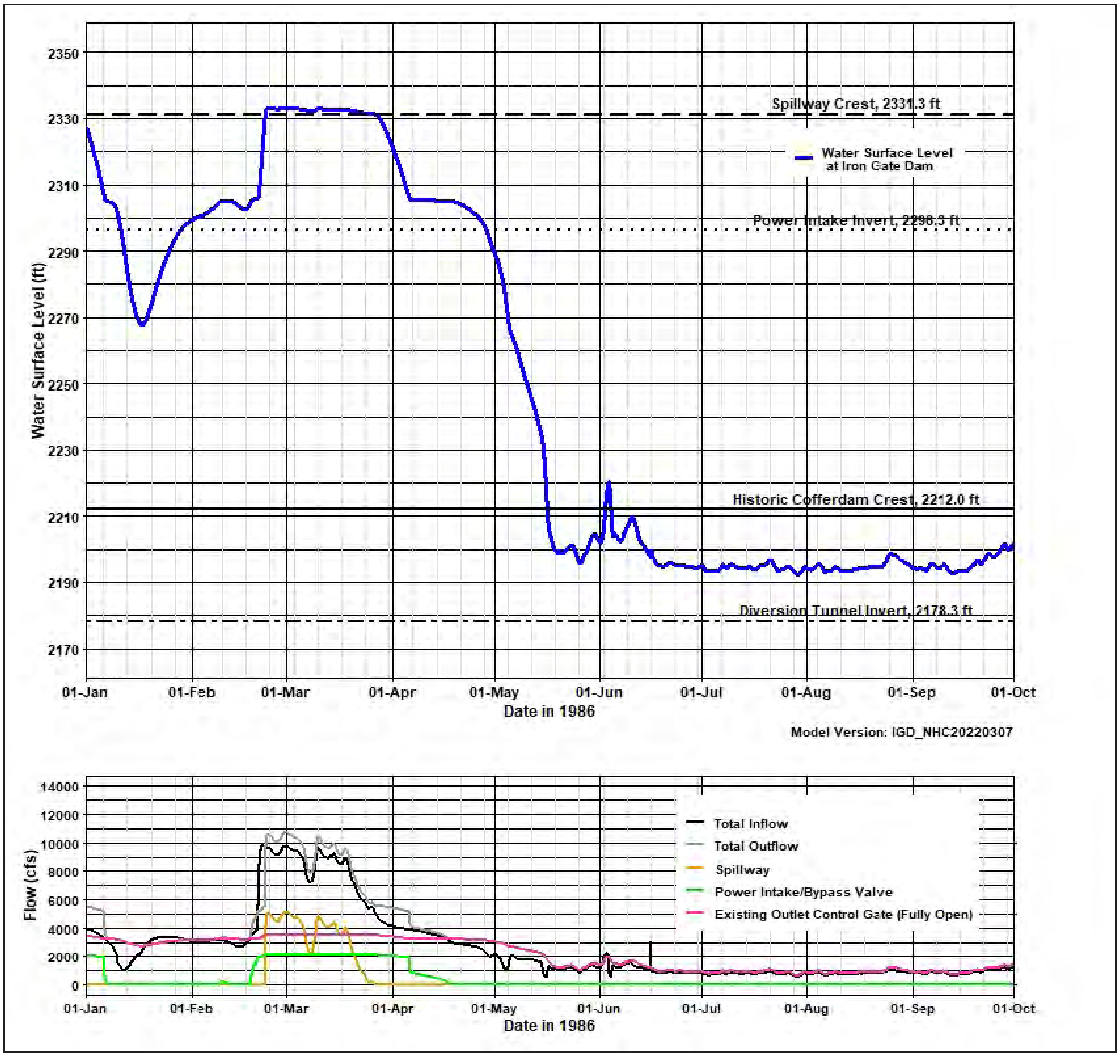

Figure 6 - Iron Gate Dam Facility Simulated Drawdown - Year 1986

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

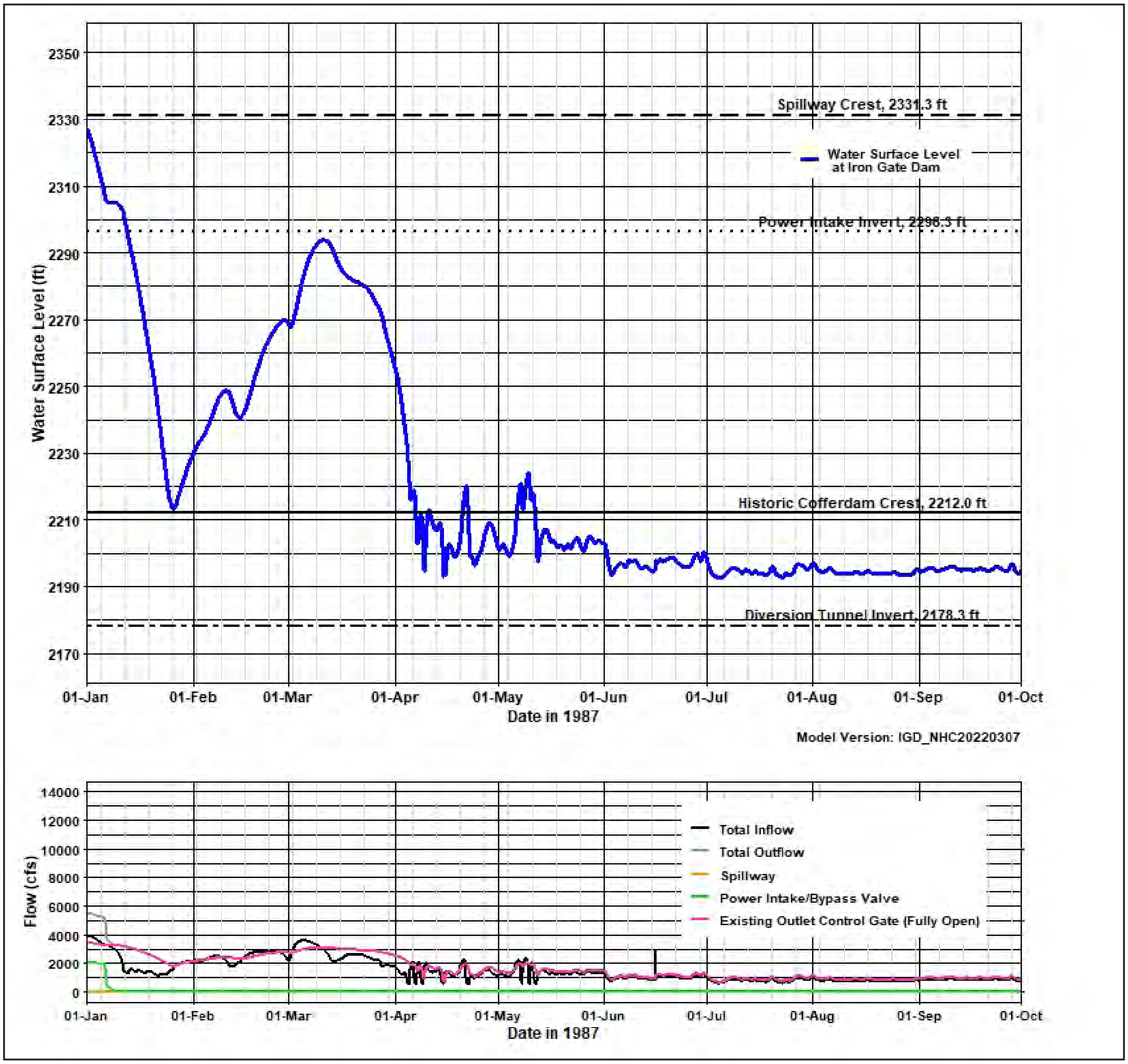

Figure 7 - Iron Gate Dam Facility Simulated Drawdown - Year 1987

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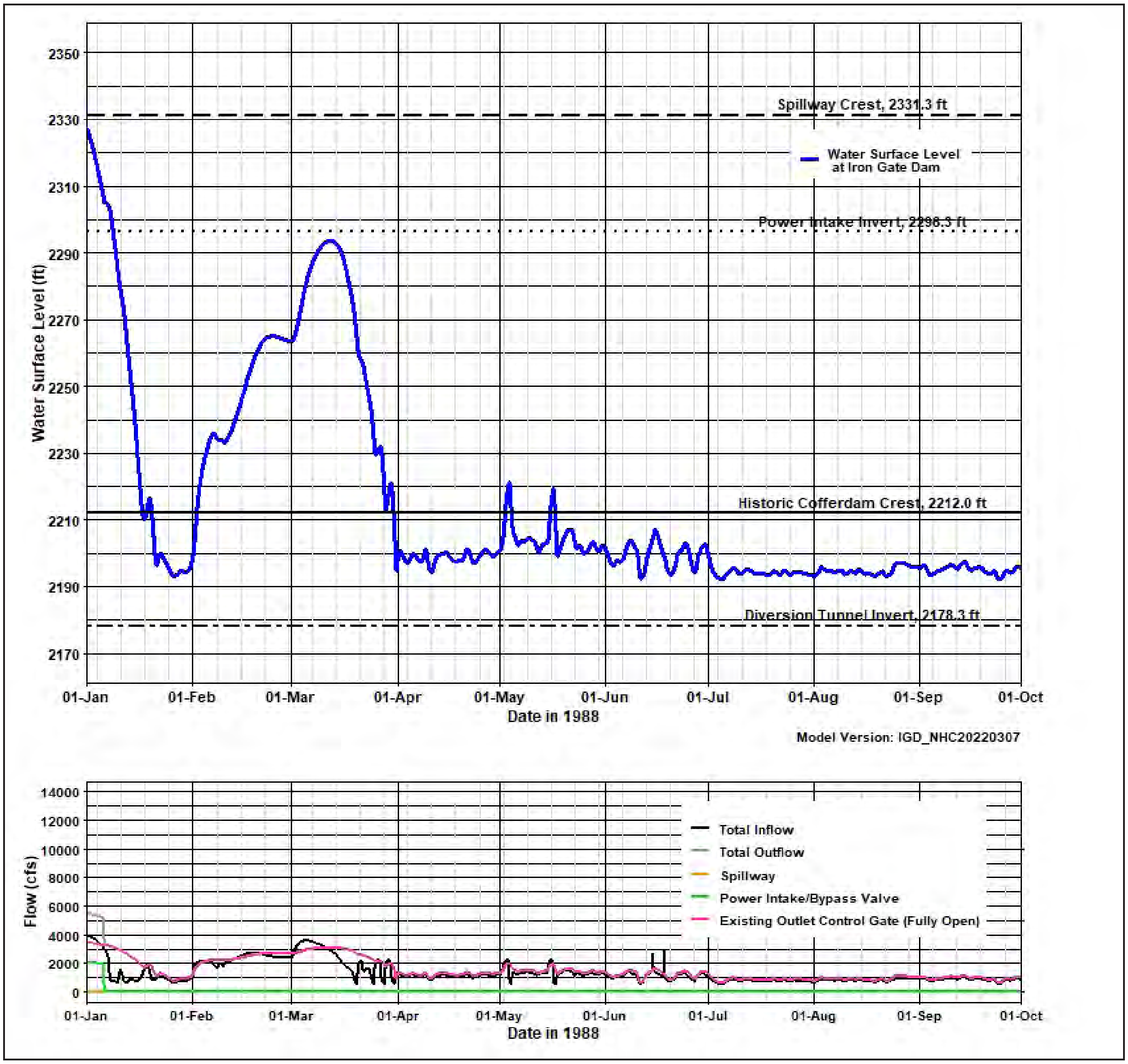

Figure 8 - Iron Gate Dam Facility Simulated Drawdown - Year 1988

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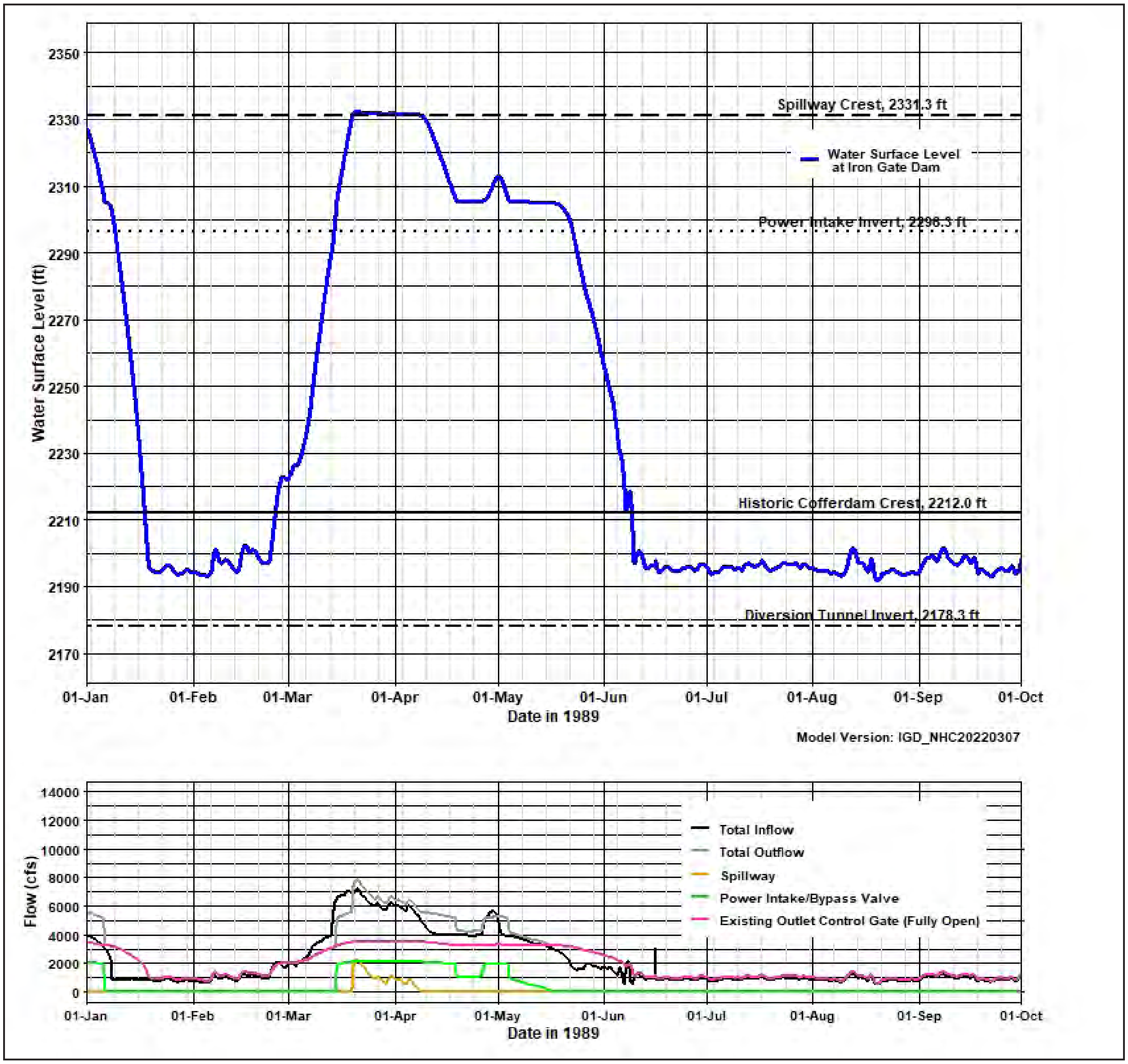

Figure 9 - Iron Gate Dam Facility Simulated Drawdown - Year 1989

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

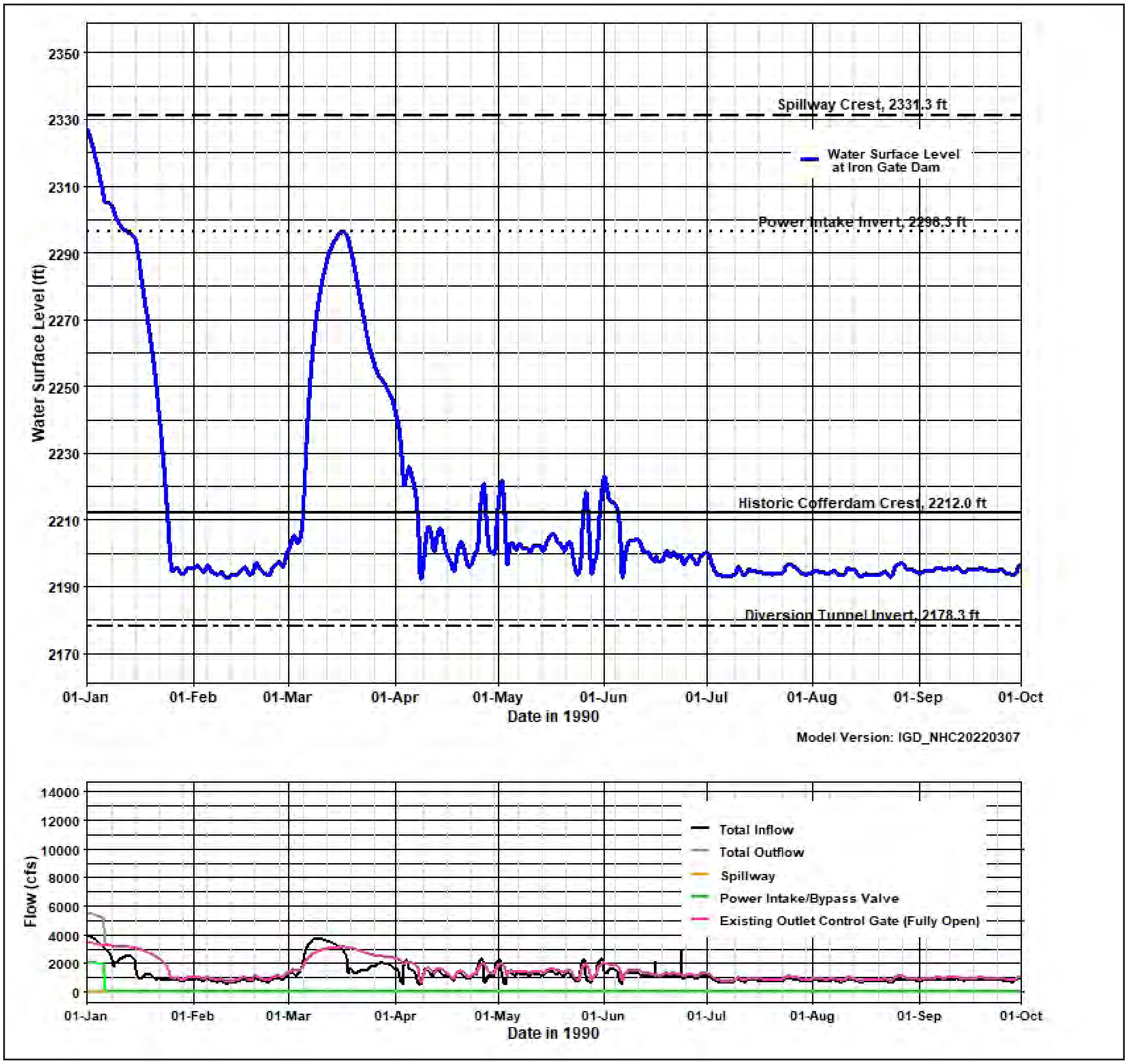

Figure 10 - Iron Gate Dam Facility Simulated Drawdown - Year 1990

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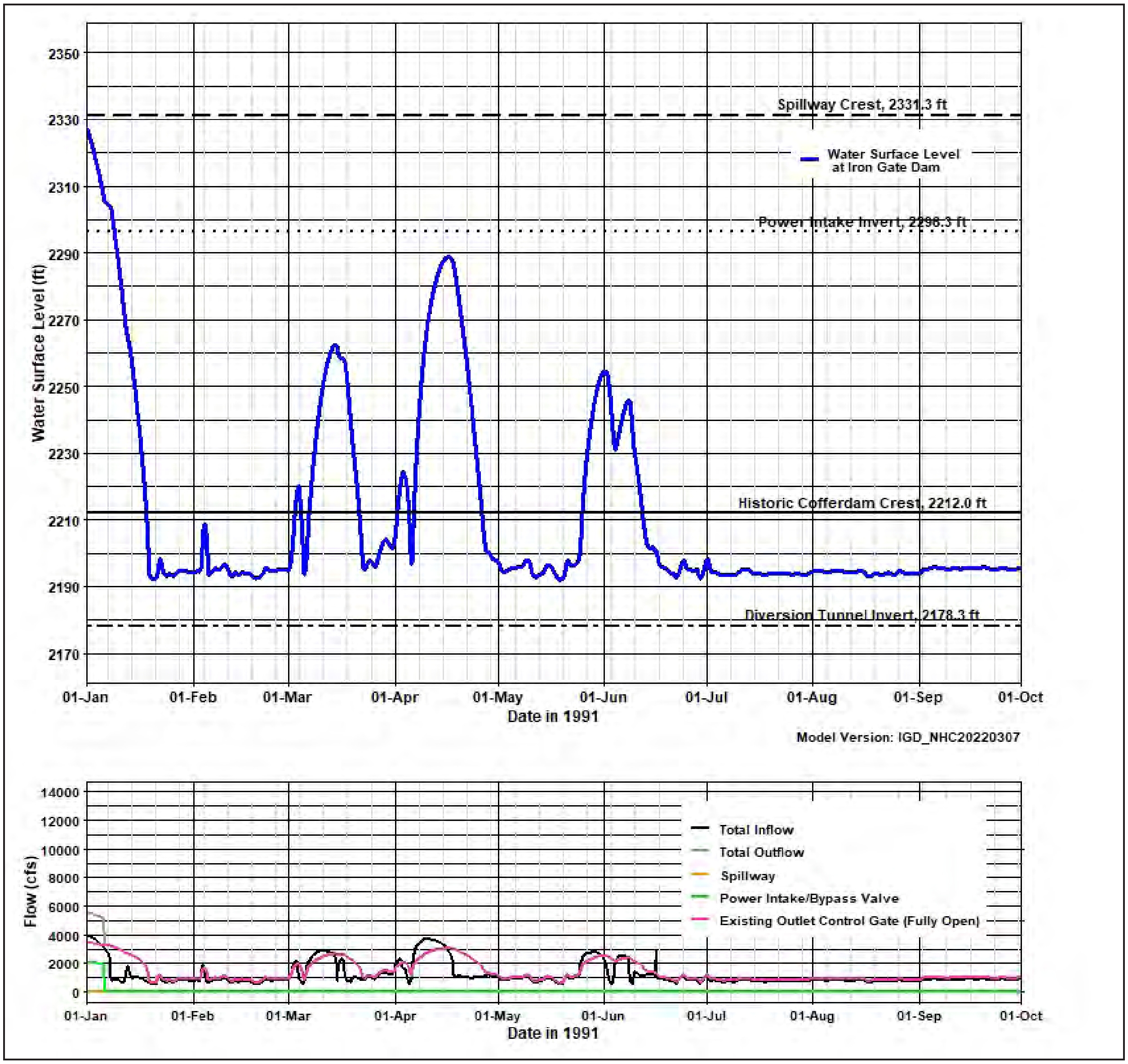

Figure 11 - Iron Gate Dam Facility Simulated Drawdown - Year 1991

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IRON GATE DAM FACILITY
SIMULATED DRAWDOWN

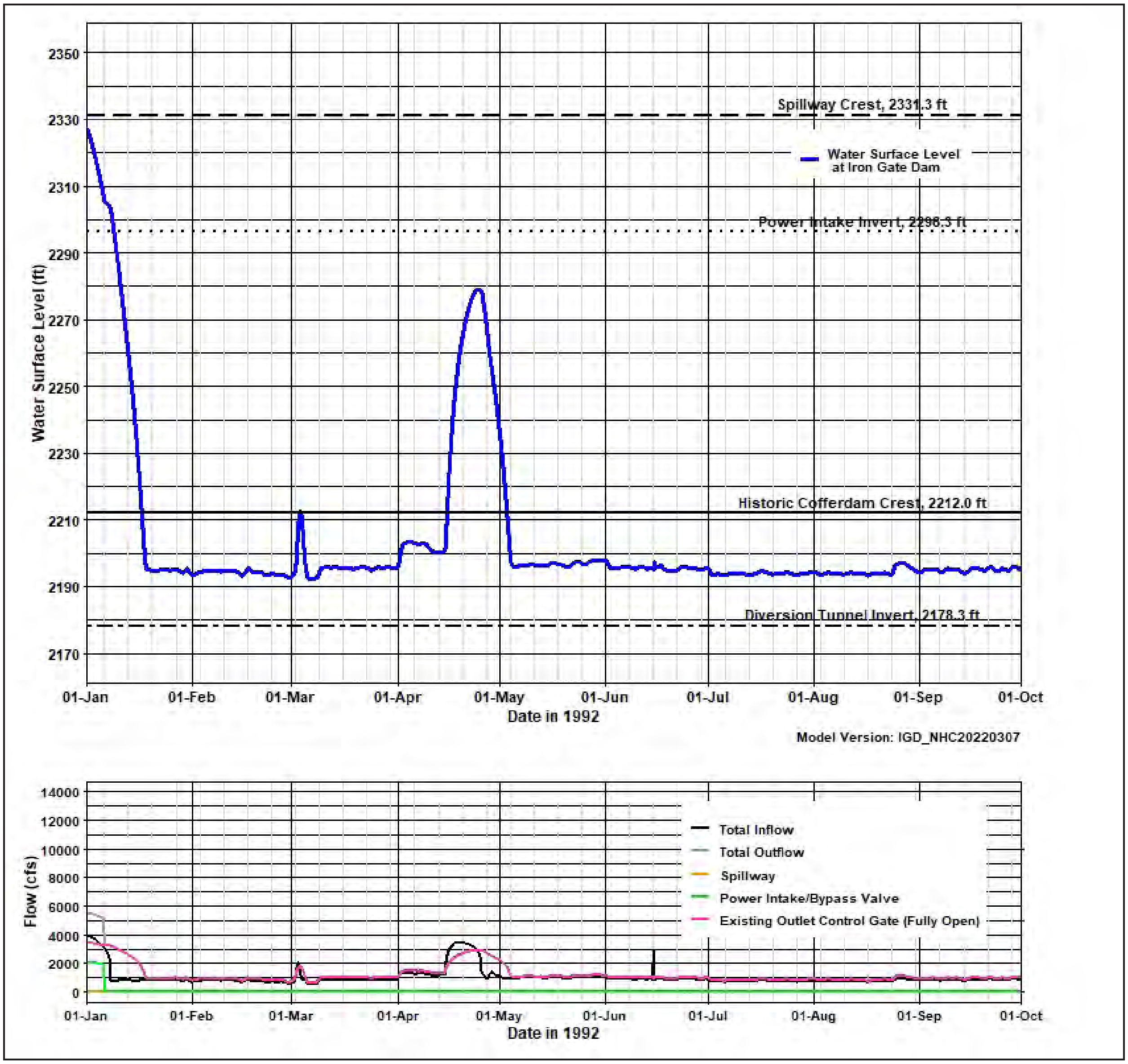

Figure 12 - Iron Gate Dam Facility Simulated Drawdown - Year 1992

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

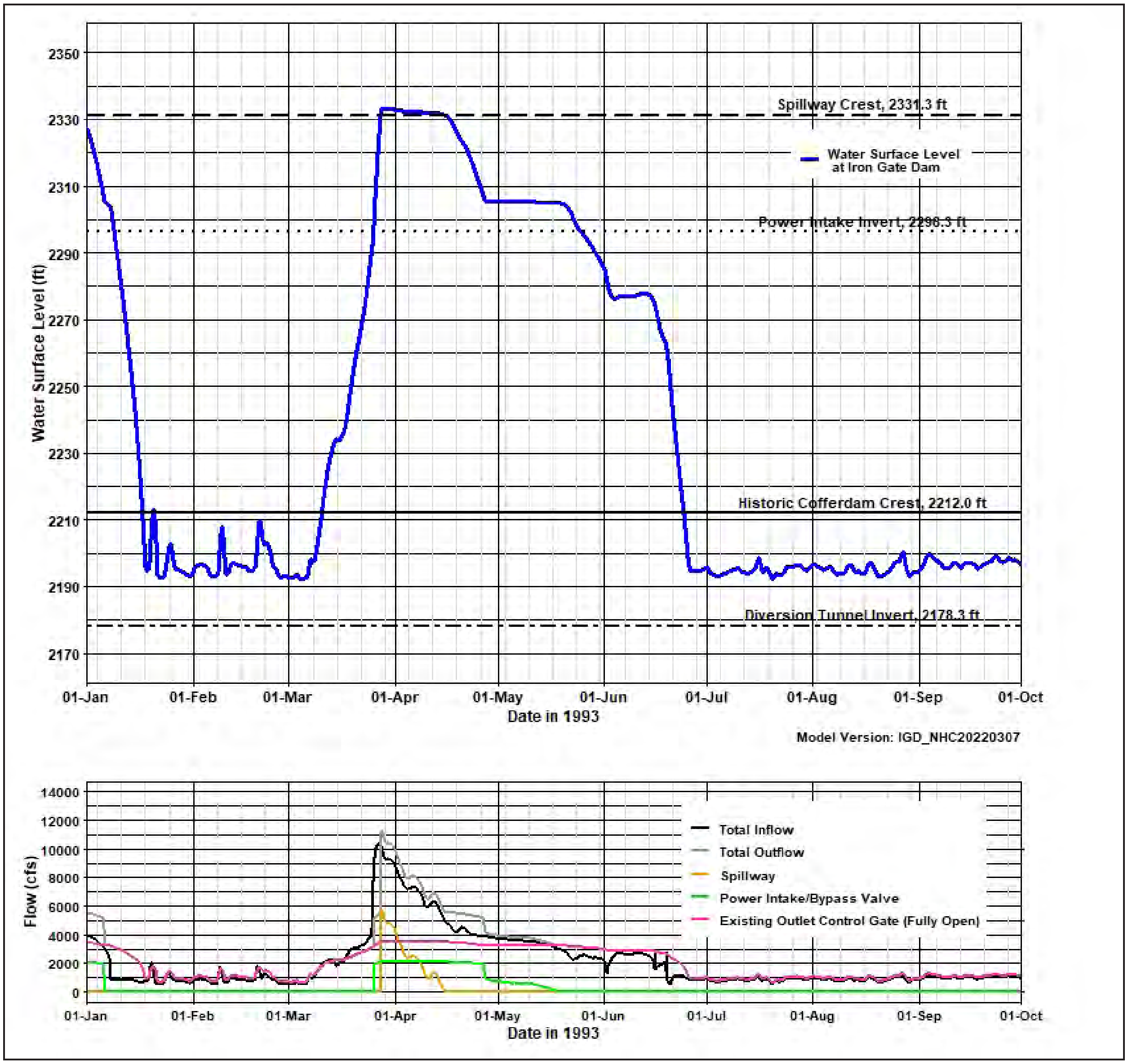

Figure 13 - Iron Gate Dam Facility Simulated Drawdown - Year 1993

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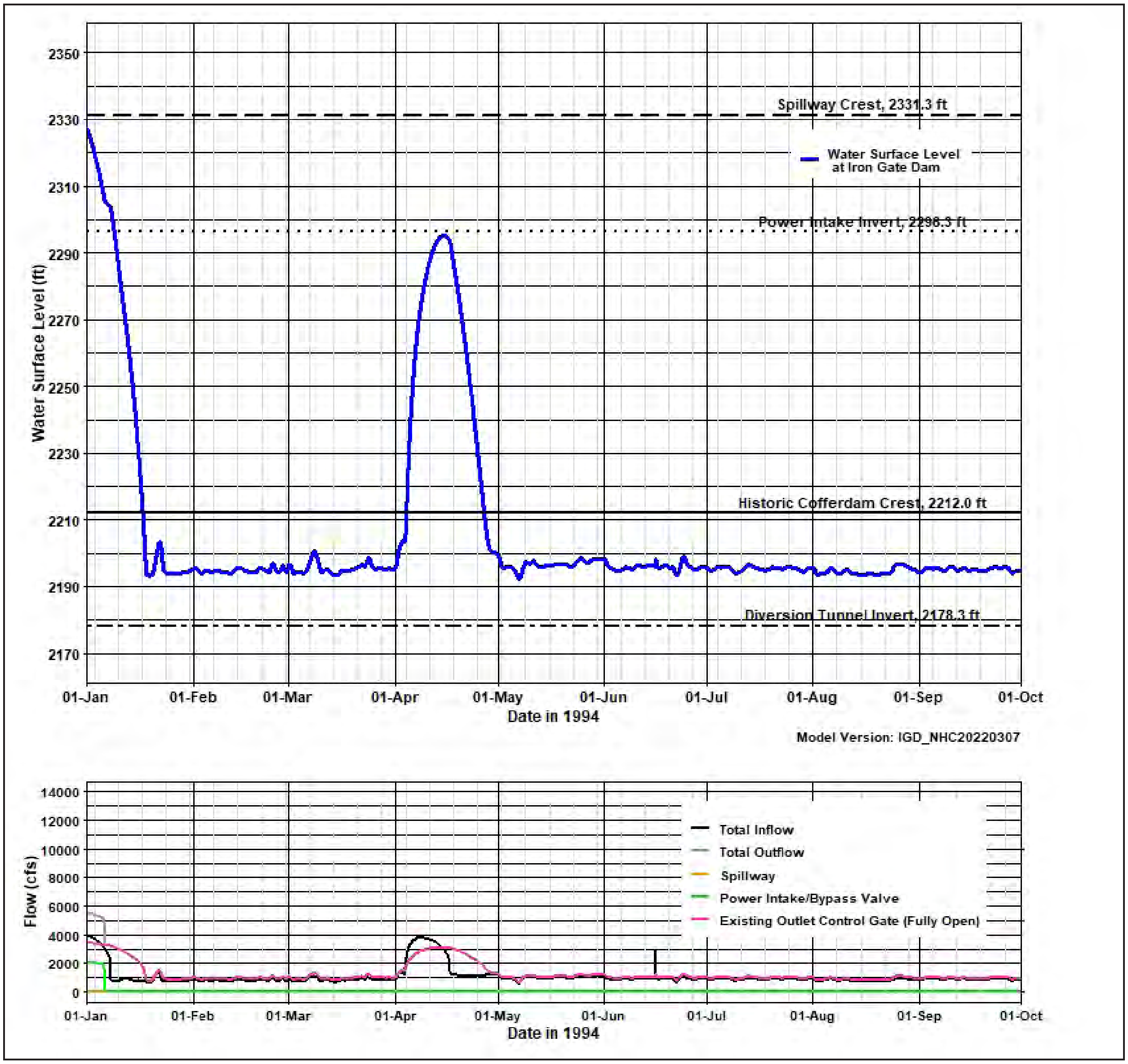

Figure 14 - Iron Gate Dam Facility Simulated Drawdown - Year 1994

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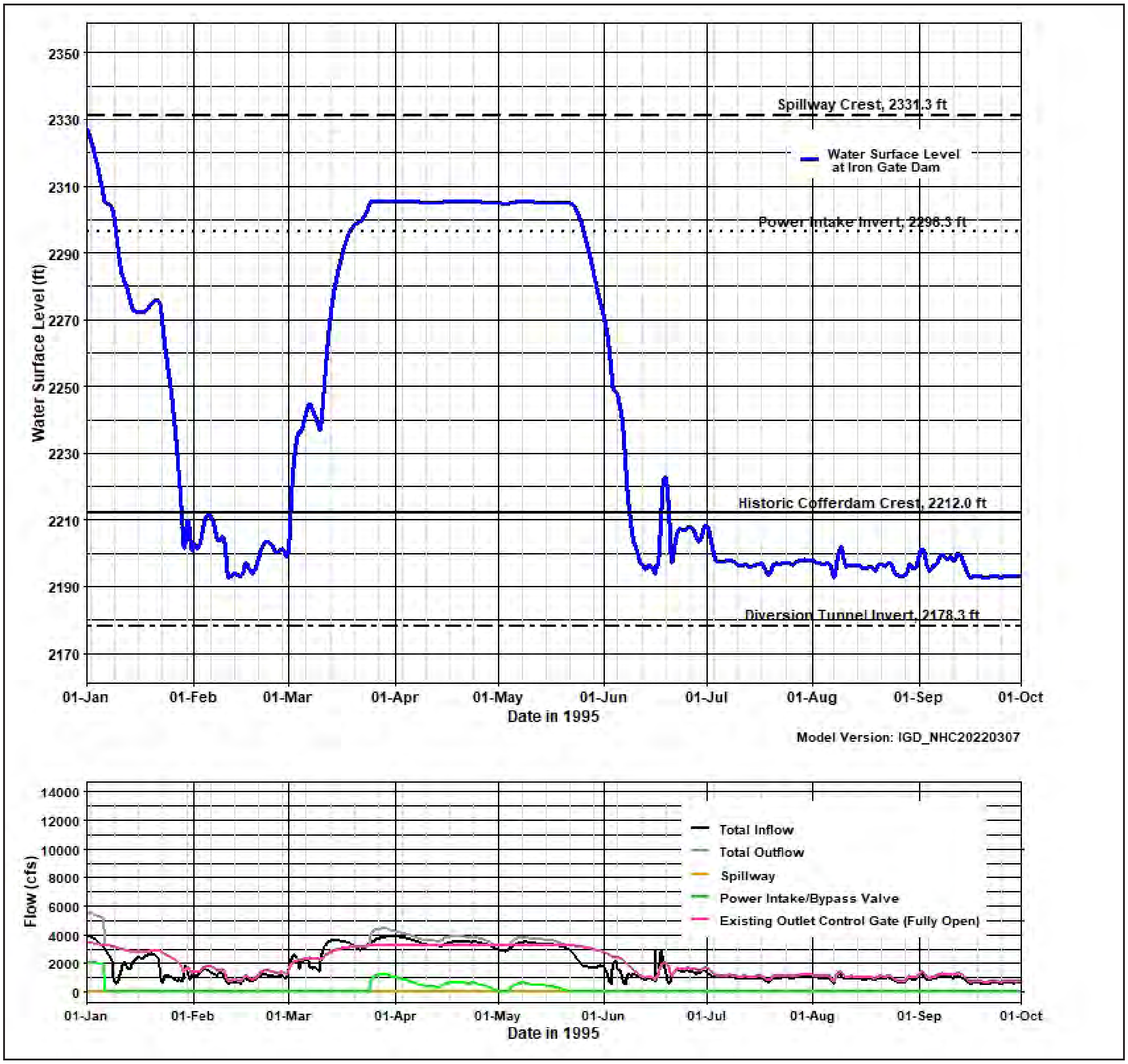

Figure 15 - Iron Gate Dam Facility Simulated Drawdown - Year 1995

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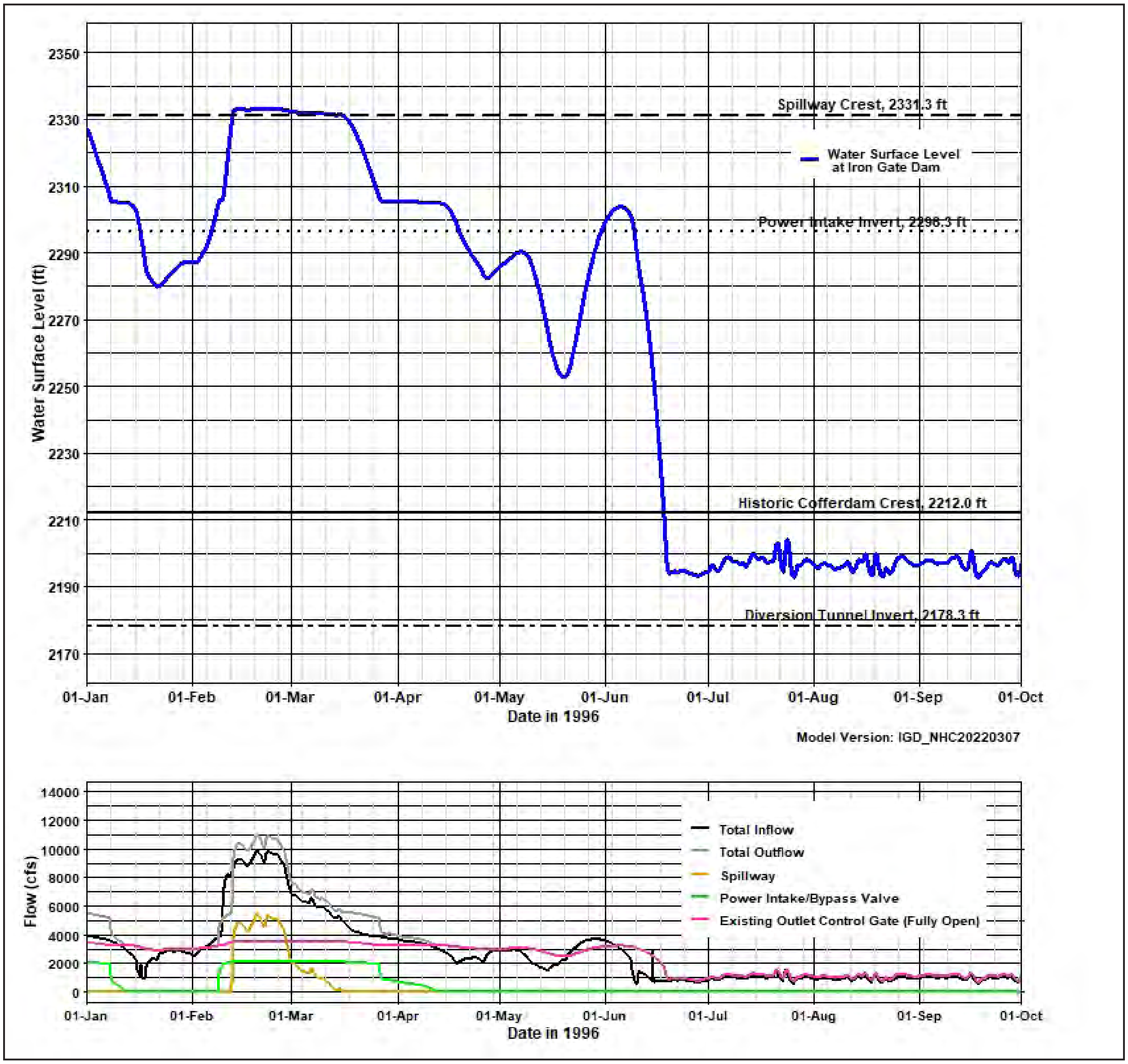

Figure 16 - Iron Gate Dam Facility Simulated Drawdown - Year 1996

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SIMULATED DRAWDOWN

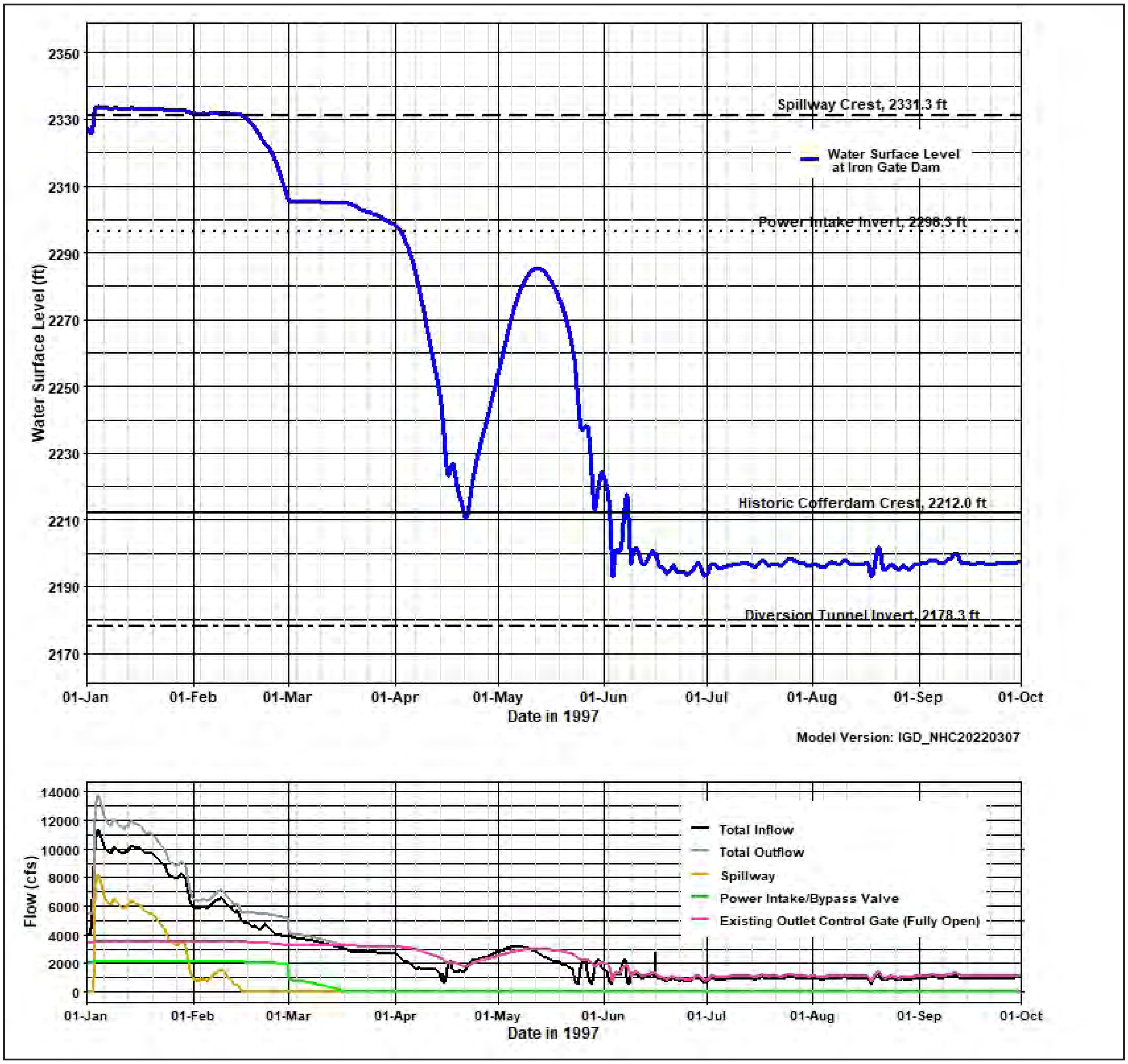

Figure 17 - Iron Gate Dam Facility Simulated Drawdown - Year 1997

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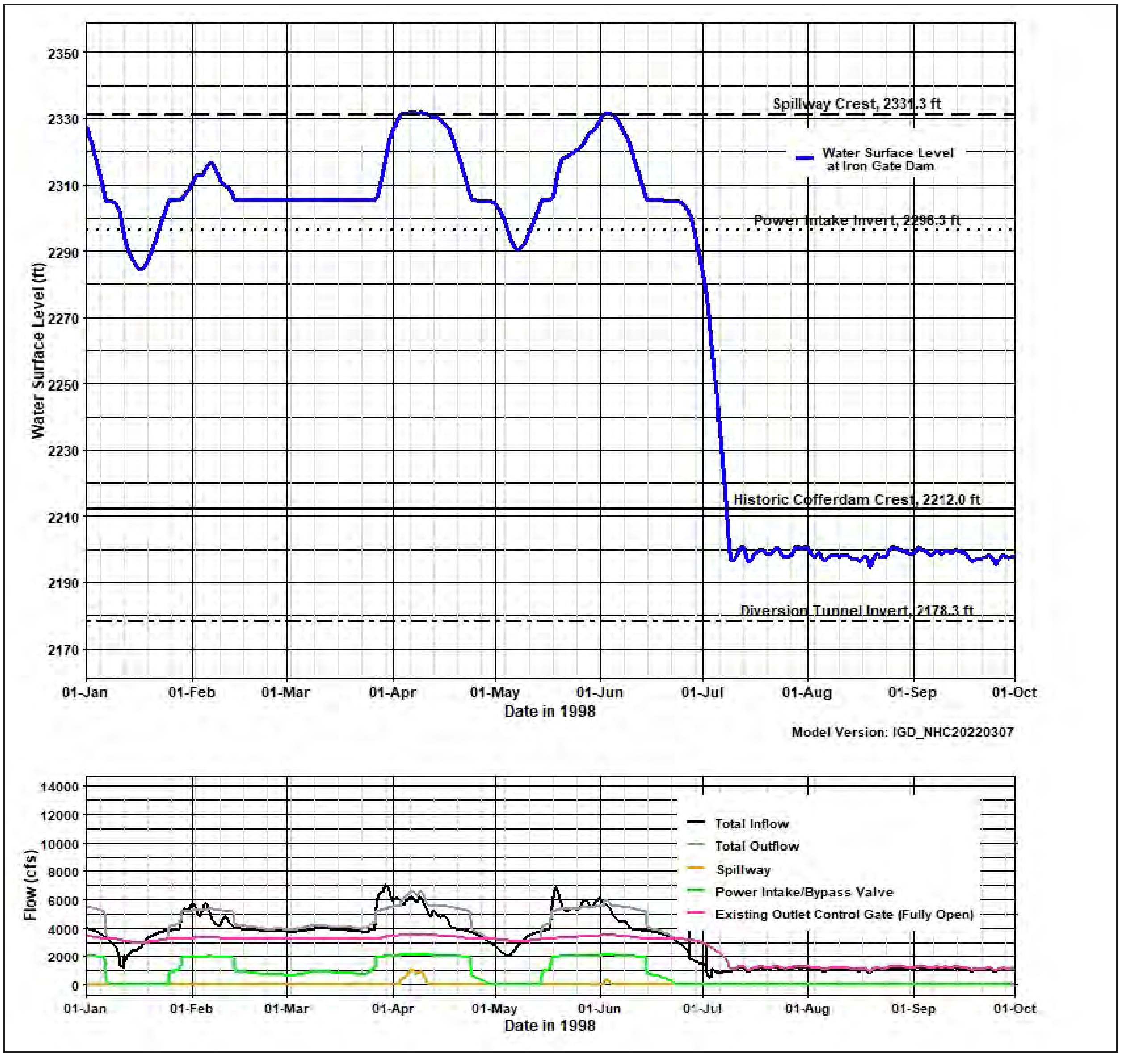

Figure 18 - Iron Gate Dam Facility Simulated Drawdown - Year 1998

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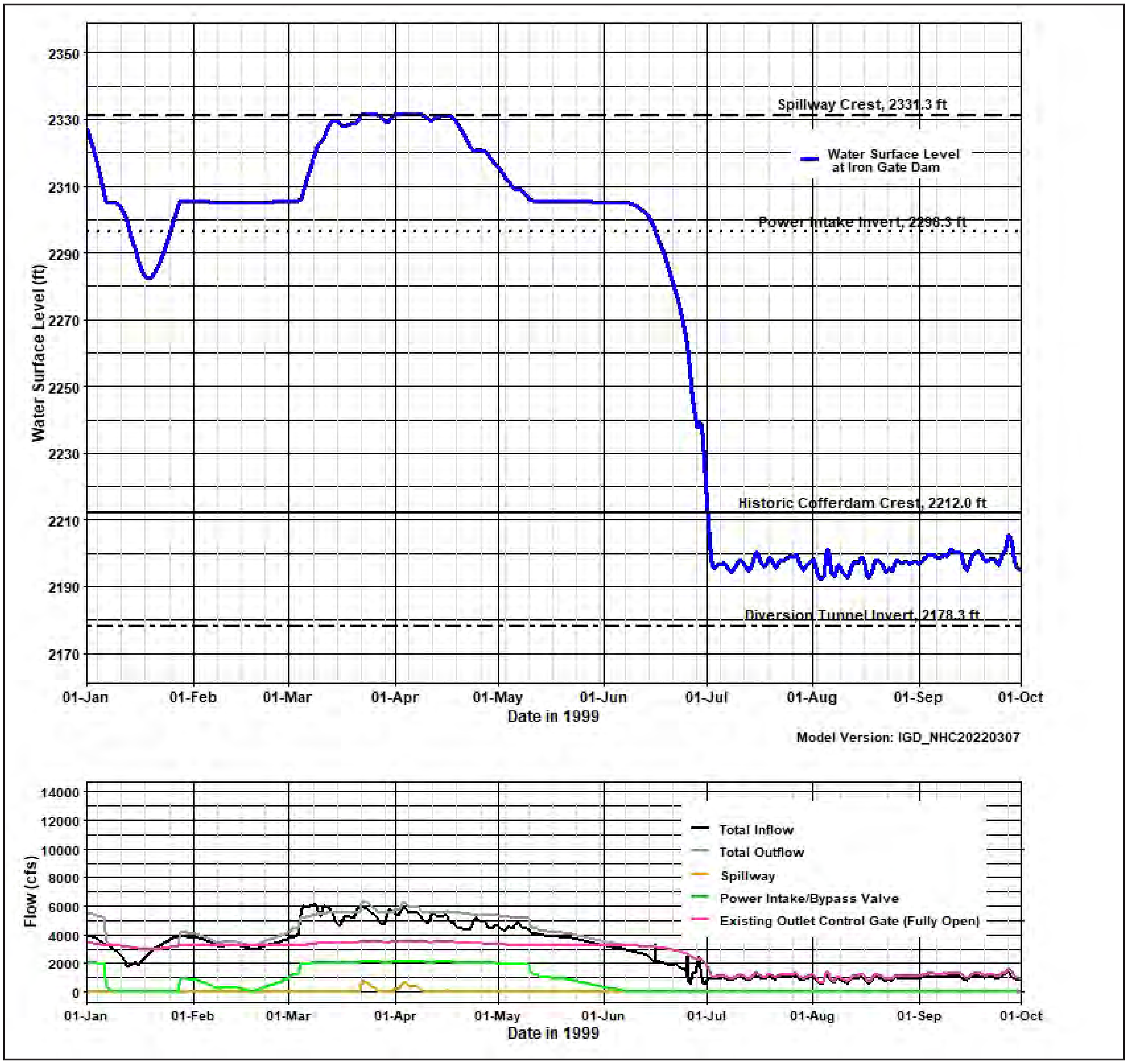

Figure 19 - Iron Gate Dam Facility Simulated Drawdown - Year 1999

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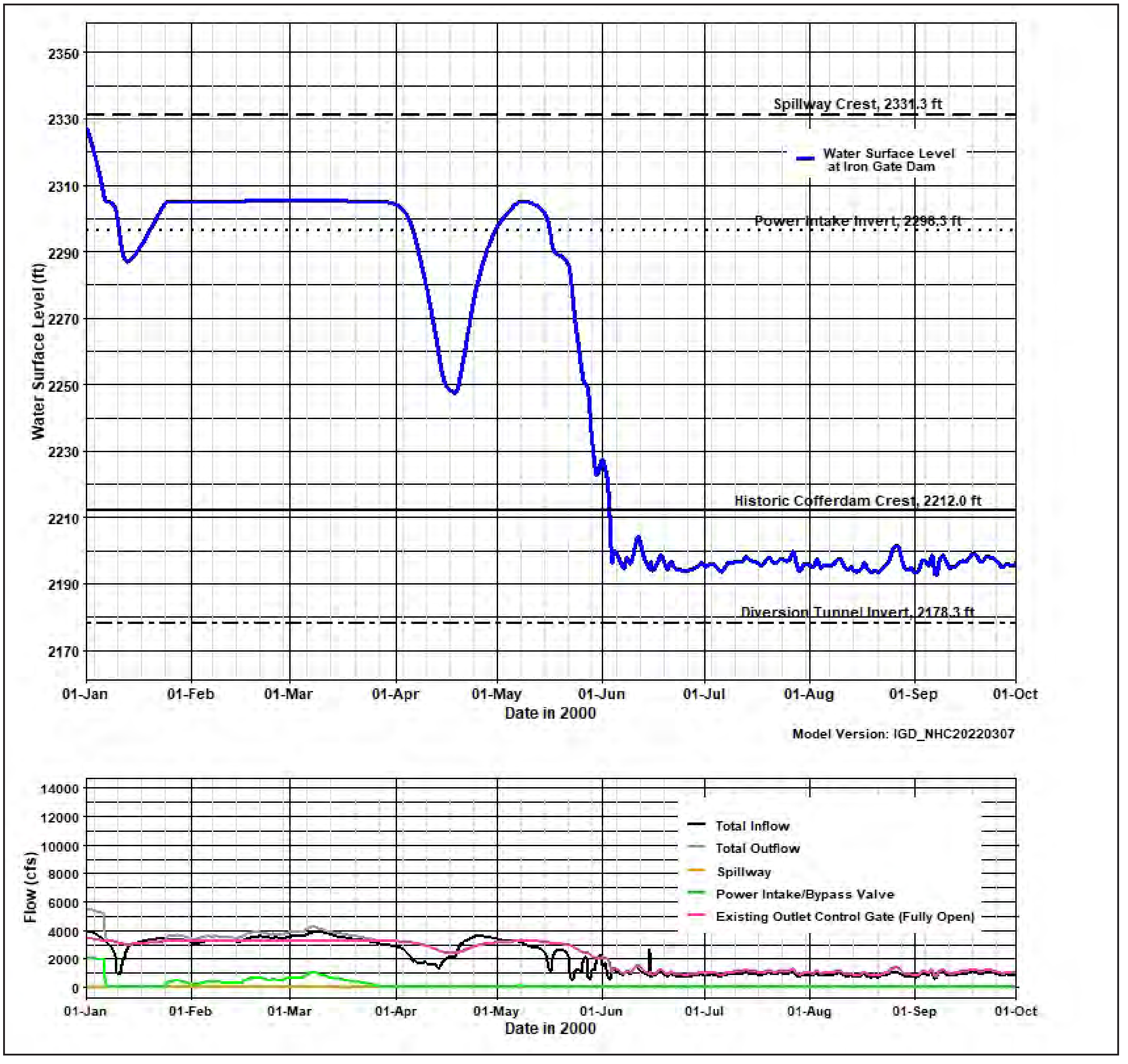

Figure 20 - Iron Gate Dam Facility Simulated Drawdown - Year 2000

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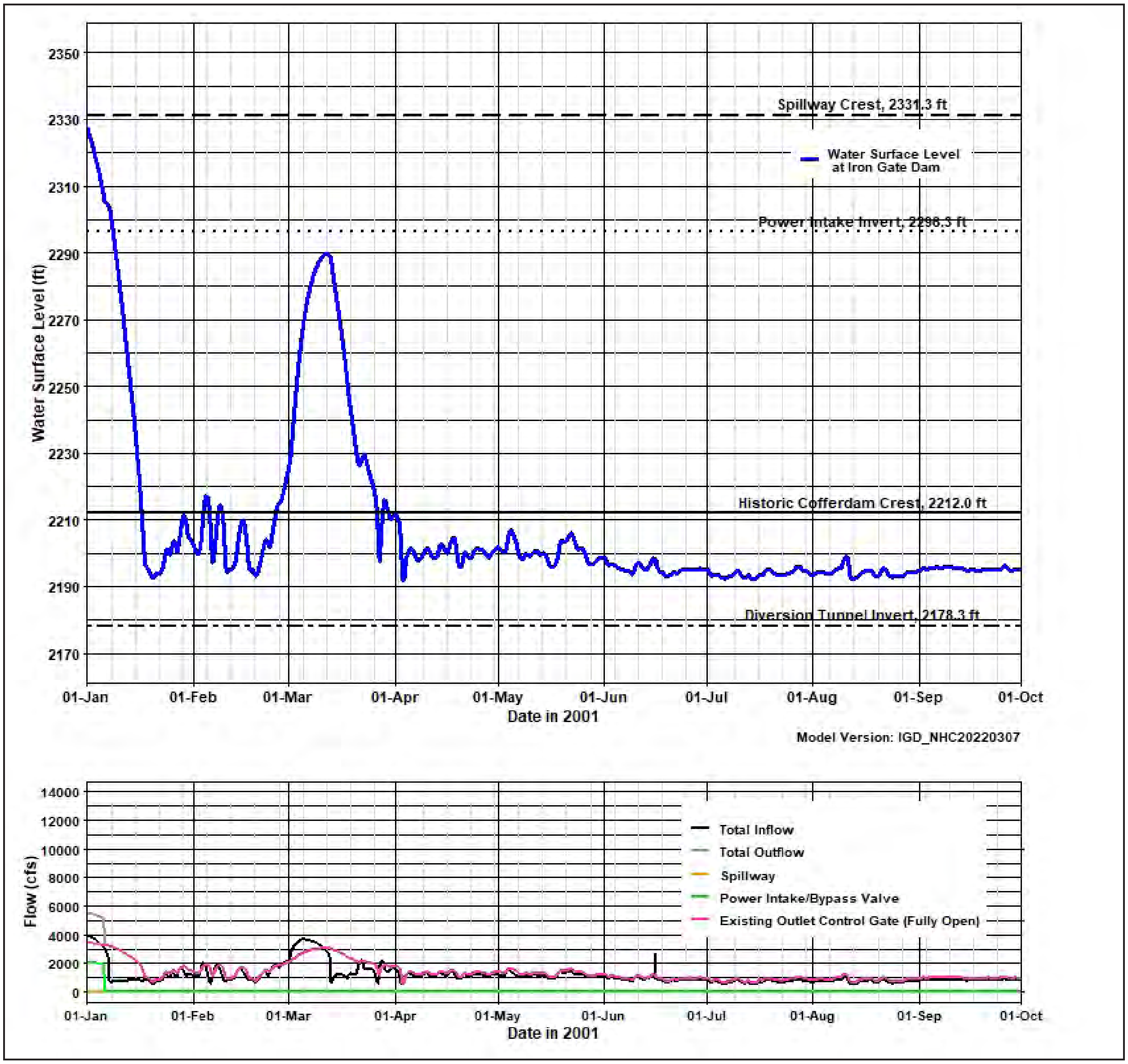

Figure 21 - Iron Gate Dam Facility Simulated Drawdown - Year 2001

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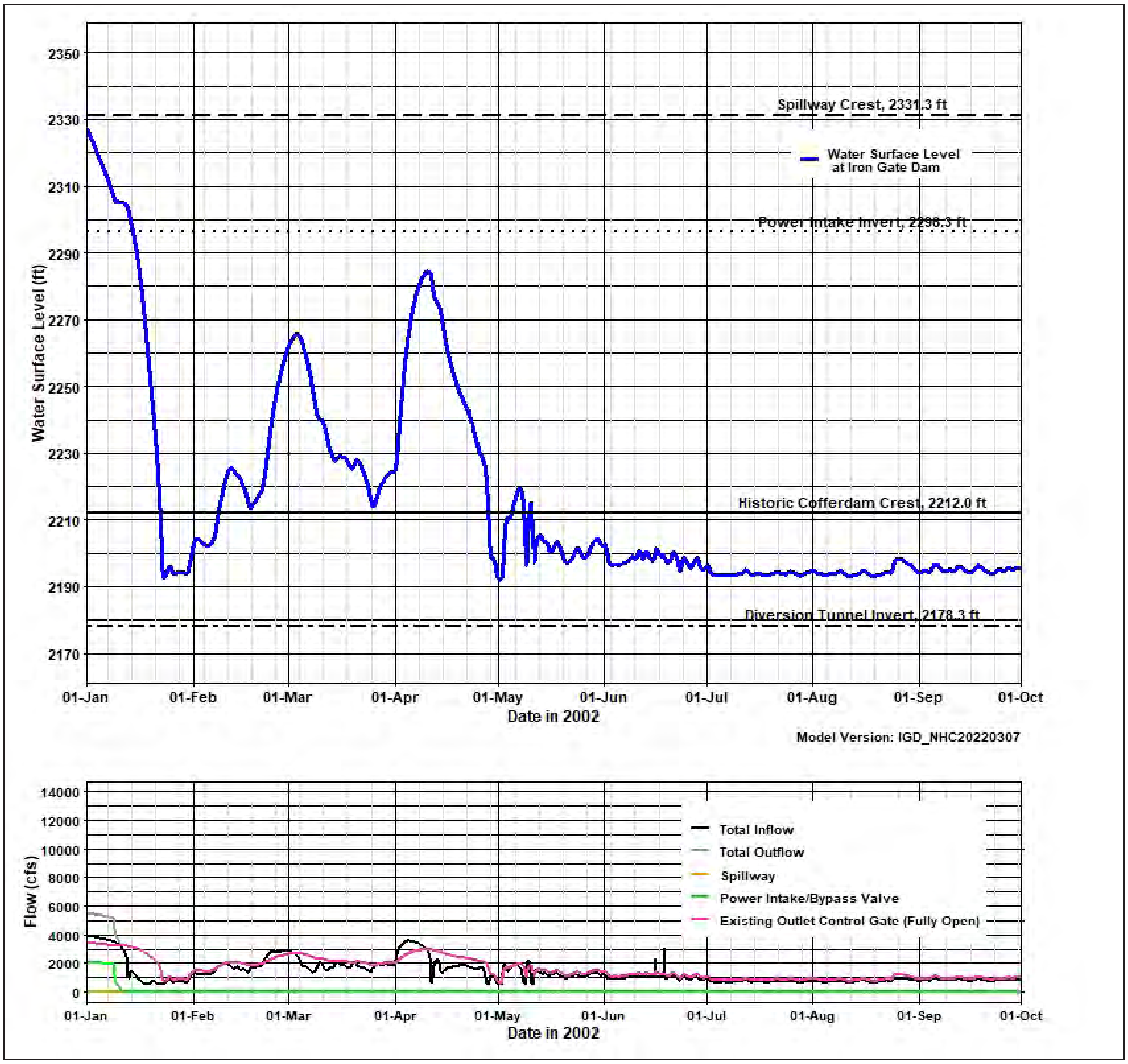

Figure 22 - Iron Gate Dam Facility Simulated Drawdown - Year 2002

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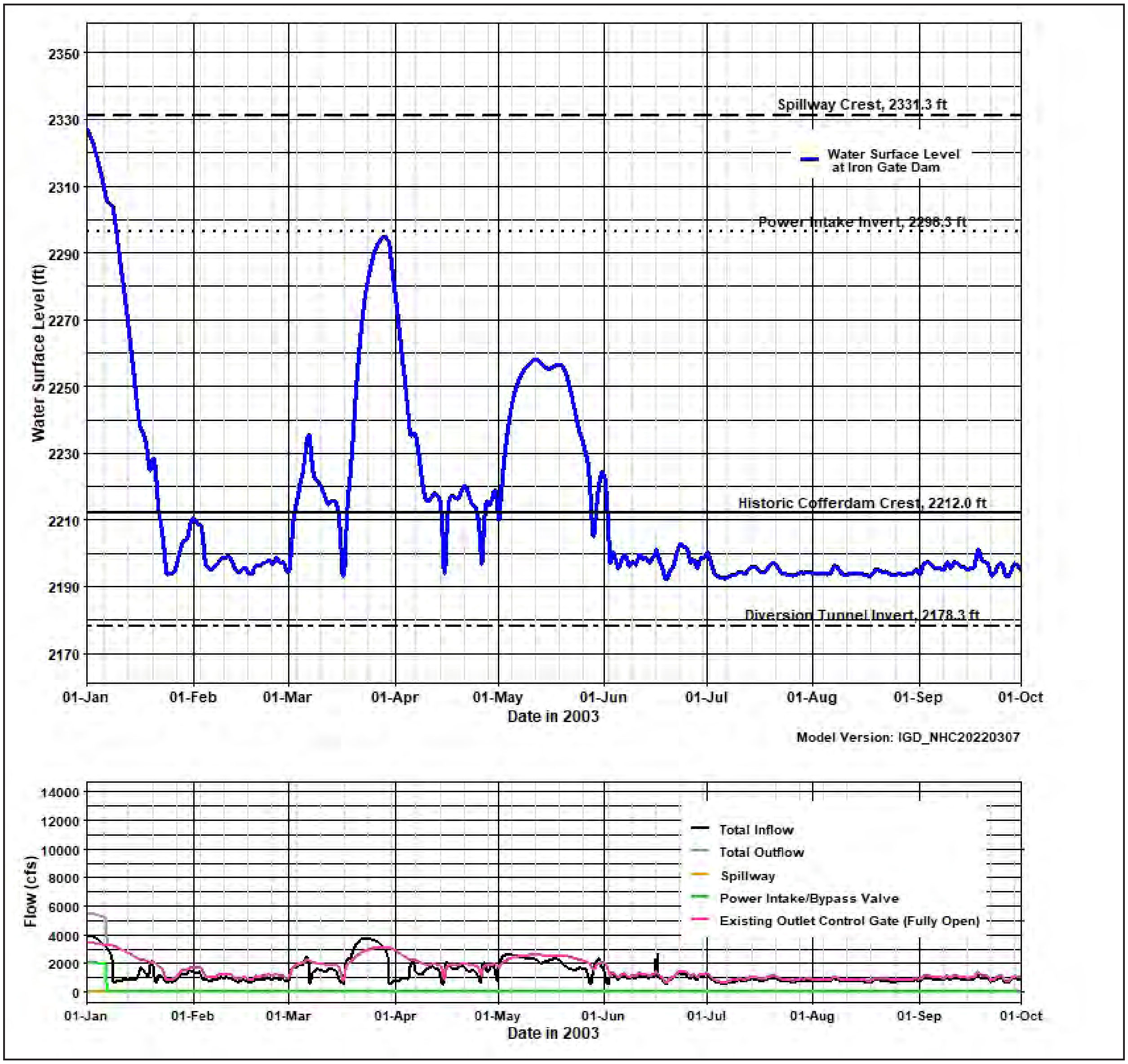

Figure 23 - Iron Gate Dam Facility Simulated Drawdown - Year 2003

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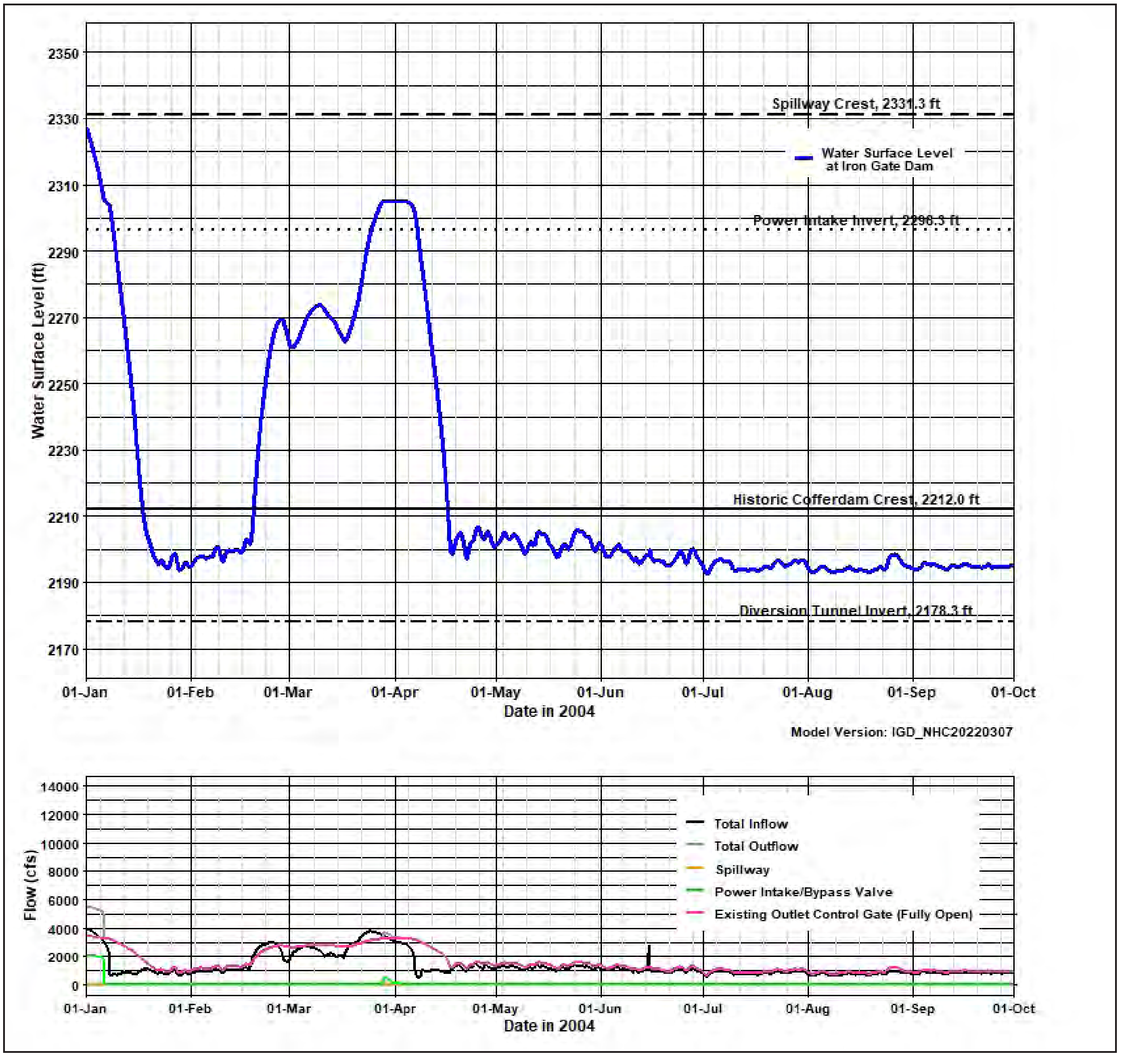

Figure 24 - Iron Gate Dam Facility Simulated Drawdown - Year 2004

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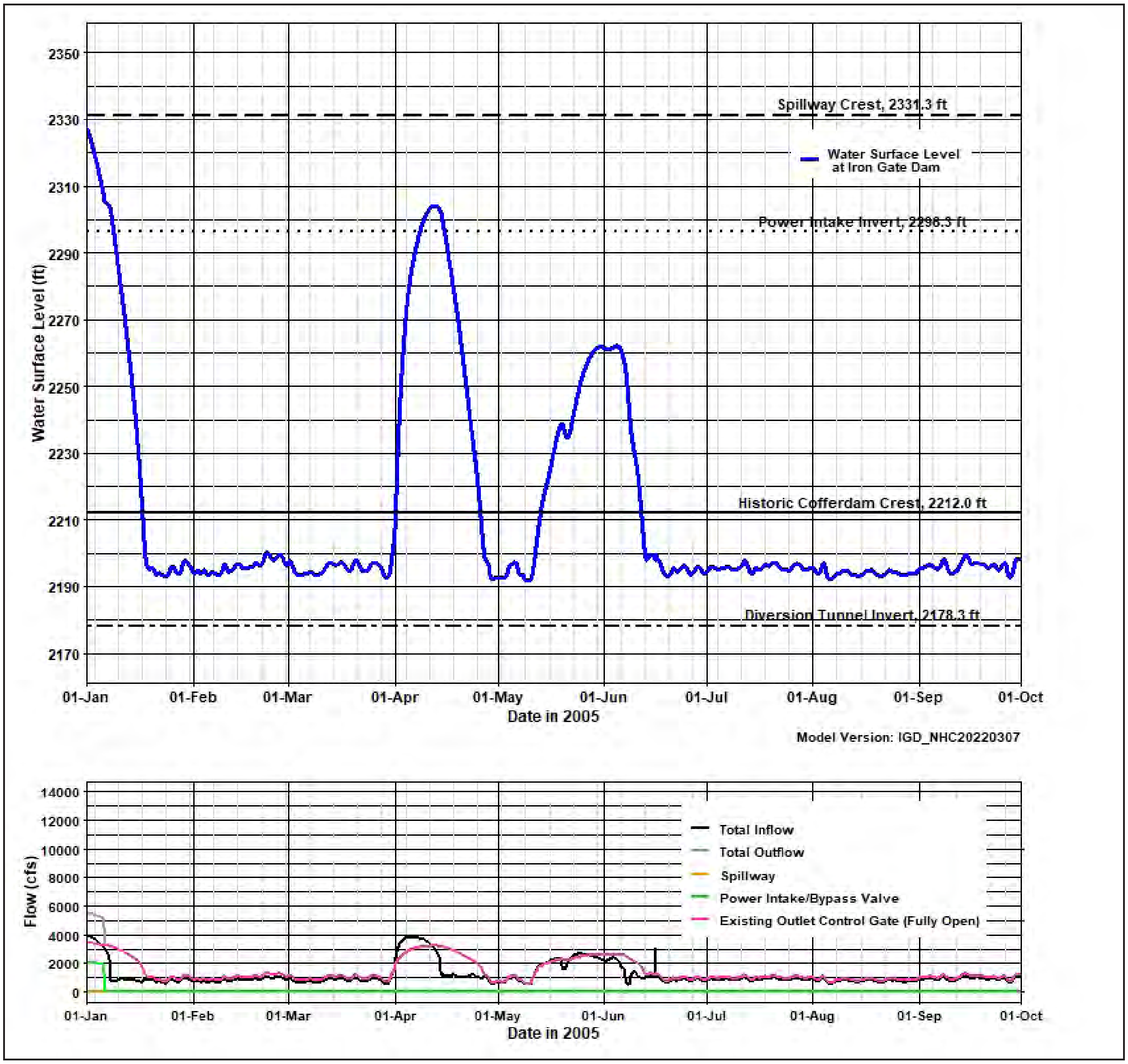

Figure 25 - Iron Gate Dam Facility Simulated Drawdown - Year 2005

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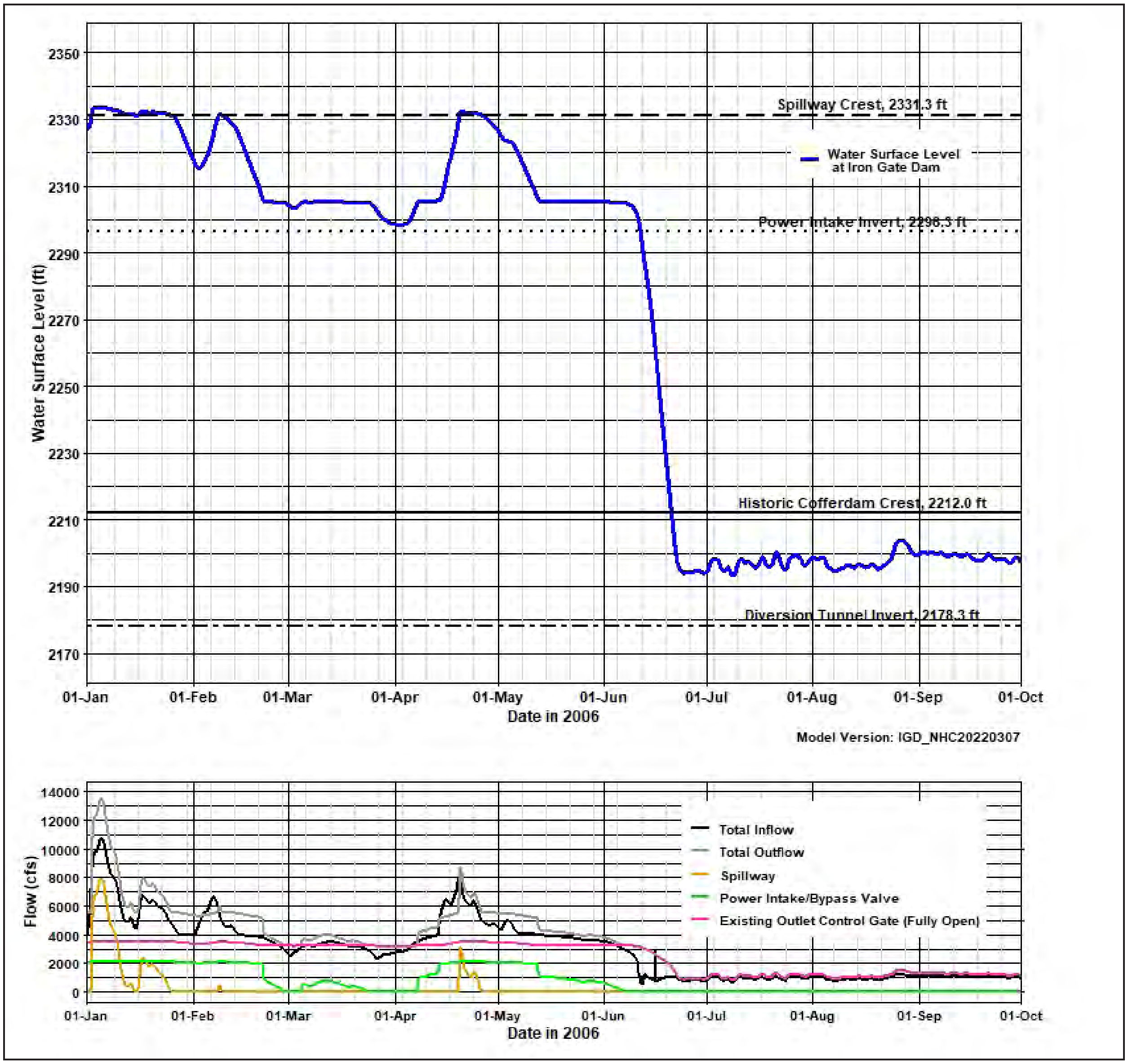

Figure 26 - Iron Gate Dam Facility Simulated Drawdown - Year 2006

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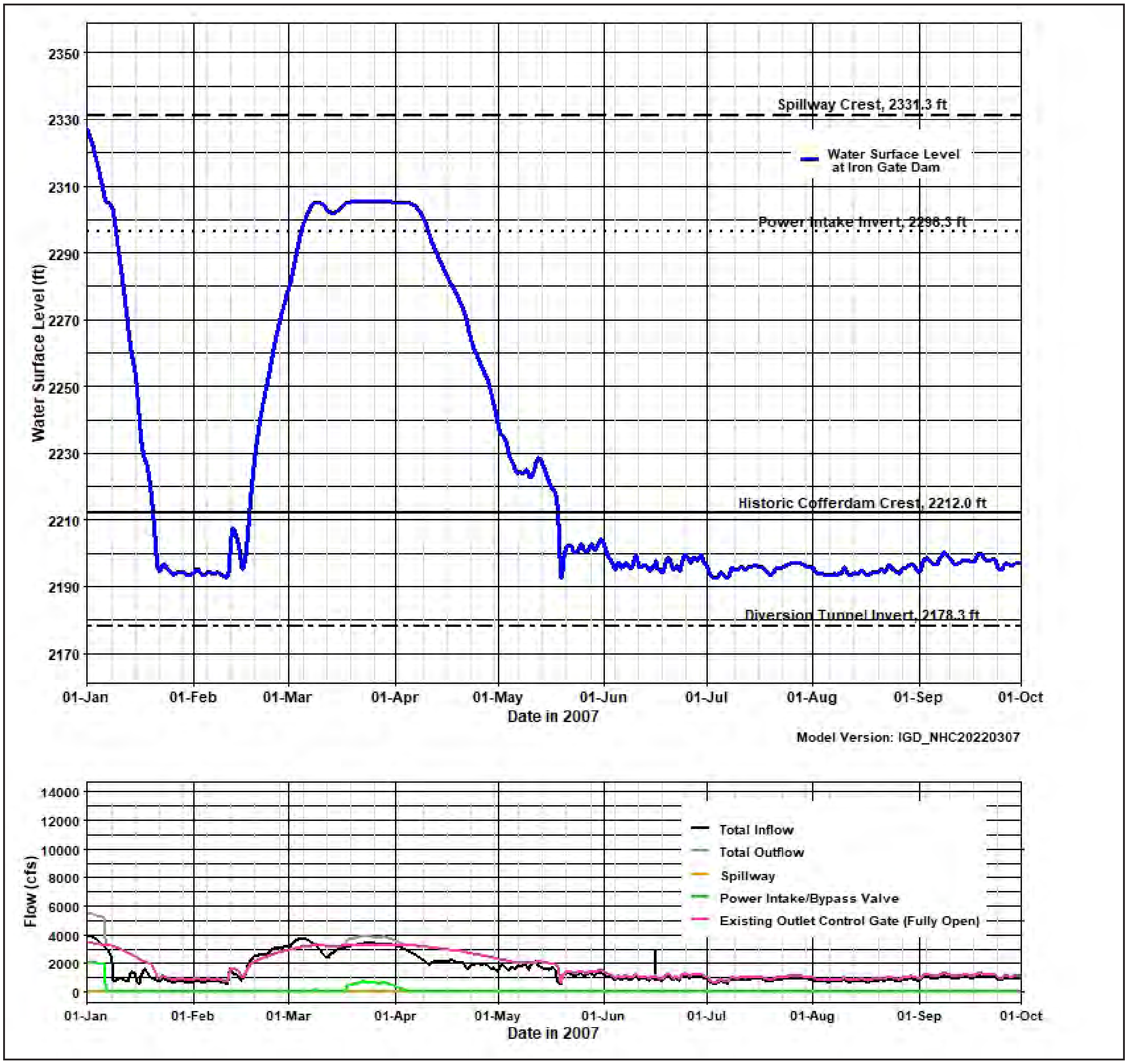

Figure 27 - Iron Gate Dam Facility Simulated Drawdown - Year 2007

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

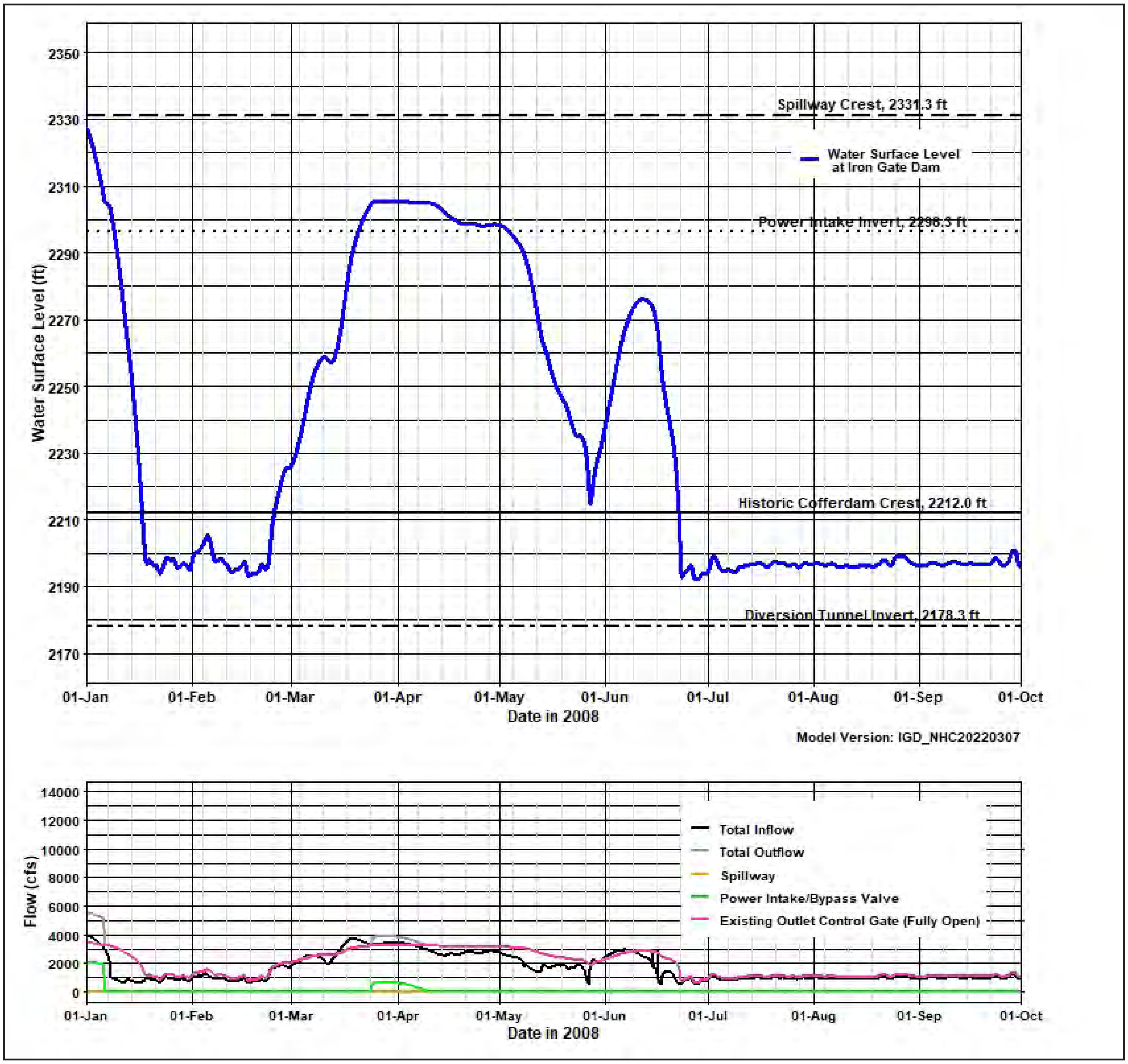

Figure 28 - Iron Gate Dam Facility Simulated Drawdown - Year 2008

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

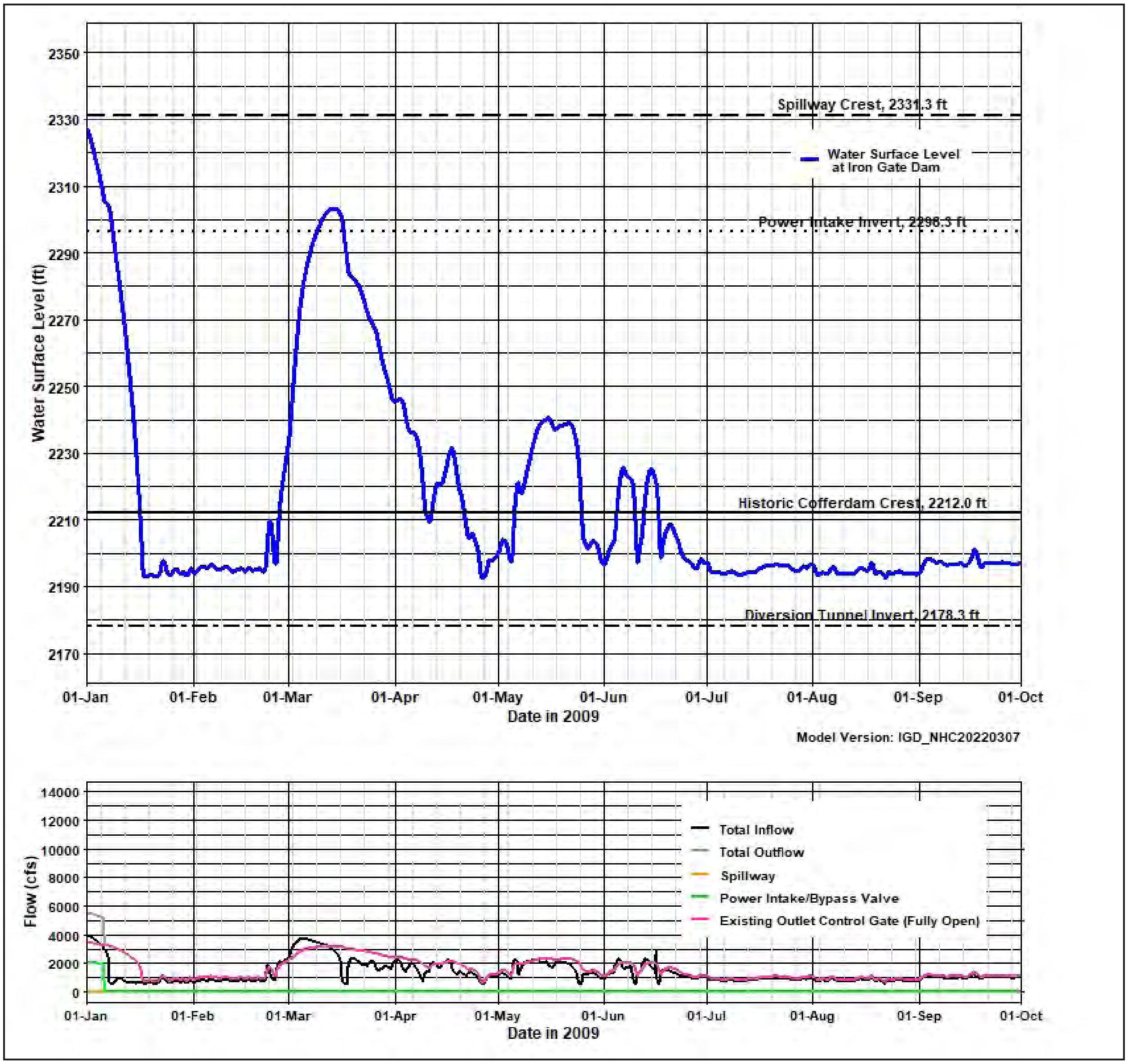

Figure 29 - Iron Gate Dam Facility Simulated Drawdown - Year 2009

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

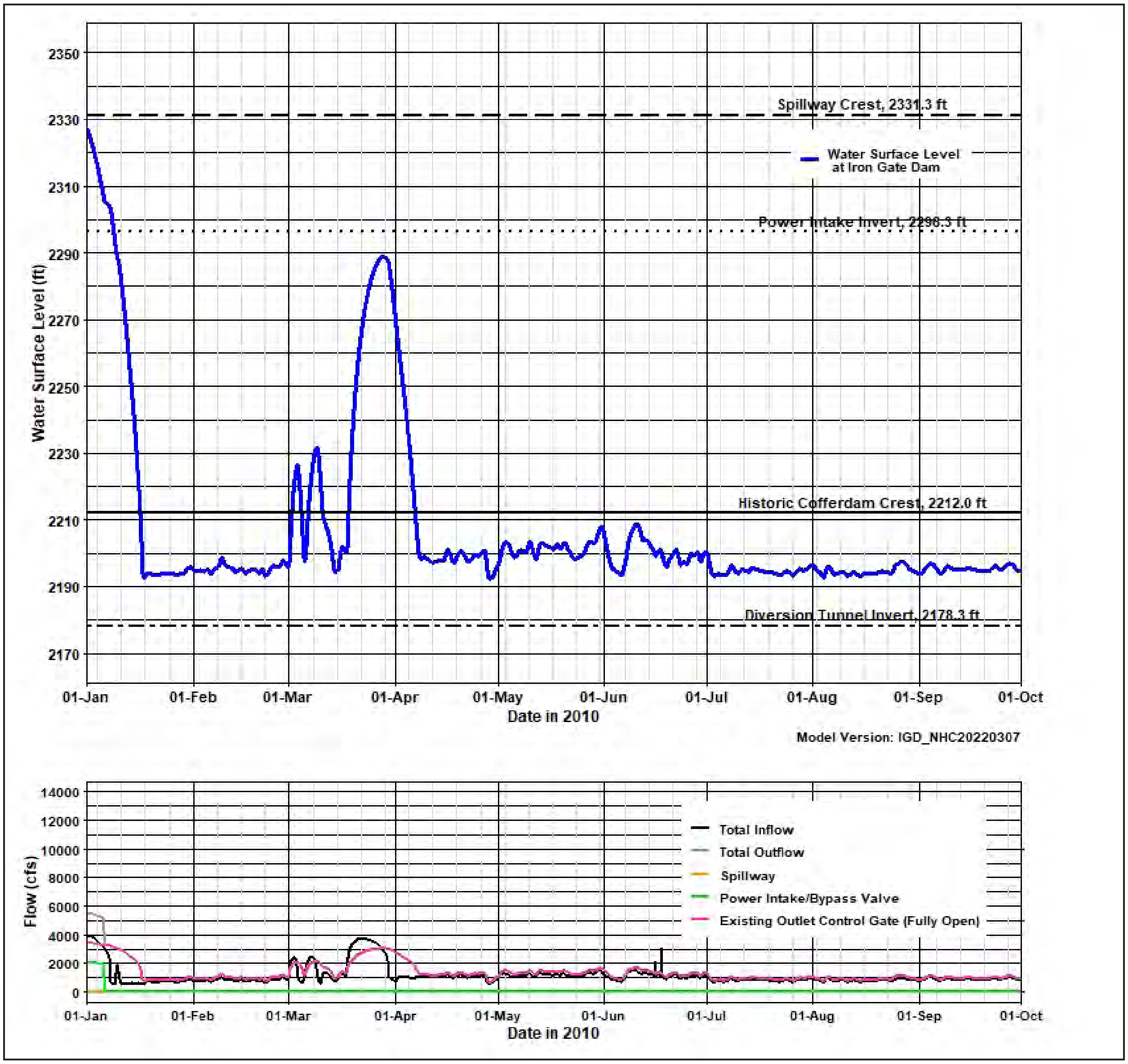

Figure 30 - Iron Gate Dam Facility Simulated Drawdown - Year 2010

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

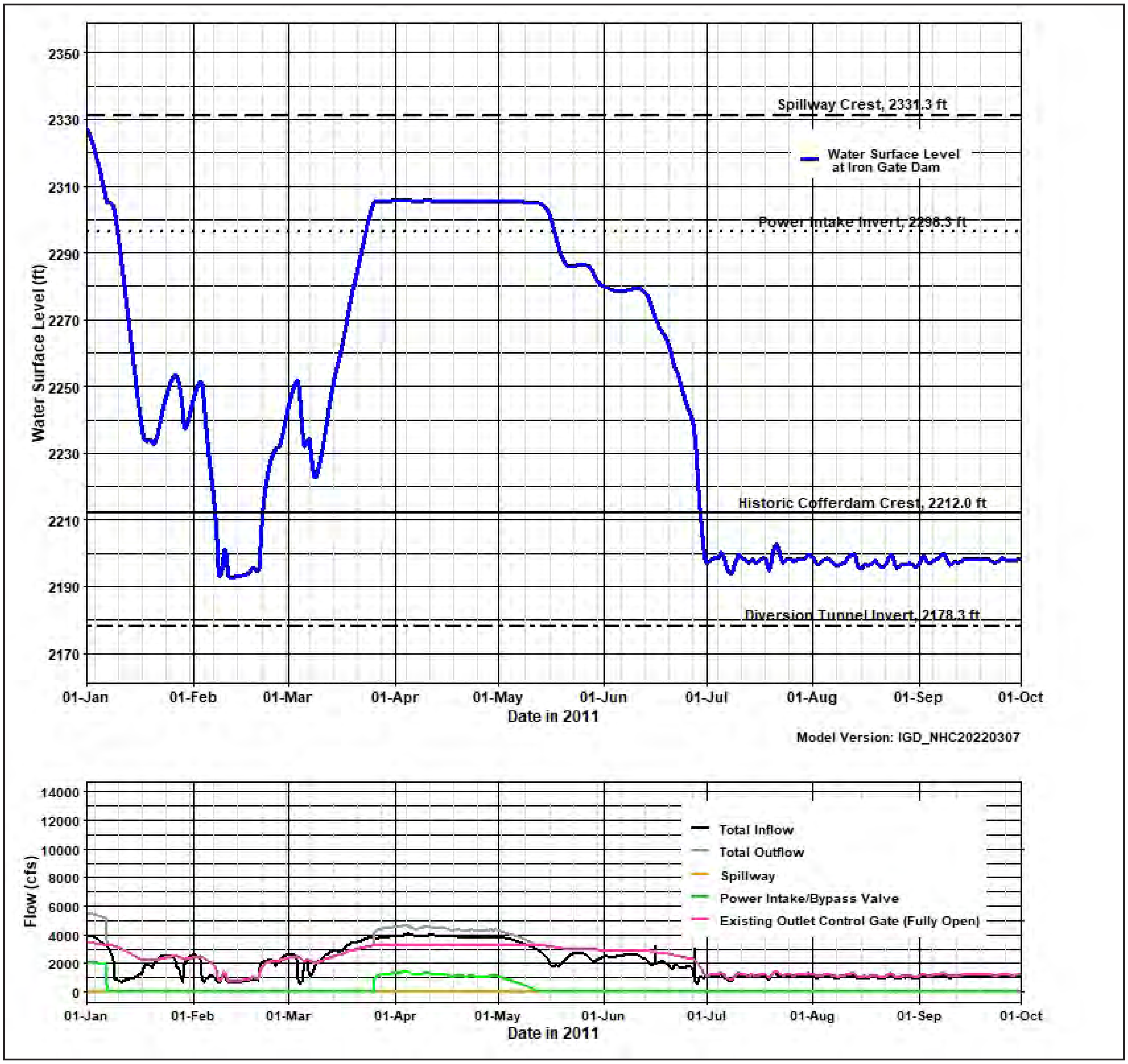

Figure 31 - Iron Gate Dam Facility Simulated Drawdown - Year 2011

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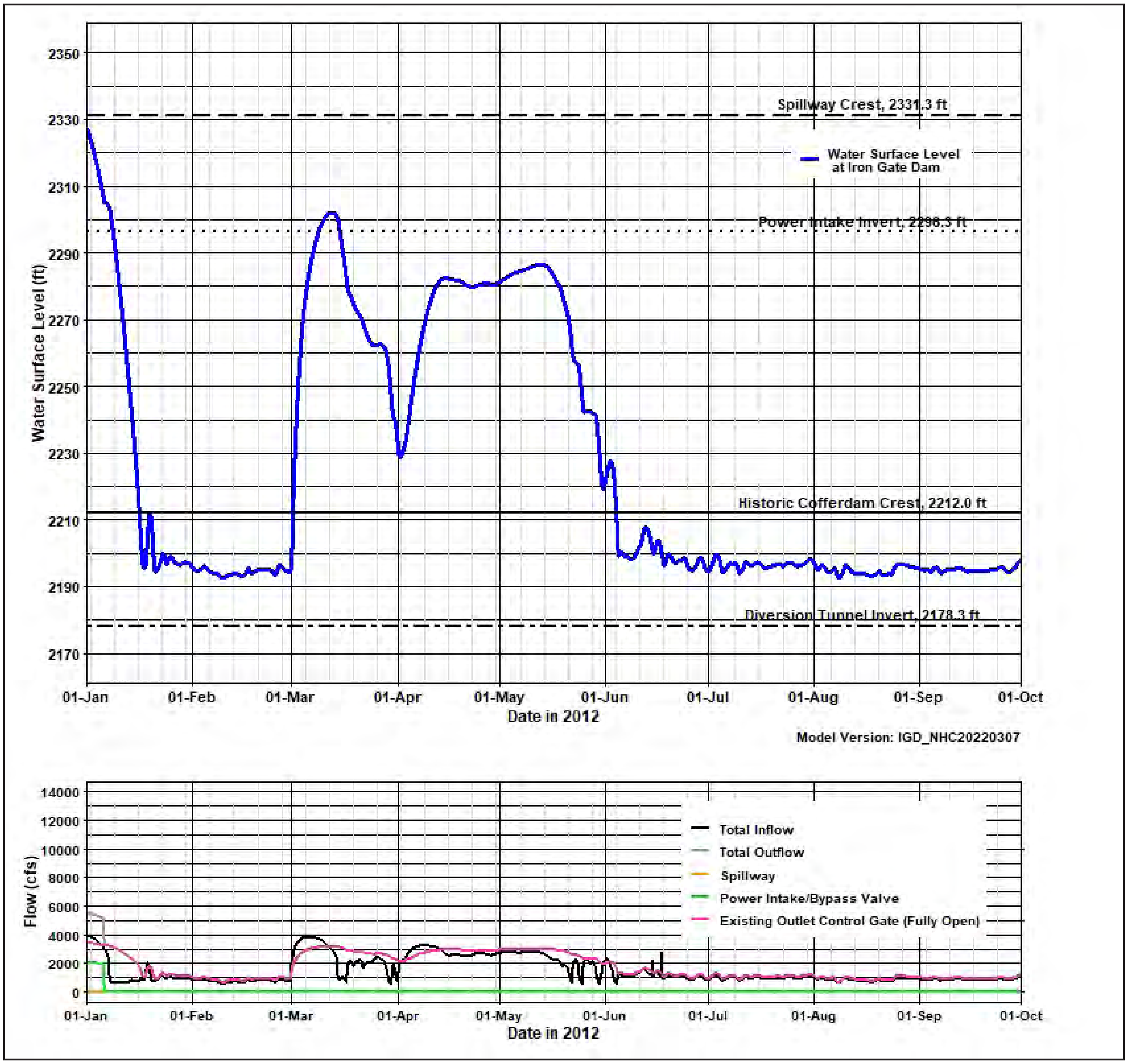

Figure 32 - Iron Gate Dam Facility Simulated Drawdown - Year 2012

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

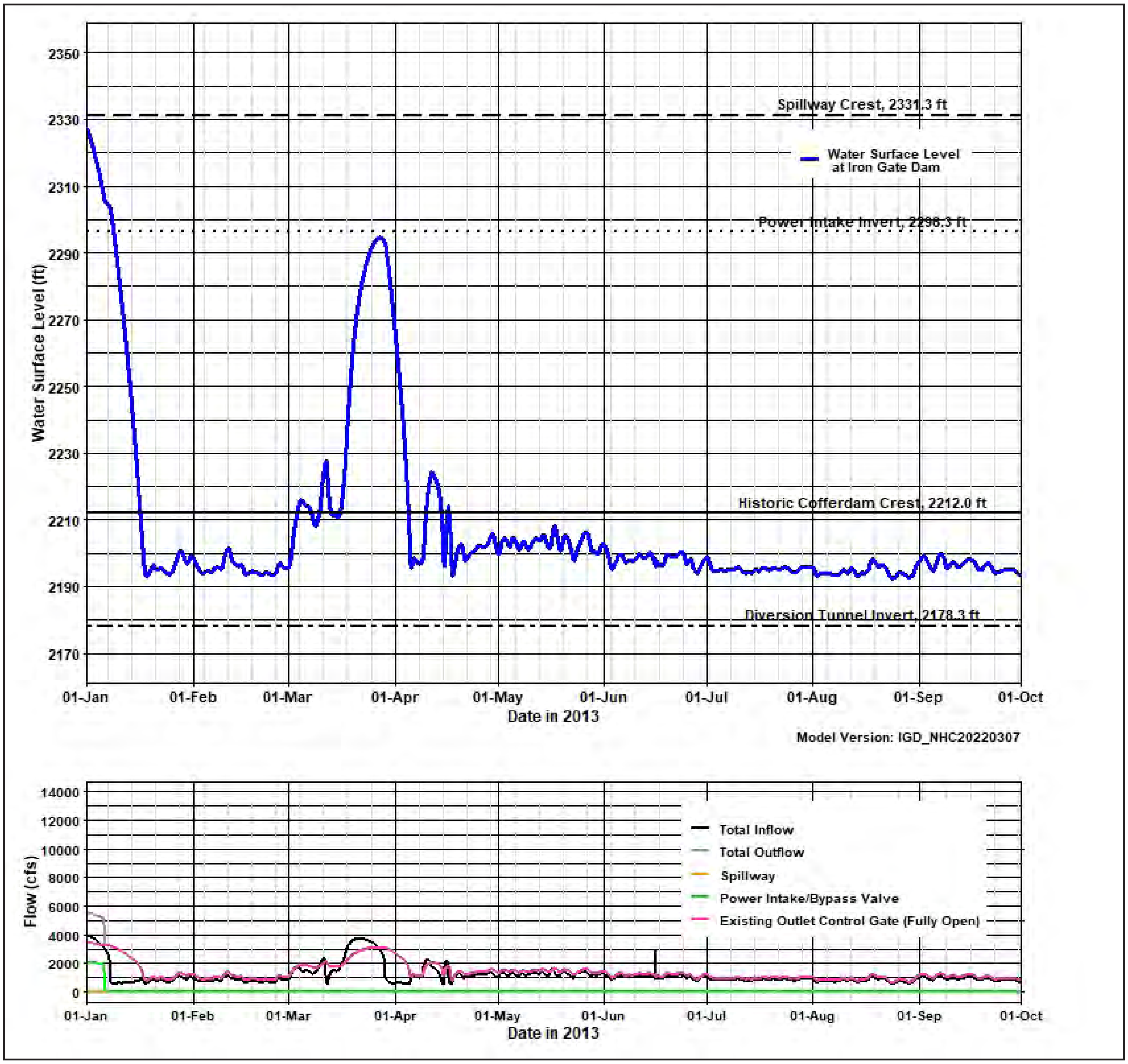

Figure 33 - Iron Gate Dam Facility Simulated Drawdown - Year 2013

April 22, 2022

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

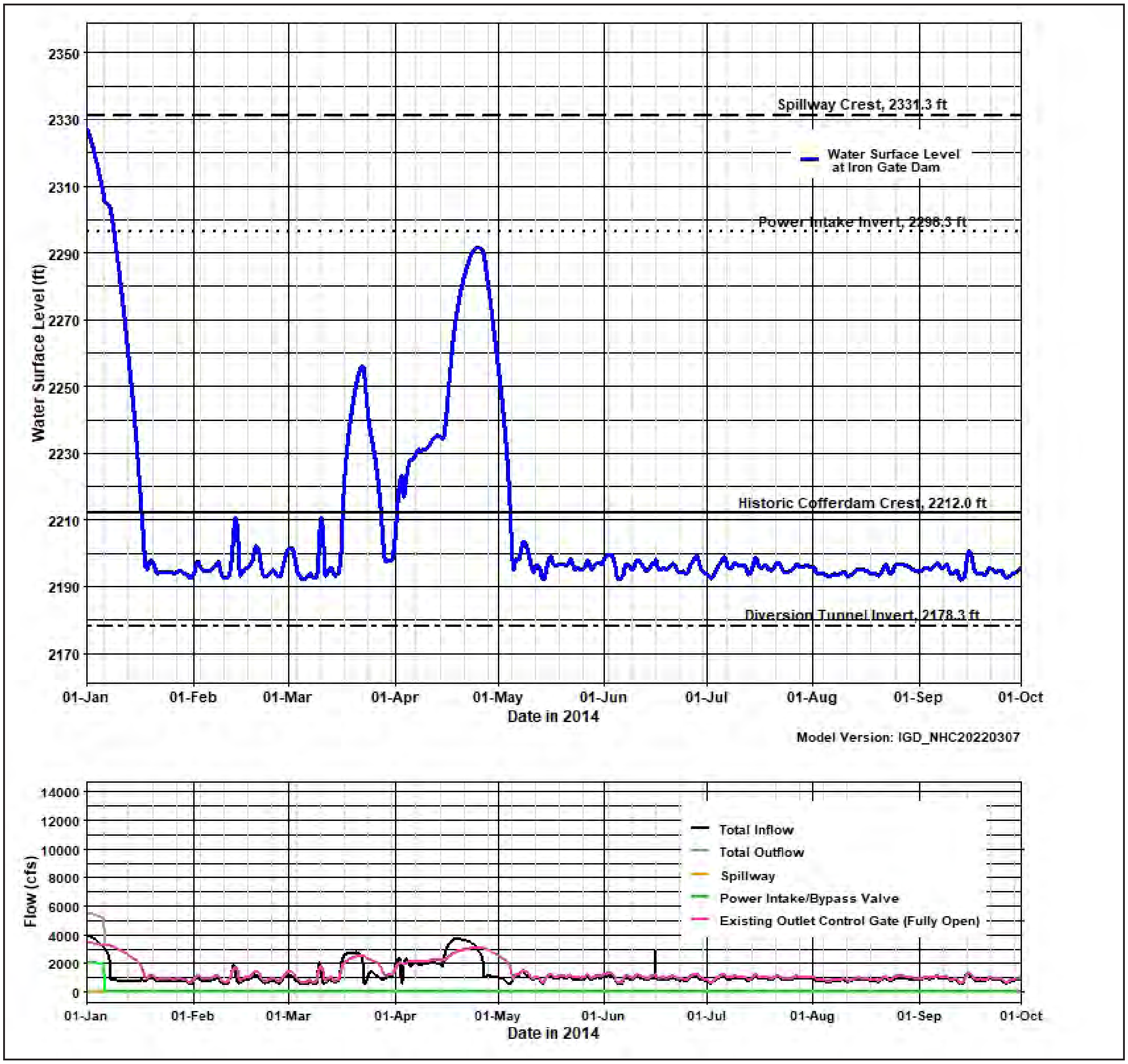

Figure 34 - Iron Gate Dam Facility Simulated Drawdown - Year 2014

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

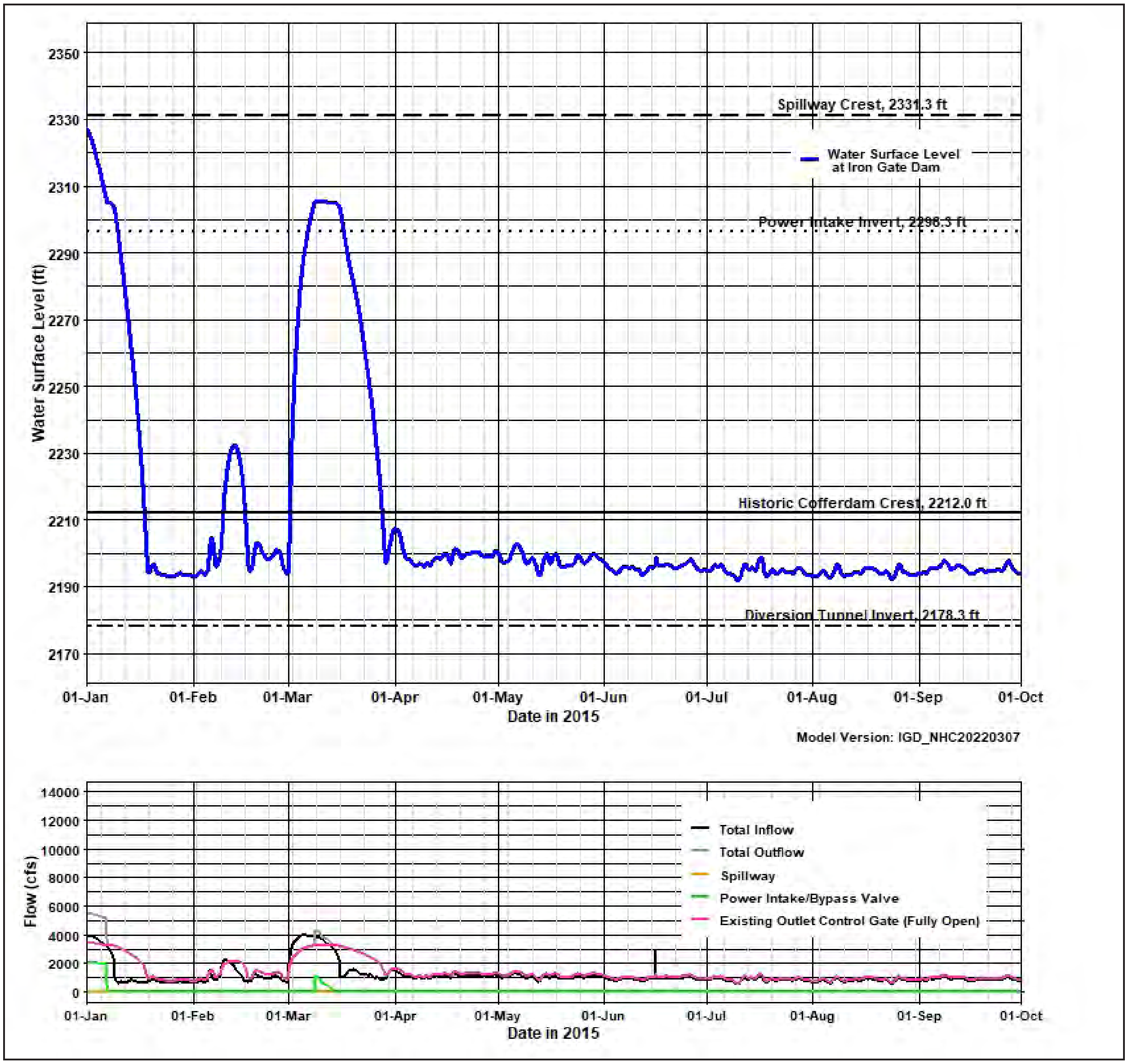

Figure 35 - Iron Gate Dam Facility Simulated Drawdown - Year 2015

April 22, 2022

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IRON GATE DAM FACILITY SIMULATED DRAWDOWN

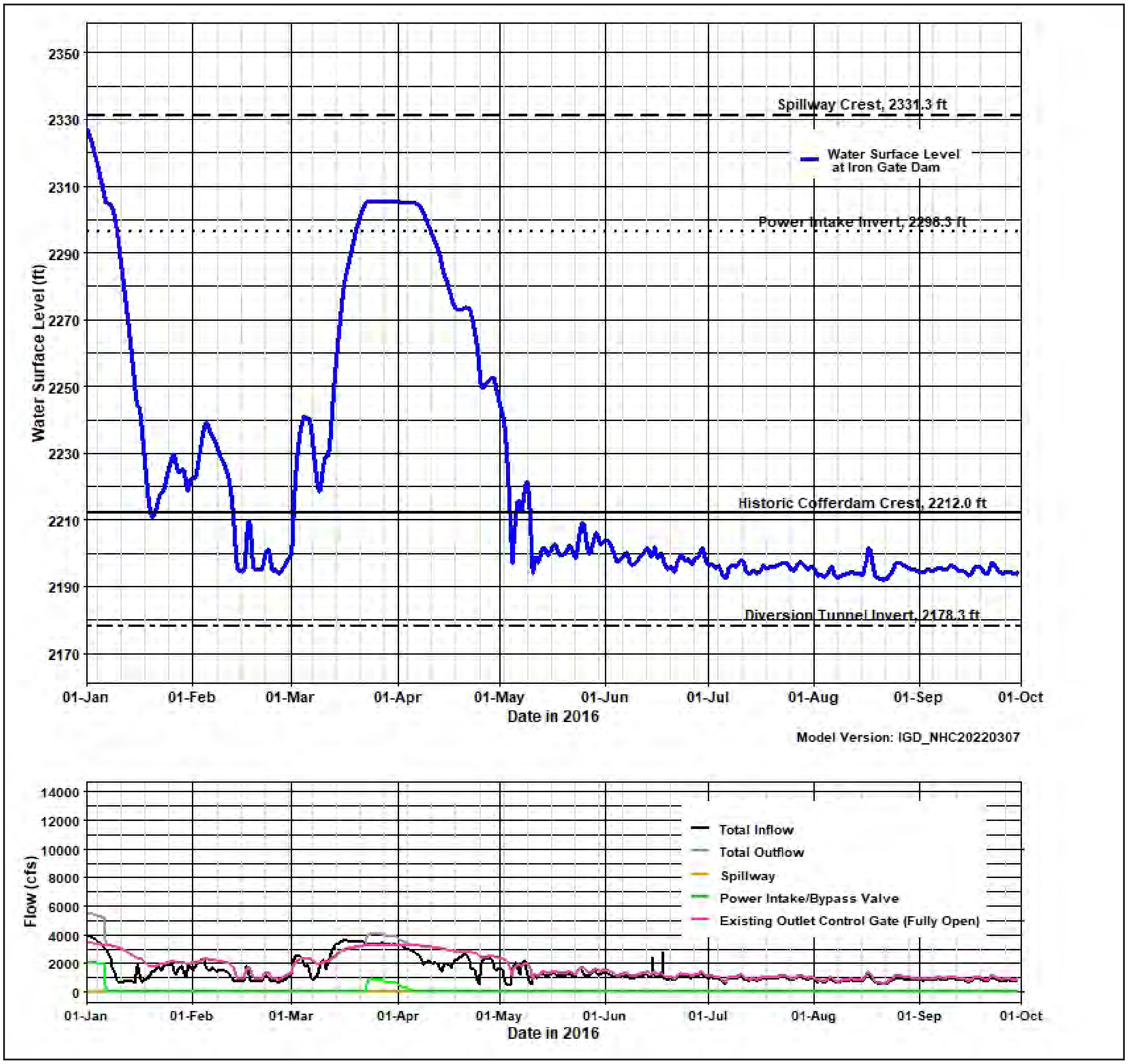

Figure 36 - Iron Gate Dam Facility Simulated Drawdown - Year 2016

April 22, 2022

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Management System Certified by: bsi ISO 9001 ISO 14001 ISO 45001 CERTIFIED

MEMORANDUM

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

1.0 INTRODUCTION

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

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

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

2.0 SURVEY RESULTS

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

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

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

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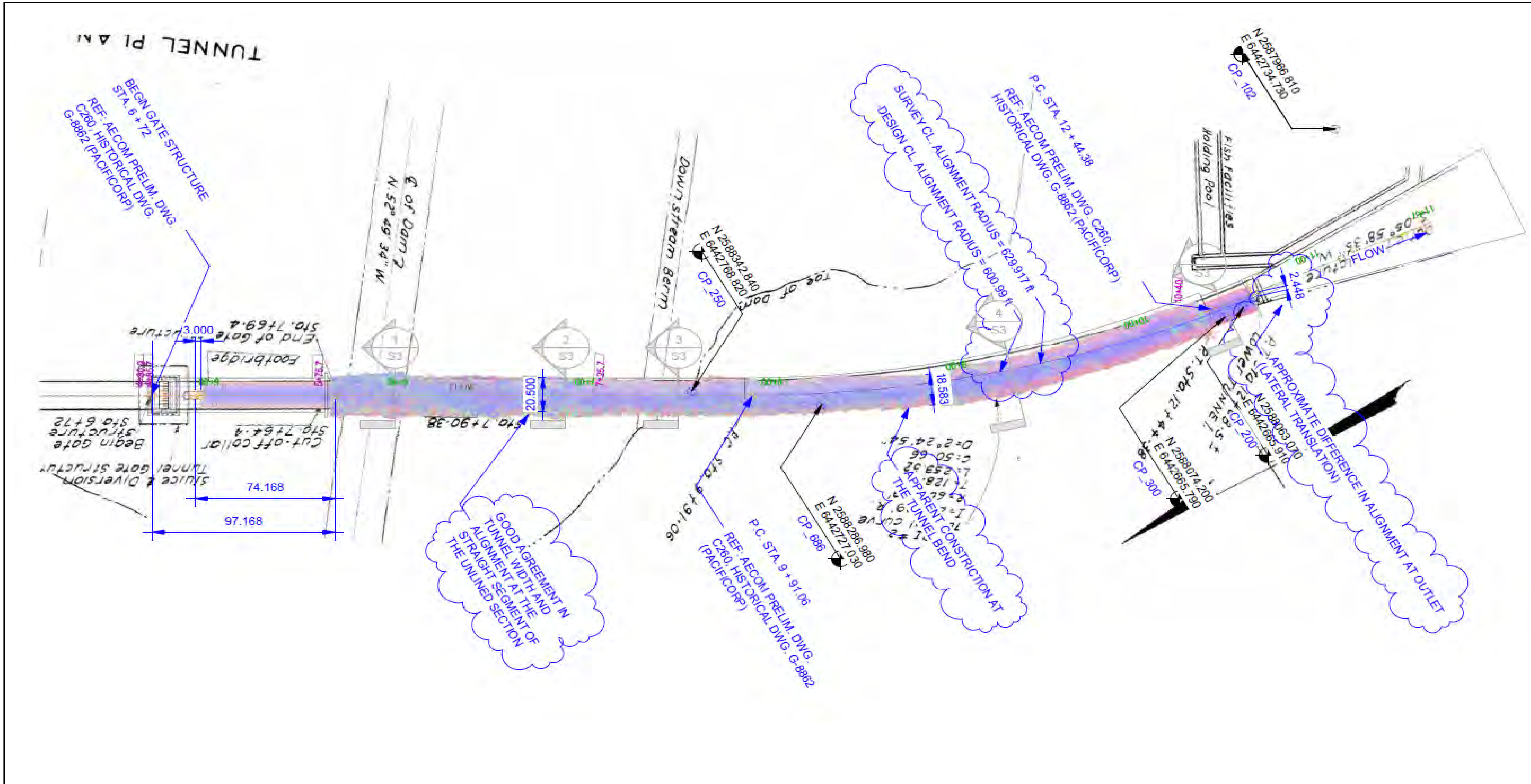
2.1 COMPARISON WITH IDEALIZED GEOMETRY USED IN 100% DESIGN DELIVERABLES

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

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

Key observations included:

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



NOTES:

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

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

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL DOWNSTREAM TUNNEL SURVEY COORDINATION PLAN

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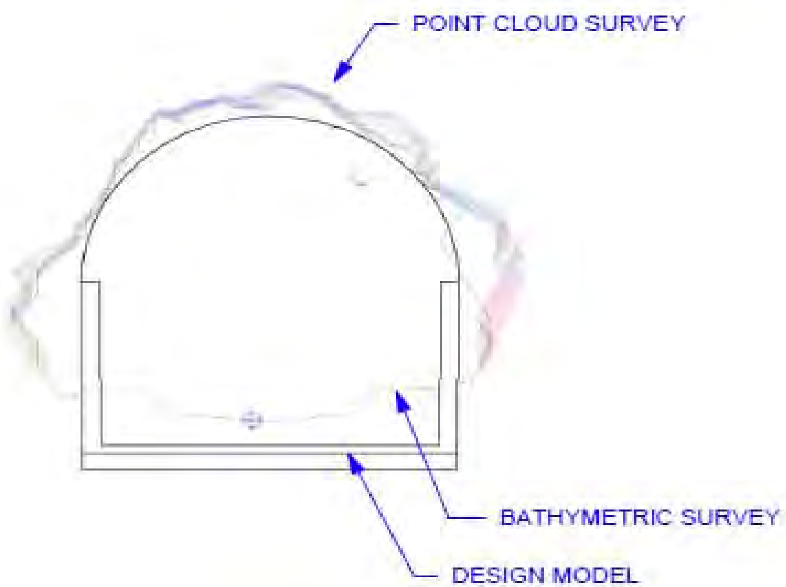
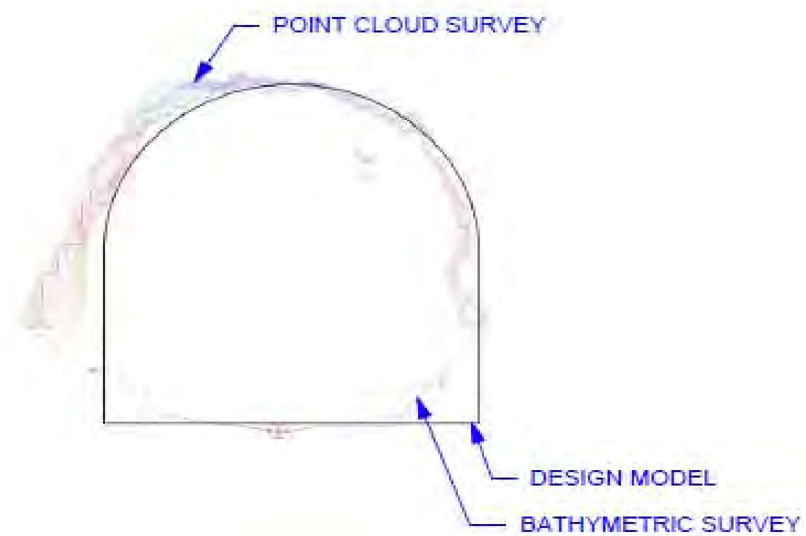
P/A NO. VA103-00640/1 | REF. NO. VA21-00482

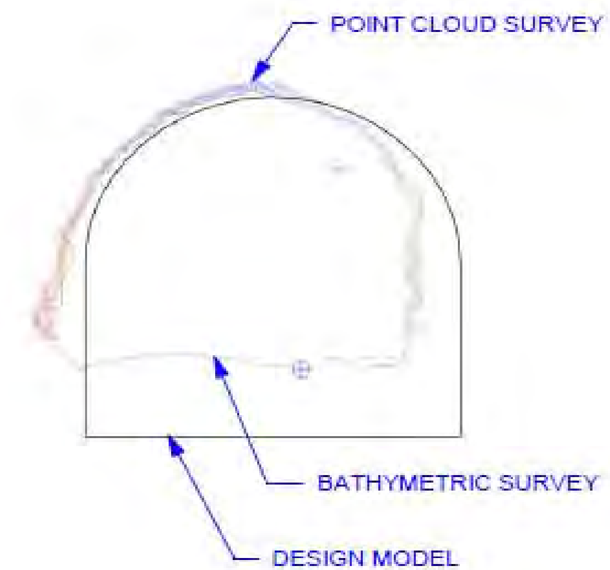
FIGURE 2.1 REV 0



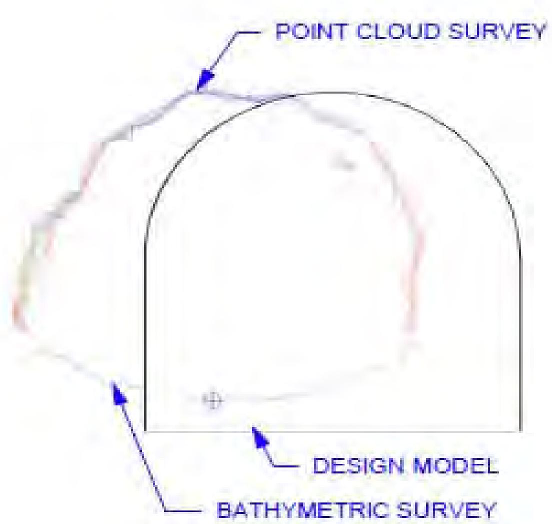
SECTION 1



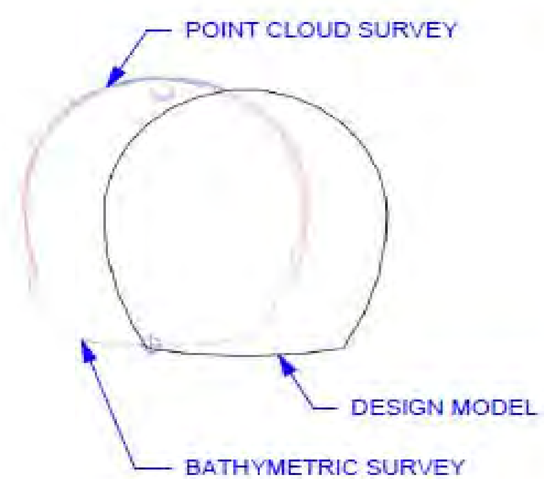
SECTION 2



SECTION 3



SECTION 4



SECTION 5

NOTES:

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

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Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
DOWNSTREAM TUNNEL SURVEY COORDINATION SECTIONS

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P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 2.2

REV 0

3.0 UPDATED MODEL GEOMETRY

3.1 SURVEY DATA MANIPULATION AND ROUGHNESS PARAMETERS

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

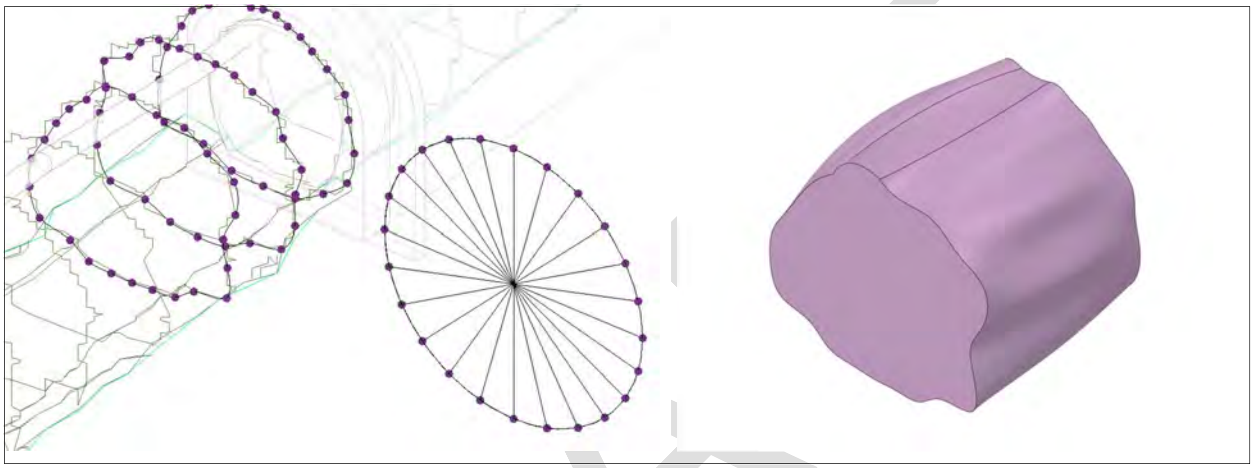

Figure 3.1 Construction of Model Geometry – Partial Segment of Tunnel

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

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

3.2 INCORPORATION OF LINER GEOMETRY

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

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

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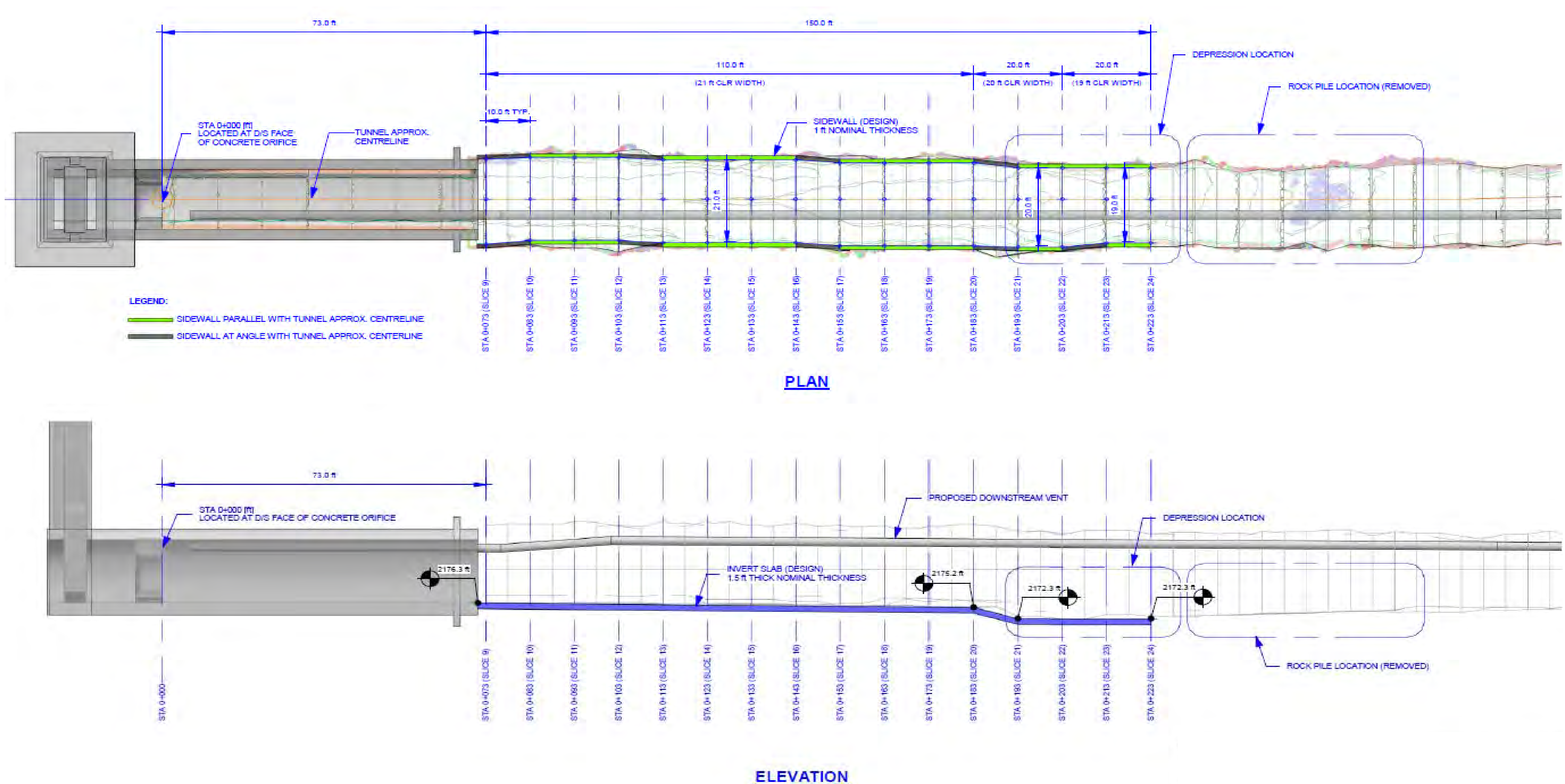
2. A "best-fit" concrete liner similar in concept to the 100% design, and reviewed by the Contractor for constructability, was incorporated from the end of the existing reinforced concrete liner to end of the low-point described above. This is approximately 150 ft in total length of concrete liner and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

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



NOTES:

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

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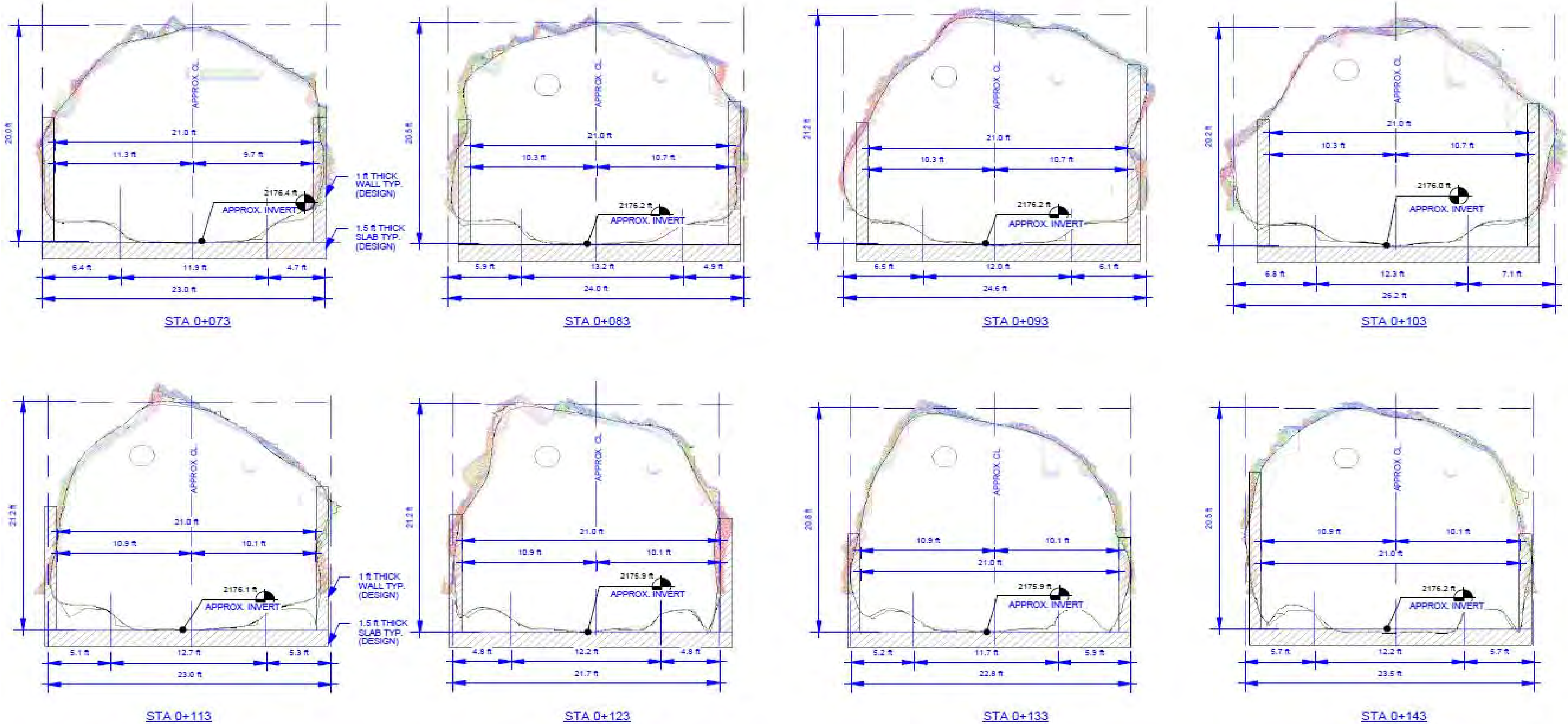
Klamath River Renewal Project

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

Knight Piésold CONSULTING

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

FIGURE 3.2 REV 0



NOTES:

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

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

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

FIGURE 3.3 REV 0

NOTES:

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

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Klamath River Renewal Project

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

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VA103-00640/1	VA21-00482

FIGURE 3.4 REV 0

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4.0 RESULTS AND OBSERVATIONS

4.1 VEOCITY REDUCTION AND ENERGY DISSIPATION

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

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

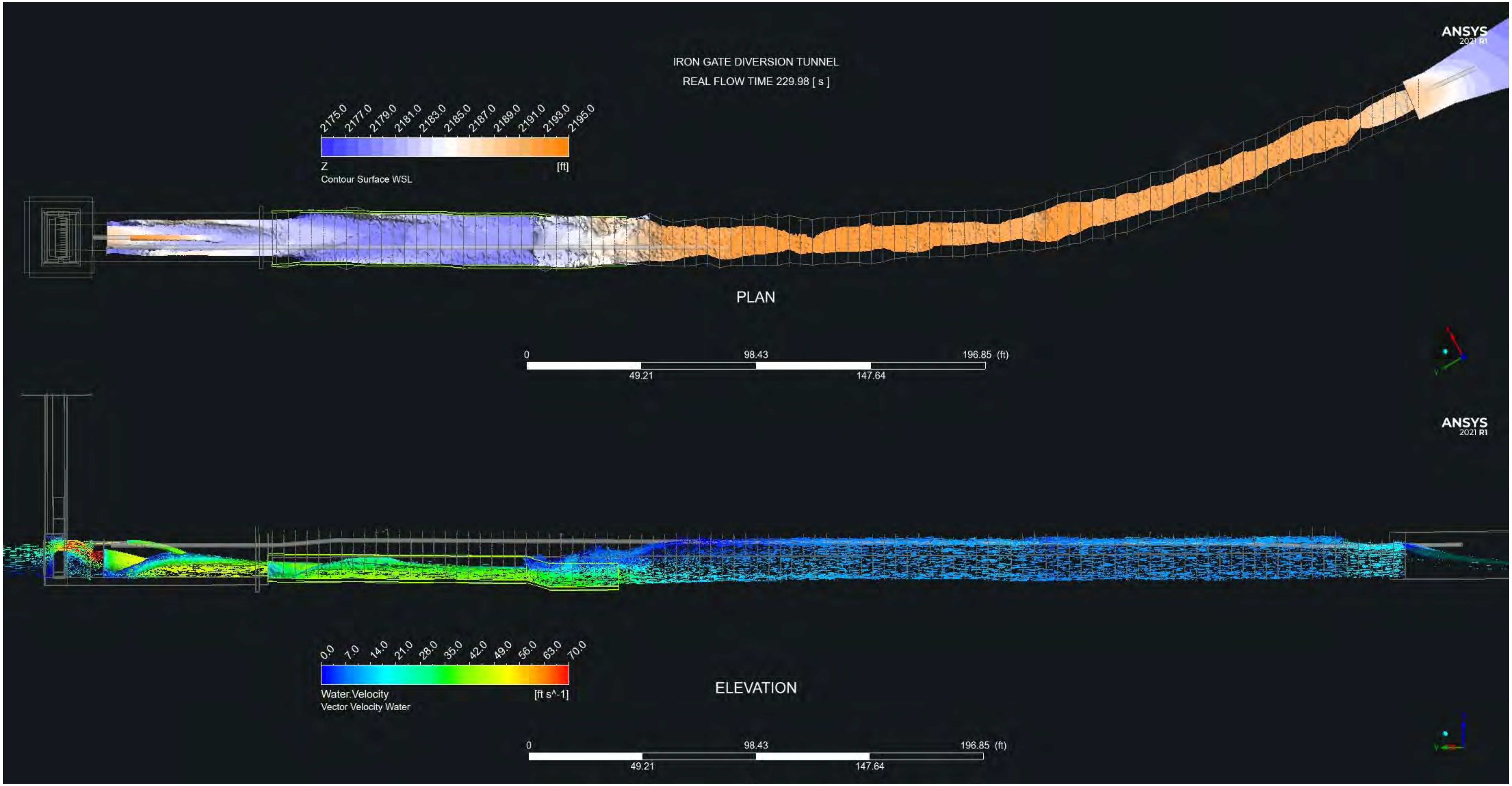
The following table summarizes the water velocities observed in the tunnel when the reservoir is discharging through the tunnel at reservoir WSL El. 2331.3.

Table 4.1 Water Velocity Range by Tunnel Region

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

Other observations demonstrated in Figures 4.1 and 4.2 include:

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



NOTES:

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

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

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

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

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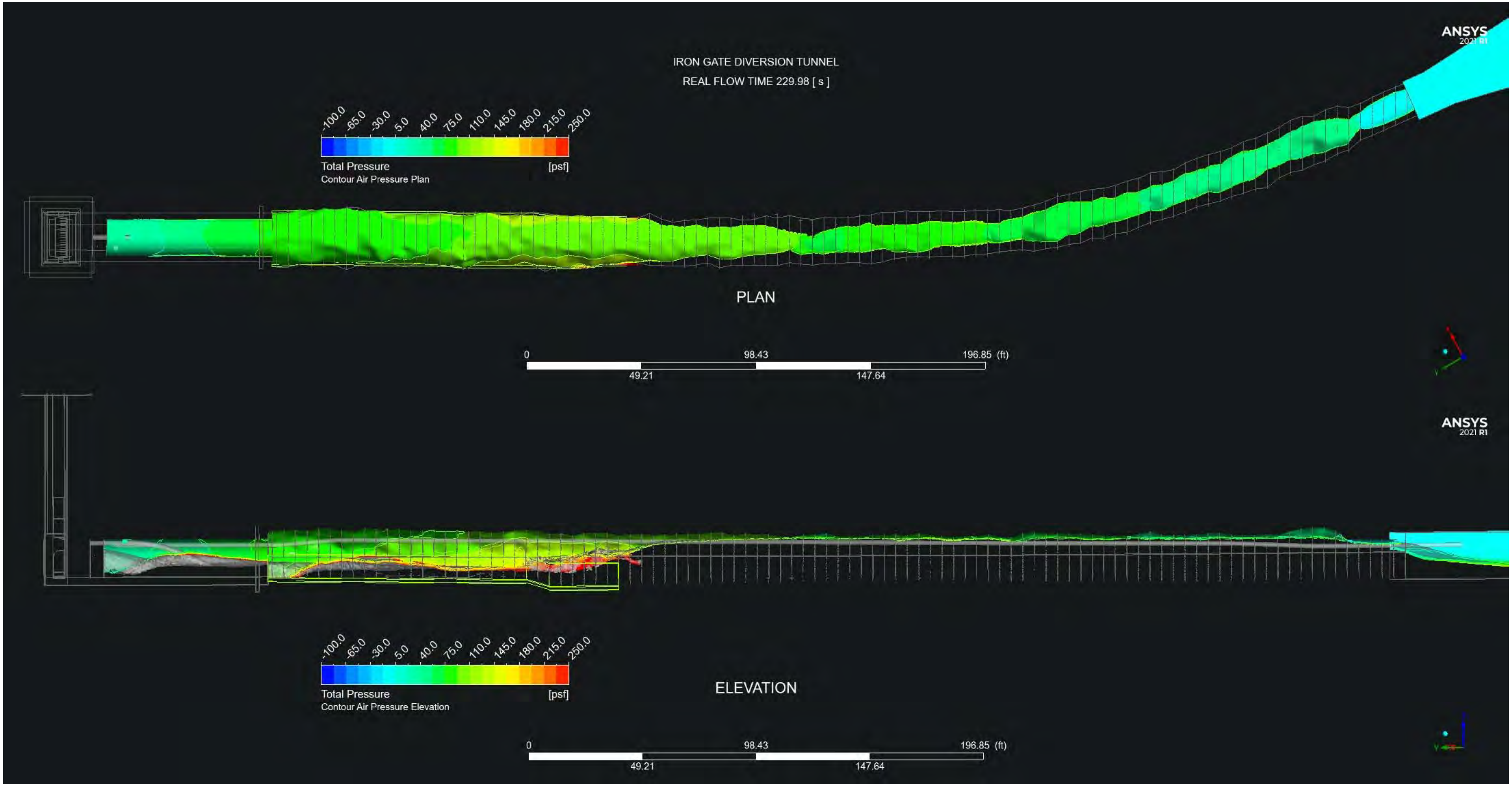
P/A NO. VA103-00640/1 | REF. NO. VA21-00482

FIGURE 4.1 REV 0



NOTES:

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

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

Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

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

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P/A NO. VA103-00640/1

REF. NO. VA21-00482

FIGURE 4.2

REV 0

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4.2 UPDATED RATING CURVE

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

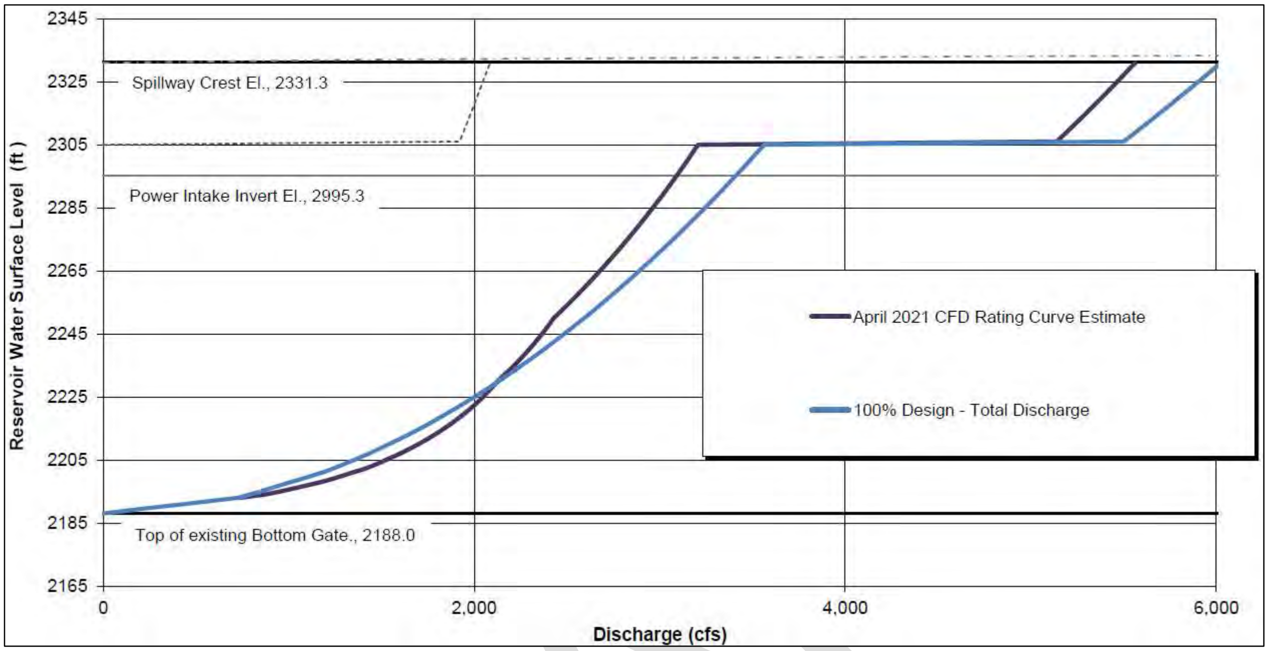

Figure 4.3 Updated Rating Curve

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

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

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

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TABLE 4.2

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

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

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

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

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

NOTES:

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

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

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

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

5.0 CONCLUSION AND RECOMMENDATIONS

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

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

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

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

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

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

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

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

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

Yours truly,
Knight Piésold

Prepared: Carlo Capucao

Prepared: Katrina Wechselberger

Reviewed: Norm Bishop

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

MEMORANDUM

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

1.0 INTRODUCTION

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

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

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

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

2.0 HYDRAULIC INPUT PARAMETERS

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

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

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

3.0 TUNNEL ARRANGEMENT & GEOMETRY CONSTRUCTION

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

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

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

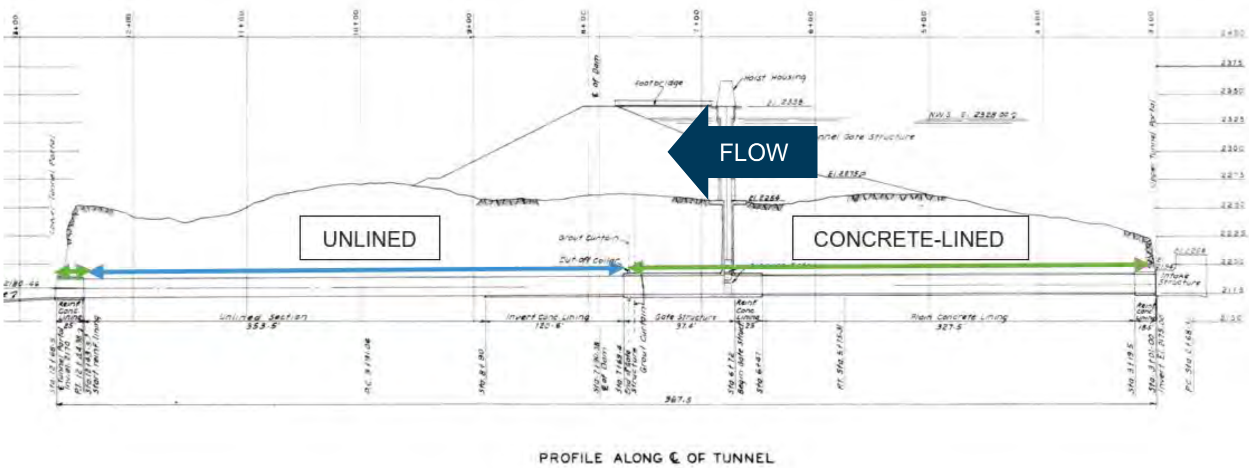


Figure 3.1 Historical Tunnel Profile

3.1 SURVEYED GEOMETRY CONSTRUCTION

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

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

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

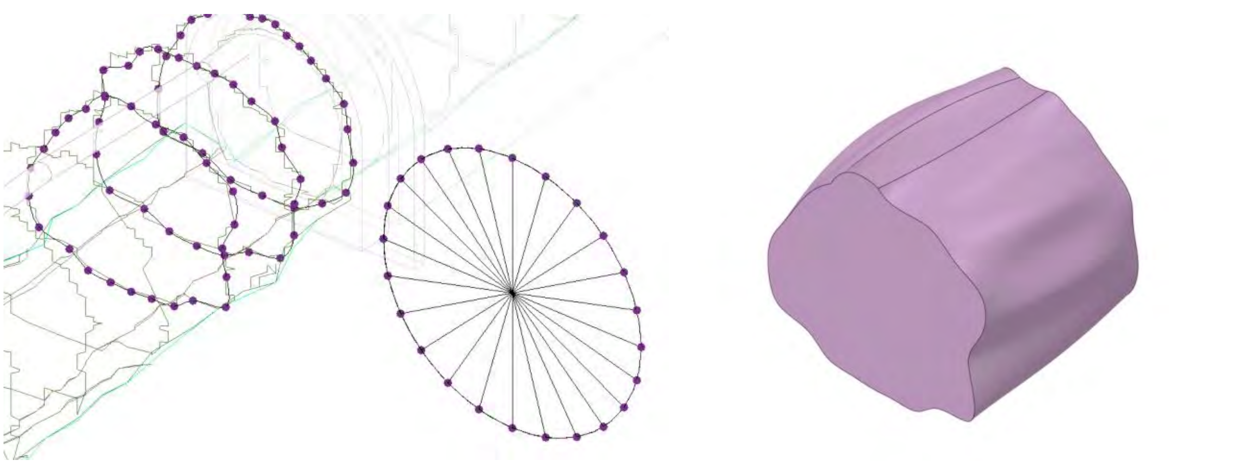


Figure 3.2 Unlined Tunnel – Construction of Model Geometry – Partial Segment of Tunnel

3.2 ADDITION OF BAFFLES

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

The CFD modelling has been completed for the following arrangements:

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

4.0 RESULTS AND PROPOSED OPTIMIZAION

4.1 EXISTING CONDITION HYDRAULIC PERFORMANCE

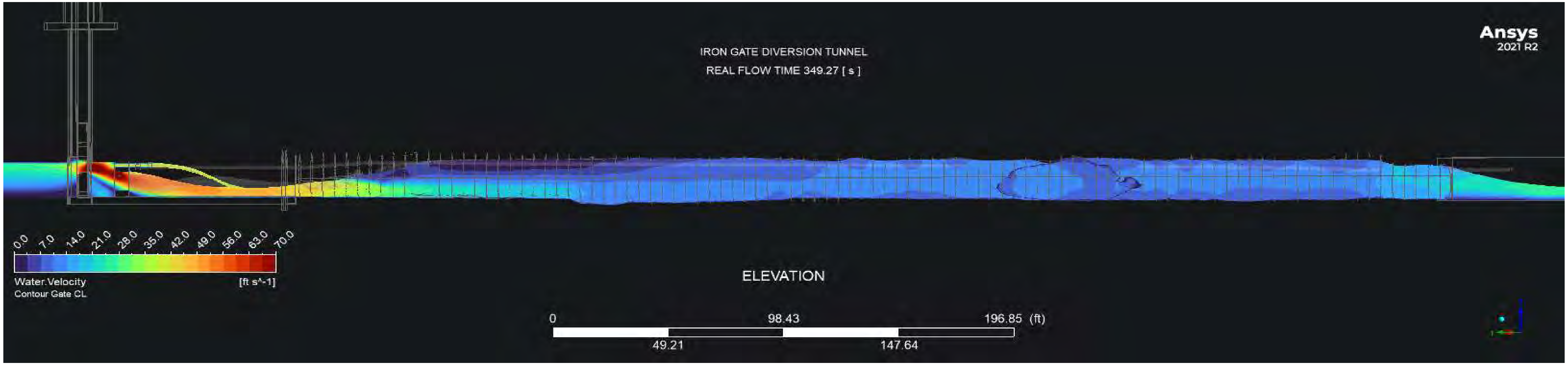
As part of the work to support the Draft 100% Design, CFD analysis, as presented in KP Memoranda VA20-01002 (2020) and VA21-00482 (2021), show that the flow regime in the tunnel downstream of the Ø9 ft orifice features a segment of supercritical, high-energy flow that transitions into subcritical, low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

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

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

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

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



HYDRAULIC DATA:

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

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Kiewit Infrastructure West Co. (California)

Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
CFD RESULTS
FLOW VELOCITY AND HYDRAULIC DATA

Knight Piésold CONSULTING

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

FIGURE 4.1 | REV 0

4.2 EXISTING CONDITION WITH BAFFLES

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

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

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

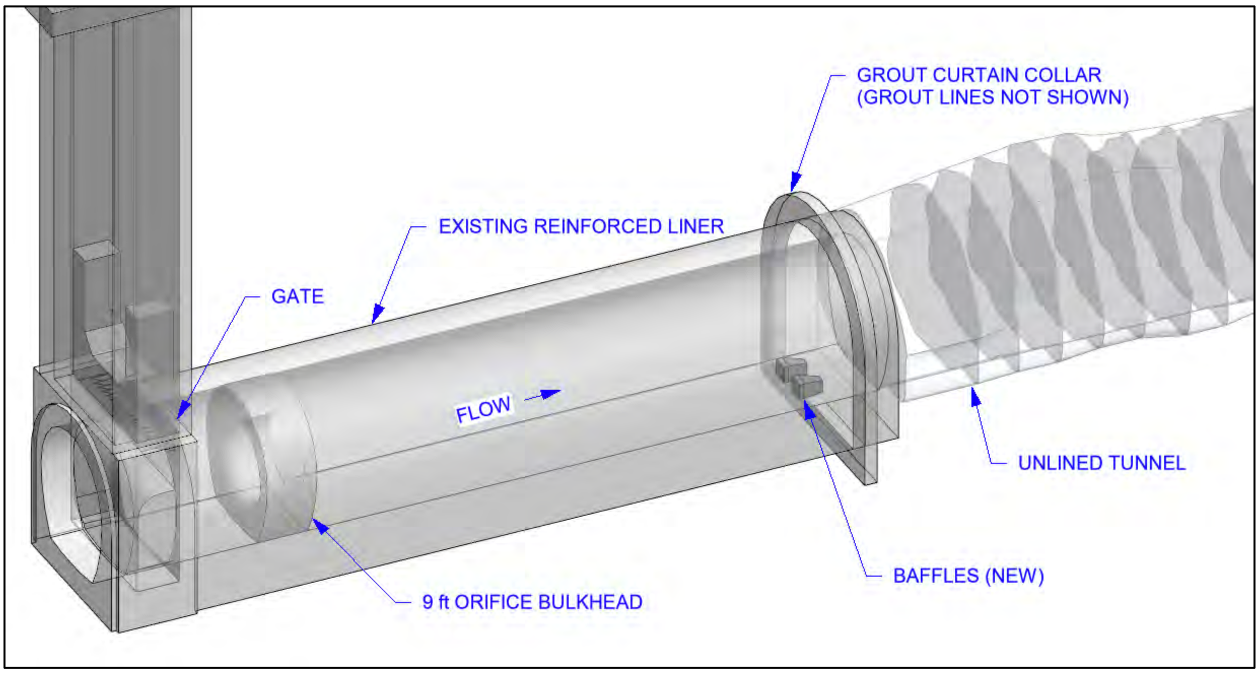
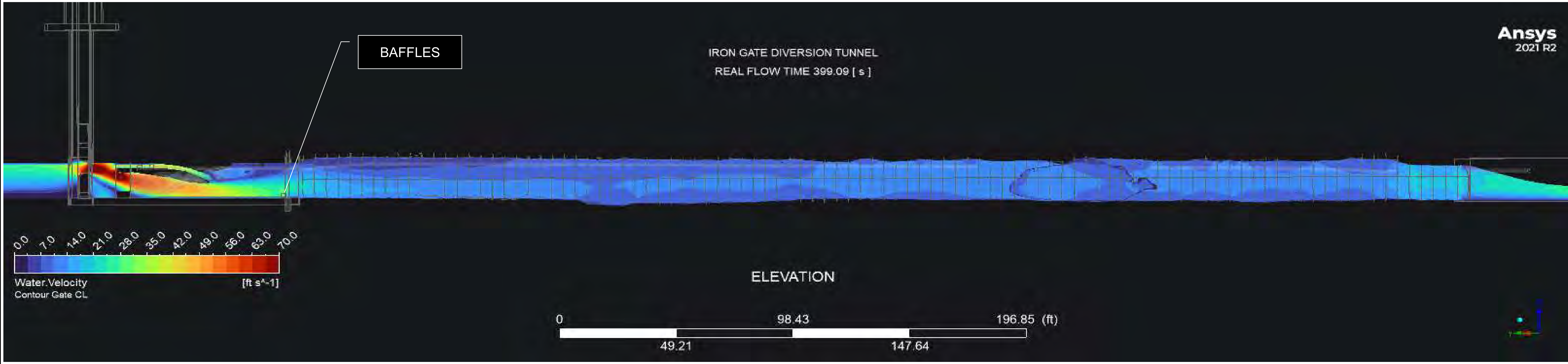


Figure 4.2 Modelled Baffle Geometry

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



HYDRAULIC DATA:

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

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Klamath River Renewal Project

IRON GATE DIVERSION TUNNEL
CFD RESULTS
FLOW VELOCITY AND HYDRAULIC DATA

Knight Piésold CONSULTING

P/A NO. VA103-00640/1

REF. NO. VA22-00905

FIGURE 4.3

REV 0

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4.3 CONCLUSION AND BAFFLE DESIGN DETAILS

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

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

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

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

Yours truly,
Knight Piésold

Prepared: Carlo Capucao

Reviewed: Katrina Wechselberger

Reviewed: Norman Bishop

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

Attachments:

C4193 Baffle Details

References

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

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

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

/eml

EXHIBIT A

100% FINAL Design Report_Appendix E2(June2022) (CEII)
(page 26 of 26)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

PAGE REDACTED IN ENTIRETY

The redacted material qualifies as CEII pursuant to the Commission's rules because it contains sensitive dam safety and construction information that (a) relates details about the production, generation, transmission, or distribution of energy, (b) could be useful to a person planning an attack on critical infrastructure, (c) is exempt from mandatory disclosure under the Freedom of Information Act, and (d) gives strategic information beyond the location of the critical infrastructure. Accordingly, the Renewal Corporation has requested confidential treatment of this material pursuant to 18 C.F.R. § 388.113.

APPENDIX F1
ROADS, BRIDGES, AND CULVERTS
DESIGN DETAILS

TABLE OF CONTENTS

1.0 INTRODUCTION

This appendix provides a comprehensive overview of the designs and design development for construction access and permanent access infrastructure for the Klamath River Renewal Project (the Project). The Project Drawings (100% Design Drawing Package, issued in conjunction with the 100% Design Report) should be reviewed with this document.

The 100% Design Drawings show the latest concepts developed by the Project Team for each of the major components.

Supporting information related to the design of the Roads, Bridges and Culverts components is provided in the following Appendices.

- Appendix A6 – Hydrology
- Appendix A7 – Design Criteria
- Appendix F2 – Supporting Information – Roads
- Appendix F3 – Hydrotechnical Design Report for Roads, Bridges and Culverts
- Appendix F4 – Geotechnical Design Report for Roads, Bridges and Culverts
- Appendix F5 – Copco Access Road Design
- Appendix F6 – Iron Gate Temporary Construction Access Road Design

Table 1.1 Scope Summary

Site	Structure	Work(s) to be Completed	Completed By	Completion Period
J.C. Boyle	Timber Bridge	Removal	Project Company	Post-Project Completion
	Topsy Grade Road Culvert	Post-drawdown monitoring for potential erosion and/or sediment accumulation	KRRC	2 years Post-Drawdown
	Keno Access Road Unnamed Culverts	Post-drawdown monitoring for potential erosion and/or sediment accumulation	KRRC	2 years Post-Drawdown
	Spencer Bridge	Post-drawdown monitoring of bridge embankments and intermediate piers (potential erosion protection and/or repair)	KRRC	2 years Post-Drawdown
	Spring Island Road	General repair and maintenance of Spring Island Road and culvert crossings, as needed, to maintain existing conditions for the Project duration	KRRC	As Required
	J.C. Boyle Intersection Improvements	Temporary improvement of the access points (at two locations) to J.C. Boyle from OR66 Highway. Includes clearing of vegetation, moving and widening of turning radii.	Project Company	As Required
Copco and Iron Gate	Beaver Creek Culverts (East and West Fork)	Post-drawdown monitoring for potential erosion and/or sediment accumulation	KRRC	2 years Post-Drawdown
	Copco Road	General repair and maintenance of Copco Road and culvert crossings, as needed, to maintain existing conditions for the Project duration	Project Company and Siskiyou County	As Required
	Copco Road Bridge	Monitor existing bridge for post-drawdown erosion at abutments and intermediate pier	KRRC	2 years Post-Drawdown
	Camp Creek Culvert	Existing Camp Creek culvert will be assessed post-drawdown and replaced by a concrete box culvert or suitable design.	Project Company	As Required
	Patricia Avenue Culverts (East and West Forks)	Post-drawdown monitoring for potential erosion and/or sediment accumulation	KRRC	2 years Post-Drawdown
	Jenny Creek Bridge	Monitor existing bridge for post-drawdown erosion at abutments.	KRRC	2 years Post-Drawdown
	Scotch Creek Culvert	Existing Scotch Creek culvert will be assessed post-drawdown and replaced by a concrete box culvert or suitable design.	Project Company	As Required
	Dry Creek Bridge	Temporary strengthening structure will be installed at the existing bridge to accommodate anticipated Project vehicle loads.	Project Company	Pre-Drawdown
	Fall Creek Bridge (Copco Road)	Temporary strengthening structure will be installed at the existing bridge to accommodate anticipated Project vehicle loads.	Project Company	Pre-Drawdown
	Fall Creek Bridge (Daggett Road)	Existing Fall Creek culvert will be replaced by a multi-plate arch culvert.	Project Company	As Required
	Fall Creek Bridge (Substation)	No work is planned at this location	-	NA
	Brush Creek Bridge	No action required; existing bridge designed for Permit Load Vehicles	-	NA
	Cottonwood Creek Bridge	No action required; existing bridge designed for Permit Load Vehicles	-	NA
	Raymond Gulch Culvert	Post-drawdown monitoring for potential erosion and/or sediment accumulation	KRRC	2 years Post-Drawdown

Site	Structure	Work(s) to be Completed	Completed By	Completion Period
Copco and Iron Gate	Ager Beswick Intersection Improvements	Temporary improvement of the access points to Iron Gate from Ager Beswick Road at Crest Lane Intersection. Includes clearing of vegetation and widening of turning radii.	Project Company	As Required

NOTES:

1. THE PROJECT COMPANY SHALL MONITOR ROAD, BRIDGE AND CULVERT SITES WITHIN THE WORK LIMITS OF EACH SITE DURING CONSTRUCTION AND UNTIL DEMOBILIZATION.
2. MONITORING AT OTHER LOCATIONS LISTED ABOVE (WHERE NO NEW CONSTRUCTION IS OCCURING) WILL BE COMPLETED, AS DETERMINED BY KRRC.

2.0 REGULATORY COMPLIANCE

The design and construction of the Roads, Bridges, and Culverts components will comply with guidelines stated in Appendix A7; however, ultimately the designs presented here-in will require approval from the appropriate governing agencies, including:

- **Klamath County, Oregon:** The Project Team has a Memorandum of Understanding (MOU) currently in place with Klamath County of Oregon and will coordinate expected construction activities with Klamath County as required.
- **Siskiyou County, California:** The Project Team is actively developing a MOU with Siskiyou County which will clarify Siskiyou County's requirements and the responsibilities of the Project Team at each of the proposed sites and usage of county roads, which are described in the following sections.
- **Fish Passage:** Compliance related to fish passage is covered in Section 4.0 of this Appendix. Agencies consulted during design for review and approval of the designs described herein include California Department of Fish and Wildlife (CDFW) and National Marine Fisheries Service (NOAA).

2.1 ROADS

Project Roads span over two counties: Klamath County in Oregon and Siskiyou County in California. While different governing agencies have jurisdiction on these roads, the focus of this design is on the lower volume County Roads which are expected to experience some construction traffic-related road degradation.

The intent of all road repairs performed will be to maintain or make better the existing road surface conditions. The existing conditions of the roads are summarized in the Project Team's Existing Conditions Assessment Report. Additional evidence of the roads existing conditions may also be gathered directly prior to beginning construction activities. The road improvements for Copco Road and other public Project roads will be determined on an as-needed basis in accordance with the County MOU's.

Some sections of new road have also been developed in association with new permanent crossing designs in Siskiyou County (i.e. Camp Creek and Scotch Creek). The general arrangement of these new alignments is provided on the Project Drawings and will require their approval of the crossing designs discussed in the following section.

Siskiyou County has recognized that in certain cases where existing conditions do not currently meet American Association of State Highway and Transportation Officials (AASHTO) standards, then the proposed design must match or exceed existing conditions.

Additional County Roads used by the Project Team in Klamath County and Siskiyou County will be monitored and maintained throughout construction as outlined in Section 6.1.3.

2.2 BRIDGES AND CROSSINGS

The bridges outlined in the following sections are located in Siskiyou County and include both construction access improvements (C6000 Drawing Series) and post-drawdown improvements (C5000 Drawing Series).

The construction access improvements outlined in Section 6 will be temporary installations throughout dam-removal related construction. The temporary strengthening structures at Dry Creek Bridge and Fall Creek Bridge along Copco Road will be utilized by both construction traffic and public traffic.

The post-drawdown improvements (i.e. new culverts at Camp Creek, Scotch Creek and Fall Creek at Daggett Road) will be permanent structures installed to withstand the post-dam flow regimes, channel incision, and provide passage for aquatic species.

The Project Team met with Scott Waite, Siskiyou County's Director of Public works, on October 28, 2019 to review each of the proposed crossing sites and discuss the county's preference for construction sequencing, in-water works, road right of ways, design criteria and general design constraints and considerations which are commonly encountered by Siskiyou County. This meeting helped to refine some of the concept arrangements at each of the structures (both temporary and permanent). The key elements related to the Siskiyou County approval of bridge designs are outlined below:

- Siskiyou County requires any permanent bridges, culverts, or road modifications to comply with AASHTO standards (as per the Design Criteria Table in Appendix A7). In certain cases where existing conditions do not meet AASHTO standards (e.g. roadway geometry at Scotch/Camp Creek), then the proposed design must match or exceed existing conditions.
- Siskiyou County has confirmed that the temporary construction access bridges are to be designed and stamped by a Professional Engineer. The temporary bridges are not required to meet AASHTO/Caltrans standards and the temporary bridge design criteria (provided in Appendix A7) have been developed by the Project Team and will be ultimately approved by the engineer of record at each site.

2.3 CULVERTS

Culvert improvements and replacements will be coordinated with Siskiyou County and Klamath County. The extent of the culvert improvements and replacements may differ from the outline proposed in Section 6.1.4. The culverts shown on the design drawings will be monitored throughout construction and repaired or replaced on an as-needed basis.

Culvert damage resulting from construction related traffic, not outlined in this report, could potentially require review and acceptance from State and Federal regulatory agencies, however most culvert crossings in the Project Area are not over major streams or tributaries which support aquatic life. Additional stream crossings which require work will be evaluated on an as-needed basis by the Project Team to determine whether improvements are required to meet fish passage criteria.

Siskiyou County understands that existing culverts may have varying conditions prior to construction and that it will not be the Project Company's liability to repair all culverts which are currently damaged. Culverts which are currently operating at a potentially reduced level of functionality have been identified in the Existing Conditions Assessment Report and an additional pre-construction culvert assessment may be employed.

3.0 DESIGN OVERVIEW

Design of all components of the Roads and Bridges scope has been developed throughout the project in consultation with the Project Company, KRRP design team and relevant governing agencies.

The progression of the concepts for each of the road and bridge components has been closely tied to the following factors:

- Opportunities identified for cost savings
- Agency Engagement (CDFW, NOAA)
- Project Company preferences on means and methods
- Field investigations to validate design assumptions (biological/geotechnical/structural)
- Location of existing utilities (i.e. overhead powerlines, subsurface piping/conduits etc.)
- Construction scheduling and haul requirements/constraints at each of the four dam facilities
- Co-ordination of the interface between new bridge structures (and associated channel profiles) and the long-term Project restoration goals

3.1 ROAD DESIGN

Road design generally follows the design intent of matching or improving upon existing conditions. The existing Copco Road features many areas of repaired and patched roadway sections. The visual assessment and available data indicate that the road does not meet standard AASHTO roadway design criteria except for some newer bridge structures and upgraded areas.

Siskiyou County has requested that any new permanent structures be designed as per AASHTO LRFD requirements and that roadway geometry should be improved upon or maintained to the extent practical. Further road design specifics are discussed in Section 6.1.

- Access to Copco 1 and Copco 2 Dam sites will be provided via Copco Road and the I5 Interstate Highway.
- Access to the south bank at Iron Gate Dam site will be provided via Ager Road, Ager Beswick Road and private roads between the Ager Beswick/Crest Lane intersection and the Iron Gate Dam site.
- Access to the J.C. Boyle Dam site will be provided directly from OR66.

3.2 BRIDGE AND CULVERT DESIGN

The following table summarizes the design scope for the Bridge and Culvert structures.

Table 3.1 Crossing Design Scope Summary

Crossing Name	Scope
Existing Timber Bridge at Dry Creek	• Design a temporary bridge strengthening system to allow the existing bridge to accommodate the anticipated Project live loads.
Existing Timber Bridge at Fall Creek	• Design a temporary bridge strengthening system to allow the existing bridge to accommodate the anticipated Project live loads.
Scotch Creek Culvert	• Box Culvert design to be completed by a PE licensed supplier (structural design of the box culvert is not discussed in this report). • Design of the civil components, (i.e. road, embankment, and channel) to support the new box culvert.
Camp Creek Culvert	• Box Culvert design to be completed by a PE licensed supplier (structural design of the box culvert is not discussed in this report). • Design of the civil components (i.e. road, embankment, and channel) to support the new box culvert.
Fall Creek Arch Culvert	• Multi-plate Arch Culvert design to be completed by a PE licensed supplier (structural design of the arch culvert is not discussed in this report). • Design of the civil components (i.e. road, embankment, and channel) to support the new arch culvert.

The Project drawings (5000 and 6000 series) illustrate the concepts for each bridge. Sections 6.0 and 7.0 of this Appendix describe each of the bridges in more detail. Topographic survey data (November 2019) is used to capture the extents of existing bridge structures and channel bathymetry to supplement the baseline Lidar data recorded in 2018/2019.

Following completion of the Project, all temporary bridges will be deconstructed and removed.

Culverts related to construction access roads may require repairs to ensure that the crossings adequately convey water without effecting the safety of the road. As construction progresses, typical road improvement details shown on the Project Drawings will serve as general repair details which can be applied as needed. If repairing an existing culvert is found to be unfeasible, the culvert will be replaced to meet or exceed the existing sizing and geometry.

Hydrologic information (including design floods) for each of the bridge and culvert sites is provided in Appendix A6.

3.3 GEOTECHNICAL DESIGN

Geotechnical design components for both construction access improvements and post-drawdown improvements are described in Appendix F4 – Geotechnical Design Report for Roads, Bridges, and Culverts.

Appendix F4 includes detailed descriptions of sub-surface site conditions, site seismicity, foundation design and analysis.

3.4 TECHNICAL SPECIFICATIONS

The technical specifications for the Roads, Bridges and Culverts are outlined in 32 50 00. The technical specifications are closely tied to the Design Drawings and the Design Criteria in Appendix A7.

3.5 FIELD INVESTIGATIONS

The following table summarizes completed field work.

Table 3.2 Field Investigations Summary Table

Investigation	Summary	Date Completed/Planned
General Investigations	Initial inventory to verify bridges and culverts identified in the Definite Plan	Jun 6, 2019
	Initial assessment of bridges and culverts identified in Definite Plan	Jun 26-27, 2019
Bridge Investigations	Ground topographic survey (including in-stream survey) at: • Lakeview Road Bridge	Nov 11-22, 2019
	Geotechnical investigation with boreholes proposed at: • Fall Creek Bridge • Dry Creek Bridge • Lakeview Road Bridge • Camp Creek Culvert • Scotch Creek Culvert • Fall Creek Bridge (Substation) • Fall Creek Bridge (Daggett Road)	Completed April 2020
Road Investigations	Roads and Bridges Borrow Source Sampling	May 15, 2020
	Copco Road Visual Condition Assessment 1	Jul 17-19, 2019
	Copco Road Visual Condition Assessment 2	Oct 16-17, 2019
	Copco Road GPR Survey • 17.5 miles along both lanes	Nov 18-24, 2019
	Copco Road Pavement Coring and Soil Testing • 18 road cores and Standard Penetration Tests (SPT)	Nov 22, 2019 to Dec 4, 2019
Culvert Investigations	Culvert Inventory • Project wide initial inventory and existing conditions assessment of culverts.	Jul 17-19, 2019
	Culvert Fish Passage Assessment • Field verification of culvert conditions and survey of existing culverts to assess fish passage at key sites identified by the Project Restoration Team. Ground topographic survey (including in-stream survey) at: • Camp Creek • Jenny Creek • Scotch Creek	Sep 25-27, 2019
	Ground topographic survey (including in-stream survey) at: • Fall Creek (Daggett Road) • Fall Creek (Substation)	Oct 28-30, 2019
Site Visit with Siskiyou County	Preliminary review of existing sites and discussion of potential designs and expectations with Scott Waite, Siskiyou County Director of Public Works	Oct 28, 2019

NOTES:

1. RESULTS OF THE KNIGHT PIÉSOLD ROAD AND BRIDGE FIELD INVESTIGATIONS ARE PROVIDED IN THE "EXISTING CONDITIONS ASSESSMENT REPORT" (KNIGHT PIÉSOLD LTD, 2022a).

3.6 TRAFFIC MANAGEMENT PLAN

This design appendix does not include detailed information related to the traffic management at each site other than general descriptions. This topic will be covered in the Project Traffic Management Plan which is currently in development. Complete temporary traffic controls will be required along Copco Road at each of the crossing sites and Project Team recommendations are shown on the Project Drawings for County approval.

3.7 SEDIMENTATION AND EROSION CONTROL

This design appendix does not include detailed information related to the Sedimentation and Erosion Control Plan at each site other than general descriptions. This topic is covered in the Project Best Management Practices (BMP) Plan which is provided in Appendix H. The proposed erosion control measures at each site are shown on the Project Drawings.

4.0 VOLITIONAL FISH PASSAGE

This section includes a summary of the work completed to ensure volitional fish passage is maintained, restored, or improved at the respective road, bridge, and culvert sites.

Transportation related structures pose a high risk of interfering with the primary restoration goal of the project of allowing volitional fish passage. All designs herein apply the design criteria related to volitional fish passage outlined in Appendix A7 and agreed upon with California Department of Fish and Wildlife (CDFW) and the National Marine Fisheries Service (NOAA). While flow characteristics required for fish to swim through different flow conditions are relatively standardized, the perquisites for when the standards need to be applied may differ depending on the governing regulatory agency.

Both California and Oregon require projects that affect stream crossings to assess and incorporate volitional fish passage under specific scenarios. California requires *“if the project affects a stream crossing on a stream where anadromous fish are found, or historically were found, an assessment of potential barriers to fish passage is done prior to commencing project design…If any structural barrier to passage exists, remediation of the problem shall be designed into the project by the implementing agency. New projects shall be constructed so that they do not present a barrier to fish passage (Streets and Highways Code – Div. 1. Article 3.5, Ch. 589, Sec. 3.)”.*

Oregon requires that a bridge must address fish passage if the native migratory fish are currently or were historically present at the location, a new or replacement bridge is planned for construction; or if over 50% of the existing bridge’s elements within, below, or above the channel are cumulatively removed, replaced, filled, or added to through time (OAR 635-412-0005(9)(a)).

The Project Company’s application of volitional fish passage criteria included examining existing structures and their relationship to the Project Company’s proposed works and then ensuring any modifications proposed at transportation structures would allow volitional fish passage, where required.

4.1 EXISTING STRUCTURES FISH PASSAGE ASSESSMENT

Culverts are evaluated as potential barriers to volitional fish passage based on their presence on streams identified as restoration priorities within the Project limits. Table 4.1 summarizes which crossings will be required to meet volitional fish passage criteria to satisfy regulatory conditions, based on the Project Company's proposed activities.

Table 4.1 Volitional Fish Passage Assessment Summary

Crossing Name	Requirement to Meet Volitional Fish Passage Criteria
AgerBeswick-66500	No
AgerBeswick-77750	No
Camp Creek	Yes – Existing structure does not meet criteria and negative drawdown related effects expected.
CopcoRoad-59000	No
CopcoRoad-60+300	No
East Fork Beaver Creek	No
Fall Creek (Substation)	Yes – Existing structure does meet criteria and does not impede movement to the Fall Creek Fish Hatchery
Fall Creek (Daggett Road)	Yes – Existing structure does not meet criteria and impedes movement to the Fall Creek Fish Hatchery
Indian Creek - Ager Beswick	No
Jenny Creek	Yes – Existing structure meets criteria. Project Company will ensure any modifications continue to meet criteria.
Keno Access Road - West	No
Scotch Creek	Yes – Existing structure does not meet criteria and negative drawdown related effects expected.
Topsy Grade-7200	No
West Fork Beaver Creek	No – Will need to meet criteria only if modifications to existing structure become required. Crossing will be monitored post-drawdown; however, no modifications are currently expected.

The Project Company determined that the existing Scotch and Camp Creek stream-road crossings do not meet fish passage criteria and will become perched once the creeks are restored due to channel incision. The existing Scotch and Camp Creek culvert crossings are located on reservoir deposits. Following drawdown, the creek profiles will adjust to an elevation lower than the existing crossing invert elevations. Furthermore, the culverts' corrugated metal inverts are exposed and if not backwatered, the steep culvert slopes would create velocities that exceed fish passage criteria.

Passage past the Fall Creek crossings at Daggett Road and Pacific Power Substation Access Road is important because these crossings are located downstream of the proposed Fall Creek fish hatchery. The Fall Creek crossing at Daggett Road consists of a 10 ft diameter corrugated metal pipe. The pipe is perched approximately 1.5 ft above the downstream scour lag deposit crest. The culvert at Daggett Road does not meet fish passage criteria for adult and juvenile salmonids. The Fall Creek at the Pacific Power Substation Access Road consists of a concrete bridge with span of 24 ft and a channel bottom width of approximately 16 ft. Flow conditions through the bridge mimic upstream and downstream flow conditions and are therefore not deemed a fish passage barrier by the National Marine Fisheries Service.

The Project Company is not proposing construction access improvements or restoration activities and does not expect any drawdown related degradation of the West Fork Beaver Creek crossing. The post-drawdown monitoring program will assess the site for signs of destabilization due to drawdown related flows. If modifications are required following the post-drawdown monitoring period, the modifications will be designed to promote volitional fish passage, as the tributary has a historic fish presence.

The Project Company does not believe remediation is required to provide fish passage at the other crossings identified in Table 4.1.

4.2 FISH PASSAGE IMPROVEMENTS

Fish passage improvements are intended to meet National Marine Fisheries Service (NMFS 2019) and CDFW criteria. NMFS allows for three methods for incorporating fish passage into crossing designs:

- Active Channel Design
- Stream Simulation Design
- Hydraulic Design

The Project Company is employing the stream simulation design method for the Camp, Scotch, and Fall Creek (Daggett Road) crossings. Stream simulation design is intended to "mimic the natural stream processes" through the crossings that are observed upstream and downstream. To this end, sediment transport and debris movement through the crossings should be similar to the upstream and downstream reaches. The proposed crossings at Camp and Scotch will include concrete box culverts embedded into the stream and will have widths near the active channel width, which allows for passage of sediment and debris, and have maximum slopes less than 6%. Camp Creek and Scotch Creek are located at the transition between the restored channels and project limits. The channels will be designed with engineered streambed to maintain stable designs through the crossings. The existing Fall Creek at Daggett Road culvert will be replaced by a multi-plate arch and will have an open bottom. The width is approximately 1.5 times the active channel width.

Hydraulic design for each of the three crossings are covered in Appendix F3 – Roads, Bridges and Culverts, Hydrotechnical Design Report.

5.0 EXISTING PROJECT BRIDGE RATINGS

KP conducted a desktop review of the existing bridges within the Project limits to assess the load carrying capacity and condition of each bridge. This information was sourced from existing bridge load ratings, inspection reports, as-built drawings and load ratings that were developed by KP based on site inspection data and typical material strength parameters.

The load rating for each bridge refers to the maximum permissible vehicle loading that is permitted on the bridge. Bridge load rating is typically expressed in terms of a standard truck load and a maximum vehicular load (i.e. permit truck load). The magnitude and distribution of such loadings are based on maximum axle weights and axle spacing for a specific design vehicle.

A summary of the proposed solutions for construction access at each Project bridge is provided in the table below.

Table 5.1 Existing Bridge Status and Proposed Actions

Existing Bridge	Bridge Load Rating According to As-Built Information	Action
Lakeview Road Bridge	40 T – 4 axle truck	No modifications planned. Project traffic exceeding the posted load limits will be routed to alternate access route via Ager Beswick/Lakeview private roads.
Dry Creek Bridge	No Rating Specified. KP inspection deemed this bridge insufficient for anticipated project loads.	Construct temporary strengthening structure to support existing bridge. Remove following Project completion.
Fall Creek Bridge at Copco Road	No Rating Specified. KP inspection deemed this bridge insufficient for anticipated project loads.	Construct temporary strengthening structure to support existing bridge. Remove following Project completion.
Copco Road Bridge	• HS20-44 • Alternate Design Load • Permit Design Load	No modification required
Jenny Creek Bridge	• HL93 • Permit Design Load	No modification required
Brush Creek Bridge	• HS20-44 • Alternate Load	No modification required
Cottonwood Creek Bridge	• HS20-44 • Permit Design Load	No modification required
Bogus Creek Bridge	• HS20-44	No modification required. Visual inspection noted some cracks in the concrete deck and that RSP requires maintenance.
Willow Creek Bridge	• HS20-44	No modification required
Klamathon River Bridge	• HS20-44 (NBI).	Not used for Construction Access Loads. Visual inspection noted cracking and excessive deflection in main central span

NOTES:

1. DATA FOR BRIDGE LOAD CAPACITIES TAKEN FROM POSTED LOAD LIMITS, AS-BUILT DRAWINGS AND THE FEDERAL HIGHWAY ADMINISTRATIONS' NATIONAL BRIDGE INVENTORY (NBI) ANNUAL INSPECTION REPORTS.
2. THIS TABLE WAS DEVELOPED FROM PUBLICLY AVAILABLE DATA AND SHOULD NOT BE CONSIDERED A COMPREHENSIVE STRUCTURAL ASSESSMENT FOR ALL PROJECT BRIDGE LOAD RATINGS.

6.0 CONSTRUCTION ACCESS IMPROVEMENTS

Construction Access improvements include repairs, upgrades, and modifications to existing transportation infrastructure and new temporary crossings to accommodate construction vehicles and equipment. The proposed improvements include:

- Improvements/repairs to the existing public and private road network to ensure the roads match or exceed existing conditions following completion of the Project. This work will be conducted in co-ordination with Siskiyou and Klamath County on an as-needed basis and will be carried out in compliance with the MOU.
- Bridge strengthening systems to accommodate live loads from Project vehicles where existing bridges have inadequate structural capacity bridge.

6.1 ROAD IMPROVEMENTS

The goal of road improvements is to maintain or improve the existing road surface conditions. Prior to construction the existing roads may be driven and recorded to provide the baseline for future road improvements, supplementary to the Existing Conditions Assessment Report (KP, 2022a). The timing and extent of these repairs will be determined by the MOU.

Site specific road improvements at each of the four dam sites will be developed as required by the Project Company to facilitate the haul plans and construction strategy at each site. In general, the Project Company will ensure that the temporary construction access roads at each of the four dam sites are well maintained and fit for purpose for the duration of the Project. Any site-specific road considerations are discussed in the respective facility appendix.

Road investigations have been performed to aid in delineating potential road repairs. These investigations are outlined as follows:

- **GPR Survey:** A GPR survey was completed on 17.5 miles of Copco Road from the Ager Road Bridge crossing the Klamath River to the Copco Dam Access Road. The survey was completed to help evaluate existing asphalt thickness and conditions and to estimate road subgrade soil/rock types and conditions. Two GPR survey passes were made along the road, one in each lane, for a total of 35 miles of survey. Each traffic lane was scanned by one pass that corresponded to the primary vehicle wheel ruts. Heading east, the survey line was on the outside lane within the outer tire tread. Heading west, the survey line was on the inside lane within the inner tire tread. Within areas of obvious asphalt and/or subgrade failure, additional GPR passes were completed to better define the horizontal and vertical extents of the failures.
- **Road Core Sampling:** Road core sampling was completed at 18 locations along the Copco Access road, and the core locations were spread out with approximately 1 core per mile of road. The asphalt was cored using a 6-inch core bit. The road subgrade was sampled using a Standard Penetration Test sampler. The road cores were located along the outside lane and were generally within the primary vehicle wheel ruts.
- **Iron Gate Alternate Route Assessment:** An assessment of potential alternate routes for the Iron Gate Dam site was undertaken during the Value Engineering phase. Access to the Iron Gate Dam site via the private roads, located south of the Klamath River and connecting to Ager Beswick Road, can be provided pending maintenance and improvements (i.e. gravel surfacing, widening at tight bends). A preliminary agreement is in place between the Project Company and private landowners for Project use. This alternate route has superseded the requirement for a temporary construction access bridge at Lakeview Road, previously outlined in the 60% DCD’s. Figures F2-1 and F2-2 provided in Appendix F2.2 show the alternate route map and key observations noted during the assessment.

6.1.1 COPCO ROAD

It is anticipated that Copco Road will serve as the access route to construction activities associated with Iron Gate, Copco No. 1, and Copco No.2. Due to the high amount of use projected through the construction period, and the types and frequencies of distresses currently present on Copco Road, some degradation of the existing road is anticipated throughout the project. The proposed pavement repairs may involve portions being re-paved prior to construction, during construction, and potentially post-project.

The potential road improvements are based on information obtained from visual inspection of the existing road surfaces. Appendix F2.1 contains photographs of various degradations currently existing on Copco Road. Two examples are shown on Figures 6.1 and 6.2 below. Asphalt pavement rehabilitation procedures currently include a mill and overlay option and an asphalt and base course replacement option to address surficial issues and subgrade issues, respectively.

Figure 6.1 Copco Road Mill and Overlay Repair Examples

Figure 6.2 Copco Road Asphalt and Basecourse Replacement Repair Examples

To facilitate the increased construction related traffic while minimizing potential delays the Project Company proposes designated pull-outs. These pull-outs utilize an aggregate-base road surface to provide vehicles, a designated place to pull to the side of the road to allow vehicles to pass easily. The pull-out locations are selected as areas that require minimal earth work. The locations of the pullouts will be finalized based on the Project Company's haul plan and schedule.

6.1.2 LAKEVIEW / AGER BESWICK ACCESS ROAD

The Project Company will co-ordinate with local landowners to maintain and modify the private roads, as required between Ager Beswick Road/Crest Lane intersection and the Iron Gate Dam site. This portion of the route includes approximately 5.8 miles of private gravel road. Figures provided in Appendix F2.2 show the general route map and a high-level overview of the conditions observed during a visual assessment completed in April 2020. The recommended improvement actions along this route include.

- Gravel re-surfacing as needed to accommodate Project vehicles (approx. length of unsurfaced road = 4.35 miles).
- Widening of tighter turns (see Appendix F2.2 Figure 1, Figure 2).
- Potential replacement of some culverts noted as being in poor condition (see Appendix F2.2 Figure 1, Figure 2).
- Some sections of Ager Beswick showed evidence of differential settlement, indicating weak subgrade conditions.

6.1.3 OTHER PROJECT ROADS

The roads in the project area and contiguous areas were surfaced with either asphalt or aggregate base rock. Based on a review of existing Project roads, it is not anticipated that any construction access improvements will be required on the roads with an asphalt surface type. Roads surfaced with aggregate base rock may require additional construction access modifications to accommodate construction vehicles within Project work areas or to repair damage caused by construction related traffic. The Project Team will regularly maintain the aggregate base road surfaces and other haul roads throughout the construction period, as per the County MOU's.

6.1.4 ROAD MONITORING AND MAINTENANCE

Project roads will require monitoring during and after construction and additional maintenance may be required on an as needed basis.

Copco Road east of Fall Creek Bridge is not maintained by Siskiyou County; this includes snow removal over the winter. Maintenance of this section of road will be performed by the Project Company during construction. Spring Island Road, which will serve as the primary access road for J.C. Boyle construction, will be maintained by the Project Team throughout construction.

6.1.5 CULVERT IMPROVEMENTS

Existing minor culverts along the construction access routes will be monitored and repaired on an as-needed basis throughout construction to ensure culverts meet or exceed their existing conditions.

Culvert damage resulting from construction related traffic damage, not outlined in this report, could potentially require review and acceptance from State and Federal regulatory agencies, however the majority of culvert crossings in the Project Area are not over major streams or tributaries which support aquatic life. Additional stream crossings which require work will be evaluated on an as-needed basis by the Project Company to determine whether improvements are required to meet fish passage criteria.

6.1.6 TEMPORARY INTERSECTION IMPROVEMENTS

Intersections at Iron Gate and J.C. Boyle will be temporarily improved to facilitate low-boy haul vehicles with larger turning radii. These improvements will require some select clearing of vegetation as required to achieve the necessary turning circles and appropriate lines of sight.

The proposed improvements for each of the following intersections are shown, as a conceptual arrangement for agency review, on the Project Drawings.

- C6500 - Crest Lane/Ager Beswick
- C6600 - OR66 Improvement 1
- C6610 - OR66 Improvement 2

6.2 FALL CREEK BRIDGE STRENGTHENING

The existing timber girder bridge at Fall Creek will be reinforced with a temporary strengthening system for the duration of the Project. A photograph of the bridge is presented on Figure 6.10.

The existing bridge has been assessed for general condition and load carrying capacity. The bridge features a single-span deck with timber girders as primary structural (load-carrying) members.

Figure 6.3 Existing Fall Creek Bridge

The Project Team was unable to source any as-built drawings or structural/geotechnical design data related to the Fall Creek bridge. As such, field measurements for the primary structural members are used to estimate the bridge's current load carrying capacity and determine applicable strengthening solutions for the bridge to pass construction traffic loads during the implementation of the Project.

Table 6.6 presents a summary of the field measurements conducted for the Fall Creek Bridge crossing.

Table 6.1 Fall Creek Bridge Geometric Properties

Bridge ID	Clear Span (ft)	Typical Girder Section (Width x Height)	Girder Spacing (ft)
Fall Creek	24.6	5.5" x 21"	1.33.

6.2.1 DESIGN LOADS

The design vehicular live load (LL) considered in the assessment of the bridges, which also forms the basis of the superstructure loading, is the HL-93 design truck load as specified in AASHTO LRFD, shown in Figure 6.11.

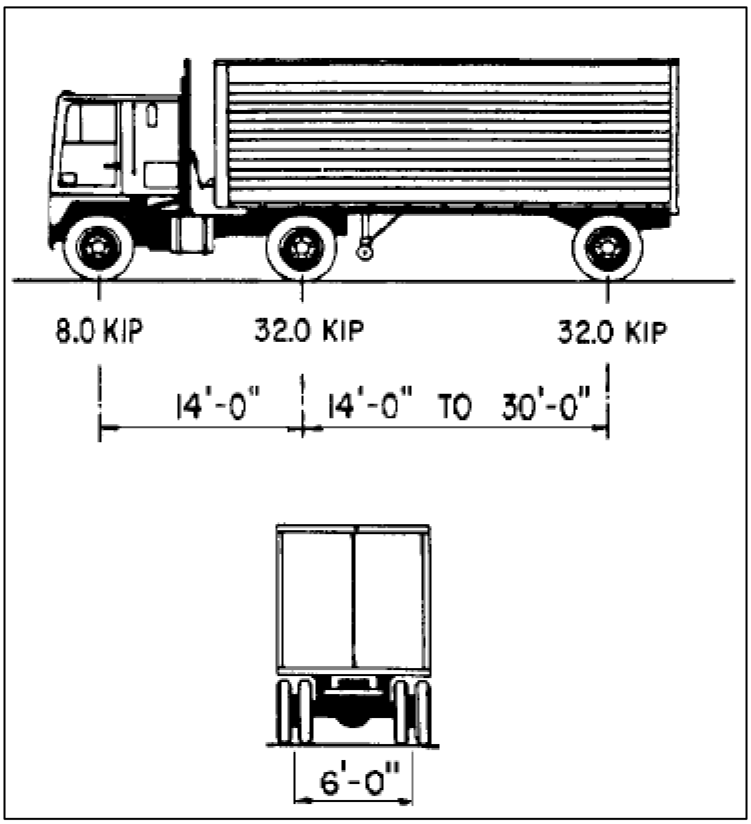

Figure 6.4 Design Truck HL-93 (AASHTO LRFD Article 3.6.1.2.2)

The dead load for structural components and non-structural attachments (DC) applied to the superstructure includes the self weight of the timber girders ($\gamma_{DFIR\text{-}LARCH}$ = 31 lbf/ft^3) and a 3.5" deck layer (measured on site) assumed to be composed of asphalt ($\gamma_{ASPHALT}$ = 150 lbf/ft^3).

AASHTO LRFD Strength II load combination is considered in the analysis. Its description is given in AASHTO LRFD Article 3.4.1:

"*Strength II – Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.*"

Load case factors are summarized as:

- Strength II Load Combination = 1.25(DC) + 1.35(LL)

Lateral load is assumed to comprise an accidental dynamic impact (collision) load from a 500 lb floating wood debris moving at a flow velocity of 1 ft/s. The resulting impact load is estimated to be 3.9 kip. This lateral load was developed as a conservative lateral load case due to low risk of wind/seismic loads for the temporary structure. The probability of floating debris directly impacting the steel girder is not considered a major structural risk. In the event of such a storm/flood event, the structure will be inspected for movement/settlement and any evidence of impact damage.

6.2.2 ESTIMATED LOAD CARRYING CAPACITY

The Project Company was unable to find reference to the type and grade of the timber girders at Fall Creek bridge. As such, the current load carrying capacity of Fall Creek Bridge's timber girders is estimated based on assumed strength properties. The representative type of timber considered is Douglas-Fir Larch. The range of strengths was established based on varying grades of timber (according to decreasing strength): Select Structural, Grade No. 1 & Btr and Grade No. 2.

Design strength for the various grades of Douglas-Fir Larch timber have been obtained in accordance with reference values and adjustment factors presented in AASHTO LRFD Section 8 – Wood Structures. The flexure and shear resistance values for the timber girders are summarized in Table 6.7:

Table 6.2 Fall Creek Bridge – Timber Girder Flexure and Shear Resistance

		Timber Grade (Douglas-Fir Larch)		
		Select Structural	**Grade No.1 and Btr**	**Grade No. 2**
Flexure Resistance	(kip-ft)	81.98	66.34	50.16
Shear Resistance	(kip)	23.73	23.73	26.89

6.2.3 LOAD RESPONSE OF EXISTING STRUCTURE

The bridge superstructure is modeled in SAP2000® to determine the maximum response due to given loads. The design vehicle live load is implemented as a moving load across the girders and as such the response is presented as an envelope of maximum and minimum values.

Figure 6.12 presents the flexure and shear response of the existing timber girder at Fall Creek Bridge.

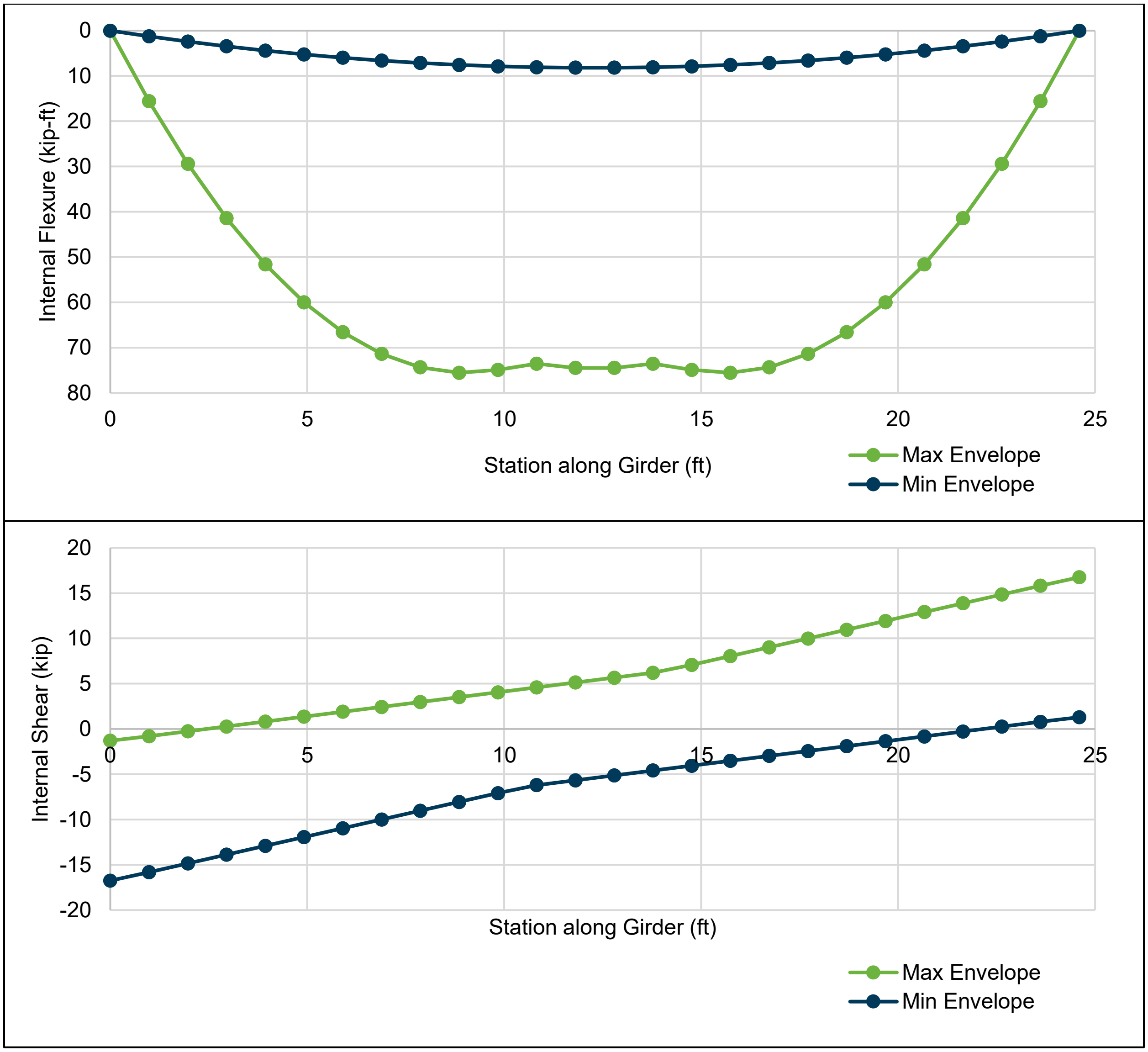

Figure 6.5 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Fall Creek Bridge Timber Girder – Existing Configuration

Analysis shows that the bridge's shear strength is adequate to resist the design loading. However, under the existing bridge configuration, only the Select Structural grade of timber meets the load carrying capacity required. Field observations have shown that the timber girders may be considered to have less strength due to the presence of several knots in the members and signs of water damage from a failing deck.

Figure 6.13 shows the deflected shape of the existing timber girders under the Strength II load combination.

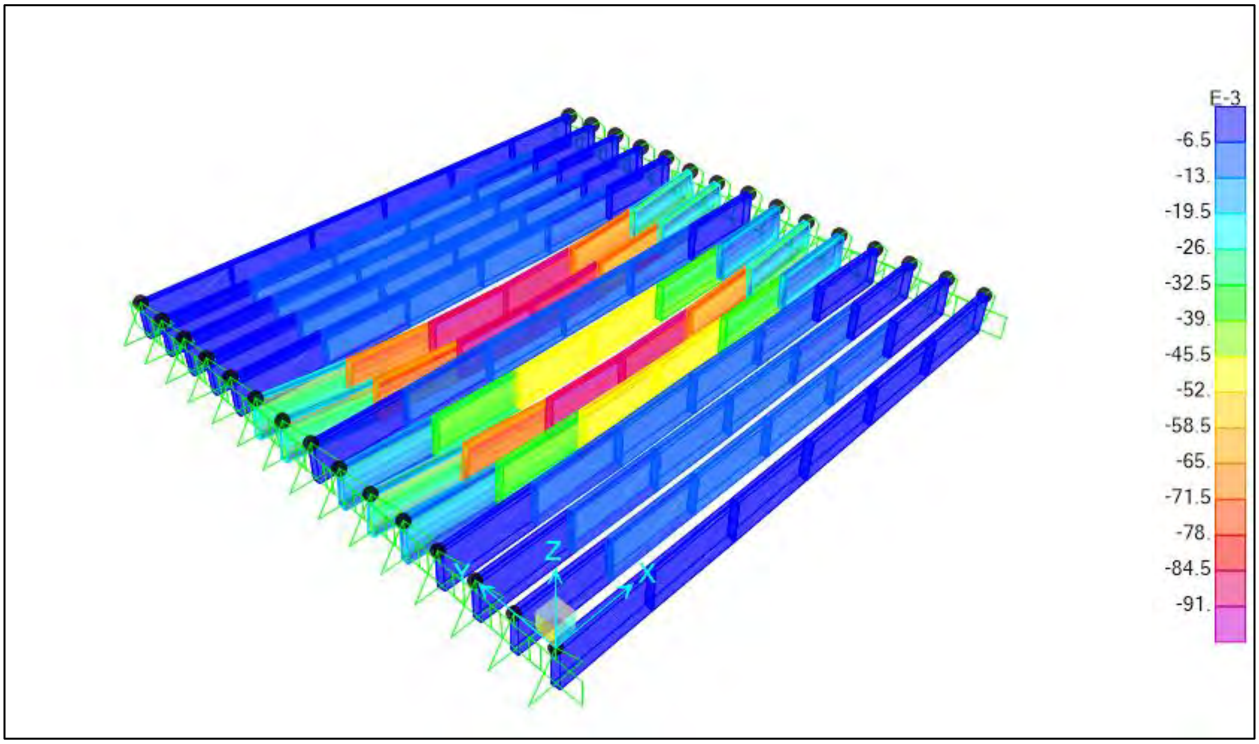

Figure 6.6 Strength II Load Combination – Fall Creek Timber Girders – Deflected Shape (10x Scale) – Vertical (Uz) Deformation Contours (ft)

6.2.4 STRENGTHENING STRUCTURE

To strengthen the timber girders, or reduce their load response under the design loads, a separate support structure will be installed, running transversely, underneath the center of the bridge span to act as an intermediate pier. The center support is composed of two 40 ft steel beams, spaced approximately 3.9 ft on-center, and oriented perpendicular to the existing bridge alignment.

The steel beams that make up the center support are sized based on their capacity to resist the design loading. The material and strength properties of the center support beams are summarized in Table 6.8. Strength properties are calculated in accordance with AASHTO LRFD Section 6 – Steel Structures.

Table 6.3 Center Support Beam Properties

Beam Section	W24x117
Grade	ASTM A992 Grade 50 (fy = 50 ksi)
Flexural Resistance	345 kip-ft
Shear Resistance	365 kip

Figure 6.14 shows the load response (flexure and shear) of the timber girders under the design loads after the center support beams are introduced.

The temporary support girders will not be fixed or attached to the existing timber girders. The top flange of the steel girders will contact the underside of the timber girders and will accommodate vertical load transfer from the timber deck. The girders will be laterally restrained at the end of the structure, at the lockblock supports, by diagonal bracing and an exterior steel band which will be tightened snug. Some additional lateral restraint of the compression flange will be provided through contact with the timber girders, but this contribution is ignored in the check for lateral torsional buckling, which assumes the unbraced length is the full length along the girder between support points.

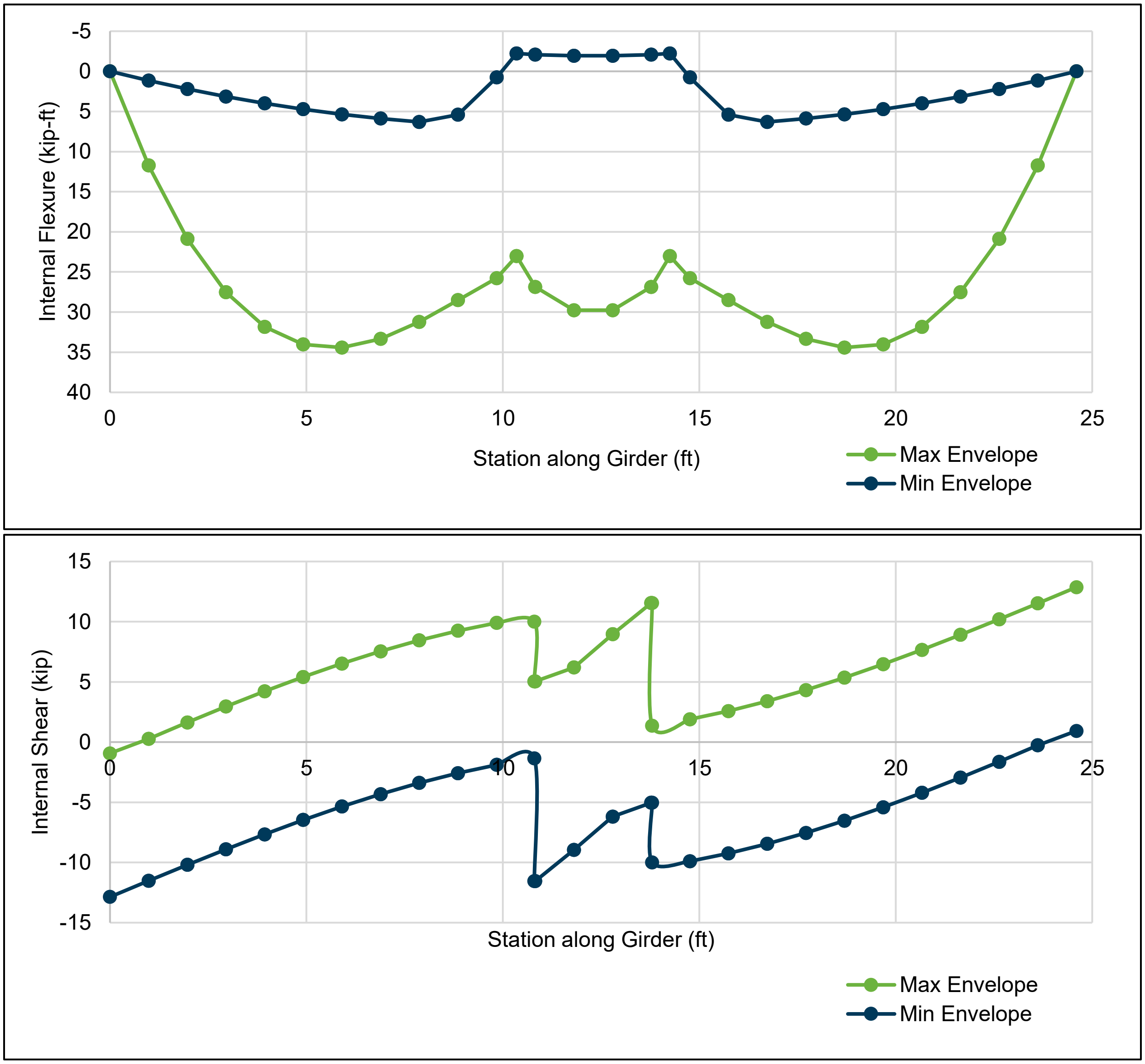

Figure 6.7 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Fall Creek Bridge Timber Girder – With Center Support Beams

The graph above shows that the maximum flexure in the timber girder is significantly reduced when the center support beams are introduced. At this supported configuration, the load response of the timber girder is found to be below the range of estimated flexural and shear strengths as shown in Table 6.7.

Similarly, the load response in the center support beam is found to be below the estimated flexural and shear resistance presented in Table 6.8. Figure 6.15 presents the flexure and shear load response of the center support beam.

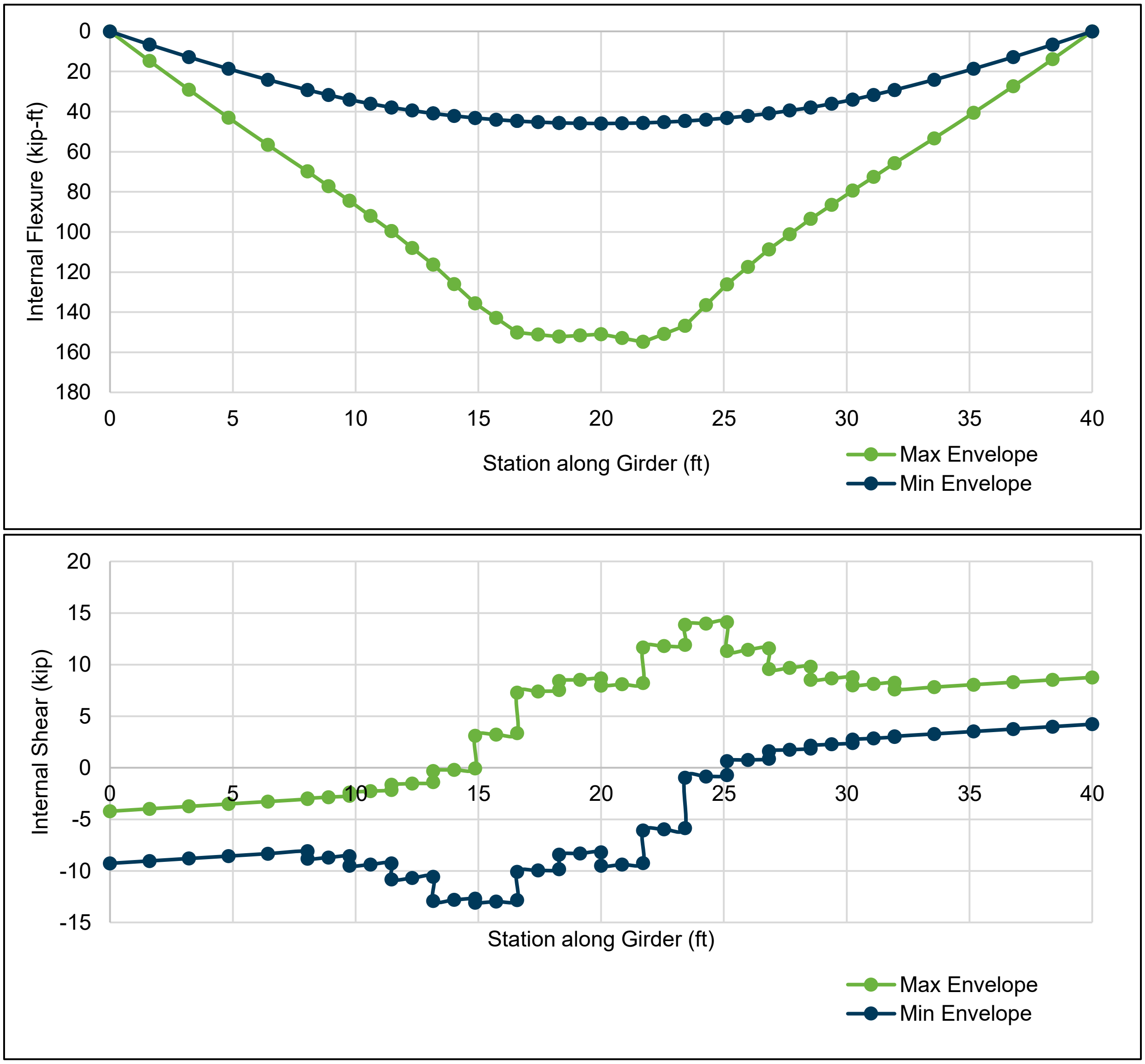

Figure 6.8 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Fall Creek Bridge Center Support Beam

Figure 6.16 shows the deflected shape of the existing timber girders with the center beam supports under the Strength II load combination.

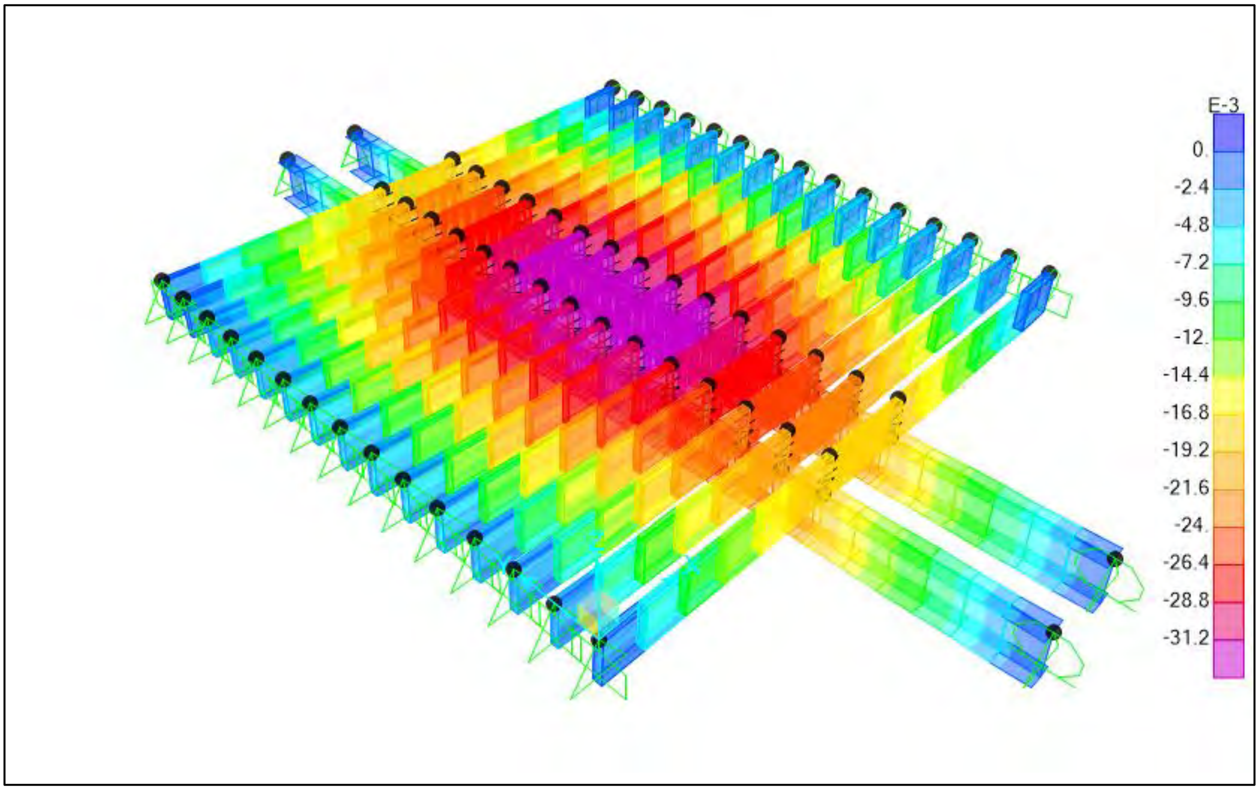

Figure 6.9 Strength II Load Combination – Fall Creek Timber Girders with Center Support Beams – Deflected Shape (10x Scale) – Vertical (Uz) Deformation Contours (ft)

The center support beams are found to decrease the overall maximum deformation of the timber girders in addition to reducing their maximum load response to the design loads.

6.2.5 CONNECTIONS

The temporary strengthening structure will be installed with the support beams, jacked into position, in contact with the underside of the existing timber girders. Fasteners will not be included in the contact interface. As such, any lateral load applied to the support structure is not expected to transfer to the existing timber bridge, and vice versa other than some minor secondary friction forces which are not expected to influence the performance of the structure.

The steel center support beams will bear on a 1’x1’x5’ timber sill and 2.5’x2.5’x5’ interlocking concrete block base at each end. The steel members’ bottom flanges will be bolted on the timber sill using Ø1” lag bolts. The timber sill will be connected to the concrete lock block using Ø1” threaded anchor rods, drilled and bonded with epoxy.

The lateral load capacity of the foundation connection was calculated based on the provisions in AASHTO LRFD Section 8 – Wood Structures and AWC-NDS Section M11 – Dowel-type Fasteners. Material properties as well as the design values calculated are summarized in Table 6.9. The strength of the bearing support and the connections are designed to adequately resist the applied loads outlined in Section 6.3.1 of this report.

Table 6.4 Center Support Beam – Bearing Connection – Material and Strength Properties

Property	Value	Notes / Reference
Timber Sill Grade	Douglas-Fir Larch, Select Structural	
Concrete Base Strength	Minimum 28-day Compressive Strength = 4,500 psi	
Lag Bolt Grade	Minimum Yield Strength = 58 000 psi	ASTM A193 Lag Screw
Threaded Anchor Rod Grade	Minimum Yield Strength = 58 000 psi	ASTM A193 Threaded Bolt
Compression / Bearing Resistance	150 kip	Compression of timber sill, perpendicular to grain (AASHTO LRFD Article 8.8.3 – Compression perpendicular to grain)
Connection Resistance (Lateral Load)	4.7 kip	Resistance of two threaded anchor rod connections (AWC-NDS Section M11 – Dowel-type fasteners)

6.2.6 FOUNDATION

The steel girders of the temporary strengthening system will be supported at each end by pre-cast concrete interlocking blocks (typically 2.5 x 2.5 x 5 ft in size) piers. The existing channel bed will be shaped to place a short steel confinement box, which will allow placement of competent material within the box, forming the foundation pad for the interlocking concrete blocks. The steel confinement box can be placed without the need to create a dry isolated work area.

The structural pad material will conform to the channel bed and create a level surface for placing the concrete blocks. There is limited headroom at Fall Creek bridge and it is anticipated that one precast block will be placed, additional support height will be gained through 12”x12” timber sills and steel shim plates and adjusting the depth of material in the steel confinement box. The strengthening structure has relatively low mass and short supports with no mechanism for lateral load transfer from the existing bridge deck, therefore seismic load cases are not considered. As a conservative engineering exercise, the interlocking concrete blocks are checked for stability against hydrodynamic forces and accidental woody debris impact loads. Geotechnical aspects of Fall Creek are discussed in Appendix F4.

It is recommended that the temporary strengthening system be visually inspected following any major storm/flood events or any noted seismic activity to check for movement/settlement and to ensure good contact is maintained between the top flange of the support girders and the existing timber girders.

Flow in Fall Creek is supplemented by upstream control structures outside of the Project work limits. Data indicates a relatively constant flow in the stream of approximately 12 ft^3/s and a flow depth of 1 ft. This flow may be suspended or reduced to allow for easier placement of the foundation pads, this operational consideration is to be determined by the Project Company and the operators of the upstream control structures.

6.2.7 SEQUENCING

At the time of this report, the following steps summarize the anticipated installation sequence at Fall Creek Bridge.

- Installation is planned for the July-October construction window, when historically the creek has reduced run-off flow. Co-ordination is required with the owners of the upstream flow control structures.
- The channel bed will be prepared for installation of the steel confinement template and the foundation pads.
- Foundation structural pads will be placed and compacted and lockblock supports will be installed.
- Support girders will be dragged under the existing timber bridge, lifted on to the lock block supports and jacked into position to achieve contact with the underside of the existing bridge deck.
- Connections and bracing will be installed prior to removing jacks.
- Due to the unknown design rating of the existing bridge decking a 1” steel traffic plate to improve local load distribution should be installed over the entire bridge deck surface.
- The bridge will be load tested to assess any settlement and the support system will be adjusted as required.
- The strengthening system should undergo visual inspection following any major storm/flood events for any signs of movement or settlement.
- Following Project completion, the strengthening structures will be deconstructed and removed and the channel bed at the foundation pads will be restored to natural conditions.

6.3 DRY CREEK BRIDGE STRENGTHENING

Dry Creek Bridge features a similar structure to Fall Creek Bridge - single-span deck with timber girders as the primary structural (load-carrying) members. A photograph of the bridge is presented on Figure 6.17.

Figure 6.10 Existing Dry Creek Bridge

Table 6.10 presents a summary of the field measurements conducted for the Dry Creek Bridge crossing.

Table 6.5 Dry Creek Bridge Geometric Properties

Bridge ID	Clear Span (ft)	Typical Girder Section (Width x Height)	Girder Spacing (ft)
Dry Creek	22.0	6" x 16"	1.35

6.3.1 DESIGN LOADS

Dry Creek Bridge shares similar design loads with Fall Creek Bridge; see Section 6.3.1 of this report.

The dead load of the deck on Dry Creek Bridge differs from Fall Creek as field measurements indicate a thicker deck layer composed of vertical 4” x 6” Douglas Fir continuous timber decking and 3” asphalt wear surface over the top of the wood decking, for a total deck thickness equal to approximately 8.5” to 9”.

6.3.2 ESTIMATED LOAD CARRYING CAPACITY

The Project team was unable to find reference to the type and grade of the timber girders. As such, the current load carrying capacity of Dry Creek Bridge’s timber girders is estimated based on assumed strength properties. The representative type of timber considered is Douglas-Fir Larch. The range of strengths is established based on varying grades of timber (according to decreasing strength): Select Structural, Grade No. 1 & Btr and Grade No. 2.

Design strength for the various grades of Douglas-Fir Larch timber are in accordance with reference values and adjustment factors presented in AASHTO LRFD Section 8 – Wood Structures. The flexure and shear resistance values for the timber girders are summarized in Table 6.11.

Table 6.6 Dry Creek Bridge – Timber Girder Flexure and Shear Resistance

		Timber Grade (Douglas-Fir Larch)		
		Select Structural	Grade No.1 and Btr	Grade No. 2
Flexure Resistance	(kip-ft)	53.37	42.86	32.24
Shear Resistance	(kip)	19.72	19.72	19.72

6.3.3 LOAD RESPONSE OF EXISTING STRUCTURE

A separate model in SAP2000® is used for Dry Creek Bridge to assess the maximum response due to given loads.

Figure 6.18 presents the flexure and shear response of the existing timber girder at Dry Creek Bridge.

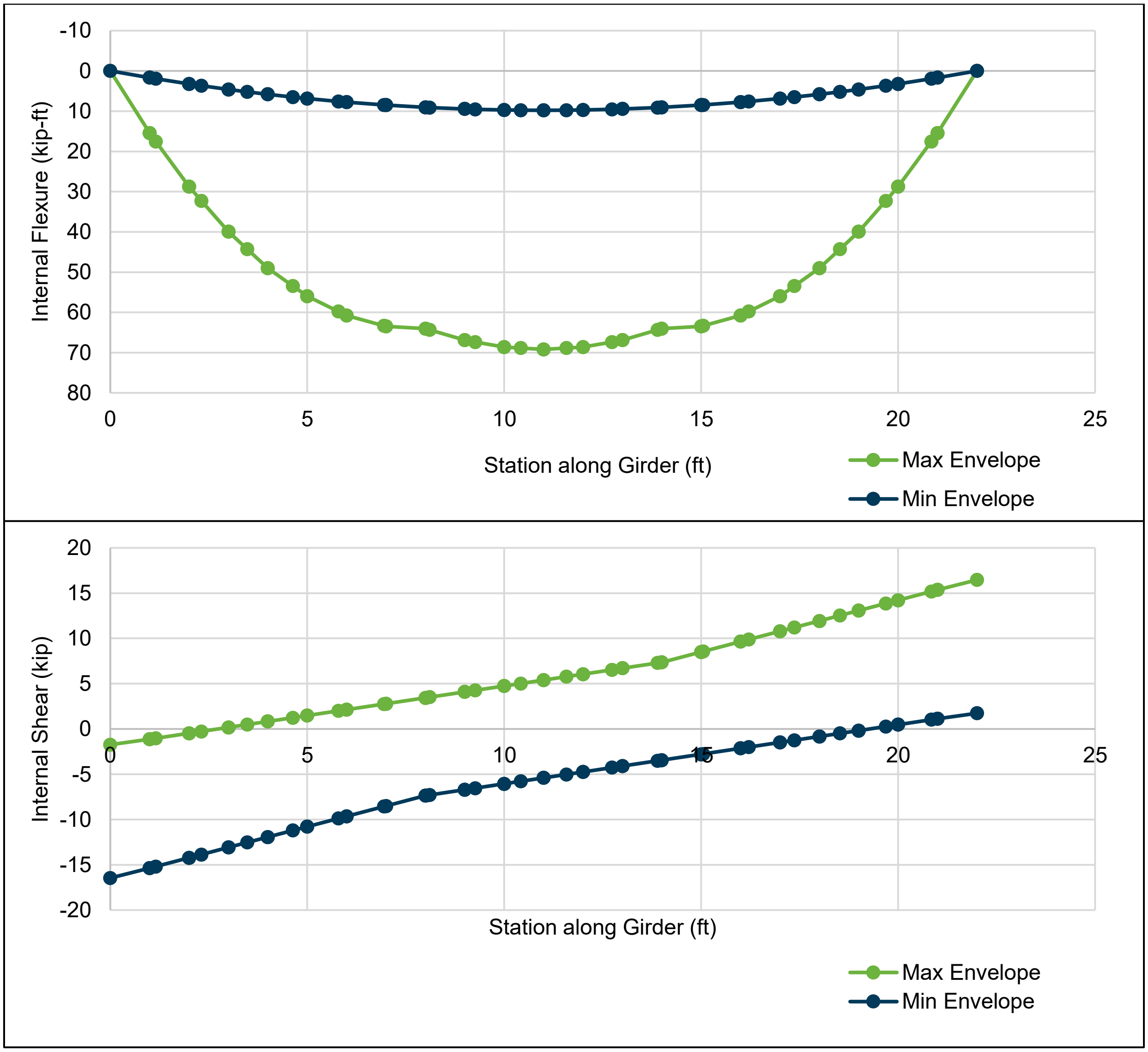

Figure 6.11 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Dry Creek Bridge Timber Girder – Existing Configuration

Analysis shows that the bridge's shear strength is adequate to resist the design loading. However, under the existing bridge configuration, the maximum flexural response of the girder exceeds the range of estimated flexural strengths according to varying timber grades (see Table 6.11).

Figure 6.19 shows the deflected shape of the existing timber girders under the Strength II load combination.

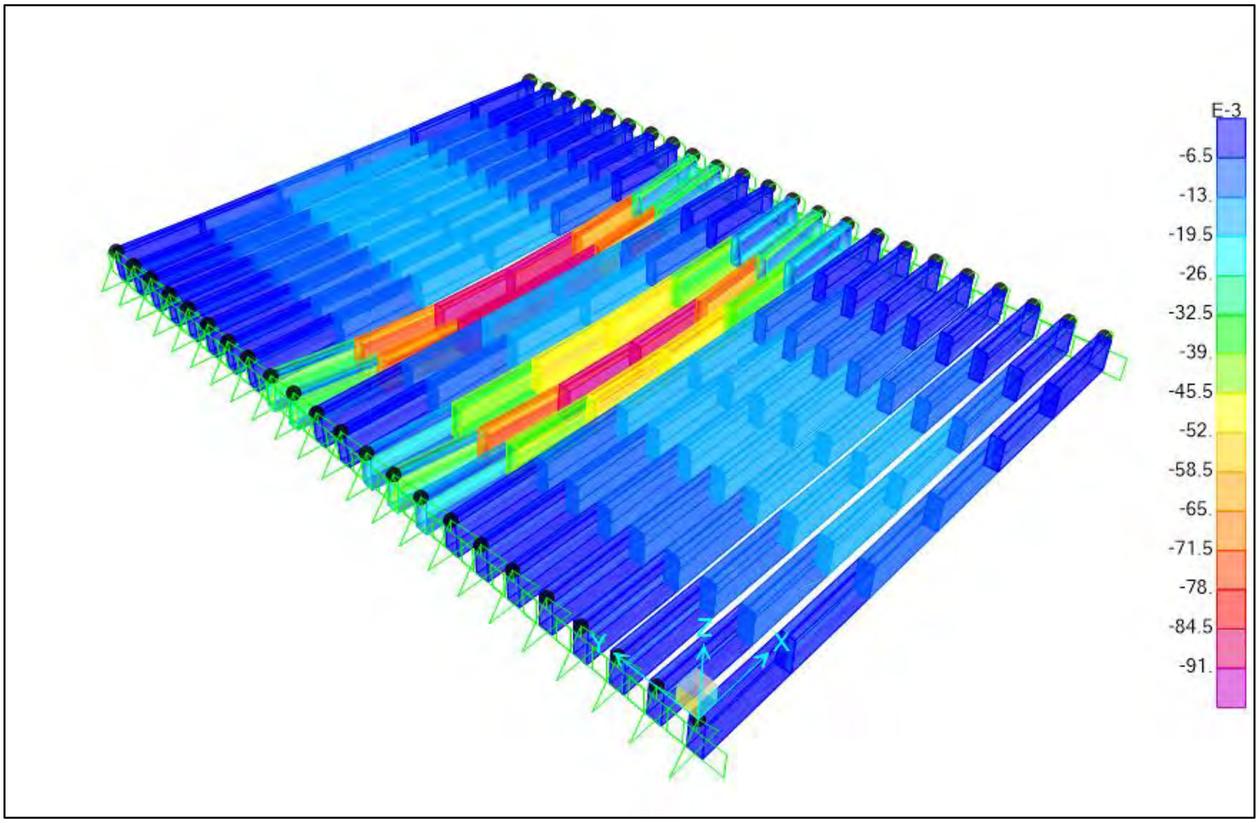

Figure 6.12 Strength II Load Combination – Dry Creek Timber Girders – Deflected Shape (10x Scale) – Vertical (Uz) Deformation Contours (ft)

6.3.4 STRENGTHENING STRUCTURE

Noting that the applied flexural load exceeds the assumed flexural resistance of the existing Dry Creek Bridge timber girders, a center support beam assembly, similar to that proposed for Fall Creek Bridge (see Section 6.3.4), is proposed at Dry Creek Bridge.

The material and strength properties of the center support beams are presented in Section 6.3.4 of this report.

Figure 6.20 shows the load response (flexure and shear) of the timber girders under the design loads after the center support beams are introduced.

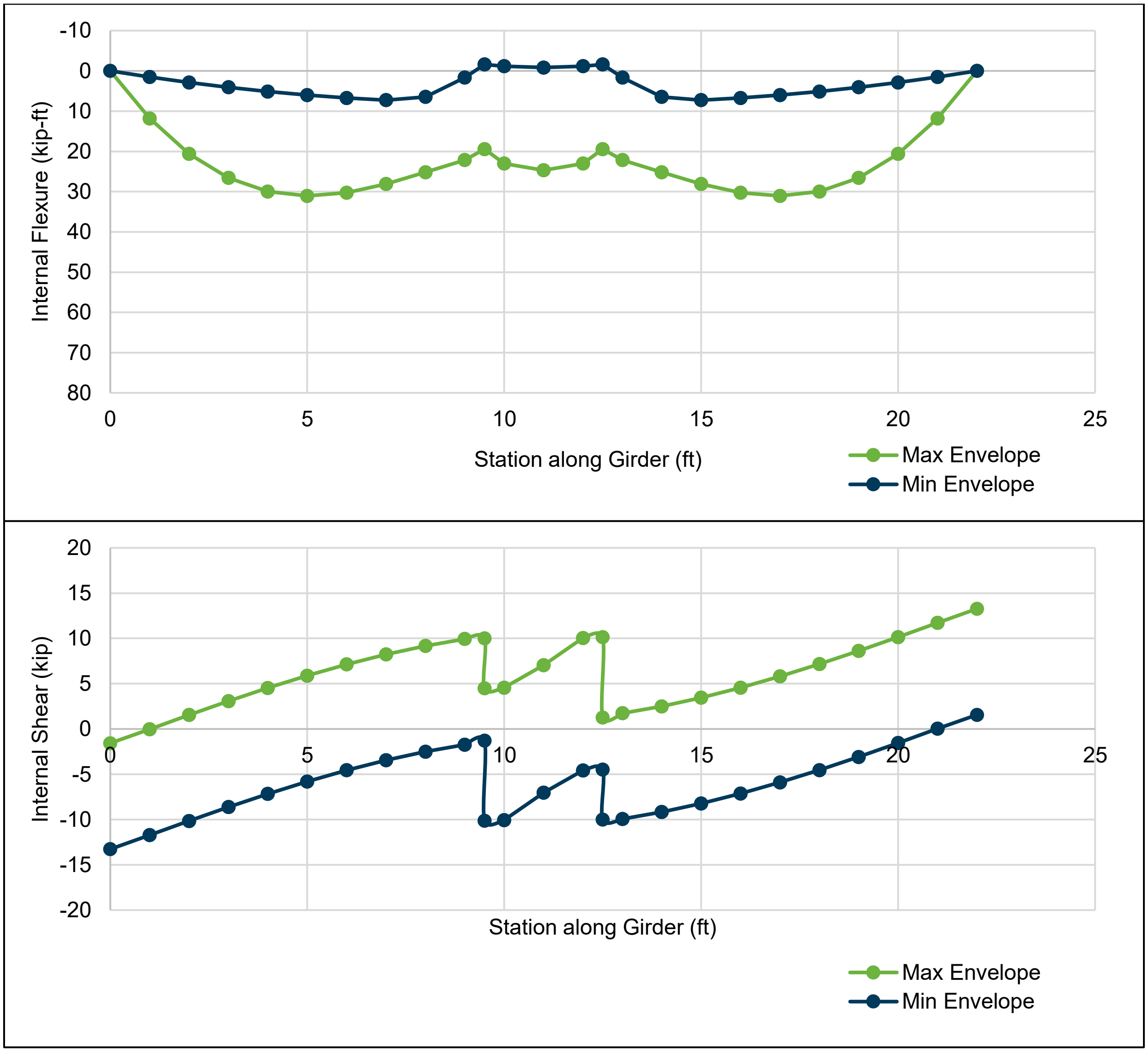

Figure 6.13 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Dry Creek Bridge Timber Girder – With Center Support Beams

The graph above shows that the maximum flexure in the timber girder is significantly reduced when the center support beams are introduced. At this supported configuration, the load response of the timber girder is found to be below the range of estimated flexural and shear strengths as shown in Table 6.11.

Similarly, the load response in the center support beam is found to be below the estimated flexural and shear resistance presented in Table 6.8 (see Section 6.3.4). Figure 6.21 presents the flexure and shear load response of the center support beam.

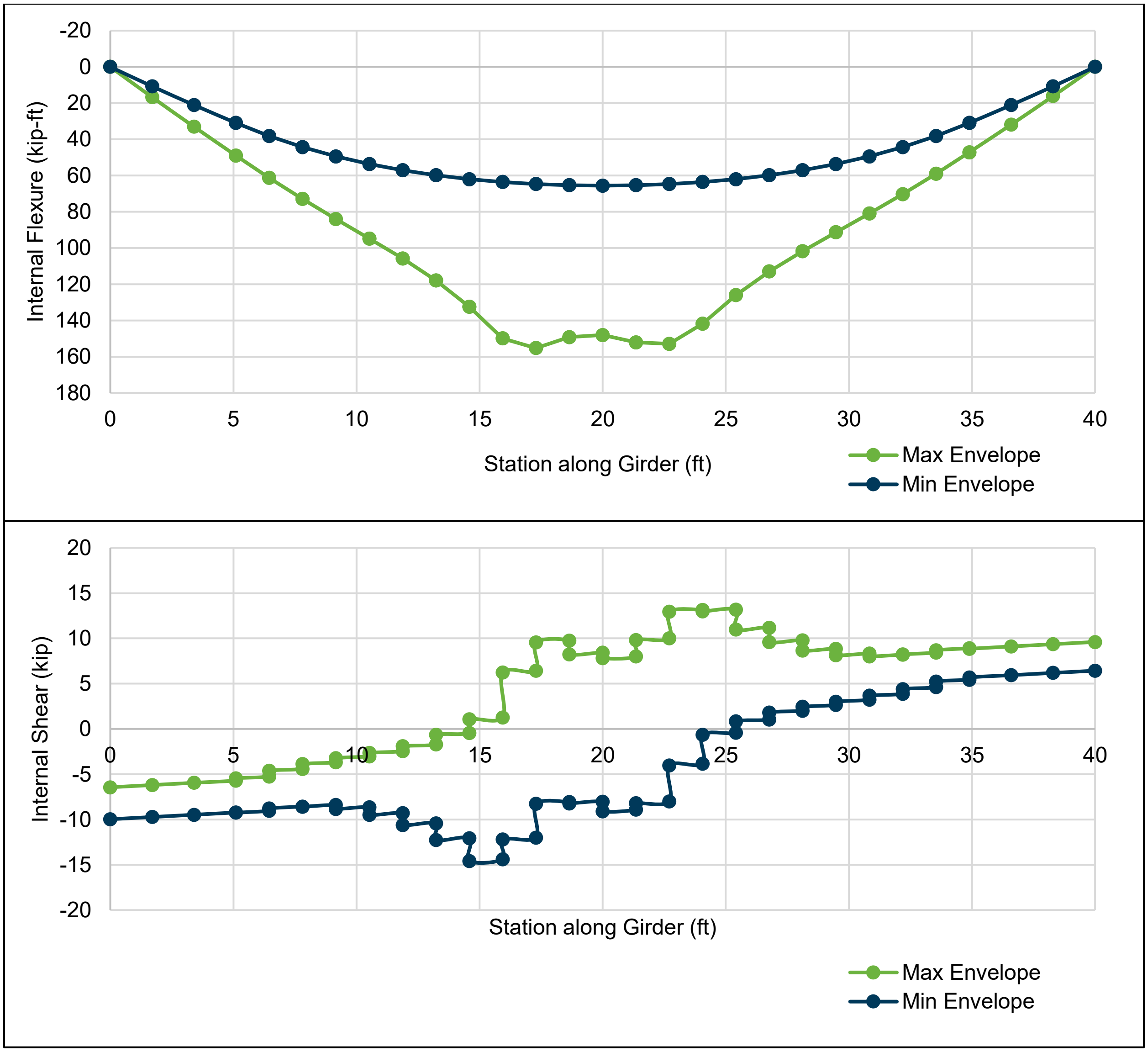

Figure 6.14 Strength II Load Combination – Internal Flexure (Top) and Internal Shear (Bottom) Response – Dry Creek Bridge Center Support Beam

Figure 6.22 shows the deflected shape of the existing timber girders with the center beam supports under the Strength II load combination.

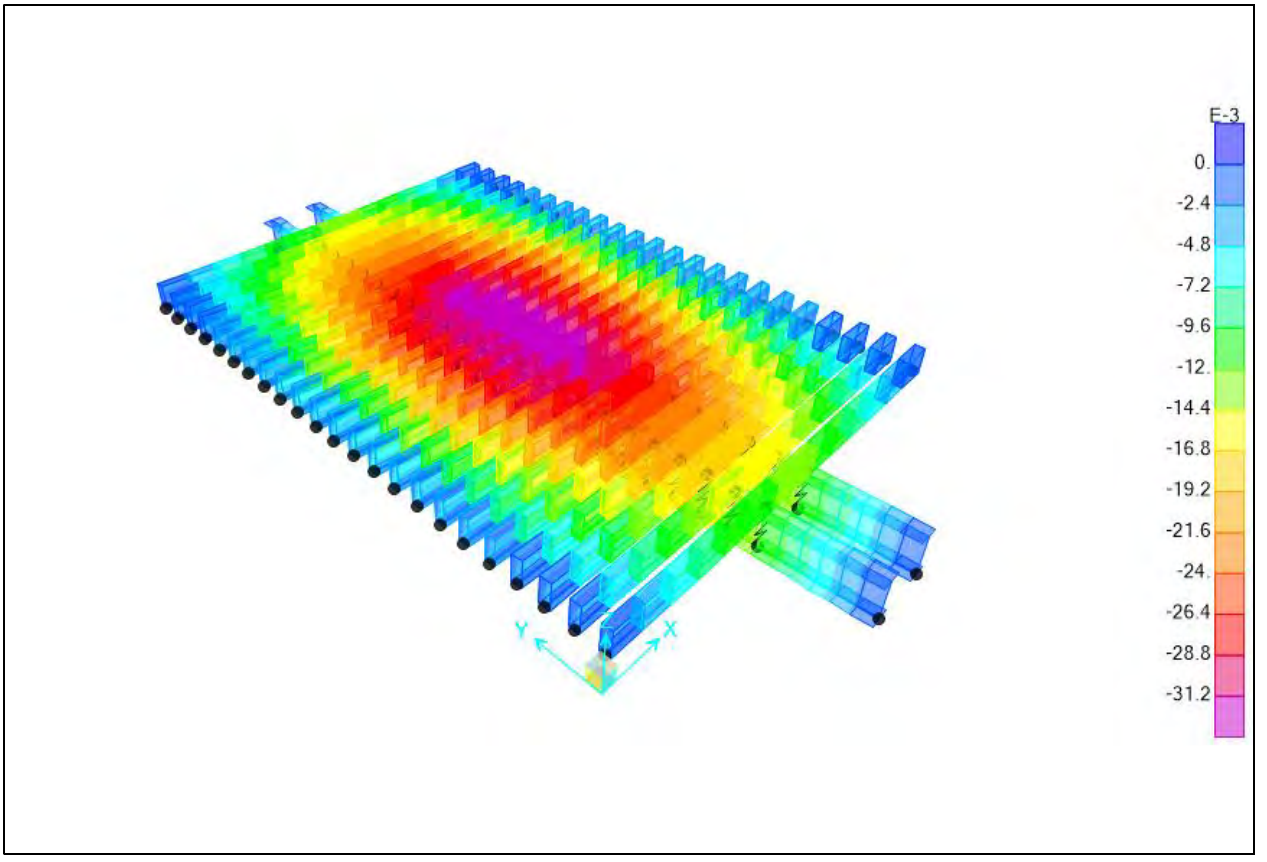

Figure 6.15 Strength II Load Combination – Dry Creek Timber Girders with Center Support Beams – Deflected Shape (10x Scale) – Vertical (Uz) Deformation Contours (ft)

The center support beams are found to decrease the overall maximum deformation of the timber girders in addition to reducing their maximum load response to the design loads.

6.3.5 CONNECTIONS

The center support beams at Dry Creek Bridge are designed with a similar approach to those at Fall Creek Bridge. For connection design details, see Section 6.3.5 of this report.

6.3.6 FOUNDATION

The foundation system at Dry Creek is similar in concept to Fall Creek, described in Section 6.3.6. Dry Creek is anticipated to have little or no flow during construction of the strengthening system.

6.3.7 SEQUENCING

At the time of this report, the following steps summarize the anticipated installation sequence at Dry Creek Bridge.

- Installation is planned for the July-October construction window, when historically the creek has little or no flow. No cofferdam or isolation is anticipated for installation.
- The channel bed will be prepared for installation of the foundation pads.

- Foundation structural pads will be placed and compacted and Lock-Block (Ultrablock) supports will be installed.
- Support girders will be dragged under the existing bridge, lifted, and jacked into position to achieve contact with the underside of the existing bridge deck.
- Connections and bracing will be installed prior to removing jacks.
- The bridge will be load tested to assess any settlement and the support system will be adjusted as required.
- The strengthening system WILL undergo visual inspection following any major storm/flood events for any signs of movement or settlement.
- Following Project completion, the strengthening structures will be deconstructed and removed and the channel bed at the foundation pads will be restored to natural condition.

7.0 POST DRAWDOWN IMPROVEMENTS

7.1 SCOTCH CREEK BOX CULVERT

The existing Scotch Creek crossing consists of a corrugated metal arch pipe with a span of approximately 10 ft and a rise of approximately 8 ft. The culvert is located along the northwest reach of the Iron Gate reservoir. The culvert is currently backwatered by Iron Gate Dam (based on photo's and site observations). Following drawdown, the creek channel will adjust downstream of the crossing cutting through the reservoir deposits. The Restoration Team will facilitate channel and floodplain adjustment through the reservoir deposits as part of the larger Project restoration efforts. The restored channel profile will extend below the existing culvert outlet creating potential barriers for volitional fish passage. The improved culvert profile will extend from the project work limits downstream approximately 75 ft at a slope of approximately 4%. The Restoration Team will transition the Scotch Creek crossing work to their restored channel. The Scotch Creek culvert design is shown on Project Drawing C5300.

National Marine Fisheries Service were consulted during the Value Engineering phase and have agreed with the design approach and culvert dimensions. The design generally matches the NOAA's Stream Simulation Design Method.

7.1.1 APPROACHES

The approaches to the new Scotch Creek crossing will match the existing road alignment. Existing road geometry does not meet AASHTO requirements (i.e. horizontal curvature). A portion of the roadway will be excavated to remove and replace the existing CSP arch culvert with the new concrete box culvert.

7.1.2 PRECAST CONCRETE BOX CULVERT

The new culvert for Scotch Creek will be a prefabricated concrete box structure. The structure will have a 15 ft span and a 12 ft rise to accommodate flood flows. The prefabricated bridge structure will be designed by suppliers (as per AASHTO LRFD to accommodate HL93 design vehicles and P-13 permit vehicles) and constructed as per the manufacturer's installation and erection plans.

7.1.3 SUBSTRUCTURE

The new box culvert will be placed as per the Project Drawings. Boreholes show competent material at Scotch Creek to support the anticipated bearing pressures induced by the new box culvert. Seismic analysis is pending supplier structure data, due to the low peak ground accelerations and observed soil conditions, seismic stability is not anticipated to be a limiting factor relative to foundation design.

Geotechnical considerations are described in Appendix F4.

7.1.4 CHANNEL RE-PROFILING

The profile of the existing channel is expected to adapt to the post dam removal flow conditions. The extent and timing of this adaption is difficult to predict. The Project Company has utilized historical photos, site survey data, Lidar, and borehole data to approximate a reasonable long-term profile based on both the existing geotechnical conditions and the historical pre-dam conditions.

The channel profile at the new Scotch Creek Culvert has been designed to pass the 1% PPE, facilitate fish passage and tie into the long-term channel restoration efforts which will occur downstream of the culvert within the Iron Gate reservoir. The Project restoration team will adaptively manage the delta deposits downstream of the Scotch Creek culvert to pass flow from the culvert to the confluence with the Klamath River. The new box culvert is designed to remain stable and avoid potential perching at the culvert outlet. The transition apron extends approximately 75 ft downstream of the new culvert. The upstream apron will extend approximately 100 ft upstream and tie into existing ground at the Project limits. The channel is expected to naturally adjust over time, and the apron tie-in points are designed as sacrificial keys which are intended to conform to any channel adjustments.

7.1.5 HYDRAULICS

The project team conducted hydraulic analyses at this crossing with the objectives of:

- Supporting the stream simulation design for fish passage
- Maintaining adequate flood flow and debris conveyance capacity to ensure long-term crossing stability

Hydraulic design at Scotch Creek Culvert is described in detail in Appendix F3.

7.1.6 SEQUENCING

At the time of this report, the following steps summarize the anticipated installation sequence at the Scotch Creek Box Culvert.

- Installation is planned for the July-October construction window, in the low flow months.
- A temporary shoo-fly detour road will be constructed to the north of the existing culvert location, which will temporarily re-route traffic from Copco Road around the work zone during installation of the new box culvert. Temporary bypass culverts will be installed to divert flow past the construction zone.
- The existing portion of Copco Road at the culvert location will be excavated as required to remove the existing CSP arch culvert.
- Subgrade will be prepared for installation of the new precast box culvert (as per supplier recommendations). The box culvert type will likely have a separate precast lid (or top half) to facilitate placement of streambed material within the box during installation.
- The box culvert will be backfilled, and the road will be constructed to match existing conditions. The temporary bypass will be closed and removed, and Copco road traffic will return to normal operation.
- The roughened channel will be constructed downstream of the new culvert, as far as the restoration tie-in point, approximately 75 ft downstream of the culvert.

- The restoration contractor will co-ordinate with the box culvert installation to ensure that the downstream delta deposits are removed to provide an effective channel to transport flow downstream to the Klamath confluence and avoid potential ponding or backwater following construction of the new box culvert.

7.2 CAMP CREEK BOX CULVERT

The existing Camp Creek crossing consists of a buried corrugated metal arch pipe with a span of about 6 ft and a rise of about 5 ft. The culvert is located along the northwest reach of the Iron Gate reservoir. The culvert is currently backwatered by Iron Gate Dam. A new concrete box culvert (15 ft span x 12 ft rise) will be installed to replace the existing culvert at Camp Creek.

Camp Creek Box culvert is identical to Scotch Creek culvert in terms of the design, construction, and sequencing strategy, see section 7.1 for reference.

The key differences between Camp Creek and Scotch Creek sites are related to the geotechnical conditions which are explained in detail in Appendix F4. In summary, the downstream delta deposits at Camp Creek have resulted in a soft layer of material which will require removal prior to commissioning the new box culvert, to avoid backwater and ponding. This work will be co-ordinated between the Project Company and the Project Restoration Team.

National Marine Fisheries Service were consulted during the Value Engineering phase and agreed with the design approach and culvert dimensions. The design generally matches the NOAA's Stream Simulation Design Method.

7.2.1 HYDRAULICS

The project team conducted hydraulic analyses at this crossing with the objectives of:

- Supporting the stream simulation design for fish passage
- Maintaining adequate flood flow and debris conveyance capacity to ensure long-term crossing stability

Hydraulic design at Camp Creek Culvert is described in detail in Appendix F3.

7.2.2 SEQUENCING

Camp Creek culvert construction will follow the same sequence as that for the Scotch Creek culvert, outlined in section 7.1.6.

7.3 FALL CREEK (AT DAGGETT ROAD) ARCH CULVERT

The Fall Creek crossing at Daggett Road is located just south of the connection with Copco Road, approximately 20 miles from the I5 interstate highway. The existing crossing includes a CMP arch pipe culvert (approximately 10 ft diameter) which passes flow through Daggett Road at the existing PacifiCorp site access gate. A photograph of the culvert is presented on Figure 7.1.

This site was not identified in the Project Agreement as a culvert requiring improvement however, following the existing structures assessment described in section 4.1 of this document, this crossing was flagged as potential replacement to meet overall KRRP objectives.

A multi-plate opened bottom arch with a bottom width of 24 ft will replace the existing culvert at Daggett Road. Design of the Daggett Road crossing has been coordinated with National Marine Fisheries Engineering and has a width that is approximately 1.5 times the active channel width.

Figure 7.1 Fall Creek Culvert at Daggett Road (Existing)

7.3.1 APPROACHES

The approaches to the new Fall Creek crossing will match the existing road alignment at Daggett Road. The existing road is owned by PacifiCorp and is used as a primary access route to the Copco No. 2 Dam site. A portion of the roadway will be excavated to remove and replace the existing CSP arch culvert with the new multi-plate arch culvert. The roadway will be reinstated following culvert installation to match the existing geometry and function.

Additional considerations at this site include the site access gate, buried utilities (power and water) and overhead power lines which are shown on the Project Drawings.

7.3.2 STRUCTURE

The new culvert for Fall Creek at Daggett Road will be a prefabricated multi-plate arch culvert. The structure will have a 24 ft span and approximately 11 ft rise to accommodate flood flows. The prefabricated structure will be designed by suppliers (AASHTO LRFD to accommodate HL93 design vehicles) and constructed as per the manufacturer’s installation and erection plans.

7.3.3 SUBSTRUCTURE

The new multi-plate arch culvert will be placed as per the Project Drawings. Geotechnical considerations are described in Appendix F4.

Geotechnical investigations conducted at the Arch Creek showed some variation in subsurface conditions. The bedrock elevation at the proposed culvert location is unknown and the Project Drawings show an assumed bedrock depth based on interpolation of the two boreholes drilled near the site. Closer proximity could not be achieved due to the location of buried utilities and traffic control requirements.

The arch culvert will be founded on precast or CIP strip footings. The detail provided in the Project Drawings shows a lined channel designed to resist scour and maintain long term channel stability.

If bedrock is encountered prior to excavating to the proposed profile depth, an adaptive detail (shown on the Project Drawings) will be employed which does not rely on scour protection. Staggered concrete lintels and/or roughness elements will be installed at regular intervals and anchored into exposed bedrock to form roughness elements on the channel bed and reduce velocities to promote aquatic organism passage.

7.3.4 FISH PASSAGE

The existing crossing on Fall Creek at Daggett Road is recognized by National Marine Fisheries as a fish passage barrier for both juvenile and adult salmonids. The crossing consists of a 60-foot-long, 10 ft diameter corrugated metal pipe that slopes at 4.3%. Flows through the crossing are supercritical. The culvert outlet is perched approximately 1.5 ft above typical late spring, summer, and fall water levels. The proposed crossing will mimic flow conditions upstream and is designed using the stream simulation method.

The proposed streambed through the multi-plate arch will slope at approximately 3.5%. The streambed will be constructed using engineered streambed material (ESM) placed between boulder buttresses. The ESM is designed using California Department of Fish and Wildlife's ESM sizing methodology. Boulder buttresses will be placed to stabilize the ESM and serve as grade control. Large boulders that project 1 to 1.5 ft above the streambed will create roughness and provide resting areas for aquatic organisms. The roughness boulders also improve passage by reducing average channel velocities and dissipate energy to increase low flow depths and help stabilize the constructed streambed.

7.3.5 HYDRAULICS

The project team has conducted hydraulic analyses at this crossing with the objectives of:

- Supporting the stream simulation design for fish passage
- Maintaining adequate flood flow and debris conveyance capacity to ensure long-term crossing stability

Hydraulic design at Fall Creek (at Daggett Road) Culvert is described in detail in Appendix F3.

7.3.6 SEQUENCING

At the time of this report, the following steps summarize the anticipated installation sequence at Fall Creek at Daggett Road Arch Culvert.

- Installation is planned for the July-October construction window, in the low flow months.
- A temporary shoo-fly detour road will be constructed to the east of the existing culvert location along Daggett Road, which will temporarily re-route traffic from Daggett Road around the work zone during installation of the new arch culvert. Temporary bypass culverts will be installed to divert flow past the construction zone. The Project Company will determine security requirements (i.e. security gates) at the shoofly road pending the overall Project Traffic Management Plan and PacifiCorp's site access requirements.
- An upstream cofferdam (i.e. concrete barrier and pond liner or similar) will be installed to divert flow while the Project Company adjust the channel profile upstream of the existing culvert.

- The existing portion of Daggett Road at the culvert location will be excavated as required to remove the existing CSP arch culvert. Existing services will be protected, moved, or rerouted during construction.
- A bypass culvert will be installed to divert Fall Creek around the construction zone.
- The roughened channel will be constructed along the new culvert profile as shown on the Project Drawings.
- Subgrade will be prepared for installation of the new multi-plate arch culvert (as per the supplier recommendations). Typically, prefabricated arch structures come with precast footings or permanent formwork with reinforcement pre-installed. Footings will be placed/poured, and the arch superstructure will be installed.
- The arch culvert will be backfilled, and the road will be constructed to match the existing conditions. The temporary bypass will be closed and removed, and Daggett Road traffic will return to normal.
- Roughness elements will be installed at the downstream end of the new culvert following removal of the temporary shoofly detour, as per the Project Drawings.

7.4 TIMBER BRIDGE DEMOLITION

The timber bridge crosses the Klamath River to the west of the J.C. Boyle intake and is located adjacent to the wood stave penstock. The bridge is 100’ long and 18’ wide, comprised of a timber deck on four longitudinal steel I-Girders (W36 x 194). The girders are diagonally braced against lateral movement. The bridge is supported at each end by a steel cap beam on four H piles, driven to an elevation of “77’ or lower” according to bridge as-built drawings.

Each abutment provides a concrete back-wall which acts like an end diaphragm bearing against the abutment backfill and supported by a steel seating plate, welded to the H-piles.

The demolition sequence for this bridge was not evaluated as part of this design report and will ultimately be defined by the Project Company.

7.5 POST DRAWDOWN MONITORING

This section describes the monitoring that will take place after drawdown occurs, to ensure the integrity of existing structures. KRRC are the Project designee for monitoring at sites which are outside of the Project Company’s direct construction footprint. KRRC will monitor the structures outlined in the following sections, during drawdown and two-years after drawdown to evaluate the post-dam performance of the structures. These sites are listed in Table 1.1 of this document.

The specific details (i.e. frequency, extent) of the monitoring plan will be developed by KRRC. The following sections are provided as a general overview of the Project monitoring objectives for Project roads, bridges, and culverts and will ultimately be decided upon by KRRC, as the Project monitoring plan is developed.

7.5.1 ROAD MONITORING

Roads adjacent to the reservoirs will be monitored during drawdown and may require repairs or improvements on an as-needed basis to maintain the current level of service. The extent and timing of these repairs will be co-ordinated between the Project Company and respective county jurisdictions (Klamath Co in Oregon and Siskiyou Co in California), based on the MOU.

7.5.2 BRIDGE MONITORING

The following bridges shall be monitored post-drawdown for a 2-year period for potential erosion or scour at the bridge embankments and intermediate piers.

- Copco Road Bridge
- Jenny Creek Bridge
- Spencer Bridge (Green Springs Highway)

The bridges in this section have been identified as having foundations which are currently near the existing reservoirs level which may be impacted by post-drawdown conditions. Based on the current conditions, and expected post-drawdown flow paths near these structures, the bridges should be monitored for any detrimental effects.

7.5.3 CULVERT MONITORING

The following culverts shall be monitored by KRRC following draw-down for a 2-year period for drawdown related sediment/debris accumulation, erosion and/or scour. These sites include:

- Beaver Creek (East Fork and West Fork)
- Keno Access Road Culverts (East and West)
- Patricia Avenue Culverts (East Fork and West Fork)
- Raymond Gulch
- Topsy Grade Road Culvert (Identified as TopsyGradeRoad-72+00 on Drawing C6710)

The Project Team did not foresee any fundamental changes required at these structures based on the current conditions and expected post-drawdown flow paths through these structures. Monitoring may include flow assessments, debris conveyance assessments, identification of channel adjustment or signs of incision that may migrate upstream and destabilize the crossing. If erosion or sedimentation are shown to negatively affect the performance of these structures, appropriate repairs may include localized riprap protection, removal of sediments/debris or alternative erosion protection measures. Culvert replacement will be required if retrofitting is deemed inadequate.

The Project Company’s assessment of habitat suitability upstream of the culverts and volitional fish passage at these culverts is summarized in Section 4.0 of this Appendix. If stabilization of the West Beaver Creek crossing is necessary following drawdown, the culvert may need to be replaced or retrofit to meet fish passage criteria.

The culvert crossing on Copco Road over the West Fork of Beaver Creek is located on the north shore of Copco Lake, and is not located on a construction access route. The Project Company is not proposing any road or culvert improvements for construction access purposes at this location. If signs of destabilization due to drawdown related flows are observed during the monitoring period and improvements become required, the crossing will be required to allow fish passage, as the tributary has a historic fish presence. The proposed restoration activities at the West Fork of Beaver Creek extend from the confluence of the Klamath River and Beaver Creek to RM 1.5 of the Main Stem of Beaver Creek and cease at a natural barrier downstream of the culvert crossing. The restoration activities proposed are not considered a trigger event.

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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 1.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 17+250

Worn paved hot-mix asphalt. Peeling of surface material with exposed alligator cracks beneath surface material. No reflective cracks protrude into surface material.

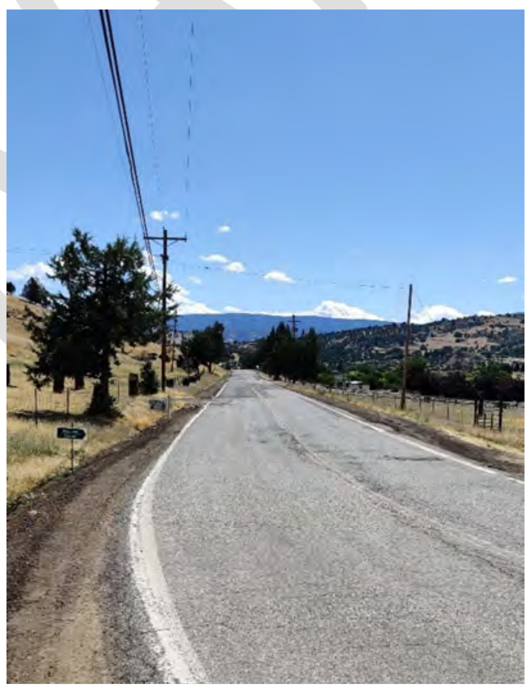
Photo No. 2.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 18+750

Hot-mix asphalt road. Low hanging powerlines going across the road.



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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 3.	
Date: 06/26/19	
Description: Copco Road Approx. Sta. 19+600 Hot-mix asphalt concrete. Pothole located on edge of road. Raveling of surface material with exposed alligator cracks beneath surface material. No reflective cracks protrude into surface material.	

Photo No. 4.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 30+300 Hot-mix asphalt concrete road with small shoulder.	

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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 5.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 31+600 Hot-mix asphalt concrete road. Cold patch covering faulting between two different pavements. Settlement and alligator cracking found at edge of old pavement.	

Photo No. 6.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 31+800 Hot-mix asphalt concrete road. Edge of road with significant spalling.	

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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 7.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 38+500 Hot-mix asphalt concrete road. Disjointed shoulder.	

Photo No. 8.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 385+00 Hot-mix asphalt concrete road. Depth of disjointed shoulder approximately 12” to 18”.	

COPCO ROAD - PHOTOGRAPHIC RECORD

KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 9.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 49+500 Hot-mix asphalt concrete road. Low handing power line going across the road.	

Photo No. 10.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 49+500 Hot-mix asphalt concrete road with alligator cracks and raveling. Numerous asphalt patches found along road.	

COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 11.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 61+800 Hot-mix asphalt concrete road. Significant spalling in pull off area. Alligator cracking and asphalt patches found along road.	

Photo No. 12.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 62+900 Transition of Hot-mix asphalt road to aggregate base road.	

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COPCO ROAD - PHOTOGRAPHIC RECORD

KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 13.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 62+900 Hot-mix asphalt concrete road. Significant spalling at edge of road. Slippage cracking found in portion of asphalt at edge of road.	

Photo No. 14.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 62+900 Hot-mix asphalt concrete road. Significant spalling at edge of road. Crushed end of CMP culvert found at edge of road.	

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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 15.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 66+400

Hot-mix asphalt road with a sharp turn.

Photo No. 16.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 66+400

Paved asphalt road with a sharp turn.

Knight Piésold CONSULTING

COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 17.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 67+700

Hot-mix asphalt and a portion of Copco Road with gravel surfacing.

Photo No. 18.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 82+000

Hot-mix asphalt with an auxiliary dirt road behind Jenny Creek bridge.

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COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 19.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 90+000 Hot-mix asphalt concrete road. Significant spalling at edge of road. Asphalt patches and alligator cracking throughout road.	

Photo No. 20.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 90+000 Rock fall found on side slope adjacent to Hot-mix asphalt concrete road.	

Knight Piésold CONSULTING

COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 21.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 98+600

Hot-mix asphalt concrete road near Fall Creek bridge. Pothole on edge of bridge.

Photo No. 22.

Date: 06/26/19

Description:

Copco Road
Approx. Sta. 98+600

Hot-mix asphalt concrete road with alligator cracks and raveling.

Knight Piésold CONSULTING

COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 23.	
Date: 06/26/19	
Description: Copco Road Approx. Sta. 117+100 Portion of Copco Road with aggregate base surfacing. Curve is a sharp turn.	

Photo No. 24.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 101+600 Aggregate base road with overhead powerlines going across the road.	

Knight Piésold CONSULTING

COPCO ROAD - PHOTOGRAPHIC RECORD
KLAMATH RIVER RENEWAL PROJECT
100% DESIGN REPORT

Photo No. 25.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 108+300 Aggregate base road with low overhead powerlines going across the road.	

Photo No. 26.	
Date: 07/17/19	
Description: Copco Road Approx. Sta. 109+000 Aggregate base road with shallow buried CMP culvert.	

Prepared By:

PREPARED FOR: KP
PROJECT: KRRP Alternative Access Routes
FIGURE F2-1: Alternative access route overview map

F2.2 - 1 of 2

N

0 2 4 Miles

Coordinate System: NAD 1983 StatePlane California I FIPS 0401 Feet
Projection: Lambert Conformal Conic
Datum: North American 1983

Date: 8/3/2020

Prepared By:

GeoServ Inc Engineering Geologists Rock - Soil - Water

PREPARED FOR: KP

PROJECT: KRRP Alternative Access Routes

FIGURE F2-2: Ager and Ager Beswick Roads route map

F2.2 - 2 of 2

Date: 8/3/2020

APPENDIX F3
ROADS, BRIDGES, AND CULVERTS – HYDROTECHNICAL DESIGN REPORT

TABLE OF CONTENTS

1.0 SCOPE

The Roads, Bridges, and Culverts Geotechnical Design Report Appendix contains an overview of the hydrotechnical design recommendations for construction access and permanent access infrastructure for the Klamath River Renewal Project (KRRP) (Table 1.1).

This document provides a comprehensive overview of the hydrotechnical design development for the Roads, Bridges and Culverts components of the KRRP.

The Project Drawings (100% Design Drawing Package) and Appendix F1 of the 100% Design Report should be reviewed in conjunction with this document.

Table 1.1 summarizes the sites included in this document.

Table 1.1 Hydrotechnical Design Scope Summary

Site	Treatment
Scotch Creek Culvert	Concrete Box Culvert
Camp Creek Culvert	Concrete Box Culvert
Fall Creek at Daggett Road	Bottomless Arch Culvert

1.1 SCOTCH CREEK AND CAMP CREEK

The Scotch, Camp and Fall Creek crossing improvements are designed to convey the 1% Annual Probable Flood (APF also referred to as the 100-year flood) and provide volitional fish passage. The stream channels downstream of the crossings are influenced by Iron Gate Reservoir water surface elevations. The crossings are designed to mimic upstream channel conditions and generally meet National Marine Fisheries Service (NMFS) stream simulation methodology. Crossing widths are approximately equal to the active channel widths or larger and the slopes of the reprofiled crossings are near bed slopes identified upstream of the crossings.

The Restoration Contractor will restore the stream channels downstream of Scotch Creek and Camp Creek, interfacing at the limits noted on the Project Drawings. Culvert designs for the Scotch and Camp road crossings extend about 70 ft and 80 ft, respectively. Geotechnical investigations suggest the transition locations between the crossing construction and the dam restoration occur in deltas formed from fine sediment and organic deposits. The road crossing will be installed prior to dam removal and downstream channel restoration. During this interim period, Scotch and Camp road crossing channels and culverts are susceptible to incision. Sacrificial toes will be installed to prevent incision from progressing upstream through the crossings. The sacrificial toes will be constructed at the downstream end of the crossing improvement channel construction and at the downstream ends of the Scotch and Camp Creek box culverts. The toe structures are comprised of erosion protection material that will partially mobilize over time and adjust naturally to a permanent stable condition to protect the newly constructed channels and crossings.

1.2 FALL CREEK AT DAGGETT ROAD

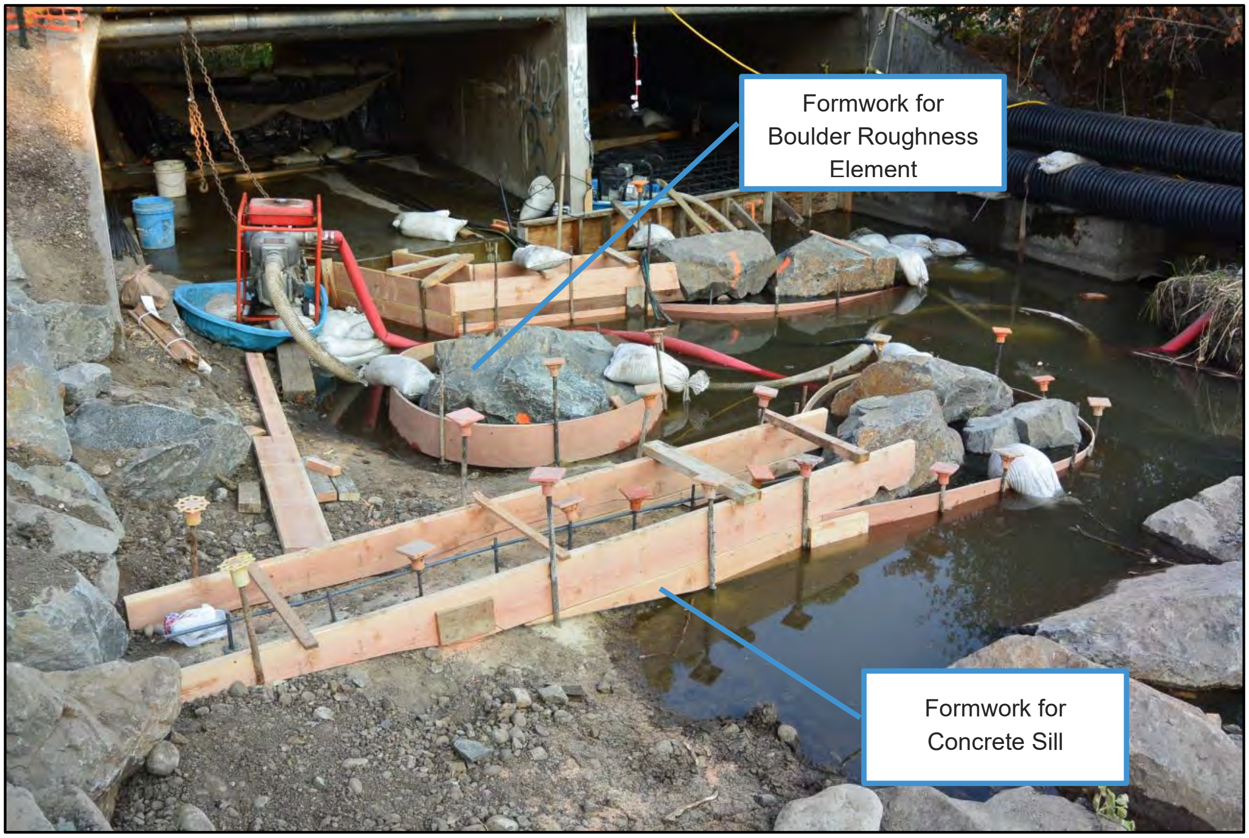
Fall Creek at Daggett Road crossing is located near the tailwater influence of Iron Gate Reservoir. The channel upstream of the crossing is largely confined by a basalt outcrop along the east side and a hillslope and road cut along the west side. Large colluvium from both sides has created reaches with step-pools and cascades upstream of the crossing. The existing 10-ft diameter corrugated metal pipe crossing influences sediment and water flow between the reaches up and downstream from the crossing. The stream channel downstream of Fall Creek is backwatered by the reservoir. Periodic drawdown events appear to mobilize fine sediments deposited because of the reservoir's water level control. Photograph 1.1 shows the creek when the reservoir was drawn down. The turbulence shown in the photograph suggests the channel downstream of the Fall Creek crossing largely consists of a steeply sloped rapid. The design assumes this condition will be present following drawdown.

The existing Daggett Road Crossing will be replaced with a 24-ft-wide open bottom arch. Engineered streambed material will be placed inside the arch and will extend approximately 30 ft downstream of the outlet to stabilize the streambed. Rock buttresses will be placed at grade to provide internal grade control and structure to the channel. Geotechnical investigations at the site are interpolated and there is a possibility that excavation and reprofiling may expose shallow bedrock. The configuration of the exposed bedrock is unknown and may create hydraulic conditions that inhibit volitional fish passage. If shallow bedrock exposures inhibit construction and installation of the engineered streambed material and boulder buttresses, concrete sills and boulder roughness elements will be installed and anchored to the shallow bedrock profile. These features will be constructed to ensure the constructed channel mimics upstream hydraulic conditions and provides volitional fish passage. The sills will be stepped at less than 1 ft and sloped to temporarily trap bed material. The bedrock will be drilled, and rebar dowels will be installed and fixed in place with epoxy. Cast-in-place concrete sills will be formed and secured to the dowels. Large rock roughness elements will be constructed in a similar manner. Rock boulders will be drilled, and rebar dowels installed. The rock roughness elements and bedrock will be tied and secured with a cast-in-place concrete pedestal. An example of this type of construction is shown in Photograph 1.2.

Currently, the channel width downstream of the Daggett Road crossing is about twice the channel width upstream of the crossing. The abrupt transition creates hydraulic and sediment transport issues. The proposed design will reduce the discontinuity by reconstructing a portion of the west bank downstream of the crossing and enhancing an existing island with large wood. These actions will add to channel complexity, create refuge areas for aquatic organisms, and help to transition flows from the crossing to the wider downstream reach.

The proposed treatments used to provide fish passage and transition flows and direct flows are commonly used. Examples of embedded crossings with engineered streambed material are shown in Photograph 1.3 and Photograph 1.4. Photograph 1.5 shows an example of a boulder buttress under construction.

Photograph 1.1 Fall Creek Stream Channel During a Drawdown Event



Photograph 1.2 Example of Cast-in-Place Rock and Concrete Sill

Photograph 1.3 Outlet at Elder Creek Roughened Channel Example

Photograph 1.4 Sunken Box Culvert with Engineered Streambed Material Example

Photograph 1.5 Larson Creek Example of Boulder Buttress Construction

2.0 DESIGN METHODOLOGY

This section briefly describes the methods used to design the Scotch, Camp, and Fall Creek crossings to convey flood flows including the 1% APF (100-year flood) and provide volitional fish passage conditions. The crossings at Scotch, Camp, Fall Creeks generally comply with NOAA Stream Simulation Design Methodology (NOAA 2019). The crossings at Scotch and Camp Creeks consist of a sunken box culvert with a span of 15 ft and a rise of 12 ft. The culverts will be embedded into the streambed by about 2.5 to 3 ft. Engineered Streambed Material will be placed within the crossings. Fall Creek will consist of an open bottom arch with a 24-ft span. Engineered streambed material will be placed in the channel bed within the crossing.

Water surface profiles, depths, and velocities for Camp and Scotch Creeks are computed using the steady-state, one-dimensional algorithms in HEC-RAS (2019). Simulations use the mixed flow condition, which allows computations of subcritical, critical, and supercritical flow conditions. Downstream channel conditions will be constructed by the Restoration Contractor. Uncertainties regarding the downstream boundary conditions preclude the use of SRH-2D, a two-dimensional hydrodynamic model.

Hydraulic characteristics for Fall Creek are calculated using SRH-2D (Lai 2008: Aquaveo 2020).

Manning’s n values for the streambed are estimated using Bathurst (1985) and are based on D84 and hydraulic radius. These values are compared with values shown in Yochum et al. (2014). Manning’s n is based on NHC’s experience with shallow overland flow along steep floodplains.

Engineered Streambed Material calculations are developed using methods prescribed in Love and Bates (2009). These calculations use the U.S. Army Corps of Engineers method for calculating rock slope protection (USACE 1994) and apply ratios to the D30 to develop a broader gradation that seals the channel bed and promotes surface flow.

3.0 ANALYSIS AND RESULTS

This section contains analyses and model results for the 1% APF design peak flow (i.e. 1% design flow). Analyses include hydraulic characterization at the 1% design flow and engineered streambed material calculations. Table 3.1 lists the 1% design flows for Scotch, Camp and Fall Creeks.

Table 3.1 Camp, Scotch, and Fall Creek Design Flows

Site	1% APF (ft^3/s)
Scotch Creek Culvert	1,070
Camp Creek Culvert	1,170
Fall Creek at Daggett Road	750

3.1 SCOTCH CREEK ANALYSIS AND RESULTS

The HEC-RAS model results incorporate the design topography and 15 ft by 12 ft embedded box culvert. The crossing width has an active channel width that mimics the upstream active channel width. The downstream active channel width is influenced by Iron Gate Reservoir and is not indicative of the post dam removal channel width. The Restoration Contractor will transition the channel from the end of the channel work related to crossing following the installation of the crossing, as shown on the Project Drawings. The model extends about 70 ft downstream of the culvert outlet and about 40 ft upstream of the inlet to the extent of the proposed channel construction. The design profile of the channel slopes at 4%. Estimated Manning's n values are shown below in Table 3.2. Figure 3.1 shows cross-section locations superimposed on the design topography and proposed culvert. The culvert inlet creates a subcritical condition upstream crossing. Flows through the crossing accelerate and become supercritical to the downstream boundary where the flow transitions to subcritical. The transition at the boundary is due to the boundary condition computation, which is computed using uniform flow with a slope of 3%. A weak hydraulic jump is likely to form at this location during extreme flood events, such as the 1% design flow. The Project Company understands the downstream channel profile will be 3% or shallower, based on ongoing co-ordination with the Project Restoration team and their assessment of assumed post-drawdown conditions. The water surface profile is shown in Figure 3.2. The design water surface elevation is about 0.7 ft lower than the culvert soffit. Hydraulic Characteristics are shown in Table 3.3.

Table 3.2 Scotch Creek Manning's n for 1% Design Flow

Location	Manning's n
Channel Bed	0.055
Overbank	0.08

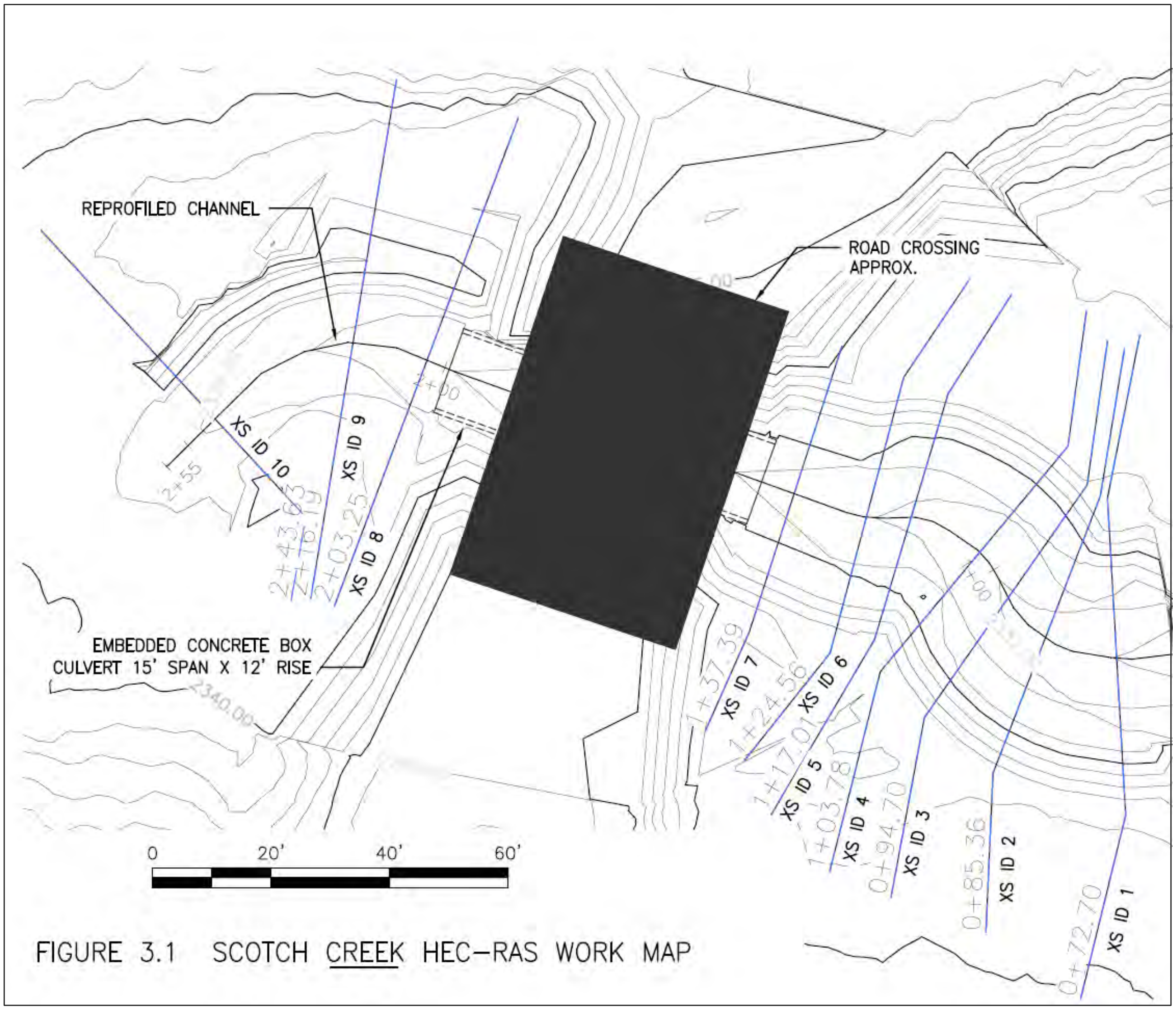


FIGURE 3.1 SCOTCH CREEK HEC–RAS WORK MAP

Figure 3.1 Scotch Creek HEC-RAS Work Map

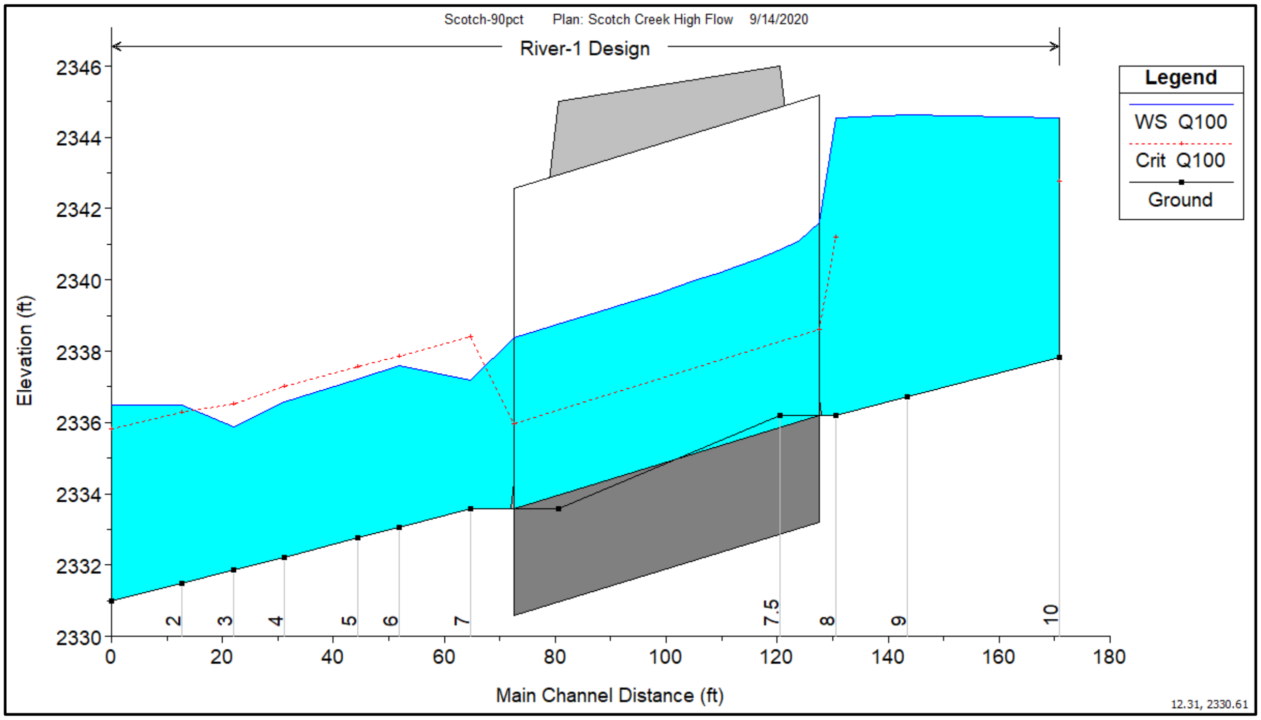


Figure 3.2 Scotch Creek HEC-RAS Longitudinal Profile

Table 3.3 Scotch Creek HEC-RAS Model Results

Cross Section ID	Min. Channel Elevation (ft)	Water Surface Elevation (ft)	Critical Water Surface (ft)	Energy Grade line Elevation (ft)	Energy Grade line Slope (ft/ft)	Velocity (ft/s)	Froude No.
10	2,337.82	2,344.53	2,342.78	2,345.44	0.008444	8.11	0.57
9	2,336.72	2,344.62		2,345.18	0.004272	6.47	0.42
8	2,336.2	2,344.56	2,341.19	2,345.12	0.003894	6.44	0.41
7	2,333.57	2,337.18	2,338.41	2,341.17	0.091441	16.83	1.68
6	2,333.06	2,337.58	2,337.85	2,339.53	0.033094	12.03	1.06
5	2,332.75	2,337.22	2,337.55	2,339.26	0.035164	12.25	1.09
4	2,332.22	2,336.59	2,337.01	2,338.76	0.038768	12.7	1.14
3	2,331.86	2,335.89	2,336.51	2,338.34	0.048003	13.29	1.25
2	2,331.49	2,336.48	2,336.3	2,337.94	0.021786	10.5	0.87
1	2,330.98	2,336.48	2,335.82	2,337.62	0.015019	9.36	0.74

Table 3.4 Scotch Creek Engineered Streambed Material Calculations

CDFW ESM		
Percent of mix.	Rock Size, ft	
	min	max
16%	3.1	4.0
34%	1.2	3.0
34%	0.07	1.1
9%	0.013	0.06
7%	SAND/SILT	

Hydraulic Characteristics			
q_f =	71.3	$ft^3/s/ft$	Unit Discharge in Main Channel
$Q_{100channel}$ =	1070	ft^3/s	1pct AEP (00-year peak flow)
$W_{channel}$ =	15	ft	Main Channel Width
So =	0.04	ft/ft	Channel Slope ft/ft

USACE (1991) Channel Bed Rock Sizing			
So =	0.04	ft/ft	Channel Slope
$q_{100channel}$ =	71.3	$ft^3/s/ft$	Unit Discharge with Concentration Factor
g =	32.2		Gravitational Acceleration
sf =	1		
D30 =	2.05	ft	

Parameters used to size CDFG ESM		
So =	0.04	ft/ft
q =	71.3	ft2/s
g =	32.2	lbm*ft/s2
n_{sed}=	0.40	-
$D_{30\text{-CORPS}}$ =	2.05	ft

CDFG Engineered Bed Material Size				
$D_{8\text{-ESM}}$ =	0.013	ft	0.2	in
$D_{16\text{-ESM}}$ =	0.07	ft	0.9	in
$D_{50\text{-ESM}}$ =	1.2	ft	14.8	in
$D_{84\text{-ESM}}$ =	3.1	ft	36.9	in
$D_{100\text{-ESM}}$ (calc) =	7.69	ft	92.2	in
$D_{100\text{-ESM}}$ (use) =	4	ft	48.0	in

3.2 CAMP CREEK ANALYSIS AND RESULTS

Because of the similar design flows and setting, the Camp Creek approach closely matches the Scotch Creek design approach. The HEC-RAS model results incorporate the design topography and 15 ft by 12 ft embedded box culvert. The crossing width has an active channel width that mimics the upstream active channel width. The downstream active channel width is influenced by Iron Gate Reservoir and is not indicative of the post dam removal channel width. The model extends about 80 ft downstream of the culvert outlet and about 95 ft upstream of the inlet. The design profile slopes at 4%. Estimated Manning's n values are shown in Table 3.5. Figure 3.3 shows cross-section locations superimposed on the design topography and proposed culvert. The culvert inlet creates a subcritical condition upstream crossing. Flows through the crossing accelerate and become supercritical to the downstream boundary where the flow transitions to subcritical. The transition at the boundary is due to the boundary condition computation, which is computed using uniform flow with a slope of 3%. A weak hydraulic jump is likely to form at this location during extreme flood events, such as the 1% design flow. The Design Team understands the downstream profile will be 3% or shallower. The water surface profile is shown in Figure 3.4. The design water surface elevation is about 0.3 ft lower than the culvert soffit. Hydraulic Characteristics are shown in Table 3.6.

Table 3.5 Camp Creek Manning's n for 1% Design Flow

Location	Manning's n
Channel Bed	0.055
Overbank	0.08

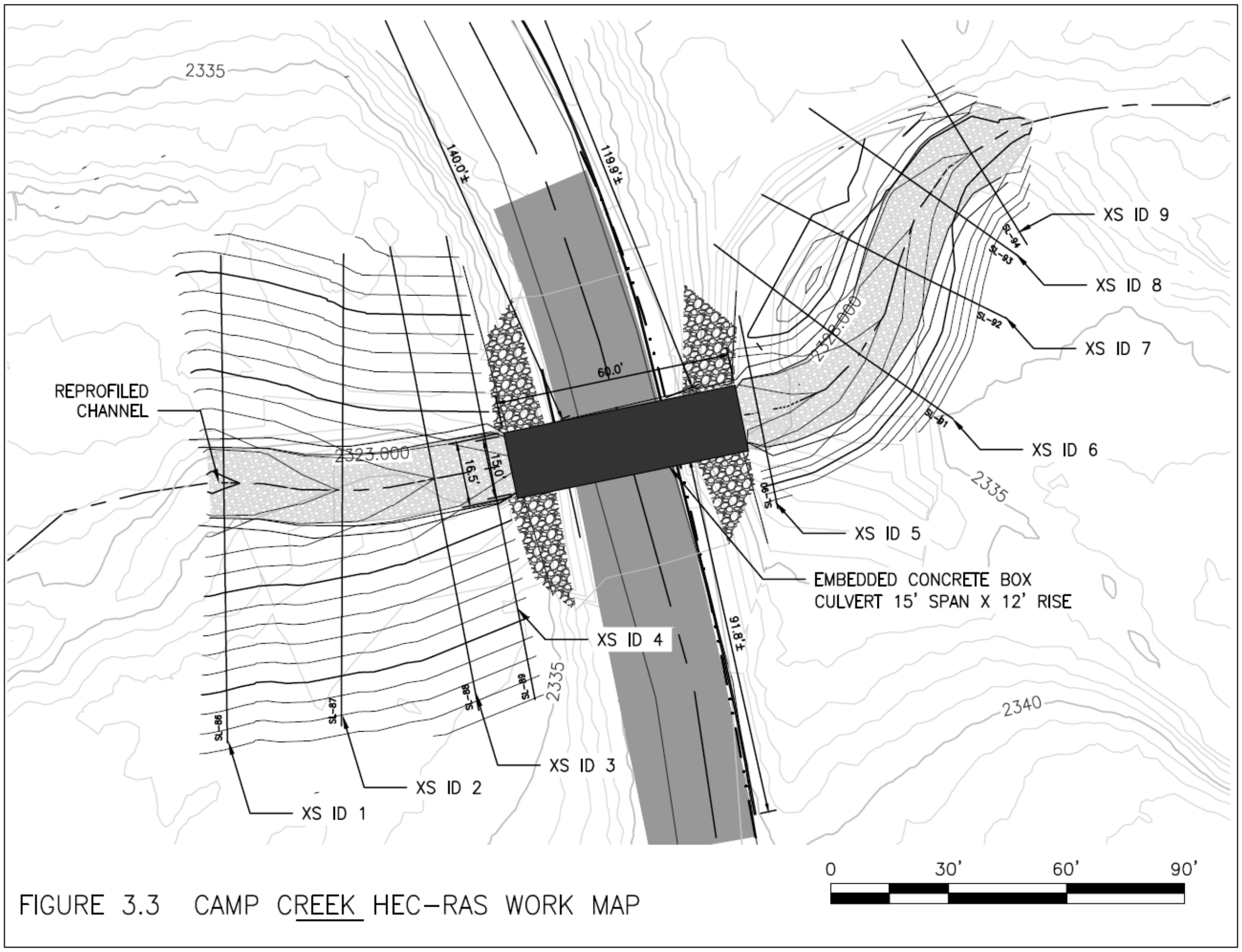


Figure 3.3 Camp Creek HEC-RAS Work Map

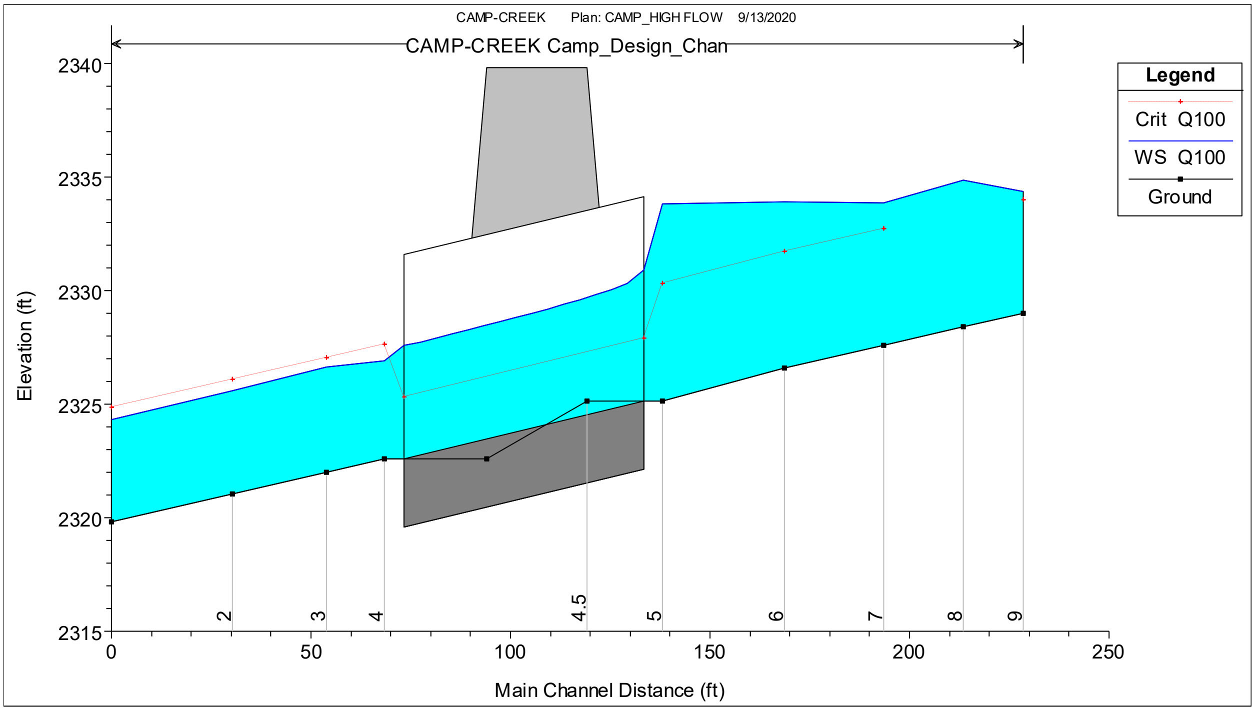


Figure 3.4 Camp Creek HEC-RAS Longitudinal Profile

Table 3.6 Camp Creek HEC-RAS Model Results

Cross Section ID	River Station	Min. Channel Elevation (ft)	Water Surface Elevation (ft)	Critical Water Surface (ft)	Energy Grade line Elevation (ft)	Energy Grade line Slope (ft/ft)	Velocity (ft/s)	Froude No.
9	1+35.00	2,328.99	2,334.38	2,334.02	2,335.93	0.020679	10.85	0.86
8	1+50.00	2,328.39	2,334.87		2,335.47	0.007411	7.32	0.54
7	1+70.00	2,327.59	2,333.87	2,332.73	2,335.2	0.012951	9.61	0.7
6	1+95.00	2,326.59	2,333.92	2,331.71	2,334.84	0.007293	8.05	0.54
5	2+25.53	2,325.15	2,333.83	2,330.3	2,334.62	0.004833	7.25	0.44
4	2+95.10	2,322.58	2,326.9	2,327.62	2,329.4	0.046736	13.81	1.25
3	3+09.65	2,322	2,326.63	2,327.03	2,328.58	0.033655	12.35	1.07
2	3+33.29	2,321.05	2,325.58	2,326.09	2,327.72	0.038031	12.89	1.13
1	3+63.58	2,319.84	2,324.32	2,324.88	2,326.54	0.039819	13.11	1.16

Table 3.7 Camp Creek Engineered Streambed Material Calculations

CDFW ESM		
Percent of Mix	Rock Size, ft	
	min	max
16%	3.3	4.0
34%	1.3	3.2
34%	0.08	1.2
9%	0.013	0.07
7%	SAND/SILT	

Hydraulic Characteristics			
q_f =	78.0	$ft^3/s/ft$	Unit Discharge in Main Channel
$Q_{100channel}$ =	1,170	ft^3/s	1pct AEP (00-year peak flow)
$W_{channel}$ =	15	ft	Main Channel Width
So =	0.04	ft/ft	Channel Slope = ft/ft

USACE (1991) Channel Bed Rock Sizing			
So =	0.04	ft/ft	Channel Slope
$q_{100channel}$ =	78.0	$ft^3/s/ft$	Unit Discharge With Concentration Factor
g =	32.2		Gravitational Acceleration
sf =	1		
D30 =	2.18	ft	

Parameters used to size CDFG ESM		
So =	0.04	ft/ft
q =	78.0	ft2/s
g =	32.2	lbm*ft/s2
n_{sed}=	0.40	-
$D_{30\text{-}CORPS}$ =	2.18	ft

CDFG Engineered Bed Material Size				
$D_{8\text{-}ESM}$ =	0.013	ft	0.2	in
$D_{16\text{-}ESM}$ =	0.08	ft	0.9	in
$D_{50\text{-}ESM}$ =	1.3	ft	15.7	in
$D_{84\text{-}ESM}$ =	3.3	ft	39.2	in
$D_{100\text{-}ESM}$ (calc) =	8.16	ft	97.9	in
$D_{100\text{-}ESM}$ (use) =	4	ft	48.0	in

3.3 FALL CREEK AT DAGGETT ROAD ANALYSIS AND RESULTS

Assessment of Fall Creek capacity to convey the 1% design flood and size engineered streambed material. Hydraulic analyses are computed using SRH-2D. Engineered streambed material is calculated using the California Department of Fish and Wildlife methodology as described in Love and Bates (2009).

The SRH-2D model requires model geometry, roughness, upstream inflow boundary condition, and downstream water surface elevations. Field survey data collected in 2019 with the design surface merged to form a single surface serves as model geometry. Model roughness is simulated using Manning's n. Figure 3.5 provides a map of Manning's n and Table 3.8 lists the roughness values. The 1% design flow is specified for the upstream boundary condition and the downstream water level boundary condition is calculated using uniform flow equations and a slope of 3%.

Model results show the 1% design flow can be conveyed through the crossing with significant freeboard. The constriction imposed by the road approaches creates critical and supercritical flow conditions within the arch crossing. Velocities range from about 10 to 11.5 ft/s near the centerline of the crossing and decrease to about 9 ft/s near the edges of the crossing. Froude numbers through the crossing and extending about 30 ft downstream of the crossing range from about 0.9 to about 1.2. Depths through the crossing range from about 3.5 to 4 ft deep. Comparison of Figure 3.6 and Figure 3.10 demonstrates the large wood structure creates head loss and dissipates the energy from flows discharging from the crossing outlet. It reduces velocities by 1 to 2 ft/s at the outlet and helps to distribute flow and reduce the longitudinal extent of critical and supercritical flow by about 30 ft.

Engineering streambed calculations are shown in Table 3.9.

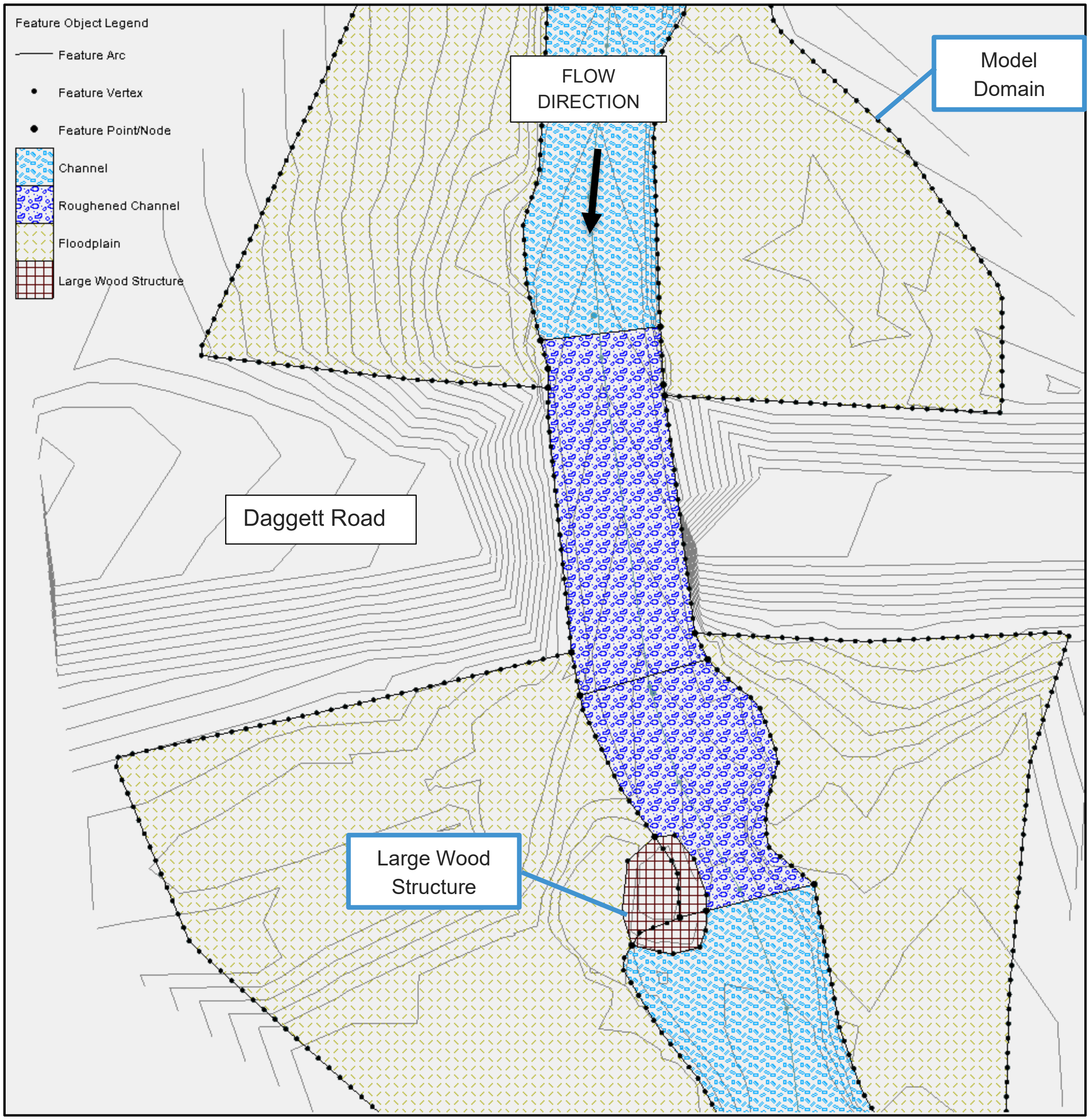


Figure 3.5 Fall Creek Manning's n Map

Table 3.8 Fall Creek Manning's n

Channel	N = 0.055
Roughened Channel	N = 0.06
Floodplain	N = 0.065
Large Wood Structure	N = 0.16

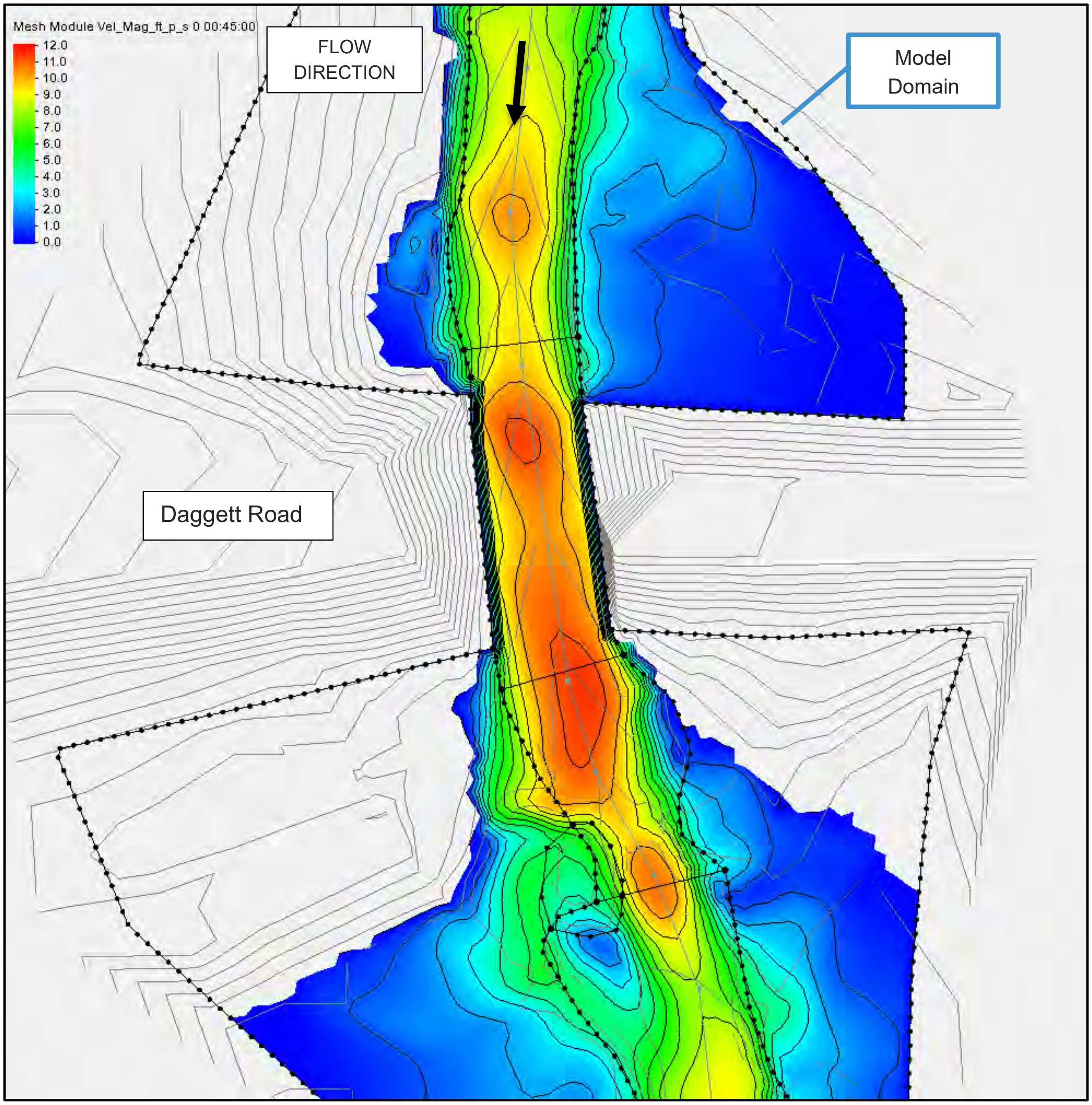


Figure 3.6 Fall Creek Velocity Contour Plot

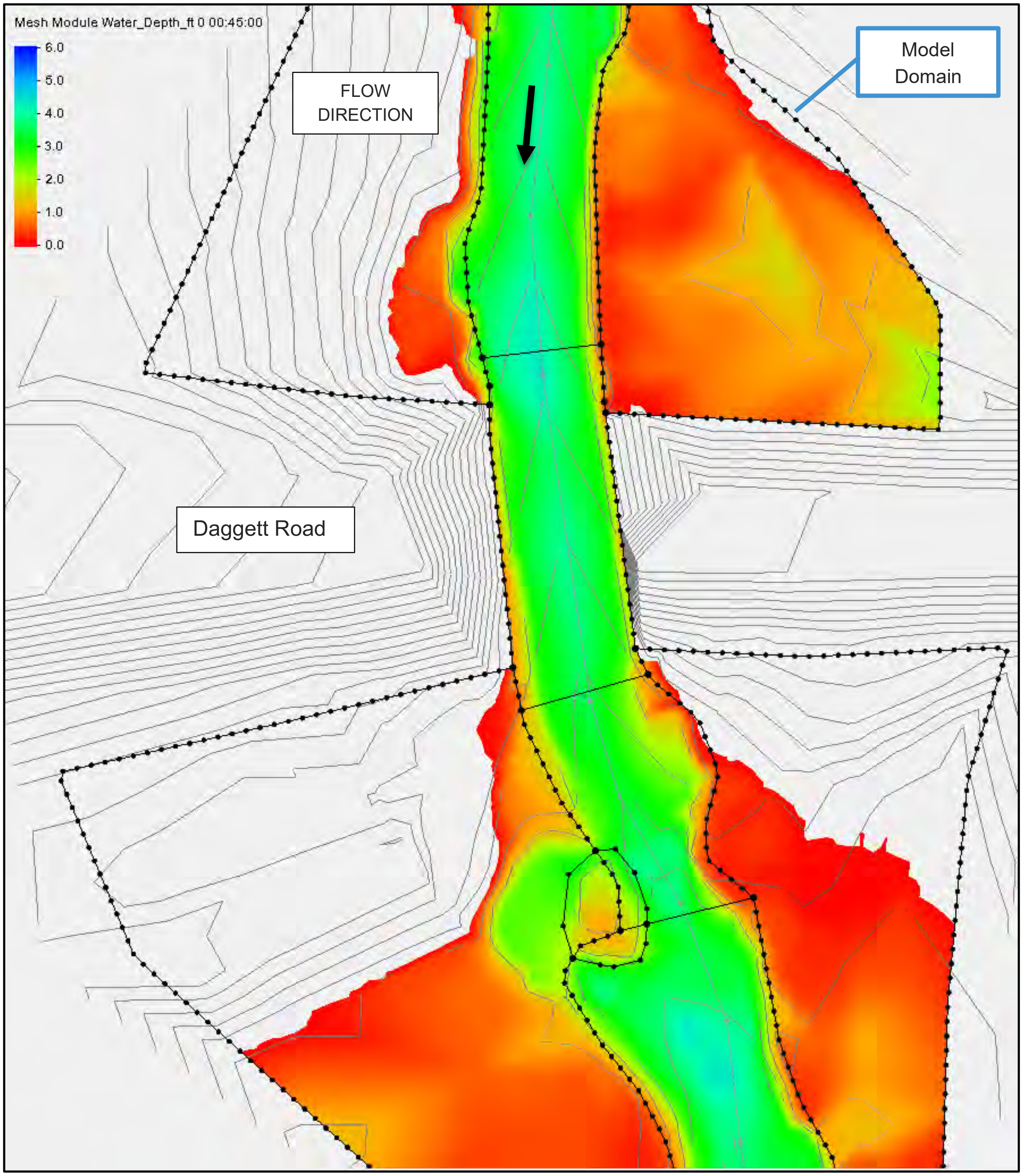


Figure 3.7 Fall Creek Depth Contour Plot

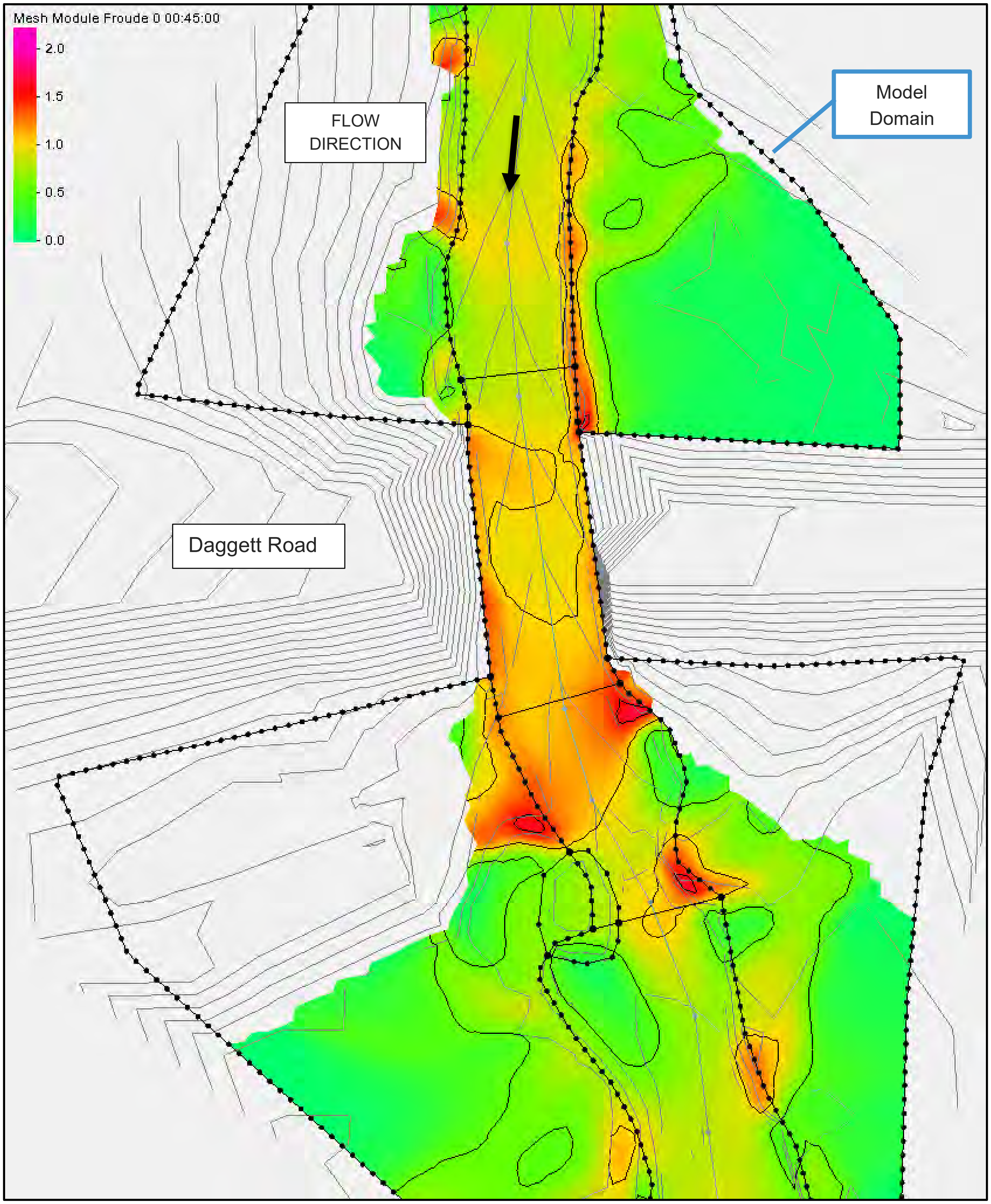


Figure 3.8 Fall Creek Froude Number Contour Plot

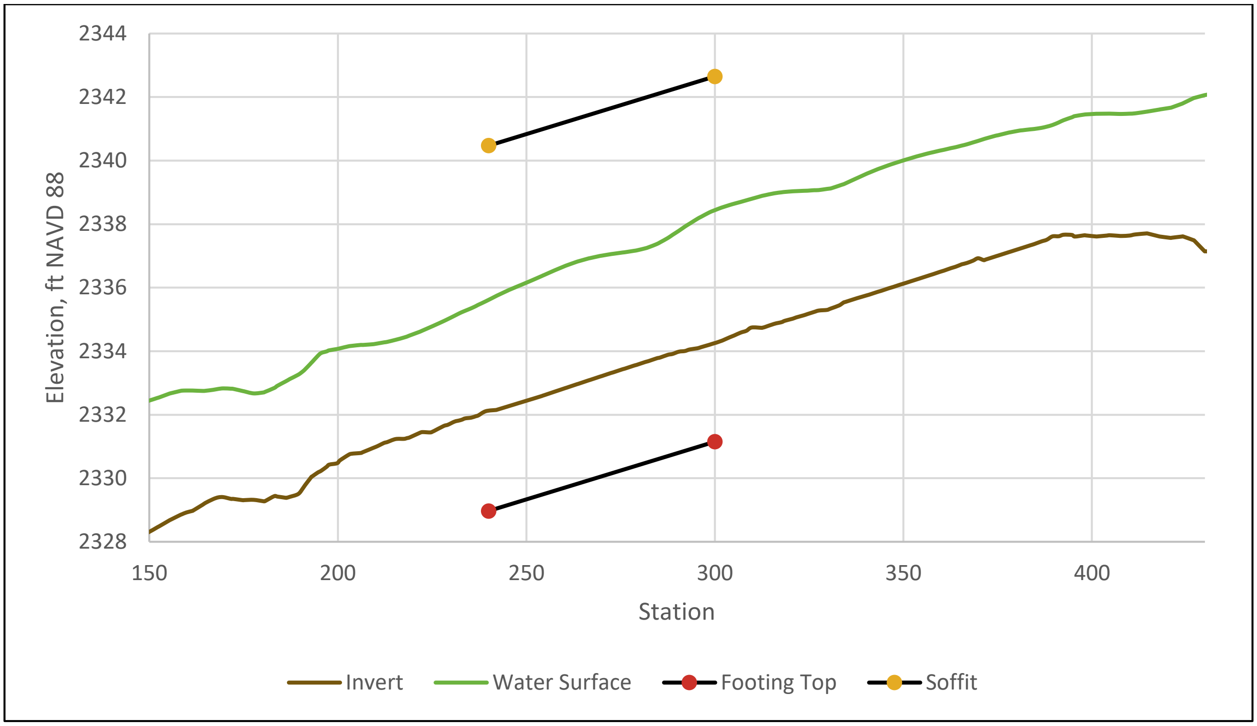


Figure 3.9 Fall Creek Longitudinal Profile

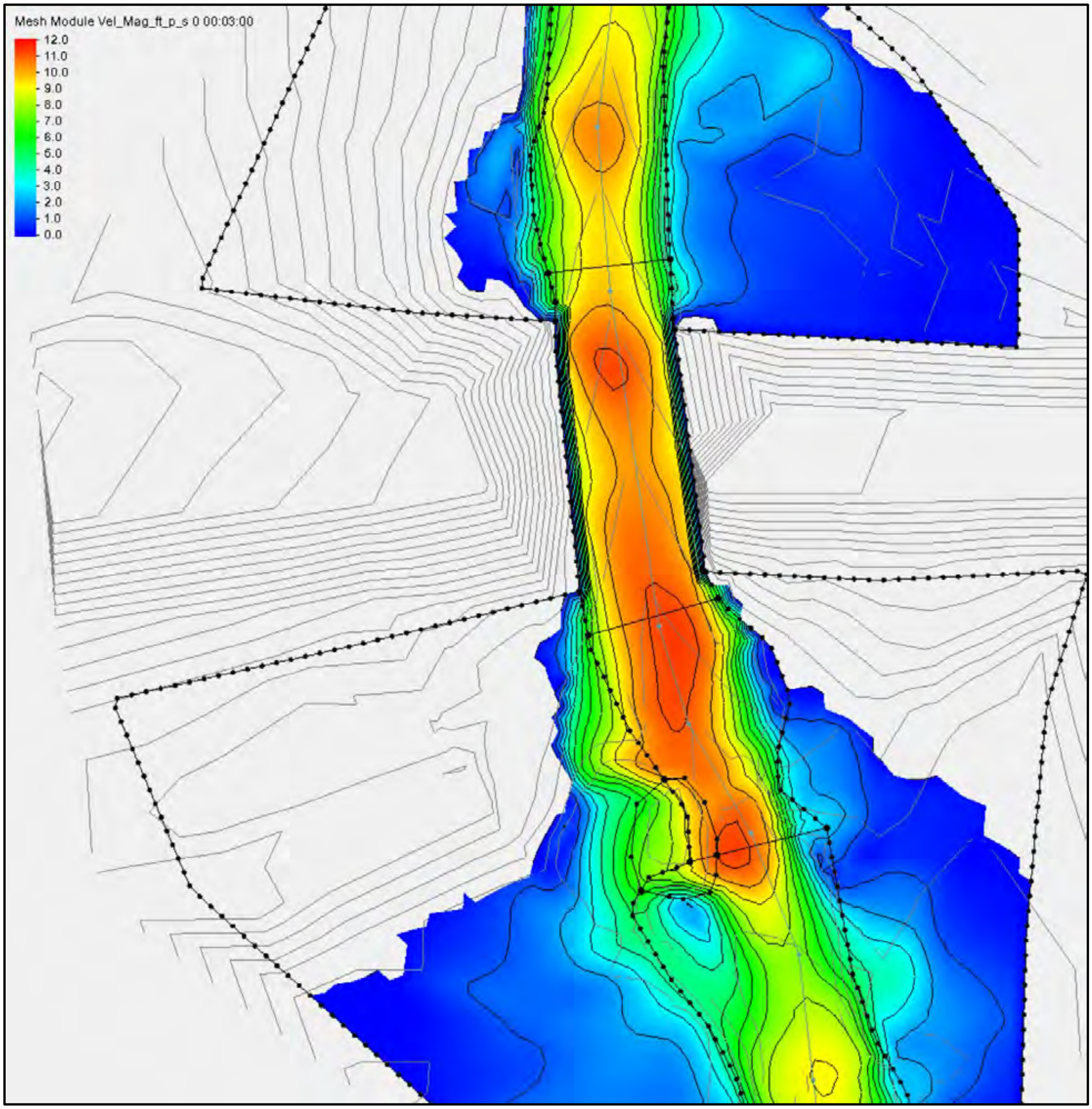


Figure 3.10 Velocity Contour Plot without Large Wood Structure

Table 3.9 Fall Creek Engineered Streambed Material Calculations

CDFW ESM		
Percent of Mix	Rock Size, ft	
	min	max
16%	1.7	4.0
34%	0.7	1.6
34%	0.04	0.6
9%	0.007	0.03
7%	SAND/SILT	

Hydraulic Characteristics			
q_f =	32.6	$ft^3/s/ft$	Unit Discharge in Main Channel
$Q_{100channel}$ =	750	ft^3/s	1pct AEP (00-year peak flow)
$W_{channel}$ =	23	ft	Main Channel Width
So =	0.0364	ft/ft	Channel Slope = ft/ft

USACE (1991) Channel Bed Rock Sizing			
So =	0.0364	ft/ft	Channel Slope
$q_{100channel}$ =	32.6	$ft^3/s/ft$	Unit Discharge with Concentration Factor
g =	32.2		Gravitational Acceleration
sf =	1		
D30 =	1.15	ft	

Parameters used to size CDFG ESM		
So =	0.0364	ft/ft
q =	32.6	ft2/s
g =	32.2	lbm*ft/s2
n_{sed}=	0.40	-
$D_{30\text{-}CORPS}$ =	1.15	ft

CDFG Engineered Bed Material Size				
$D_{8\text{-}ESM}$ =	0.007	ft	0.1	in
$D_{16\text{-}ESM}$ =	0.04	ft	0.5	in
$D_{50\text{-}ESM}$ =	0.7	ft	8.3	in
$D_{84\text{-}ESM}$ =	1.7	ft	20.8	in
$D_{100\text{-}ESM}$ (calc) =	4.33	ft	51.9	in
$D_{100\text{-}ESM}$ (use) =	4	ft	48.0	in

4.0 CONCLUSIONS

Numerical modeling results indicate all the crossings should pass the 1% design flow. The crossing dimensions having similar slope and active channel widths to upstream reaches suggest volitional fish passage past the crossings should mimic upstream and likely downstream conditions. Scotch and Camp Creeks have high unit discharges and are near their maximum conveyance capacities. These crossings should be inspected following large flow events and debris should be removed from the inlet when present. Fall Creek has lower unit discharges during extreme events and is likely to perform better during extreme events. The roughened channel and roughness features at the outlet of Fall Creek dissipates energy during high flows and improves channel stability near the transition with the existing channel downstream of the crossing improvements. The existing channel downstream of the crossing improvements will adjust following dam removal. The constructed features near the outlet will stabilize the transition from the roughened channel at the new crossing to the self-adjustment of Fall Creek downstream. These elements also provide refuge and habitat for aquatic organisms, which is important because of the upstream hatchery as this is one of the few cold-water perennial streams on the Klamath River.

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APPENDIX F4.1
ROADS, BRIDGES, AND CULVERTS – GEOTECHNICAL DESIGN REPORT

TABLE OF CONTENTS

1.0 SCOPE

This Roads, Bridges, and Culverts Geotechnical Design Report Appendix contains an overview of the geotechnical design recommendations for construction access and permanent access infrastructure for the Klamath River Renewal Project (KRRP) (Table 1.1).

This document is intended to provide a comprehensive overview of the geotechnical design development for the Roads, Bridges and Culverts components of the KRRP. The Project Drawings (100% Design Drawing Package) and Appendix F1 of the 100% Design Report should be reviewed in conjunction with this document. Refer to Appendix F4.2 for the supporting figures noted in this report. Geotechnical Data Reports are included in Appendix F4.3 for the Copco Road Surface and Sub-Surface Geotechnical Data Report and F4.4 for the Geotechnical Data report for the site investigations at the bridge and culvert sites listed below in Table 1.1.

The design for each of the components is ongoing and in the absence of specific site data, assumptions have been made (e.g. deferred superstructure design). As this data is confirmed, designs will be confirmed or revised as needed. The 100% Design Drawings show the latest concepts developed by the Project Team for each of the major components.

The list of sites for the Roads, Bridges and Culverts components is provided in Table 1.1.

Table 1.1 Scope Summary

Site	Treatment
Copco Road at Dry Creek Bridge	Temporary Support Structure
Scotch Creek Culvert	Concrete Box Culvert
Camp Creek Culvert	Concrete Box Culvert
Fall Creek at Daggett Road	Bottomless Arch Culvert
Fall Creek at Copco Road Bridge	Temporary Support Structure

2.0 METHODS

This investigation used the 100% Project Drawings and information obtained during the Value Engineering phase to develop geotechnical design parameters and help progress the KRRP transportation infrastructure design. The geotechnical data used as part of this investigation are documented in GeoServ, Inc. (2020a and 2020b) geotechnical data reports. This investigation was completed to obtain information on the engineering properties of the rock, soil, groundwater, and to inform the designs and construction techniques for each site. The engineering properties of the project area rocks and soils were assessed using industry standard methods (CDC 2001, Williamson 1984, and BOR 2001). The rocks and soils were classified and assessed following the most recent ASTM methods.

The soil and rock test holes (i.e., bore holes) were located at each site to characterize the spatial distribution of rock and soil types and engineering properties. This sampling scheme was intended to assess the horizontal and vertical distribution of soil or rock near the ground surface. The bore holes were drilled vertically using a rotary auger drill rig. The holes were drilled in late 2019 and early 2020 and extended below the shallow soil horizon and the expected structure foundation sub-grade elevation. For each bore hole, the soil depth, color, particle size and volume, relative density, particle angularity and shape, moisture content, strength, cohesion, and compaction were logged and visually noted or measured in a soil laboratory (GeoServ,Inc. 2020a and 2020b)

A Standard Penetration Test (SPT) was completed every 5 feet of drilled depth or at specific depths (e.g., slide shear surface) to help measure and quantify the relative density and strength of the soil and rock. The tests were completed following ASTM 1586. Split spoon core samples were collected, photographed, and field classified. Bulk and carved soil samples were collected at various depths within each bore hole.

The geotechnical design parameters were measured and/or calculated using the available field and laboratory testing data following standard methods. The ultimate and allowable foundation bearing capacities were calculated using the Meyerhof (1956) method following AASHTO design guidance. Foundation settlement was modeled using the Burland and Burbridge (1984) method following AASHTO design guidance.

3.0 REGIONAL GEOLOGIC AND TECTONIC SETTING

The rocks that underlie the project area are of the Cascade Range Geomorphic Province (Figure 3.1, F4.2) (Jennings, et. al. 1977). There are several different mapped rock types within the project area; however, most of them are extrusive igneous rock types (i.e., volcanic). The regional and local topography are an expression of these relatively young Tertiary volcanic rocks and Quaternary colluvial and alluvial rocks. As mapped by Luedke, et. al. (1981) (I-1091-C), all of the project area is underlain by Tertiary volcanic rocks that have been reworked by modern denudation processes. Most of the exposed rock is rhyolite, dacite, and basalt.

Historically and presently, this region has been subject to fault activity. Figure 3.1 shows the location and distribution of the Alquist-Priolo Fault Zones Map and Holocene faults (i.e., active faults). The Richter magnitude scale is used to quantify the amount of seismic energy released by an earthquake. Earthquakes with a magnitude greater than three can be felt by most people, but significant damage usually only occurs in earthquakes that have a magnitude greater than five (USGS 1989). Several earthquakes with a magnitude of five or greater, have had an epicenter within 100 miles of the project area in the last 100 years. However, the region has regularly experienced localized smaller magnitude (between three and five) earthquakes over the last 100 years within 50 miles of the project area.

The mapped soil types are shown on Figure 3.2 (see Appendix F4.2). Most of the soils are on 5 to 50 percent slopes and are residuum weathered from volcanic rock. There are several other soil types making up less than 5% of the project area. Given that the bulk of the soils are completely weathered volcanic rock, they have a poor to fair rating as a potential source of sand, gravel, and road fill (NRCS 2020).

4.0 LOCAL GEOLOGIC CONDITIONS

4.1 SUBSURFACE CONDITIONS

4.1.1 COPCO ROAD AT DRY CREEK BRIDGE

The observed subsurface material at this site consists of fill made up of cohesive sandy gravel/cobble clay with soft to very stiff consistency. Below the fill layer, there is in-place native rock. Most of the in-place material is hard volcanic rock varying from fresh to very weathered into clay with gravel and cobbles. No groundwater was observed within the boreholes. The streambed material was only observed at the surface and consists of alluvium with small to large gravel and small cobble. The alluvial material is mobilized frequently during flooding. The depth to rock under the alluvium is unknown. The temporary bridge support structure will likely be founded on the shallow alluvium.

4.1.2 SCOTCH CREEK CULVERT

The observed subsurface material at this site is fill to about 15 ft. below ground surface (bgs) along the existing road prism, an alluvial sandy to clayey gravel, and weathered volcanic rock. The fill is made up of sandy clay. The alluvium is deposited on top of very dense weathered volcanic rock. The alluvium observed downstream of the road is likely modern delta deposits caused by backwater from Iron Gate Reservoir. At this site, the delta deposits tend to be coarse sand to large gravel. Downstream of the crossing, the delta deposits become finer with more clay, silt, and sand. The USGS mapped the rock as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. The new culvert will likely be founded on volcanic rock. Directly below the existing road grade, groundwater was found at 15 ft. bgs. For the bore holes downstream of the road, no groundwater was found.

4.1.3 CAMP CREEK CULVERT

The observed subsurface material at this site is fill 5 ft. to 10 ft. below ground surface (bgs) along the existing road prism, an alluvial sandy to clayey gravel, and weathered volcanic rock. The fill is made up of sandy clay. The alluvium is deposited on top of a mix of clay, gravel, and boulders. Volcanic rock was found at 10 ft. to 30 ft. bgs along the existing road. Downstream of the existing road crossing, there are two distinct layers of alluvium, loose alluvial sandy clay to clayey sand and medium dense well graded sand. No volcanic rock was encountered at 22 ft. bgs. From 0 ft. to 18 ft. bgs, the alluvium is likely sediment deposited in the Camp Creek delta on top of the original stream channel. The upper layer of alluvial material is loose and liquefiable.

Directly below the existing road grade, groundwater was found at about 15 ft. bgs. For the bore holes downstream of the road, was encountered at 3ft. and 4 ft. bgs. The groundwater was perched above the modern stream channel with the surface water in the stream 2 ft. to 3 ft. lower than the water level measured in the boreholes. The shallow groundwater will likely need to be mitigated during construction.

4.1.4 FALL CREEK AT DAGGETT ROAD

The observed subsurface material at this site is fill 3 ft. to 11 ft. bgs along the existing road prism, and it is a clayey sand and gravel. Below the fill is a 2.5 ft. thick layer of loose to stiff sandy clay. Below the clay is a very dense weathered volcanic rock. The USGS mapped the rock type as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. The new culvert will likely be founded on the volcanic rock. No groundwater was observed within the boreholes.

4.1.5 FALL CREEK AT COPCO ROAD BRIDGE

The observed subsurface material at this site consists of fill made up of rock rubble and cohesive sandy gravel/cobble clay with soft to very stiff consistency. Below the fill layer, there is in-place native rock. Most of the in-place material is hard volcanic rock. No groundwater was observed within the boreholes. The streambed material was only observed at the surface and consists of alluvium with large gravel, cobble, and boulders. The alluvial material is mobilized infrequently during flooding. The depth to rock under the alluvium is unknown. The temporary bridge support structure will likely be founded on the shallow alluvium.

5.0 GEOLOGIC HAZARDS

5.1 ACTIVE FAULTS AND SEISMIC HAZARD ASSESSMENTS

Project construction and implementation would be subject to a low to moderate risk of damage from fault movement. Fault movement has the potential to affect the stability of the proposed structure(s). According to the CDC (2000), the closest known inactive fault is approximately 16 miles east of the project area (Figure 3.1). Most of the faults east of the project area are considered active, and the most recent events were 4.3 and 4.4 magnitude earthquakes in 1974 and 2005, respectively. To initiate the dominant seismic hazards of the area, an earthquake would have a magnitude of 8.5 or greater (CDC, 1996).

Seismic movement from earthquakes has the potential to affect the stability of the proposed structure(s). According to the CDC (1997) and CDC (2006), the project area is not within a mapped Alquist-Priolo Earthquake Hazard Zone. It is likely that the proposed structure(s) will be impacted by the effects of a large magnitude earthquake due to proximity to known active fault zones. The proposed structure(s) will likely be subjected to frequent smaller magnitude earthquakes. Small earthquakes may cause minor settling or shifting of unconsolidated sediments. Overall, there is a low to moderate risk of damaging earthquakes (Peterson 1996, Peterson 1999, and Toppozada, 2000).

5.2 LIQUEFACTION

Liquefaction typically occurs as a result of seismic events that cause the sudden loss of soil shear strength. The cyclic loading from an earthquake triggers liquefaction. The risk of liquefaction is based on the expected seismic event, soil properties, and groundwater depth. For liquefaction to occur the following must be present:

- Granular soils
- Low soil density
- High groundwater table

The project area rock or soils are granular in nature and lie atop dense volcanic rock. The risk of adverse impacts from liquefaction at the project area is low if the foundations are properly prepared and dewatered.

5.3 FLOODING HAZARD POTENTIAL

The flood hazard potential is addressed in Appendix F3 Hydrotechnical Design Report for Roads, Bridges, and Culverts.

5.4 DAM INUNDATION HAZARD POTENTIAL

The dam inundation hazard potential is addressed in Appendix F1 Roads, Bridges, and Culverts Design Details.

5.5 STREAM SCOUR

The stream scour hazard potential is addressed in Appendix F3 Hydrotechnical Design Report for Roads, Bridges, and Culverts.

5.6 EXPANSIVE SOILS

Potentially expansive clay soil was encountered was encountered during the subsurface investigation at the bridge and culvert sites to include Dry Creek Bridge, Camp Creek Culvert, and Fall Creek at Daggett Road Culvert. Expansive clay soil was also found along the construction road access routes. The presence of very soft expansive clay appears to coincide with road segments that are fill and actively or potentially failing. At the bridge and culvert sites, the risk of expansive soils is low if the foundations are prepared following the Project Drawings. Road failure repairs should follow the Project Drawing typical details, however, site specific designs need to be developed to mitigate the expansive clay.

5.7 VOLCANIC HAZARDS

The project area is not within an area with recent volcanic activity, and the project area is in a zone that could be impacted by a volcanic eruption. Quantifying the volcanic risk to the project area is beyond the scope of this investigation. Overall, the risk of adverse impacts from volcanic activity at the project area is moderate to low.

5.8 SLOPE STABILITY

The project area is within a region with moderate to high landslide susceptibility. Based on the bridge and culvert site location, topography, and subsurface geology there is a low to moderate modern landslide risk. The stream road crossings are susceptible to debris torrents that occur within the stream channel. This is especially a moderate risk during infrequent flood events and after large and severe wildland fires. For example, Jenny Creek Bridge failed in the flood of 1996 partially due to sediment and debris laden flood waters.

There are several active and dormant landslides along the construction access roads (GeoServ, Inc. 2020a). These sections of the road system have a high landslide susceptibility, especially along Copco Road between the Klamathon Bridge and Fall Creek Bridge. The landslides tend to be translational debris slides. The slide planes tend to occur in the weathered volcanic rock and clay horizon where the clay soils are very soft, the rock dips adversely, and there is perched shallow groundwater. Some areas with hard volcanic rock overlain by clay soils, have an ash layer about 5 ft. to 7 ft. bgs with that has very low shear strength (e.g., near Camp Creek Campground). These areas tend to have hummocky topography and rapid soil creep. Road segments where the prism is mainly fill, commonly fail in the landslide prone areas (GeoServ, Inc. 2020a).

5.9 TSUNAMIS AND SEICHE

Based on site location, elevation, and tsunami hazard mapping from the CGS website (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=tsunami) the site is not in a tsunami inundation hazard zone. In addition, oscillatory waves (seiches) are considered unlikely due to the absence of large confined bodies of water in the site area.

5.10 EROSION AND SEDIMENTATION

There is a high erosion risk given that construction at the bridge and culvert sites will occur within and adjacent to stream channels and wetlands. Any construction related disturbance to the soils will increase the erosion risk, and temporary and permanent erosion control measures need to be implemented, per the Project Drawings and Technical Specifications, to keep storm water from discharging site soils and nutrients into the stream channels. Conceptual erosion and sediment control plans, to include dewatering plans, have been developed for each of the bridge and culvert sites (see Project Drawings).

During construction, the contractor needs to implement the Temporary Erosion Control Plans as prescribed on the Project Drawings and California Construction General Permit Storm Water Pollution Prevention Plan (SWPPP) (Erosion and Sedimentation Control, Section 31 25 00) (California Water Board 2010a).

Post construction, the contractor needs to implement final erosion and sediment control measures that follow the Action Plan for the Klamath River Total Maximum Daily Loads (California Water Board 2010b). The final measures shall be implemented as shown on the Project Drawings and include embankment and disturbed area erosion control and controllable sediment discharge BMPs.

5.11 WILDLAND FIRE

The potential risk of wildfire depends on several factors, such as, abundance of flammable vegetation, high winds, topography, and seasonal weather. For the project area, there is a high threat of fire during the dry summer and fall periods due to chaparral and conifer vegetation and high winds. The project area has an extreme to elevated potential for wildfire hazard.

6.0 EARTHWORKS

6.1 SITE PREPARATION

Each project site should be stripped of vegetation and organic debris within the work limits. These materials should be stock piled and may be used as ground cover and revegetation efforts at the end of the project or disposed of offsite. Voids left from removal of debris should be replaced with native fill compacted to 90 percent relative compaction.

6.2 TRENCHES AND CUT-SLOPES

There are three culvert excavation sites that will have deep trenches, and all three of them have similar soil types, mainly Type C soil. Given the measured soil conditions, it is likely that the excavations can be sloped at 1.5H:1V with 4 ft. benches to 20 ft. bgs assuming that the Type C soil is homogeneous. At sites where the culvert excavation is greater than 20 ft. deep (likely Camp Creek Culvert), trench plates or other shoring methods will be needed due to depth of excavation. In addition, presence of saturated, medium dense, non-cohesive gravel excavations will need to be dewatered if water is present. Other shoring methods may be needed depending on the actual excavation depth and type of soil encountered during construction (OSHA 29 CFR 1926.650, 29 CFR 1926.651, and 29 CFR 1926.652). Shoring below 20 ft. bgs needs to be designed by a registered Professional Engineer. During construction, unusual changes in rock or soil strata should be evaluated by the Engineer or designated representative.

For permanent cut-slopes in soil or weathered rock, the slope angle should be no steeper than 2H:1V, and erosion control measures should be implemented to help ensure long-term slope stability. For permanent cut-slopes in hard rock, the slope angle should be no steeper than 1H:1V. Final cut-slope angles may vary depending on the rock and soil conditions encountered. Variations in cut-slope angle can be field fit during construction as approved by the Engineer.

6.3 MATERIALS

Any construction Excavation and fill materials for the various components of the culvert and bridge designs should follow the specifications listed in Table 6.1 according to the type and intended use. These material and placement and compaction, and testing specifications are based on AASHTO criteria and the KRRP material gradation (i.e., Sheets G0050 and G0051). The foundation subgrade material types are not on Sheets G0050 and G0051.

Table 6.1 Excavation and Fill Material Types, Specifications and Testing for Road, Bridge, and Culvert Sites (5000 and 6000 Series Drawings) Foundations

KP Material Type	Material Type	Material Specifications	Placement and Compaction Specifications	Compaction Test Type
Site Specific	Structural Sub-Grade	Firm and Unyielding Native Material free of debris, rocks > 4", and organics	Scarified subgrade to 8" depth, moisture conditioned to within 2% optimum moisture, and re-compacted to at least 95% relative compaction or until firm and unyielding under vibratory roller	ASTM D 698
E3	Foundation Structural Fill	Native Soil or Imported Granular Fill free of debris, rocks > 4", and organics with a Plasticity Index < 12, Liquid Limit < 35, and < 35% Passing No. 200 sieve	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction within 2% of optimum moisture or until firm and unyielding under vibratory roller	ASTM D 698
E5	Road Embankment Fill	Native Soil or Imported Granular Fill free of debris, rocks > 4", and organics with a Plasticity Index < 12, Liquid Limit < 35, and between 15% to 35% Passing No. 200 sieve	Placed in 8" to 12" loose lifts and compacted to at least 95% relative compaction or until firm and unyielding under vibratory roller	ASTM D 698
E7	Erosion Protection (EP)	Crushed rock material with minimum D_{50} of 8", 21", 36" for E7a, E7b, E7c, respectively, that consists of angular, durable rock and gravel, free from slaking or decomposition under the action of alternate wetting and drying, free of hazardous or deleterious material, and shall have a specific gravity > 2.35, absorption < 4.2%, and a durability index > 52	Placed with heavy equipment, not dropped more than 2', compacted until firm and unyielding under mechanical movement of heavy equipment (worked in with appropriately sized excavator)	ASTM D 698
E11	Aggregate Base	CalTrans Class II Aggregate Base	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction and 2% optimum moisture or until firm and unyielding under vibratory roller	ASTM D 1557
E13	Drain Rock	Crushed drain rock shall be imported material that consists of angular, durable rock and gravel free from slaking or decomposition under the action of alternate wetting and drying, free of hazardous or deleterious material	Placed in 4" to 6" loose lifts and compacted to at least 95% relative compaction or until firm and unyielding	ASTM D 698

Backfill material for permanent road embankments shall be as per the Project drawings and Technical Specifications in addition to meeting the following placement requirements.

1. Compaction to the 95% relative density, to be achieved through the following observed method specification.
2. Minimum of 4 passes with a minimum 20,000 lb vibratory roller, proof rolled (e.g., loaded 10 cubic yard minimum dump truck) to test for visible deflection, as measured every other lift.
3. For course granular fill (E3, E5, E7a), vibratory roller shall have a sheeps foot drum.
4. For fine granular fill (E11), vibratory roller shall have a smooth drum.

Material placed in permanent road embankments shall be free of any rocks larger than 4 in. and organic debris and shall have a plasticity index of less than 12. Material shall be moisture conditioned, as approved by the Engineer during placement.

Fill material (E5) placed in permanent road embankments shall have a fines content of less than 35% No.200 sieve.

Material shall be placed in maximum 1 ft. lifts and moisture conditioned to optimum levels, as approved by the Engineer during placement.

Backfill material for temporary road embankments shall be as per the Project drawings in addition to meeting the following placement requirements.

Compaction to 90% relative density, to be achieved through the following observed method specification.

1. Minimum of 4 passes with a 20,000 lb vibratory roller, proof rolled (e.g., loaded 10 cubic yard minimum dump truck) to test for visible deflection, as measured every other lift.
2. For course granular fill (E3, E5, E7a), material shall be compacted through track packing (18-ton minimum vehicle weight) as an alternative to vibratory rolling.
3. For fine granular fill (E11), vibratory roller shall have a smooth drum.

Material shall be placed in maximum 18 in. to 24 in. lifts and moisture conditioned to optimum levels, as approved by the Engineer during placement.

Material shall be free of organic debris and shall be moisture conditioned, as approved by the Engineer during placement.

7.0 FOUNDATIONS

7.1 STRUCTURE FOUNDATION VERTICAL AND LATERAL ALLOWABLE BEARING CAPACITIES

Table 7.1 lists the material properties and vertical and lateral load recommendations. The allowable vertical bearing capacity for bridge abutments and concrete structures assume that the structures are founded on firm and unyielding soil and/or rock. Very soft and firm cohesive soils and very loose and loose cohesionless soils will be over-excavated within the foundation footprints.

For lateral loads, horizontal shear forces are assumed to be offset by frictional forces between the base of footings and the finished subgrade material. Since the subgrade is likely to be made up of firm and unyielding material, shallow footings may be designed to resist lateral loads using the coefficients of friction of listed in Table 7.1 (total frictional resistance equals the coefficient of friction times the dead load). A specified design passive resistance value using an equivalent fluid weight is per foot of depth and a maximum value of 1,250 psf. The passive resistance values include a 1.5 factor of safety. The top 1 ft. of soil can be neglected for the passive resistance calculations. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

Table 7.1 Foundation Material Properties and Vertical and Lateral Allowable Bearing Capacities and Design Load Values

Site/Type	Foundation Material Description	Average Dry Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)	Allowable Bearing Capacity (psf)	Coeff. of Friction	Lateral Passive Resistance (psf/f)	Coeff. of Active Earth Pressure
Copco Road at Dry Creek Bridge	Sandy Gravel and Weathered Rock	131	222	31.2	5,968	0.3	300	0.32
Scotch Creek Culvert	Weathered Volcanic Rock	150	0	35.0	17,441	0.3	300	0.27
Camp Creek Culvert	Well Graded Sand with Gravel	122	0	36.0	17,033	0.3	250	0.26
Fall Creek at Daggett Road	Weathered Volcanic Rock	132	0	35.0	5,509	0.3	300	0.27
Fall Creek at Copco Road Bridge	Sandy Gravel and Weathered Rock	155	0	33.0	5,529	0.3	300	0.29

7.1.1 DRY CREEK BRIDGE AT COPCO ROAD TEMPORARY SUPPORT FOUNDATION

For the temporary bridge support foundations, the subgrade (i.e., native streambed) has an allowable bearing capacity of 5,968 psf (Table 7.1). Loose debris and gravel/cobble should be removed prior to compacting the and constructing the leveling pads.

7.1.2 SCOTCH CREEK CULVERT FOUNDATION

For the permanent culvert support foundation, the subgrade (i.e., native cohesionless soil or rock) has an allowable bearing capacity of 4,000 psf (Table 7.1). Loose debris and gravel/cobble should be removed prior to compacting the subgrade.

7.1.3 CAMP CREEK CULVERT FOUNDATION

For the permanent culvert support foundation, the subgrade (i.e., native cohesionless soil or rock) has an allowable bearing capacity of 2,500 psf (Table 7.1). Loose debris and gravel/cobble should be removed prior to compacting the subgrade. The excavation trench at this site is likely to be greater than 20 ft. bgs and may require additional trench shoring mitigation measures (C5203).

7.1.4 FALL CREEK AT DAGGETT ROAD CULVERT FOUNDATION

For the permanent culvert support foundation, the subgrade (i.e., native cohesionless soil or rock) has an allowable bearing capacity of 5,509 psf (Table 7.1). Loose debris and gravel/cobble should be removed prior to compacting the subgrade. Given the uncertainty in the actual subgrade conditions at this site, the Project Drawings show a shallow bedrock alternative (C5003).

7.1.5 IRON GATE FISH LADDER BRIDGE FOUNDATION

As of this report, the subsurface conditions at this site are unknown, especially on the east side of the fish ladder. The bridge abutments need an allowable bearing capacity of at least 3,000 psf. Subgrade and foundation conditions need to be field verified during construction by the Engineer or designated representative.

7.2 FOUNDATION AND DIFFERENTIAL SETTLEMENT

Foundation settlement was analyzed using the available soil data at each of the sites (GeoServ, Inc. 2020b). The potential settlement was analyzed using the Burland & Burbridge (1984) method assuming 1 in. of allowable settlement. All footings will be reinforced as required by the Engineer to provide structural continuity, to permit strong spanning of local irregularities and to be rigid enough to accommodate potential differential movements (as described below) estimated to be about ½ in. over 20 linear ft. Based on the conditions observed at the bridge and culvert sites, the total structure settlement is expected to be on the order of 1 in. for static compression and ½ in. for dynamic settlement in the event a large seismic event. Differential settlements on the order of ½ in. and ¼ in. are recommended for static and dynamic settlements, respectively.

Differential settlement is the tendency for native material and engineered fill material to settle at differing rates over time when loaded with structures, foundations, or other loads. Differential settlement typically occurs when a structure is placed partially on fill and partially on native material and may cause cracking and other problematic effects to foundation/structure. When a structure is placed on both cut and fill there are two possible ways to limit differential settlement from occurring. One of the following options should be followed:

- The entire area of the structure/foundation can be over-excavated to a depth so that when backfilled with engineered fill to final grade (planned footing bottom) the entire structure/foundation is placed on a uniform thickness of engineered fill above native soil.
- The foundation/footings in the area of fill extend to the depth of the native soils. This deepening of the foundation/footing can be backfilled using unreinforced concrete or "lean mix" to the planned bottom of footing elevation that corresponds with the footings resting on the "cut" area native soils.

7.3 FOUNDATION SETBACK

The bottoms of trenches or other excavations placed adjacent to the perimeter of any foundation(s) should be above an imaginary plane that projects at a 45 degree angle down from the lowest outermost edge of the foundation. Where trenches pass through the plane the trench should be installed perpendicular to the face of the foundation for a distance of at least the depth of the foundation. Deepening of the affected foundation is considered an effective means of attaining the prescribed setbacks.

7.4 FOUNDATION SEISMIC DESIGN

The seismic calculation tables are summarized in Table 7.3 and were developed using the recommended AASHTO seismic design parameters for the permanent and temporary bridge and culvert structures. The shallow subsurface material is classified using the site-specific soil and rock conditions. This classification is based on field observations and the measured engineering soil properties. For temporary structures, the seismic design criteria are based on a 100-year return period (this is equal to a 10% probability of exceedance in 10 years) per the Caltrans Site Seismicity for Temporary Bridges and Stage Construction Memo to Designers dated May 2011 (Table 7.3). For permanent structures, the seismic design criteria are based on a 7% probability of exceedance in 75 years (equal to a 1000 year return period) per the AASHTO LRFD Bridge Design Specifications (Table 7.3).

Table 7.2 Seismic Design Criteria

Site	Type	Site Class	Return Period (years)	PGA	S_1	S_s	S_{D1}	S_{DS}
Copco Road at Dry Creek Bridge	Temporary	B	100	0.036	0.043	0.089	0.043	0.089
Scotch Creek Culvert	Permanent	D	1,000	0.253	0.212	0.423	0.424	0.634
Camp Creek Culvert	Permanent	D	1,000	0.253	0.212	0.423	0.424	0.634
Fall Creek at Daggett Road	Permanent	D	1,000	0.253	0.212	0.423	0.424	0.634
Fall Creek at Copco Road Bridge	Temporary	B	100	0.036	0.043	0.091	0.043	0.091

8.0 SCOTCH CREEK AND CAMP CREEK CULVERTS DELTA DEPOSITS

Scotch and Camp Creeks have modern delta deposits that have built up as a result of backwater from Iron Gate Reservoir (Figure 8.1 and Figure 8.2, Appendix F4.2). The new culvert inverts will be set at or close to the pre-dam channel elevation. Given the horizontal and vertical extents of the delta deposits downstream of the new culverts, there are potential backwater effects that need to be mitigated if these culverts are constructed before reservoir drawdown. As of this report, the reservoir drawdown is scheduled to occur after the new culverts have been installed. If the culverts are installation before reservoir drawdown, there will be large wedges of sediment downstream of the culverts that will be above the finished grade of the new culverts and, during flooding, backwater will inundate the new culverts. To mitigate potential damage to the new culverts from backwater, the delta sediment deposits downstream of the culverts should be removed to below the stations shown on Figure 8.1 and Figure 8.2. The Project Drawings show the location of the berm just downstream of the roughened stream channel.

9.0 COPCO ROAD FILL SLOPE FAILURES

There are several sections of Copco Road that are actively failing due to poor subgrade conditions (GeoServ, Inc. 2020a) (Figure 9.1, F4.2). In order to repair and mitigate existing fill slope movement and increase road bearing capacity, several road segments have been identified that need fill slope stabilization treatments. The recommended mitigation measure is construction of rock fills along the outer edge of the road prism. Stabilization measures are needed to provide a stable road prism.

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APPENDIX F4.2
ROADS, BRIDGES, AND CULVERTS – SUPPORTING FIGURES

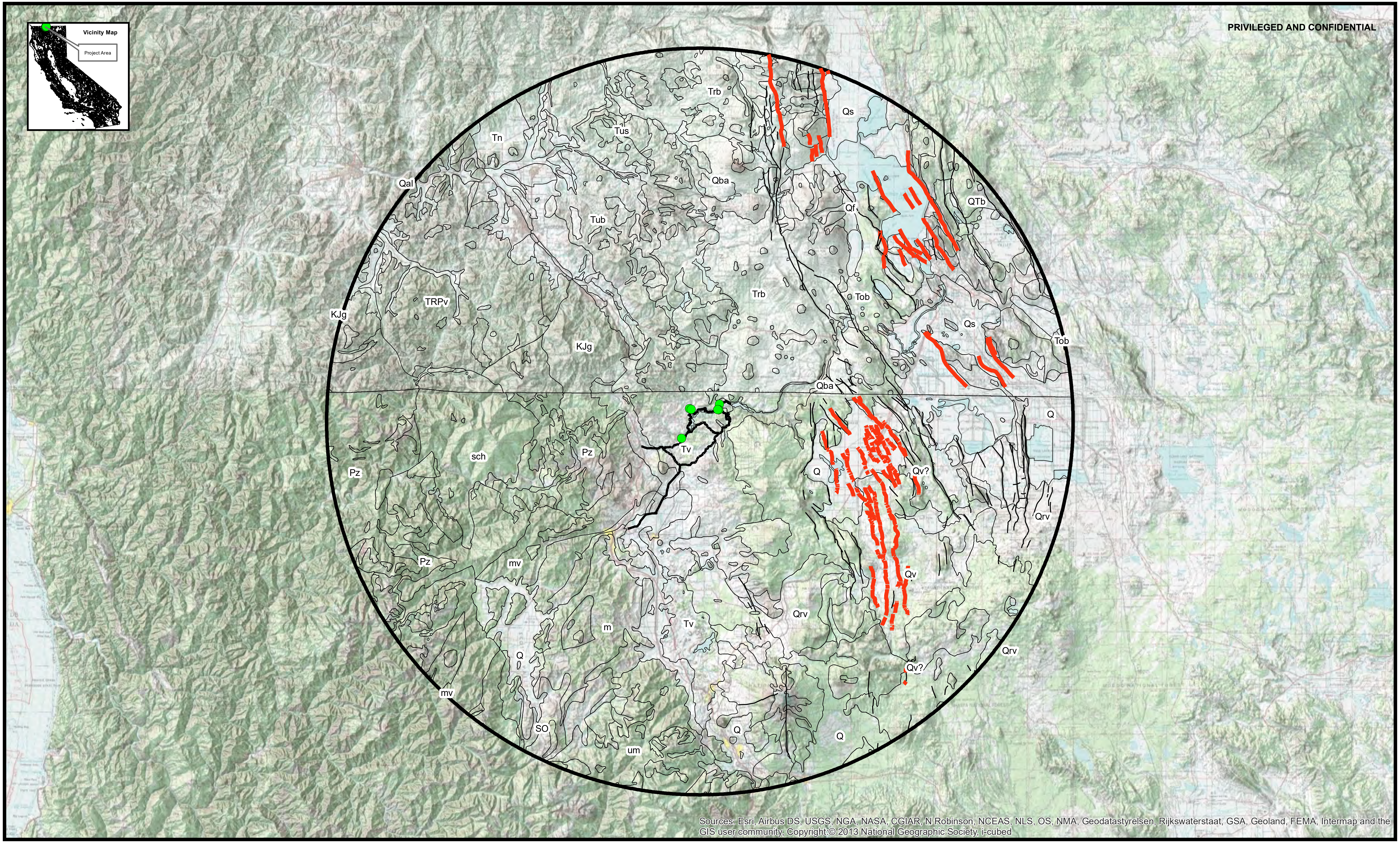


Figure 3.1 Project Area Geology, 50 mile Fault Circle, and Fault Map (red lines = active faults)

Date: 10/2/2020

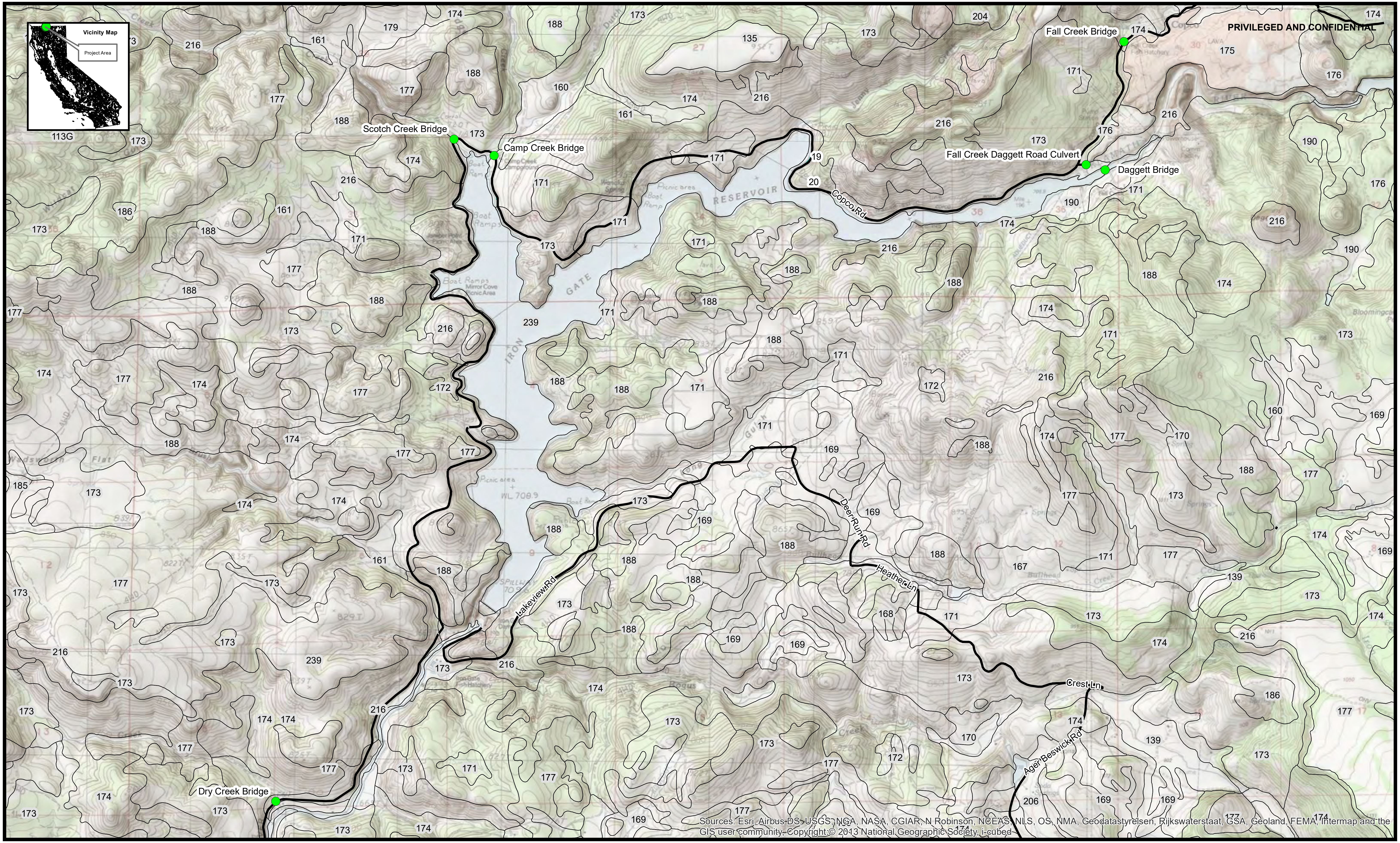
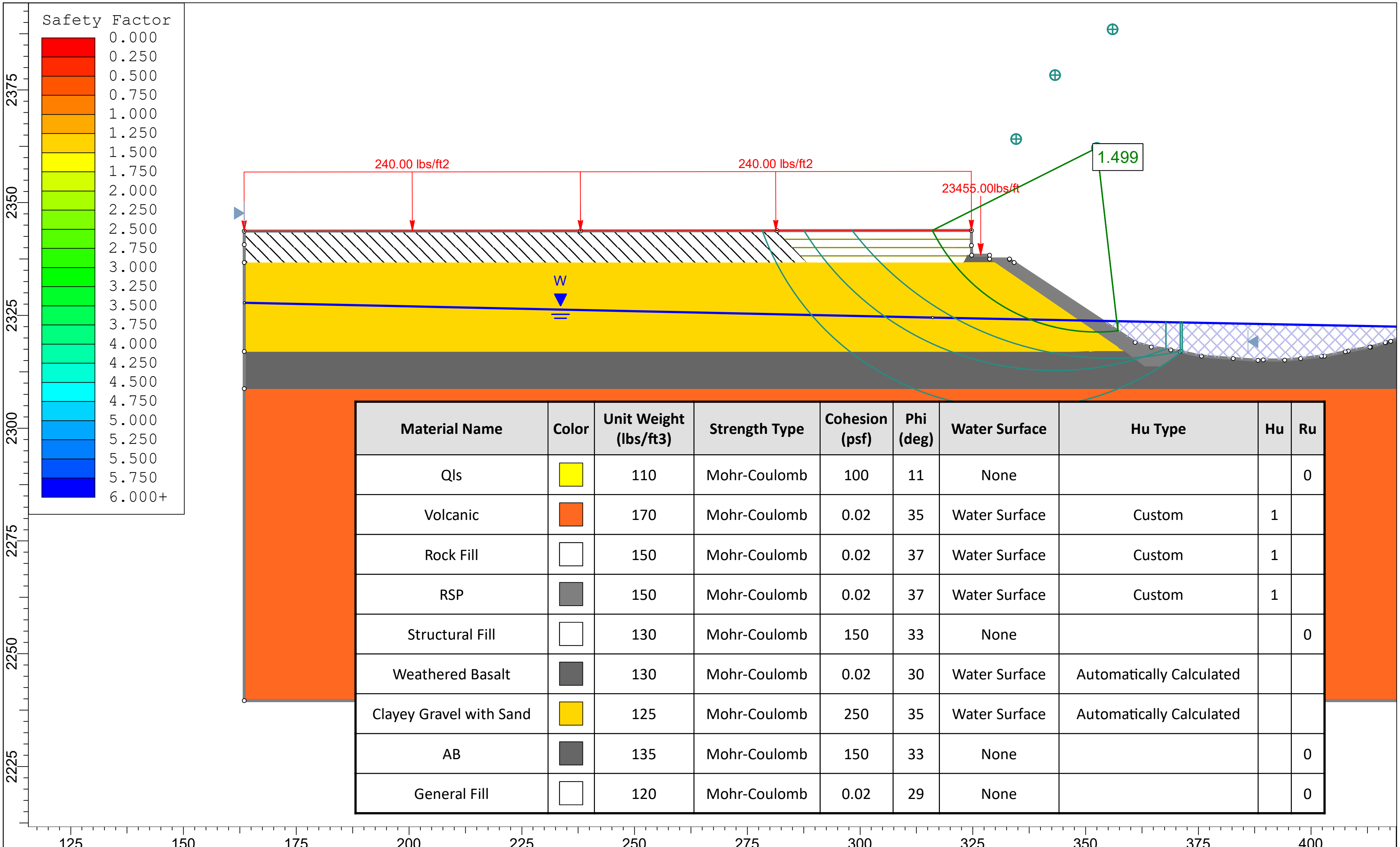


Figure 3.2 Project Area (Green Dots) Soils Map

Date: 10/2/2020



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Qls		110	Mohr-Coulomb	100	11	None			0
Volcanic		170	Mohr-Coulomb	0.02	35	Water Surface	Custom	1	
Rock Fill		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
RSP		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
Structural Fill		130	Mohr-Coulomb	150	33	None			0
Weathered Basalt		130	Mohr-Coulomb	0.02	30	Water Surface	Automatically Calculated		
Clayey Gravel with Sand		125	Mohr-Coulomb	250	35	Water Surface	Automatically Calculated		
AB		135	Mohr-Coulomb	150	33	None			0
General Fill		120	Mohr-Coulomb	0.02	29	None			0

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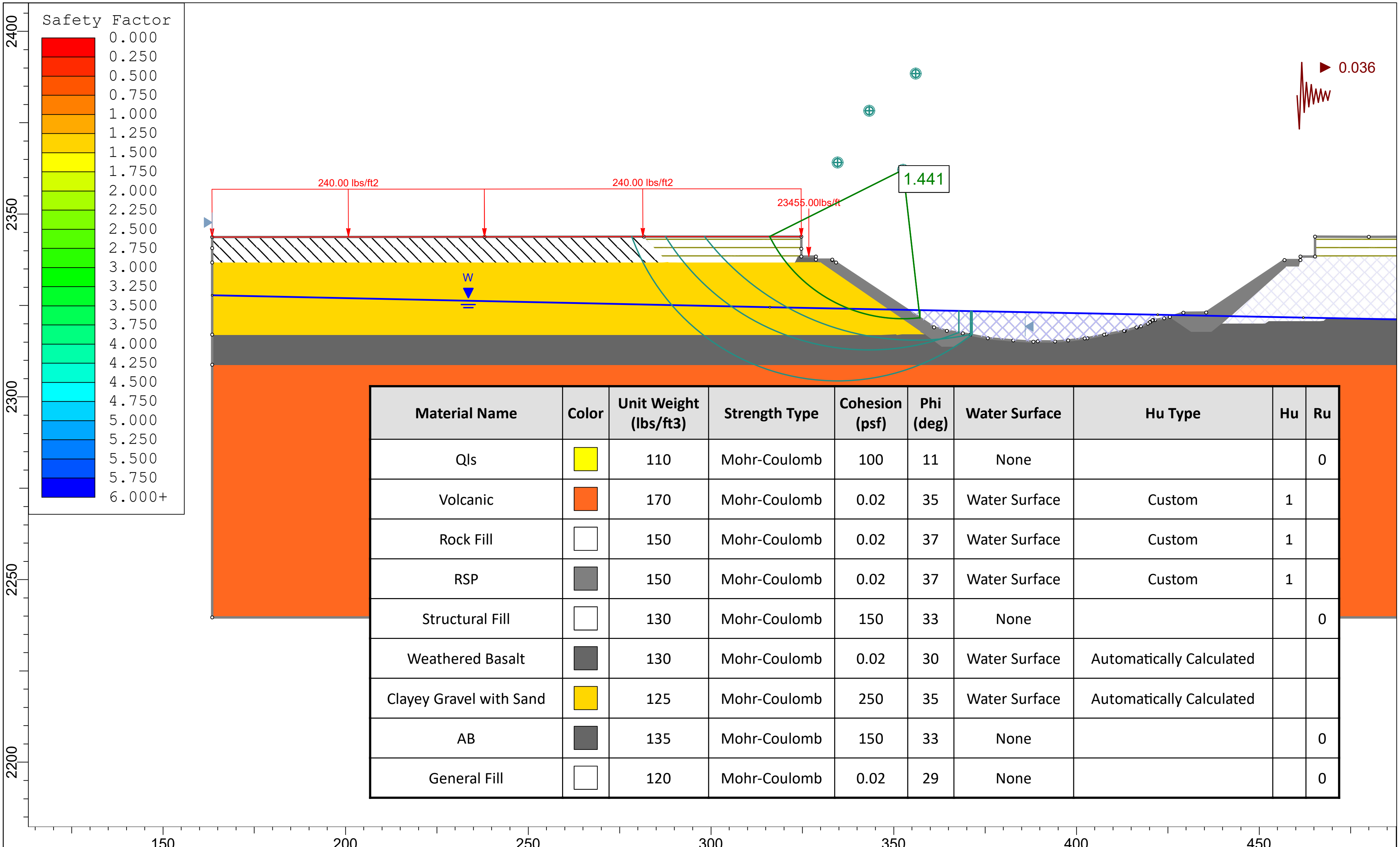
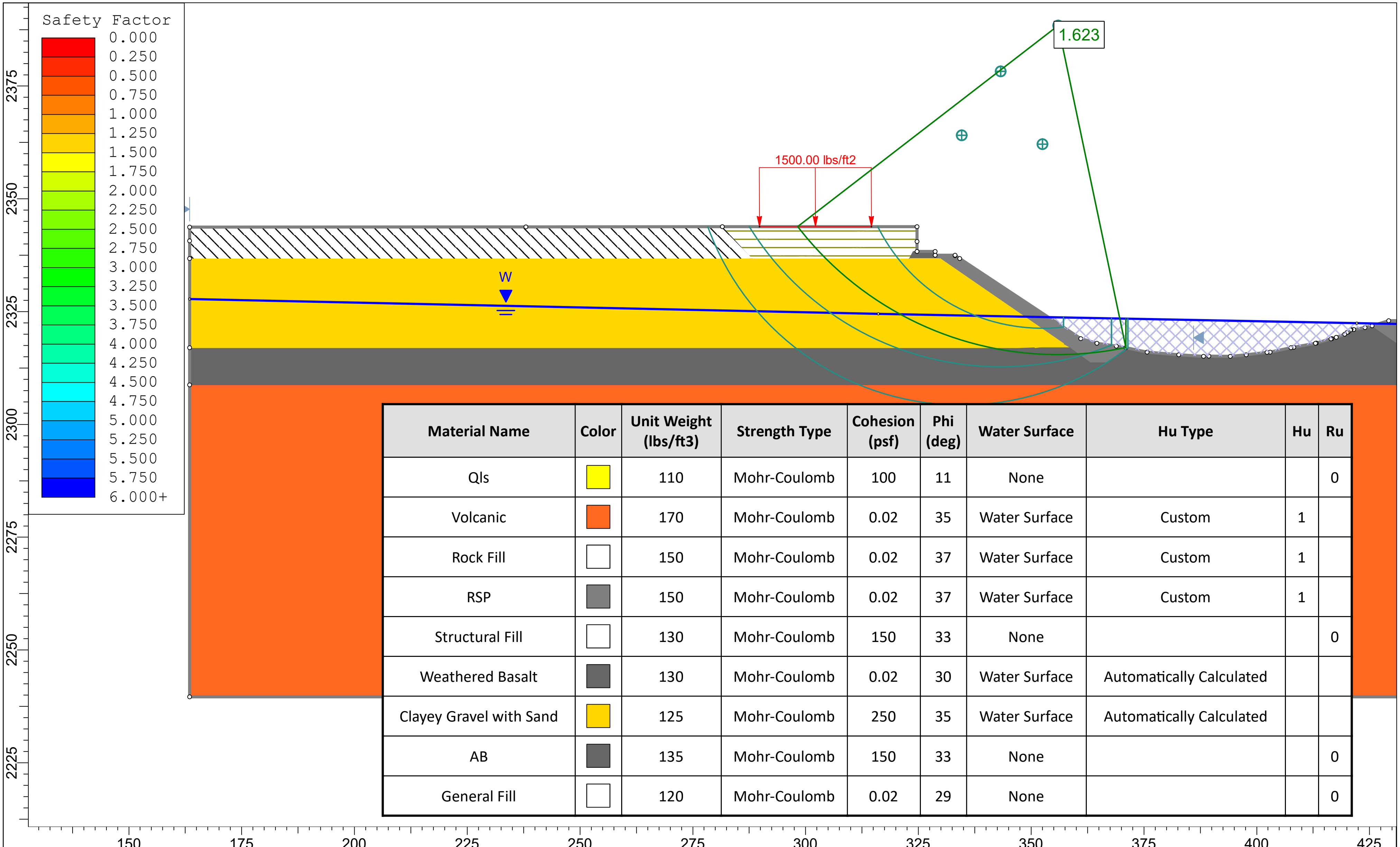
Project	Figure 7.1 Daggett Temporary Construction Access Bridge River Right Abutment Slope Static Conditions Slope Stability Model Results
Analysis Description	Spencer
Drawn By	James Fitzgerald
Company	GSI
Date	10/7/2020
File Name	KRRP_Daggett_Bridge_RR_Abutment_Wall_Design_Static_201005.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Qls		110	Mohr-Coulomb	100	11	None			0
Volcanic		170	Mohr-Coulomb	0.02	35	Water Surface	Custom	1	
Rock Fill		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
RSP		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
Structural Fill		130	Mohr-Coulomb	150	33	None			0
Weathered Basalt		130	Mohr-Coulomb	0.02	30	Water Surface	Automatically Calculated		
Clayey Gravel with Sand		125	Mohr-Coulomb	250	35	Water Surface	Automatically Calculated		
AB		135	Mohr-Coulomb	150	33	None			0
General Fill		120	Mohr-Coulomb	0.02	29	None			0

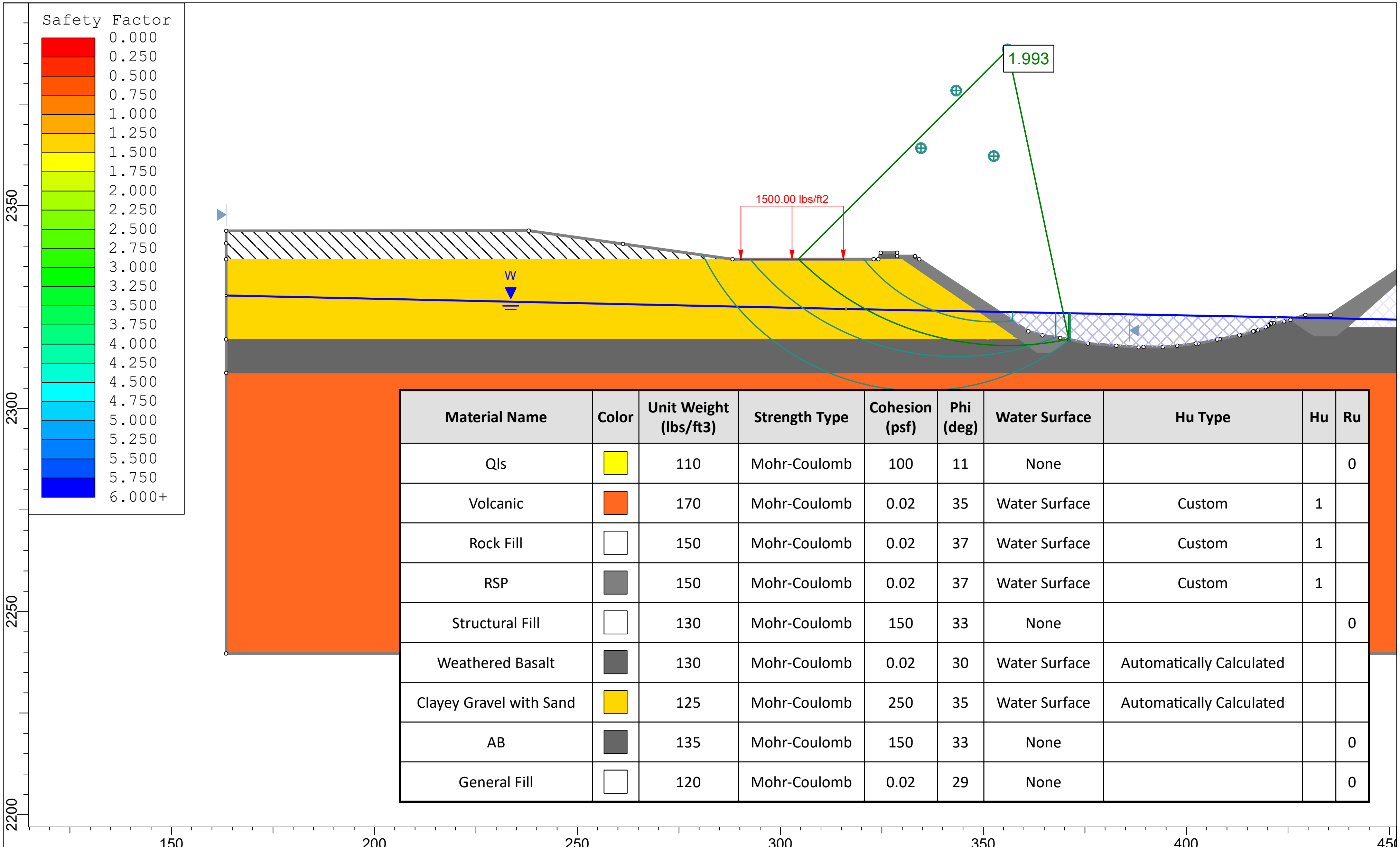
Project	Figure 7.2 Daggett Temporary Construction Access Bridge River Right Abutment Slope Seismic Conditions Slope Stability Model Results		
Analysis Description	Spencer		
Drawn By	James Fitzgerald	*Company*	GSI
Date	10/7/2020	*File Name*	RRP_Daggett_Bridge_RR_Abutment_Wall_Design_Seismic_201005.slin

SLIDEINTERPRET 6.039



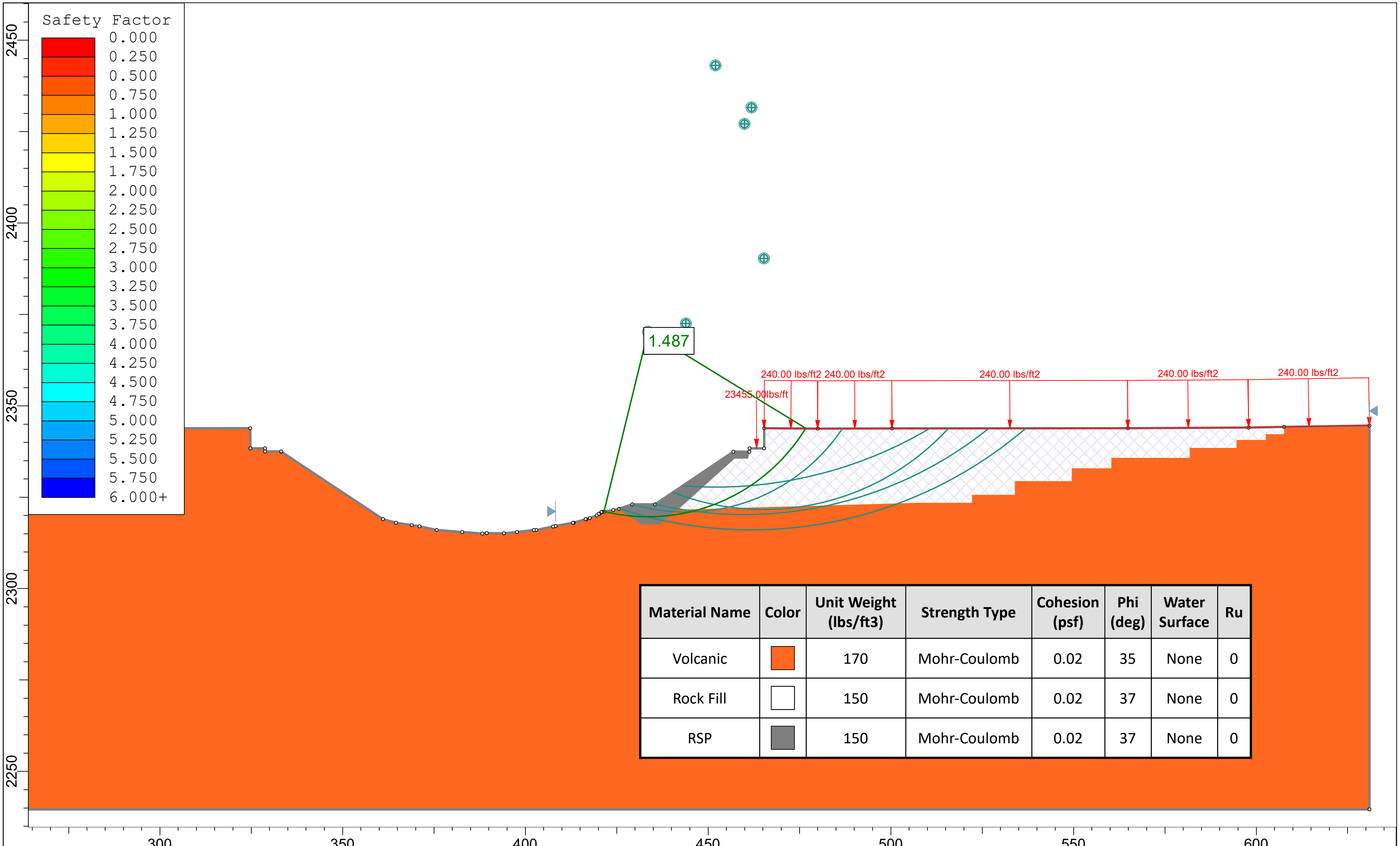
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Qls		110	Mohr-Coulomb	100	11	None			0
Volcanic		170	Mohr-Coulomb	0.02	35	Water Surface	Custom	1	
Rock Fill		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
RSP		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
Structural Fill		130	Mohr-Coulomb	150	33	None			0
Weathered Basalt		130	Mohr-Coulomb	0.02	30	Water Surface	Automatically Calculated		
Clayey Gravel with Sand		125	Mohr-Coulomb	250	35	Water Surface	Automatically Calculated		
AB		135	Mohr-Coulomb	150	33	None			0
General Fill		120	Mohr-Coulomb	0.02	29	None			0

GEOSERV	*Project* Figure 7.3 Daggett Temporary Construction Access Bridge River Right Crane Pad Fill (Alternative 1) Static Conditions Slope Stability Model Results		
	Analysis Description Spencer		
	Drawn By James Fitzgerald	*Company* GSI	
SLIDEINTERPRET 6.039	*Date* 10/7/2020	*File Name* KRRP_Daggett_Bridge_RR_Crane_Pad_Design_Static_Alt1_201005.slim	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Qls		110	Mohr-Coulomb	100	11	None			0
Volcanic		170	Mohr-Coulomb	0.02	35	Water Surface	Custom	1	
Rock Fill		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
RSP		150	Mohr-Coulomb	0.02	37	Water Surface	Custom	1	
Structural Fill		130	Mohr-Coulomb	150	33	None			0
Weathered Basalt		130	Mohr-Coulomb	0.02	30	Water Surface	Automatically Calculated		
Clayey Gravel with Sand		125	Mohr-Coulomb	250	35	Water Surface	Automatically Calculated		
AB		135	Mohr-Coulomb	150	33	None			0
General Fill		120	Mohr-Coulomb	0.02	29	None			0

GEOSERV	*Project* Figure 7.4 Daggett Temporary Construction Access Bridge River Right Crane Pad Fill (Alternative 2) Static Conditions Slope Stability Model Results	
	Analysis Description Spencer	
	Drawn By James Fitzgerald	*Company* GSI
SLIDEINTERPRET 6.039	*Date* 10/7/2020	*File Name* KRRP_Daggett_Bridge_RR_Crane_Pad_Design_Static_Alt1_201005.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Volcanic		170	Mohr-Coulomb	0.02	35	None	0
Rock Fill		150	Mohr-Coulomb	0.02	37	None	0
RSP		150	Mohr-Coulomb	0.02	37	None	0

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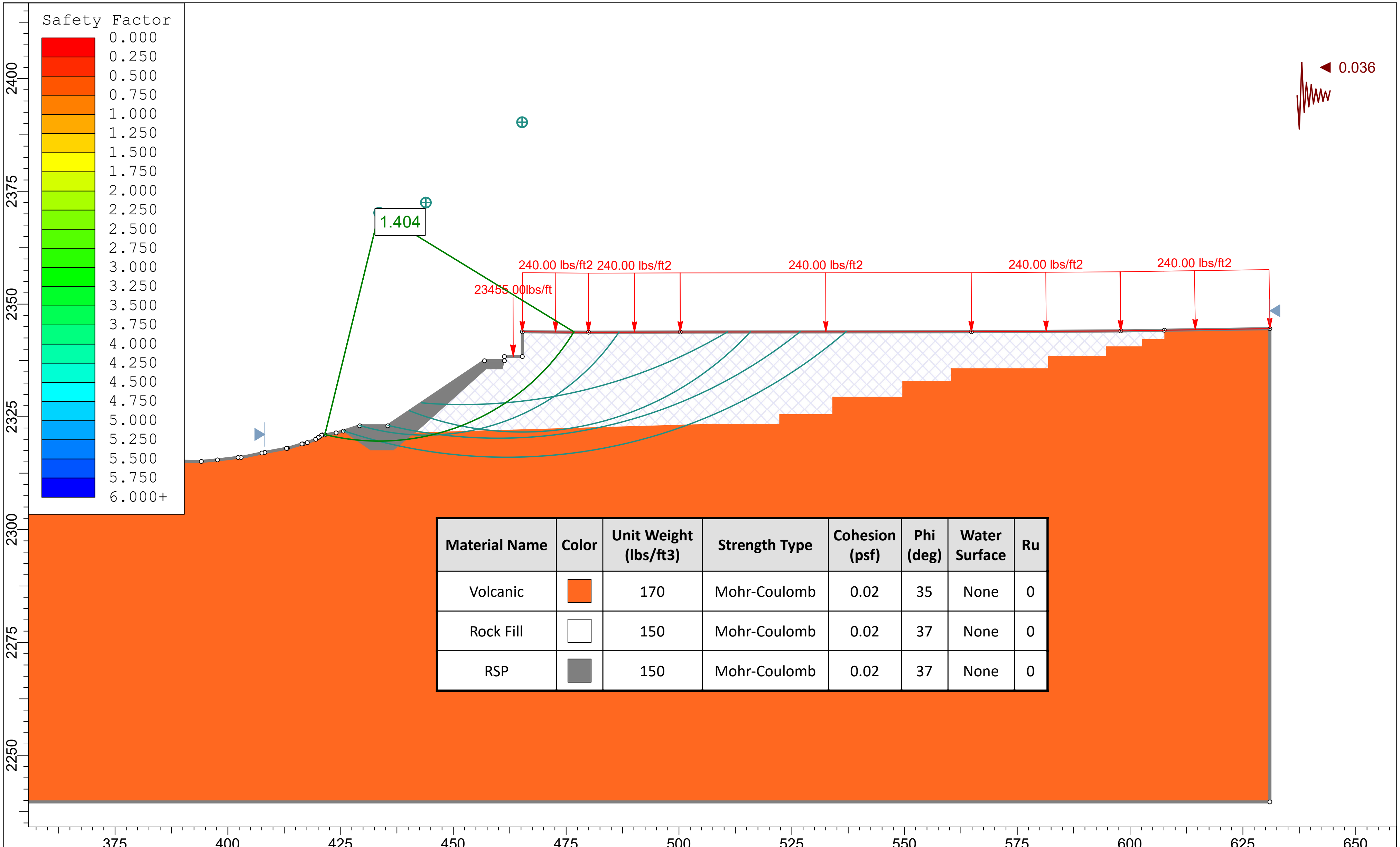
Project	Figure 7.5 Daggett Temporary Construction Access Bridge River Left Rock Fill Static Conditions Slope Stability Model Results
Analysis Description	Army Corp #1
Drawn By	James Fitzgerald
Company	GSI
Date	10/7/2020
File Name	KRRP_Daggett_Bridge_RL_Abutment_Wall_Design_Static_201005.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Volcanic		170	Mohr-Coulomb	0.02	35	None	0
Rock Fill		150	Mohr-Coulomb	0.02	37	None	0
RSP		150	Mohr-Coulomb	0.02	37	None	0

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Project
Figure 7.6 Daggett Temporary Construction Access Bridge River Left Rock Fill Seismic Conditions Slope Stability Model Results

Analysis Description
Army Corp #1

Drawn By
James Fitzgerald

Company
GSI

Date
10/7/2020

File Name
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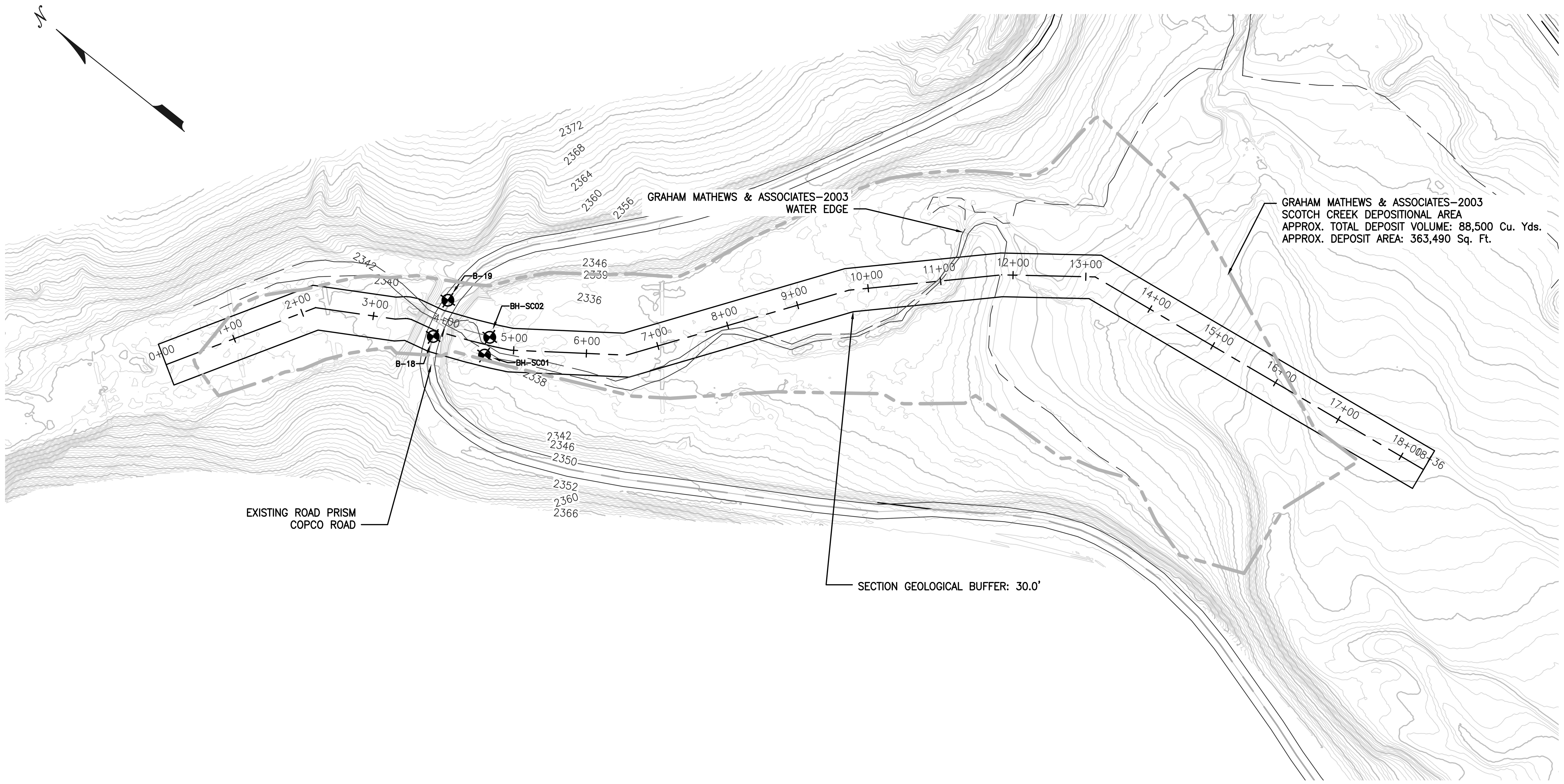
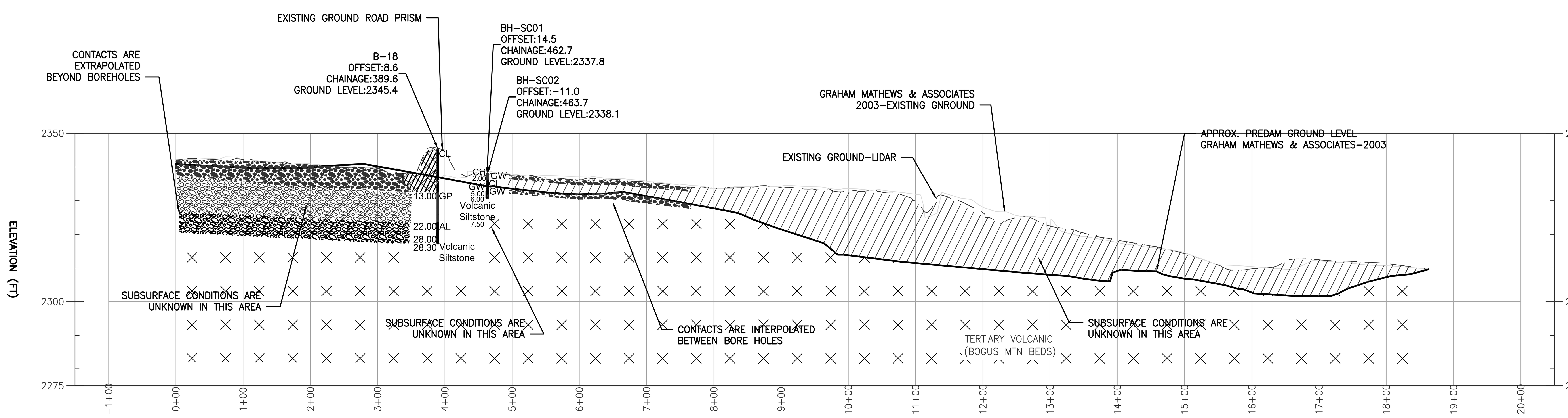


PLAN
1"= 100'



SECTION–1
1"= 100'; 5H : 1V

LEGEND				
HATCH	STRATA DESC.	STRENGTH/ DENSITY DESC.	CALC. BEARING CAPACITY (PSF)	UNDRAINED SHEAR STRENGTH (PSF)
	AC	NA		
	AL	MEDIUM DENSE		
	CH	VERY STIFF		2,214
	CL	MEDIUM STIFF/STIFF		1,107
	FILL	SEE INNER STRATA		
	GP	MEDIUM DENSE		
	GW	MEDIUM DENSE	3,305–9,148	
	Volcanic Siltstone	WEAK TO VERY DENSE	17,878	

0 100 200 300
1" = 100'

FOR INFORMATION ONLY

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
SCOTCH CREEK BRIDGE GEOTECHNICAL DATA SUMMARY

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 5/19/20

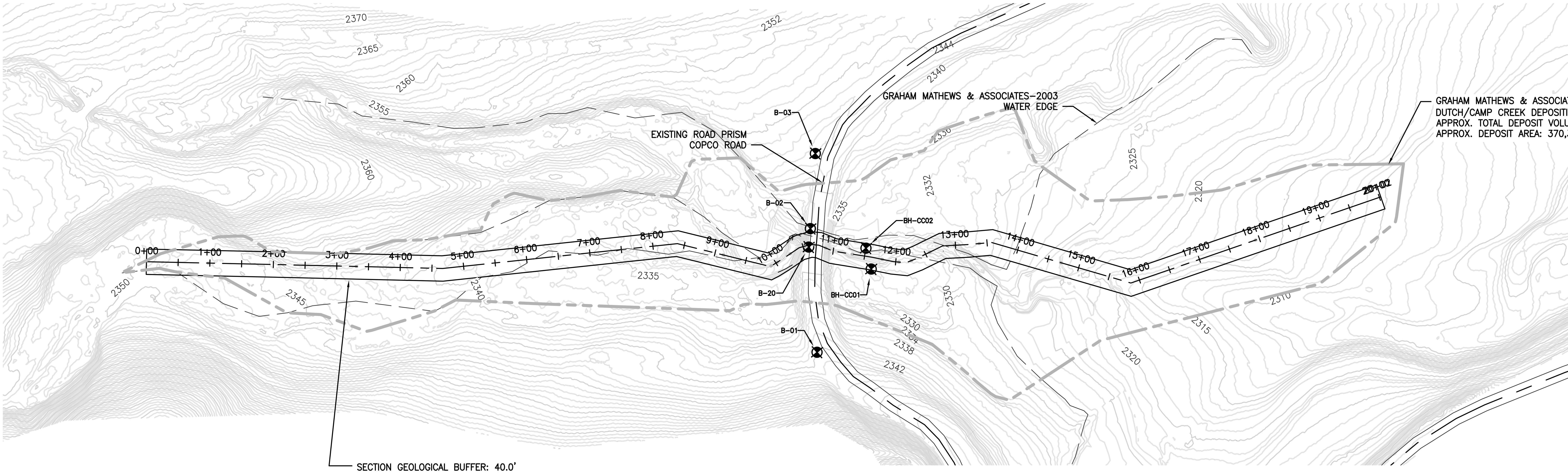
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

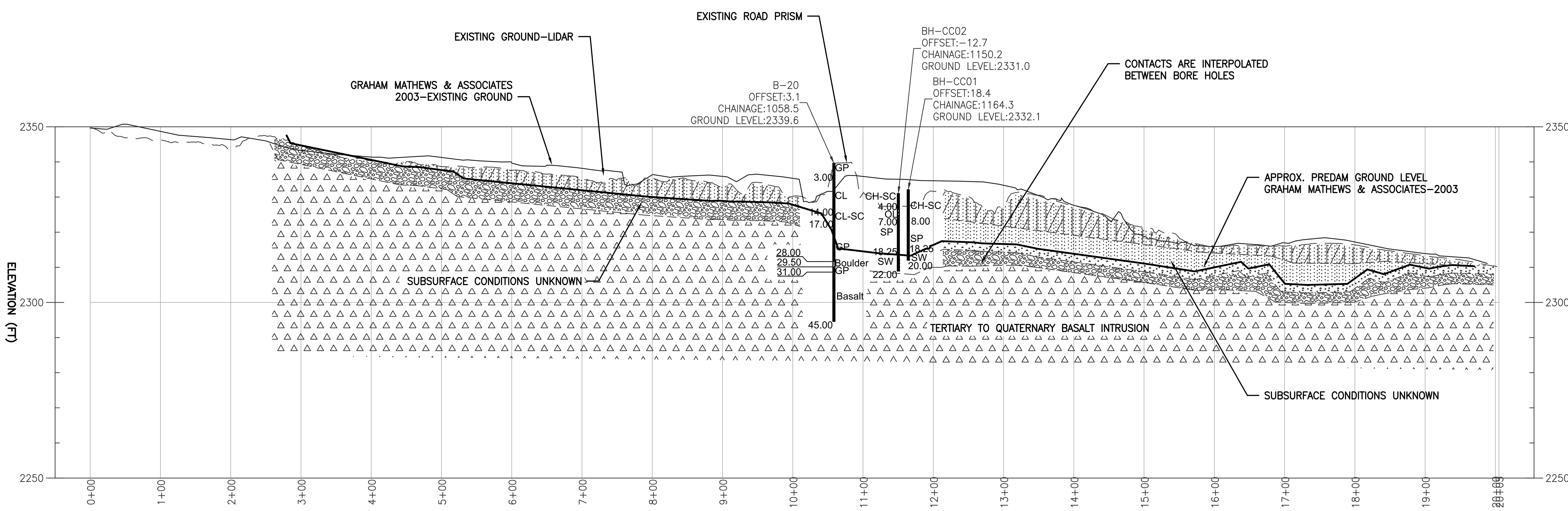
CHECKED: JF

FIGURE 8.1



PLAN

1"= 100'



SECTION–1

1"= 100'; 5H = 1V

LEGEND					
	STRATA DESCRIPTION	STRENGTH/ DENSITY DESC.	CALC. BEARING CAPACITY (PSF)	UNDRAINED SHEAR STRENGTH (PSF)
	BASALT	MODERATELY STRONG		
	BOULDER			
	BOULDER & COBBLE			
	CL	MEDIUM STIFF		
	FILL	SEE INNER STRATUM		
	CH-SC	VERY LOOSE	196-355	
	GC	VERY STIFF		
	GP	MEDIUM DENSE TO DENSE		
	OL	VERY SOFT		123
	SP	LOOSE TO MEDIUM DENSE	196-3,008	
	SW	VERY LOOSE TO DENSE	1,878-3,008	
	VOLCANIC BRECCIA	VERY WEAK		
	VOLCANIC SILTSTONE	WEAK TO MOD. STRONG		

FOR INFORMATION ONLY

0 100 200 300

1" = 100'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:

CAMP CREEK BRIDGE GEOTECHNICAL DATA SUMMARY

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 5/19/20

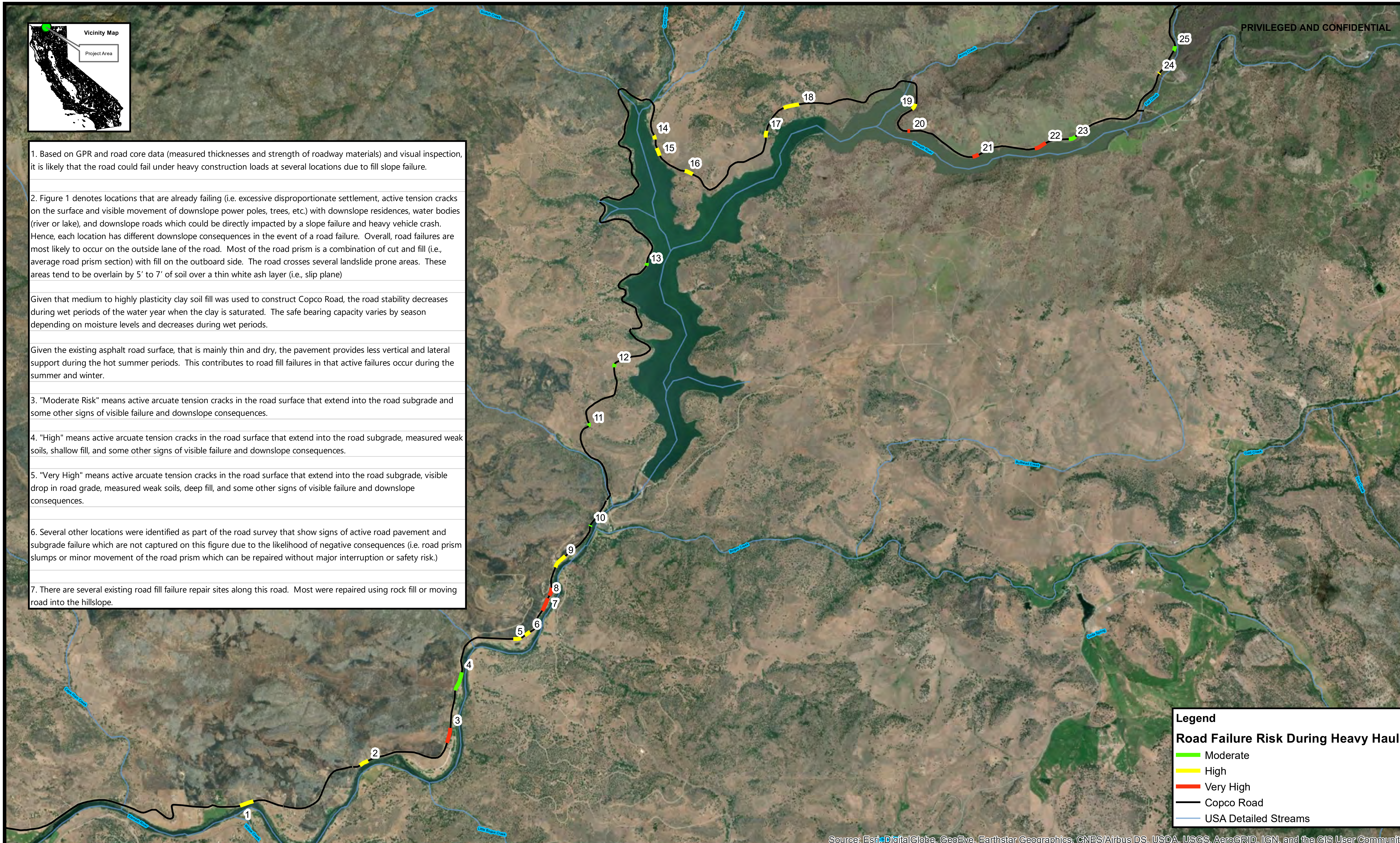
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

CHECKED: JF

FIGURE 8.2



1. Based on GPR and road core data (measured thicknesses and strength of roadway materials) and visual inspection, it is likely that the road could fail under heavy construction loads at several locations due to fill slope failure.

2. Figure 1 denotes locations that are already failing (i.e. excessive disproportionate settlement, active tension cracks on the surface and visible movement of downslope power poles, trees, etc.) with downslope residences, water bodies (river or lake), and downslope roads which could be directly impacted by a slope failure and heavy vehicle crash. Hence, each location has different downslope consequences in the event of a road failure. Overall, road failures are most likely to occur on the outside lane of the road. Most of the road prism is a combination of cut and fill (i.e., average road prism section) with fill on the outboard side. The road crosses several landslide prone areas. These areas tend to be overlain by 5' to 7' of soil over a thin white ash layer (i.e., slip plane)

Given that medium to highly plasticity clay soil fill was used to construct Copco Road, the road stability decreases during wet periods of the water year when the clay is saturated. The safe bearing capacity varies by season depending on moisture levels and decreases during wet periods.

Given the existing asphalt road surface, that is mainly thin and dry, the pavement provides less vertical and lateral support during the hot summer periods. This contributes to road fill failures in that active failures occur during the summer and winter.

3. "Moderate Risk" means active arcuate tension cracks in the road surface that extend into the road subgrade and some other signs of visible failure and downslope consequences.

4. "High" means active arcuate tension cracks in the road surface that extend into the road subgrade, measured weak soils, shallow fill, and some other signs of visible failure and downslope consequences.

5. "Very High" means active arcuate tension cracks in the road surface that extend into the road subgrade, visible drop in road grade, measured weak soils, deep fill, and some other signs of visible failure and downslope consequences.

6. Several other locations were identified as part of the road survey that show signs of active road pavement and subgrade failure which are not captured on this figure due to the likelihood of negative consequences (i.e. road prism slumps or minor movement of the road prism which can be repaired without major interruption or safety risk.)

7. There are several existing road fill failure repair sites along this road. Most were repaired using rock fill or moving road into the hillslope.

Prepared By:

GeoServ Inc Engineering Geologists Rock - Soil - Water

Figure 9.1 Copco Road Fill Failure Risk Map

N

0 0.5 1 Miles

Coordinate System: NAD 1983 StatePlane California I FIPS 0401 Feet
Projection: Lambert Conformal Conic
Datum: North American 1983

Date: 10/4/2020

October 7, 2020 GSI Project #: 190725

Knight Piésold Ltd. (KP)

Subject: KRRP Copco Road Surface and Subsurface Geotechnical Survey Technical Memorandum

Dear Knight Piésold:

In accordance with your request and authorization of GeoServ, Inc. (GSI) has prepared the enclosed Geotechnical Survey based on the requirements and proposed project specifics identified during our review. Specifically, this technical memorandum (memo) provides a summary of the methods used to survey Copco Road from the Klamathon Bride to the Copco Dam Road intersection. The memo also includes Appendix A that shows and lists relevant data and diagrams to include:

- Survey Field Road Core Test Results
- Road Core Logs
- Summary Photographs
- Figure showing Road Fill Failure Segments
- Ground Penetrating Radar Survey Diagrams of Road Fill Failure Segments

Data and results presented in this technical memorandum are preliminary and subject to change. Additional analyses and interpretations need to be made from the survey data. General design recommendations are included for road fill failure segments. If you have any questions regarding the data and results, please do not hesitate to contact this office. The opportunity to be of service is appreciated.

Respectfully submitted,

James Fitzgerald, Senior Geologist
GeoServ, Inc.
624 South Mount Shasta Blvd.
Mount Shasta, CA 96067
(530) 227-8963
jf@geoscienceserv.com

KRRP Copco Road Surface and Subsurface Geotechnical Survey Technical Memorandum

Prepared for: Knight Piésold Ltd. (KP)

Prepared by: GeoServ, Inc. (GSI)

First Draft Report Date: April 6, 2020

Second Draft Report Date: October 7, 2020

Summary

GSI completed a surface and subsurface road survey of 17.5 miles of Copco/Iron Gate Lake Road (Copco Road). The survey included drilling 18 road cores and surveying both traffic lanes with ground penetrating radar (GPR) survey equipment. These data were used to characterize surface/subsurface road conditions. This report includes a summary of the methods used for data collection and analysis, data results, preliminary conclusions, and limitation and assumptions (see Appendix A for survey data). Copco Road is a rural Siskiyou County Road with an asphalt and gravel surface that accesses both the Iron Gate and Copco dams, as well as recreational areas and private properties. This survey focused on Copco Road starting at the Klamathon Bridge on the west end and Copco Dam on the east end (Appendix A: Sheet C1).

Assessment of the Copco Road surface and shallow subsurface was accomplished through advancement of 18 road cores spread evenly along the road survey segment (Appendix A: Sheet C1). The road cores were used to help determine asphalt, aggregate base, and native fill thickness, depth to bedrock, fill conditions, groundwater conditions, and road bearing capacity. To provide indirect data on the shallow subsurface and to allow for interpolation and extrapolation between drill sites, a GPR survey was completed along each lane of the surveyed road segments. The direct and indirect data were compiled and analyzed to give an estimate of average asphalt thickness and condition, aggregate base conditions, and cut and fill conditions.

Asphalt: Most of the Copco Road surface is paved with asphalt that is in fair to poor condition based on the direct and indirect measurements taken as part of this survey. There are short sections of gravel surface road. The average measured asphalt thickness is 2" and is in fair to poor condition.

Asphalt Subgrade: Directly under the pavement there is either aggregate base rock with moderate to high density or native fill material with moderate to high density.

Road Subgrade: The road prism is a combination of cut and fill with most of the prism having both cut and fill. Overall, most of the fill is native material locally sourced from the cut areas. The native fill tends to be firm to very stiff cohesive gravelly clay with moderate to high plasticity.

Methods

Direct Measurements: Road core sampling was completed at 18 locations along Copco Road, and the core locations were spread out with about 1 core per mile of road surveyed (Appendix A: Sheet C1 and Table 1). The asphalt was cored using a 6" diamond core bit. The road subgrade was sampled using a 6" hollow stem auger and a Standard Penetration Test (SPT) 1.5 in. inner diameter sampler. The tests were completed following ASTM 1586. Split spoon core samples were collected, photographed, and field classified. Bulk and carved soil samples were collected at various depths within each bore hole. The road cores were located along the outside lane and were generally within the outside primary vehicle wheel tread.

Indirect Measurements: GPR survey was completed on 17.5 miles of Copco Road from the Klamathon Bridge crossing the Klamath River to the Copco Dam Access Road. The survey was completed to help evaluate existing asphalt thickness and condition and to estimate road subgrade soil/rock types and condition. Two GPR survey passes were made along the road, one in each lane, for a total of 35 miles of survey. Each traffic lane was scanned by one pass that corresponded with the primary vehicle wheel tread. Heading east, the survey line was on the outside lane within the outer tire tread. Heading west, the survey line was on the inside lane within the inner tire tread. Within areas of obvious asphalt and/or subgrade failure, additional GPR passes were completed to better define the horizontal and vertical extents of the failures.

Results

In general, drilling of the road surface and prism was accomplished with minimal drilling effort. Total road core depth to auger refusal ranged from 0.8' to 7.8' below ground surface (bgs) (Appendix A: Sheets C2-C13 and Road Core Logs). Even with the presence of clay rich soils, the road core and GPR data correlate relatively well, and general conclusions of road condition can be estimated with relatively good certainty. A summary of the measured and estimated asphalt, aggregate base, road subgrade conditions is shown in Appendix A: Sheets C2-C13 and Table 1.

The survey data indicate that in areas where an asphalt surface is present asphalt thickness is typically 1.5"-2". In road segments where repairs have taken place, the asphalt thickness generally increases, with the thickest measured asphalt at 6.25" in a repaired segment. Asphalt was typically dry with partial cracking visible on the road surface, areas of apparent subgrade failure show larger arcuate shaped cracking along the perimeter of the failing area as well as alligator cracking along some sections. It appears that repairs on the roadway typically consist of additional layer(s) of asphalt being placed on top of a failing section of road to make grade/alignment adjustment to bring the roadway surface back up to grade. Road segments with newer asphalt have a higher asphalt density, less cracking, and higher oil content.

Inferred from the road core and GPR data correlation, it appears that most of the surveyed road segment is underlain by between 4" to 6" of aggregate base rock. Recently repaired areas have up to 1' of base. The directly observed aggregate base rock is typically a cohesionless medium dense to dense ¾" minus gravel (Appendix A: Table 1).

The measured native fill thickness along the surveyed road ranges from 0' to 7.5' with the thickest areas being associated with placement of culverts and fill across drainages and swales. The native fill thickness also varies from lane to lane as most of the roadway required the use of cut and fill construction methods in order to provide a level road surface and proper road alignment for vehicle traffic. Fill material most commonly consists of locally or adjacently sourced native soil and rock placed during original road building efforts. Fill material typically consists of cohesive sandy/gravelly/cobble clay with firm to very stiff consistency (Appendix A: Table 1). For the directly observed native fill, the sand is very fine to coarse, the clay has medium to high plasticity, gravels are less than 1" in diameter, and cobbles are about 2.5" in diameter.

For fill areas of the road prism, below the aggregate base rock or native fill material, there is in-place native soil and rock. Most of the in-place material is hard volcanic rock varying from fresh to very weathered into clay with gravel and cobbles (Appendix A: Table 1).

No groundwater was observed within the road cores or GPR data (Appendix A: Road Core Logs). Groundwater levels can fluctuate from season to season and year to year. Given that this survey was completed during a dry time of year, shallow groundwater may be present during wet times of the year.

Conclusions

Overall, the surveyed road segments with full bench cuts are founded on hard bedrock and are relatively stable. Road segments constructed using native fill are relatively unstable. The segments that are full bench cuts have good to fair road surface and subgrade conditions whereas segments that are cut/fill or all fill have fair to poor surface and subsurface conditions.

Based on the data interpretations and visual road assessment, there are several likely main causes of poor road surface condition. Those likely causes are road prisms that are founded on relatively uncompacted expansive clay soil, these is very little or no aggregate base present under the asphalt, the asphalt surface layers are relatively thin, and the asphalt is relatively old and has little to no maintenance since being constructed. Road segments assessed to be in poor condition tend to have an irregular surface, less aggregate base rock, and old and dry asphalt (e.g., alligator cracking). Also, road segments with a combination of cut and fill (i.e., sliver fills) tend to have outboard edge failures with arcuate shaped drops in the road prism. These fill failures are likely result from a lack of keyways into in-place native rock and soil on the outboard edge of the road, poor compaction of expansive clay soils, and heavy live loads. In addition, a white volcanic ash layer, acting as a landslide slip plane, was noted at several locations at 5’ to 7’ bgs.

Based on GPR and road core data (measured thicknesses and strength of roadway materials) and visual inspection, it is likely that the road could fail under heavy construction loads at several locations due to fill slope failure.

Sheet 1 denotes locations that are already failing (i.e. excessive disproportionate settlement, active tension cracks on the surface and visible movement of downslope power poles, trees, etc.) with downslope residences, water bodies (river or lake), and downslope roads which could be directly impacted by a slope failure and/or heavy vehicle crash(s). Hence, each location has different downslope consequences in the event of a road failure. Overall, road failures are most likely to occur on the outboard (downslope) edge of the road. Most of the road prism is a combination of cut and fill (i.e., average road prism section) with fill on the outboard side.

Given that medium to highly plasticity clay soil was used as fill material to construct Copco Road, the road stability decreases during wet periods of the water year when the clay is saturated. The safe bearing capacity varies by season depending on moisture levels and decreases during wet periods or when the fill soils are saturated.

The existing asphalt road surface is relatively thin and has a low oil content (dry, friable), the pavement provides less vertical and lateral support during the hot summer periods. The relatively dry nature of the asphalt also allows for increased cracking of the surface which intern creates conduits for surface water to infiltrate the subgrade materials. This contributes to road fill failures in active failures to continually occur during both summer and winter.

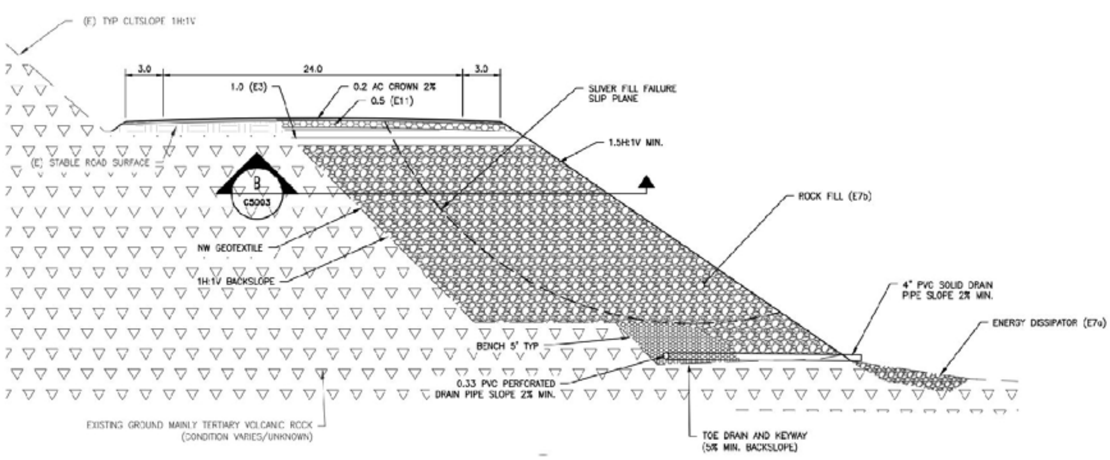
Using existing data and current downslope configurations(i.e. possible impacted entities) specific road segments/area of known failure were assessed as to their relative risk of failure and possible failure impacts to that area. Three rankings were used, they are as follows:

"Moderate Risk" - A road segment/area with active arcuate tension cracks in the road surface that extend into the road subgrade, other signs of visible failure and downslope consequences.

"High Risk" - A road segment/area with active arcuate tension cracks in the road surface that extend into the road subgrade, measured weak soils, relatively shallow fill, other signs of visible failure and downslope consequences.

"Very High Risk" - A road segment/area with active arcuate tension cracks in the road surface that extend into the road subgrade, visible drop in road grade, measured weak soils, deep fill, and some other signs of visible failure and downslope consequences.

In total 24 areas of Moderate, High, and Very High Risk were identified. Of those 8 ranked as Very High Risk, 11 as High Risk, and 5 as Moderate Risk level. All sites were numbered sequentially from west to east on Copco Road and can be seen on Figure 1 and GPR Radargrams summaries for the Very High Risk segments/areas are in Appendix A. Several other locations were identified as part of the road survey that show signs of active road pavement and subgrade failure which are not captured on this Sheet due to the likelihood of negative consequences (i.e. road prism slumps or minor movement of the road prism which can be repaired without major interruption or safety risk.) There are several existing road fill failure repair sites along this road. Most were repaired using rock fill (see below) or moving road further into the hill-slope/cut-slope.



Limitations and Assumptions

The analysis and conclusions presented in this report have been conducted according to current geologic and engineering practice and the standard of care exercised by reputable professional consultants performing similar tasks in this area. The conclusions made are preliminary and subject to change. This is a preliminary summary and interpretation of these data. No other warranty, expressed or implied, is made regarding the conclusions and opinions expressed in this report. Variations may exist and conditions not observed or described in this report may be encountered during future assessments. GSI's conclusions are based on an analysis of the observed conditions and data available at the time of this report.

Data for this survey is inherently limited given the density of direct measurements (i.e., one road core per mile of survey). The point data at road core locations have the most objective and greatest certainty in the accuracy of conclusions made from these data. GPR data have the most uncertainty given the indirect nature of non-visual testing. The GPR data do have the most coverage relative to the road core data. The correlation between road core data and GPR data is limited to extrapolation between road cores. The conclusions made herein assume that asphalt composition and thickness between known points is relatively constant and that the aggregate base material is from the same source with similar thickness, and that native fill material is the same from station to station. Also assumed is that the aggregate base differs greatly from native fill material in gradation, density, and plasticity. It follows that fill compaction and or composition varies from adjacent native fill and in place material(s) allowing for differentiation with the return signal detected by the GPR equipment. As of this report, the laboratory testing of soil and rock samples has not occurred and is forthcoming.

Risk assessment of road segments/areas are limited to area that are known to be or may be in the process of roadway failure. It is possible that a road/subgrade failure is occurring in areas outside of those described or that a failure could occur at any point or time in the roadway surface. GSI assumes no liability in the event that a roadway failure occurs at any time along any segment of the roadway or road subgrade area.

APPENDIX A

Table 1. Road Core Data Summary and Field Tested Parameters.

STA	Distance (feet)	Borehole Number	Depth (feet)	Depth (meters)	GW Depth (feet)	Type	Material Type	Cohesion Type	N	N60	N1,60	Blows/ Foot	Field Measured Dry Soil Unit Weight (pcf)	Field Measured Saturated Soil Unit Weight (pcf)	Relative Density (N60)	Friction Angle (N60)	Friction Angle	Undrained Shear Strength (N60) (psf)	Cohesionless Soil Density	Cohesive Soil Consistency
180+60	18,060	RC-CR-001	1.0	0.3	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	119	147	184	114	46.9	49.5		Very Dense	
236+20	23,620	RC-CR-002	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	8	6.4	10.9	8	76	110	47			1,963		Firm
236+20	23,620	RC-CR-002	2.0	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	12	9.6	16.3	12	106	129	53			5,117		Stiff
236+20	23,620	RC-CR-002	4.0	1.2	No Water	Fill	Sandy Clay with Gravel	Cohesive	9	7.2	12.2	9	106	129	43			3,843		Stiff
236+20	23,620	RC-CR-002	5.0	1.5	No Water	Fill	Sandy Clay with Gravel	Cohesive	10	8.0	13.6	100	106	129	44			4,261		Stiff
236+20	23,620	RC-CR-002	7.0	2.1	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	62.8	125	143	188	88	47.0	49.5		Very Dense	
220+57	22,057	RC-CR-003	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	16	12.8	21.8	16	110	131	65			6,809		Very Stiff
220+57	22,057	RC-CR-003	2.0	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	27	21.6	36.7	27	110	131	77			11,508		Very Stiff
220+57	22,057	RC-CR-003	4.0	1.2	No Water	Fill	Sandy Clay with Gravel	Cohesive	25	20.0	8.0	25	110	131	41			10,652		Very Stiff
315+66	31,566	RC-CR-004	1.0	0.3	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	120	147	185	114	46.9	49.5		Very Dense	
386+17	38,617	RC-CR-005	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	45	36.0	61.2	45	131	133	120	38.0	41.0		Dense	
386+17	38,617	RC-CR-005	1.7	0.5	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	125	147	188	114	46.9	49.5		Very Dense	
430+68	43,068	RC-CR-006	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	50	40.0	68.0	50	134	137	114	46.9	49.5		Very Dense	
470+56	47,056	RC-CR-007	0.5	0.2	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	120	147	185	114	46.9	49.5		Very Dense	
507+44	50,744	RC-CR-008	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	13	10.4	17.7	13	106	129	59			5,535		Stiff
507+44	50,744	RC-CR-008	3.0	0.9	No Water	Fill	Sandy Clay with Gravel	Cohesive	5	4.0	6.8	5	76	110	34			2,130		Firm
507+44	50,744	RC-CR-008	4.5	1.4	No Water	Fill	Sandy Clay with Gravel	Cohesive	16	12.8	21.8	16	110	131	65			6,809		Very Stiff
507+44	50,744	RC-CR-008	6.0	1.8	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	120	147	185	114	46.9	49.5		Very Dense	
552+05	55,205	RC-CR-009	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	33	26.4	44.9	33	112	133	90			14,056		Hard
552+05	55,205	RC-CR-009	2.5	0.8	No Water	Fill	Sandy Clay with Gravel	Cohesive	13	10.4	17.7	13	106	129	59			5,535		Stiff
698+00	69,800	RC-CR-009A	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	17	13.6	23.1	17	110	131	67			7,205		Very Stiff
698+00	69,800	RC-CR-009A	2.5	0.8	No Water	Fill	Sandy Clay with Gravel	Cohesive	20	16.0	27.2	20	110	131	65			8,521		Very Stiff
698+00	69,800	RC-CR-009A	4.5	1.4	No Water	Native Rock	Weathered Volcanic	Cohesionless	28	22.4	38.1	28	126	118	71	41.0	43.0		Medium Dense	
739+58	73,958	RC-CR-010	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	19	15.2	25.8	19	110	131	70			8,104		Very Stiff
739+58	73,958	RC-CR-010	2.0	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	40	32.0	54.4	40	112	133	92			17,043		Hard
831+92	83,192	RC-CR-010A	1.0	0.3	No Water	Fill	Sandy Clay with Gravel	Cohesive	19	15.2	25.8	19	110	131	70			8,104		Very Stiff
831+92	83,192	RC-CR-010A	2.0	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	16	12.8	21.8	16	110	131	65			6,809		Very Stiff
831+92	83,192	RC-CR-010A	4.0	1.2	No Water	Fill	Sandy Clay with Gravel	Cohesive	10	8.0	13.6	10	106	129	44			4,261		Stiff
753+85	75,385	RC-CR-010B	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	18	14.4	24.5	18	111	113	73	28.0	30.0		Medium Dense	
753+85	75,385	RC-CR-010B	2	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	7	5.6	9.5	7	76	110	41			2,987		Firm
753+85	75,385	RC-CR-010B	3.5	1.1	No Water	Fill	Sandy Clay with Gravel	Cohesive	6	4.8	8.2	6	76	110	36			2,548		Firm
753+85	75,385	RC-CR-010B	5	1.5	No Water	Fill	Sandy Clay with Gravel	Cohesive	3	2.4	4.1	3	76	110	25			1,274		Firm
861+30	86,130	RC-CR-011	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	24	19.2	32.6	24	115	118	83	30.0	32.0		Medium Dense	
861+30	86,130	RC-CR-011	2	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	10	8.0	13.6	10	106	129	44			4,261		Stiff
861+30	86,130	RC-CR-011	3.5	1.1	No Water	Fill	Sandy Clay with Gravel	Cohesive	22	17.6	30.0	22	110	131	65			9,378		Very Stiff
861+30	86,130	RC-CR-011	5	1.5	No Water	Fill	Sandy Clay with Gravel	Cohesive	20	16.0	27.2	20	110	131	60			8,521		Very Stiff
918+36	91,836	RC-CR-011A	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	35	28.0	47.6	35	123	126	99	33.0	36.0		Dense	
918+36	91,836	RC-CR-011A	2	0.6	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	68.0	120	147	185	114	46.9	49.5		Very Dense	
960+49	96,049	RC-CR-012	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	28	22.4	38.1	28	118	121	89	31.0	33.0		Medium Dense	
960+49	96,049	RC-CR-012	2.5	0.8	No Water	Native Rock	Weathered Volcanic	Cohesionless	37	29.6	50.3	37	134	125	86	44.0	46.0		Dense	
1019+33	101,933	RC-CR-013	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	38	30.4	51.7	38	125	128	102	34.0	36.0		Dense	
1019+33	101,933	RC-CR-013	2	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	50	40.0	68.0	50	112	133	113			21,303		Hard
1059+30	105,930	RC-CR-014	0.5	0.2	No Water	AB	Aggregate Base Rock	Cohesionless	16	12.8	21.8	16	110	112	69	31.0	33.0		Medium Dense	
1059+30	105,930	RC-CR-014	2	0.6	No Water	Fill	Sandy Clay with Gravel	Cohesive	18	14.4	24.5	18	110	131	64			7,665		Very Stiff
1059+30	105,930	RC-CR-014	3.5	1.1	No Water	Fill	Sandy Clay with Gravel	Cohesive	24	19.2	32.6	24	110	131	68			10,234		Very Stiff
1059+30	105,930	RC-CR-014	5	1.5	No Water	Fill	Sandy Clay with Gravel	Cohesive	11	8.8	15.0	11	106	129	46			4,678		Stiff
1059+30	105,930	RC-CR-014	6.5	2.0	No Water	Fill	Sandy Clay with Gravel	Cohesive	32	25.6	41.2	32	112	133	72			13,638		Hard
1059+30	105,930	RC-CR-014	7.9	2.4	No Water	Native Rock	Weathered Volcanic	Cohesionless	50	40.0	59.0	120	141	185	86	47.0	50.0		Very Dense	

SHEET INDEX

DRAWING #	TITLE	REVISION	DATE
C1	COPCO ROAD GPR PROJECT AREA		2/19/20
C2	COPCO ROAD STA 157+00.0–236+20.0		2/19/20
C3	COPCO ROAD STA 236+20.0–328+60.0		2/19/20
C4	COPCO ROAD STA 328+60.0–407+80.0		2/19/20
C5	COPCO ROAD STA 394+60.0–473+80.0		2/19/20
C6	COPCO ROAD STA 473+80.0–539+80.0		2/19/20
C7	COPCO ROAD STA 539+80.0–619+00.0		2/19/20
C8	COPCO ROAD STA 632+20.0–711+40.0		2/19/20
C9	COPCO ROAD STA 711+40.0–790+60.0		2/19/20
C10	COPCO ROAD STA 790+60.0–869+80.0		2/19/20
C11	COPCO ROAD STA 869+80.0–949+00.0		2/19/20
C12	COPCO ROAD STA 949+00.0–1028+20.0		2/19/20
C13	COPCO ROAD STA 2028+20.0–1092+00		2/19/20

PLAN
1"= 2000'

0 2000 4000 6000
SCALE: 1" = 2000'

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD GPR
PROJECT AREA

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

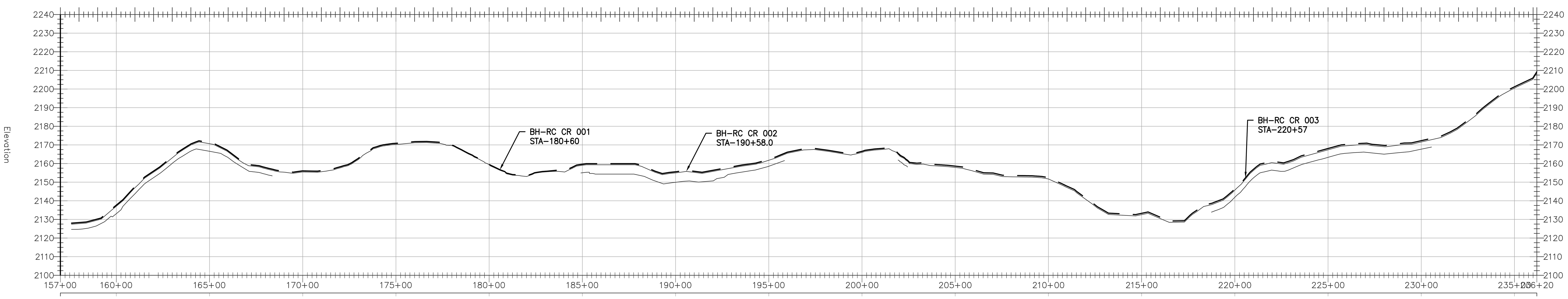
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

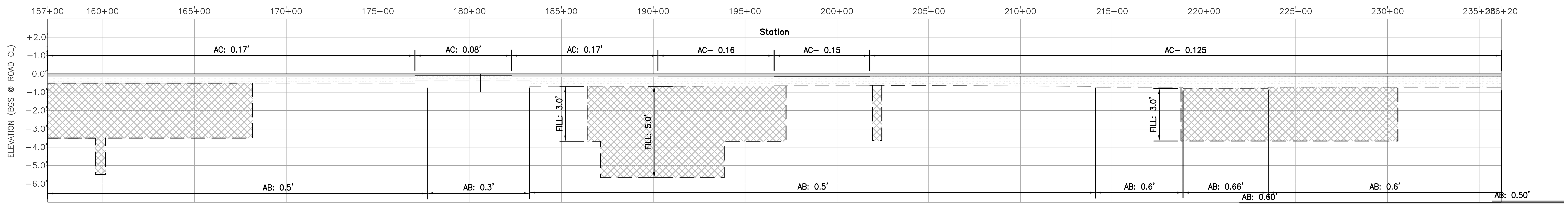
CHECKED: JF

SHEET: **C1**



ELEVATION

1"= 300'; 1H = 10V



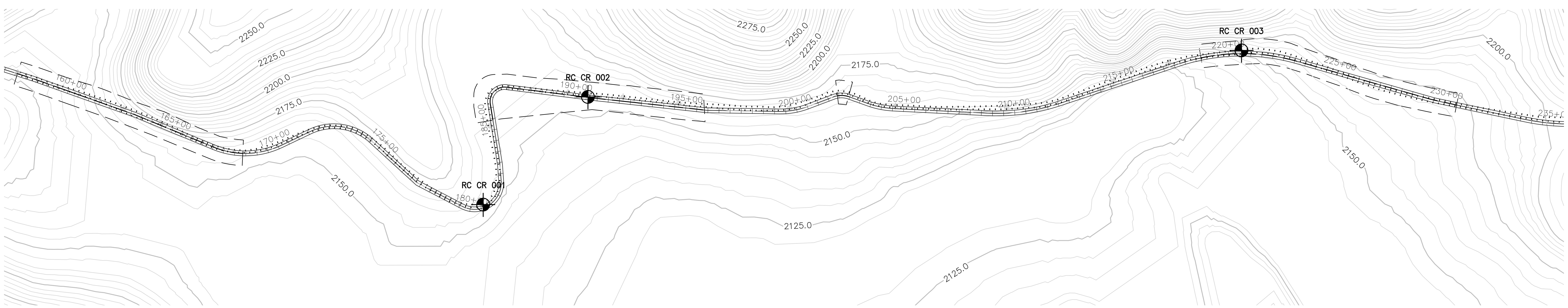
ELEVATION

HORZ. SCALE 1"= 300'

NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'-4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.

LEGEND

- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS-25'



PLAN

1"= 300'

0 300 600 900

SCALE: 1" = 300'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5340, 550) EASTBOUND LANE;
(4730, 5875) WEST BOUND LANE.

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA 157+00-236+20 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

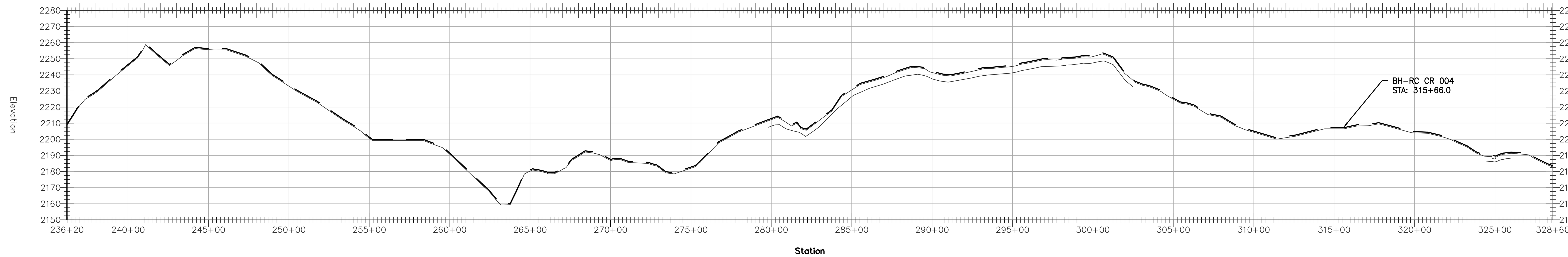
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

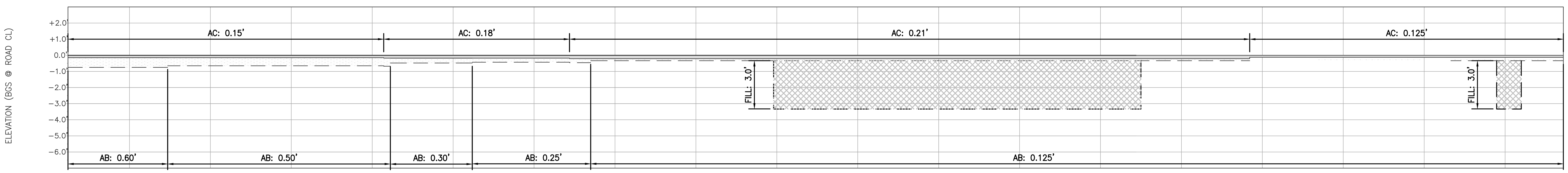
CHECKED: JF

SHEET: **C2**



ELEVATION

1"= 350' 1 H = 10 V



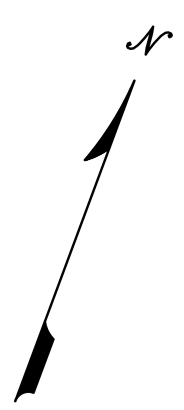
ELEVATION

HORZ. SCALE 1"= 350'

NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'–2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'–4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'–6'.

LEGEND

	EXISTING ASPHALT
	EXISTING AGGREGATE BASE Rx
	EXISTING FILL
	COPCO GPR MAPPING TICKS–25'



NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (10665, 5365) EASTBOUND LANE;
(5380, 5920, 4730) WEST BOUND LANE.

PLAN

1"= 350'

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227–8963
FAX: (530) 926–8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA
236+20–328+60
PLAN SECTION &
PROFILE

REVISIONS:

PROJECT NO:
190725

ISSUE DATE:
2/19/20

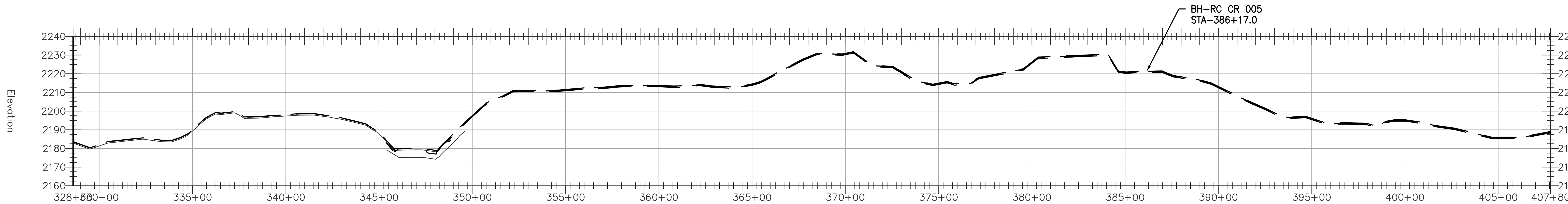
SCALE:
AS NOTED

DRAWN BY:
KF

ENGINEERED:

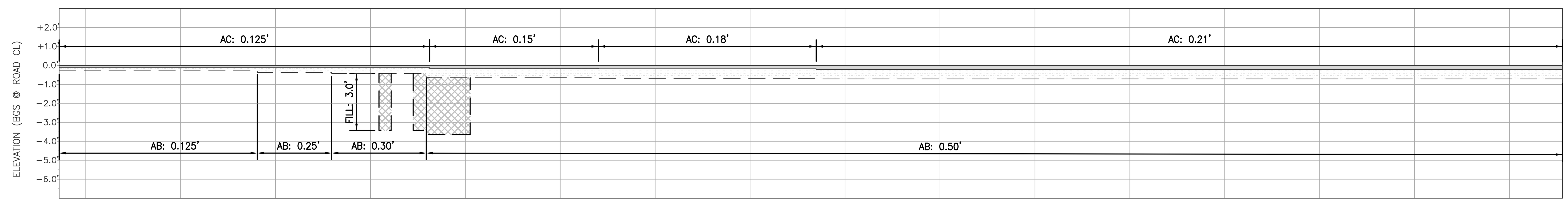
CHECKED:
JF

SHEET:
C3



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 300'

NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
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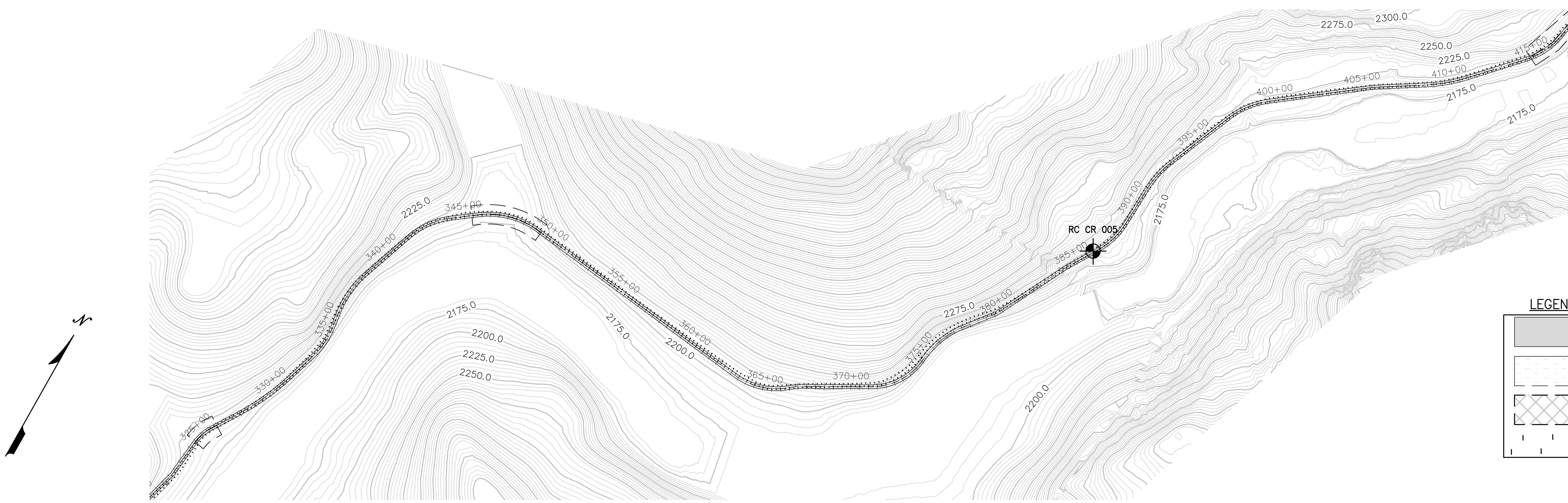
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.



LEGEND

	EXISTING ASPHALT
	EXISTING AGGREGATE BASE Rx
	EXISTING FILL
	COPCO GPR MAPPING TICKS-25'

PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5340, 550) EASTBOUND LANE;
(4730, 5875) WEST BOUND LANE.

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:

COPCO ROAD STA 328+60-407+80 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

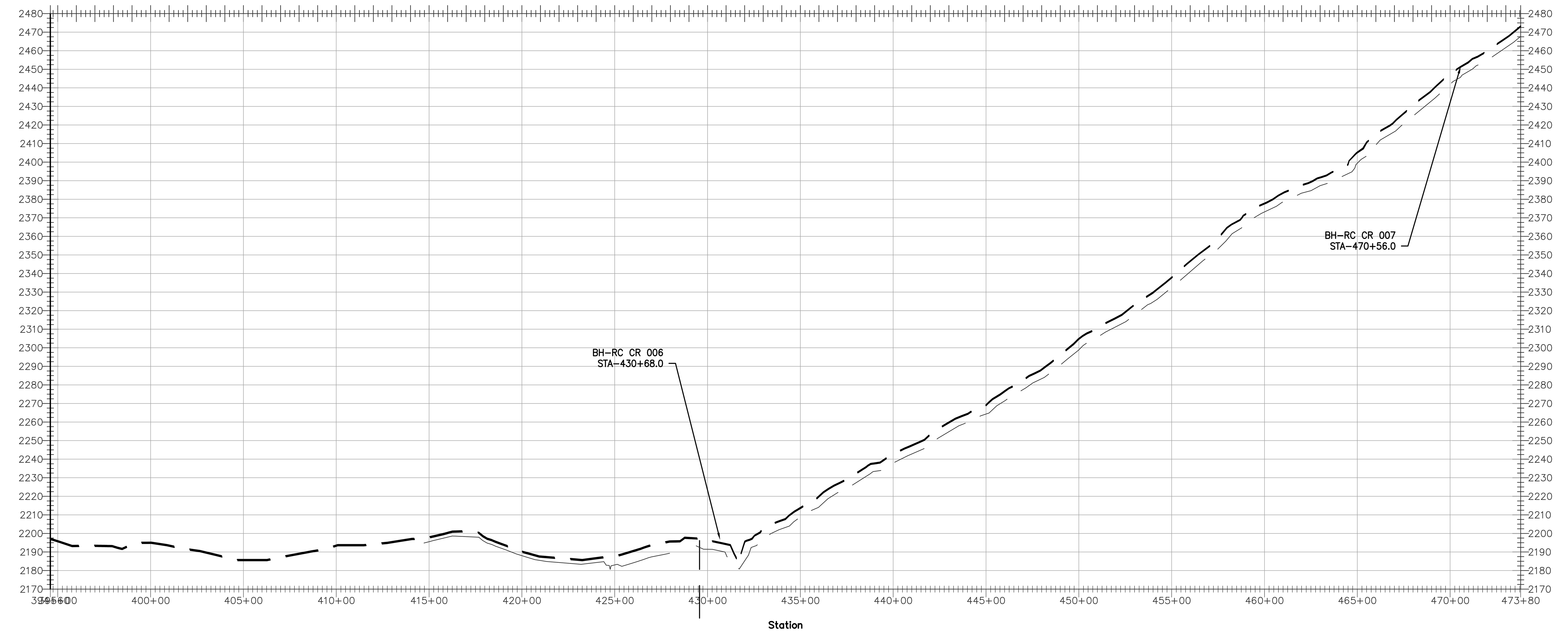
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

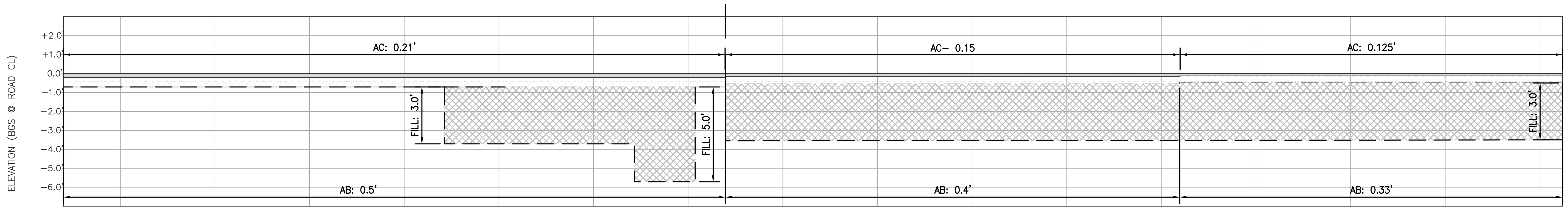
CHECKED: JF

SHEET: C4



ELEVATION

1"= 350' 1 H = 10 V

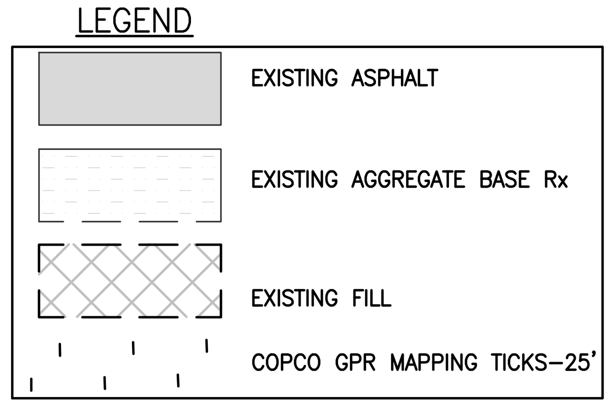


ELEVATION

HORZ. SCALE 1"= 350'

NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'-4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.

LEGEND



PLAN

1"= 350

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5546, 5520) EASTBOUND LANE;
(5550, 5521) WEST BOUND LANE.

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA 394+60-473+80 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

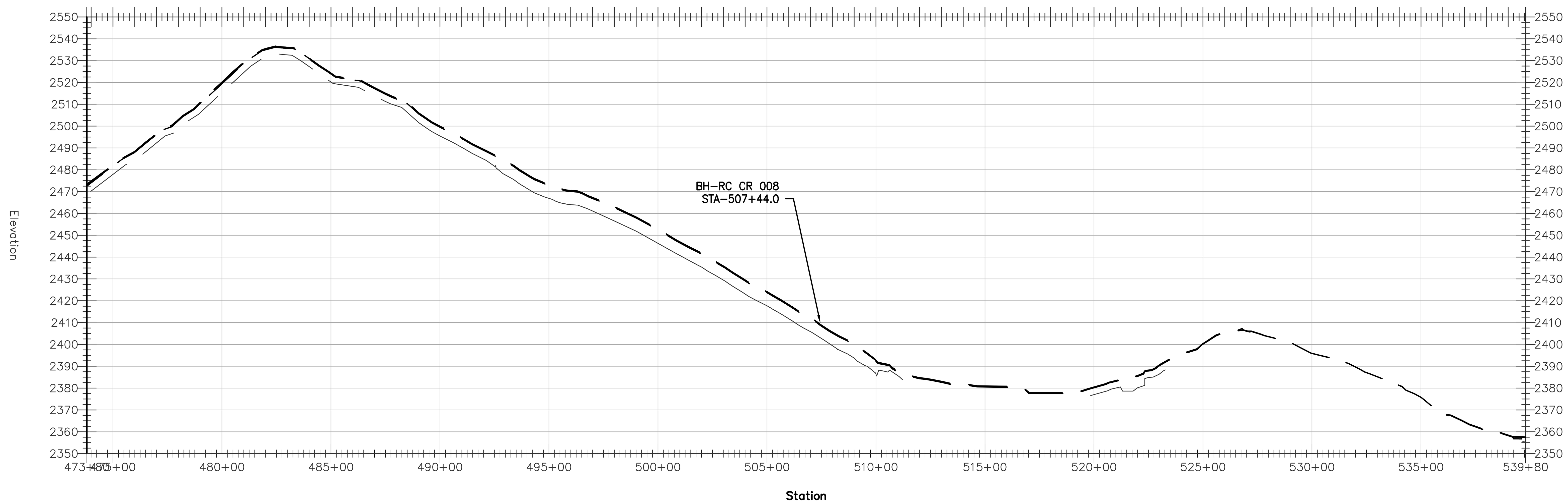
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

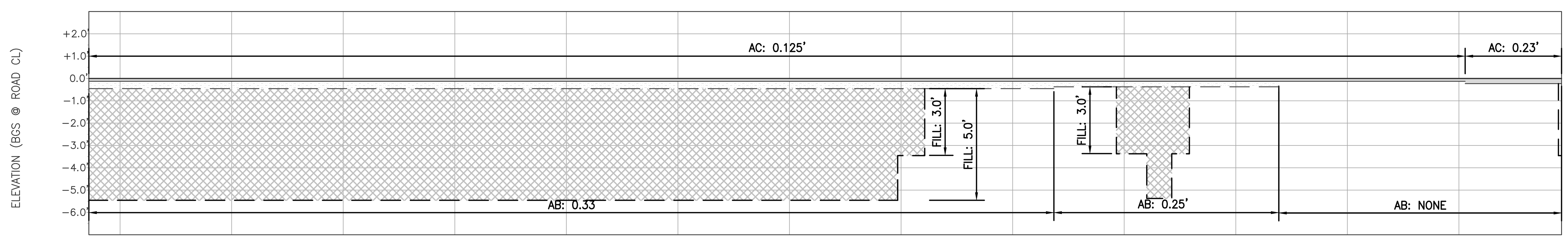
CHECKED: JF

SHEET: **C5**



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 350'

NOTES:

1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'-4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.

LEGEND

	EXISTING ASPHALT
	EXISTING AGGREGATE BASE Rx
	EXISTING FILL
	COPCO GPR MAPPING TICKS-25'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5384, 4632, 5546) EASTBOUND LANE;
(5420, 900, 4430, 5550) WEST BOUND LANE.

PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA
473+80-539+80
PLAN SECTION &
PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

SCALE: AS NOTED

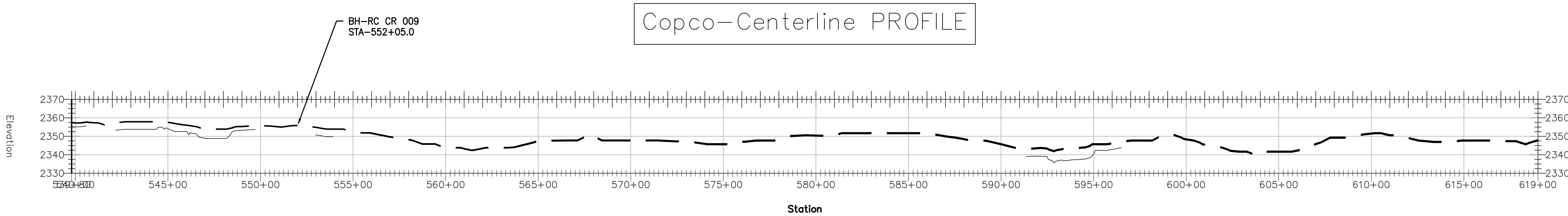
DRAWN BY: KF

ENGINEERED:

CHECKED: JF

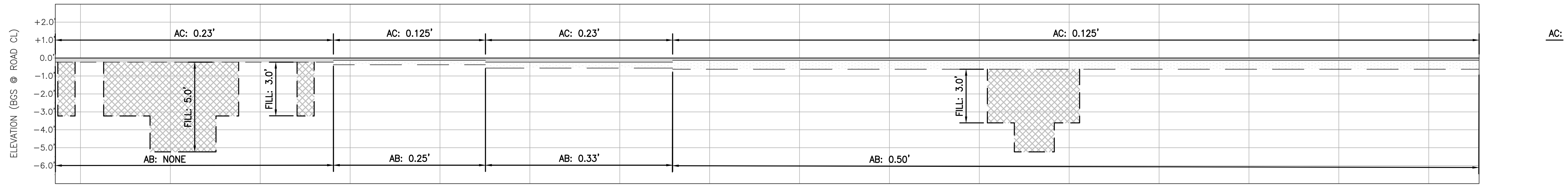
SHEET: C6

Copco–Centerline PROFILE



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 350'

NOTES:
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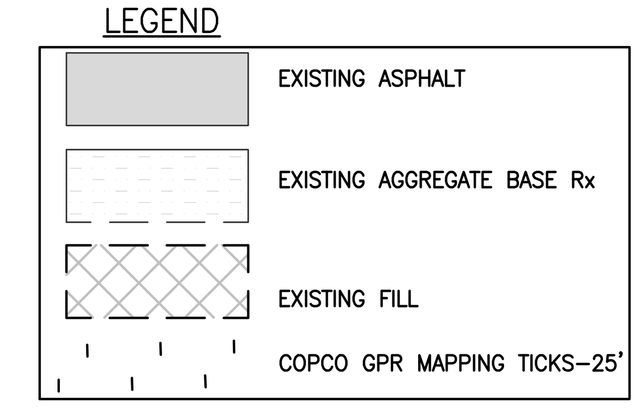
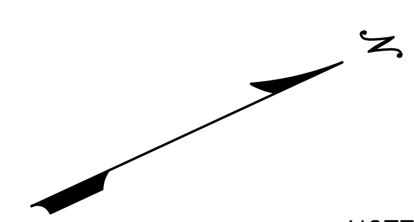
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'–2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'–4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'–6'.

LEGEND

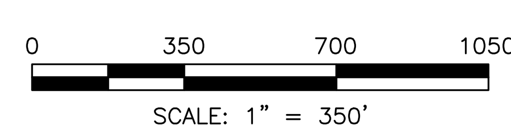




NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (1090, 5360) EASTBOUND LANE;
(4200, 5400) WEST BOUND LANE.

PLAN

1"= 350



GEOSERV

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P.O. BOX 831
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FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA
539+80–619+00
PLAN SECTION &
PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

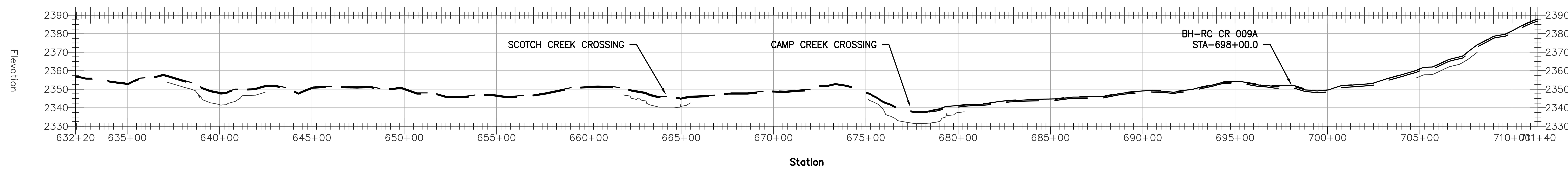
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

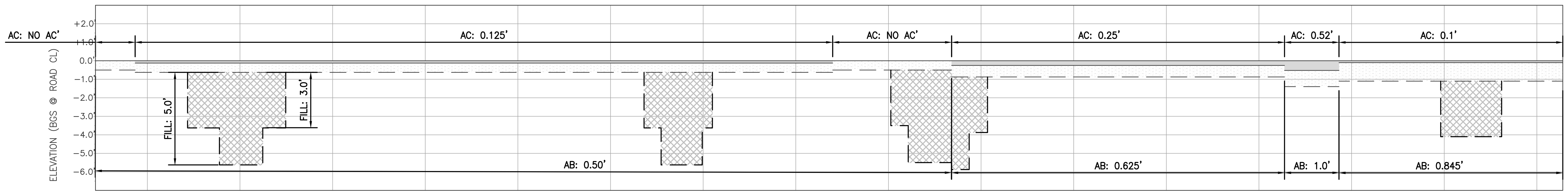
CHECKED: JF

SHEET: **C7**



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 350'

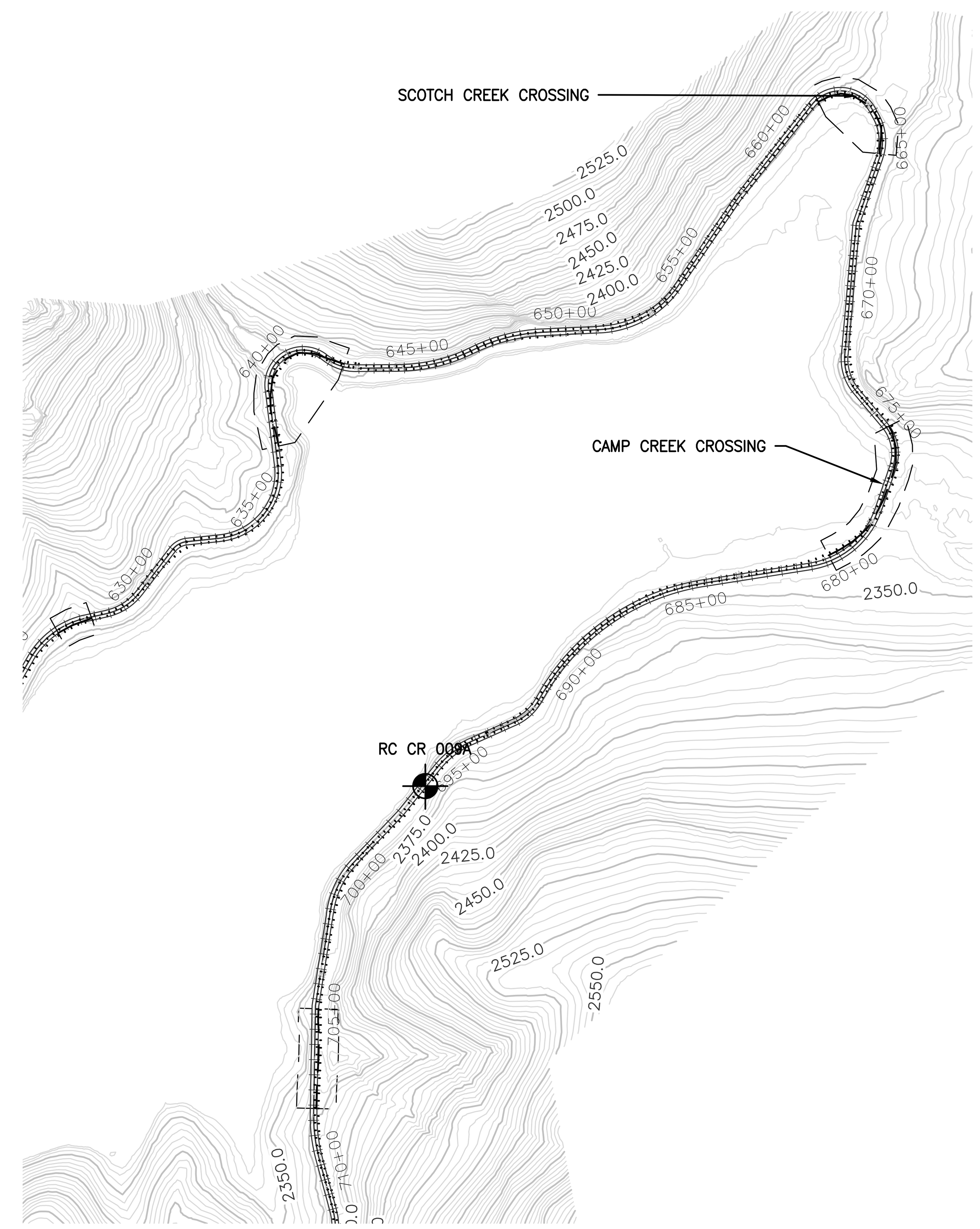
NOTES:

1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'-4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.

LEGEND

- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS-25'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5360, 5310, 5395) EASTBOUND LANE;
(5370, 5340) WEST BOUND LANE.



PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA 632+20-711+40 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

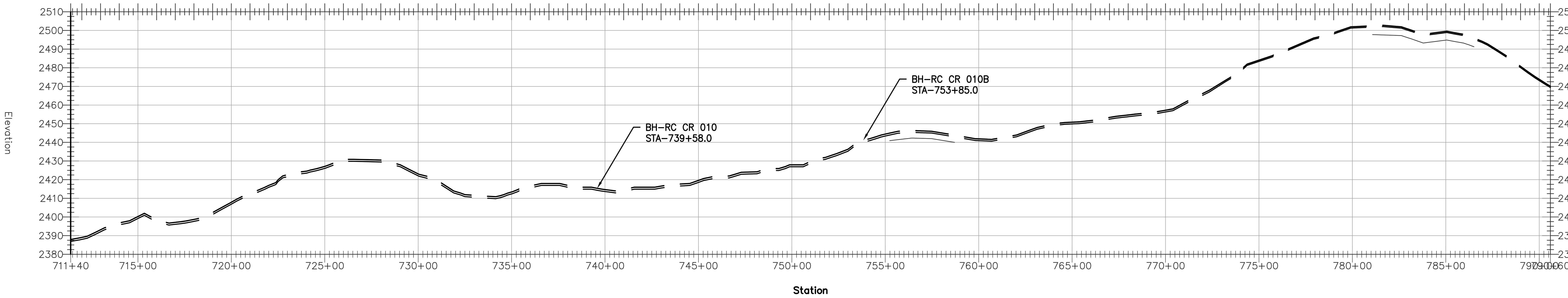
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

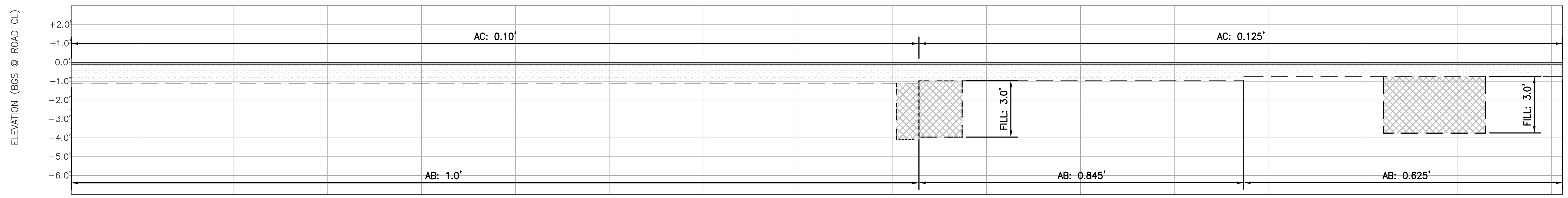
CHECKED: JF

SHEET: C8



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 350'

NOTES:
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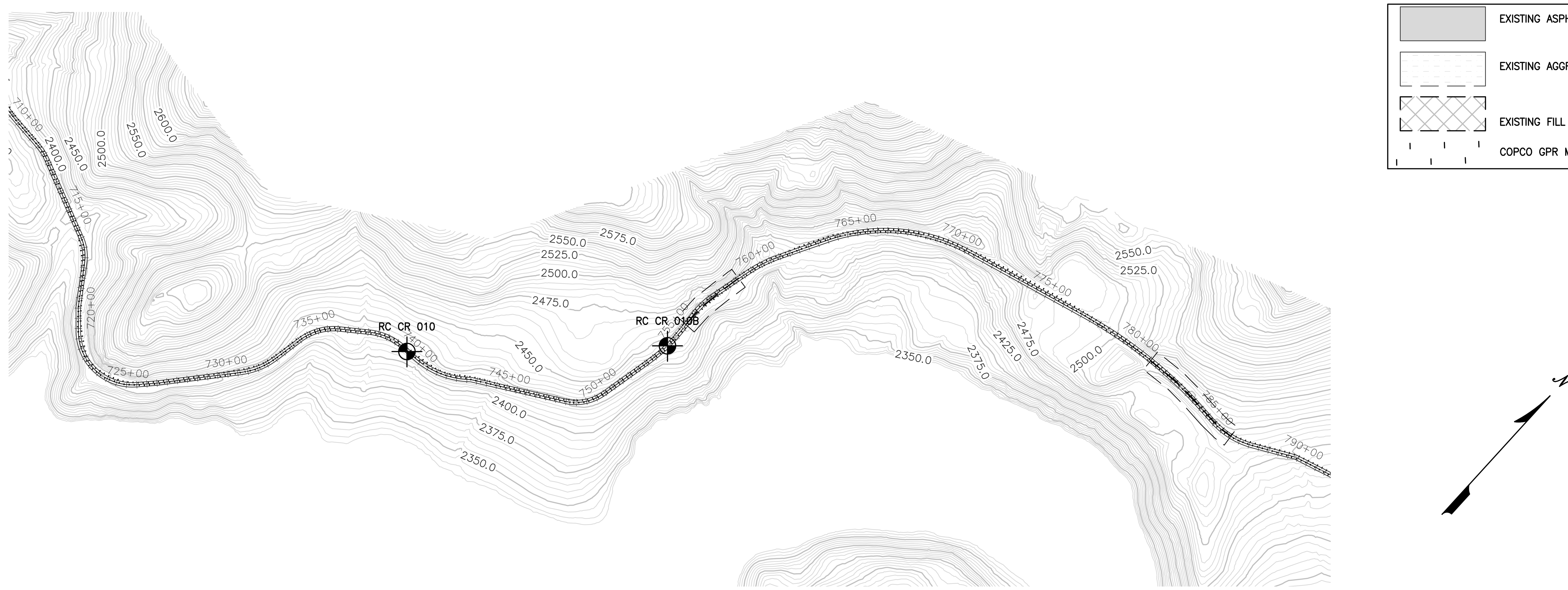
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'–2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'–4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'–6'.

LEGEND

- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS–25'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5310, 5395, 5385) EASTBOUND LANE;
(5365) WEST BOUND LANE.



PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA
711+40–790+60
PLAN SECTION &
PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

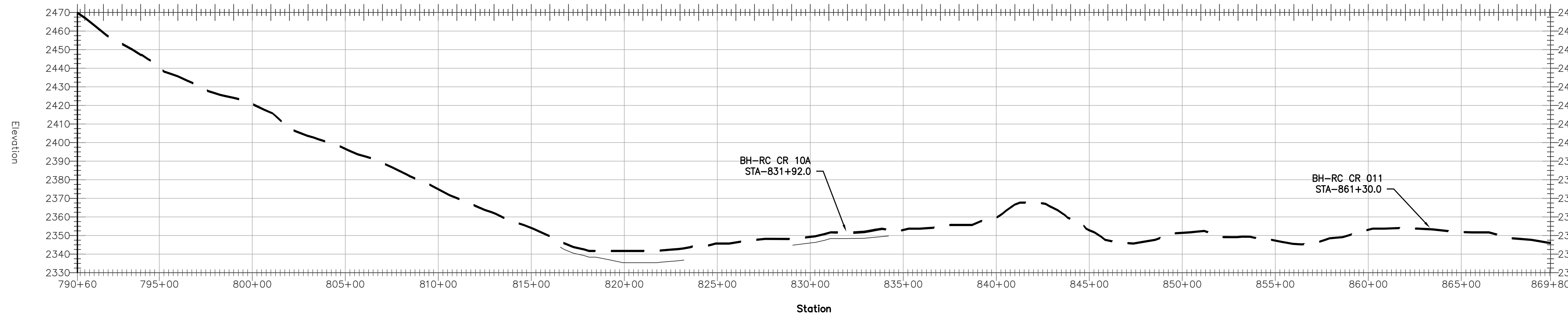
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

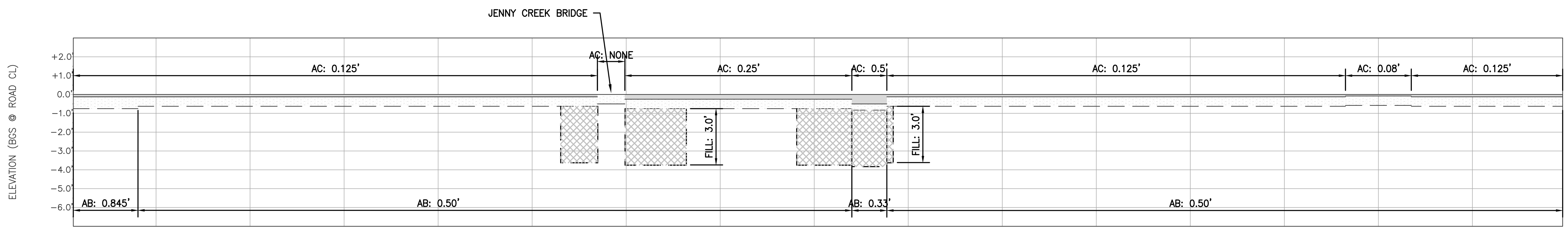
CHECKED: JF

SHEET: C9



ELEVATION

1"= 350' 1 H = 10 V



ELEVATION

HORZ. SCALE 1"= 350'

NOTES:
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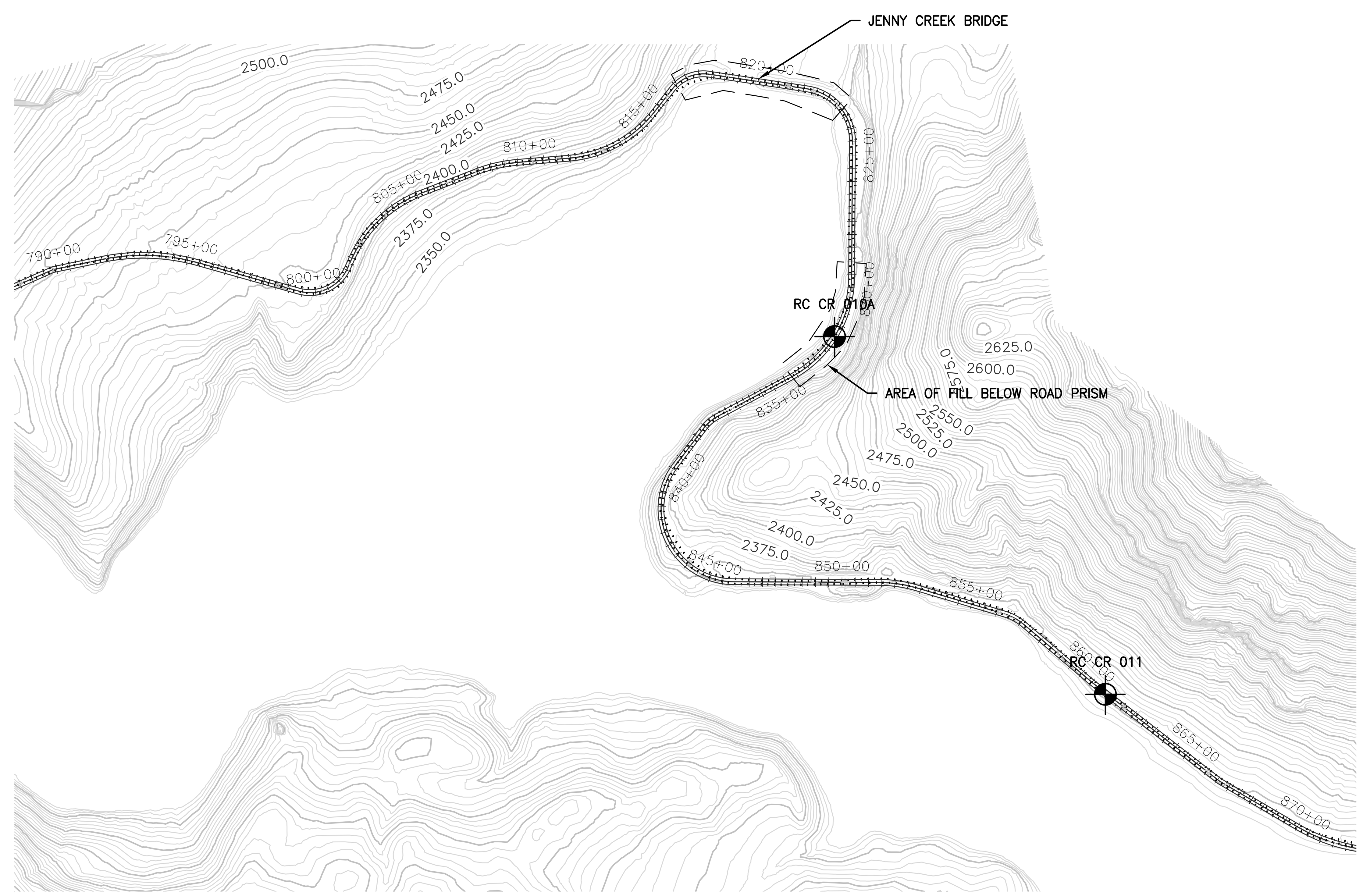


LEGEND

	EXISTING ASPHALT
	EXISTING AGGREGATE BASE Rx
	EXISTING FILL
	COPCO GPR MAPPING TICKS–25'

N

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5385, 5345) EASTBOUND LANE;
(5530, 3565) WEST BOUND LANE.

PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227–8963
FAX: (530) 926–8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:

COPCO ROAD STA
790+60–869+80
PLAN SECTION &
PROFILE

REVISIONS:

PROJECT NO:
190725

ISSUE DATE:
2/19/20

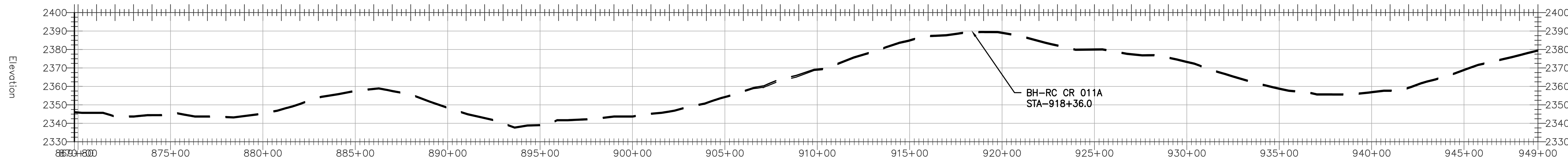
SCALE:
AS NOTED

DRAWN BY:
KF

ENGINEERED:

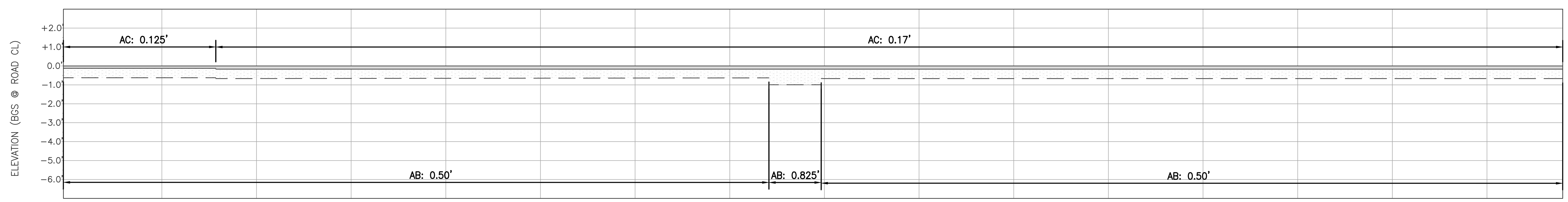
CHECKED:
JF

SHEET:
C10



ELEVATION

1"= 350' 1 H = 10 V



NOTES:
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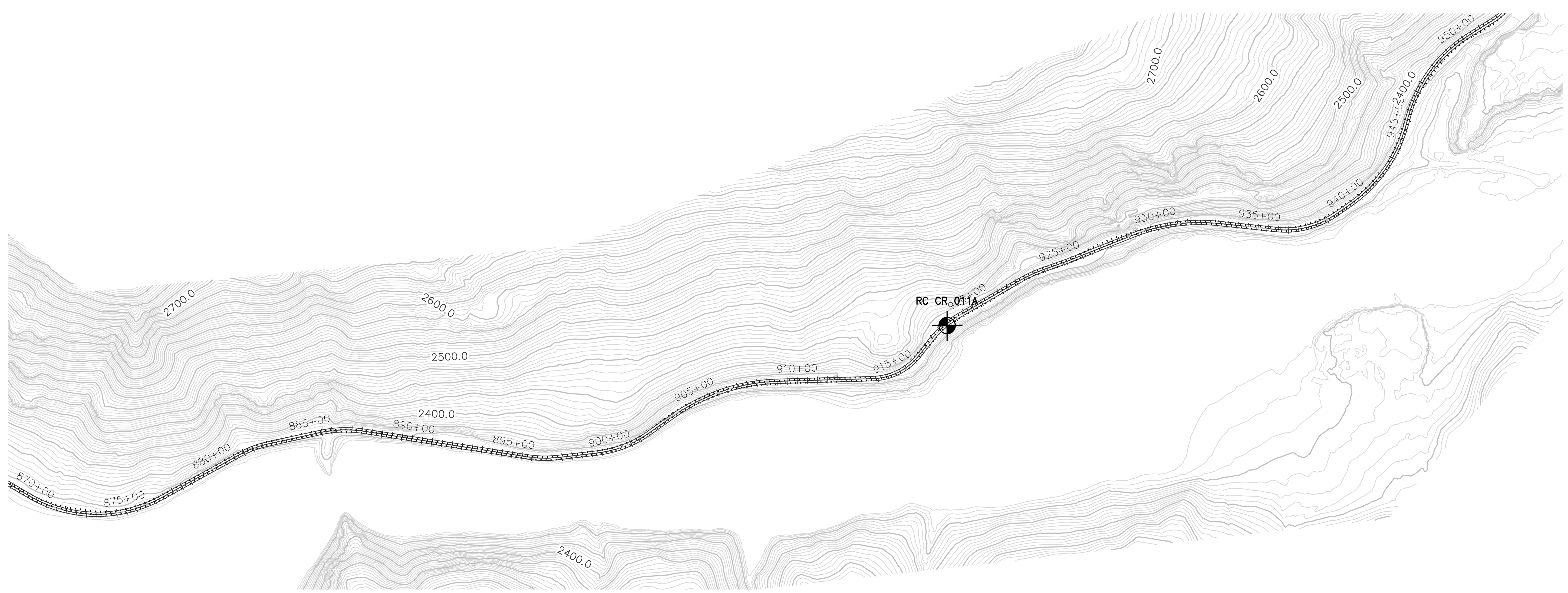
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'–6'.

ELEVATION

HORZ. SCALE 1"= 350'

LEGEND

- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS–25'



NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5345, 5320) EASTBOUND LANE;
(5280, 1921, 3530) WEST BOUND LANE.

PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

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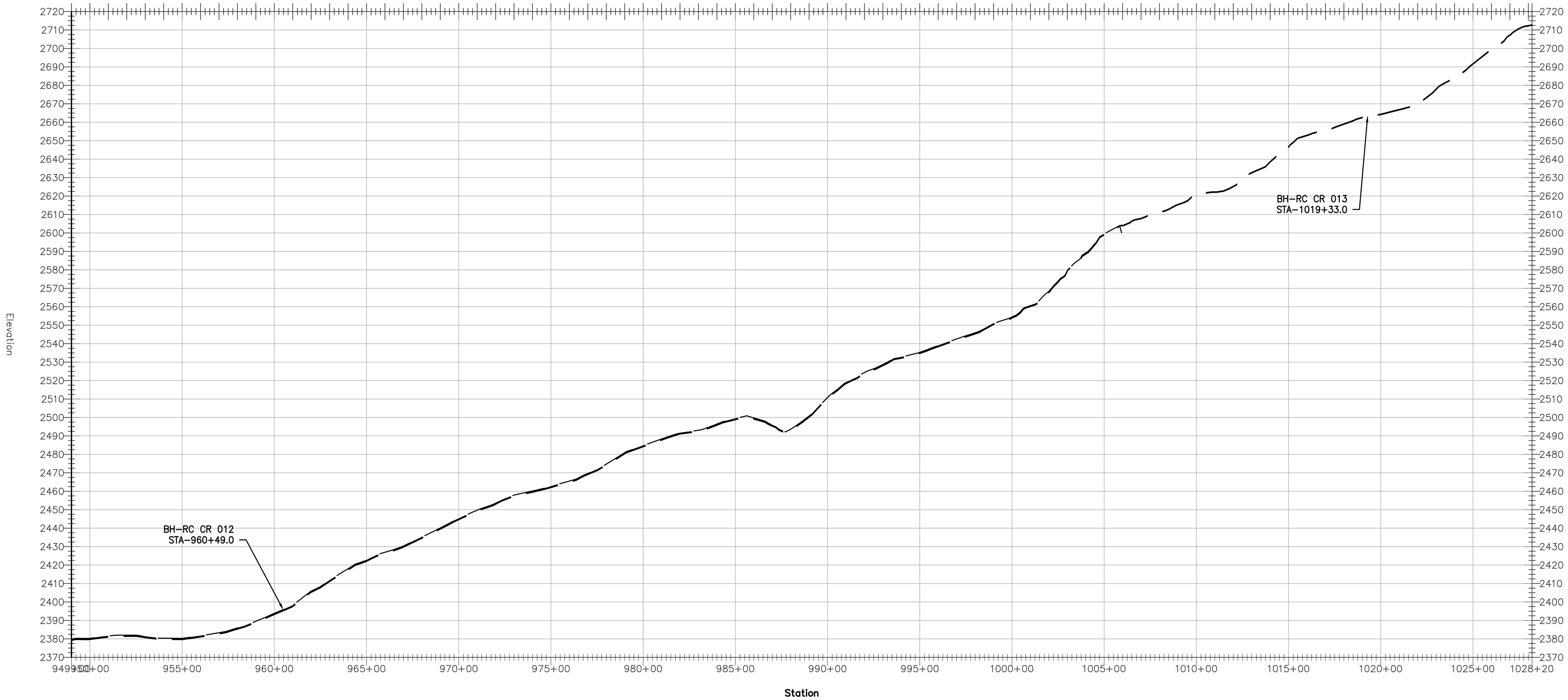
PH: (530) 227–8963
FAX: (530) 926–8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA
869+80–949+00
PLAN SECTION &
PROFILE

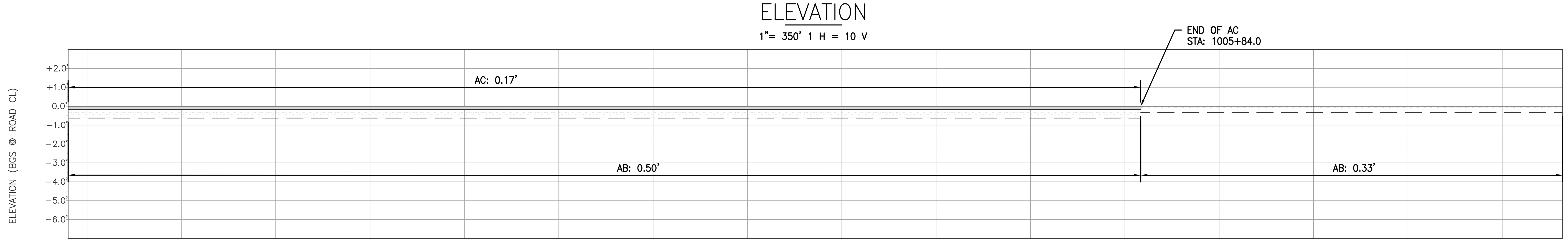
REVISIONS:

PROJECT NO:	190725
ISSUE DATE:	2/19/20
SCALE:	AS NOTED
DRAWN BY:	KF
ENGINEERED:	
CHECKED:	JF
SHEET:	C11



ELEVATION

1"= 350' 1 H = 10 V



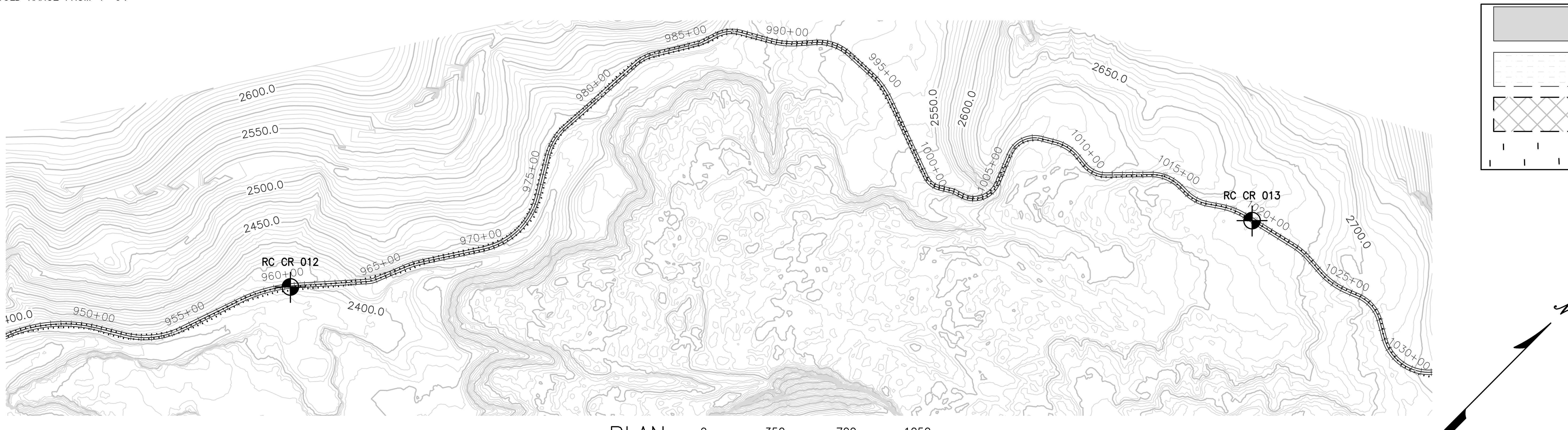
NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'-2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'-4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'-6'.

ELEVATION

HORZ. SCALE 1"= 350'

LEGEND

- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS-25'



NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (5320, 5140) EASTBOUND LANE;
(4435, 5280) WEST BOUND LANE.

PLAN

1"= 350

0 350 700 1050

SCALE: 1" = 350'

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

ROAD CONDITION SURVEY, COPCO ROAD, HORNBROOK, SISKIYOU COUNTY

SHEET NAME:

COPCO ROAD STA 949+00-1028+20 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 2/19/20

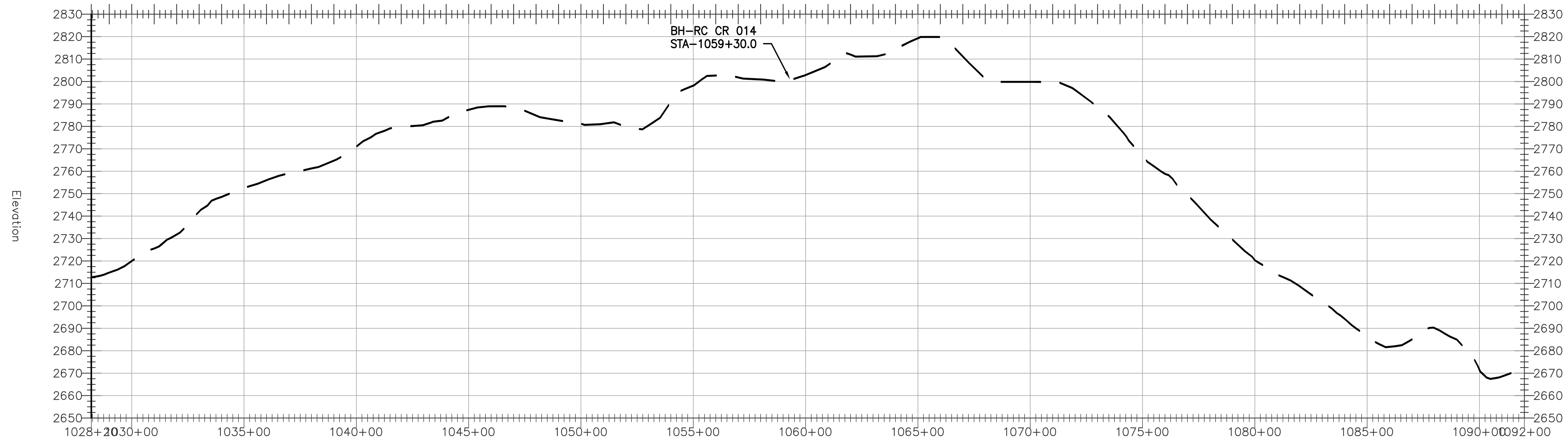
SCALE: AS NOTED

DRAWN BY: KF

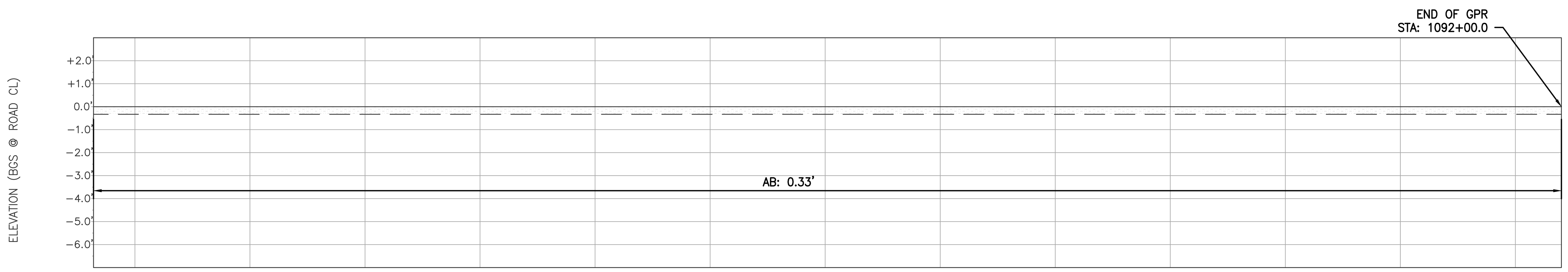
ENGINEERED:

CHECKED: JF

SHEET: C12



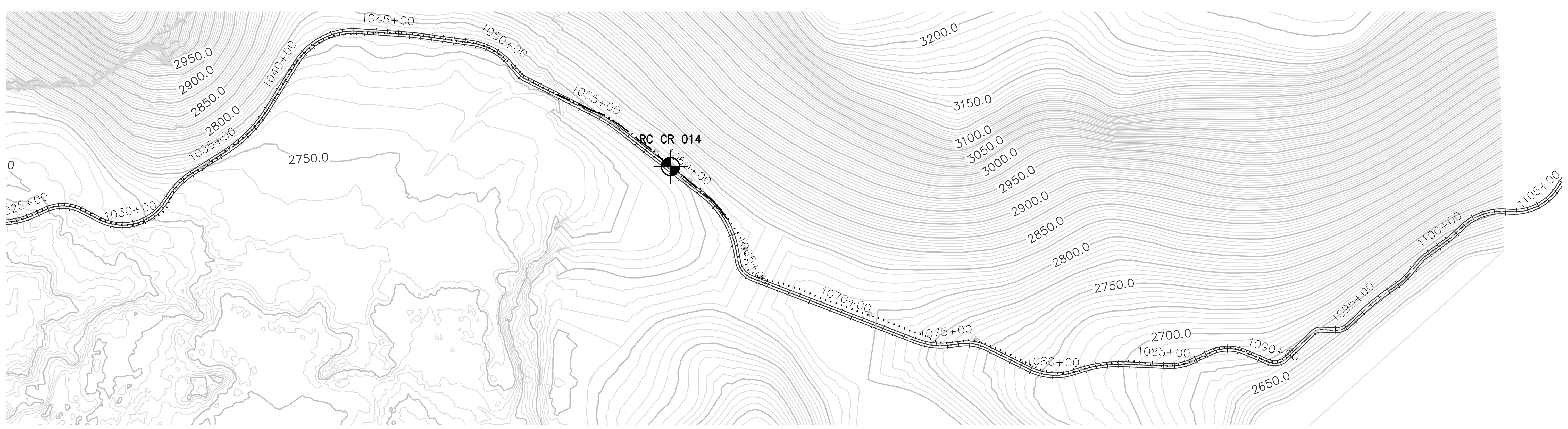
ELEVATION
1"= 350' 1 H = 10 V



ELEVATION
HORZ. SCALE 1"= 350'

NOTES:
1. DIMENSIONS SHOWN ARE OF MATERIAL THICKNESS
2. AREAS OUTSIDE OF MAPPED FILL COULD HAVE A FILL DEPTH RANGING IN 0'–2'.
3. MAPPED FILL AREAS SHOWN AT A 3.0' THICKNESS COULD RANGE FROM 2'–4'.
4. MAPPED FILL AREAS SHOWN AT A 5.0' THICKNESS COULD RANGE FROM 4'–6'.

LEGEND
- EXISTING ASPHALT
- EXISTING AGGREGATE BASE Rx
- EXISTING FILL
- COPCO GPR MAPPING TICKS–25'



PLAN
1"= 350

0 350 700 1050
SCALE: 1" = 350'

NOTE:
FOR GROUND PENETRATION LOGS SEE APPENDIX #
FOOTAGE LOGS: (4660, 2030) EASTBOUND LANE;
NO MAPPING DONE ON THE WEST BOUND LANE.

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227–8963
FAX: (530) 926–8921

ROAD CONDITION SURVEY,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD STA 949+00–1028+20 PLAN SECTION & PROFILE

REVISIONS:

PROJECT NO: 190725
ISSUE DATE: 2/19/20
SCALE: AS NOTED
DRAWN BY: KF
ENGINEERED:
CHECKED: JF
SHEET: C13

BORE HOLE SOIL AND SYMBOL LEGEND

CH - Inorganic Clays of High Plasticity, Fat Clays.

CL - Inorganic Clays or Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays.

Fill - Artificial Fill.

GC - Clayey Gravels, Gravel-Sand-Clay Mix.

GP - Poorly-Graded Gravels and Gravel-Sand Mix, Little or No Fines.

GP - Well-Graded Gravels and Gravel-Sand Mix, Little or No Fines.

OL - Organic Silts and Organic Silty Clays of Low Plasticity.

SC - Clayey Sands, Sand-Clay Mix.

SP - Poorly-Graded Sands and Gravelly Sands, Little or No Fines.

SM - Silty Sands and Sand-Silt Mix.

SW - Well-Graded Sands and Gravelly Sands, Little or No Fines.

Weathered Rock - Weathered Quaternary Rock; Mainly Breccia.

Weathered Volcanic Rock - Weathered Tertiary Flows; Mainly Basalt and Andesite.

Volcanic Siltstone - Weathered Tertiary Flows.

Volcanic Breccia.

SYMBOL LEGEND

SPT 1.5" Sample

Bulk Sample

Groundwater Level (After Completion)

Groundwater Level (Before Completion)

LOG COLUMN DESCRIPTION

1. Elevation: Elevation (in feet) referenced to mean sea level (MSL).
2. Depth: Distance (in feet) below the collar of the borehole.
3. Recovery: Amount (in percent) of core recovered from the coring interval; calculated as length of core recovered divided by run length.
4. Soils and Lithology: A graphic log of material encountered using symbols to represent differing soil and rock types; symbols are explained above.
5. Description: Lithologic description in this order: rock type, color, texture, grain size, weathering, strength, and other features.
6. Drill Rate: Time marking start and finish of each run; drill rate (ft/min).
7. Drill Torque (lb/ft): Measure of total down pressure relative to material strength and drill bit refusal.
8. Compressive Strength: measured load at failure use pocket penetrometer.
9. Field Notes and Other Tests: Comments regarding drilling and sampling made by driller or logger.

GEOSERV

REPORT TITLE KRRP Roads, Bridges, and Culverts		
COUNTY Siskiyou		
PROJECT OR BRIDGE NAME Geotechnical Data Reports		
PREPARED BY JF, JS, KF	DATE 10/07/20	SHEET 1

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.89861862, -122.50257862	RC-CR-001

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-180+60	2157.76'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019		ND	NA	1.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt												AC Thickness: 0.083'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.			20									
	1		Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered to fresh; gravel to cobble subangular grains; weathering decreases with depth.			50: 0.42'			50/ 60/ 66	49.5	46,786				
	2														
2155															
	3														
	4														
	5														

	REPORT TITLE				HOLE ID
GEOSERV	Road Condition Survey, Copco Road				RC-CR-001
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 1

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.89997054, -122.50084894	RC-CR-002
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-190+58.0	2156.44'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				7.0'

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Remarks
	0	Asphalt		100					AC Thickness: 0.16'
		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.	8/4/4	50	8/ 8.4/ 10.9			2,464	
2155	1	SC: SANDY CLAY with GRAVEL (Fill-SC); firm to stiff; moist to dry; reddish brown to brown; low to medium plasticity; <1" subangular to angular GRAVEL; very fine to coarse SAND							
	2		3/6/6	50	12/ 9.5/ 16.5			5,491	
	3–4		4/5/4	75	9/ 7.2/ 12.5			4,804	
	5		3/4/6	50	10/ 9/ 12.6			5,100	
2150	6.5	Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.							
	7		50:0.08'	0	50/ 40/ 62.6	49.5	45,750		

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Column headings also printed: Material Graphics, Sample Location, Sample Number, Blows per foot, Drilling Method, Casing Depth.

GEOSERV

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-002
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 2

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.90057592, -122.49007457	RC-CR-003
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-220+57	2152.45'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				4.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Underdrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.124'
	1		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.			12/5/11		75	16/ 17.6/ 21.6			6,800			
2150	2		SC: SANDY CLAY with GRAVEL (Fill-SC); very stiff; moist; brown; low to medium plasticity; <1" subangular to angular GRAVEL; very fine to coarse SAND.			5/11/16		100	27/ 21.6/ 35.7			11,508			
	3–4		SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); very stiff; moist; reddish brown; low to medium plasticity; <1" subangular to angular GRAVEL; <2.5" subangular COBBLE; very fine to coarse SAND			8/10/15		70	25/ 20/ 2			10,897			
	5														

	REPORT TITLE				HOLE ID
GEOSERV	Road Condition Survey, Copco Road				RC-CR-003
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 3

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.90960758, -122.46340715	RC-CR-004
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-315+66	2207.32'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				0.8

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.125'
			Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered to fresh; gravel to cobble subangular grains; weathering decreases with depth.			20/ 50:0.33'		0	50	48.0	48,780				
	1														
	2														
2205															
	3														
	4														
	5														

	REPORT TITLE				HOLE ID
GEOSERV	Road Condition Survey, Copco Road				RC-CR-004
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 4

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)			HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.9227841, -122.44965139			RC-CR-005
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)			SURFACE ELEVATION
GeoServ, Inc.			STA-386+17			2221.34'
DRILLING METHOD			DRILLING RIG			BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver			6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE			HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer			
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				ND	NA	2.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1.60	Friction Angle	Calc. Bearing Capacity (psf)	Underdrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.208'
	1		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dense; reddish brown; dry; <3/4" diameter.			10/11/34		90	45/ 36/ 61.2	41.0	33,721				
2220	2		Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered to fresh; gravel to cobble subangular grains; weathering decreases with depth.			50: 0.46'		85	50/ 40/ 68	49.5	48,700				
	3														
	4														
	5														

GEOSERV	REPORT TITLE Road Condition Survey, Copco Road			HOLE ID RC-CR-005
	DIST.	COUNTY Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME KRRP			
	BRIDGE NUMBER NA	PREPARED BY KF	DATE 2/18/20	SHEET 5

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.93238193, -122.44157609	RC-CR-006

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-430+68	2195.36'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019		ND	NA	1.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Underdrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							
2195			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.												AC Thickness: 0.208'
			SC: SANDY CLAY with GRAVEL (Fill-SC); stiff; moist; brown; low to medium plasticity; <1" subangular to angular GRAVEL; very fine to coarse SAND.			18/27/39		50	[illegible]	[illegible]	[illegible]				
	1		Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.												
	2														
	3														
	4														
	5														

GEOSERV

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-006
DIST.	COUNTY Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME KRRP				
BRIDGE NUMBER NA	PREPARED BY KF		DATE 2/18/20	SHEET 6

LOGGED BY: JF & JS	BEGIN DATE: Nov 25th, 2019	COMPLETION DATE: Nov 25th, 2019	BOREHOLE LOCATION (Lat/Long or North/East and Datum): 41.94152973, -122.44469268	HOLE ID: **RC-CR-007**
DRILLING CONTRACTOR: GeoServ, Inc.			BOREHOLE LOCATION (Offset, Station, Line): STA-470+56	SURFACE ELEVATION: 2451.54'
DRILLING METHOD: Hollow Stem Auger			DRILLING RIG: Little Beaver	BOREHOLE DIAMETER: 6"
SAMPLER TYPE(S) AND SIZES (ID): CA Split Spoon 1.5"			SPT HAMMER TYPE: Safety Hammer	HAMMER EFFICIENCY, ERi:
BOREHOLE BACKFILL AND COMPLETION: Bentonite Chip, Nov 25th, 2019			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING: 0.8'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Underdrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.			38/50:0		100	[illegible]	[illegible]	[illegible]				AC Thickness: 0.124'
			Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered to fresh; gravel to cobble subangular grains; weathering decreases with depth.												
	1														
2450															
	2														
	3														
	4														
	5														

GEOSERV	REPORT TITLE: Road Condition Survey, Copco Road			HOLE ID: RC-CR-007
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP			
	BRIDGE NUMBER: NA	PREPARED BY: KF	DATE: 2/18/20	SHEET: 7

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.94752309, -122.43652720	RC-CR-008

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-507+44	2409.19'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION	**GROUNDWATER READINGS** — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	**TOTAL DEPTH OF BORING**
Bentonite Chip, Nov 25th, 2019		6.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.125'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.			13/8/5		100	13/ 10.4/ 17.7			[illegible]			
	1		SC: SANDY CLAY with GRAVEL (Fill-SC); firm to very stiff; moist to dry; reddish brown to brown; low to medium plasticity; <1" subangular to angular GRAVEL; very fine to coarse SAND												
	3					3/3/2		95	5/ 4/ 6.6			[illegible]			
2405	4					3/4/12		100	16/ 12.8/ 21.6			[illegible]			
	5		Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.			50:0.33'		75	50/ 40/ 68	49.5	45,700				
	6														
	7														
	8														
2400	9														
	10														

GEOSERV

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-008
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 8

LOGGED BY: JF & JS	BEGIN DATE: Nov 25th, 2019	COMPLETION DATE: Nov 25th, 2019	BOREHOLE LOCATION (Lat/Long or North/East and Datum): 41.95554070, -122.43910704	HOLE ID: RC-CR-009
DRILLING CONTRACTOR: GeoServ, Inc.			BOREHOLE LOCATION (Offset, Station, Line): STA-552+05	SURFACE ELEVATION: 2355.96'
DRILLING METHOD: Hollow Stem Auger			DRILLING RIG: Little Beaver	BOREHOLE DIAMETER: 6"
SAMPLER TYPE(S) AND SIZES (ID): CA Split Spoon 1.5"			SPT HAMMER TYPE: Safety Hammer	HAMMER EFFICIENCY, ERi:
BOREHOLE BACKFILL AND COMPLETION: Bentonite Chip, Nov 25th, 2019			GROUNDWATER READINGS: DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING: 2.6'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					85							AC Thickness: 0.229'
			SC: SANDY CLAY with GRAVEL and COBBLE (Fill-SC); hard to stiff; moist to dry; reddish brown; low to medium plasticity; <1" subangular to angular GRAVEL; <2.5" subangular COBBLE; very fine to coarse SAND			7/13/20		100	[illegible]/ 26.4/ 44.9			[illegible]			
2355	1														
	2					8/8/5		100	[illegible]			[illegible]			
	3														
	4														
	5														

GEOSERV	REPORT TITLE: Road Condition Survey, Copco Road			HOLE ID: RC-CR-009
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP			
	BRIDGE NUMBER: NA	PREPARED BY: KF	DATE: 2/18/20	SHEET: 9

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)		HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.96907399, -122.43490124		RC-CR-009A
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)		SURFACE ELEVATION
GeoServ, Inc.			STA-698+00		2352.0'
DRILLING METHOD			DRILLING RIG		BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver		6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE		HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer		
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS: DURING DRILLING ND	AFTER DRILLING (DATE) NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019					5.0'

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Remarks
	0	Asphalt		100					AC Thickness: 0.52'
		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.							
2350	1–2	SC: SANDY CLAY with GRAVEL (Fill-SC); firm to very stiff; moist; brown; low to medium plasticity; 1" subangular to angular GRAVEL; very fine to coarse SAND	12/10/7	100	[illegible]			[illegible]	
	3–4	SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); firm to very stiff; moist to dry; reddish brow; low to medium plasticity; <1" subangular to angular GRAVEL; <2.5" subangular COBBLE; very fine to coarse SAND	5/4/16	95	[illegible]			[illegible]	
			13/11/17	100	[illegible]	[illegible]	[illegible]		
2345	5	Weathered Volcanic Rx: NATIVE ROCK; medium dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.							

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-009A
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 10

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Nov 25th, 2019	Nov 25th, 2019	41.96769722, -122.42322706	RC-CR-010
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-739+58	2415.39'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				3.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.104'
2415			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dense; reddish brown; dry; <3/4" diameter.			12/11/8		80	[illegible]			[illegible]			
	1		SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); very stiff to hard; moist; brown; low to medium plasticity; <1" subangular to angular GRAVEL; <2" subangular COBBLE at 2.0' bgs; very fine to coarse SAND.												
	2					6/22/18		25	[illegible]			[illegible]			
	3														
	4														
	5														

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 4th, 2019	Dec 4th, 2019	41.97357373, -122.39873857	RC-CR-010A

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-831+92	2352.0'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019		ND	NA	5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.5'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); very dense; reddish brown; dry; <3/4" diameter.												
	1		SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); very stiff to firm; moist; dark brown; low to medium plasticity; <1" subangular to angular GRAVEL; few 2"-4" subangular COBBLE at 2.5' bgs; very fine to coarse SAND.			8/9/10		25	19/ 15.2/ 25.6			[illegible]			
2350	2					6/9/7		80	16/ 12.8/ 21.6			[illegible]			
	3		Weathered Volcanic Rx: NATIVE ROCK; medium dense; low to medium plasticity; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.												
	4					11/5/5		60	10/ 8/ 13.6			[illegible]			
	5														

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-010A
DIST.	**COUNTY**	**ROUTE**	**POSTMILE**	**EA**
	Siskiyou			
PROJECT OR BRIDGE NAME				
KRRP				
BRIDGE NUMBER	**PREPARED BY**		**DATE**	**SHEET**
NA	KF		2/18/20	12

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 4th, 2019	Dec 4th, 2019	41.96998006, -122.41964575	RC-CR-010B
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-753+85	2441.27'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019				6.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.104'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); medium dense; reddish brown; dry; <3/4" diameter.			20/13/5		80	[illegible]	[illegible]	[illegible]				
2440	1		SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); firm; moist; brown; low to medium plasticity; <1" subangular to angular GRAVEL; <4" subangular to angular COBBLE at 2.0' bgs; very fine to coarse SAND.												
	2					5/3/4		50	[illegible]			[illegible]			
	3					4/3/3		60	[illegible]			[illegible]			
	5					2/1/2		50	[illegible]			[illegible]			
	6														
2435	7														
	8														
	9														
	10														

GEOSERV

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-010B
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 13

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 4th, 2019	Dec 4th, 2019	41.96990519, -122.39491649	RC-CR-011

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-861+30	2353.69'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Dec 4th, 2019		ND	NA	6.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.083'
	0–1		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); medium dense; reddish brown; dry; <3/4" diameter.			18/12/12		80	[illegible]	[illegible]	[illegible]				
	1–5		SC: SANDY CLAY with GRAVEL (Fill-SC); stiff to very stiff; moist to dry; reddish brown to dark brown; low to high plasticity; <1/2" subangular to angular GRAVEL; very fine to coarse SAND			13/5/5		80	[illegible]			[illegible]			
2350						4/8/14		80	[illegible]			11,717			
	5–6		Weathered Volcanic Rx: NATIVE ROCK; very dense; weathering to clay; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.			7/9/11		80	[illegible]			[illegible]			
2345	6–10														

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-011
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 14

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 3rd, 2019	Dec 3rd, 2019	41.97099703, -122.37556893	RC-CR-011A
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-918+36	2389.63'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Dec 3rd, 2019				2.3'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Underdrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.166'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dense; reddish brown; dry; <3/4" diameter.												
	1		Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered to fresh; gravel to cobble subangular grains; weathering decreases with depth.			14/16/19		80	35/ 39/ 47.6	56	15,667				
	2					24/ 50:0.33'		80	50/ 40/ 65	[illegible]	[illegible]				
	3														
	4														
2385															
	5														

	REPORT TITLE				HOLE ID
GEOSERV	Road Condition Survey, Copco Road				RC-CR-011A
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 15

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 3rd, 2019	Dec 3rd, 2019	41.97727691, -122.36430688	RC-CR-012

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-960+49	2395.86'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Dec 3rd, 2019		ND	NA	3.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Asphalt					100							AC Thickness: 0.166'
			GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); medium dense; reddish brown; dry; <3/4" diameter.												
2395	1		SC: SANDY CLAY with GRAVEL (Fill-SC); loose to moderately stiff; moist; reddish brown to dark brown; low to high plasticity; <1/2" subangular to angular GRAVEL; very fine to coarse SAND			17/15/13		80	28/ 23.4/ 56.1	53	10,284				
	2		Weathered Volcanic Rx: NATIVE ROCK; dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.			7/11/26		80	37/ 29.6/ 55.5	36	10,181				
	3														
	4														
	5														

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-012
DIST.	**COUNTY**	**ROUTE**	**POSTMILE**	**EA**
	Siskiyou			
PROJECT OR BRIDGE NAME				
KRRP				
BRIDGE NUMBER	**PREPARED BY**		**DATE**	**SHEET**
NA	KF		2/18/20	16

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 3rd, 2019	Dec 3rd, 2019	41.98730504, -122.35332753	RC-CR-013

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	STA-1019+33	2663.56'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
CA Split Spoon 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, Nov 25th, 2019		ND	NA	3.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dense; reddish brown; dry; <3/4" diameter.			17/17/21		80	[illegible]	[illegible]	[illegible]				
			SC: SANDY CLAY with GRAVEL and COBBLES (Fill-SC); hard; moist to dry; reddish brown to dark brown; low to medium plasticity; <1" subangular to angular GRAVEL; <4" subangular COBBLE; very fine to coarse SAND.												
	1														
						20/18/33		80	[illegible]			[illegible]			
	2														
	3														
2660															
	4														
	5														

GEOSERV	REPORT TITLE				HOLE ID
	Road Condition Survey, Copco Road				RC-CR-013
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: KRRP				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 17

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Dec 3rd, 2019	Dec 3rd, 2019	41.97727691, -122.36430688	RC-CR-014
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			STA-1059+30	2801.27'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Little Beaver	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
CA Split Spoon 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): NA	TOTAL DEPTH OF BORING
Bentonite Chip, Dec 3rd, 2019				7.8'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1.60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); medium dense; reddish brown; dry; <3/4" diameter.												
			SC: SANDY CLAY with GRAVEL (Fill-SC); very stiff to hard; moist; reddish brown to dark brown; low to high plasticity; <1/2" subangular to angular GRAVEL; very fine to coarse SAND			9/6/10		90	[illegible]	[illegible]	[illegible]				
2800	1														
	2					7/9/9		100	[illegible]			[illegible]			
	3					9/11/13		100	[illegible]			[illegible]			
	4														
	5					8/4/7		100	[illegible]			[illegible]			
2795	6														
	7					10/13/19		90	[illegible]			[illegible]			
			Weathered Volcanic Rx: NATIVE ROCK; very dense; cohesionless; highly weathered; gravel to cobble subangular grains; weathering decreases with depth.			50:0.33'		80	[illegible]	50	[illegible]				
	8														
	9														
	10														

GEOSERV

REPORT TITLE				HOLE ID
Road Condition Survey, Copco Road				RC-CR-014
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: KRRP				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 2/18/20	SHEET: 18

Photographs 1. Road Core RC-CR-001 (**STA 180+60.0**) SPT sample taken from 0-1.5' bgs.

Photograph 2. Asphalt core sample at Road Core RC-CR-002 (**STA 236+20.0**).

Photograph 3/4/5. SPT samples taken at 0-1.5’ bgs -**Left**, 1.5-3’ bgs -**Middle**, & 3-4.5’ bgs -**Right** (CR-RC-002).

Photograph 6 & 7. SPT sample taken at 4.5-6.0’ bgs-**Left**, & 6.5-6.584’ bgs-**Right** (CR-RC-002).

Photograph 7. Asphalt core sample at Road Core RC-CR-003 (**STA 220+57.0**).

Photographs 8/9/10. SPT samples taken at 0-1.5’ bgs-**Left**, 1.5’-3.0’ bgs-**Middle**, & 3.0-4.5’ bgs-**Right** (CR-RC-003).

Photograph 11. Looking at asphalt coring at Road Core RC-CR-004 (**STA 315+66.0**).

Photograph 12. Asphalt core sample at Road Core RC-CR-004.

Photograph 13. SPT sample taken at 0-0.8’ bgs (CR-RC-004).

Photograph 14. Asphalt core sample at Road Core RC-CR-005 (**STA 386+17.0**).

Photograph 15 & 16. SPT samples taken at 0-1.5’ bgs -**Left**, & 1.5-1.958’ bgs-**Right** (CR-RC-005)

Photograph 17. Looking at asphalt coring at Road Core RC-CR-006 (**STA 430+68.0**).

Photograph 18. SPT sample taken at 0-1.5’ bgs (CR-RC-006)

Photograph 19. Asphalt core sample at Road Core RC-CR-007 (**STA 470+56.0**)

Photograph 20. SPT sample taken at 0-0.8’ bgs (CR-RC-007)

Photograph 21. Asphalt core sample at Road Core RC-CR-008. (**507+44.0**)

Photographs 22 & 23. SPT samples taken at 0-1.5’ bgs-**Left**, & 2.5-4.0’ bgs-**Right** (CR-RC-008)

Photograph 24. SPT sample taken at 4.0-5.5’ bgs (CR-RC-008)

Photograph 25. Asphalt core sample at Road Core RC-CR-009 (**STA 552+05**).

Photograph 26 & 27. SPT samples taken at 0-1.5’ bgs **-Left**, & 1.5-3.0’ bgs **-Right** (CR-RC-009)

Photograph 28. Asphalt core sample at Road Core RC-CR-09A, (**STA-739+58.0**).

Photograph 29. Looking at SPT sample taken from 0.5-2.5’ bgs (RC-CR-09A).

Photograph 30. Looking at Road Core location RC-CR-010 (**STA 739+58.0**).

Photograph 31. Looking at Road Core location RC-CR-010A (**STA 831+92.0**).

Photograph 32. Looking at Road Core location RC-CR-010B (**STA 753+85**).

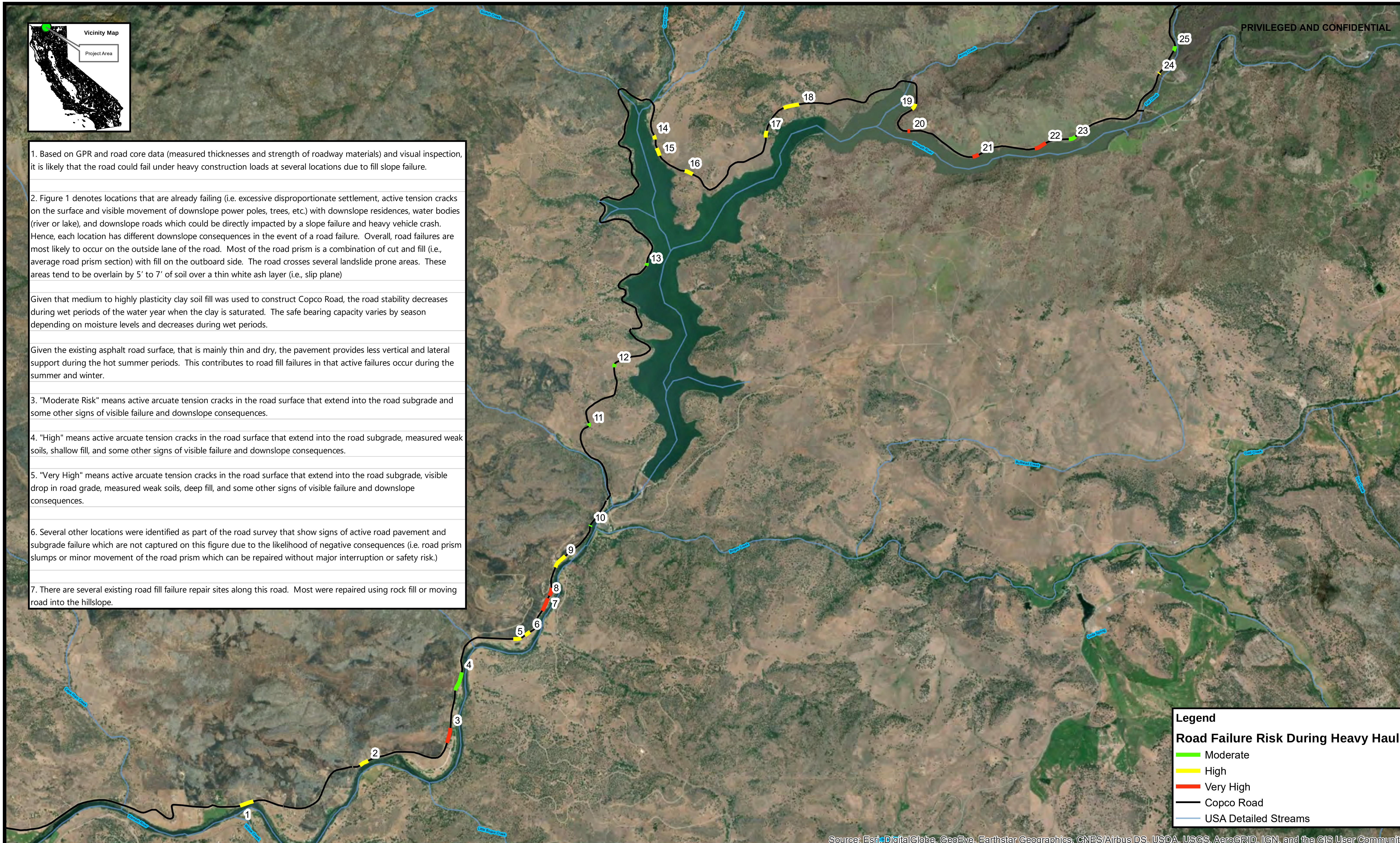
Photograph 33. Looking at Road Core location RC-CR-011 (**STA 861+30.0**).

Photograph 34. Looking at Road Core location RC-CR-011A (**STA 918+36.0**).

Photograph 35. Looking at Road Core location RC-CR-012 (**STA 960+49.0**).

Photograph 36. Looking at Road Core location RC-CR-013 (**STA 1019+33**).

Photograph 37. Looking at Road Core location RC-CR-013 (**STA 1059+30**).



1. Based on GPR and road core data (measured thicknesses and strength of roadway materials) and visual inspection, it is likely that the road could fail under heavy construction loads at several locations due to fill slope failure.

2. Figure 1 denotes locations that are already failing (i.e. excessive disproportionate settlement, active tension cracks on the surface and visible movement of downslope power poles, trees, etc.) with downslope residences, water bodies (river or lake), and downslope roads which could be directly impacted by a slope failure and heavy vehicle crash. Hence, each location has different downslope consequences in the event of a road failure. Overall, road failures are most likely to occur on the outside lane of the road. Most of the road prism is a combination of cut and fill (i.e., average road prism section) with fill on the outboard side. The road crosses several landslide prone areas. These areas tend to be overlain by 5' to 7' of soil over a thin white ash layer (i.e., slip plane)

Given that medium to highly plasticity clay soil fill was used to construct Copco Road, the road stability decreases during wet periods of the water year when the clay is saturated. The safe bearing capacity varies by season depending on moisture levels and decreases during wet periods.

Given the existing asphalt road surface, that is mainly thin and dry, the pavement provides less vertical and lateral support during the hot summer periods. This contributes to road fill failures in that active failures occur during the summer and winter.

3. "Moderate Risk" means active arcuate tension cracks in the road surface that extend into the road subgrade and some other signs of visible failure and downslope consequences.

4. "High" means active arcuate tension cracks in the road surface that extend into the road subgrade, measured weak soils, shallow fill, and some other signs of visible failure and downslope consequences.

5. "Very High" means active arcuate tension cracks in the road surface that extend into the road subgrade, visible drop in road grade, measured weak soils, deep fill, and some other signs of visible failure and downslope consequences.

6. Several other locations were identified as part of the road survey that show signs of active road pavement and subgrade failure which are not captured on this figure due to the likelihood of negative consequences (i.e. road prism slumps or minor movement of the road prism which can be repaired without major interruption or safety risk.)

7. There are several existing road fill failure repair sites along this road. Most were repaired using rock fill or moving road into the hillslope.

Prepared By:

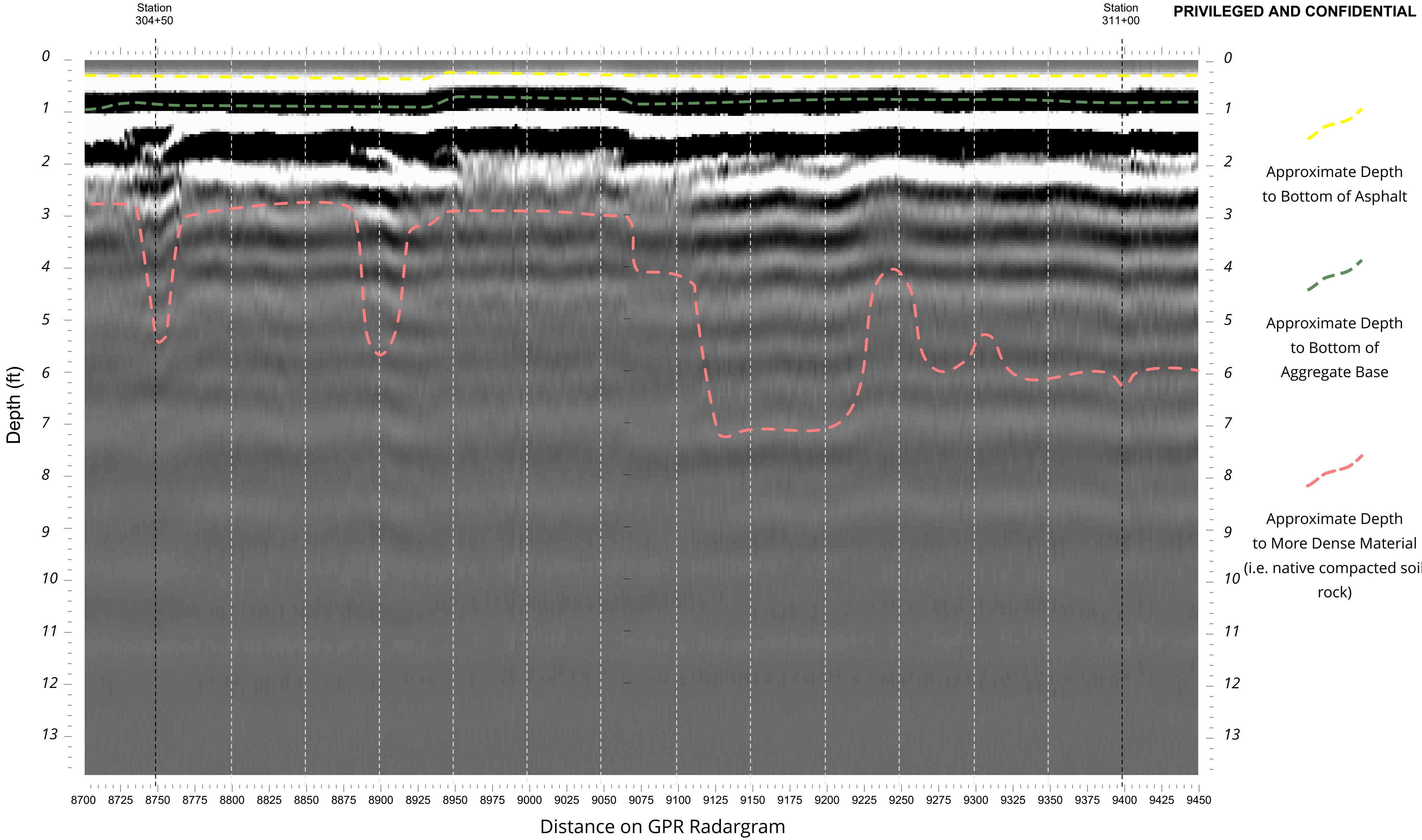
GeoServ Inc Engineering Geologists Rock - Soil - Water

Figure 1 Copco Road Fill Failure Risk Map

0 0.5 1 Miles

Coordinate System: NAD 1983 StatePlane California I FIPS 0401 Feet
Projection: Lambert Conformal Conic
Datum: North American 1983

Date: 10/5/2020



Source: GPR Radargram file 0.0044

GeoServ Geoscience Services

GPR Radargram High Risk Area 3

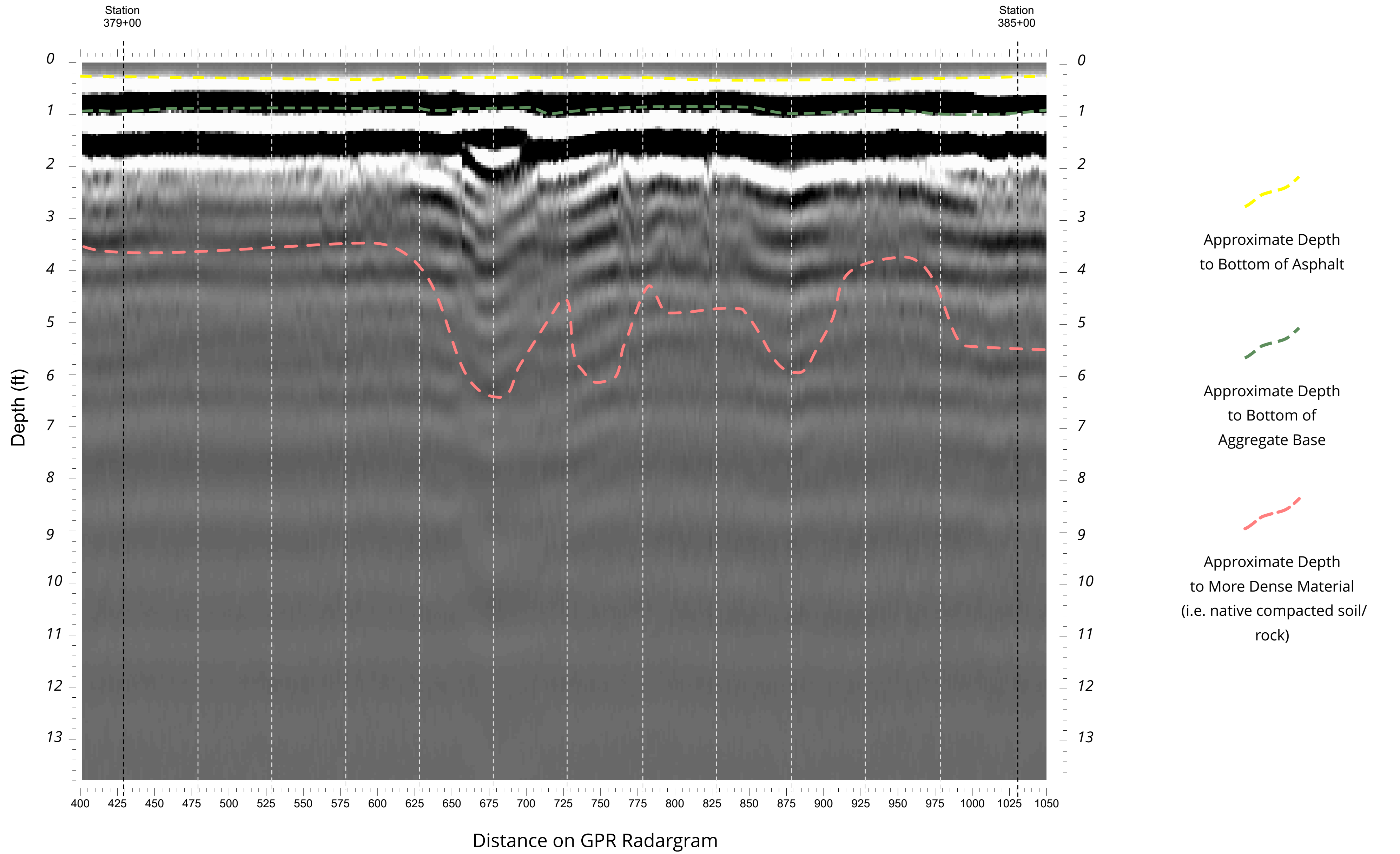
Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0046

GEOSERV Geoscience Services

GPR Radargram High Risk Area 7

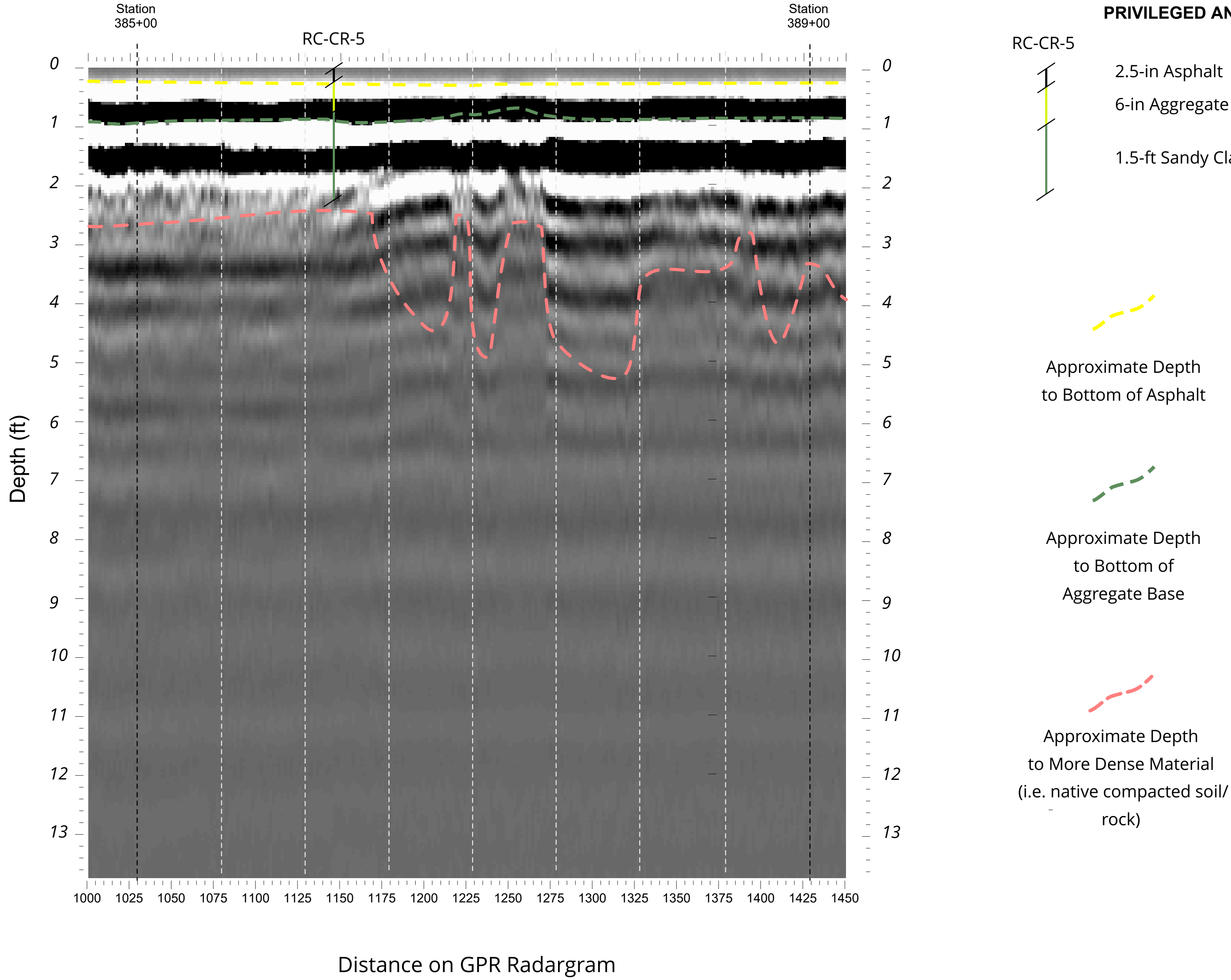
Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0046

GeoServ
Geoscience Services

GPR Radargram High Risk Area 8

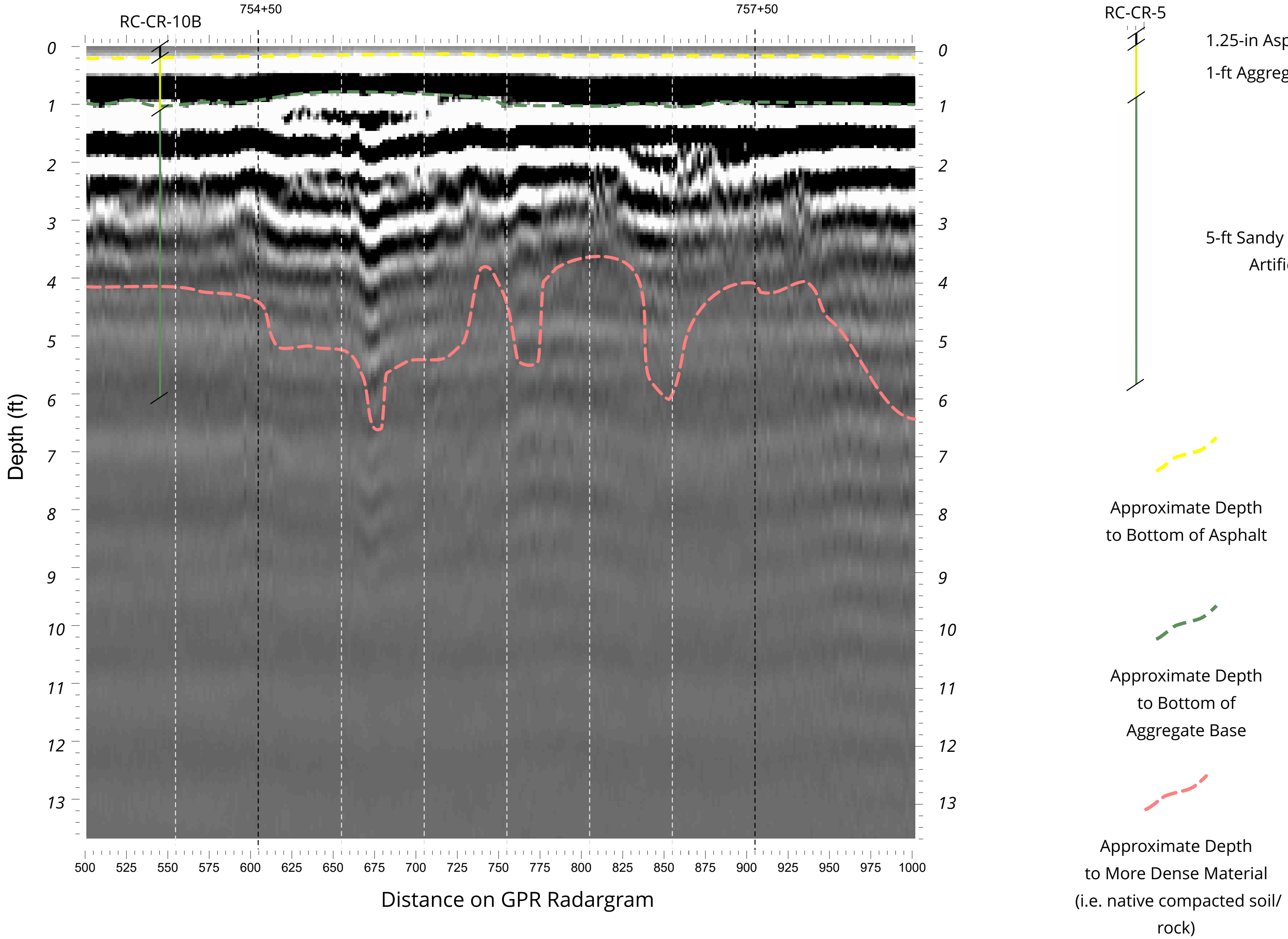
Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0018

GeoServ Geoscience Services

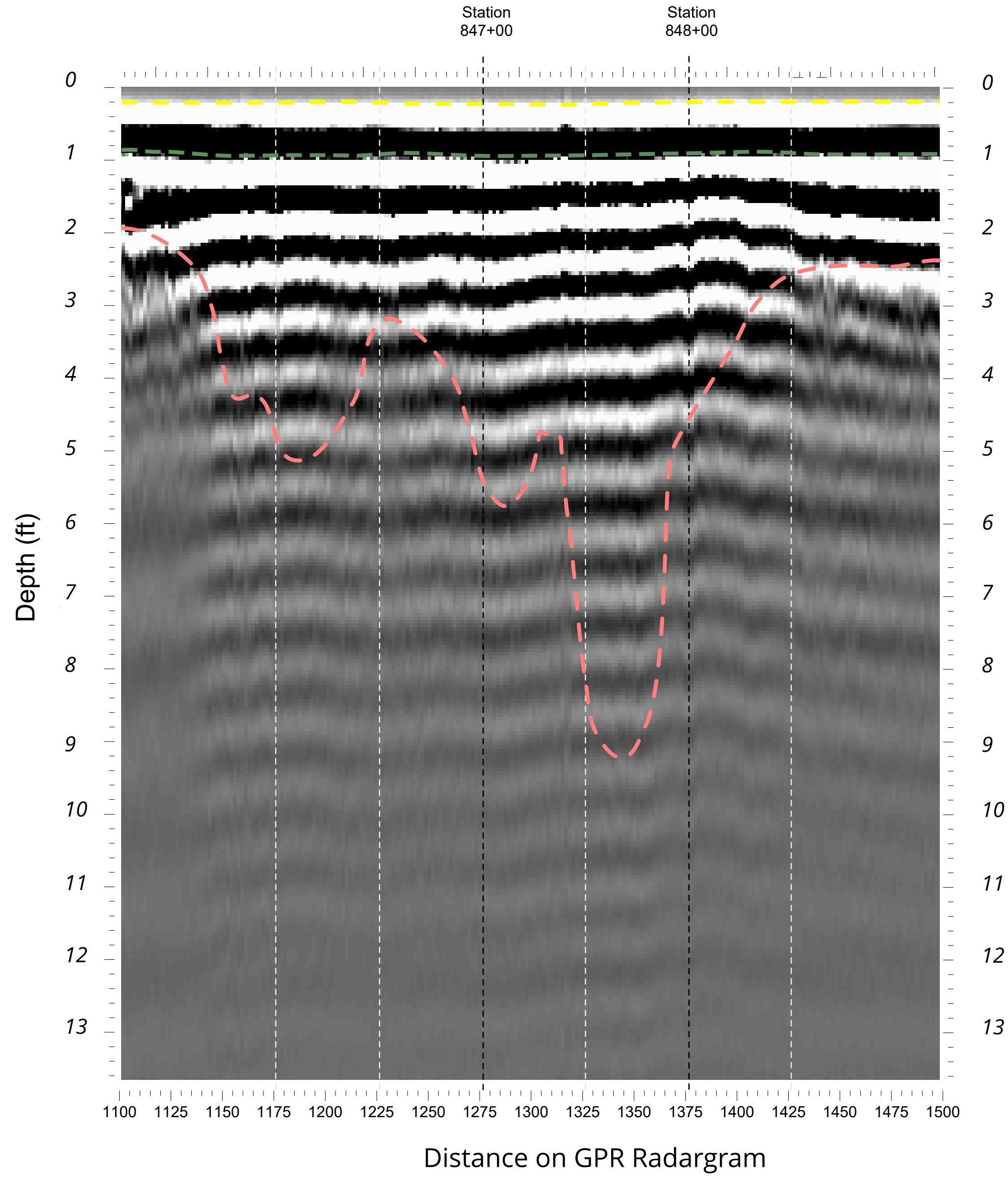
GPR Radargram High Risk Area 17

Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:



Approximate Depth to Bottom of Asphalt

Approximate Depth to Bottom of Aggregate Base

Approximate Depth to More Dense Material (i.e. native compacted soil/ rock)

Source: GPR Radargram file 0.0020

GeoServ Geoscience Services

GPR Radargram High Risk Area 20

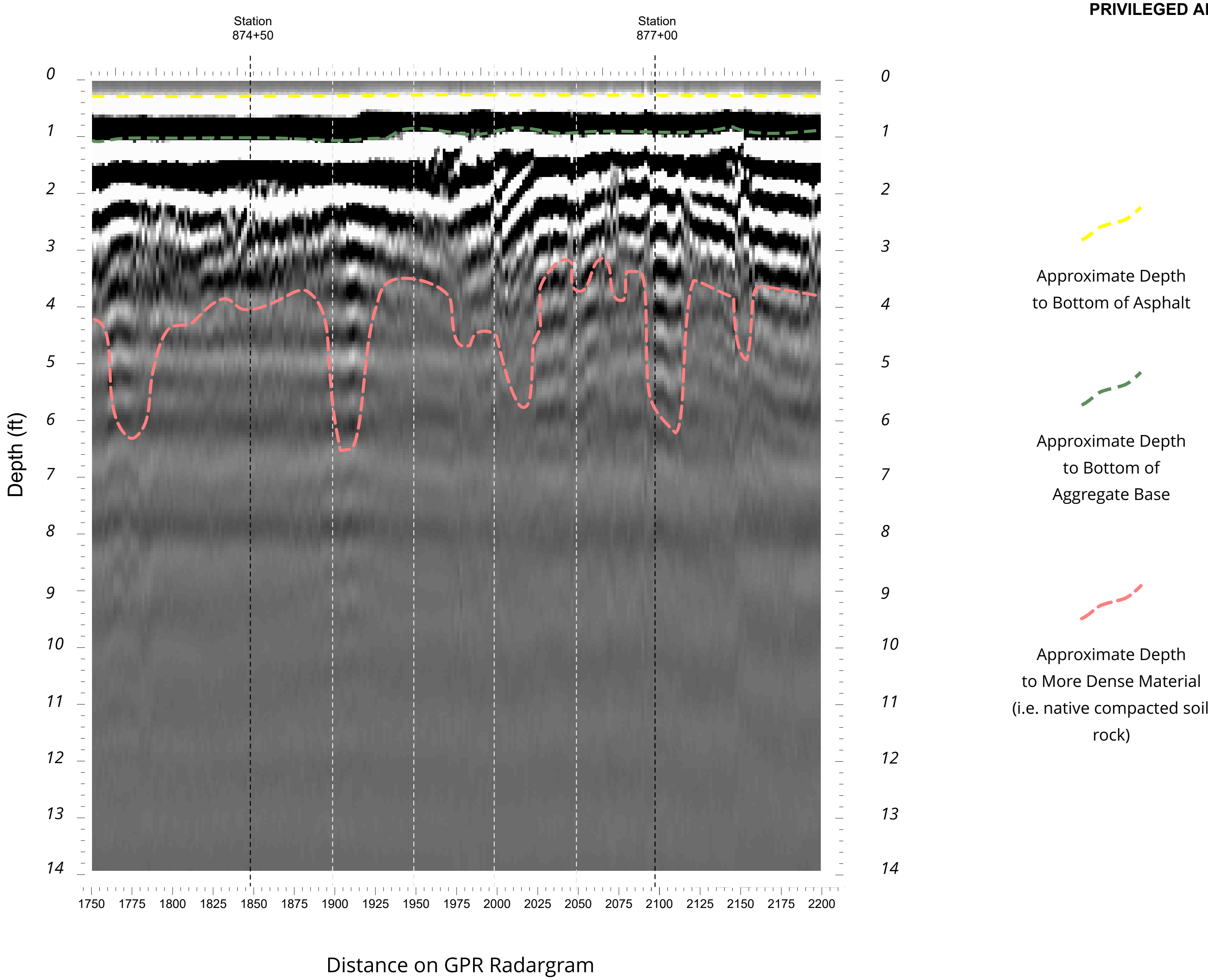
Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0022

GeoServ Geoscience Services

GPR Radargram High Risk Area 21

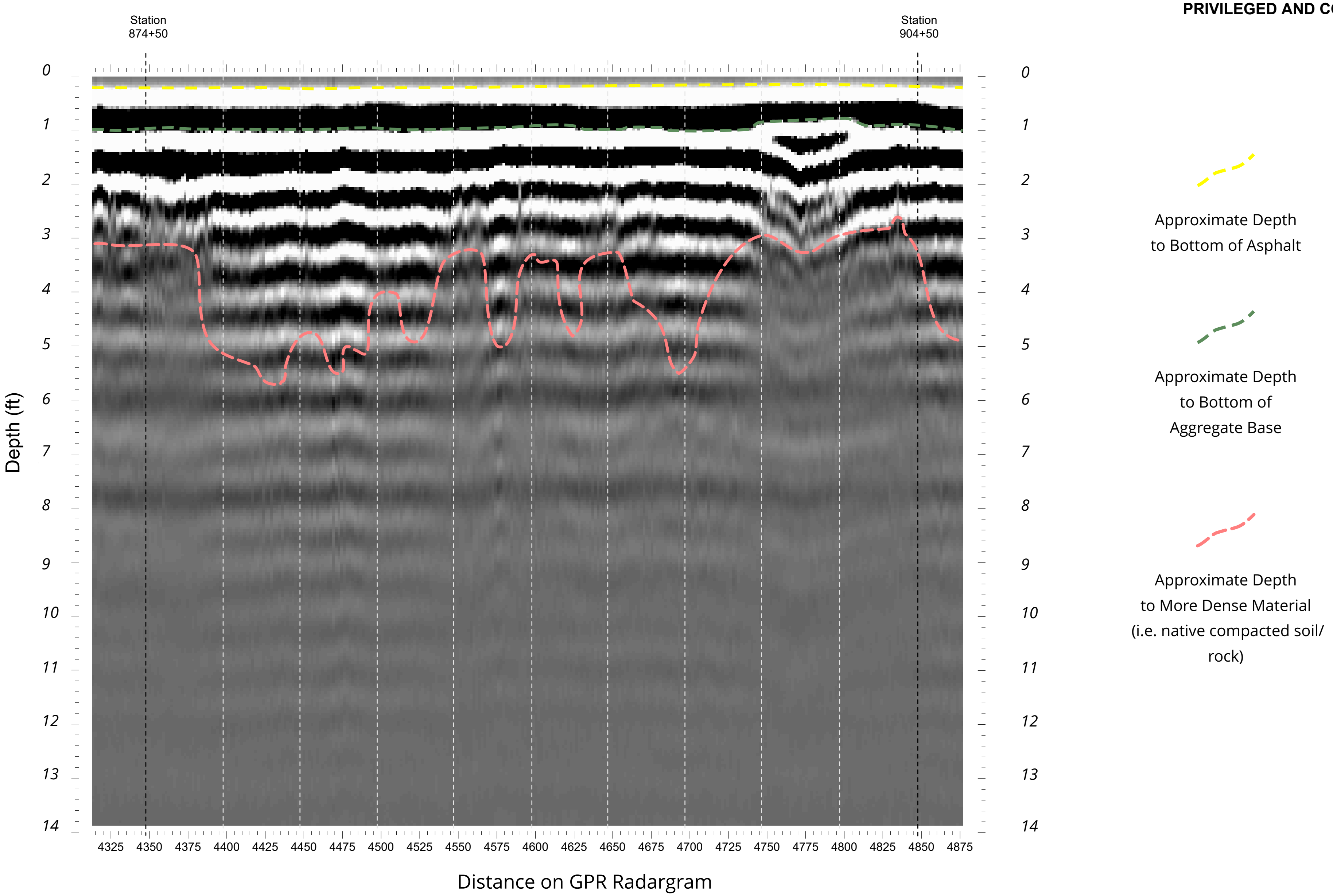
Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

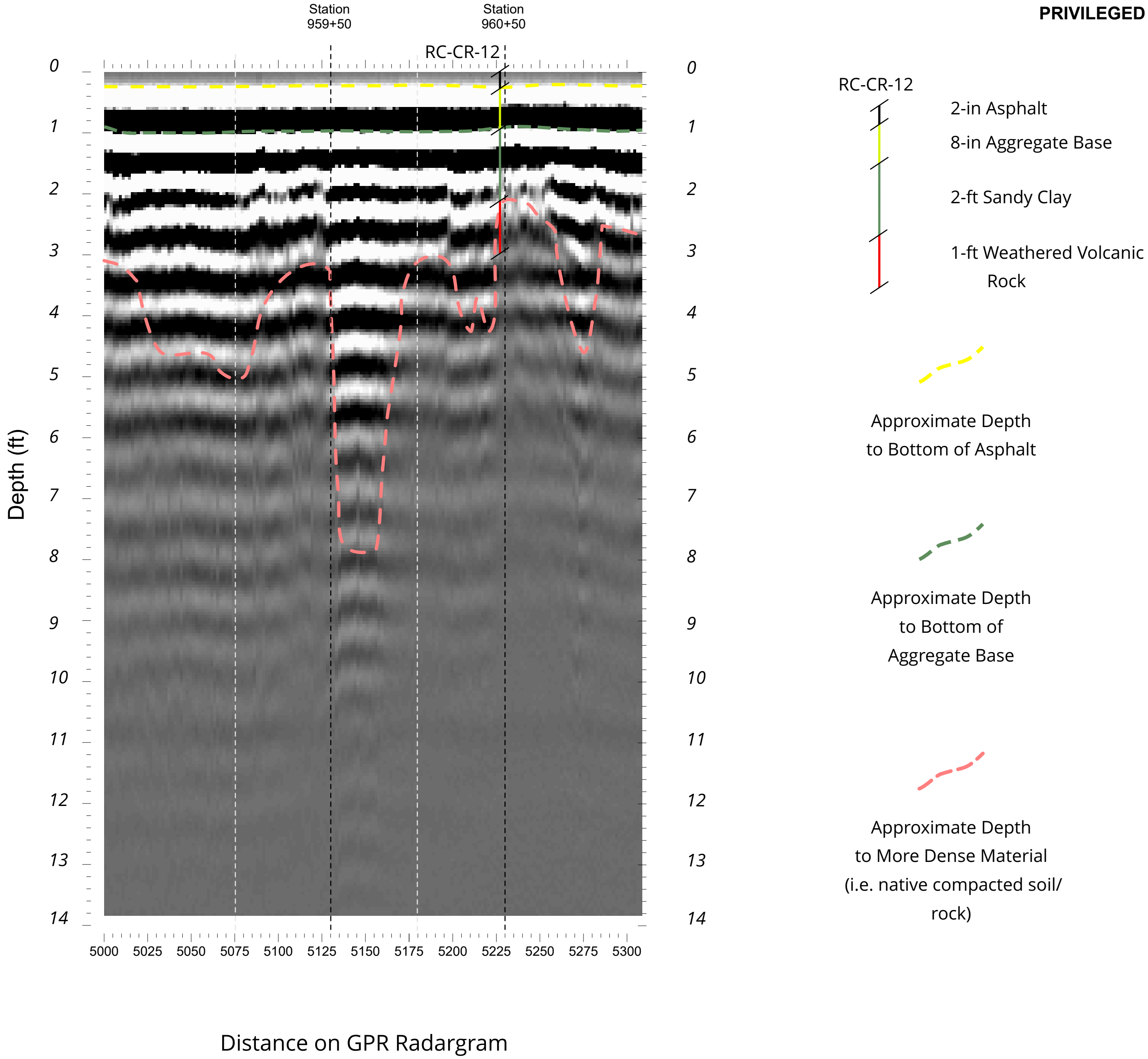
PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0022

GeoServ Geoscience Services	GPR Radargram High Risk Area 22	
	Project: KRRP Road Condition Survey	
	Client: KRRP	Scale: As noted
		Plate #:

PRIVILEGED AND CONFIDENTIAL



Source: GPR Radargram file 0.0023

GeoServ Geoscience Services

GPR Radargram High Risk Area 23

Project: KRRP Road Condition Survey

Client: KRRP

Scale: As noted

Plate #:

October 7, 2020 GSI Project #: 190725

Knight Piésold Ltd. (KP)

Subject: KRRP Transportation Geotechnical Data Report

Dear Knight Piésold:

In accordance with your request and authorization of GeoServ, Inc. (GSI) has prepared the enclosed Geotechnical Data Report based on the requirements and proposed project specifics identified during our review. Specifically, this report provides a summary of the methods used to collect geotechnical data and the data results for the following sites:

Figure 1 - Copco Road at Dry Creek Bridge
Figure 2 - Lakeview Road Bridge
Figure 3 - Scotch Creek Culvert
Figure 4 - Camp Creek Culvert
Figure 5 - Fall Creek at Daggett Road
Figure 6 - Fall Creek at Substation Road Bridge
Figure 7 - Fall Creek at Copco Road Bridge
Figure 8 - Daggett Road Temporary Construction Access Bridge

The memo includes Appendix A that shows and lists relevant data and diagrams to include:

- Borehole Locations and Logs
- Borehole Data
- Site Summary Photographs
- Available Laboratory Data

Data and results presented in this report are preliminary and subject to change. Additional analyses and interpretations need to be made from the data at the 100% design phase. Data analysis, interpretation, and design recommendations are not included at this time pending input from KP. If you have any questions regarding the data and results, please do not hesitate to contact this office. The opportunity to be of service is appreciated.

Respectfully submitted,

James Fitzgerald, Senior Geologist
GeoServ, Inc.
624 South Mount Shasta Blvd.
Mount Shasta, CA 96067
(530) 227-8963
jf@geoscienceserv.com

KRRP Transportation Geotechnical Data Report

Prepared for: Knight Piésold Ltd. (KP)

Prepared by: GeoServ, Inc. (GSI)

Initial Draft Report Date: June 24, 2020

Second Draft Report Date: July 14, 2020

Third Draft Report Date: October 7, 2020

Summary

GSI completed a subsurface geotechnical investigation at seven sites associated with the transportation system needed for KRRP construction access and post dam drawdown road improvements. The investigation included compiling existing data and information and drilling geotechnical borings. These data were used to characterize and measure subsurface conditions. This report includes a summary of the methods used for data collection, presents the geotechnical data, and lists the data limitations.

Field investigation of the transportation sites was accomplished through advancement of 18 geotechnical borings at the following sites:

Site	Borehole ID
Copco Road at Dry Creek Bridge	BH-DR01
	BH-DR02
	BH-DR03
	BH-DR04
Lakeview Road Bridge	BH-A01
	BH-A02
Scotch Creek Culvert	BH-SC01
	BH-SC02
Camp Creek Culvert	BH-CC01
	BH-CC02
Fall Creek Culvert at Daggett Road	BH-DG03
	BH-DG04
Fall Creek Culvert at Substation	BH-DG01
	BH-DG02
Copco Road at Fall Creek Bridge	BH-FL01
	BH-FL02
	BH-FL03
	BH-FL04

This data reports incorporates soil bore data collected by a previous investigation (AECOM 2018) at sites included in the KRRP Road, Bridge, Culvert site investigation to include:

1. Soil bores at Scotch Creek
2. Soil bores at Camp Creek
3. Soil bores at Daggett Road Bridge

The borehole locations are shown on the following figures:
Figure 1 - Copco Road at Dry Creek Bridge
Figure 2 - Lakeview Bridge
Figure 3 - Scotch Creek Culvert
Figure 4 - Camp Creek Culvert
Figure 5 - Fall Creek at Daggett Road
Figure 6 - Fall Creek at Substation Road Bridge
Figure 7 - Fall Creek at Copco Road Bridge
Figure 8 - Daggett Road Bridge

Methods

This investigation was completed to obtain information on the engineering properties of site fill, soil, rock, and groundwater at sites associated with the project road, bridge, and culvert improvements sites. The engineering properties of the site rocks and soils were assessed using industry standard methods (BOR 2001 and Williamson 1984). The rocks and soils were classified and assessed following the most recent ASTM methods.

Eighteen (18) boreholes were advanced at 7 project sites using either a Lonestar Auger Drill, Deere 35G Limited Access Drill, or a T1 Air Hammer Drill. The drilling tools included a 6" hollow stem auger and a 10" tri-cone bit. Standard Penetration Test (SPT) and bulk samples were taken in each borehole. Relatively undisturbed samples were taken with a 1.5" inner diameter SPT sampler at 2.5' to 5' intervals or at changes in soil/rock type. At Lakeview Bridge, once the rock layer was reached, the holes were advanced with the T1 Air Hammer Drill with a 10" Tri-Cone bit.

Borehole logs and summary figures were drafted following CalTrans standards. For each borehole, the rock/soil depth, color, particle size and volume, relative density/consistency, particle angularity and shape, moisture content, strength, cohesion, plasticity, and compaction were visually noted and field classified. SPT tests were completed following ASTM 1586. Split spoon core samples were collected, photographed, and field classified. The recovery of un-disturbed samples was limited given the material characteristics. The borehole logs are shown in Appendix A. A subset of the samples were sent to a soil laboratory and tested for gradation, plasticity, and strength (Appendix A). Field and laboratory measured soil and rock properties are summarized in Appendix A – Table 1. Summary photos of each site are included in Appendix A.

This report includes the data for each site and does not provide data analysis, interpretation, or design recommendations. At Scotch and Camp Creeks, that have had existing geotechnical data, their historic data was combined with the GSI data to help characterize the horizontal and vertical extent of subsurface conditions (Figure 3 and Figure 4).

Results

Copco Road at Dry Creek Bridge

Drilling at this site was accomplished with moderate to high drilling effort. The boreholes were located as close to the existing bridge abutments as possible (Figure 1). For all four boreholes, there is a layer of rock rubble and native fill at the surface. That fill likely extends down to the base of the abutments. The total depth drilled to auger refusal ranged from 5.5' to 11.5' below ground surface (bgs) (Appendix A – Table 1 and Borehole Logs). The measured fill thickness ranges from 5' to 7.5' bgs. The material consists of cohesive sandy gravel/cobble clay with soft to very stiff consistency (Appendix A – Table 1 and Borehole Logs). For the directly observed fill bulk samples, the sand is very fine to coarse, the clay has medium to high plasticity, gravels are less than 1" in diameter, and cobbles are about 2.5" in diameter. Below the fill layer, there is in-place native rock. Most of the in-place material is hard volcanic rock varying from fresh to very weathered into clay with gravel and cobbles. No groundwater was observed within the boreholes.

Lakeview Bridge
Drilling at this site was accomplished with moderate to high drilling effort. Boreholes BH-AB01 and BH-AB02 were located on the right bank of the river on the shoulder of Copco Road and the boat ramp (Figure 2). At BH-AB01 and BH-AB02 depth to refusal ranged from 35' and 30' bgs, respectively (Appendix A – Table 1 and Borehole Logs). The right river bank has three prominent layers of material, an upper artificial fill (containing: gravels, cobbles, and boulders), a clay rich material, and a volcanic bedrock material to at least 35' bgs. The artificial fill layer was encountered to a depth of about 5' bgs. The upper layer was rock rubble likely placed as part of road construction. The fill was generally loose near the surface and dense before the clay soil was encountered. The clay soil is stiff and moist from ~5' to 18' bgs. At 18' bgs, the stiff clay soil transitioned to a soft organic sandy clay in BH-AB01 and a loose gravelly clay in BH-AB02. The thickness of these soft and loose layers ranges from 2.5' to 5.0'. Below the weaker layer of gravelly clay and sandy clay is a very dense weathered volcanic rock. The USGS mapped the dominant geological unit in the area as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. The degree of weathering decreased with depth at 35.0' bgs in BH-AB-01 and 30.0' bgs in BH-AB02. The depth to bedrock in BH-AB01 and BH-AB02 correlated well. Groundwater was encountered in BH-AB01 at 13.0' bgs and in BH-AB02 at 10.0' bgs. The observed groundwater depths were well above the river water level. It appears that there is perched shallow groundwater flowing along the soil-rock contact.

Scotch Creek Culvert
Drilling at this site was accomplished with moderate to high drilling effort. Boreholes BH-SC01 and BH-SC02 were located on the right and left banks, respectively, of Scotch Creek just downstream of Copco Road (Figure 3). At BHSC-01 and BH-SC02 depth to refusal ranged from 7.5' and 7' bgs, respectively (Appendix A – Table 1 and Borehole Logs). The right and left streambanks have two prominent layers of material, alluvial sandy to clayey gravel and weathered volcanic rock (at a relatively shallow depth). The upper layer of clay, sand, and gravel is stiff/dense and moist from 0' to 7' bgs. At about 7' bgs, the alluvium transitioned to a very dense weathered volcanic rock. The USGS mapped the dominant geological unit in the area as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. The degree of weathering decreased with depth at 7.5' bgs at BH-SC01. No groundwater was not encountered within the boreholes.

Camp Creek Culvert
Drilling at this site was accomplished with low to moderate drilling effort. Boreholes BH-CC01 and BH-CC02 were located on the left and right banks, respectively, of Camp Creek just downstream of Copco Road (Figure 4). At BH-CC01 and BH-CC02 depth to refusal ranged from 20' and 22' bgs, respectively (Appendix A – Table 1 and Borehole Logs). The right and left streambanks have two prominent layers of material, loose alluvial sandy clay to clayey sand and medium dense well graded sand. No bedrock was encountered in either borehole. From 0' to 18' bgs, the alluvium is likely

sediment deposited in Camp Creek delta on top of the original stream channel (Figure 4). The upper layer of alluvial material is loose and liquefiable given that during drilling sand flowed up into the auger. Groundwater was encountered in both boreholes between 3' and 4' bgs. The groundwater was perched above the stream with the surface water in the stream 2' to 3' lower than the water level measured in the boreholes.

Fall Creek at Daggett Road

Drilling at this site was accomplished with low to high drilling effort. The boreholes were located as close to the existing culvert as possible (Figure 5); however, given the road width, underground utilities, and the need to keep the road open during drilling, the holes had to be located at a less than ideal proximity to the culvert (Figure 5). For BH-DG03, the top of the borehole was located adjacent to the road at the toe of the road fillslope. The fill consists of medium dense clayey sand and gravel and extends to about 10.5' bgs (Appendix A – Table 1 and Borehole Logs). Below the fill is a 2.5' thick layer of loose to stiff sandy clay. Below the clay is a very dense weathered volcanic rock. The USGS mapped the dominant geological unit in the area as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. For BH-DG04, the borehole was located in the road shoulder about 40' west of the existing culvert. The top 3' is fill consisting of loose to medium dense clayey sand and gravel (Appendix A – Table 1 and Borehole Logs). Below the fill there is a stiff sandy silty clay with gravel to 6.5' bgs. Below the clay a very dense weathered volcanic rock similar to the rock encountered in BH-DG03. No groundwater was observed within the boreholes.

Fall Creek at Substation Road Bridge

Drilling at this site was accomplished with medium to high drilling effort. The boreholes were located as close to the existing bridge as possible (Figure 6); however, given the road width and the need to keep the road open during drilling, the holes had to be located at a less than ideal distance from the bridge (Figure 6). For BH-DG02, there is fill that consists of medium dense sandy gravel to about 1.5' bgs (Appendix A – Table 1 and Borehole Logs). Below the fill is stiff sandy clay with gravel to 9.5' bgs. Below the clay is a very dense weathered volcanic rock was encountered to at least about 11' bgs. The USGS mapped the dominant geological unit in the area as Tertiary volcanic rock; minor pyroclastic deposits that correlates to the observed rock. For BH-DG01, there is fill that consists of very stiff gravelly clay to 7' bgs (Appendix A – Table 1 and Borehole Logs). Below the fill is a stiff to very stiff gravely clay with sand to 9.0' bgs. Auger refusal was met in this hole before hitting rock. No groundwater was observed within the boreholes.

Fall Creek at Copco Road Bridge

Drilling at this site was accomplished with high drilling effort. The boreholes were located as close to the existing bridge abutments as possible (Figure 7). At the surface there was a layer of rock rubble that extends to the base of the abutments in most locations. Only one borehole could be advanced through the rock rubble layer (i.e., BH-FC1). The total depth drilled to auger refusal ranged from 2' to 6.1' bgs (Appendix A – Table 1 and Borehole Logs). The fill consists loose to medium dense clayey sand and gravel. No groundwater was observed within the boreholes.

Daggett Road Bridge

Drilling at this site was completed by AECOM (2018), and based on their borehole logs, drilling was accomplished with low to high drilling effort. The boreholes were located on the north and south sides of the existing bridge and within the Klamath River just downstream of the bridge (Figure 8).

The observed subsurface material at this site consists of fill made up of rock rubble and sandy clay. Below the fill layer, there is in-place native rock. Most of the in-place material is weathered volcanic rock. Groundwater was observed within the borehole on the north side of the bridge at about 17' bgs and within the borehole located in the river bed (i.e., river water was 1' deep at the time of drilling). The streambed material was only observed at the center of the existing bridge and consist of a thin layer of alluvium, several feet of weathered volcanic rock. The amount of weathering decreases with depth and hard rock was found at about 5' bgs. The alluvial material is mobilized infrequently during flooding. The temporary bridge rock fill will likely be founded on the shallow alluvium and/or weathered volcanic rock.

Limitations

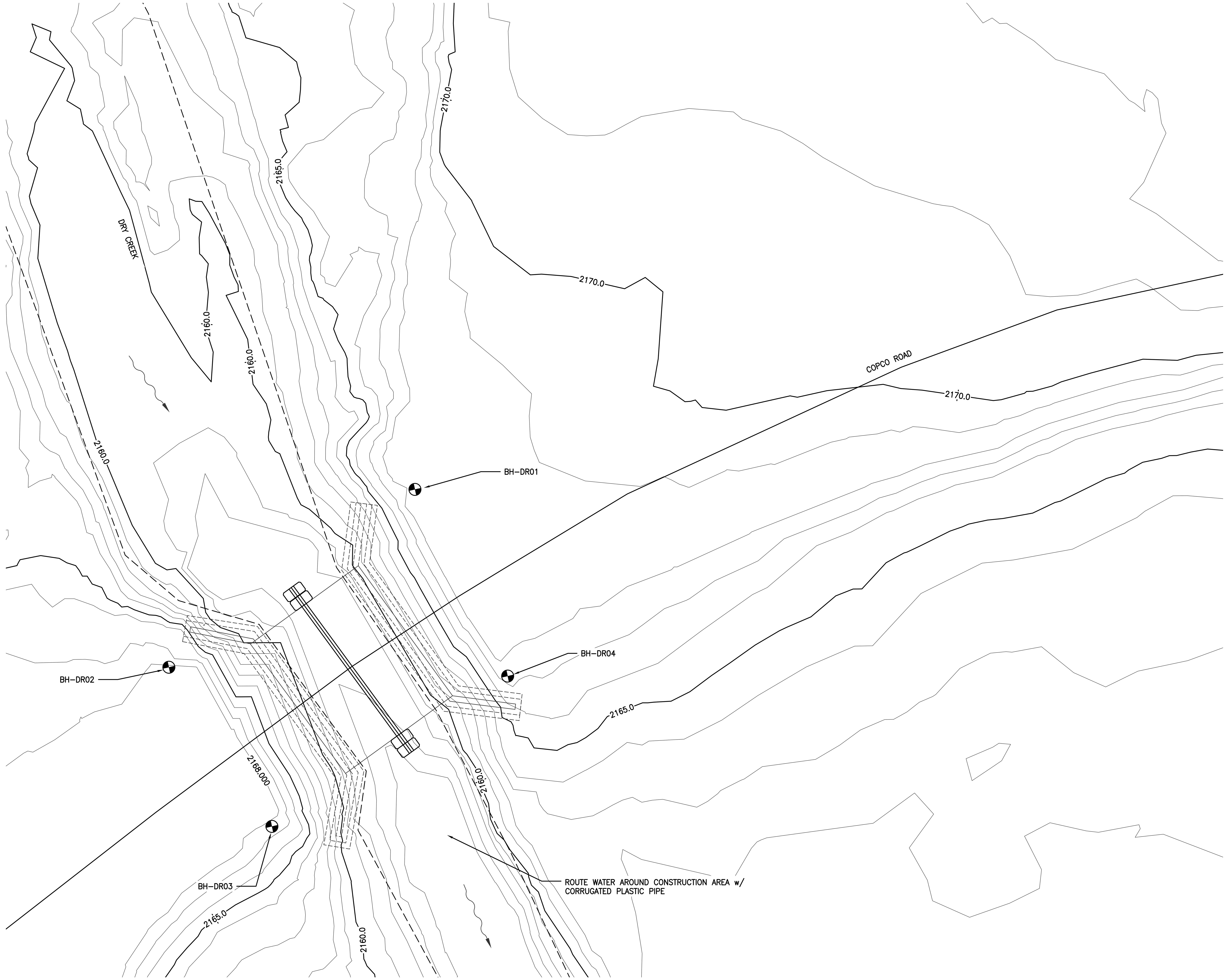
The geotechnical data presented in this report were collected following current geologic and engineering practice and the standard of care exercised by reputable professional consultants performing similar tasks in this area. The data are preliminary and subject to change. No other warranty, expressed or implied, is made regarding the data in this report. Variations may exist and conditions not observed or measured as part of this effort may exist at the site(s).

References

AECOM Technical Services, Inc., 2018. Klamath River Renewal Project, Geotechnical Data Report.

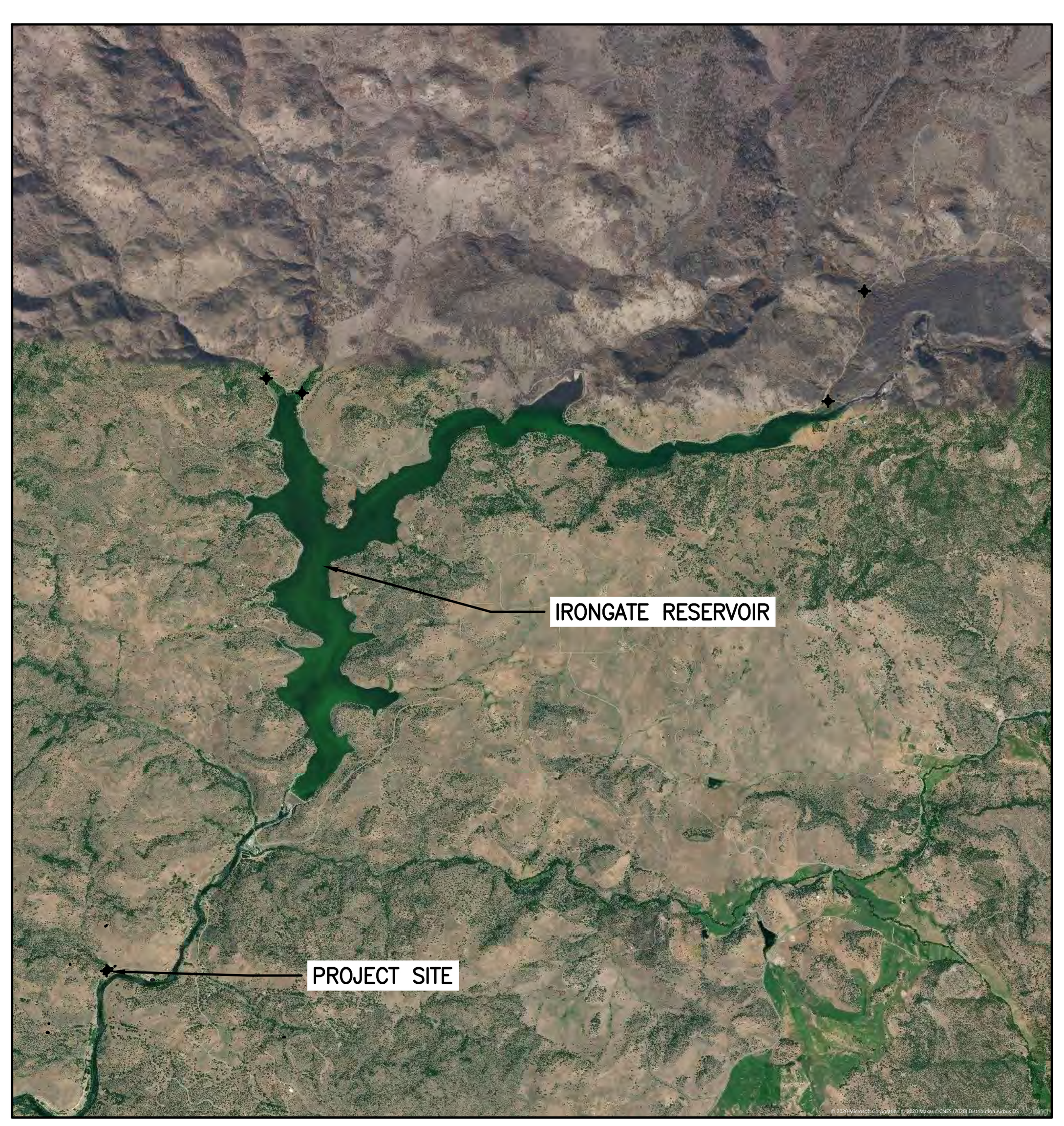
U.S. Bureau of Reclamation (BOR), 2001. Engineering Geology Field Manual, Second Edition, Volume I.

Williamson, D.A., 1984, Unified Rock Classification System: Bulletin of the Association of Engineering Geologists, Vol. XXI, No. 3, p. 345-354.



PLAN
1"= 10'

0 10 20 30
1" = 10'



N

VICINITY MAP
1" = 1 MILE

FOR INFORMATION ONLY

GEOSERV

GEOSERV, INC.

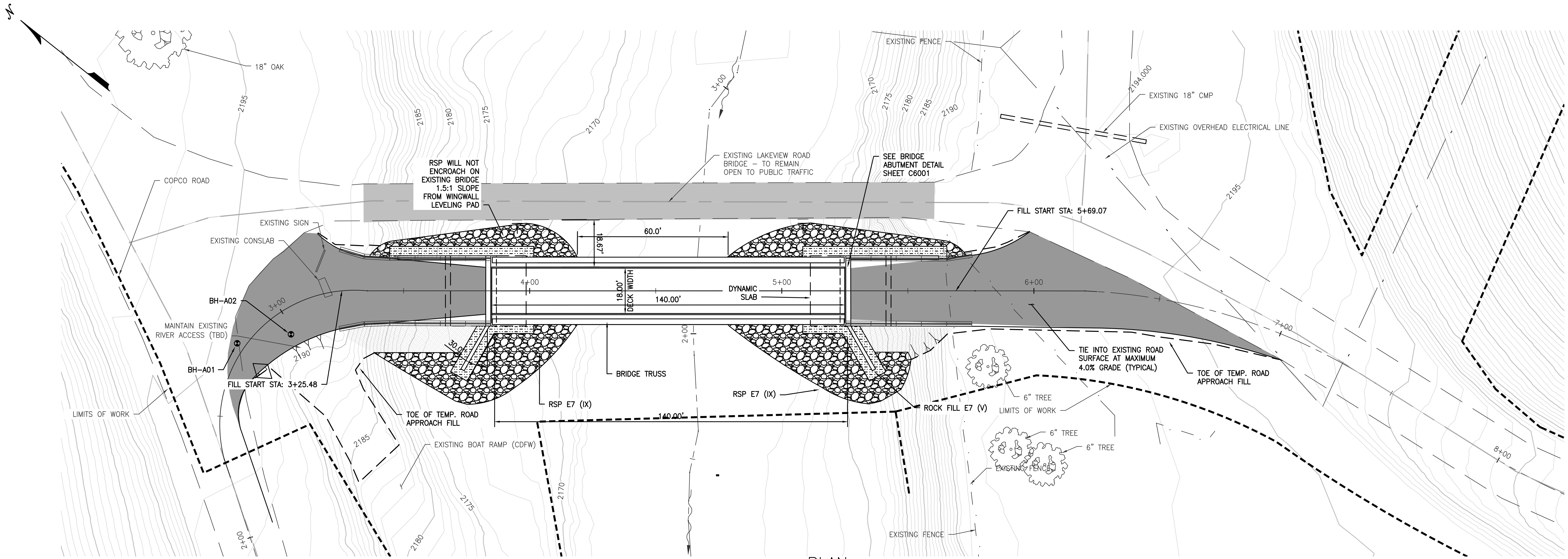
P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD at DRY CREEK BRIDGE

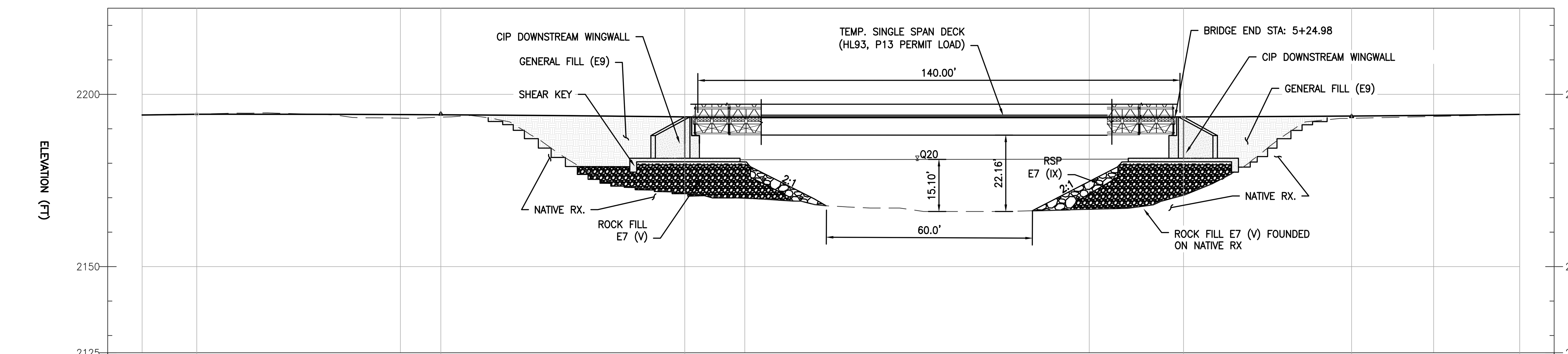
REVISIONS:

PROJECT NO: 190725
ISSUE DATE: 7/14/20
SCALE: AS NOTED
DRAWN BY: KF
ENGINEERED:
CHECKED: JF
FIGURE: 1



PLAN

1" = 20'



STATION (FT)	2+25	2+40.87	3+00	3+11.72	3+82.57	4+00	5+00	5+27.38	3+32.13	6+00	6+25
ELEVATION (FT)		2194.22		2194.29	2193.44			2193.44	2347.40		2194.18
POINT DESCRIPTION		BVC		PVI	EVC			BVC	PVI		EVC

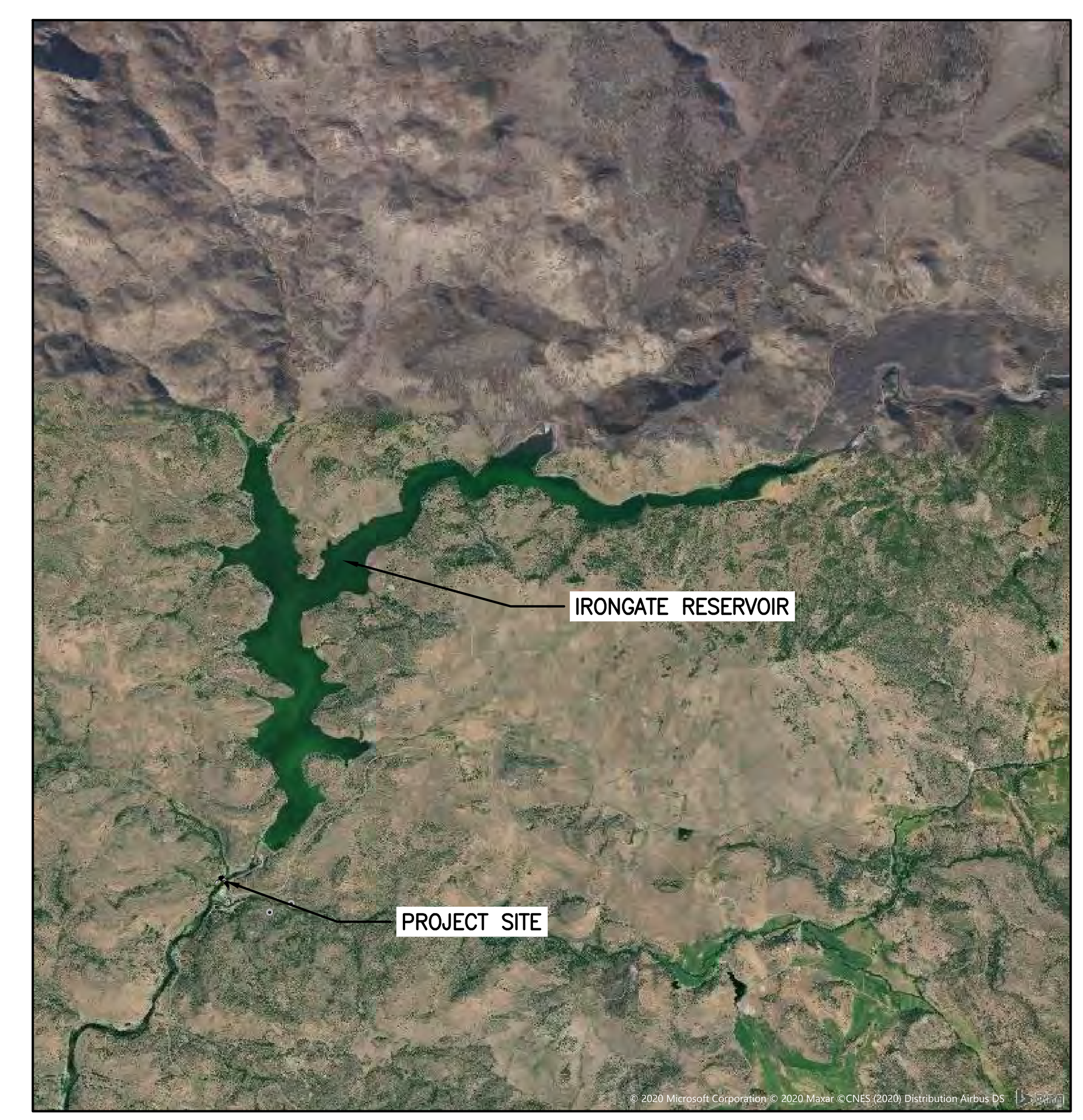
SLOPE/CURVATURE				
1.00% SLOPE	(CREST) K=108.0 LVC=141.70	0.00% SLOPE	(SAG) K=108.0 LVC=97.62	

ELEVATION

1" = 20'; 1H = 1V

0 20 40 60

1" = 20'



VICINITY MAP

1" = 1 MILE

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GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION, COPCO ROAD, HORNBROOK, SISKIYOU COUNTY

SHEET NAME:

LAKEVIEW ROAD BRIDGE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 7/14/20

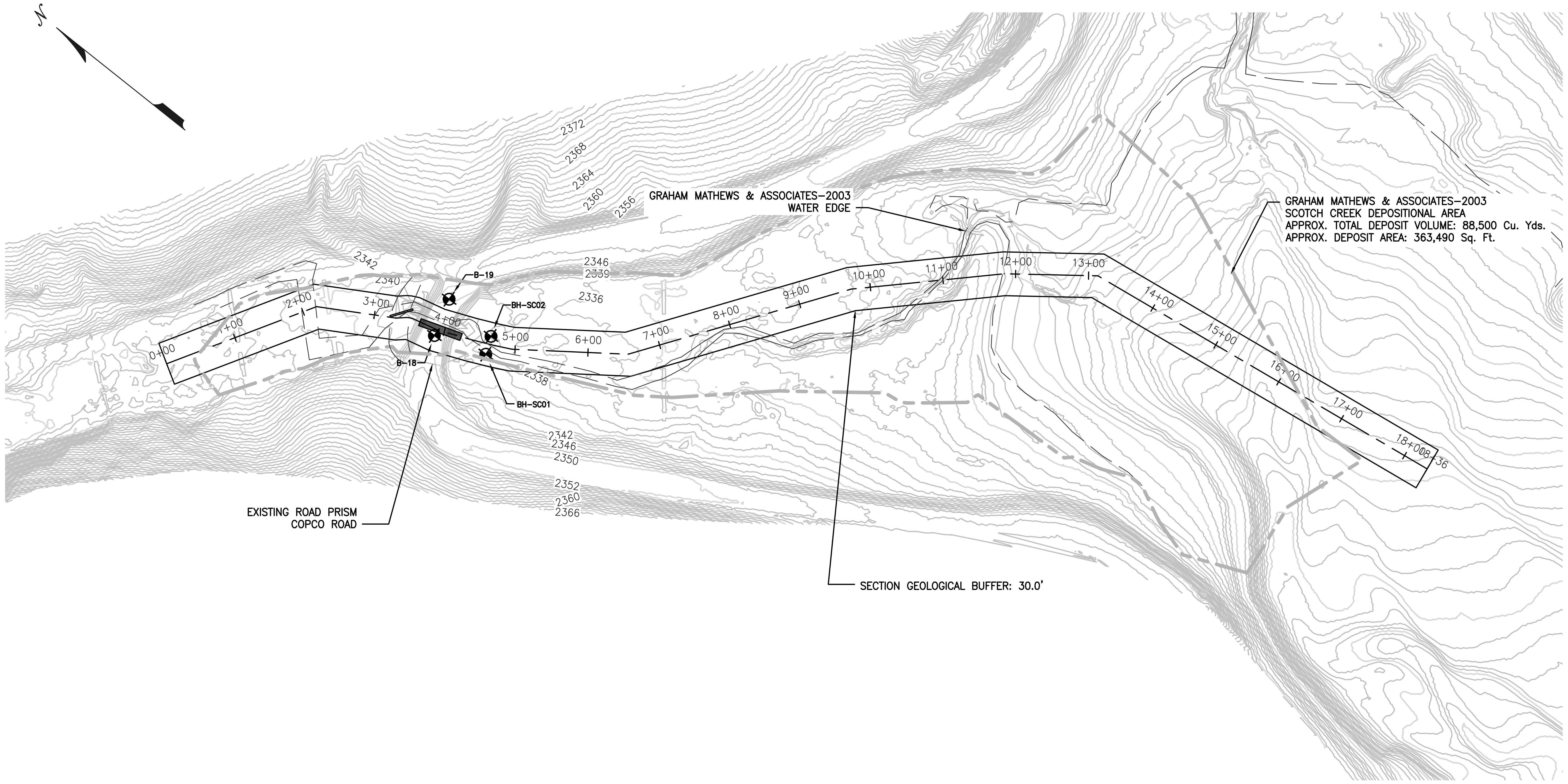
SCALE: AS NOTED

DRAWN BY: KF

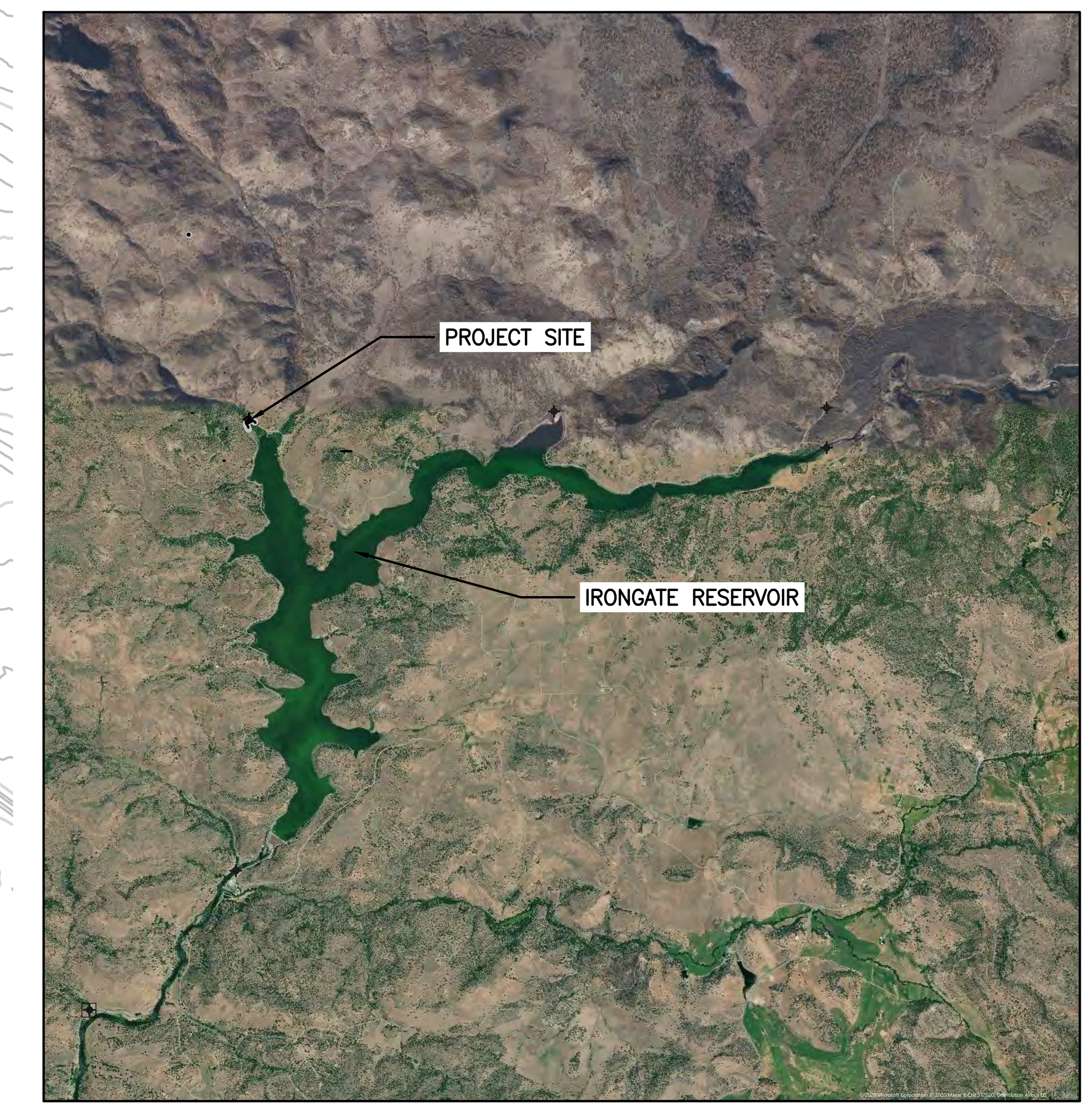
ENGINEERED:

CHECKED: JF

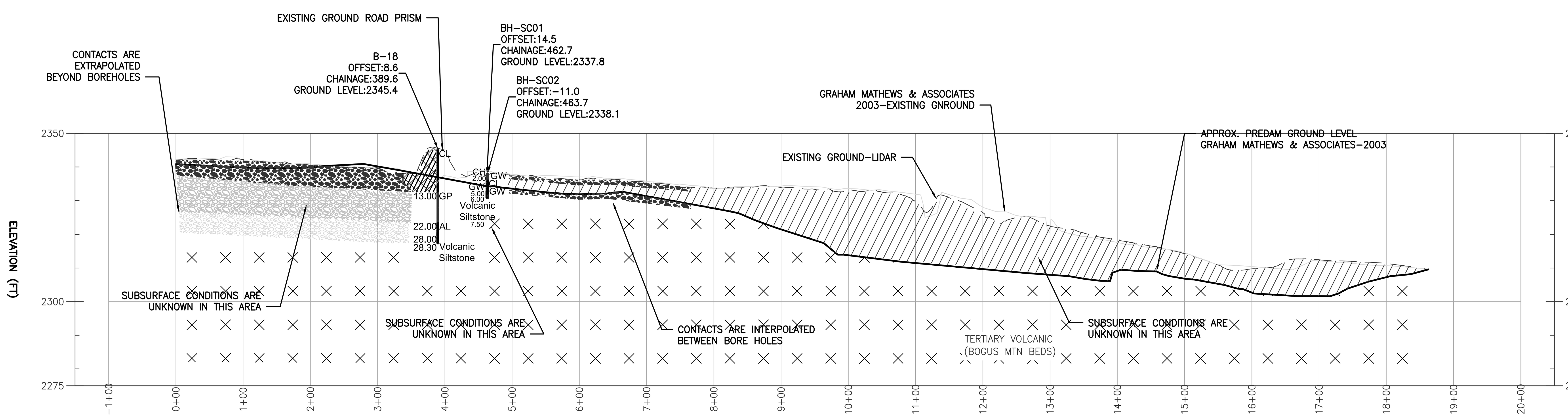
FIGURE: 2



PLAN
1"= 100'



VICINITY MAP
1" = 1 MILE



SECTION–1
1"= 100'; 5H : 1V

LEGEND				
HATCH / STRATA DESC.	STRENGTH/ DENSITY DESC.	CALC. BEARING CAPACITY (PSF)	UNDRAINED SHEAR STRENGTH (PSF)	
AC	NA			
AL	MEDIUM DENSE			
CH	VERY STIFF		2,214	
CL	MEDIUM STIFF/STIFF		1,107	
FILL	SEE INNER STRATA			
GP	MEDIUM DENSE			
GW	MEDIUM DENSE	3,305–9,148		
Volcanic Siltstone	WEAK TO VERY DENSE	17,878		

0 100 200 300
1" = 100'

FOR INFORMATION ONLY

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227–8963
FAX: (530) 926–8921

SITE INVESTIGATION, COPCO ROAD, HORNBROOK, SISKIYOU COUNTY

SHEET NAME:
SCOTCH CREEK CULVERT

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 7/14/20

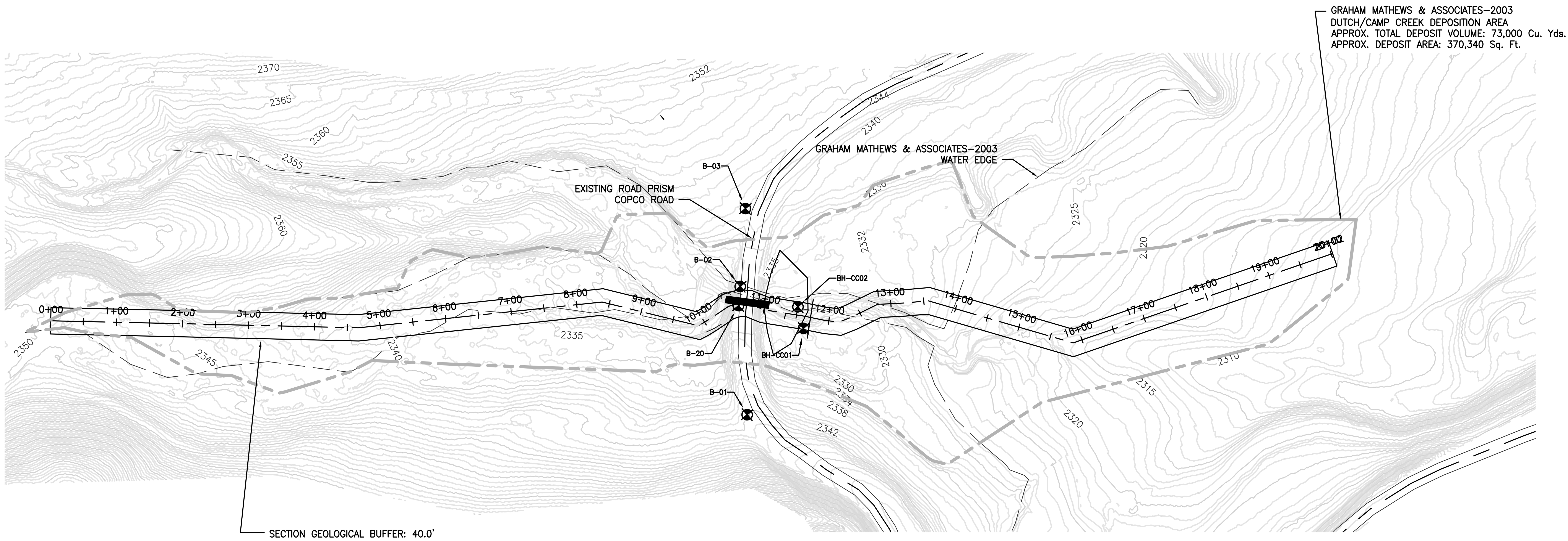
SCALE: AS NOTED

DRAWN BY: KF

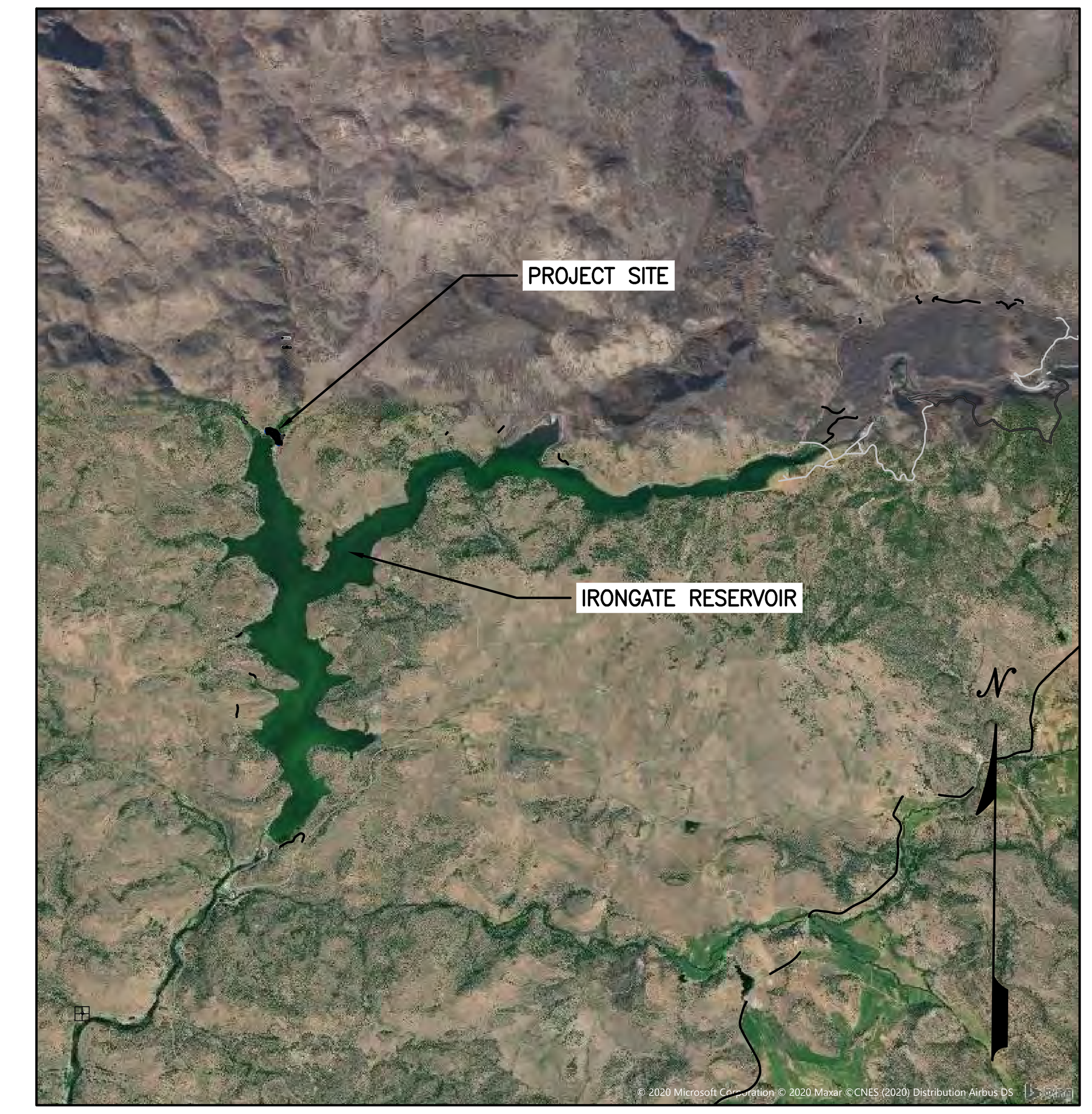
ENGINEERED:

CHECKED: JF

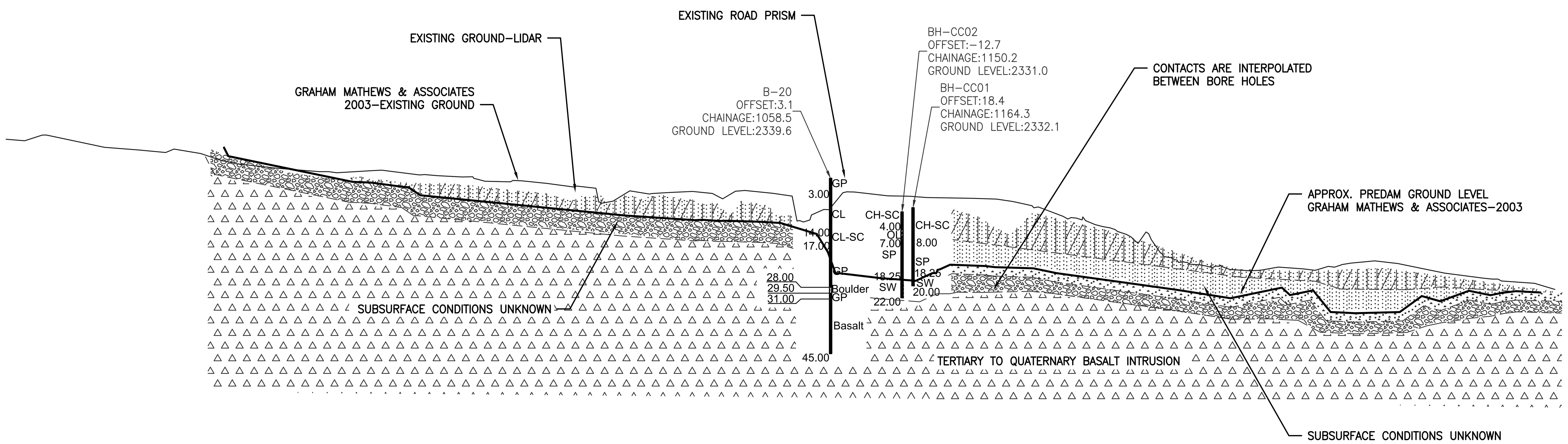
FIGURE: 3



PLAN
1"= 100'



VICINITY MAP
1" = 1 MILE



SECTION–1
1"= 100'; 5H = 1V

LEGEND

	STRATA DESCRIPTION	STRENGTH/ DENSITY DESC.	CALC. BEARING CAPACITY (PSF)	UNDRAINED SHEAR STRENGTH (PSF)
	BASALT	MODERATELY STRONG		
	BOULDER			
	BOULDER & COBBLE			
	CL	MEDIUM STIFF		
	FILL	SEE INNER STRATUM		
	CH-SC	VERY LOOSE	196-355	
	GC	VERY STIFF		
	GP	MEDIUM DENSE TO DENSE		
	OL	VERY SOFT		123
	SP	LOOSE TO MEDIUM DENSE	196-3,008	
	SW	VERY LOOSE TO DENSE	1,878-3,008	
	VOLCANIC BRECCIA	VERY WEAK		
	VOLCANIC SILTSTONE	WEAK TO MOD. STRONG		

0 100 200 300
1" = 100'

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227–8963
FAX: (530) 926–8921

SITE INVESTIGATION, COPCO ROAD, HORNBROOK, SISKIYOU COUNTY

SHEET NAME:
CAMP CREEK CULVERT

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 7/14/20

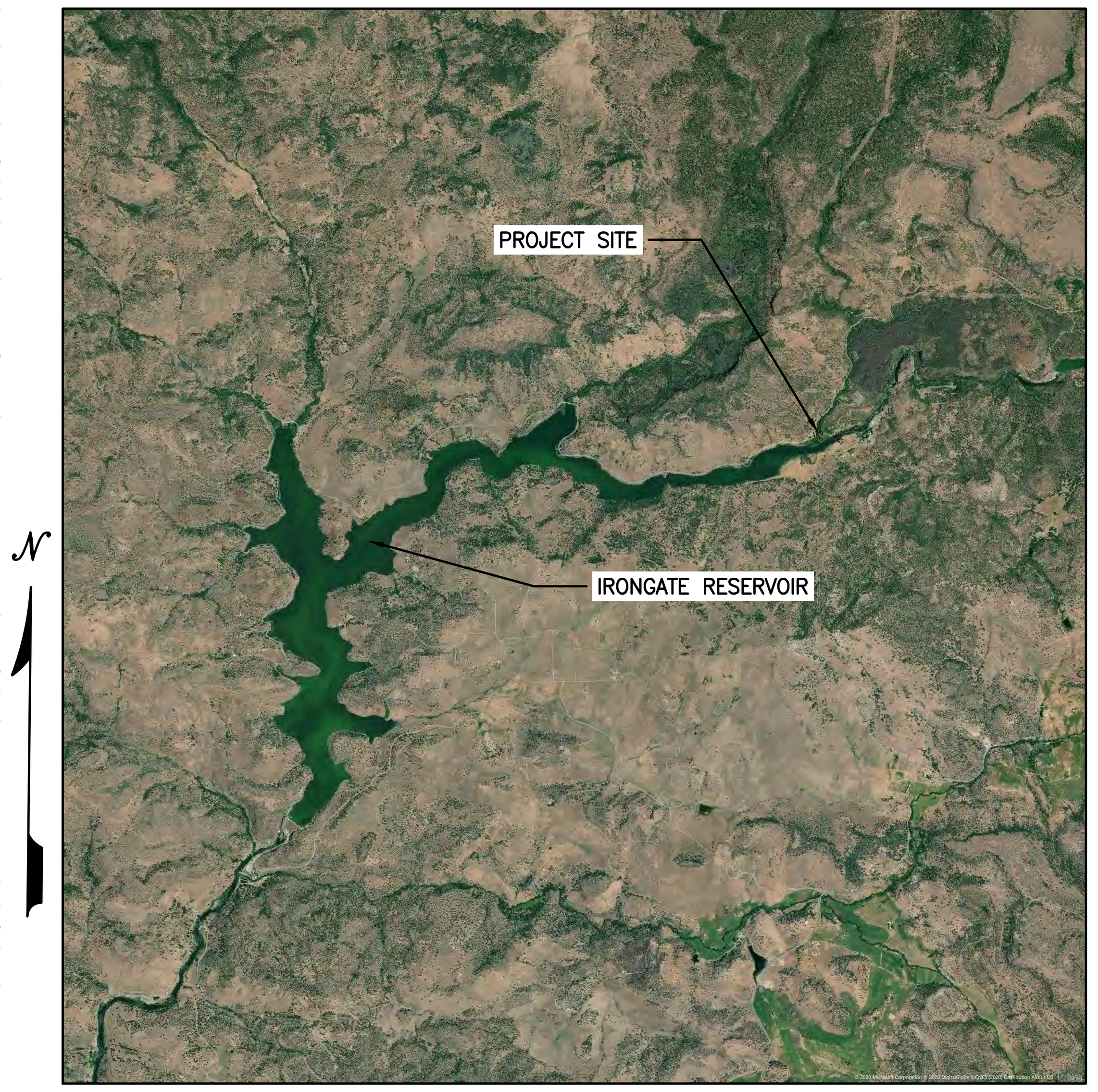
SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

CHECKED: JF

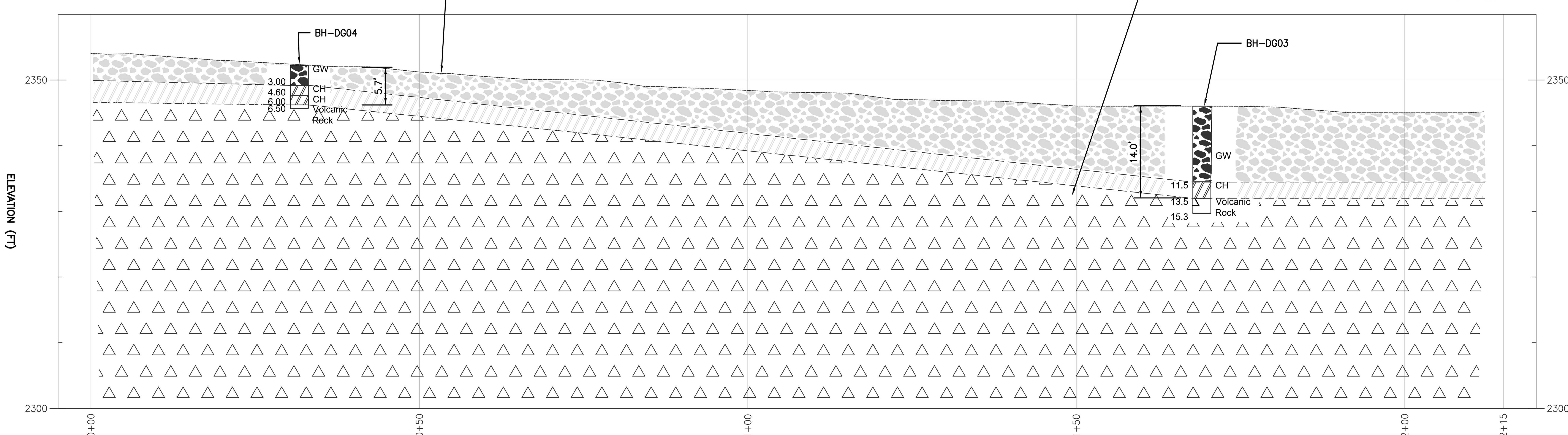
FIGURE: 4



VICINITY MAP
1" = 1 MILE

PLAN
1" = 20'

0 20 30 40
1" = 20'



SECTION
1" = 10'; 1H = 1V

LEGEND			
	CH	SANDY SILTY GRAVEL WITH CLAY TO CLAY	LOOSE TO VERY STIFF
	FILL-GW	CLAYEY SAND WITH GRAVEL COBBLES AND BOULDERS	LOOSE TO MODERATELY DENSE
	Volcanic Rock	WEATHERED Rx, HIGHLY WEATHERED TO FRESH BASALT AND ANDESITE	VERY DENSE

FOR INFORMATION ONLY

0 10 20 30
1" = 10'

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
FALL CREEK CULVERT at DAGGETT ROAD

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 7/14/20

SCALE: AS NOTED

DRAWN BY: KF

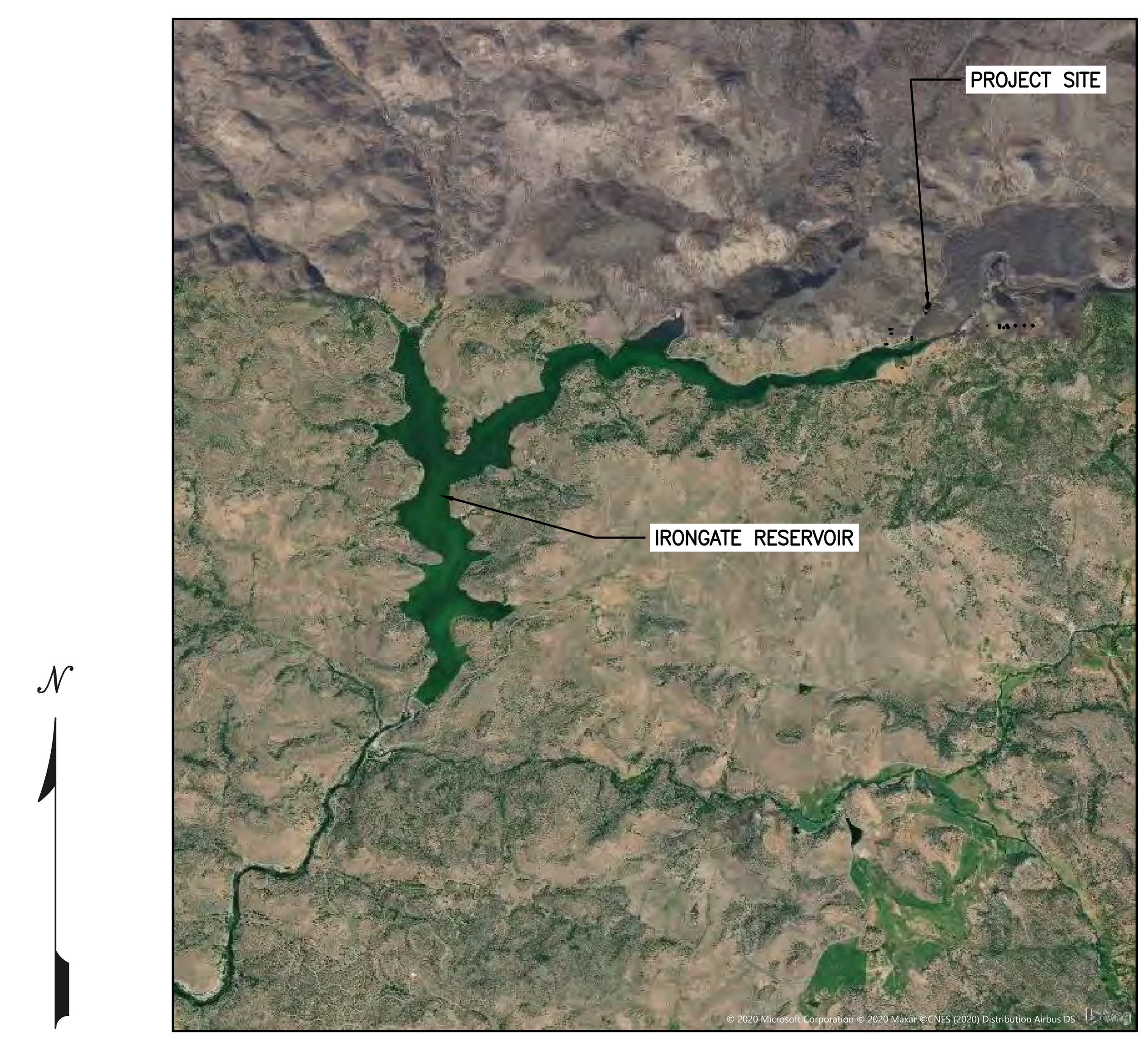
ENGINEERED:

CHECKED: JF

FIGURE: 5

PLAN
1"= 10'

0 10 20 30
1" = 10'



VICINITY MAP
1" = 1 MILE

FOR INFORMATION ONLY

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P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
FALL CREEK CULVERT
at SUBSTATION

REVISIONS:

PROJECT NO:
190725

ISSUE DATE:
7/14/20

SCALE:
AS NOTED

DRAWN BY:
KF

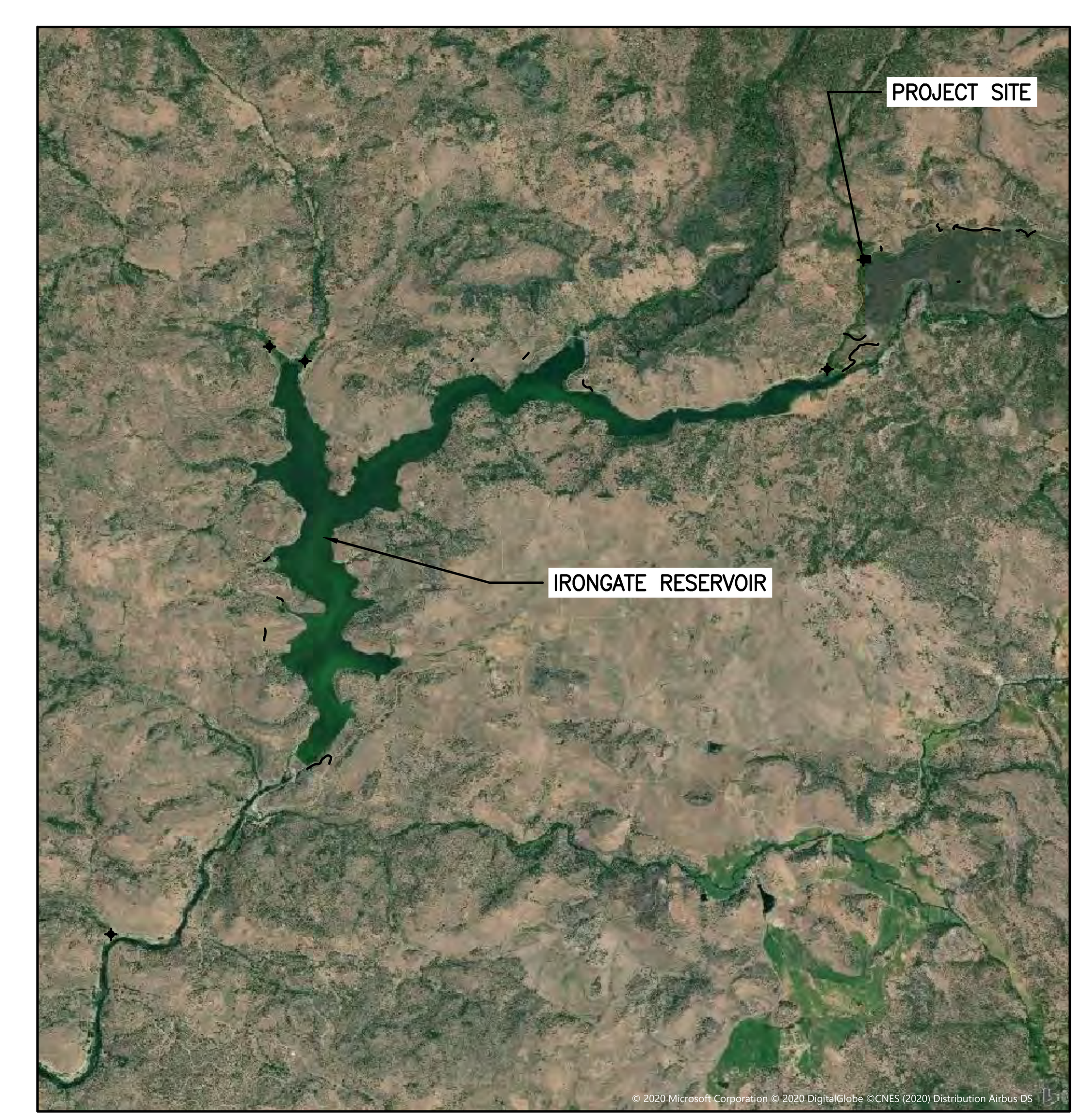
ENGINEERED:

CHECKED:
JF

FIGURE:
6

PLAN
1"= 10'

0 10 20 30
1" = 10'



VICINITY MAP
1" = 1 MILE

FOR INFORMATION ONLY

GEOSERV

GEOSERV, INC.

P.O. BOX 831
MOUNT SHASTA, CA 96067
PH: (530) 227-8963
FAX: (530) 926-8921

SITE INVESTIGATION,
COPCO ROAD, HORNBROOK,
SISKIYOU COUNTY

SHEET NAME:
COPCO ROAD at FALL CREEK BRIDGE

REVISIONS:

PROJECT NO: 190725

ISSUE DATE: 7/14/20

SCALE: AS NOTED

DRAWN BY: KF

ENGINEERED:

CHECKED: JF

FIGURE: 7

EXHIBIT A

100% FINAL Design Report_Appendix F4.4 (June2022) (CEII)
(page 15 of 91)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

PAGE REDACTED IN ENTIRETY

The redacted material qualifies as CEII pursuant to the Commission's rules because it contains sensitive dam safety and construction information that (a) relates details about the production, generation, transmission, or distribution of energy, (b) could be useful to a person planning an attack on critical infrastructure, (c) is exempt from mandatory disclosure under the Freedom of Information Act, and (d) gives strategic information beyond the location of the critical infrastructure. Accordingly, the Renewal Corporation has requested confidential treatment of this material pursuant to 18 C.F.R. § 388.113.

APPENDIX A

Borehole Logs and Data

Table 1. KRRP Transportation Geotechnical Data Borehole Data Summary Table

Feature	Borehole Number	Depth (feet)	Groundwater Depth (feet)	Material Type	Soil Type	N	N60	N1,60	Blows /Foot	Field Measured Soil Dry Unit Weight (pcf)	Field Measured Soil Wet Unit Weight (pcf)	Relative Density (N60)	Relative Density	Field Measured Friction Angle (deg)	Lab Measured Friction Angle (deg)	Lab Measured Cohesion (psf)	Undrained Shear Strength (N60) (psf)	Lab Measured Plasticity	Cohesionless Soil Density	Cohesive Soil Consistency
Camp Creek Culvert	BH-CC01	19	3.0	Well Graded Sand with Gravel	Cohesionless	34	19.6	24.9	34	123	123	55	49	30	37.5	607.0			Dense	
Camp Creek Culvert	BH-CC01	7.5	3.0	Poorly Graded Sand	Cohesionless	12	9.6	16.3	12	107	107	52	46	26					Medium Dense	
Camp Creek Culvert	BH-CC01	1	No Water	Sandy Clay to Clayey Sand	Cohesionless	3	2.4	4.1	3	90	90	24	22	20					Very Loose	
Camp Creek Culvert	BH-CC01	5	3.0	Sandy Clay to Clayey Sand	Cohesionless	2	1.6	2.7	2	100	100	20	19	19					Very Loose	
Camp Creek Culvert	BH-CC02	21.5	4.0	Clayey Sand	Cohesionless	33	26.0	31.0	33	122	122	67	75	41	43.4	39.0			Dense	
Camp Creek Culvert	BH-CC02	13	4.0	Poorly Graded Sand	Cohesionless	10	8.0	11.4	10	105	105	35	32	25					Loose	
Camp Creek Culvert	BH-CC02	7.5	4.0	Poorly Graded Sand	Cohesionless	14	11.2	19.0	14	108	108	42	38	27					Medium Dense	
Camp Creek Culvert	BH-CC02	1	4.0	Sandy Clay to Clayey Sand	Cohesionless	3	2.4	4.1	3	101	100	24	22	20					Very Loose	
Camp Creek Culvert	BH-CC02	19	4.0	Well Graded Sand with Trace Gravel	Cohesionless	3	2.4	3.0	3	100	100	18	16	20					Very Loose	
Camp Creek Culvert	BH-CC02	21	4.0	Well Graded Sand with Trace Gravel	Cohesionless	3	2.4	2.9	3	100	100	18	16	20					Very Loose	
Camp Creek Culvert	BH-CC02	5	4.0	Organic Debris with Sand	Cohesionless	1	0.8	1.4	1	43	89	16	14				123			Very Soft
Copco Road at Dry Creek Bridge	BH-DR01	3	No Water	Clayey Sand	Cohesive	13	10.4	17.7	13	106	129	56					5,538			Stiff
Copco Road at Dry Creek Bridge	BH-DR02	5.5	No Water	Sandy Clay	Cohesive	10	8.0	13.6	10	76	110	44					4,260			Firm
Copco Road at Dry Creek Bridge	BH-DR02	8	No Water	Sandy Clay	Cohesive	7	5.6	8.2	7	76	110	34					2,982			Firm
Copco Road at Dry Creek Bridge	BH-DR02	10.5	No Water	Sandy Clay	Cohesive	22	17.6	22.4	22	43	89	63			31.2	222.0	9,372	25		Very Stiff
Copco Road at Dry Creek Bridge	BH-DR03	8.5	No Water	Sandy Clay	Cohesive	9	7.2	10.2	9	106	129	39					3,834			Stiff
Copco Road at Dry Creek Bridge	BH-DR03	6	No Water	Sandy Clay	Cohesive	2	1.6	2.7	2	43	89	18					852			Very Soft
Copco Road at Dry Creek Bridge	BH-DR04	6	No Water	Sandy Clay	Cohesive	5	4.0	6.8	5	76	110	30					2,130			Firm
Copco Road at Dry Creek Bridge	BH-DR04	10	No Water	Sandy Clay	Cohesive	11	8.8	11.5	11	106	129	42					4,686			Stiff
Fall Creek at Copco Road Bridge	BH-FC01	3	No Water	Silty Clay with Gravel	Cohesive	7	5.6	9.5	7	76	110						1,713			Firm
Fall Creek at Copco Road Bridge	BH-FC01	4.5	No Water	Silty Clay with Gravel	Cohesive	14	11.2	19.0	14	106	129						3,425			Stiff
Fall Creek at Substation Road Bridge	BH-DG01	3.5	No Water	Gravelly Clay with Sand	Cohesive	34	27.2	46.2	34	106	129						8,354			Hard
Fall Creek at Substation Road Bridge	BH-DG01	7.5	No Water	Gravelly Clay with Sand	Cohesive	26	20.8	31.2	26	43	89						6,391			Very Stiff
Fall Creek at Substation Road Bridge	BH-DG02	8.5	No Water	Rock	Cohesionless	33	26.4	25.3	33	155	155	61	69	44					Dense	
Fall Creek at Substation Road Bridge	BH-DG02	3.5	No Water	Clay with Sandy Gravel	Cohesive	14	11.2	19.0	14	106	129						3,446			Stiff
Fall Creek Culvert at Daggett Road	BH-DG03	15	No Water	Rock	Cohesionless	50	40.0	46.8	100	132	170	99	113	49					Very Dense	
Fall Creek Culvert at Daggett Road	BH-DG03	11	No Water	Sandy Clay	Cohesive	9	7.2	9.1	9	106	129						2,214	46		Stiff
Fall Creek Culvert at Daggett Road	BH-DG04	3.5	No Water	Sandy Silty Clay	Cohesive	12	9.6	16.3	12	106	129						2,945			Stiff
Fall Creek Culvert at Daggett Road	BH-DG04	5	No Water	Clay	Cohesive	21	16.8	28.6	21	43	89						5,159			Very Stiff
Lakeview Road Bridge	BH-AB01	25	13	Rock	Cohesionless	33	26.0	24.7	33	155	155	63	57	45					Dense	
Lakeview Road Bridge	BH-AB01	10	No Water	Clay with Gravel	Cohesionless	25	20.0	32.6	25	116	116	58	64	42					Medium Dense	
Lakeview Road Bridge	BH-AB01	6.5	No Water	Clay with Gravel	Cohesionless	21	16.8	28.6	21	113	113	60	66	41					Medium Dense	
Lakeview Road Bridge	BH-AB01	3	No Water	Fill	Cohesionless	20	16.0	27.2	20	113	113	59	65	41					Medium Dense	
Lakeview Road Bridge	BH-AB01	15	13	Clay with Gravel	Cohesive	15	12.0	13.2	15	106	129						3,697			Stiff
Lakeview Road Bridge	BH-AB01	20	13	Sandy Clay	Cohesive	26	20.5	16.7	26	43	89						5,033			Very Stiff
Lakeview Road Bridge	BH-AB02	20	10	Rock	Cohesionless	28	17.2	18.4	28	155	155	49	54	41					Medium Dense	
Lakeview Road Bridge	BH-AB02	15	10	Clay with Sand	Cohesive	3	2.4	2.8	3	76	110						737			Firm
Lakeview Road Bridge	BH-AB02	6.5	No Water	Clay with Gravel	Cohesive	18	14.4	23.2	18	106	129						4,428			Stiff
Lakeview Road Bridge	BH-AB02	10	10	Clay with Gravel	Cohesive	15	12.0	15.7	15	106	129						3,697			Stiff
Scotch Creek Culvert	BH-SC01	6.5	No Water	Clayey Gravel and Sand	Cohesionless	21	16.8	29.0	21	113	113	78	69	33					Medium Dense	
Scotch Creek Culvert	BH-SC01	1	No Water	Sandy Gravely Cobbles	Cohesionless	15	12.0	20.0	15	109	109	66	60	30					Medium Dense	
Scotch Creek Culvert	BH-SC01	7	No Water	Rock	Cohesionless	50	40.0	68.0	110	170	170	100	90	37					Very Dense	
Scotch Creek Culvert	BH-SC01	4	No Water	Sandy Clay	Cohesive	9	7.2	12.2	9	106	129	35	31				1,107			Stiff
Scotch Creek Culvert	BH-SC02	3.5	No Water	Sandy Gravely Cobbles	Cohesionless	31	24.8	42.2	31	120	120	84	74	32					Dense	
Scotch Creek Culvert	BH-SC02	6.5	No Water	Rock	Cohesionless	50	40.0	68.0	112	170	170	100	90	37					Very Dense	
Scotch Creek Culvert	BH-SC02	1	No Water	Sandy Clay with Cobbles	Cohesive	18	14.4	24.5	18	43	89	73	65				2,214			Very Stiff
Daggett Road Bridge at KR	B-15	15.5	No Water	Clayey Gravel and Sand	Non-cohesive	50	40.0	54.8	50	134				31					Very Dense	

BORE HOLE SOIL AND SYMBOL LEGEND

CH - Inorganic Clays of High Plasticity, Fat Clays.

CL - Inorganic Clays or Low to Medium Plasticity, Gravelly Clays, Sandy Clays, Silty Clays, Lean Clays.

Fill - Artificial Fill.

GC - Clayey Gravels, Gravel-Sand-Clay Mix.

GP - Poorly-Graded Gravels and Gravel-Sand Mix, Little or No Fines.

GP - Well-Graded Gravels and Gravel-Sand Mix, Little or No Fines.

OL - Organic Silts and Organic Silty Clays of Low Plasticity.

SC - Clayey Sands, Sand-Clay Mix.

SP - Poorly-Graded Sands and Gravelly Sands, Little or No Fines.

SM - Silty Sands and Sand-Silt Mix.

SW - Well-Graded Sands and Gravelly Sands, Little or No Fines.

Weathered Rock - Weathered Quaternary Rock; Mainly Breccia.

Weathered Volcanic Rock - Weathered Tertiary Flows; Mainly Basalt and Andesite.

Volcanic Siltstone - Weathered Tertiary Flows.

Volcanic Breccia.

SYMBOL LEGEND

SPT 1.5" Sample

Bulk Sample

Groundwater Level (After Completion)

Groundwater Level (Before Completion)

LOG COLUMN DESCRIPTION

1. Elevation: Elevation (in feet) referenced to mean sea level (MSL).
2. Depth: Distance (in feet) below the collar of the borehole.
3. Recovery: Amount (in percent) of core recovered from the coring interval; calculated as length of core recovered divided by run length.
4. Soils and Lithology: A graphic log of material encountered using symbols to represent differing soil and rock types; symbols are explained above.
5. Description: Lithologic description in this order: rock type, color, texture, grain size, weathering, strength, and other features.
6. Drill Rate: Time marking start and finish of each run; drill rate (ft/min).
7. Drill Torque (lb/ft): Measure of total down pressure relative to material strength and drill bit refusal.
8. Compressive Strength: measured load at failure use pocket penetrometer.
9. Field Notes and Other Tests: Comments regarding drilling and sampling made by driller or logger.

REPORT TITLE	KRRP Roads, Bridges, and Culverts
COUNTY	Siskiyou
PROJECT OR BRIDGE NAME	Geotechnical Data Reports
PREPARED BY	JF, JS, KF
DATE	10/07/20
SHEET	1

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 16th, 2020	April 16th, 2020	41.97665, -122.36415	BH-DG01

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2387.0'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020		ND	ND	10.9'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–1.5		Fill: ARTIFICIAL FILL - SANDY GRAVEL: (GW): dark brown; dry to moist; medium dense; SAND very fine to coarse; GRAVEL< 1.5" dia., angular to subangular.												
2385	1.5–9.5		CH: CLAY with SANDY GRAVEL: (CH); dark brown; moist; firm; CLAY medium to high plasticity; sand very fine; GRAVEL < 0.375" dia, angular to subangular, GRAVEL occurs below 5.5' bgs.												
	3.5–5					3/5/9	14	80	14/11.3/19			3,446			Bulk sample taken @ 1.5'-5.5' bgs
2380	8.5–10					5/10/23	33	80	33/26.4/25.3	44	9,003				Harder drilling @ 9.5' bgs
	9.5–10.9		Weathered Rx: WEATHERED ROCK; high weathering to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.												Auger Refusal @ 10.9' bgs
2375	12														

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-DG01
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Fall Creek culvert at Substation				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 13

GEOSERV

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 18th, 2020	April 18th, 2020	41.97676, -122.36455	BH-DG02

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2386.0'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020		ND	ND	9.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL - GRAVELLY CLAY with SAND (CH): dark reddish brown; moist; stiff to very stiff; CLAY medium to high plasticity; SAND coarse; GRAVEL< 1.0" dia., angular to subrounded.												
2385	1														
	2														
	3														
	4					7/14/20	34	30	34/27.2 /85.5			3,858			
	5														
2380	6														
	7		CH: GRAVELLY CLAY with SAND: (CH); dark reddish brown; moist; stiff to very stiff; CLAY medium to high plasticity; SAND coarse; GRAVEL < 0.5" dia, subrounded to round.												Easier drilling @ 7.0' bgs
	8					7/12/14	26	50	26/20.8 /51.2			5,891			Auger Refusal @ 9.0' bgs
	9														
	10														
2375	11														
	12														
	13														

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-DG02
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Fall Creek culvert at Substation				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 14

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 16th, 2020	April 16th, 2020	41.97318588, -122.36643994	BH-DG03
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			NA	2345'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Deere 35G	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING
Bentonite Chip, 4/18/2020				9.3'

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Remarks
2345	0	Fill: ARTIFICIAL FILL-CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 8" dia., subangular.								
2340	5									
2335	10									
	10.5	CH: SANDY CLAY (CH); dark brown; moist; very soft to firm; CLAY medium to high plasticity; SAND coarse to very coarse; trace organic debris.	3/4/5	9	30	9/7.8/ 9.1			2,214	PI 46
	13	Weathered Rx: WEATHERED ROCK; highly weathered to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.								Harder drilling @ 13.0' bgs Max Drill Effort/Torque @ 14.0' bgs
2330	15		50:5"		50	50/40/ 46.6	49	17,557		Auger Refusal @ 15.3' bgs below existing top of road

REPORT TITLE				HOLE ID
Geotechnical Investigation, Daggett Road				BH-DG03
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Fall Creek Culvert at Daggett Road				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 11

GEOSERV

LOGGED BY JF & KF	BEGIN DATE April 17th, 2020	COMPLETION DATE April 17th, 2020	BOREHOLE LOCATION (Lat/Long or North/East and Datum) 41.97329581, -122.36694418	HOLE ID BH-DG04
DRILLING CONTRACTOR GeoServ, Inc.			BOREHOLE LOCATION (Offset, Station, Line) NA	SURFACE ELEVATION 2352.2'
DRILLING METHOD Hollow Stem Auger			DRILLING RIG Lonestar Drill	BOREHOLE DIAMETER 6"
SAMPLER TYPE(S) AND SIZES (ID) SPT 1.5"			SPT HAMMER TYPE Safety Hammer	HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION Bentonite Chip, 4/18/2020			GROUNDWATER READINGS — DURING DRILLING: ND — AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING 6.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–3		Fill: ARTIFICIAL FILL-CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 8" dia., subangular.												
2350															
	3–5		CH: SANDY SILTY CLAY WITH GRAVEL (CH); dark brown; dry to moist; very soft to firm; CLAY medium to high plasticity; SAND coarse to very coarse; GRAVEL less than 1.5in diameter angular to subangular.			7/6/6	12	20	12/9.6/16.[illegible]			3,845			
	5–6		Clay: CLAY (CH); dark brown, moist; firm to very stiff; clay medium to high plasticity; harder drilling with depth.			5/8/13	21	90	21/16.8/28.8			5,150			Hard Drilling Max Drill Effort/Torque @ 5.0' bgs Auger Refusal @ 6.5' bgs.
	6–6.5		Weathered Rx: WEATHERED ROCK; highly weathered to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.												
2345	7														
2340	12														

GEOSERV

REPORT TITLE Geotechnical Investigation, Daggett Road					HOLE ID BH-DG04
DIST.	COUNTY Siskiyou	ROUTE	POSTMILE		EA
PROJECT OR BRIDGE NAME Fall Creek Culvert at Daggett Road					
BRIDGE NUMBER NA		PREPARED BY KF		DATE 7/14/2020	SHEET 12

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Jan 29th, 2020	Jan 30th, 2020	41.93146, -122.4421	BH-A01

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	None	2194.21'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Air Rotary Drill	T1	6" to 10"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement, 1/31/20		18.0'	13.0' (1/30/20)	35'

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Remarks
	0–1	GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dry; <3/4" diameter.								Hollow Stem Auger	
2190	1–6	GP: GRAVELS COBBLES, and BOULDERS (Fill-GP); loose to dense; dry; subangular to angular GRAVEL/COBBLE/BOULDER; <18" BOULDER.	3/10/10	20	0	20/16.0 /27.0	41	4,850			Sample NO. 1.1-STP
2185	6–18	CH: CLAY with GRAVEL (Fill-SC); firm; moist; reddish brown; CLAY medium to high plasticity; <1/2" subround to angular GRAVEL.	4/9/12	21	30	21/16.8 /28.6	41	5,305			Sample NO. 1.2-SPT
			10/12/13	25	100	58/68/ 45	42	6,767			Sample NO. 1.3-SPT
2180			5/6/9	15	100	15/12.0 /15.2			3,897		Sample NO. 1.4-SPT
2175	18–23	CH: CLAY with GRAVEL to Gravelly Clay (CH-GC); very soft to firm; moist to wet; dark grey; CLAY medium to high plasticity; <1/2" subround to angular GRAVEL.	2/16/10	26	60	26/20.5 /16.7			5,033		Sample NO. 1.5-SPT
2170	23–35	Weathered Volcanic Rx: NATIVE WEATHERED ROCK; hard; CLAYEY GRAVEL WITH SAND; Clay medium to high plasticity; <0.375" subangular to angular GRAVEL; coarse to very coarse SAND; weathering decreases with depth.	50: 0.25'		10	33/26.0 /24.7	45	9,774		Tri-Cone	Sample NO. 1.6-SPT
2165											
2160	35–36										

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-A01
DIST.	COUNTY Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Lakeview Road Bridge				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 1

GEOSERV

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	Jan 30th, 2020	Jan 30th, 2020	41.93143, -122.44202	BH-A02

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	None	2188.92'

DRILLING METHOD	DRILLING RIG	BOREHOLE DIAMETER
Air Rotary Drill	T1	6" to 10"

SAMPLER TYPE(S) AND SIZES (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement, 1/31/20		15.5'	10.0' (1/31/20)	30'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–1		GW: WELL GRADED AGGREGATE BASE ROCK (FILL-GW); dry; <3/4" diameter.										Hollow Stem Auger		
2185 (at 4)	1–5		GP: GRAVELS COBBLES, and BOULDERS (Fill-GP); loose to dense; dry; subangular to angular GRAVEL/COBBLE/BOULDER; <18" BOULDER.												
2180 (at 9), 2175 (at 14)	5–15.5		CH: CLAY with GRAVEL (Fill-CH); firm; moist; reddish brown; CLAY medium to high plasticity; <1/2" subround to angular GRAVEL.												
	7.5–9			SPT		5/7/11	18	30	18/14.4/23.2			4,476			Sample NO. 2.1-SPT
	10		(groundwater)												
	10.5–12			SPT		4/7/8	15	100	15/12.0/15.7			3,692			Sample NO. 2.2-SPT
	15.5–18		SC: CLAY with SAND to SAND with ORGANIC DEBRIS (CH-SC); very soft to firm; moist to wet; greenish grey; CLAY medium to high plasticity; fine to very fine SAND; organic debris throughout up to 1/8"x1/2" in size.												
	15.5–17			SPT		1/1/2	3	100	3/2.4/2.6			737			Sample NO. 2.3-SPT
2170 (at 19), 2165 (at 24), 2160 (at 29)	18–30		Weathered Volcanic Rx: NATIVE VOLCANIC WEATHERED ROCK; hard; CLAYEY GRAVEL WITH SAND; Clay medium to high plasticity; <0.375" subangular to angular GRAVEL; coarse to very coarse SAND; weathering decreases with depth; preserved amygdules.										Tri-Cone		
	20.5–22			SPT		18/10/18	28	60	28/17.2/18.4	41	5,472				Sample NO. 2.4-SPT
	25–30														Sample NO. 2.5-SPT
2155 (at 34)	30–36														

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-A02
DIST.	COUNTY	ROUTE	POSTMILE	EA
	Siskiyou			
PROJECT OR BRIDGE NAME				
Lakeview Road Bridge				
BRIDGE NUMBER	PREPARED BY		DATE	SHEET
NA	KF		7/14/2020	6

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 14th, 2020	April 14th, 2020	41.97509531, -122.44002545	BH-SC01

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2338'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/14/2020		ND	ND	7.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–2		GW: SANDY GRAVELY COBBLES and BOULDERS (GW); tannish brown; dry to moist; medium dense; SAND fine to coarse; GRAVEL < 1.5" dia. and subangular to rounded; COBBLE <5" dia. and subangular to rounded; BOULDER < 12" dia. and subangular to rounded.			7/8/9	15	60	15/12.0/20.0	30.0	3,000				
2335 (at 3 ft)	2–5		CL: SANDY CLAY (CL); reddish brown; moist; firm; CLAY medium plasticity; SAND fine.			3/4/5	9	100	9/7.3/12.2			1,107			Medium Drilling Torque/Effort Bulk sample 2'-6' Medium Drilling Torque/Effort
	5–7		GW: CLAYEY GRAVEL and SAND (GW); reddish brown; moist; firm; CLAY medium plasticity; SAND fine to coarse; GRAVEL < 1.5" angular to subangular.			7/14/7	21	60	21/16.8/29.0	38.8	5,205				
2330 (at 8 ft)	7–7.5		Weathered Volcanic Rx: VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; very dense. --Tertiary Volcanics (BOGUS MOUNTAIN BEDS, undifferentiated)--			50-0"		0	50/40.0/68.0	37.0	17,676				Max. Drilling Torque/Effort Auger refusal @ 7.5'

Depth scale: 0 1 2 3 4 5 6 7 8 9 10 11 12 13

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-SC01
DIST.	COUNTY Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME Scotch Creek Culvert				
BRIDGE NUMBER NA	PREPARED BY KF		DATE 7/14/2020	SHEET 7

LOGGED BY: JF & JS	BEGIN DATE: April 13th, 2020	COMPLETION DATE: April 13th, 2020	BOREHOLE LOCATION (Lat/Long or North/East and Datum): 41.97511937, -122.43993749	HOLE ID: **BH-SC02**
DRILLING CONTRACTOR: GeoServ, Inc.			BOREHOLE LOCATION (Offset, Station, Line): NA	SURFACE ELEVATION: 2338.1'
DRILLING METHOD: Hollow Stem Auger			DRILLING RIG: Lonestar Drill	BOREHOLE DIAMETER: 6"
SAMPLER TYPE(S) AND SIZES (ID): SPT 1.5"			SPT HAMMER TYPE: Safety Hammer	HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION: Bentonite Chip, 4/14/2020			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING: 7.0'

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Remarks
	0–2	CH: SANDY CLAY with COBBLES and ORGANIC DEBRIS (CH); dark brown; moist to wet; firm; CLAY medium to high plasticity; Sand very fine to fine; COBBLE <6" dia. and subangular to rounded; ORGANIC DEBRIS < 0.5" dia. roots..	3/4/14	18	60	18/14.4/24.5			2,214	Medium Drill Effort/Torque
2335	2–5	GW: SANDY GRAVEL with COBBLES (GW); tannish brown; moist to wet; medium dense; SAND fine to coarse; GRAVEL < 1.5" sub rounded to rounded; COBBLES < 4" dia. and subrounded to rounded.	6/14/17	31	100	31/24.8/43.2	32.6	9,188		Max. Drilling Torque/Effort
	5–7	Weathered Volcanic Rx: VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; very dense. --Tertiary Volcanics (BOGUS MOUNTAIN BEDS, undifferentiated)--	50-6"		0	50/40.0/63.0	37.0	17,675		Auger refusal @ 7.0'
2330	8									
2325	13									

Other columns: Material Graphics, Sample Location, Sample Number, Drilling Method, Casing Depth.

GEOSERV

REPORT TITLE: Geotechnical Investigation, Copco Road				HOLE ID: BH-SC02
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Scotch Creek Culvert				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 8

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil Boring B-18

Sheet 1 of 1

Date(s) Drilled	**10/11/2018**	Logged By	**P. Respess**	Checked By	
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**6-inch flight auger**	Total Depth of Borehole	**28.3 feet**
Drill Rig Type	**Truck Mounted Mobile B-53**	Drilling Contractor	**Gregg Drilling**	NAVD 88 Ground Surface Elevation	**2347 feet**
Groundwater Level(s)	**15.0 feet below ground surface (10/11/2018)**	Sampling Method(s)	**2.5-inch ID ModCal, SPT**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**Scotch Creek**	Coordinate Location	**N 2603261 E 6442042**

Elevation feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resistance	SAMPLES Recovery (feet)	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Fines Content (% <#200 Sieve)	REMARKS AND OTHER TESTS
	0						2.5-inches ASPHALT roadway				Start 10/11/2018; hollow stem auger 0-28ft.
							GRAVEL				
2345							SANDY LEAN CLAY (CL); medium stiff to stiff; reddish brown; 80-90% medium plasticity FINES; 10-20% fine to coarse grained SAND; occasional GRAVEL and COBBLE --FILL--				Smooth drilling
	5						GRAVEL				Rig chatter
2340											Return to smooth drilling to 13ft.
	10										
2335											
							POORLY GRADED GRAVEL with SAND (GP); medium dense; varied dark grey with purple, red, and yellowish brown; fine to coarse angular GRAVEL, COBBLES, and BOUDLERS; fine to coarse grained SAND --FILL--*(continued)*				Rig chatter
	15						▽				
2330											
							BOUDLER				Driller indicates hard rock at 18ft.
	20										
2325							ALLUVIUM				Driller indicates smooth, consistent drilling 22-25ft.
	25		S-01	50/3	0						
			S-02	100/4			BOUDLER, basalt				
2320											
			S-03	100/4			VOLCANIC SILTSTONE; reddish purple; slightly weathered to fresh; weak to moderately strong; very thinly laminated --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS)?--				
	30						**TOTAL DEPTH = 28.3 FEET**				

Report: GEO_10B1_OAK; File: ROCK CORES.GPJ; 11/14/2018 B-18

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-19

Sheet 1 of 3

Date(s) Drilled	**10/11/2018**	Logged By	**P. Respess**	Checked By	
Drilling Method	**Hollow Stem Auger, Rotary Wash, HQ-3 Rock Core**	Drill Bit Size/Type	**3 7/8-inch tricone; 3 7/8-inch #6 HQ bit**	Total Depth of Borehole	**37.5 feet**
Drill Rig Type	**Truck Mounted Mobile B-53**	Drilling Contractor	**Gregg Drilling**	NAVD 88 Ground Surface Elevation	**2346 feet**
Groundwater Level	**15.0 feet below ground surface (10/11/2018)**	Sampling Methods	**2.5-inch ID ModCal, SPT, HQ Core Barrel**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**Scotch Creek**	Coordinate Location	**N 2603258 E 6442034**

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2346	0								2.5-inches ASPHALT roadway GRAVEL --ROAD BASE--						Start 10/11/2018; hollow stem auger 0-23ft.
	1								SANDY LEAN CLAY (CL); medium stiff to stiff; reddish brown; 80-90% medium plasticity FINES; 10-20% fine to coarse grained SAND; occasional GRAVEL and COBBLE --FILL--						
2344	2														
	3														
2342	4														
	5								Becomes light reddish brown, with low to medium plasticity FINES		S01	9 8 10			S-01 One liner retained (6-6.5ft.)
2340	6														
	7														
2338	8														
	9														
2336	10										S02	9 7 7			S-02 One liner retained (11-11.5ft.)
	11														
2334	12														
	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-19

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-19

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13							POORLY GRADED GRAVEL with SAND (GP); medium dense; dark grey with some yellow brown; fine to coarse angular GRAVEL, COBBLES, and BOULDERS; fine to coarse grained SAND; trace to little low plasticity FINES; moist to wet --FILL--*(continued)*						Rig chatter
2332	14													
	15							▽		S03	6, 4, 4			S-03 One liner retained (16-16.5ft.)
2330	16							SANDY SILT (ML); loose; dark grey; fine grained SAND; low plasticity FINES; wet						
	17							POORLY GRADED GRAVEL with SAND (GP); medium dense; dark grey with some yellow brown; fine to coarse angular GRAVEL, COBBLES, and BOUDLERS; fine to coarse grained SAND; trace to little low plasticity FINES; moist to wet						Continued rig chatter
2328	18													
	19													
2326	20									S04	12, 21, 12			S-04 One liner retained (21-21.5ft.)
	21													
2324	22							ALLUVIUM						
	23							BOUDLER, basalt						
2322	24													Switch to rotary wash drilling with 3 7/8-inch tricone bit; yellowish brown clayey cuttings with rounded gravel 24.5-28ft.
	25													
2320	26													
	27													
2318	28							VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; weak; very thinly laminated --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--						Reddish purple clayey and rock cuttings
	29													

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-19

Sheet 3 of 3

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29								VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; weak; very thinly laminated --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--*(continued)*						
2316	30		1		0		m		Becomes weak to moderately strong		S05	50/3		1150	Switch to HQ rock coring with 3 7/8-inch diamond bit; all breaks mechanical
	31	1		100	0	100	m m m m							[8]	
2314	32				0		m m m m							1208 1215	
	33				0		m m m								
2312	34				0										
	35	2		86	0	86	m							[13]	
2310	36				0		m								0.7 ft. of core slipped out of core barrel; left in hole prior to grouting
	37				NA		NR							1238	
									TOTAL DEPTH = 37.5 FEET						
2308	38														
	39														
2306	40														
	41														
2304	42														
	43														
2302	44														
	45														

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 13th, 2020	April 13th, 2020	41.97385602, -122.43609419	BH-CC01

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2332.1'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Neat Cement, 4/14/2020		3.0'	3.0'	20.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–8		SC: SANDY CLAY TO CLAYEY SAND (CH-SC); tan to dark brown; moist; loose; CLAY medium to high plasticity; SAND very fine to fine.			2/1/2	3	30	3/3.4/4.1	34.4	[illegible]				Min. Drill Effort/Torque
2330	2														
	3														
	5					1/1/1	2	30	2/1.8/2.1	19.0	[illegible]				SPT Sample Interference due to sand flowing into auger.
2325	7					2/3/9	12	70	12/9.6/16.1	[illegible]	[illegible]				Sand flowing up into HSA
	8–18.5		SP: POORLY GRADED SAND (SP); dark grey; wet; loose; SAND coarse to very coarse.												
	11														SPT Sample Interference due to sand flowing into auger.
2320	12.5					16/10/10	10	50							Sand flowing up into HSA
	16														Min. Drill Effort/Torque
2315	17														
	18.5–20		SW: WELL GRADED SAND WITH GRAVEL (SW); dark grey; wet; medium dense; SAND fine to very coarse.			17/15/19	34	70	24/19.6/(24.9)	30.5 (Lab 7.5)	6,579				Harder Drilling @ 18.5' Lab Coh. 607psf Max. Drill Effort/Torque
	20														Auger Refusal @ 20.0'

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-CC01
DIST.	COUNTY	ROUTE	POSTMILE	EA
	Siskiyou			
PROJECT OR BRIDGE NAME				
Camp Creek Culvert				
BRIDGE NUMBER	PREPARED BY		DATE	SHEET
NA	KF		7/14/2020	9

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 13th, 2020	April 13th, 2020	41.97381485, -122.43598248	BH-CC02
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			NA	2331'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS: DURING DRILLING 4.0'; AFTER DRILLING (DATE) 4.0'	TOTAL DEPTH OF BORING
Neat Cement, 4/14/2020				22.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
2330	0–4		SC: SANDY CLAY TO CLAYEY SAND (CH-SC); tan to dark brown; moist; loose; CLAY medium to high plasticity; Sand very fine to fine.			2/1/2	3	30	3/2.4/4.1	[illegible]	[illegible]				Min. Drill Effort/Torque
2325	4–7		OL: ORGANIC DEBRIS WITH SAND (OL); greyish brown; wet; loose; ORGANIC DEBRIS < 0.5" dia. plant matter; SAND coarse to very coarse.			1-12"/1	1	90	[illegible]			[illegible]			SPT Sample Interference due to sand flowing into auger.
2320, 2315	7–18		SP: POORLY GRADED SAND (SP): dark grey; wet; loose; SAND coarse to very coarse.			3/6/8	14	90	14/11.2/19.0	[illegible]	[illegible]				Sand flowing up into HSA
															SPT Sample Interference due to sand flowing into auger.
						4/5/5	10	90	10/8.0/11.4	24.8	1,876				Sand flowing up into HSA
															Min. Drill Effort/Torque
															Harder Drilling @ 18.5'
2310	18–22		SW: WELL GRADED SAND WITH TRACE GRAVEL (SW); dark grey; wet; medium dense; SAND fine to very coarse; GRAVEL < 0.75" sub rounded to rounded.			1/2/1	3	50	[illegible]	20.4	349				
						1/1/2	3	30	[illegible]	20.4 (Lab 42.4)	349				Lab Coh=39.0psf
	22–23														Auger Refusal @ 22.0'

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-CC02
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Camp Creek Culvert				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 10

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-01

Sheet 1 of 2

Date(s) Drilled	9/27/2018	Logged By	S. Janowski	Checked By	
Drilling Method	Hollow Stem Auger, HQ-3 Rock Core	Drill Bit Size/Type	6-inch flight auger, HQ-3 wireline diamond bit	Total Depth of Borehole	25.5 feet
Drill Rig Type	Truck Mounted Mobile B-53	Drilling Contractor	Gregg Drilling	NAVD 88 Ground Surface Elevation	2346 feet
Groundwater Level	Not encountered before rotary wash drilling	Sampling Methods	2.5-inch ID ModCal; SPT; HQ Core Barrel	Hammer Data	Automatic hammer; 140 lbs, 30-inch drop
Borehole Backfill	Cement grout to ground surface	Borehole Location	Camp Creek Bridge	Coordinate Location	N 2602866 E 6443027

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2346	0								2-inches GRAVEL roadway						Start 9:00 9/27/2018; hang auger 0.0-5.0ft.
									CLAYEY GRAVEL (GC); very stiff; yellowish brown (10YR 5/6); 60% subangular GRAVEL to 1-inch; 30% low plasticity FINES; 10% fine grained SAND; moist --ROAD FILL--						
	1														
2344	2														
	3														
2342	4														pp = 2.75 tsf
	5								LEAN CLAY with GRAVEL and SAND (CL); very stiff; dark yellowish brown (10YR 4/4); 80% medium plasticity FINES; 10% fine grained SAND; 10% subangular GRAVEL to 1/2-inch; moist --ALLUVIUM--			6			Hollow stem auger 5.0ft. to 9.0ft.
2340	6										S-01	7	0		pp = 2.25 tsf
												8			
	7											6			
											S-02	7	0		
2338	8											8			
									GRAVEL and COBBLES in a SANDY LEAN CLAY matrix; GRAVEL and COBBLES are subrounded Basalt						
	9		1		NA							19		1014	Auger refusal at 9.0ft.; advance 4.5-inch casing to 9.0ft. and switch to rotary wash drilling with 3 7/8-inch tricone bit.
2336	10	1		80		NA					S-03	17	11	[13]	
					NA							10		1021	
	11	2		100		NA								1037 [9] 1044	
					NA		m							1056	
2334	12						NR								75% fluid circulation
		3		60	NA	NA								[25]	
	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-01

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-01

Sheet 2 of 2

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13		1		NA				GRAVEL and COBBLES in a SANDY LEAN CLAY matrix; GRAVEL and COBBLES are subrounded Basalt --ALLUVIUM--*(continued)*						75% fluid circulation
2332	14	4		80	NA	NA								1102 / 1112 [9] 1119	
	15	5		87	NA	NA			VOLCANIC BRECCIA; dark reddish brown (10R 3/4); highly weathered; very weak; highly fractured; friable --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--					1124 [23]	
2330	16				NA		m		Becomes yellowish grey (5Y 7/2), moderately weathered					1128	
	17	6		94	0	88*								1136	
2328	18				>6		m m		Becomes greyish brown (5YR 3/2) Intensely fractured						
	19				0									[16]	*Rock does not meet soundness criteria for RQD calculation
2326	20				1		1 2		1: 15, V, T-VN, H+Uk, Fi, Pl, ? 2: 60, J, N-W, Sd, Fi, Wa, ?						
	21						m m							1155	
2324	22	7		70	NA	0	m m							1244 [15] 1248	
	23	8		0	NA	0								1254 [12] 1259	
2322	24	9		50	NA	0								1304 [12]	
	25				>6									1314	
									TOTAL DEPTH = 25.5 FEET						
2320	26														
	27														
2318	28														
	29														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-01

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil Boring B-02

Sheet 1 of 2

Date(s) Drilled	**10/12/2018**	Logged By	**P. Respess**	Checked By	
Drilling Method	**Hollow Stem Auger**	Drill Bit Size/Type	**6-inch flight auger**	Total Depth of Borehole	**31.4 feet**
Drill Rig Type	**Truck Mounted Mobile B-53**	Drilling Contractor	**Gregg Drilling**	NAVD 88 Ground Surface Elevation	**2341 feet**
Groundwater Level(s)	**13.5 feet below ground surface 10/12/2018**	Sampling Method(s)	**SPT**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**Camp Creek Bridge**	Coordinate Location	**N 2602747 E 6443180**

Elevation feet	Depth, feet	SAMPLES Type	SAMPLES Number	SAMPLES Sampling Resistance	SAMPLES Recovery (feet)	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Fines Content (%<#200 Sieve)	REMARKS AND OTHER TESTS
	0						2-inches GRAVEL roadway				Start 9:00 9/27/2018; hollow stem auger 0-31ft.
2340							POORLY GRADED GRAVEL (GP); dense; fine to coarse GRAVEL and COBBLES; fine to corase grained SAND; little no plasticity FINES; moist --FILL--				
							LEAN CLAY (CL); medium stiff; brown; medium plasticity FINES; trace fine grained SAND; occasional GRAVEL and COBBLE				
	5										Logged from auger cuttings and rig chatter
2335											
	10										
2330							∇				
	15										
2325											
	20						POORLY GRADED GRAVEL with SAND (GP); medium dense to dense; fine to coarse GRAVEL to BOULDERS; fine ot coarse grained SAND; some no plasticity FINES --ALLUVIUM--				Rig chatter indicated rocky layer
2320							BOULDER, basalt				
	25										
2315							BOULDER, basalt				
			S-01	14 14 44	100						
	30										

Report: GEO_10B1_OAK; File: ROCK CORES.GPJ; 11/14/2018 B-02

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil Boring B-02

Elevation feet	Depth, feet	SAMPLES Type	Number	Sampling Resistance	Recovery (feet)	Graphic Log	MATERIAL DESCRIPTION	Water Content, %	Plasticity Index	Fines Content (%<#200 Sieve)	REMARKS AND OTHER TESTS
	30						[As Above] --ALLUVIUM--*(continued)*				
2310			S-02	50/0	0		BASALT; dark grey; slightly weathered to fresh; moderately strong --TERTIARY to QUATERNARY INTRUSIVE BASALT--				S-02 attempted at 31.4; logged from flake in shoe
							TOTAL DEPTH = 31.4 FEET				
	35										
2305											
	40										
2300											
	45										
2295											
	50										
2290											
	55										
2285											
	60										
2280											
	65										

Report: GEO_10B1_OAK; File: ROCK CORES.GPJ; 11/14/2018 B-02

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-03

Sheet 1 of 2

Date(s) Drilled	**10/12/2018-10/16/2018**	Logged By	**P. Respess**	Checked By	
Drilling Method	**Hollow Stem Auger, Rotary Wash, HQ-3 Rock Core**	Drill Bit Size/Type	**6-inch flight auger, 3 7/8-inch tricone, 3 7/8-inch diamond core bit**	Total Depth of Borehole	**27.3 feet**
Drill Rig Type	**Truck Mounted Mobile B-53**	Drilling Contractor	**Gregg Drilling**	NAVD 88 Ground Surface Elevation	**2341 feet**
Groundwater Level	**Not encountered before rotary wash drilling**	Sampling Methods	**2.5-inch ID ModCal, HQ Core Barrel**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**Camp Creek**	Coordinate Location	**N 2602664 E 6443265**

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	0								POORLY GRADED GRAVEL with SAND (GP); medium dense to dense; fine to coarse GRAVEL to BOULDERS; fine to corase grained SAND; some no plasticity FINES; dry to moist --FILL--						Start 12:00 10/12/2018; hang auger 0.0-5.0ft.
2340	1														
	2														
2338	3								BOULDER and COBBLES; 3.0-4.8ft.: BOULDER --ALLUVIUM--						
	4														End of day 10/12/2018 Begin day 10/15/2018
2336	5														Switch to rotary wash drilling with 3 7/8-inch tricone bit
	6								BOULDER						
2334	7														Advance 4.5-inch casing to 5ft.
	8								BOULDERS and COBBLES						
2332	9														
	10														
2330	11								BOULDERS and COBBLES						
	12														
2328	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-03

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-03

Sheet 2 of 2

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing	Number	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2328	13								BOULDER and COBBLES --ALLUVIUM--*(continued)*						Reddish clay cuttings
	14								VOLCANIC SILTSTONE; reddish brown to olive grey; moderately to highly weathered; very weak to weak; very thinnly laminated; locally clayey --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--						
2326	15										S-01	14 19 23			S-01 One liner retained (16-16.5ft.)
	16														
2324	17														
	18														
2322	19														
	20										S-02	27 31 46			Rig chatter at 20ft. indicates rocky layer
2320	21														S-02 One liner retained (21-21.5ft.)
	22		1		NA		NR	m	becomes moderately weathered; weak					0810	End of day 10/15/2018 Begin day 10/16/2018
2318	23				1			m 1							Advance 4.5-inch casing to 22ft. Switch to HQ-3 rock coring at 22.3ft.
	24				1			m m m m 2	1: J, 30, N, Cl, Fi, Pl, S-SR (dissolution voids along joint) 2: J, 10-15, VN, Cl, Fi, Pl-Wa, S-SR						
2316	25	1		82	0	78*	NR	m						[7]	
	26				0										26.3-27.3ft. driller reports harder drilling condition
2314	27				NA		NR	m m						0852	
									TOTAL DEPTH = 27.3 FEET						
	28														
2312	29														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-03

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-19

Sheet 2 of 3

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13								POORLY GRADED GRAVEL with SAND (GP); medium dense; dark grey with some yellow brown; fine to coarse angular GRAVEL, COBBLES, and BOUDLERS; fine to coarse grained SAND; trace to little low plasticity FINES; moist to wet --FILL--*(continued)*						Rig chatter
2332	14														
	15								▽		S03	6, 4, 4			S-03 One liner retained (16-16.5ft.)
2330	16								SANDY SILT (ML); loose; dark grey; fine grained SAND; low plasticity FINES; wet						
	17								POORLY GRADED GRAVEL with SAND (GP); medium dense; dark grey with some yellow brown; fine to coarse angular GRAVEL, COBBLES, and BOUDLERS; fine to coarse grained SAND; trace to little low plasticity FINES; moist to wet						Continued rig chatter
2328	18														
	19														
2326	20										S04	12, 21, 12			S-04 One liner retained (21-21.5ft.)
	21														
2324	22								ALLUVIUM						
	23								BOUDLER, basalt						
2322	24														Switch to rotary wash drilling with 3 7/8-inch tricone bit; yellowish brown clayey cuttings with rounded gravel 24.5-28ft.
	25														
2320	26														
	27														
2318	28								VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; weak; very thinly laminated --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--						Reddish purple clayey and rock cuttings
	29														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-19

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-19

Sheet 3 of 3

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29								VOLCANIC SILTY CLAYSTONE/SILTSTONE; reddish purple; slightly weathered; weak; very thinly laminated --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--*(continued)*						
2316	30		1		0		m		Becomes weak to moderately strong		S05	50/3		1150	Switch to HQ rock coring with 3 7/8-inch diamond bit; all breaks mechanical
	31	1		100	0	100	m m m m							[8]	
2314	32				0		m m m m							1208 / 1215	
	33				0		m m m								
2312	34				0										
	35	2		86	0	86	m							[13]	
2310	36				0		m								0.7 ft. of core slipped out of core barrel; left in hole prior to grouting
	37				NA		NR							1238	
									TOTAL DEPTH = 37.5 FEET						
2308	38														
	39														
2306	40														
	41														
2304	42														
	43														
2302	44														
	45														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-19

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-20

Sheet 1 of 4

Date(s) Drilled	**10/10/2018**	Logged By	**P. Respess**	Checked By	
Drilling Method	**Hollow Stem Auger, Rotary Wash, HQ-3 Rock Core**	Drill Bit Size/Type	**3 7/8-inch tricone; 3 7/8-inch #6 HQ bit**	Total Depth of Borehole	**47.0 feet**
Drill Rig Type	**Truck Mounted Mobile B-53**	Drilling Contractor	**Gregg Drilling**	NAVD 88 Ground Surface Elevation	**2341 feet**
Groundwater Level	**14.5 feet below ground surface 10/10/2018**	Sampling Methods	**2.5-inch ID ModCal, SPT, HQ Core Barrel**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**Camp Creek**	Coordinate Location	**N 2602768 E 6443160**

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	0								2.5-inches Aggregate base						Start 9:00 10/10/2018; hollow stem auger 0-28ft.
									POORLY GRADED GRAVEL (GP); dense; fine to coarse GRAVEL and COBBLES, little no plasticity FINES; moist --FILL--						
2340	1														
	2														
2338	3								LEAN CLAY (CL); medium stiff; brown; medium plasticity FINES; trace fine grained SAND; occasional GRAVEL and COBBLES; moist						
	4														
2336	5										S01	3, 5, 6			S-01 One liner retained (5.5-6ft.)
	6														
2334	7														
	8														
2332	9														
	10										S02	4, 4, 6			S-02 One liner retained (10.5-11ft.)
2330	11														
	12														
2328	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-20

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-20

Sheet 2 of 4

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2328	13								LEAN CLAY (CL); medium stiff; brown; medium plasticity FINES; trace fine grained SAND; occasional GRAVEL and COBBLES; moist --FILL--*(continued)*						
	14								SANDY CLAY to CLAYEY SAND (CL-SC); medium stiff; olive brown; ~ 50% medium plasticity FINES; ~50% fine to coarse grained SAND and fine GRAVEL ∇						
2326	15										S03	7 8 7			S-03 One liner retained (16-16.5ft.)
	16														
2324	17								POORLY GRADED GRAVEL with SAND (GP); medium dense to dense; fine to coarse grained SAND; fine to coarse GRAVEL with COBBLES and BOULDERS, wet --ALLUVIUM--						
	18														
2322	19														
	20										S04	4 5 6			S-04 One liner retained (21-21.5ft.)
2320	21														
	22														
2318	23														
	24														
2316	25										S05	22 24 18			S-05 One liner retained (26-26.5ft.)
	26														
2314	27														
	28								BOULDER: 28-29.5 ft.						Switch to rotary wash drilling with 3 7/8-inch tricone bit at 28ft.
2312	29														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-20

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-20

Sheet 3 of 4

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing	Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2312	29									BOULDER: 28-29.5 ft. --ALLUVIUM--*(continued)*						
	29.5									POORLY GRADED GRAVEL with SAND (GP); medium dense to dense; fine to coarse grained SAND; fine to coarse GRAVEL with COBBLES and BOULDERS						Skip sample; rig behavior indicates gravel and cobbles
2310	31									BASALT; dark grey; slightly weathered; moderately strong; with Fe staining around joints; chlorite and quartz infilling; numerous healed fractures --TERTIARY to QUATERNARY INTRUSIVE BASALT--						
2306	35															Skip sample; rig behavior indicates gravel and cobbles
	40		1		3			1 2 3 m 4				S06	50/1.5		1305	Switch to HQ rock coring with 3 7/8-inch diamond bit
2300	41	1		100	2	30		5							[7]	
	42				2			1 6		1: 60, J, N, Fe+Ch, Pa, Wa-Pl, SR 2: 70-90, J, VN, Fe, Pa, Wa, SR 3: 70, J/V, Vn, Qz, Pa, Wa, SR 4: 60, V, VN, Qz, Pa-Sp, Wa-Pl, SR 5: 40, J/V, N, Qz+Ch, Fi, Wa, ? 6: 40, J, VN, Ch, Pa-Su, Pl-Wa, SR					1328 1338	
2298	43				0											
	44	2		100	1	79		1		1: 40, J, VN, Ch, Fi, Pl, ?					[12]	
2296	45															

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-20

Sheet 4 of 4

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2296	45		1		1		2		BASALT; dark grey; slightly weathered; moderately strong; with Fe staining around joints; chlorite and quartz infilling; numerous healed fractures --TERTIARY to QUATERNARY INTRUSIVE BASALT--*(continued)*						
	46				2		3, 4		2: 60, J/V, W (20mm), Ch, Fi, Wa, ? 3: 60, J, N, Ch, Sp, SR, ? 4: 70, J, VN, Ch, Sp, SR						
2294	47								**TOTAL DEPTH = 47.0 FEET**					1400	
	48														
2292	49														
	50														
2290	51														
	52														
2288	53														
	54														
2286	55														
	56														
2284	57														
	58														
2282	59														
	60														
2280	61														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: ROCK CORES.GPJ; 11/14/2018 B-20

LOGGED BY: JF & JS — BEGIN DATE: April 16th, 2020 — COMPLETION DATE: April 16th, 2020	BOREHOLE LOCATION (Lat/Long or North/East and Datum): 41.97318588, -122.36643994		HOLE ID: **BH-DG03**
DRILLING CONTRACTOR: GeoServ, Inc.	BOREHOLE LOCATION (Offset, Station, Line): NA		SURFACE ELEVATION: 2345'
DRILLING METHOD: Hollow Stem Auger	DRILLING RIG: Deere 35G		BOREHOLE DIAMETER: 6"
SAMPLER TYPE(S) AND SIZES (ID): SPT 1.5"	SPT HAMMER TYPE: Safety Hammer		HAMMER EFFICIENCY, ERi
BOREHOLE BACKFILL AND COMPLETION: Bentonite Chip, 4/18/2020	GROUNDWATER READINGS — DURING DRILLING: ND	AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING: 9.3'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
2345	0		Fill: ARTIFICIAL FILL-CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 8" dia., subangular.												
2340	5														
2335	10														
	11		CH: SANDY CLAY (CH); dark brown; moist; very soft to firm; CLAY medium to high plasticity; SAND coarse to very coarse; trace organic debris.			3/4/5	9	30	9/7.2/ 9.1			2,314			PI 46
	13		Weathered Rx: WEATHERED ROCK; highly weathered to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.												Harder drilling @ 13.0' bgs Max Drill Effort/Torque @ 14.0' bgs
2330	15					50:5"		50	50/40/ 46.6	49	17,657				Auger Refusal @ 15.3' bgs below existing top of road
	16														
	17														

GEOSERV	REPORT TITLE: Geotechnical Investigation, Daggett Road				HOLE ID: BH-DG03
	DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
	PROJECT OR BRIDGE NAME: Fall Creek Culvert at Daggett Road				
	BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 11

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & KF	April 17th, 2020	April 17th, 2020	41.97329581, -122.36694418	BH-DG04

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2352.2'

DRILLING METHOD	DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger	Lonestar Drill	6"

SAMPLER TYPE(S) AND SIZES (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/18/2020		ND	ND	6.5'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL-CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 8" dia., subangular.												
2350	2														
	3		CH: SANDY SILTY CLAY WITH GRAVEL (CH); dark brown; dry to moist; very soft to firm; CLAY medium to high plasticity; SAND coarse to very coarse; GRAVEL less than 1.5in diameter angular to subangular.												
	4					7/6/6	12	20	12/9.6/16.5			2,945			
	5		Clay: CLAY (CH); dark brown, moist; firm to very stiff; clay medium to high plasticity; harder drilling with depth.			5/8/13	21	90	21/16.6/26.8			5,150			Hard Drilling Max Drill Effort/Torque @ 5.0' bgs Auger Refusal @ 6.5' bgs.
	6		Weathered Rx: WEATHERED ROCK; highly weathered to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.												
2345	7														
	8														
	9														
	10														
	11														
2340	12														
	13														
	14														
	15														
	16														
	17														

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Daggett Road				BH-DG04
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Fall Creek Culvert at Daggett Road				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 12

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 16th, 2020	April 16th, 2020	41.97665, -122.36415	BH-DG01
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			NA	2387.0'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020				10.9'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–1.5		Fill: ARTIFICIAL FILL - SANDY GRAVEL: (GW): dark brown; dry to moist; medium dense; SAND very fine to coarse; GRAVEL< 1.5" dia., angular to subangular.												
2385	1.5–9.5		CH: CLAY with SANDY GRAVEL: (CH); dark brown; moist; firm; CLAY medium to high plasticity; sand very fine; GRAVEL < 0.375" dia, angular to subangular, GRAVEL occurs below 5.5' bgs.												
	3.5–5					3/5/9	14	80	14/11.3/19			3,446			Bulk sample taken @ 1.5'-5.5' bgs
2380	7														
	8.5–10					5/10/23	33	80	33/26.4/25.3	44	9,003				Harder drilling @ 9.5' bgs
	9.5–10.9		Weathered Rx: WEATHERED ROCK; high weathering to nearly fresh fragments of andesite/basalt; level of weathering decreases with depth.												Auger Refusal @ 10.9' bgs
2375	12														

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-DG01
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Fall Creek culvert at Substation				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 13

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & JS	April 18th, 2020	April 18th, 2020	41.97676, -122.36455	BH-DG02

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2386.0'

DRILLING METHOD	DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger	Lonestar Drill	6"

SAMPLER TYPE(S) AND SIZES (ID)	SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020		ND	ND	9.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL - GRAVELLY CLAY with SAND (CH): dark reddish brown; moist; stiff to very stiff; CLAY medium to high plasticity; SAND coarse; GRAVEL< 1.0" dia., angular to subrounded.												
2385	1														
	2														
	3														
	4					7/14/20	34	30	34/27.2 /48.5			[illegible]			
	5														
2380	6														
	7		CH: GRAVELLY CLAY with SAND: (CH); dark reddish brown; moist; stiff to very stiff; CLAY medium to high plasticity; SAND coarse; GRAVEL < 0.5" dia, subrounded to round.												Easier drilling @ 7.0' bgs
	8					7/12/14	26	50	26/20.8 /31.2			[illegible]			
	9														Auger Refusal @ 9.0' bgs
	10														
2375	11														
	12														
	13														

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation				BH-DG02
DIST.	COUNTY	ROUTE	POSTMILE	EA
	Siskiyou			
PROJECT OR BRIDGE NAME				
Fall Creek culvert at Substation				
BRIDGE NUMBER	PREPARED BY		DATE	SHEET
NA	KF		7/14/2020	14

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & KF	April 16th, 2020	April 16th, 2020	41.98390289, -122.36220577	BH-FL01
DRILLING CONTRACTOR			BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.			NA	2494.87'
DRILLING METHOD			DRILLING RIG	BOREHOLE DIAMETER
Hollow Stem Auger			Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)			SPT HAMMER TYPE	HAMMER EFFICIENCY, ERi
SPT 1.5"			Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION			GROUNDWATER READINGS — DURING DRILLING: ND; AFTER DRILLING (DATE): ND	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020				2.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL-CLAYEY SAND AND GRAVEL WITH BOULDERS AND COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 12" dia., subangular.												Boulders and RSP
	1														
	2														Refusal due to boulders @ 2.0'
	3														
	4														
2490	5														
	6														
	7														

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-FL01
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Copco Road at Fall Creek Bridge				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 15

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & KF	April 16th, 2020	April 16th, 2020	41.98390289, -122.36220577	BH-FL02

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2494.88'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hollow Stem Auger	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020		ND	ND	6.1'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL - CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS , 8" dia., subangular.												Boulders and RSP
	1		CH: SILTY CLAY with GRAVEL and TRACE COBBLE (CH); reddish brown; moist; moderately stiff; CLAY medium to high plasticity; GRAVEL < 1" dia., subrounded; COBBLE < 4" dia, subangular.												
	2														
	3														Hand cleared to 3' bgs
	4					2/3/4	7	10	7/5.0/9.5			1,713			
2490	5					9/8/6	14	30	14/11.2/19.0			3,425			
	6														Refusal @ 6.1' bgs
	7														

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-FL02
DIST.	COUNTY	ROUTE	POSTMILE	EA
	Siskiyou			
PROJECT OR BRIDGE NAME				
Copco Road at Fall Creek Bridge				
BRIDGE NUMBER	PREPARED BY	DATE	SHEET	
NA	KF	7/14/2020	16	

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & KF	April 16th, 2020	April 16th, 2020	41.98406971, -122.36207834	BH-FL03

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2493.27'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hand Cleared	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	
BOREHOLE BACKFILL AND COMPLETION	**GROUNDWATER READINGS** — DURING DRILLING: ND; AFTER DRILLING (DATE): ND	**TOTAL DEPTH OF BORING**
Bentonite Chip, 4/19/2020		3.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0		Fill: ARTIFICIAL FILL - CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 12" dia., subangular.												Boulders and RSP
	1														
	2														Hand cleared to 3.0' bgs
	3														Refusal due to boulders @ 3.0'
2490															
	4														
	5														
	6														
	7														

GEOSERV

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-FL03
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Copco Road at Fall Creek Bridge				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 17

LOGGED BY	BEGIN DATE	COMPLETION DATE	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID
JF & KF	April 16th, 2020	April 16th, 2020	41.98399944, -122.36201060	BH-FL04

DRILLING CONTRACTOR	BOREHOLE LOCATION (Offset, Station, Line)	SURFACE ELEVATION
GeoServ, Inc.	NA	2494.17'
DRILLING METHOD	**DRILLING RIG**	**BOREHOLE DIAMETER**
Hand Cleared	Lonestar Drill	6"
SAMPLER TYPE(S) AND SIZES (ID)	**SPT HAMMER TYPE**	**HAMMER EFFICIENCY, ERi**
SPT 1.5"	Safety Hammer	

BOREHOLE BACKFILL AND COMPLETION	GROUNDWATER READINGS	DURING DRILLING	AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING
Bentonite Chip, 4/19/2020		ND	ND	2.0'

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	N/ N60/ N1,60	Friction Angle	Calc. Bearing Capacity (psf)	Undrained Shear Strength (psf)	Drilling Method	Casing Depth	Remarks
	0–2		Fill: ARTIFICIAL FILL - CLAYEY SAND and GRAVEL with BOULDERS and COBBLES (FILL); light to dark brown; dry to moist; loose to medium dense; CLAY medium to high plasticity; SAND fine to very coarse; GRAVEL < 1.5" dia., angular to subangular; COBBLES and BOULDERS, 12" dia., subangular.												Boulders and RSP Hand cleared to 2.0' bgs Refusal due to boulders @ 2.0'
2490	4														

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7

REPORT TITLE				HOLE ID
Geotechnical Investigation, Copco Road				BH-FL04
DIST.	COUNTY: Siskiyou	ROUTE	POSTMILE	EA
PROJECT OR BRIDGE NAME: Copco Road at Fall Creek Bridge				
BRIDGE NUMBER: NA	PREPARED BY: KF		DATE: 7/14/2020	SHEET: 18

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Sheet 1 of 4

Date(s) Drilled	1/22/2019-1/23/209	Logged By	S. Janowski	Checked By	P. Respess
Drilling Method	Solid Stem Auger, HQ-3 Rock Core	Drill Bit Size/Type	4-inch solid stem auger, 4-inch diamond coring bit	Total Depth of Borehole	51.5 feet
Drill Rig Type	Truck Mounted CME 75	Drilling Contractor	Taber Drilling	NAVD 88 Ground Surface Elevation	2344 feet
Groundwater Level	11.7' 1/23/2019	Sampling Methods	2.5-inch ID ModCal, SPT, HQ Core Barrel	Hammer Data	Automatic hammer; 140 lbs, 30-inch drop
Borehole Backfill	Cement grout to ground surface	Borehole Location	North end of Daggett Road Bridge	Coordinate Location	N 2602349 E 6462482

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery,%	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2344	0								SANDY LEAN CLAY with GRAVEL (CL); very stiff; moist; dark brown (10yr3/3); 20% subrounded to rounded GRAVEL to 3/4"; 20% fine- to medium-grained SAND; 60% medium plasticity FINES --FILL--						
	1														
2342	2														
	3														
2340	4														
	5										1-1 1-2	6 6 8	78		pp=3.0 tsf
2338	6														
	7														
2336	8								CLAYEY GRAVEL with SAND (SC); very dense; moist; yellowish brown to dark brown; interbedded layers of gravel with clay and sand --ALLUVIUM--						Fill estimate based on height of slope embankment
	9														
2334	10										2	100/1"	100		
	11								▼						
2332	12														
	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-15

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Sheet 2 of 4

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13								CLAYEY GRAVEL with SAND (SC); very dense; moist; yellowish brown to dark brown; interbedded layers of gravel with clay and sand --ALLUVIUM-- *(continued)*						
2330	14														
	15											16			SA: G=42%; S=27%; F=31%
2328	16										3	20	78		
												34			
	17								BASALT BOULDERS and COBBLES in SAND & GRAVEL matrix; medium dark gray (N4) to dark gray (N3), strong, some boulders are scoriaceous, matrix washed out --ALLUVIUM--						Rig chatter
2326	18														
	19														
2324	20											17			
											4	50	61		
	21	1	1	100	NA	0						50		0921 [6] 0926	End of day 1/22/2019
														0933	Begin day 1/23/2019; AM water level=11.7' bgs Switch to HQ rock core
2322	22				NA										
	23				NA		NR								
2320	24	2		2	NA	0								[100]	
	25				NA										
2318	26				NA									0936	
											5	65	100		
	27				NA									0956	
2316	28	3		71	NA	27*	NR								*Rock does not meet soundness criteria for RQD calculation
									VOLCANICLASTIC BRECCIA --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--						
	29														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-15

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Sheet 3 of 4

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29	3	1	71	5	27*	1,1; 1; 1	△ △	VOLCANICLASTIC BRECCIA; light olive gray (5Y5/2); moderately weathered; weak; highly to intensely fractured; angular clasts to 1/2" --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--					[68]	
2314	30				6		1; 1	△ △	1: 20°, J, MW, Sd, Sp, Wa, R Becomes grayish blue-green (5BG5/2); slightly weathered; moderately strong						*Rock does not meet soundness criteria for RQD calculation
	31				0		M	△ △						1000 / 1008	
2312	32				3		1; 1; M; 1; M	△ △	1: 30°, J, MW, Sd+Fe, Sp+Su, Wa, R						
	33				1		M; 1	△ △							
2310	34	4		100	0	82		△ △	Becomes slightly fractured					[60]	
	35				0			△ △							
2308	36				0		M	△ △						1013 / 1017	
	37				0		M; M	△ △							
2306	38				0		M; M	△ △	Becomes light olive gray (5Y5/2); moderately weathered; weak; highly fractured						
	39	5		100	1	96*	1	△ △	1: 15°, J, MW, Fe, Su, Wa, VR					[75]	*Rock does not meet soundness criteria for RQD calculation
2304	40		2		1		M; M; 2	△ △	2: 60°, J/Sh, MW, Fe+Mn+Sd, Su+Sp, Wa, R						
	41				2		1	△ △	Becomes grayish blue-green (5BG5/2); slightly weathered 1: 20°, J, MW, No, No, Wa-St, VR					1021 / 1024	
2302	42				1		1; M	△ △	Becomes moderately fractured						
	43				0		M	△ △							
2300	44	6		100	0	72	M	△ △	Becomes weak to very weak					[43]	
	45														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-15

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Sheet 4 of 4

Elevation, feet	Depth, feet	Run No. (Rock Core)	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type (Soil Samples)	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	45		2		4		1, 1	△ △	VOLCANICLASTIC BRECCIA; grayish blue-green (5BG5/2); slightly weathered; weak to very weak; highly fractured; angular clasts to mostly to 1/2", occasionally to 1.5" --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*						
		6		100		72		△ △							
-2298	46						M, M	△ △							
					0		M	△ △						1031 / 1035	UCS = 1546 psi
	47						M	△ △							
					0			△ △							
-2296	48						1	△ △							
					1			△ △	1: 20°, J, MW, No, No, Wa, R 2: 15°, J, N-VN, Sd+Si, So-Pa, Pl, R-SR						
	49	7		100		94	2,2	△ △						[43]	
					2			△ △							
-2294	50						2, 2	△ △							
					2			△ △							
	51						M	△ △							
					0			△ △						1042	
	51.5								**TOTAL DEPTH = 51.5 FEET** **Grout mix: 30 gallons of water, six 47# bags of cement, no bentonite**						
-2292	52														
	53														
-2290	54														
	55														
-2288	56														
	57														
-2286	58														
	59														
-2284	60														
	61														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-15

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-16

Sheet 1 of 2

Date(s) Drilled	1/12/2019	Logged By	P. Respess	Checked By	S. Janowski
Drilling Method	Rotary Wash, HQ-3 Rock Core	Drill Bit Size/Type	3-7/8-inch tricone, 3 3/4-inch diamond coring bit	Total Depth of Borehole	24.5 feet
Drill Rig Type	Barge Mounted CME-45	Drilling Contractor	Taber Drilling	NAVD 88 Ground Surface Elevation	2319 feet
Groundwater Level	12 feet above ground surface	Sampling Methods	SPT, HQ Core Barrel	Hammer Data	Automatic hammer; 140 lbs, 30-inch drop
Borehole Backfill	Bentonite cement grout to ground surface	Borehole Location	12' downstream of Daggett Road bridge	Coordinate Location	N 2602237 E 6462573

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Soil Samples: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	0								VOLCANICLASTIC BRECCIA; gray-green; completely weathered; extremely weak; fine-grained matrix; dark gray-black angular clasts up to 1/4"-1/2"; slightly fractured with widely-spaced natural fractures; numerous mechanical breaks --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--	SPT	1	3			12' of water in river at time of drilling
	1											5			
2318												15			5" HWT casing driven to 14' (refusal) Tricone to 15' and continue with HQ core High Water Circulation Return (WCR)
	2														
	3						M		Becomes moderately to slightly weathered; moderately strong; slightly fractured; multi-colored clasts up to 2"					1024	
2316			1		0		M, M, M		Broken mechanical						
		1		100		100								[90]	
	4						M								
	4.5				0		M, M							1025 / 1029	
2314	5														
					0										
	6						M								
					0										
2312	7	2		100		100	1		1: 20°, J, N, No, No, Wa, SR					[150]	
					1		M								
	8														
					0										
2310	9														
	9.5				0		M							1031 / 1034	
	10														
					0		M								
2308	11														
					0										
	12	3		100		100	M							[100]	
					0		M								
	13														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-16

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-16

Sheet 2 of 2

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Number	MATERIAL DESCRIPTION	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2306	13	3	1	100	0	100	M	VOLCANICLASTIC BRECCIA; gray-green; moderately to slightly weathered; moderately strong; slightly fractured; multi-colored clasts up to 2"; numerous mechanical breaks. Becomes clasts up to 3-4" at 13.8' --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*	1037	High WCR
2304	14.5–19.5	4	2	100	0, 0, 1, 0, 0	100	M, M, M, 1, M, M, M, M, M	1: 30°, J, N, No, No, Pl-Wa, SR	1040; [150]; 1042	
2298	19.5–24.5	5		96	0, 0, 0, 0, 0, NA	96	M, M, M, 1 (NR)		1045; [75]; 1049	
	24.5							**TOTAL DEPTH = 24.5 FEET** **15 gallons of grout: 6 sack mix with 5% bentonite**		

Elevation scale: 2306, 2304, 2302, 2300, 2298, 2296, 2294, 2292. Depth scale: 13–29 feet.

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-16

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-17

Date(s) Drilled	1/22/2019	Logged By	S. Janowski	Checked By	P. Respess
Drilling Method	Solid Stem Auger, HQ-3 Rock Core	Drill Bit Size/Type	4-inch solid stem auger, 4-inch diamond coring bit	Total Depth of Borehole	41.5 feet
Drill Rig Type	Truck Mounted CME 75	Drilling Contractor	Taber Drilling	NAVD 88 Ground Surface Elevation	2341 feet
Groundwater Level	Not encountered before HQ rock coring	Sampling Methods	2.5-inch ID ModCal, SPT, HQ Core Barrel	Hammer Data	Automatic hammer; 140 lbs, 30-inch drop
Borehole Backfill	Cement grout to ground surface	Borehole Location	South end of Daggett Road Bridge	Coordinate Location	N 2602195 E 6462721

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	0								GRAVELLY CLAY with SAND (CL); stiff; moist; dark brown (7.5YR3/3); subangular to subrounded GRAVEL to 1/2"; medium-grained SAND; medium plasticity FINES --FILL--						
2340	1														
	2														
2338	3														
	4														
2336	5										1-1 1-2	5 50/5"	100		pp=1.0 tsf Driller adds water to facilitate advancement
2334	6														
	7														
2332	8								SANDY GRAVEL (GP); very dense; moist; brown; subangular to subrounded GRAVEL to 2.25"; medium- to coarse-grained SAND --ALLUVIUM--						
	9														
2330	10										2-1 2-2	42 50/4"	100		
	11														
	12														
2328	13								VOLCANICLASTIC BRECCIA --TERTIARY VOLCANICS--						Driller felt change during advancement

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-17

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-17

Sheet 2 of 3

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	MATERIAL DESCRIPTION	Soil Samples Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13							VOLCANICLASTIC BRECCIA; greenish-gray (5G6/1); slightly weathered; moderately strong; slightly fractured; angular clasts to 1/2" in fine matrix --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*						
	14													
2326														
	15		1		0		M			3	50/4"	100	1110	Switch to HQ core
	16	1		100		100							[45]	
2324					0		M						1112 / 1147	
	17				0									
	18													UCS = 2130 psi
2322					0		M, M							
	19	2		100		100	M						[75]	
					0		M							
	20													
2320					0									
	21				0								1151 / 1216	
	22													
2318					0									
	23						1	Very weak; 1: 20°, J, VN, Cl+Sd, Sp, Pl, S-SR						
					1									
	24	3		100		84	1, 1	Weak					[100]	
2316					2									
	25						M							
					0									
	26						M, M							
2314					0								1219 / 1223	
	27				0									
	28	4		100		100	M						[75]	
2312					0									
	29													

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-17

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-17

Sheet 3 of 3

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	MATERIAL DESCRIPTION	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29		2		0			VOLCANICLASTIC BRECCIA; greenish-gray (5G6/1); slightly weathered; moderately strong; slightly fractured; angular clasts to 1/2" in fine matrix --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*		
2310	30	4		100	0	100			[75]	
	31				0		M		1227 / 1231	
2308	32				0			Coarser clasts to 2"		
	33				0					
2306	34	5		100	1	86			[100]	
	35				3		1	Light brownish gray (5YR6/1); moderately weathered; weak; highly fractured 1: 60°, J/Sh, W, Ca+Mn+Fe, Pa+Su IR, VR-Slk (rough grooves)		
2304	36				0		M	Abundant mechanical fractures	1234 / 1239	
	37				0		M, M			
2302	38				0		M			
	39	6		100	0	100			[75]	
2300	40				0		M	1.5" Clast		UCS = 2985 psi
	41				0				1243	
2298	42							**TOTAL DEPTH = 41.5 FEET** **Grout mix: 20 gallons of water, five 47# bags of cement, no bentonite**		
	43									
2296	44									
	45									

Columns also printed: Lithology; SOIL SAMPLES: Type, Number, Blows / 6 in., Recovery, %

#
Copco Road at Dry Creek Bridge Photo 1 – BH-DR02 Sample 2.2 from 8-9.5 ft bgs.

Copco Road at Dry Creek Bridge Photo 2 – BH-DR03 Sample 3.2 from 8-9.5 ft bgs.

Copco Road at Dry Creek Bridge Photo 3 – BH-DR04 Sample 4.2 from 9.5-11 ft bgs.

Copco Road at Dry Creek Bridge Photo 4 – BH-DR02 (far cone) location viewed from BH-DR03 looking to the northwest, Copco Road at Dry Creek Bridge in background.

#
Lakeview Road Bridge Photo 1 – BH-AB01 Sample 1.1 from 7-9.5 ft bgs.

Lakeview Road Bridge Photo 2 – BH-AB01 Sample 1.2 from 10-11.5 ft bgs.

#

Lakeview Road Bridge Photo 3 – BH-AB01 Sample 1.3 from 15-16.5 ft bgs.

Lakeview Road Bridge Photo 4 – BH-AB01 Sample 1.4 from 20-21.5 ft bgs.

Lakeview Road Bridge Photo 5 – BH-AB01 Sample 1.5 from 25-25.25 ft bgs.

Lakeview Road Bridge Photo 6 – BH-AB02 Sample 2.3 from 15-16.5 ft bgs.

Lakeview Road Bridge Photo 7 – BH-AB2 Sample 2.4 from 20-21.5 ft bgs.

Lakeview Road Bridge Photo 8 – BH-AB01 location looking south.

Lakeview Road Bridge Photo 9 – BH-AB02 Location looking southwest.

#

Scotch Creek Culvert Photo 1 – BH-SC01 Sample 1.1 from 0-1.5 ft bgs.

#

Scotch Creek Culvert Photo 2 – BH-SC02 Sample 2.1 from 0-1.5 ft bgs.#

Scotch Creek Culvert Photo 3 – BH-SC02 Sample 2.2 from 3.5-5 ft bgs.

#

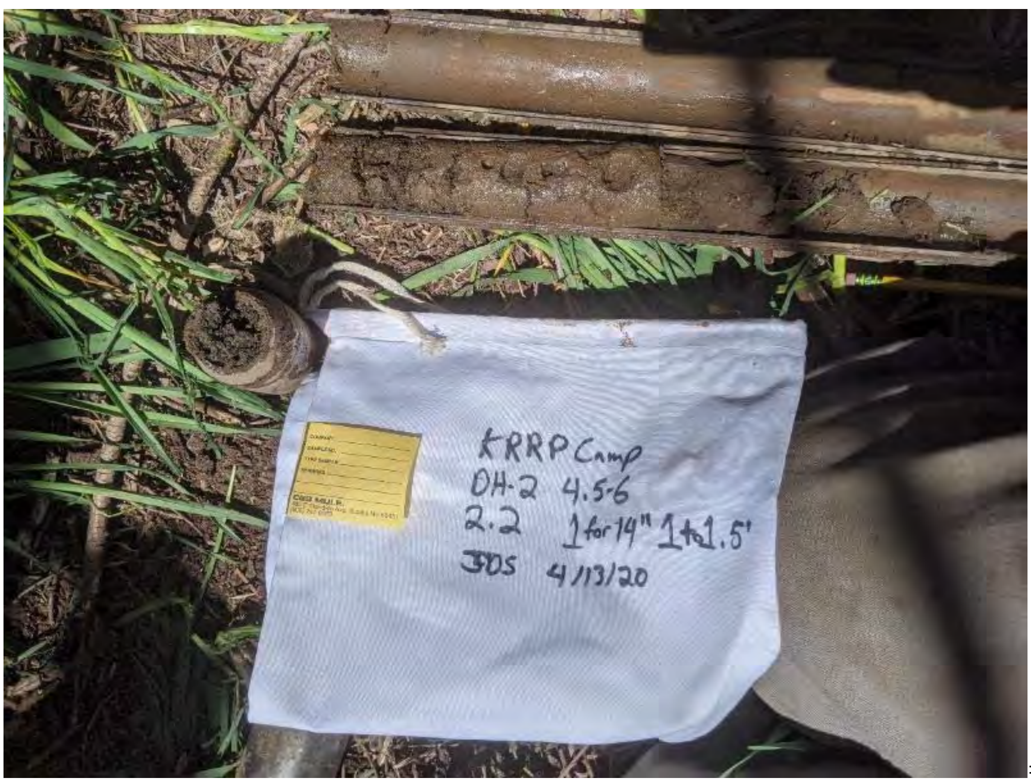
Scotch Creek Culvert Photo 4 – BH-SC02 Sample 2.3 from 6-7.5 ft bgs.

Scotch Creek Culvert Photo 5 – BH-SC01 location looking south.

Scotch Creek Culvert Photo 6 – BH-SC02 Location looking southwest.

#

Camp Creek Culvert Photo 1 – BH-CC01 Sample 1.3 from 7-8.5 ft bgs.



#

Camp Creek Culvert Photo 2 – BH-CC02 Sample 2.2 from 4-6 ft bgs.

Camp Creek Culvert Photo 3 – BH-CC01 immediately after drilling completion, ground water present in borehole.

Camp Creek Culvert Photo 4 – BH-CC02 location looking North East, Camp Creek Culvert to the right of picture frame (not pictured).

#

Fall Creek at Daggett Road Photo 1 – BH-DG03 looking west.

Fall Creek at Daggett Road Photo 2 – BH-DG03 looking south.

#

Fall Creek at Daggett Road Photo 3 – BH-DG04 bulk sample at 5 ft bgs.

Fall Creek at Daggett Road Photo 4 – BH-DG04 looking south-east.

#

Fall Creek at Substation Road Photo 1 – BH-DG02 Sample 1.1 from 3.5-5 ft bgs.

Fall Creek at Substation Road Photo 2 – BH-DG02 Sample 1.3 from 8.5-10 ft bgs.

#

Fall Creek at Substation Road Photo 3 – BH-DG01 Sample 2.1 from 3.5-5 ft bgs.

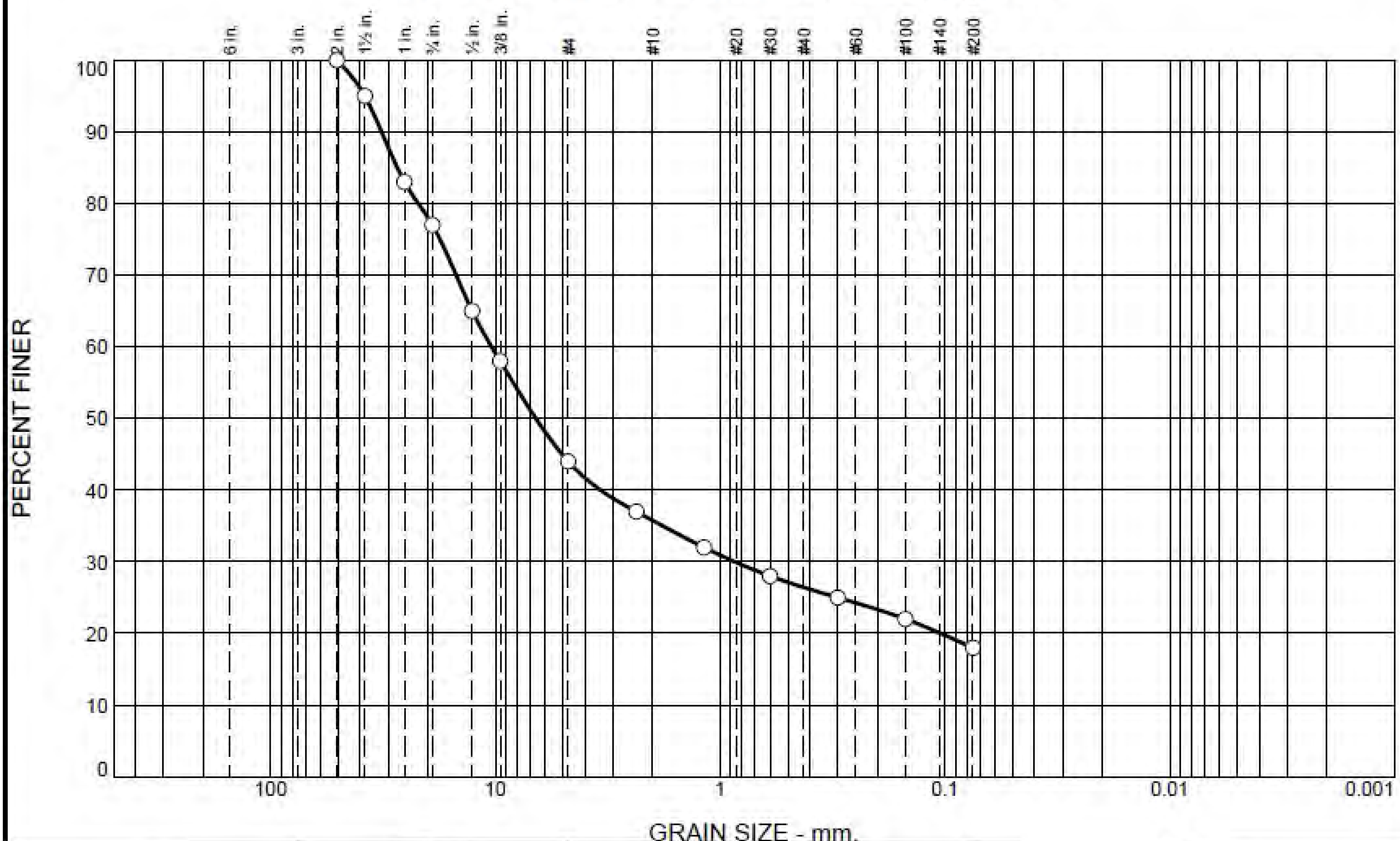
Fall Creek at Substation Road Photo 4 – BH-DG01 location in foreground to the left (white circle), BH-FCSSR-01 location at back of drill rig trailer behind stop sign in background, looking west-northwest.


#
Fall Creek at Copco Road Photo 1 – BH-FC03 in foreground, BH-FC01 and BH-FC02 across the bridge in background on left and right respectively, view is looking west-southwest.

Fall Creek at Copco Road Photo 2 – BH-FC04 in foreground, BH-FC02 across the bridge in background, view is looking west-southwest, Fall Creek upstream to the right.

Particle Size Distribution Report

PRIVILEGED AND CONFIDENTIAL



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	23	33	8	10	8	18	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100		
1 1/2"	95		
1"	83		
3/4"	77		
1/2"	65		
3/8"	58		
#4	44		
#8	37		
#16	32		
#30	28		
#50	25		
#100	22		
#200	18		

* (no specification provided)

Material Description

Dark Brown Clayey Gravel with Sand

Atterberg Limits

PL= 19 LL= 44 PI= 25

Coefficients

D_{90}= 32.0836 D_{85}= 27.3453 D_{60}= 10.4077
D_{50}= 6.6082 D_{30}= 0.8621 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= GC AASHTO= A-2-7(0)

Remarks

Material tested in accordance with ASTM D6913.

Sample Number: 1

Date: 07/13/2020

MATERIALS TESTING INCORPORATED M T I

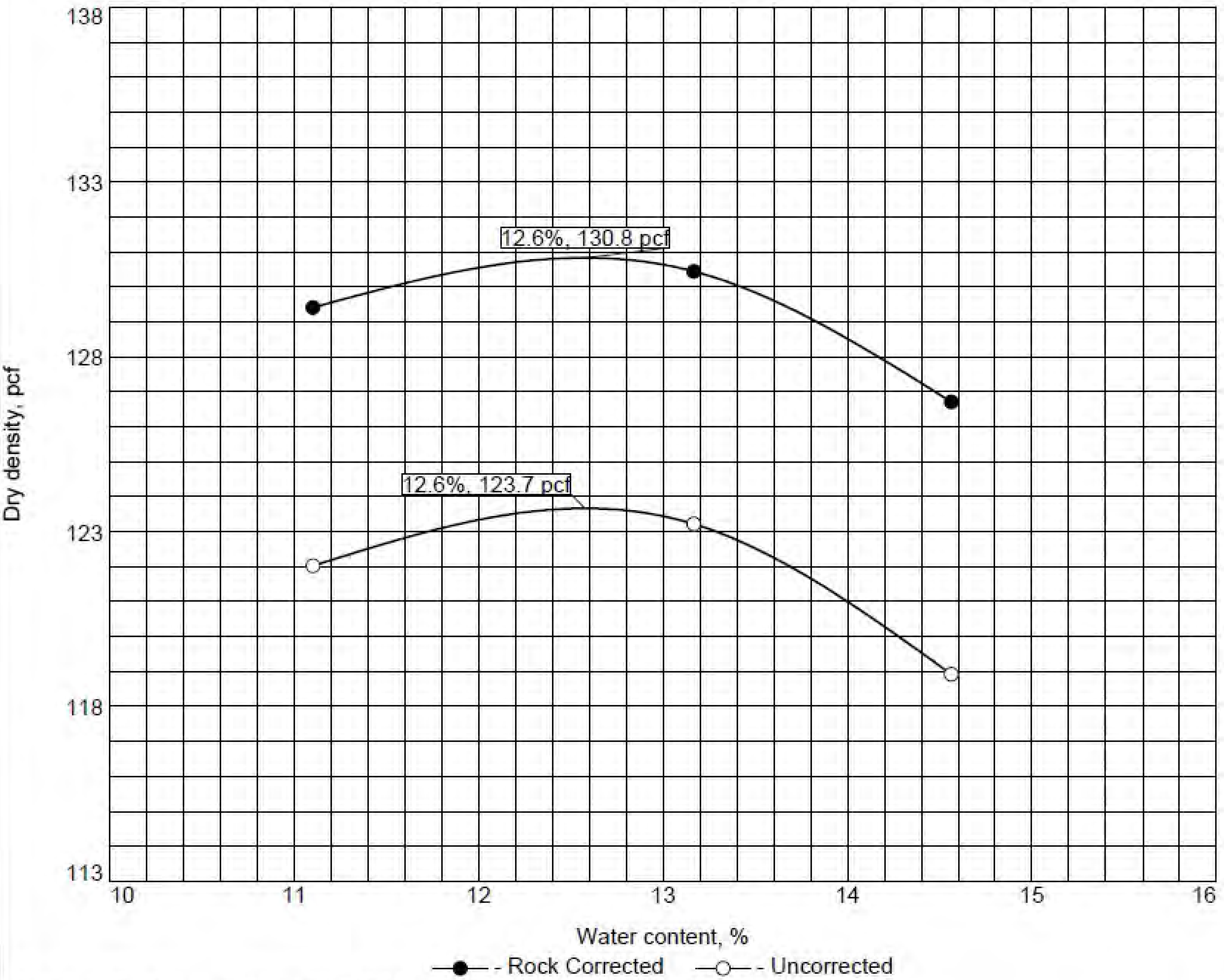
Client: GeoServ, Inc.
Project: KRRP Dry Creek Site Investigation

Project No: 3155-025 **Figure** 0300-001

Tested By: John Hubbard

COMPACTION TEST REPORT

PRIVILEGED AND CONFIDENTIAL



Test specification: ASTM D1557-12 Method C (with uncorrected) Modified
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
	GC	A-2-7(0)		2.60	44	25	23	18

ROCK CORRECTED TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 130.8 pcf Optimum moisture = 12.6 %	Dark Brown Clayey Gravel with Sand

Project No. 3155-025 **Client:** GeoServ, Inc.
Project: KRRP Dry Creek Site Investigation

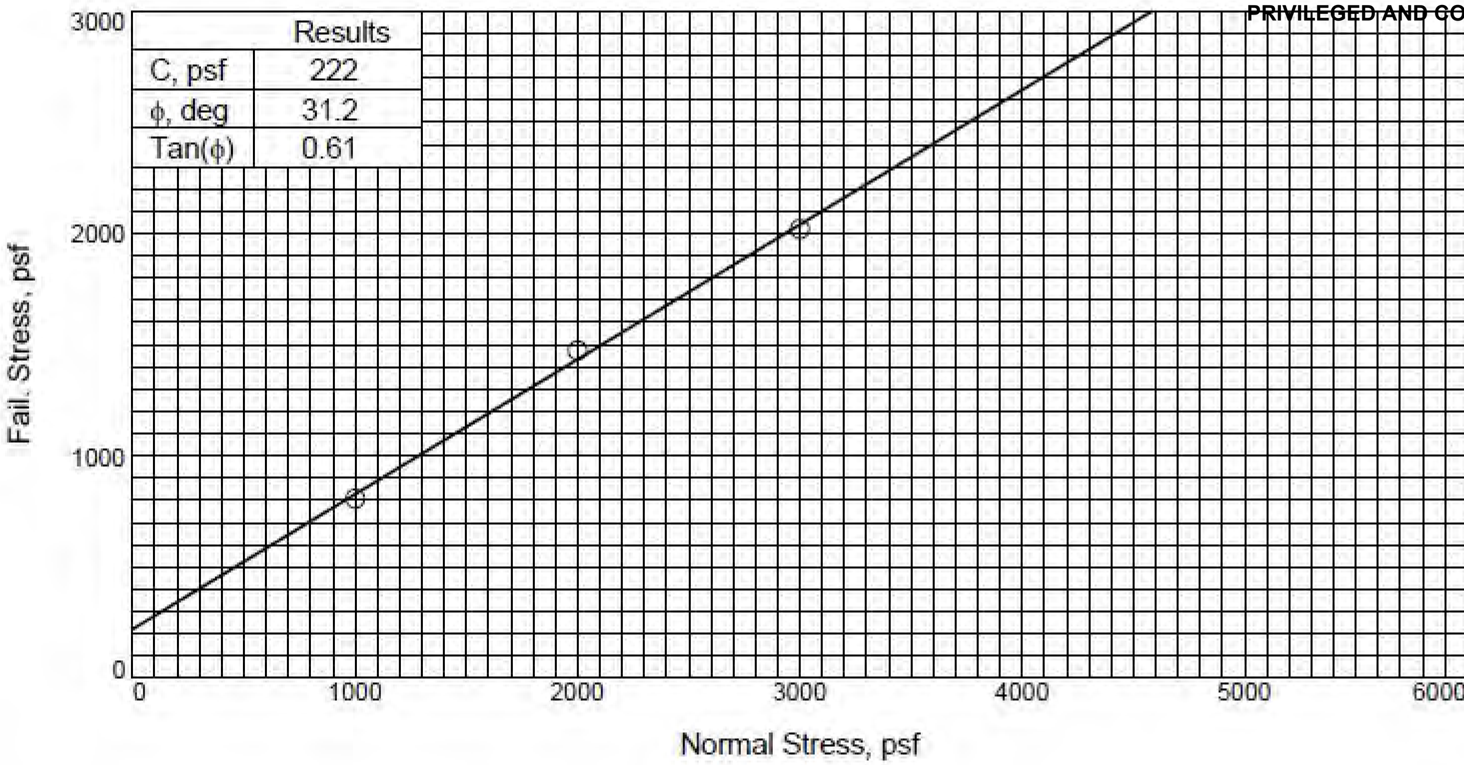
○ **Sample Number:** 1

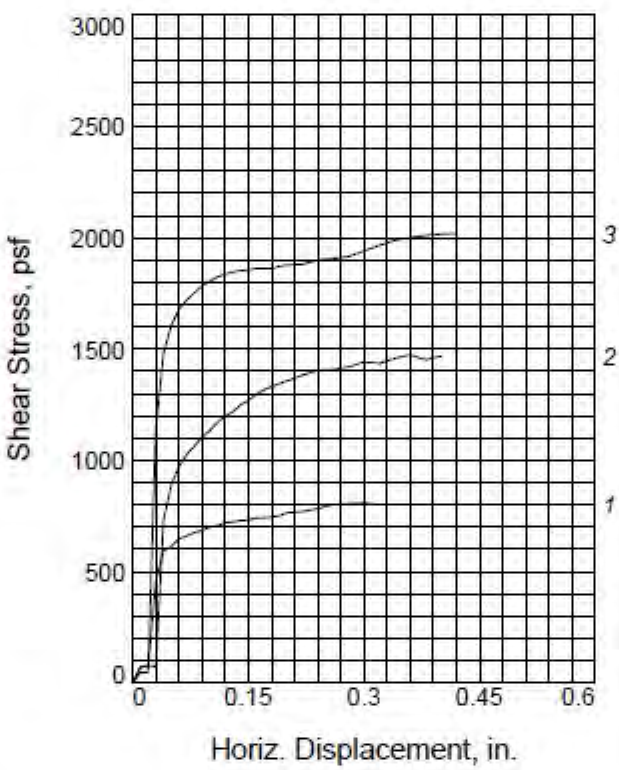
Remarks:
Curve #1
07/01/2020

Figure 0300-002

Tested By: Jack Bianchin

Results	
C, psf	222
ϕ, deg	31.2
Tan(ϕ)	0.61





	Sample No.	1	2	3
Initial	Water Content, %	17.6	17.6	17.6
	Dry Density, pcf	117.5	117.5	117.5
	Saturation, %	109.3	109.3	109.3
	Void Ratio	0.4348	0.4348	0.4348
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	29.2	26.2	25.6
	Dry Density, pcf	85.2	112.8	113.7
	Saturation, %	80.6	143.1	143.2
	Void Ratio	0.9786	0.4936	0.4826
	Diameter, in.	2.41	2.41	2.41
	Height, in.	1.38	1.04	1.03
	Normal Stress, psf	1000	2000	3000
	Fail. Stress, psf	808	1474	2020
	Displacement, in.	0.30	0.36	0.42
	Ult. Stress, psf			
	Displacement, in.			
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Remolded
Description: Dark Brown Clayey Gravel with Sand

LL= 44 **PL=** 19 **PI=** 25
Specific Gravity= 2.70
Remarks: Material tested in accorance with ASTM D3080. Remolded to 95% Maximum Uncorrected at 5% above Optimum Moisture.

Figure 0300-003

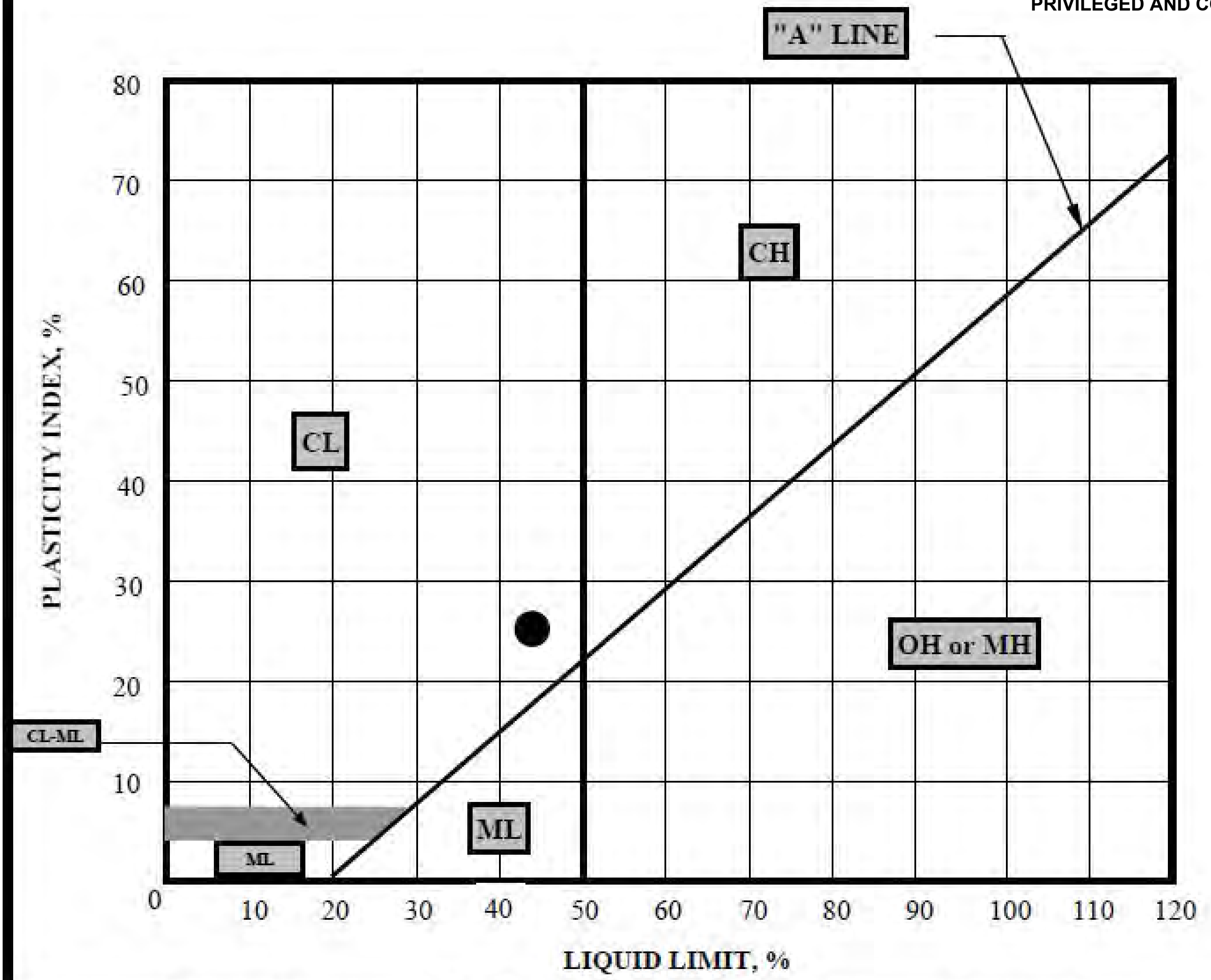
Client: GeoServ, Inc.

Project: KRRP Dry Creek Site Investigation

Sample Number: 1

Proj. No.: 3155-025 **Date Sampled:** 07/13/2020

Tested By: Cindy Gooden



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	1	---	N/A	44	19	25	N/A	GC

Note: Atterberg Limits tested in accordance with ASTM D4318.

MATERIALS TESTING M T I INCORPORATED

Materials Testing, Inc.

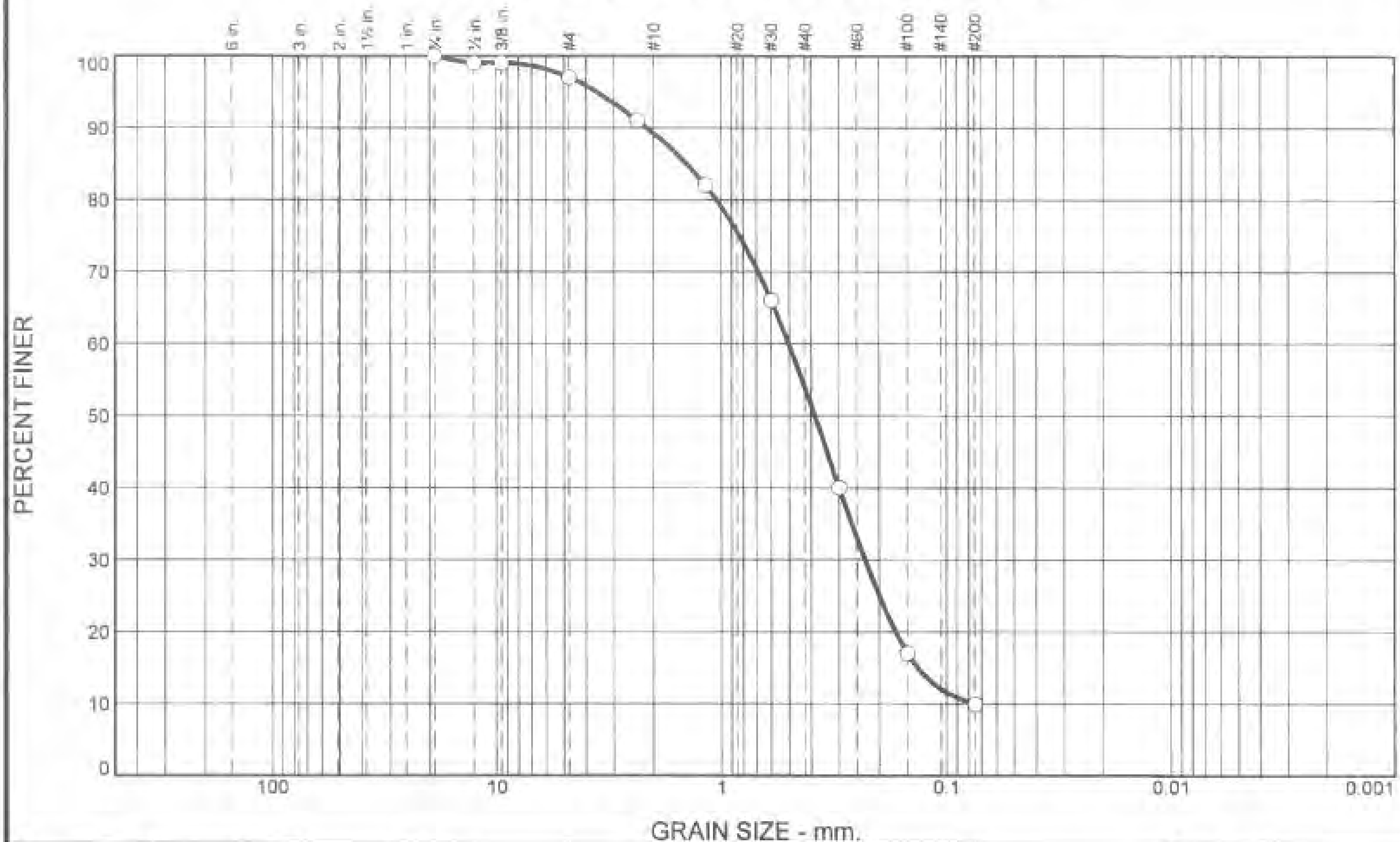
PLASTICITY CHART AND DATA

KRRP Dry Creek Site Investigation

Project No:	Date:	Figure No:
3155-025	7/13/2020	0300-004

Particle Size Distribution Report

PRIVILEGED AND CONFIDENTIAL



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	3	8	35	44	10	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100		
1/2"	99		
3/8"	99		
#4	97		
#8	91		
#16	82		
#30	66		
#50	40		
#100	17		
#200	9.9		

* (no specification provided)

Material Description

Brown Sand with Clay (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.1478 D_{85}= 1.4304 D_{60}= 0.5025
D_{50}= 0.3858 D_{30}= 0.2314 D_{15}= 0.1351
D_{10}= 0.0766 C_u= 6.56 C_c= 1.39

Classification

USCS= SW-SC AASHTO=

Remarks

Material tested in accordance with ASTM D6913.

Location: BH-CC-01
Sample Number: 1 **Depth:** 7.0' - 20.0' **Date:** 05/29/2020

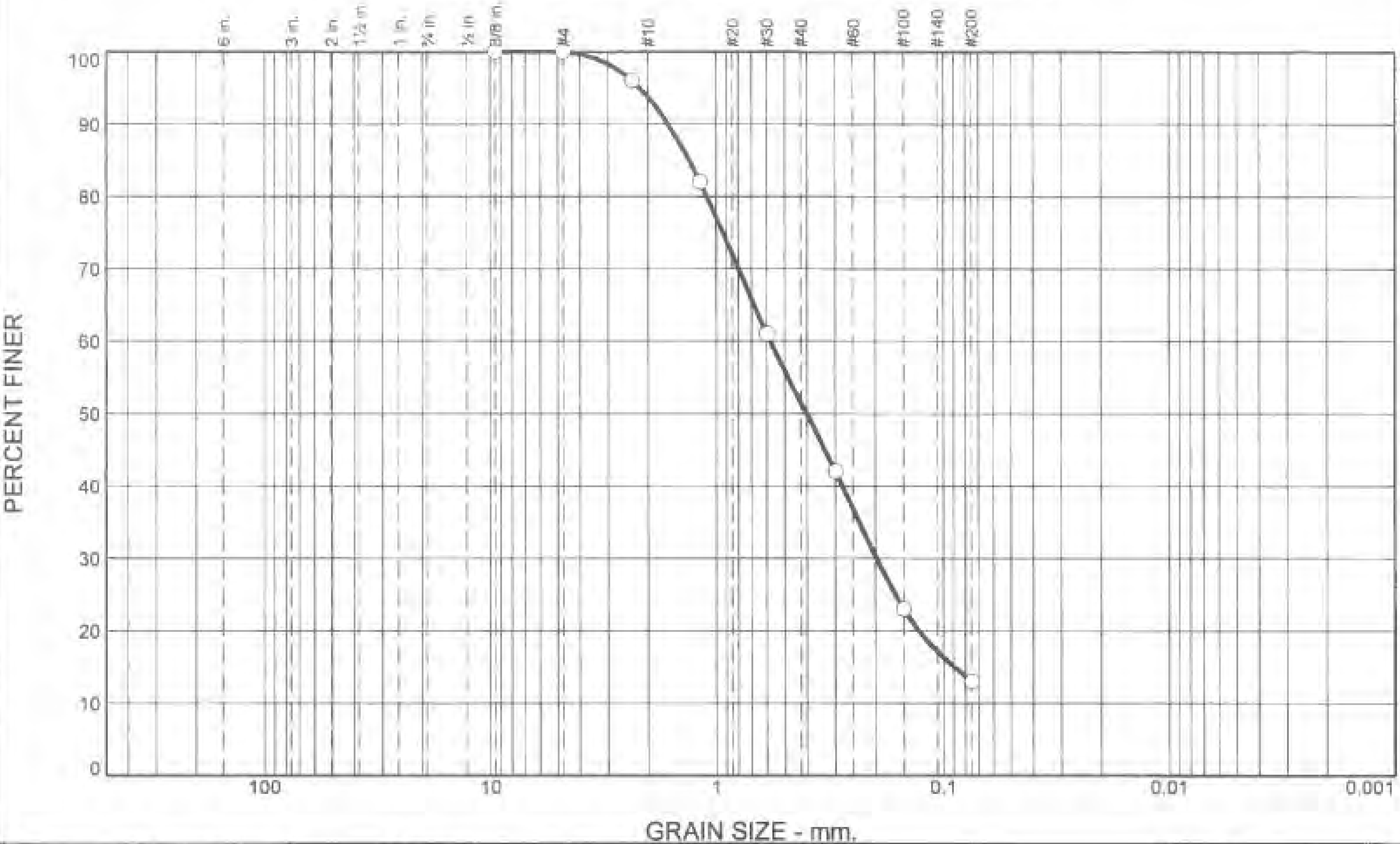
MATERIALS TESTING INCORPORATED M T I

Client: GeoServ, Inc.
Project: KRRP Camp Creek Site Investigation
Project No: 3155-023 **Figure** 0300-001

Tested By: Allante Blocker

Particle Size Distribution Report

PRIVILEGED AND CONFIDENTIAL



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	6	43	38	13	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100		
#4	100		
#8	96		
#16	82		
#30	61		
#50	42		
#100	23		
#200	13		

* (no specification provided)

Material Description

Gray Clayey Sand (visual)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.6382 D_{85}= 1.3199 D_{60}= 0.5800
D_{50}= 0.4030 D_{30}= 0.1982 D_{15}= 0.0898
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

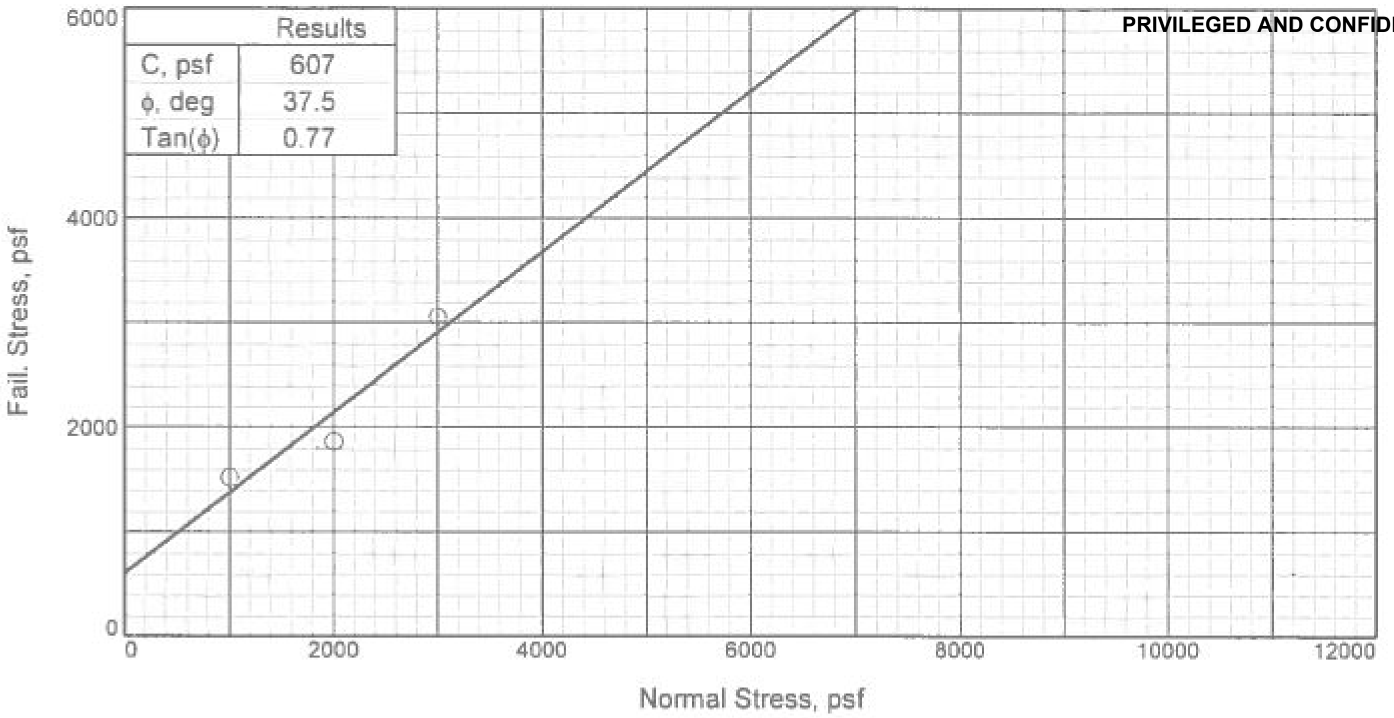
Material tested in accordance with ASTM D6913.

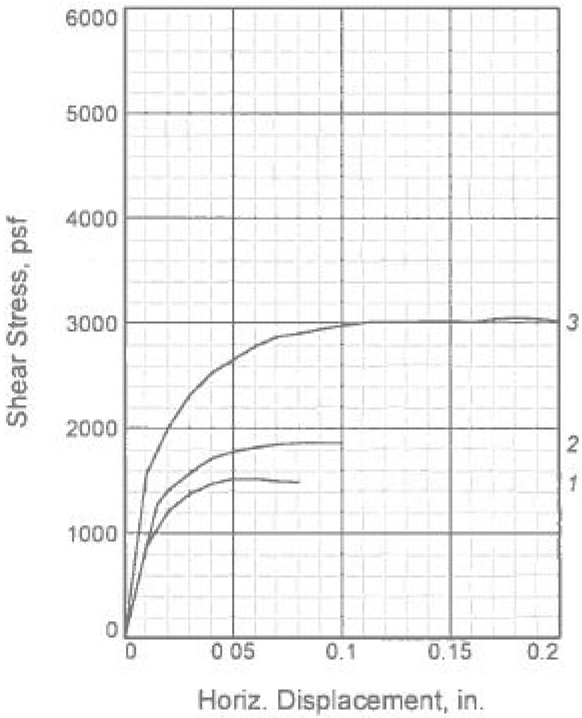
Location: BH-CC-02
Sample Number: 2 **Depth:** 7.0' - 20.0' **Date:** 05/29/2020

MATERIALS TESTING INCORPORATED M T I

Client: GeoServ, Inc.
Project: KRRP Camp Creek Site Investigation
Project No: 3155-023 **Figure** 0300-002

Tested By: Jack Bianchin





	Sample No.	1	2	3
Initial	Water Content, %	15.0	15.0	15.0
	Dry Density, pcf	90.0	90.0	90.0
	Saturation, %	44.2	44.2	44.2
	Void Ratio	0.9566	0.9566	0.9566
	Diameter, in.	1.94	1.94	1.94
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	26.2	28.4	26.8
	Dry Density, pcf	87.8	86.8	87.7
	Saturation, %	73.6	77.8	75.1
	Void Ratio	1.0043	1.0272	1.0063
	Diameter, in.	1.94	1.94	1.94
	Height, in.	1.02	1.04	1.03
	Normal Stress, psf	1000	2000	3000
	Fail. Stress, psf	1515	1861	3050
	Displacement, in.	0.05	0.09	0.18
	Ult. Stress, psf			
	Displacement, in.			
	Strain rate, in./min.	0.002	0.002	0.002

Sample Type: Remold
Description: Brown Sand with Clay (visual)

Specific Gravity= 2.82
Remarks: Material tested in accordance with ASTM D3080.
Remolded to 90 p.c.f. @ 15% Moisture.

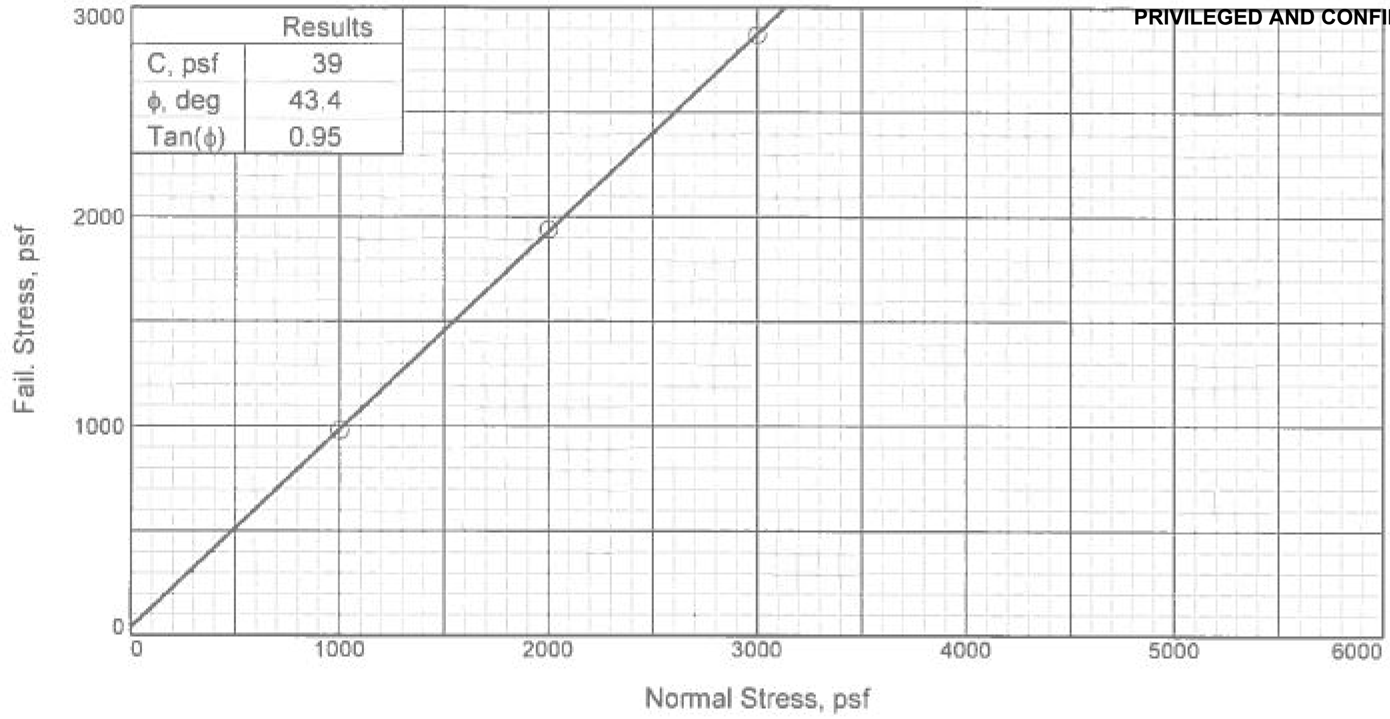
Figure 0300-003

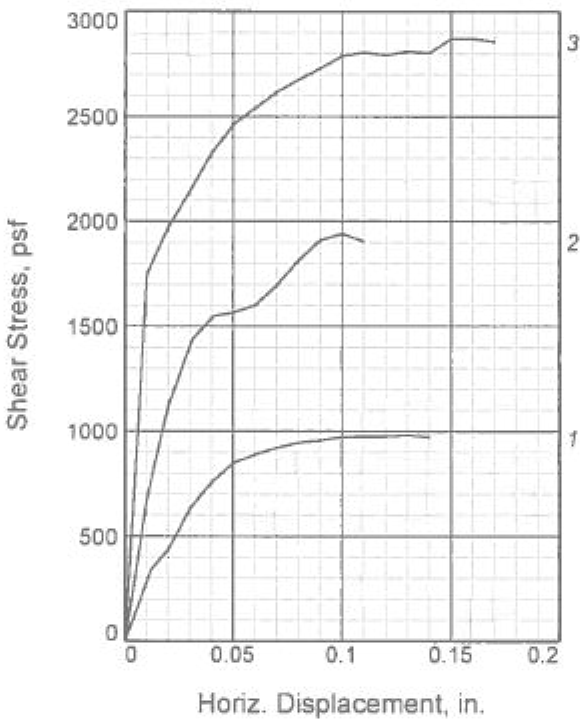
Client: GeoServ, Inc.

Project: KRRP Camp Creek Site Investigation

Location: BH-CC-01
Sample Number: 1 **Depth:** 7.0' - 20.0'
Proj. No.: 3155-023 **Date Sampled:** 05/29/2020

Tested By: Cindy Gooden





	Sample No.	1	2	3
Initial	Water Content, %	15.0	15.0	15.0
	Dry Density, pcf	90.0	90.0	90.0
	Saturation, %	46.2	46.2	46.2
	Void Ratio	0.8803	0.8803	0.8803
	Diameter, in.	1.94	1.94	1.94
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	26.4	24.6	25.8
	Dry Density, pcf	87.4	88.3	88.4
	Saturation, %	76.6	72.8	76.4
	Void Ratio	0.9356	0.9169	0.9145
	Diameter, in.	1.94	1.94	1.94
	Height, in.	1.03	1.02	1.02
Normal Stress, psf		1000	2000	3000
Fail. Stress, psf		979	1939	2869
Displacement, in.		0.13	0.10	0.15
Ult. Stress, psf				
Displacement, in.				
Strain rate, in./min.		0.002	0.002	0.002

Sample Type: Remold
Description: Gray Clayey Sand (visual)

Specific Gravity= 2.71
Remarks: Material tested in accordance with ASTM D3080.
Remolded to 90 p.c.f. @ 15% Moisture.

Figure 0300-004

Client: GeoServ, Inc.

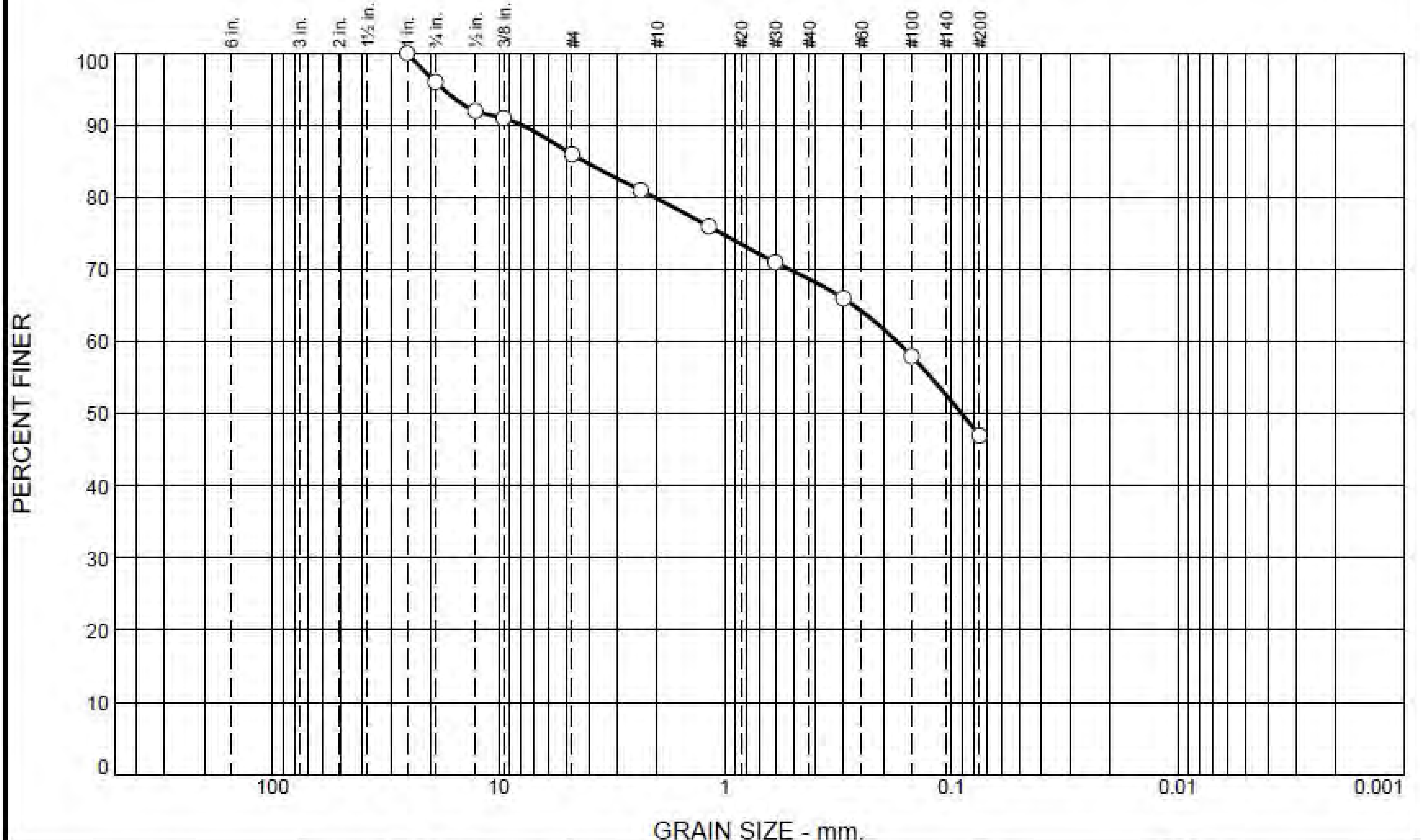
Project: KRRP Camp Creek Site Investigation

Location: BH-CC-02
Sample Number: 2 **Depth:** 7.0' - 20.0'
Proj. No.: 3155-023 **Date Sampled:** 05/29/2020

Tested By: Cindy Gooden

Particle Size Distribution Report

PRIVILEGED AND CONFIDENTIAL



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	4	10	6	11	22	47	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100		
3/4"	96		
1/2"	92		
3/8"	91		
#4	86		
#8	81		
#16	76		
#30	71		
#50	66		
#100	58		
#200	47		

* (no specification provided)

Material Description

Brown Clayey Sand

Atterberg Limits

PL= 21 LL= 67 PI= 46

Coefficients

D_{90}= 7.8598 D_{85}= 4.1801 D_{60}= 0.1737
D_{50}= 0.0898 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-7-6(16)

Remarks

Material testd in accordance with ASTM D6913.

Sample Number: 1

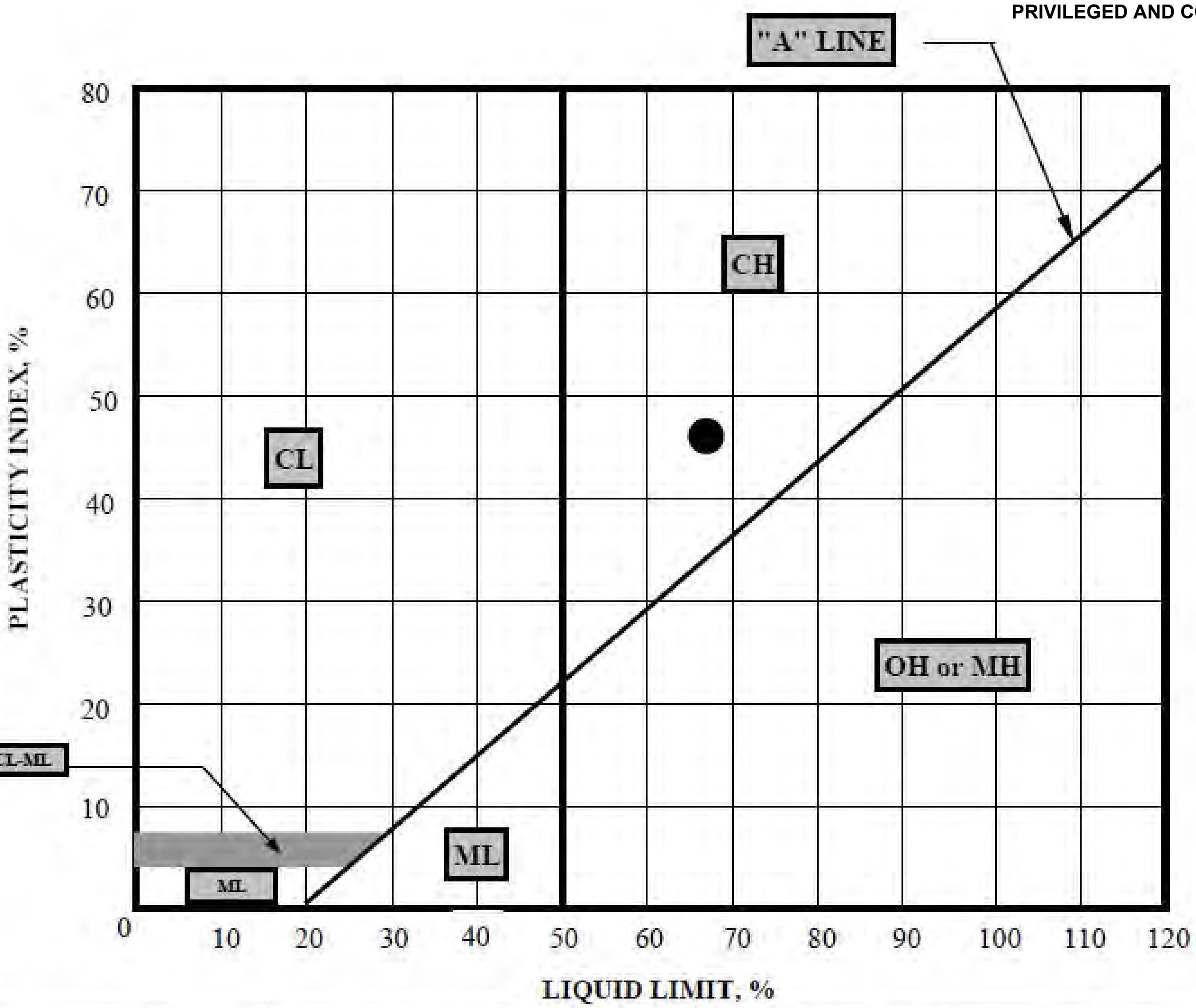
Date: 07/09/2020

MATERIALS TESTING INCORPORATED M T I

Client: GeoServ, Inc.
Project: KRRP Fall Creek at Daggett Road Site Investigation

Project No: 3155-026 **Figure** 0300-001

Tested By: John Hubbard



KEY SYMBOL	SAMPLE NUMBER	DEPTH	NATURAL MOISTURE CONTENT, %	LIQUID LIMIT, LL, %	PLASTIC LIMIT, PL, %	PLASTICITY INDEX, PI, %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
●	1	---	N/A	67	21	46	N/A	SC

Note: Atterberg Limits tested in accordance with ASTM D4318.

MATERIALS TESTING M T I INCORPORATED

Materials Testing, Inc.

PLASTICITY CHART AND DATA

KRRP Fall Creek @ Daggett Road
Site Investigation

Project No:	Date:	Figure No:
3155-026	7/9/2020	0300-002

March 11, 2021

Knight Piésold
KRRP Project Office
4650 Business Center Drive
Fairfield, California
USA, 94534
www.knightpiesold.com

Mr. Erik Esparza
Design-Build Coordination Manager
Kiewit Infrastructure West Co.
4650 Business Center Drive
Fairfield, California
USA, 94534

Dear Erik,

RE: KRRP - COPCO NO. 1 AND COPCO NO. 2 TEMPORARY CONSTRUCTION ACCESS ROAD – DESIGN CRITERIA

This letter is intended to outline the design criteria for the temporary construction access road to access Copco No. 1 and Copco No. 2 dam sites from the right bank as part of the Klamath River Renewal Project. Following acceptance of the criteria herein, KP will proceed with detailed stability analysis of the proposed alignment (appended).

1.0 PRIMARY DESIGN OBJECTIVES

- Design a stable temporary haul road to access Copco No. 1 and Copco No. 2 dams.
- Avoid any material entering the wetted perimeter of the Klamath River.
- Maximize usable excavated rock for use as erosion protection and general fill in other project areas.
- Locate spoil areas within the project limits to reduce labor and haul costs.
- Design will be California PE stamped by GeoServ.

2.0 DESIGN CRITERIA

2.1 GENERAL HAUL ROAD DESIGN CRITERIA

- Copco 1 Access Road shall be designed as a temporary structure (i.e., 2 - 3-year design life).
- Copco 1 Access Road is for Contractor (and PacifiCorp temporarily) use only during construction and will be left as-is following Project completion. Following Project completion, road access will be blocked or prevented via rock/earthfill berm or other approved methods. A posted permanent sign will be needed at project completion at the barrier indicating danger of rock fall and slope stability for pedestrians. Proceed at your own risk.
- Runaway vehicles are not considered in this design due to tight spatial constraints.
- The road design is based on site conditions, temporary haul road industry best practices, equipment specifications and Contractor inputs.
- Copco 1 Access Road maintenance of driving surfaces, and any daily or weekly maintenance required for continued compliance to the design geometry for the duration of use, will be the responsibility of the Contractor.
- Following a snow event - road shall be ploughed clear and if icy, adhesion shall be improved through placing cinder/salts etc. or as per Contractor's road maintenance plan.
- Slope hazard mitigation shall be managed by the Contractor.

- Temporary road signage will be as required by the Contractor.
- Best Management Practices (BMP's) will be implemented.
- On-site materials will be used for haul road construction.

2.2 VEHICLE LOADING

- The road is designed to accommodate the maximum imposed tire contact pressure due to a fully loaded CAT 745C haul vehicle, as defined by the Contractor.
- Other construction equipment may be used, provided the maximum applied loads do not exceed the fully loaded CAT 745C.
- Vehicle Load Factor (dynamic impact/braking etc.) = 1.25.

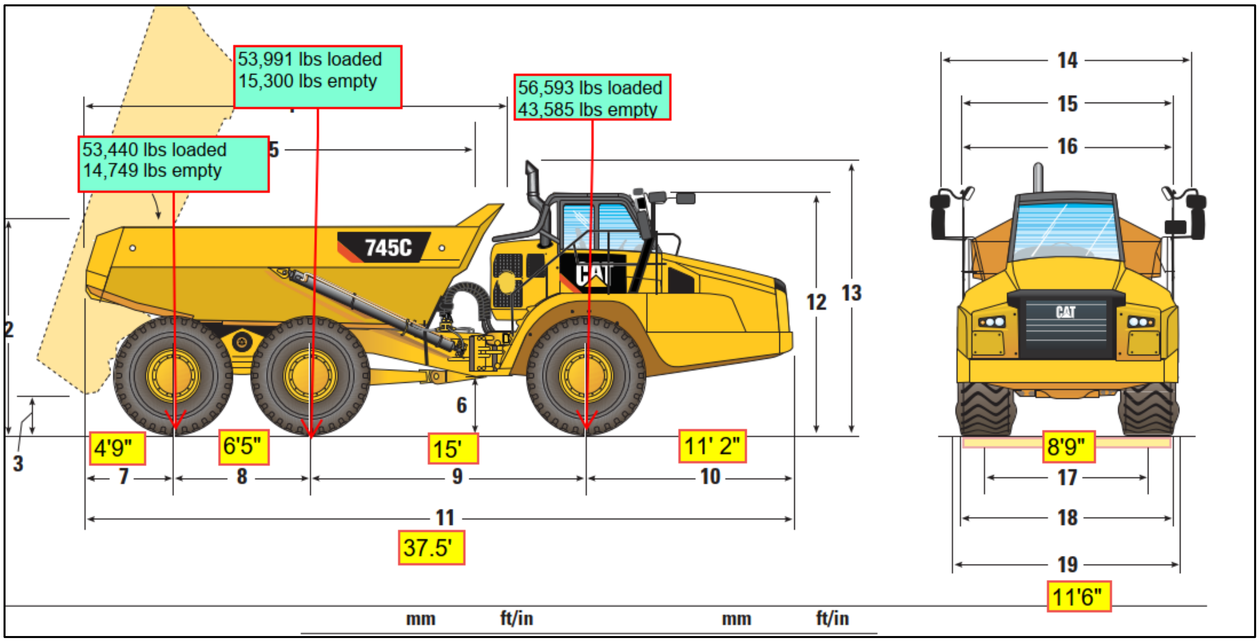


Figure 1 **CAT 745C Articulated Truck – Unfactored Nominal Axle Loads**
[(CAT Product Specifications Sheet)]

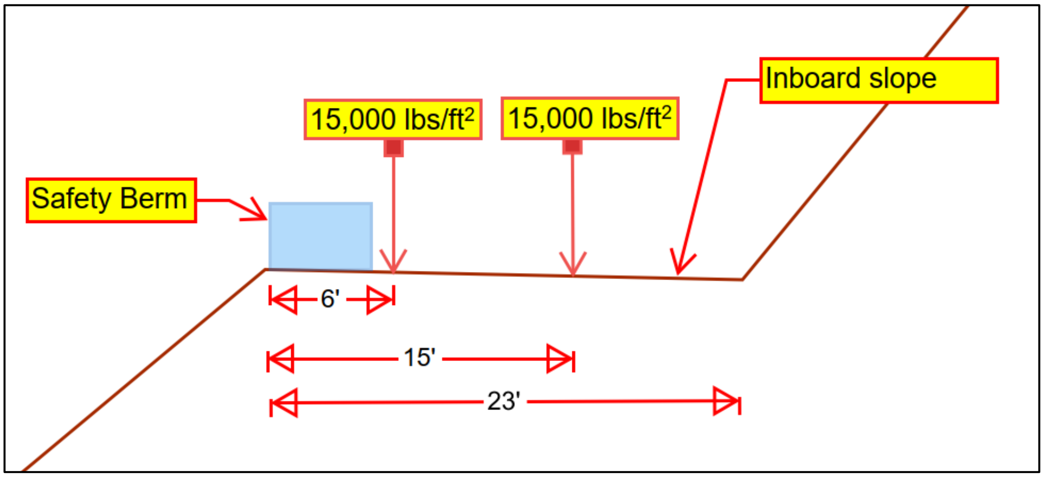


Figure 2 2D Slope Section, CAT 745C Axle Loading (based on tire contact pressure)

2.3 ROAD DESIGN CRITERIA

- 23 ft wide road - single lane (one way) traffic only.
  - 5 ft wide x 3 ft high safety berm (gabion baskets or earthfill/rock berm - TBD by Contractor).
  - 18 ft wide driving surface.
- Safety Berm is not designed for impact. Safety berm provides operators with visual/tactile feedback for vehicle alignment and road vehicular positioning.
- Maximum allowable road grade is 20%.
- Road will slope inwards from the outer edge to direct surface run-off. Drainage culverts and water management to be provided by the Contractor.
- Minimum allowable outside vehicle turning radius = 35 ft.

2.4 SLOPE STABILITY (GRANULAR MATERIALS)

- Includes all slopes comprised of loose/granular material (soils, gravel, rubble).
- Required factor of safety for limit equilibrium under static conditions will be 1.5.
- Inputs based on site investigation (conduction by GeoServ, January 2021) and industry best practice for temporary haul roads (per MSHA and best practices for Forestry/Mining Haul road applications).
- Peak Flood Events (considered for toe stability of sidecast material zones)
  - Copco No. 1 Flood Event (1% Probable Flood, Post Drawdown WSL) EL. 2,503.6
  - Copco No. 2 Flood Event (1% Probable Flood, Post Drawdown WSL) EL. 2,480.3
  - Copco Access Road (1% Probable Flood, Post Drawdown WSL) EL. 2,488.0
  - Freeboard Assumed for long term slope stability 2 ft
- Seismic Peak Ground Acceleration (100 Year Return Period) 0.036g
- Sidecast spoiled material - slope stability factor of safety (FOS) 1.05

2.5 SLOPE STABILITY (ROCK)

- Includes all slopes cut into solid rock (e.g., columnar basalt).
- Required factor of safety for limit equilibrium under static conditions (deep rock slope plane failure, toppling failure) will be 1.05.
- Required factor of safety for limit equilibrium under seismic conditions will be 1.05.

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- Design does not include assessment of overhangs, detached portions of basalt, potential surface raveling, surface freeze thaw action, or other similar rock face hazards.
- Rock slope hazard mitigation (i.e., weathered/fractured rock in the first 5 ft of rock slope surface) will be managed by the Contractor.

3.0 CLOSING

We trust the information contained herein meets your needs at this time.

Please do not hesitate to contact any of the undersigned if you have any questions or comments.

Yours truly,
Knight Piésold

Prepared: James O'Reilly

Reviewed: Norm Bishop

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

References:

U.S. Department of Interior, Bureau of Mines. Design of Surface Mine Haulage Roads – A Manual, Information Circular 8758, 2001.

Weaver, W.E., Weppner, E.M. and Hagans, D.K., Handbook for Forest, Ranch and Rural Roads: A Guide for Planning, Designing, Constructing, Reconstructing, Upgrading, Maintaining and Closing Wildland Roads (Rev. 1st ed.), Mendocino County Resource Conservation District, Ukiah, California, 2015.

Oregon Department of Forestry, State Forests Program, Forest Roads Manual, 2000.

Copy To: Nick Drury
Gary Jara
Jim Fitzgerald
Chuck Schlumberger

APPENDIX F5.2
COPCO NO. 1 ACCESS ROAD DESIGN

TABLE OF CONTENTS

1.0 SCOPE

The Copco No.1 Access Road Geotechnical Design Report Appendix contains an overview of the geotechnical design recommendations for temporary construction access roads for Copco dam removal as part of the Klamath River Renewal Project (KRRP).

This document is intended to provide a comprehensive overview of the geotechnical design development for the Copco No. 1 temporary construction access roads as part of the KRRP. The Project Drawings (100%

Design Drawing Package) and Appendix F1 of the 100% Design Report should be reviewed in conjunction with this document. The supporting figures and geotechnical data used for design are documented in the KRRP Geotechnical Data Report, Appendix I – Copco No. 1 Access Road Geotechnical Report.

2.0 METHODS

This investigation used the 100% Project Drawings and information obtained during the Site Investigation (SI) phase to develop geotechnical design parameters and help progress the KRRP transportation infrastructure design. This SI was completed to obtain information on the engineering properties of the rock, soil, groundwater, and to inform the designs and construction techniques for the proposed roads. The engineering properties of the project area rocks and soils were assessed using industry standard methods (e.g., CDC 2001, Williamson 1984, and BOR 2001). The rocks and soils were classified and assessed following the most recent ASTM methods.

The SI was completed in January 2021 (GSI 2021). The test pits, seismic refraction (SR), ground penetrating radar (GPR), and bedrock mapping sites were located along the proposed road alignment in safe accessible locations to characterize the spatial distribution of the terrane, rock, soil, and water conditions. This sampling scheme was intended to assess the spatial and temporal distribution of soil or rock near the ground surface. The SI comprised the following data collection:

- 12 test pits with logs and samples
- 2,940 ft of Seismic Refraction (SR) surveys over 11 transects
- 3,689 ft of Ground Penetrating Radar (GPR) surveys over 17 transects
- Bedrock outcrop surface maps and samples

Design criteria for the temporary construction access roads documented in KP Letter VA21-00436 "KRRP - Copco No. 1 And Copco No. 2 Temporary Construction Access Road – Design Criteria", (March 11, 2021), refer to Appendix F5.1 of this report.

- This phase of the analysis did not evaluate the permanent slope stability of the road prism rock and fill slopes.
- Static Factor of Safety (FOS) for temporary rock cut-slopes = 1.05.
- Seismic FOS for temporary rock cut-slopes = 1.0 (PGA = 0.036 - 100-year return interval).
- Static Factor of Safety (FOS) for temporary rock fill slopes = 1.5.
- Seismic FOS for temporary rock fill slopes = 1.1 (PGA = 0.036 - 100-year return interval).
- For rock cut and fill slopes, construction traffic line loads per VA21-00436.
- All temporary road prisms are on full bench rock cuts or rock fills founded on stable rock or rock rubble.
- New side cast spoils placed over existing loose fill, at least 2 feet above the 1% flood water surface level, and do not increase finished slope height or angle.

3.0 GEOLOGIC CONDITIONS

3.1 SURFACE AND SUBSURFACE CONDITIONS

The project area is underlain mainly by igneous rocks (Figure 1). The rock types include columnar basalt, blocky basalt, and cinder/pyroclastic flows that were erupted within the last 1 to 2 thousand years. The sequence of eruptions is a complex sequence of cinder cone eruptions interspersed with basalt eruptions. During this period of eruptions, the Klamath River was likely blocked by basalt flows, and available geologic data indicate that the river was dammed forming a lake 35 ft. higher than the present reservoir elevation (CDMG 1983). The modern outcrops express the sequence of eruptions where the base of the cinder cone to the north is covered by basalt that flowed from the south (Figure 2). Subsequently, the Klamath River incised through the basalt dam creating the present day river canyon. Diatomaceous earth deposits were used to understand the elevation and extent of the basalt dam (CDMG 1983). Though time, the river cut through the hard and soft rock forming the near vertical canyon walls and intermediate sloped benches formed by softer rock. The blocky and columnar basalt outcrops continuously shed columns of basalt forming colluvial slopes at the base of the outcrops. Occasionally, the rock slope faces topple in mass shedding large areas of loose rock into the river canyon.

The basalt and cinder outcrops were cut into to access the Copco No. 1 and No. 2 dam sites to include railroad and equipment access roads. The rock slopes were blasted and excavated to access the dam sites. Historical photographs show that there were intermittent large rock falls that blocked the rail and road access routes.

During construction, the spoils were side cast, and rail and road prisms are mainly composed of cut and fill. The side cast fill slopes are still present today and cover the lower half of the slopes within the project area (Figure 2). The existing road is supported by loose fill slopes on the outside edge of the road. The fill slopes tend to be composed of loose soil, rock, and metal and wood debris. The existing road traverse the fill slopes and there is active erosion and shallow failures, mainly along the outboard edge.

The SI results indicate that there are several existing road segments with loose uncompacted fill. SR and GPR data taken along and perpendicular to the roads match the direct observations made in the test pits and show that there are large wedges of loose fill along the outside edge of the road from east to west (Figure 2). The fill tends to be between 5 ft. and 20 ft. thick. The surveys detected large voids withing the road prisms especially at the lower end near Copco No. 1 dam.

The full bench rock cut road prisms tend to be stable less the frequent rock fall or topple from the cut-slope. The upper portion of the existing road tends to be in full bench or through cuts.

4.0 GEOLOGIC HAZARDS

4.1 ACTIVE FAULTS AND SEISMIC HAZARD ASSESSMENTS

Project construction and implementation would be subject to a low to moderate risk of damage from fault movement. Fault movement has the potential to affect the stability of the proposed structure(s). According to the CDC (2000), the closest known inactive fault is 16 miles east of the project area. Most of the faults east of the project area are considered active, and the most recent events were 4.3 and 4.4 magnitude earthquakes in 1974 and 2005, respectively. To initiate the dominant seismic hazards of the area, an earthquake would have a magnitude of 8.5 or greater (CDC, 1996).

Seismic movement from earthquakes has the potential to affect the stability of the proposed roads. According to the CDC (1997) and CDC (2006), the project area is not within a mapped Alquist-Priolo Earthquake Hazard Zone. It is unlikely that the proposed roads will be impacted by the effects of a large magnitude earthquake given that they are temporary. The proposed roads could be subjected to frequent smaller magnitude earthquakes. Small earthquakes may cause minor settling or shifting of unconsolidated sediments. Overall, there is a low risk of damaging earthquakes (Peterson 1996, Peterson 1999, and Toppozada, 2000).

4.2 LIQUEFACTION

Liquefaction typically occurs as a result of seismic events that cause the sudden loss of soil shear strength. The cyclic loading from an earthquake triggers liquefaction. The risk of liquefaction is based on the expected seismic event, soil properties, and groundwater depth. For liquefaction to occur the following must be present:

- Granular soils
- Low soil density
- High groundwater table

The project area rock or soils are granular in nature and lie atop dense volcanic rock. The risk of adverse impacts from liquefaction at the project area is low.

4.3 FLOODING HAZARD POTENTIAL

The flood hazard potential is addressed in Appendix F3 Hydrotechnical Design Report for Roads, Bridges, and Culverts.

4.4 DAM INUNDATION HAZARD POTENTIAL

The dam inundation hazard potential is addressed in Appendix F1 Roads, Bridges, and Culverts Design Details.

4.5 STREAM SCOUR

The stream scour hazard potential is addressed in Appendix F3 Hydrotechnical Design Report for Roads, Bridges, and Culverts.

4.6 EXPANSIVE SOILS

Potentially expansive clay soils were not encountered as part of the SI and the risk is low for the proposed road.

4.7 VOLCANIC HAZARDS

The project area is not within an area with recent volcanic activity, and the project area is in a zone that could be impacted by a volcanic eruption. Quantifying the volcanic risk to the project area is beyond the scope of this investigation. Overall, the risk of adverse impacts from volcanic activity at the project area is moderate to low.

4.8 SLOPE STABILITY

The project area is within a region with moderate to high landslide susceptibility. Based on the road locations, topography, and subsurface geology there is a high modern landslide risk. The loose fill slopes are prone to soil creep and shallow debris flows. The rock slopes are prone to rock topple. There are several existing rock slopes with active rock topple along the proposed road alignments. These sections of the roads have a high susceptibility to rock fall and large rock block topple failures.

4.9 TSUNAMIS AND SEICHE

Based on site location, elevation, and tsunami hazard mapping from the CGS website (http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=tsunami) the site is not in a tsunami inundation hazard zone. In addition, oscillatory waves (seiches) are considered unlikely due to the absence of large confined bodies of water in the site area.

4.10 EROSION AND SEDIMENTATION

There is a high erosion and sedimentation risk given that the proposed roads are adjacent to the Klamath River. Any construction related disturbance to the soils will increase the erosion risk, and temporary and permanent erosion control measures need to be implemented, per the Project Drawings and Technical Specifications, to keep storm water from discharging site soils and nutrients into the stream channels. Conceptual erosion and sediment control plans have been developed for each of the proposed road segments (see Project Drawings).

During construction, the contractor needs to implement the Temporary Erosion Control Plans as prescribed on the Project Drawings and California Construction General Permit Storm Water Pollution Prevention Plan (SWPPP) (Erosion and Sedimentation Control, Section 31 25 00) (California Water Board 2010a).

Post construction, the contractor needs to implement final erosion and sediment control measures that follow the Action Plan for the Klamath River Total Maximum Daily Loads (California Water Board 2010b). The final measures shall be implemented as shown on the Project Drawings and include embankment and disturbed area erosion control and controllable sediment discharge BMPs.

4.11 WILDLAND FIRE

The potential risk of wildfire depends on several factors, such as, abundance of flammable vegetation, high winds, topography, and seasonal weather. For the project area, there is a high threat of fire during the dry summer and fall periods due to chaparral and conifer vegetation and high winds. The project area has an extreme to elevated potential for wildfire hazard.

5.0 EARTHWORKS

5.1 SITE PREPARATION

Each project site should be stripped of vegetation and organic debris within the work limits. These materials should be stock piled and may be used as ground cover and revegetation efforts at the end of the project or disposed of offsite. Voids left from removal of debris should be replaced with native fill compacted to 90 percent relative compaction.

Project area stripping should include the demolition and removal of all existing structures including concrete foundations, metal debris, utility poles, underground utilities, concrete debris, vegetation, and other organic material in all the new road corridor and staging/spoils areas. Loose, weak, or otherwise unstable soil or rock in the road alignment corridor should be excavated and evaluated by KP for possible re-use as engineered fill. Utilities that extend into the construction area scheduled to be abandoned should be properly capped at the perimeter of the construction zone or moved as directed in the plans.

It is anticipated that large voids may be encountered during road construction, and voids large than 5 ft. across and 5 ft. deep should be reported to KP immediately and evaluated accordingly by the Project Engineer or designated representative. Based on the SI results, it is likely that abandon blasting adits, voids between large blocks of rock, and areas with decomposed organic matter (e.g., wood). The SI identified several locations with potential adits, and one adit that is known to exist near the north side of Copco No. 1 dam.

5.2 TRENCHES

Given the measured soil conditions, it is likely that the excavations can be sloped at 0.25H:1V with 4 ft. benches to 20 ft. bgs assuming that the Type A soil is homogeneous. At sites where the excavation is greater than 20 ft. deep, trench plates or other shoring methods will be needed due to depth of excavation. In addition, presence of saturated, medium dense, non-cohesive gravel excavations will need to be dewatered if water is present. Other shoring methods may be needed depending on the actual excavation depth and type of soil encountered during construction (OSHA 29 CFR 1926.650, 29 CFR 1926.651, and 29 CFR 1926.652). Shoring below 20 ft. bgs needs to be designed by a registered Professional Engineer. During construction, unusual changes in rock or soil strata should be evaluated by the Engineer or designated representative.

For temporary cut-slopes in soil or weathered rock, the slope angle should be no steeper than 1H:1V. For temporary cut-slopes in hard rock, the slope angle should be no steeper than 0.25H:1V. Final cut-slope angles may vary depending on the rock and soil conditions encountered. Variations in cut-slope angle can be field fit during construction as approved by the Engineer

5.3 CUT-SLOPES

This section provides rock slope recommendations for removal of bedrock in the back slope. Rock slope recommendations are provided for new construction access roads. The rock slope recommendations are based the following:

1. Rock type
2. Discontinuity (bedding, joints, fractures) orientation and frequency
3. Cut Height
4. Weathering
5. Presence of erodible material
6. Road orientation

An optimum rock slope design minimizes risk to the project and also minimizes the amount of excavation and stabilization required. Proper design includes selection of an optimum “safe” cut-slope angle together with an appropriate rock fall catchment area. The cut-slope is often referred to as a “cut-slope angle” vertical to horizontal (e.g., 1/4V:1H). The rock catchment area includes the flat ditch area plus the inslope that ends at the shoulder. The inslope normally varies between 1V:6H and 1V:4H.

Cut-slope angles were derived from an evaluation of rock mass characteristics, attained from a combination of measurement made of exposed bedrock faces. Additional factors considered for cut-slope selection include site conditions (groundwater, roadway orientation, and others) and professional judgement.

The rock slope design process was a trade off between stability and economics. Steep slopes and narrow ditches are usually less expensive to construct than the safer. Given that the geologic structure and type of rock vary considerably at each slope position within the project area, it is difficult to provide general guidelines for design recommendations that fit all circumstances. The following guidelines are created to fit typical conditions common to project area.

Soft rocks, which include principally cinder and pyroclastic rock, can be excavated without blasting. Hard basalt rock, will likely require blasting to excavate, include igneous, metamorphic rocks and carbonates. Tall rock cuts (10 ft to 30 ft in height) should closely follow the design criteria and Project Drawings. In the hard rock types, controlled blasting techniques may be required for final shaping of the cut face. Composite slopes, consisting of both soft and hard rock types (particularly with hard overlying soft) are susceptible to differential erosion and require careful consideration and field review by the Project Engineer or designated representative. Typically, the hard rock layer will be set back about 10 ft from the face of the underlying softer rock (e.g., cinder or loose rock rubble), with an impermeable bench constructed on top of the soft rock layer.

5.4 MATERIALS

Any construction Excavation and fill materials for the various components of the proposed road designs should follow the specifications listed in Table 5.1 according to the type and intended use. These material and placement and compaction, and testing specifications are based on the Copco Access Road design criteria.

Table 5.1 Excavation and Fill Material Types, Specifications and Testing for Copco Temporary Construction Access Roads

KP Material Type	Material Type	Material Specifications	Placement and Compaction Specifications	Compaction Test Type
Site Specific	Structural Sub-Grade	Firm and Unyielding Native Material free of debris, rocks > 4", and organics	Scarified subgrade to 8" depth, moisture conditioned to within 2% optimum moisture, and re-compacted to at least 95% relative compaction or until firm and unyielding under vibratory roller	ASTM D 698
Rock Fill	Rip-Rap	Crushed rock material generally 21" to 36" that consists of angular, durable rock and gravel, and <30% fines	Placed with heavy equipment, not dropped more than 2', compacted until firm and unyielding under mechanical movement of heavy equipment (worked in with appropriately sized excavator or bull dozer)	ASTM D 698

The proposed roads will be built using on-site material. Given that the majority of the roads will be full bench rock cut, there is limited need for material specifications.

For the rock fill along U-300, the material placed in temporary road embankments shall be hard, durable, angular, and shall have a fines content of less than 35% No.200 sieve.

Material shall be placed in maximum 1 ft. lifts and moisture conditioned to optimum levels, as approved by the Engineer during placement.

Backfill material for the temporary rock fill shall be as per the Project drawings in addition to meeting the following placement requirements.

Compaction to 90% relative density, to be achieved through the following observed method specification.

1. Minimum of 4 passes with a 20,000 lb vibratory roller, proof rolled (e.g., loaded 10 cubic yard minimum dump truck) to test for visible deflection, as measured every other lift.
2. For course granular fill (i.e., rock fill), material shall be compacted through track packing (18-ton minimum vehicle weight) as an alternative to vibratory rolling.

Material shall be placed in maximum 18 in. to 24 in. lifts and moisture conditioned to optimum levels, as approved by the Engineer during placement.

Material shall be free of organic debris and shall be moisture conditioned, as approved by the Engineer during placement.

5.5 TEMPORARY ROAD SEISMIC DESIGN

The seismic calculation tables are summarized in Table 5.2 and were developed using the recommended AASHTO seismic design parameters for temporary roads. The shallow subsurface material is classified using the site-specific soil and rock conditions. This classification is based on field observations and the measured engineering soil properties. For temporary structures, the seismic design criteria are based on a 100-year return period (this is equal to a 10% probability of exceedance in 10 years).

Table 5.2 Seismic Design Criteria

Site	Type	Site Class	Return Period (years)	PGA	S_1	S_s	S_{D1}	S_{DS}
Copco Construction Access Roads	Temporary	B	100	0.036	0.043	0.089	0.043	0.089

References:

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March 18, 2022

Knight Piésold
KRRP Project Office
4650 Business Center Drive
Fairfield, California
USA, 94534
www.knightpiesold.com

Mr. Erik Esparza
Design-Build Coordination Manager
Kiewit Infrastructure West Co.
4650 Business Center Drive
Fairfield, California
USA, 94534

Dear Erik,

RE: KRRP – IRON GATE TEMPORARY CONSTRUCTION ACCESS ROAD – DESIGN CRITERIA (DRAFT)

This letter is intended to outline the design criteria for the temporary construction access road to access Iron Gate dam site from the left bank upstream of the dam as part of the Klamath River Renewal Project. Following acceptance of the criteria herein, Knight Piésold's (KP's) road design sub-consultant, Geoserv, Inc., will proceed with a preliminary road stability analysis and preliminary design of the proposed alignment.

Much of the proposed alignment is currently underwater in the Iron Gate Reservoir. In addition, no subsurface investigation data were available at the time of the preliminary design. As a result, the slope stability analysis and road design will be preliminary and subject to change. Site conditions will be reviewed by the Engineer following reservoir drawdown and the design will be updated as required prior to road construction.

1.0 PRIMARY DESIGN OBJECTIVES

- Design stable <u>temporary</u> haul roads to access Iron Gate dam.
- Avoid any material entering the post drawdown wetted perimeter of the Klamath River.
- Maximize usable soil for use as construction access road fill.
- Locate spoil areas within the project limits to reduce labor and haul costs.
- Design will be California PE stamped by GeoServ, Inc.

2.0 DESIGN CRITERIA

2.1 GENERAL HAUL ROAD DESIGN CRITERIA

- Iron Gate Access Road shall be designed as a <u>temporary</u> structure (i.e., 2 - 3-year design life).
- Iron Gate Access Road is for Contractor use only during construction and will be left as-is following Project completion. Following Project completion, road access will be blocked or prevented via rock/earthfill berm or other approved methods. A posted permanent sign will be needed at project completion at the barrier indicating danger of rock fall and slope stability for pedestrians. Proceed at your own risk.
- Runaway vehicles are not considered in this design due to tight spatial constraints.
- The road design is based on site conditions, temporary haul road industry best practices, equipment specifications and Contractor inputs.

- Iron Gate Access Road maintenance of driving surfaces, and any daily or weekly maintenance required for continued compliance to the design geometry for the duration of use, will be the responsibility of the Contractor.
- Following a snow event - road shall be ploughed clear and if icy, adhesion shall be improved through placing cinder/salts etc. or as per Contractor's road maintenance plan.
- Slope hazard mitigation shall be managed by the Contractor.
- Temporary road signage will be as required by the Contractor.
- Best Management Practices (BMP's) will be implemented.
- On-site materials will be used for haul road construction.

2.2 VEHICLE LOADING

- The road is designed to accommodate the maximum imposed tire contact pressure due to a fully loaded CAT 745C haul vehicle, as defined by the Contractor.
- Other construction equipment may be used, provided the maximum applied loads do not exceed the fully loaded CAT 745C.
- Vehicle Load Factor (dynamic impact/braking etc.) = 1.25.

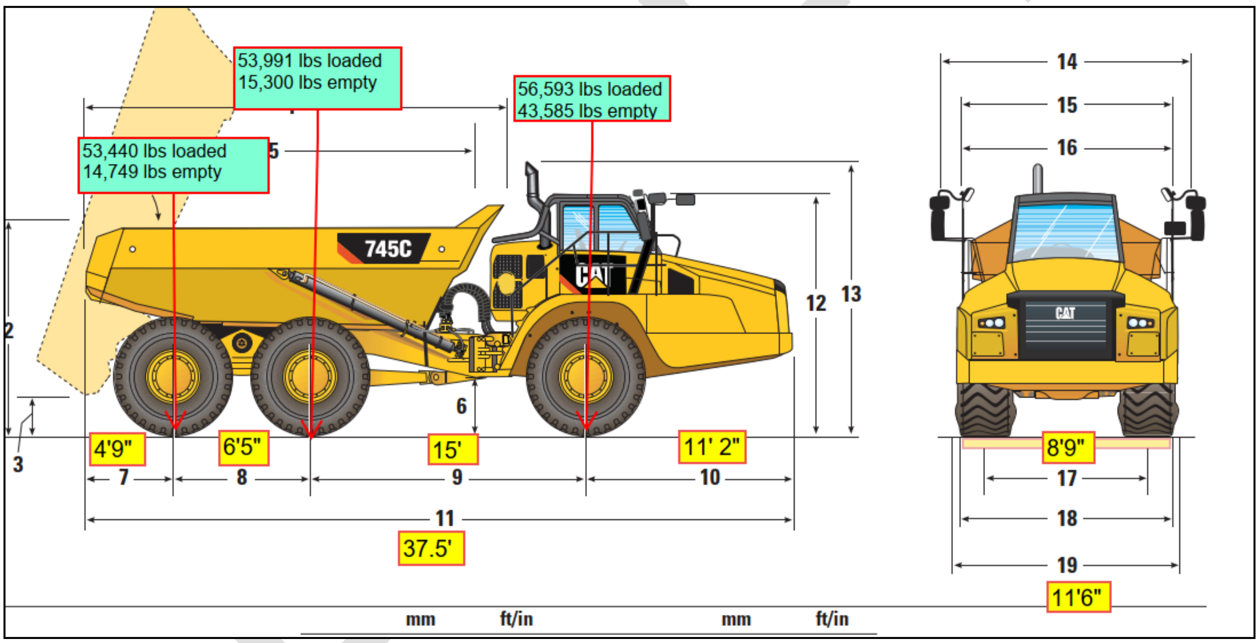

Figure 1 - CAT 745C Articulated Truck – Unfactored Nominal Axle Loads
[(CAT Product Specifications Sheet)]

Knight Piésold CONSULTING

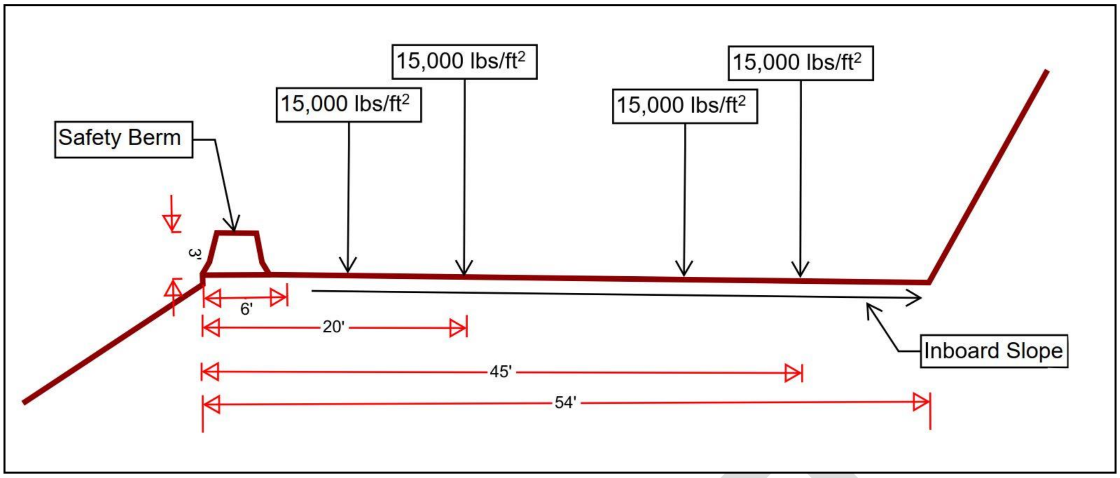

Figure 2 - 2D Slope Section, CAT 745C Axle Loading (based on tire contact pressure)

2.3 ROAD DESIGN CRITERIA

- 54 ft wide road - two way traffic (double lane).
  - 6 ft wide x 3 ft high safety berm (gabion baskets or earthfill/rock berm - TBD by Contractor).
  - 48 ft wide driving surface.
- Safety Berm is not designed for impact. Safety berm provides operators with visual/tactile feedback for vehicle alignment and road vehicular positioning.
- Maximum allowable road grade is 20%.
- Road will slope inwards from the outer edge to direct surface run-off. Drainage culverts and water management to be provided by the Contractor.
- Minimum allowable outside vehicle turning radius = 35 ft.

2.4 SLOPE STABILITY (GRANULAR MATERIALS)

- Includes all slopes comprised of loose/granular material (soils, gravel, rubble).
- Required factor of safety for limit equilibrium under static conditions will be 1.5.
- Inputs based on industry best practice for temporary haul roads (per MSHA and best practices for Forestry/Mining Haul Road applications). Actual soil and rock conditions to be field verified during construction.
- Peak Flood Events (considered for toe stability of sidecast material zones).
  - Iron Gate Flood Event (1% Probable Flood, Post Drawdown WSL) EL. 2,190 ft
  - Freeboard Assumed for long term slope stability 2 ft
- Seismic Peak Ground Acceleration (100 Year Return Period) 0.036g
- Sidecast spoiled material - slope stability factor of safety (FOS) 1.05

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3.0 CLOSING

We trust the information contained herein meets your needs at this time.

Please do not hesitate to contact any of the undersigned if you have any questions or comments.

Yours truly,
Knight Piésold

Prepared:	Craig Nistor	Reviewed:	Norm Bishop

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

Copy To: Nick Drury, Kiewit Infrastructure West Co.

DRAWDOWN MODEL REPORT
FOR THE KLAMATH RIVER RENEWAL PROJECT

100% DESIGN REPORT

Prepared for:

Knight Piésold
Fairfield, California

On behalf of:

The Klamath River Renewal Corporation
Berkeley, California

Prepared by:

Northwest Hydraulic Consultants Inc.
Seattle, Washington

March 10, 2022

NHC Ref. No. 2004947

DISCLAIMER

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CREDITS AND ACKNOWLEDGEMENTS

Northwest Hydraulic Consultants staff who contributed to this effort include:

• Nayyar Minhaj Asif	Engineer
• Natalia Leon Barrios	Engineer
• Jaron Brown	Engineer
• Jessica Lammers	Engineer
• Jeremy Payne	Engineer
• Kayla Kassa	Junior Engineer
• Alex Anderson	Senior Engineer
• Carla Bracken	Senior Technical Editor
• Todd Bennett	Principal
• Nancy Sims	Principal
• Pepe Vasquez	Principal

Report Reviewed by:

Erik Rowland, State of Oregon, P.E.
Principal

Henry Fehlman, State of California,
P.E. Principal

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APPENDICES

1 INTRODUCTION

As part of the Klamath River Renewal Project (KRRP), a one-dimensional (1D) Hydraulic Engineering Center River Analysis System (HEC-RAS) hydraulic model (HEC, 2019) has been developed to assess the reservoir hydraulics during the drawdown of J.C. Boyle, Copco No. 1, Copco No. 2, and Iron Gate Reservoirs on the Klamath River, located in Oregon and California. The model was developed using combined LiDAR, bathymetric surveys and a Digital Elevation Model (DEM) of the Klamath River (GMA, 2018). Inflows to the model are from the 2019 Biological Opinion (BiOp) flows (USBR, 2018).

Under contract to Knight Piésold (KP), Northwest Hydraulic Consultants Inc. (NHC) was tasked with developing a HEC-RAS model (HEC, 2019) (USACE, 2019).

1.1 Scope of Work

The primary purposes of the drawdown model and this report are to present simulated reservoir water surface elevations (WSEs) for the four reservoirs during the drawdown year as they relate to drawdown operations. These assessments were performed under a wide range of flow conditions to provide an assessment of the magnitude and timing of expected reservoir WSEs, inflows, and outflows. The proposed drawdown operations for each facility were evaluated using the HEC-RAS model for various flow conditions that could occur during the drawdown period. The entire 36-year record (October 1980 to September 2016) of daily average BiOp flows were used in the drawdown model.

The HEC-RAS model initiates drawdown for all the facilities on January 1 of the drawdown year. The HEC-RAS model simulates inflows, outflows and reservoir WSE through the drawdown period and for the post-drawdown period, prior to the final dam breach and establishment of the volitional fish channels. The pre-drawdown period, which is the period wherein temporary access, and dam and tunnel modifications are constructed, are not included in the HEC-RAS model.

1.2 Vertical Datum

All elevations in this report are relative to the National American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

2 MODEL DEVELOPMENT

2.1 General

Three separate HEC-RAS models were used to simulate drawdown and operation of the reservoirs during drawdown for J.C. Boyle reservoir, Copco No. 1 and No. 2 reservoirs (the two Copco facilities are combined in one HEC-RAS model), and Iron Gate reservoir. The extent of each model domain and cross-section locations are shown on Figure 1. The outflow from the upstream facilities was used as the inflow into the next downstream reservoir (e.g. outflow from J.C. Boyle model is the inflow into Copco Lake).

HEC-RAS model cross-sections are based on the topobathymetric data (GMA, 2018) and reach lengths (i.e. the distance between HEC-RAS model cross-sections) were defined to represent, as best possible, a range of storage and conveyance conditions for both high reservoir stage and low-flow immediately after drawdown. Section 2.4 discusses checks of the reservoir volume between the HEC-RAS model (the hydraulic model volumes are based on reach length and cross-section area) and the bathymetric data. The low flow channel is estimated based on the 2018 bathymetric data and represents a very approximate riverine condition, though that is considered sufficient for this drawdown study work. As discussed in Section 2.3.1, the DEM was modified near the Copco No. 1 Historic Diversion Tunnel.

2.2 Hydraulic Model Inflows, Local Inflows, and Downstream Boundary Assumptions

Daily average 2019 BiOp flows, from October 1980 through September 2016, were provided at the USGS station Klamath River at Keno, Oregon (USGS 11509500), and at the USGS station Klamath River below Iron Gate Dam, California (USGS 11516530) (USBR, 2018). These flows were applied for the simulations discussed in this report. The Keno flow was specified as the HEC-RAS model inflow into the riverine reach upstream of the J.C. Boyle reservoir. Local inflow was determined based on the difference between the Keno and Iron Gate BiOp flows. These local inflows were applied to the HEC-RAS model, with each reach of the study area receiving a share of the inflows proportional to the approximate local drainage area within that reach. Based on this method, the difference between Keno and Iron Gate BiOp flows were applied as follows; 20 percent to the J.C. Boyle Dam reach, 30 percent to the Copco reach, and 40 percent to the Iron Gate Dam reach. The remaining 10 percent of the local inflow enters downstream of Iron Gate Dam. The time distribution of this local inflow volume was assumed to follow that at Keno. The downstream boundary for each model was assumed as the normal depth of the average downstream slope.

2.3 Digital Elevation Model and Structure Elevation Data

Table 1 lists elevations (El.) for key structure features used in the HEC-RAS model. Figure 2 through Figure 4 show profile views of the dam and reservoir portions of the HEC-RAS model for each of the dams with the elevations of relevant dam features.

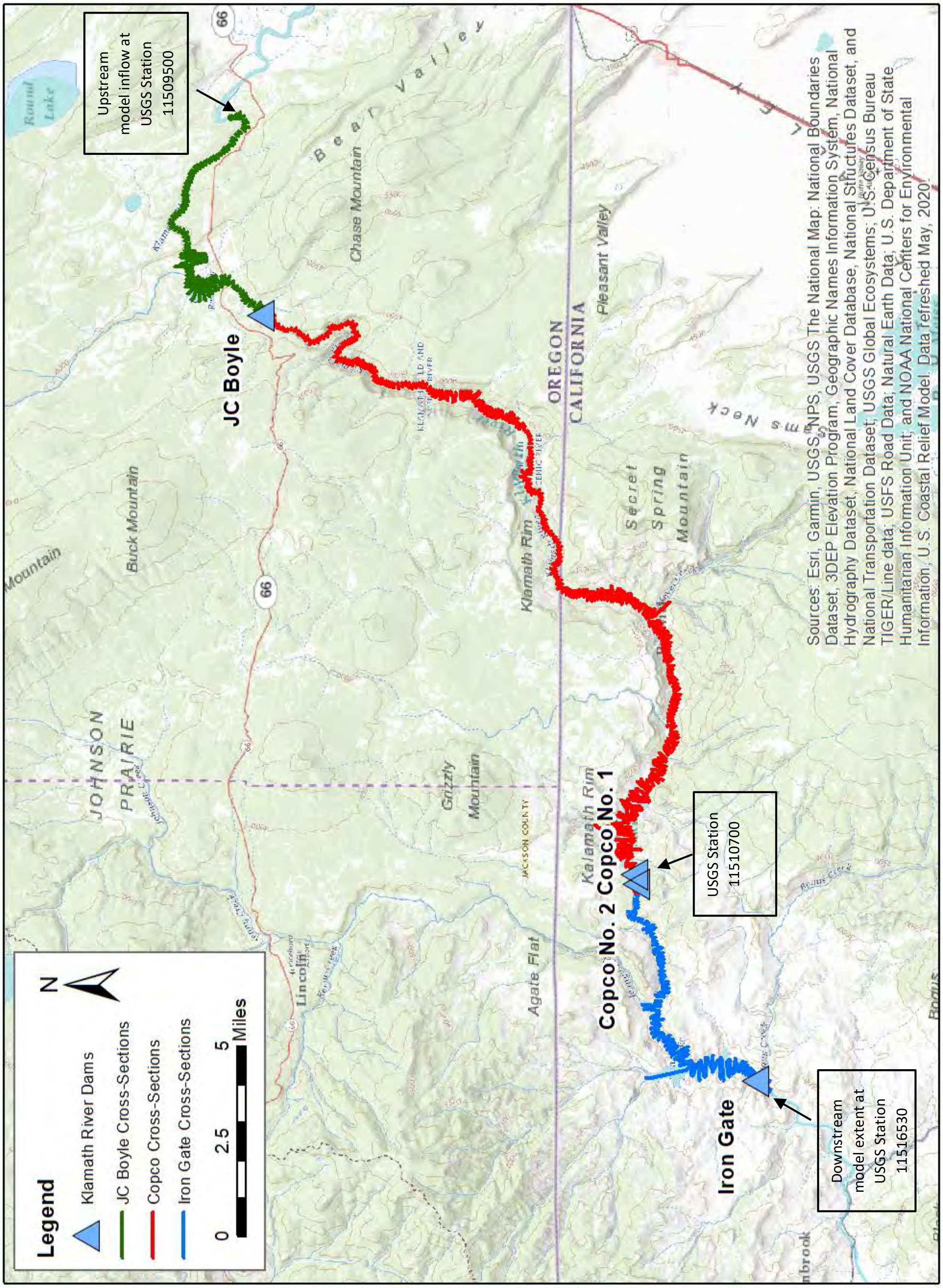

Figure 1. Vicinity Map.

Table 1. Dam feature elevations.

Dam	Dam Feature	Elevation (feet)
J.C. Boyle	Dam Crest	3803.7
	Spillway Crest	3785.2
	Power Intake Invert	3771.7
	Historic Cofferdam Crest	3770
	Diversion Culvert #1 Invert	3755.2
	Diversion Culvert #2 Invert	3755.2
Copco No. 1	Dam Crest	2616.5
	Spillway Crest	2597.1
	Historic Cofferdam Crest	2515
	Historic Diversion Tunnel Invert	2495
	Low-Level Outlet Invert (to be added prior to reservoir drawdown)*	2492.5 inlet 2477.3 outlet
Copco No. 2	Dam Crest	2496.5
	Spillway Crest	2476.5
	Spillway Bay No. 1 Invert (post dam removal elevation)*	2459.5
Iron Gate	Dam Sheet Pile	2351.3
	Dam Crest	2346.3
	Spillway Crest	2331.3
	Historic Cofferdam Crest	2212.0
	Historic Diversion Tunnel Outlet Invert	2178.3

**Notes a change from existing conditions*

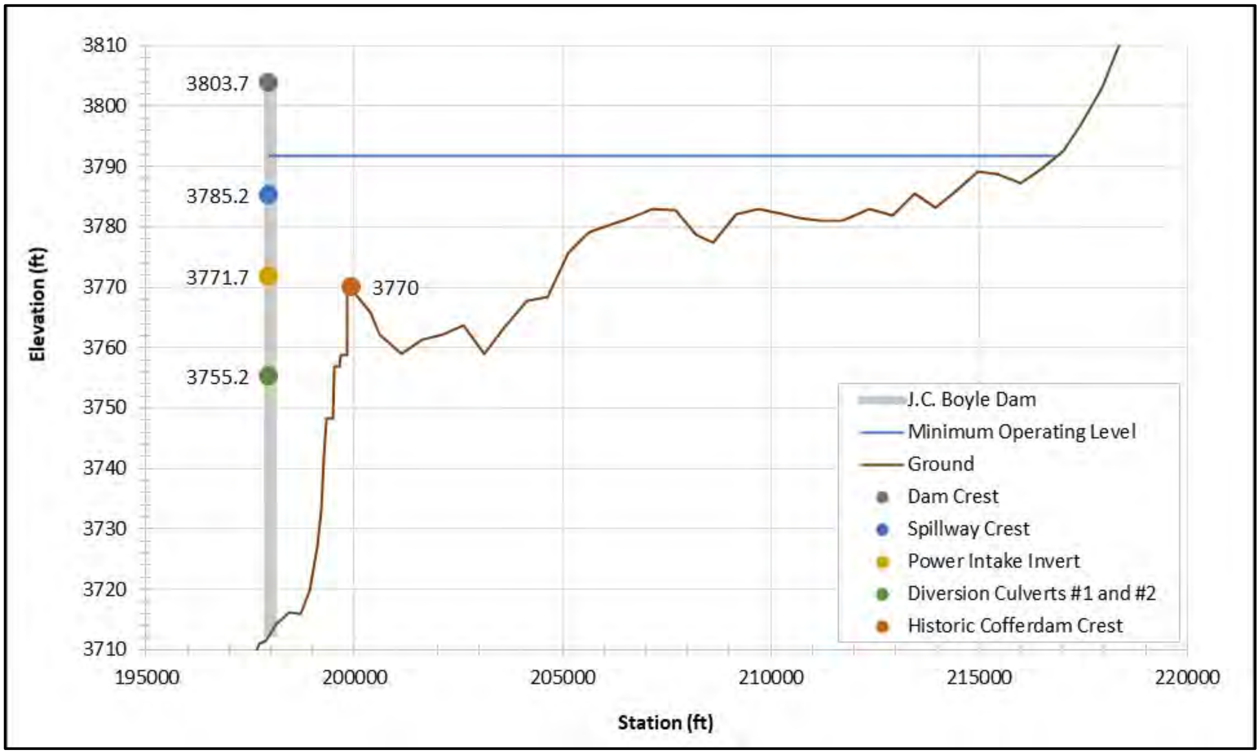

Figure 2. Profile of the dam and reservoir portion of J.C. Boyle HEC-RAS hydraulic model.

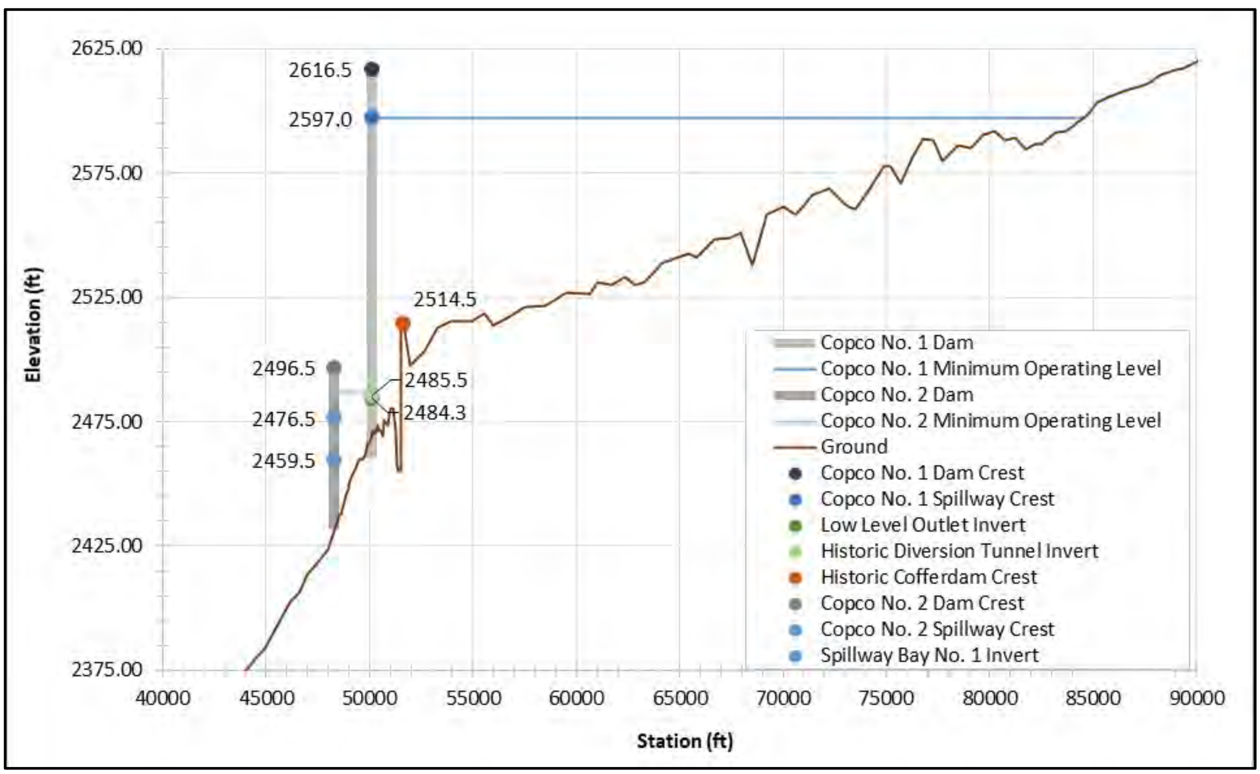

Figure 3. Profile of the dam and reservoir portion of Copco No. 1 and Copco No. 2 HEC-RAS hydraulic model.

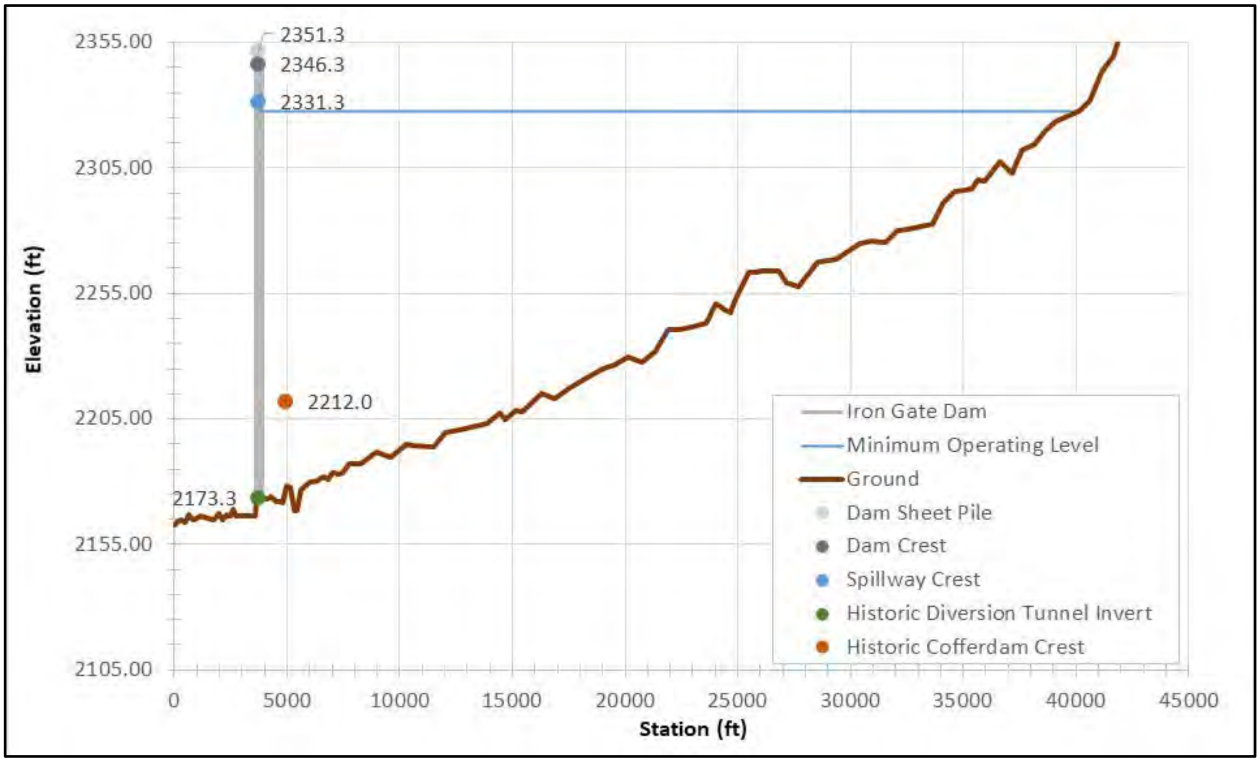

Figure 4. Profile of the dam and reservoir portion of Iron Gate HEC-RAS hydraulic model.

2.3.1 Digital Elevation Model Modifications

Sediment has accumulated near the entrance to the Copco No. 1 Historic Diversion Tunnel and the sediment will require excavation prior to the diversion tunnel opening. For the 100% design modeling, sediment was assumed to be removed to the Historic Diversion Tunnel intake, with approximately 1:1 side slopes. This represents a slight modification from the excavation plan as per KP Drawing C2120. However, this is not expected to impact the modeling results. No other modifications were made to the DEM.

2.4 Hydraulic Model Calibration and Validation

The model was validated to show that it can replicate observed reservoir stage. In a reservoir water balance, inflow plus change in reservoir storage equals outflow, and if these values are correct, then the hydraulic model should replicate observed stage.

Reservoir storage is a function of volume, and therefore the representation of the three main reservoirs within HEC-RAS (calculated up to the dam or spillway crest based on cross-section shape and the specified reach length between cross-sections) were compared to that of the topobathymetric data from approximately the spillway crest to at or near the historic coffer dam. Figure 5 shows this comparison (these plots are shown with the same volume scale for comparison purposes) and the values are generally within 10 percent when compared at 10-foot increments.

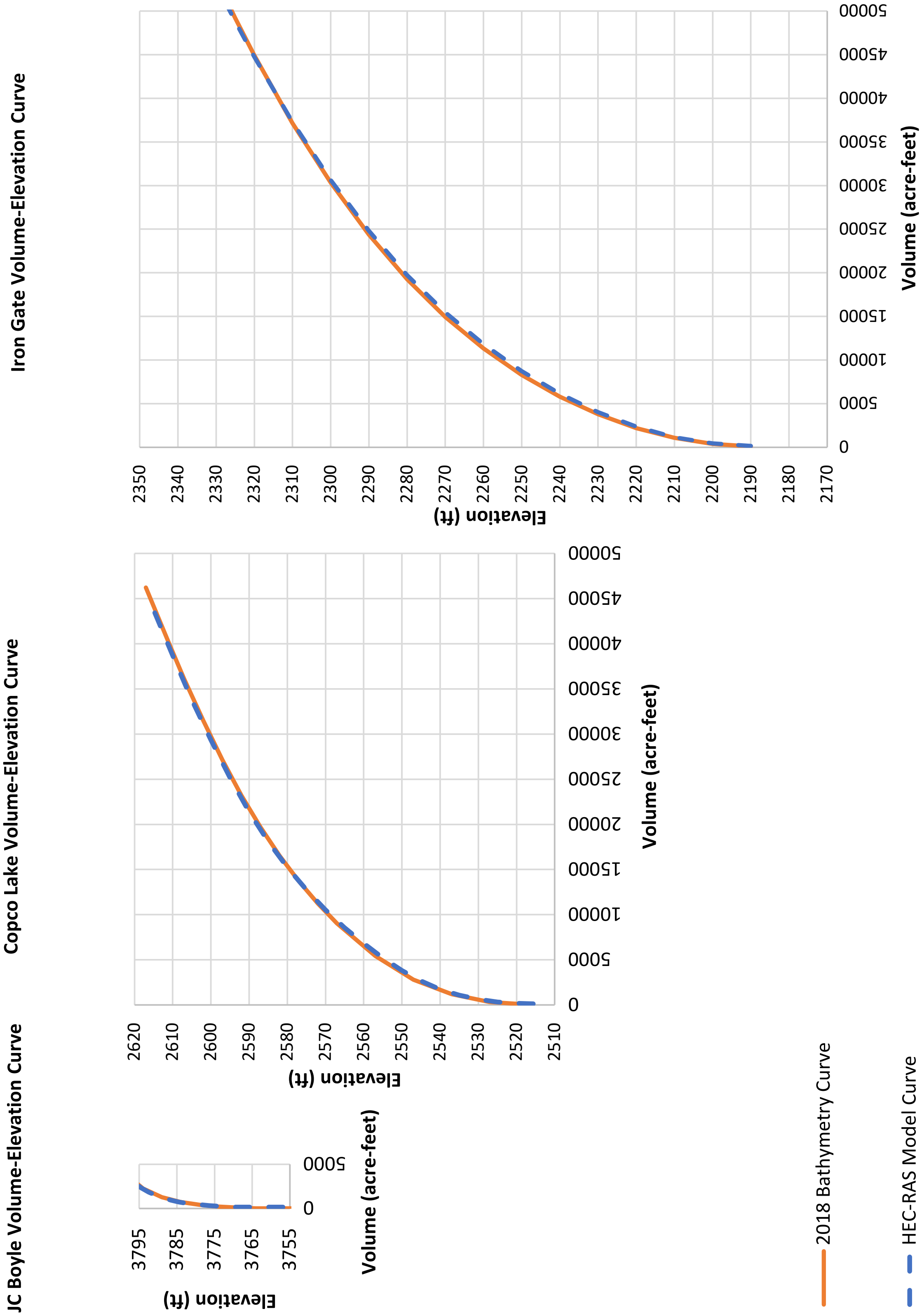

Figure 5. J.C. Boyle, Copco Lake and Iron Gate Reservoir HEC-RAS models volume (dashed blue) compared to topobathymetric data (solid orange).

In addition to storage, all inflows and outflows must be known or estimated to complete a water balance for the reservoirs. Gaged reservoir inflow and outflow data are available, however the local inflow between these points is also necessary to complete the water balance and evaluate the models' capability to replicate observed stage. As a proof of concept, the local inflows were roughly determined (Appendix A) to create an observed match between the simulated and observed stage within a portion of the normal operating pool range. Figure 6 to Figure 8 show for J.C. Boyle, Copco No. 1, and Iron Gate reservoirs, respectively, simulated stage with the estimated local inflow for a yearlong simulation at each reservoir. Root Mean Square Error for differences in simulated and gaged reservoir stage for these periods are 0.20, 0.16, and 0.18 for J.C. Boyle, Copco No.1, and Iron Gate reservoirs, respectively. Given the uncertainty of the local inflow, and that not knowing this accurately has a significant effect on the water balance, further evaluation was not conducted for this study.

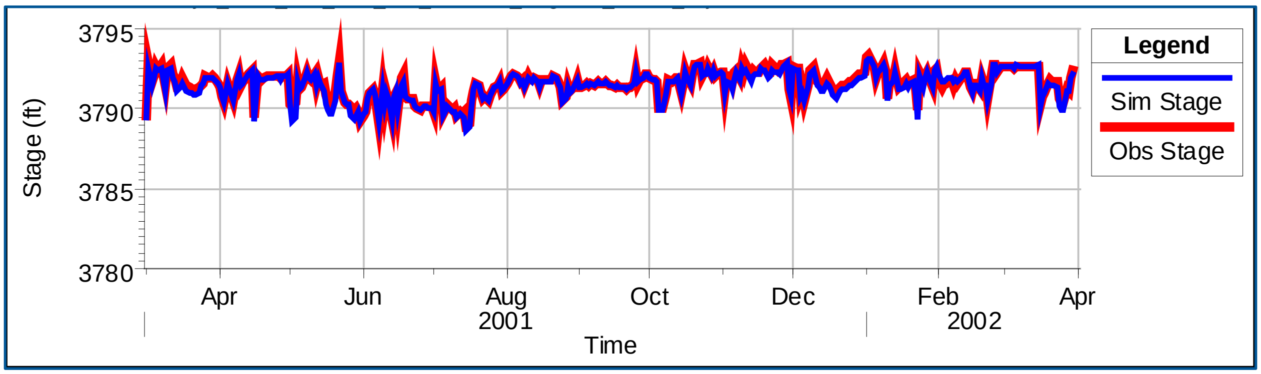

Figure 6. J.C. Boyle Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

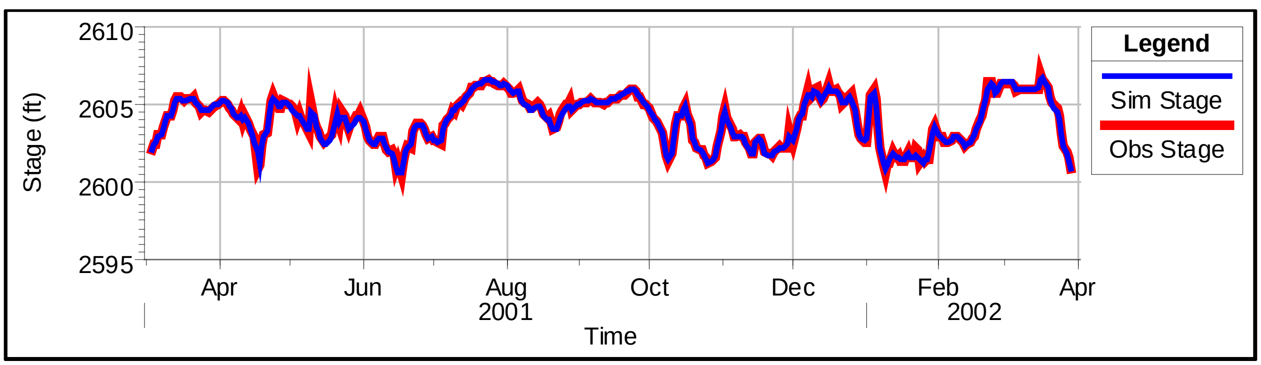

Figure 7. Copco No. 1 Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

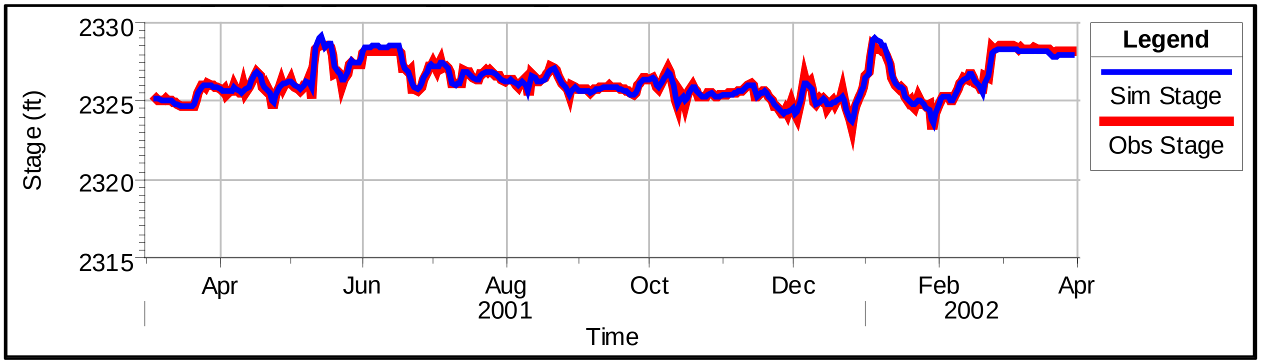

Figure 8. Iron Gate Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

The hydraulic model was calibrated to existing data for the riverine portions of the study area and also validated to show that it accurately simulates reservoir stage conditions within the range of normal pool operations. Figure 9 to Figure 11 show the different simulations run with Manning's roughness values of 0.04, 0.05, 0.06, and 0.07 to test model sensitivity for J.C. Boyle, Copco No.1, and Iron Gate, respectively. The local rating curve data from the three USGS gages were input as observed time series for each of the dams. The model became unstable for J.C. Boyle and Copco No. 1 simulations with a Manning's roughness value of 0.04. Manning's n was calibrated based on three USGS gaging stations within the study reach, and the value of n = 0.05 and n = 0.06 selected for the main channel and overbanks, respectively to match the best fit lines in Figure 9 to Figure 11 (Appendix A). Root Mean Square Error for differences in simulated and measured stage are 0.65, 0.35, and 0.40 for riverine sites upstream of J.C. Boyle, Copco No.1, and Iron Gate reservoirs, respectively. Simulated flows below 8,000 cfs are typically within one foot of measured USGS values. Figure 12 shows a time series example of simulated versus observed stage at USGS Station 11510700 upstream of the Copco No. 1 Reservoir for a range of flows. Sensitivity of model results to Manning's n is discussed in Section 2.5.

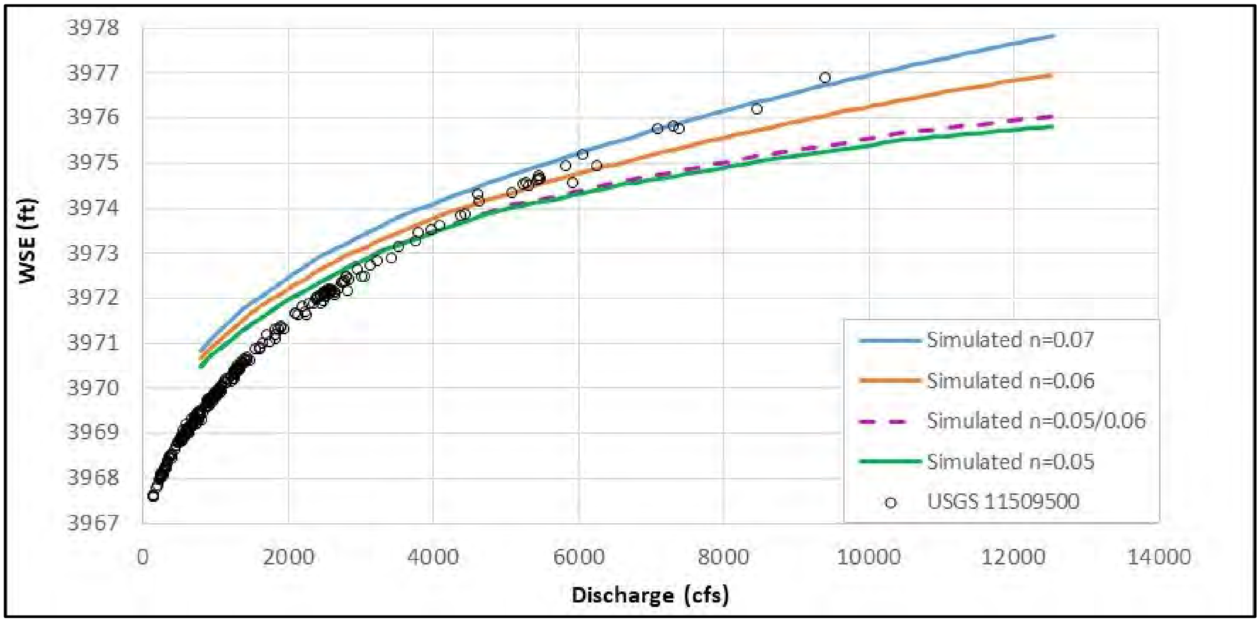

Figure 9. Simulated Stage Discharge Curves for a Range of Manning's n Values compared to Measured Values at a USGS Station 11509500 Upstream of the J.C. Boyle Reservoir

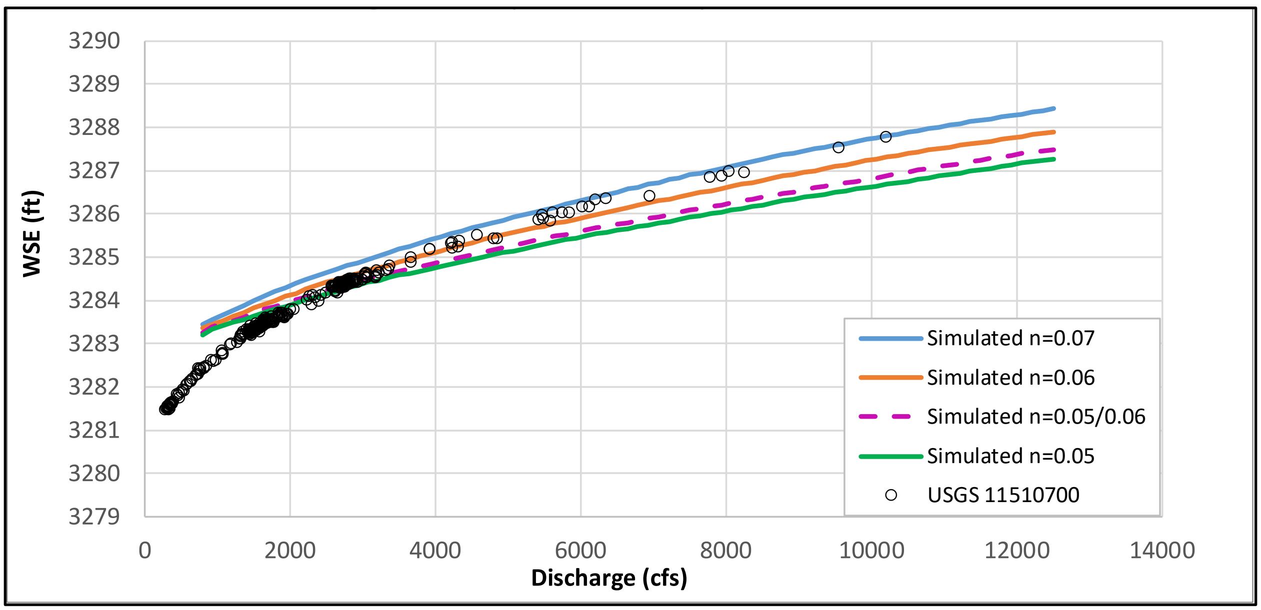

Figure 10. Simulated Stage Discharge Curves for a Range of Manning's n Values compared to Measured Values at a USGS Station 11510700 Upstream of the Copco No. 1 Reservoir

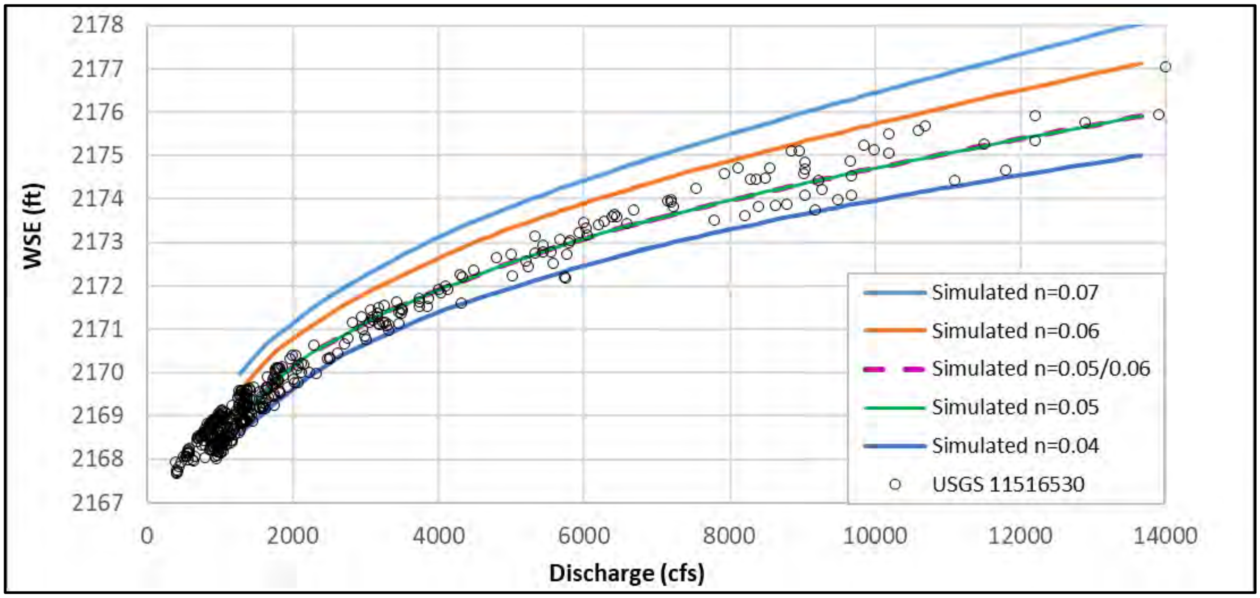

Figure 11. Simulated Stage Discharge Curves for a Range of Manning's n Values compared to Measured Values at a USGS Station 11516530 Downstream of the Iron Gate Reservoir

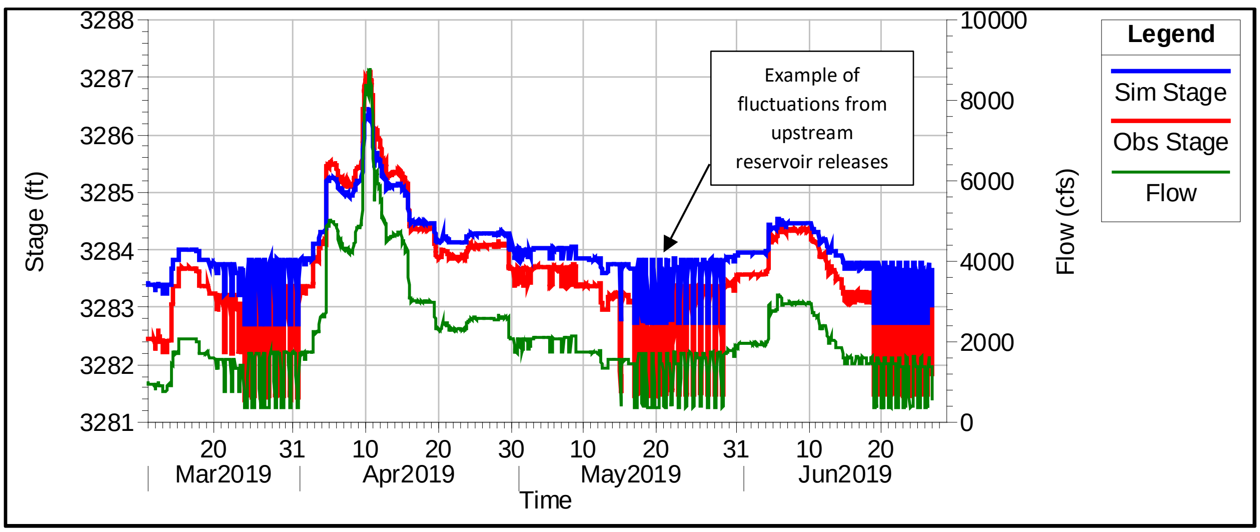

Figure 12. Example Simulated versus Observed Stage at a USGS Station 11510700 Upstream of the Copco No. 1 Reservoir

2.5 Sensitivity Analyses

Several sensitivity analyses were conducted to understand how adjustments to model parameters would affect the simulated results. When results show a high variability to modifying a specific model parameter, then additional attention should be made in selecting an appropriate value for that parameter. Sensitivity analyses evaluated varying Manning's n through the unregulated, riverine portions of the hydraulic model, and through the reservoirs, both with and without dams in place (the latter investigating sensitivity to the simulated drawdown condition) as well as varying the computational time step, and the effect of varying the output time step on downstream model results (where the upstream model output is used for input into the downstream model). Additional simulations have been conducted to evaluate different Manning's n values as well as other model parameters for all three reservoirs as discussed in the NHC technical memo on model sensitivity (Appendix A). These tests included:

- Removing the dams and testing the sensitivity of travel time, attenuation and stage to varying n (in 0.01 increments from 0.04 to 0.07) – to see how the Manning's n values affect simulation results during the drawdown condition.
- Through the riverine portion only (with dams in place) testing sensitivity of travel time, attenuation and stage to varying n (0.01 increments from 0.04 to 0.07).
- Through the reservoir portion only (with dams in place) testing sensitivity of travel time, attenuation and stage to varying n (n values of 0.03, 0.05 and 0.07).
- Varying the computational timestep (5, 15, 30 and 60 second timesteps).
- Varying reservoir volume by +/- 10%.

None of these had a significant effect on models results, with difference in water surface of typically less than a foot and time difference of less than an hour.

2.6 Hydraulic Modeling of Dam Structure Operations During Drawdown and Post-Drawdown

2.6.1 General

"Rules" are used in the hydraulic modeling to specify outflow from the dams through the various outlet structures. Computational fluid dynamics (CFD) methods were used to determine rating curves for the outlet structures at all four dams (NHC, 2020b; NHC, 2020c; NHC, 2020d; NHC, 2020e), and then the HEC-RAS rules were used to dictate when a specific outlet structure is active based on the specified drawdown operating criteria presented in the 90% Design Report (KP, 2020a). Further, at Iron Gate, the rating curve of the existing diversion tunnel was updated as per KP (2021) following a tunnel survey completed by Yurok Tribe between November 17 and November 20, 2020. For the simulations, all reservoirs are assumed lowered to their minimum operating levels and starting at that level when simulated drawdown begins on January 1 of each drawdown year.

2.6.2 J.C. Boyle

The drawdown of the J.C. Boyle reservoir will utilize the spillway, power intake, and two low-level diversion culverts. The drawdown operations specified in the HEC-RAS model for J.C. Boyle are as follows:

- Stage 1 – Drawdown using spillway gates:
  - Initial WSE is the minimum operating level (El. 3791.7 feet).

- Drawdown is initiated on January 1 and is regulated using the spillway gates at a target rate of 5 feet/day.

- Stage 2 – Drawdown using power intake to lower the reservoir levels to below the spillway crest:
  - The power intake opens on January 2. Flow through the power intake is not regulated.
- Stage 3 – Opening of Diversion Culvert #1:
  - Diversion Culvert #1 opens once the reservoir WSE is at or below El. 3783.2 feet (which is 2 feet below the spillway crest) for a period of 24 hours.
  - No operational controls exist for the culvert.
  - The power intake permanently closes when Diversion Culvert #1 is opened. Once the power intake is closed, it remains closed.
- Stage 4 – Opening of Diversion Culvert #2:
  - Diversion Culvert #2 is delayed until after the freshet.
  - Diversion Culvert #2 opens on or after June 10 and at a reservoir WSE at or below El. 3783.2 feet (which is 2.0 ft below the invert of the spillway crest) for a period of 24 hours.
  - No operational controls exist for the culvert.

The rating curves used in the HEC-RAS model for the J.C. Boyle facility are shown on Figure 13. The rating curve was developed using computational fluid dynamics (CFD) by NHC (2020b).

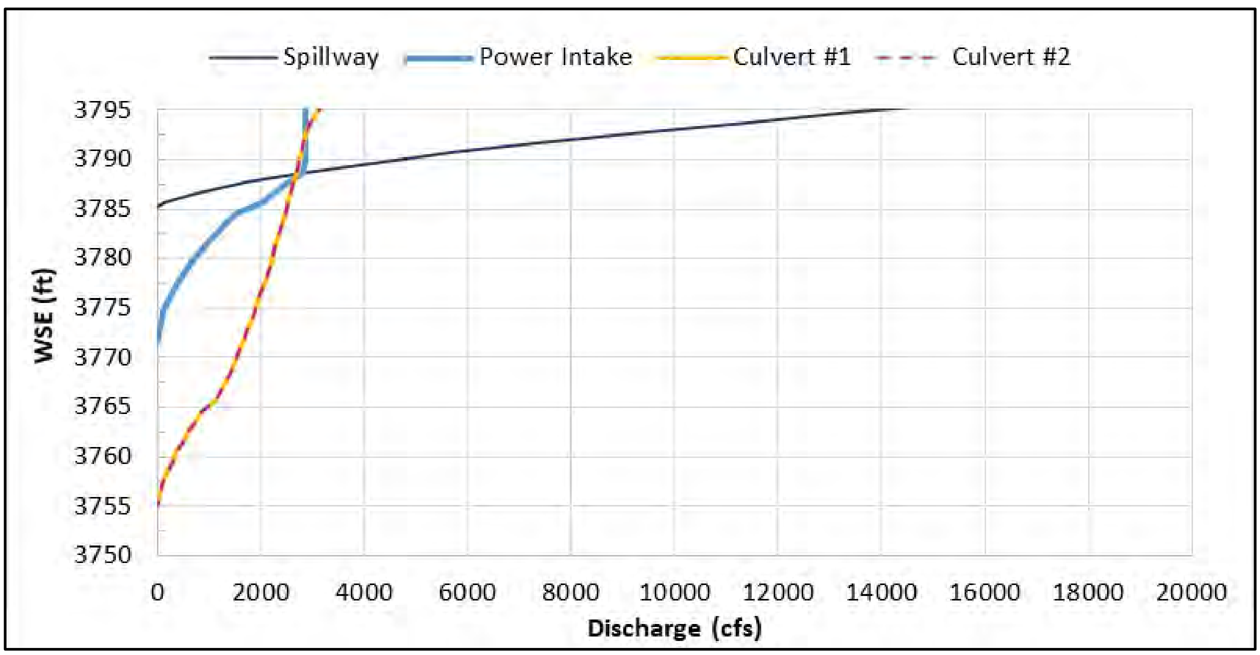

Figure 13. Stage versus flow relationships at J.C. Boyle Dam for simulating outflow at the dam.

2.6.3 Copco No. 1

The drawdown of the Copco No. 1 reservoir will be completed utilizing the spillway and by constructing a new low-level outlet with a 10.0 feet orifice inlet diameter and a 10.5 feet by 15 feet "D" shaped tunnel with a 10.5 feet diameter steel pipe at the outlet of the low-level outlet. The historic diversion tunnel will be used to further lower the water level in the reservoir after the majority of drawdown has occurred. The drawdown operations specified in the HEC-RAS model for Copco No.1 are as follows:

- Drawdown Phase – Opening of new low-level outlet:
  - Initial WSE is at the crest of the spillway (El. 2597 feet).
  - Drawdown is initiated on January 1 when the low-level outlet is opened.
  - The pre-drawdown phase is not included in the HEC-RAS model.
  - No powerhouse flows are included in the HEC-RAS model.
- Diversion Stage – Opening of historic diversion tunnel:
  - The historic diversion tunnel opens after June 15 of the drawdown year and once the reservoir WSE is at or below 2530 feet, which is approximately 20 feet above the top of the existing intake structure. Initially only a 5-foot opening is assumed, and once the water level drops below El. 2516 feet, then an 18 feet opening is assumed. The model assumes the historic diversion tunnel is opened instantaneously between these opening heights.

The rating curves used in the HEC-RAS model for the Copco No. 1 facility are shown on Figure 14. They were developed using computational fluid dynamics (CFD) by NHC (2020c).

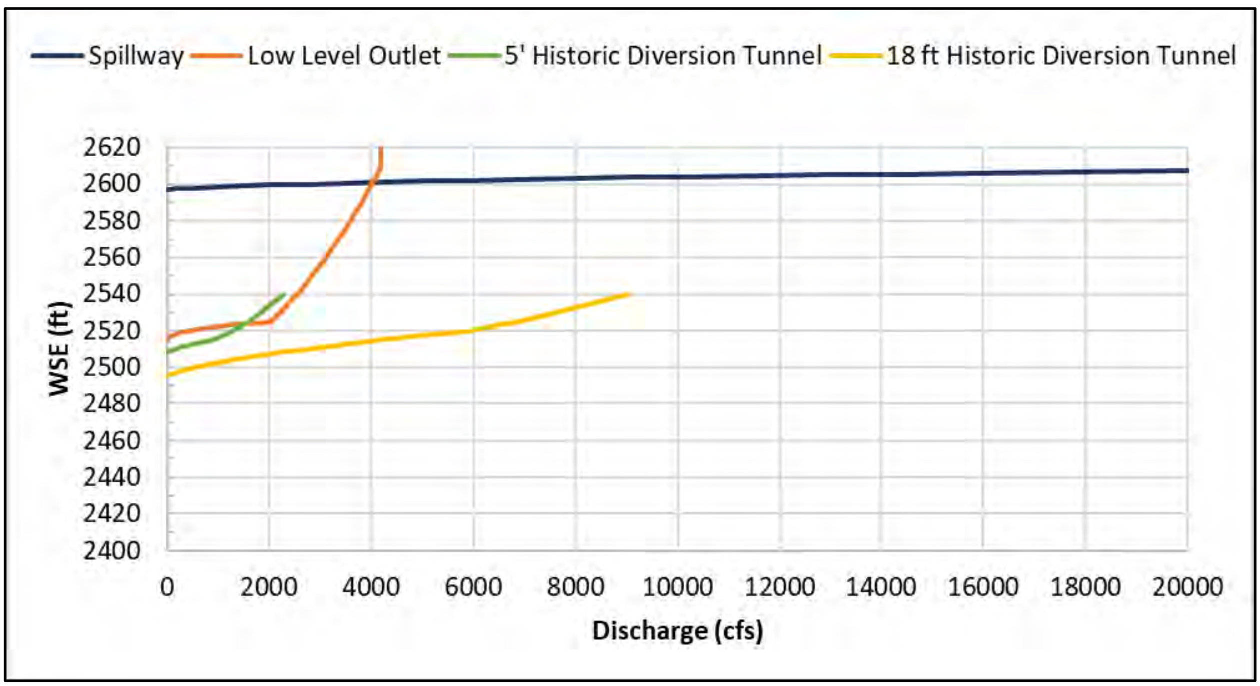

Figure 14. Stage versus flow relationships at Copco No. 1 Dam.

2.6.4 Copco No. 2

The drawdown of the Copco No. 2 reservoir will be completed by opening the exiting spillway gates and removing the concrete plug at Spillway Bay No. 1. The drawdown operations specified in the HEC-RAS model for Copco No.2 are limited as follows:

- Drawdown Phase – Opening of the Spillway Bay No. 1:
  - Initial WSE is at the normal water level (El 2,486.5 ft).
  - Drawdown is initiated on January 1 when the concrete plug at Spillway Bay No. 1 is removed.
- Passing flow through the conveyance system to the powerhouse is not included in the HEC-RAS model.
- The pre-drawdown works, which involves fully opening the spillway gates to construct a temporary working platform downstream of the Spillway Bays No. 2 through No. 5 and the partial removal of the ogee for Spillway Bay No. 1, are not included in the HEC-RAS model. The lateral removal of the dam was not simulated for the post-drawdown and the same rating curve was applied for the entire simulation.

The rating curves used in the HEC-RAS model for the Copco No. 2 facility are shown on Figure 15. The rating curve was developed using computational fluid dynamics (CFD) by NHC (2020d).

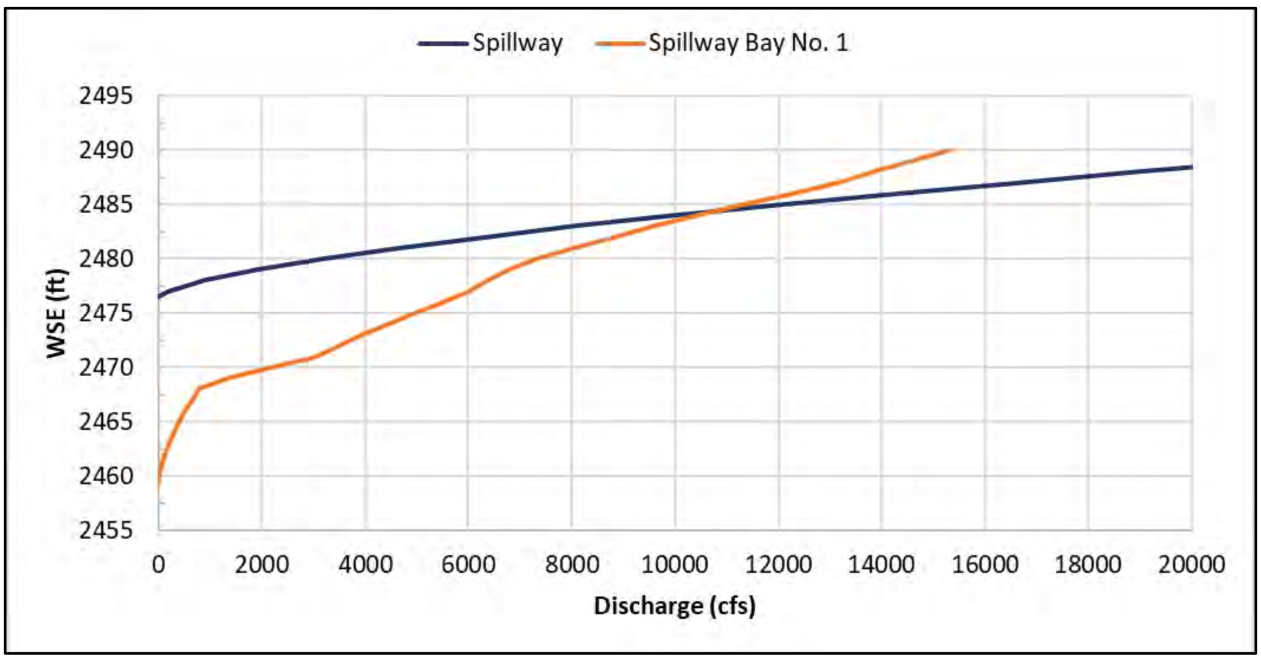

Figure 15. Stage versus flow relationships at Copco No. 2 Dam.

2.6.5 Iron Gate

The drawdown of the Iron Gate reservoir will utilize the spillway, power intake using hydraulic turbine or by-pass (Howell-Bunger valve), and existing diversion tunnel. The flow through the existing diversion tunnel will be controlled by the existing upper gate. The drawdown operations specified in the HEC-RAS model for Iron Gate are as follows:

- Drawdown Phase – Opening the existing upper gate in the diversion tunnel:
  - The initial WSE is at the minimum operating level (El. 2327.3 feet).
  - Drawdown is initiated on January 1 by fully opening the existing upper gate in the diversion tunnel (57 inches), and by opening the power intake and the bypass valve.

The rating curves used in the HEC-RAS model for the Iron Gate facility are shown on Figure 16. The rating curve was developed using computational fluid dynamics (CFD) by NHC (2020e) and KP (2021) following a tunnel survey completed by Yurok Tribe between November 17 and November 20, 2020.

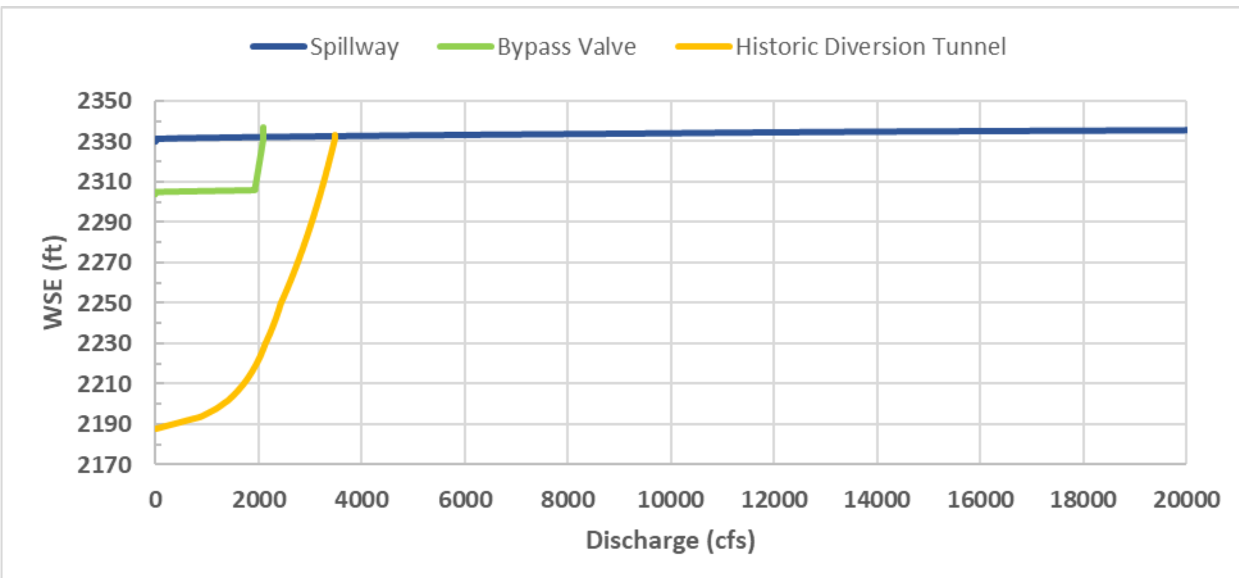

Figure 16. Stage versus flow relationships at Iron Gate Dam for simulating outflow at the dam.

3 SIMULATED RESERVOIR DRAWDOWN

The simulation results are described in detail in the following sections. The results highlight key elevation and time triggers for the hydraulic operational controls of the reservoirs for a variety of hydrologic conditions. All 36 simulation periods (1981 through 2016) were evaluated for each reservoir to ensure the efficacy and functionality of the proposed drawdown operations. Stage and flow plots for each simulation can be found in Appendices B through E. A drawdown plot for selected simulations is shown with text boxes helping to describe what is occurring in the simulation.

3.1 J.C. Boyle

3.1.1 Simulated Drawdown Results

Stage and flow results for select J.C. Boyle simulations are provided in Figure 17 to Figure 22 (these include simulation years 1987, 1993, 1997 and 2006 to show how the RAS "rules" operate under a range for flow conditions). The 1987 simulation is representative of typical hydrologic conditions based on BiOp flow volumes. The 1997 and 2006 simulations included extended periods of high flows and discharge, providing confirmation of certain proposed operational controls. The 1993 simulation provided a good example of peak flow attenuation between the four reservoirs. Both elevation controls (e.g. Culvert #1 will not open until the water surface elevation is below El. 3783.2 feet), and time controls (e.g. the power intake will not open until January 2) are utilized for this reservoir.

A stage and flow profile plot for the full 1987 simulation is provided in Figure 17. A finer resolution profile showing the key operational triggers at the beginning of the simulation is provided in Figure 18. The simulation begins at the minimum operating level of El. 3791.7 feet. The drawdown over the spillway is 2.7 feet, which is less than the target drawdown rate of 5 feet per day. The power intake opens on January 2, dropping the WSE 9.2 feet over the next 24 hours. The WSE drops below 3783.2 feet on January 2 and stays below this elevation for a minimum of 24 hours. This results in Culvert #1 opening on January 3. Once Culvert #1 is open, the power intake is closed permanently and cannot be reopened.

Tests were also completed to ensure that Culvert #2 was operating correctly under a variety of hydrologic and hydraulic conditions. Figure 19 and Figure 20 provide a weeklong snapshot of the 1987 and 2006 simulations, respectively, when Culvert #2 is activated. In Figure 19, Culvert #1 is activated at a previous time step (see Figure 18), and the WSE remains below 3783.2 for the 24 hours preceding June 11. On June 11, Culvert #2 is activated and remains open along with Culvert #1 for the remainder of the simulation. Another possible scenario is when Culvert #1 and #2 open at the same time. In the 2006 simulation (Figure 20), the WSE drops below El. 3783.2 feet on June 10, and stays below this elevation for 24 hours, causing both Culverts #1 and #2 to open on June 11. This confirms that both the elevation and time operational controls are functioning correctly.

Figure 21 is a stage and flow profile plot for the full 1997 simulation. Similar to the 2006 simulation, the 1997 simulation is an example of extended high headwater conditions, requiring the power intake to remain open until the middle of April. The WSE drops below El. 3783.2 feet for the first time on April 13 and stays under this elevation for 24 hours. Culvert #1 is opened on April 14, at which time the power intake is closed permanently for the remainder of the simulation. The WSE remains below El. 3783.2 feet for 24 hours prior to June 11, allowing Culvert #2 to open on June 11.

As previously mentioned, the HEC-RAS models represent a series of reservoirs by simulating the outflow from one reservoir into the next downstream reservoir. This means that rapid increases in flow within one reservoir should be observed within a close time frame in the immediate downstream reservoir. To confirm that the reservoirs are acting in series, the outflows from all four reservoirs were plotted during a storm in the 1993 simulation (Figure 22). The J.C. Boyle peak is on March 26 at 13:52. The Copco No. 1 and No. 2 peaks are on March 27 at 05:05, and the Iron Gate peak is on March 28 at 10:23. The peak flow from J.C. Boyle is attenuated approximately 16 hours before reaching Copco No. 1. The distance between Copco No. 1 and No. 2, along with the small capacity of Copco No. 2, resulted in no attenuation between the two Copco reservoirs. The peak discharge between Copco No. 2 and Iron Gate is attenuated approximately 29 hours. Longer attenuation between these two reservoirs was anticipated based on the distance between them, and the increased storage capacity of the Iron Gate reservoir. Figure 22 confirms that the reservoirs are acting in series, and Figure 17 through Figure 21 confirm that the operations controls are functioning as designed.

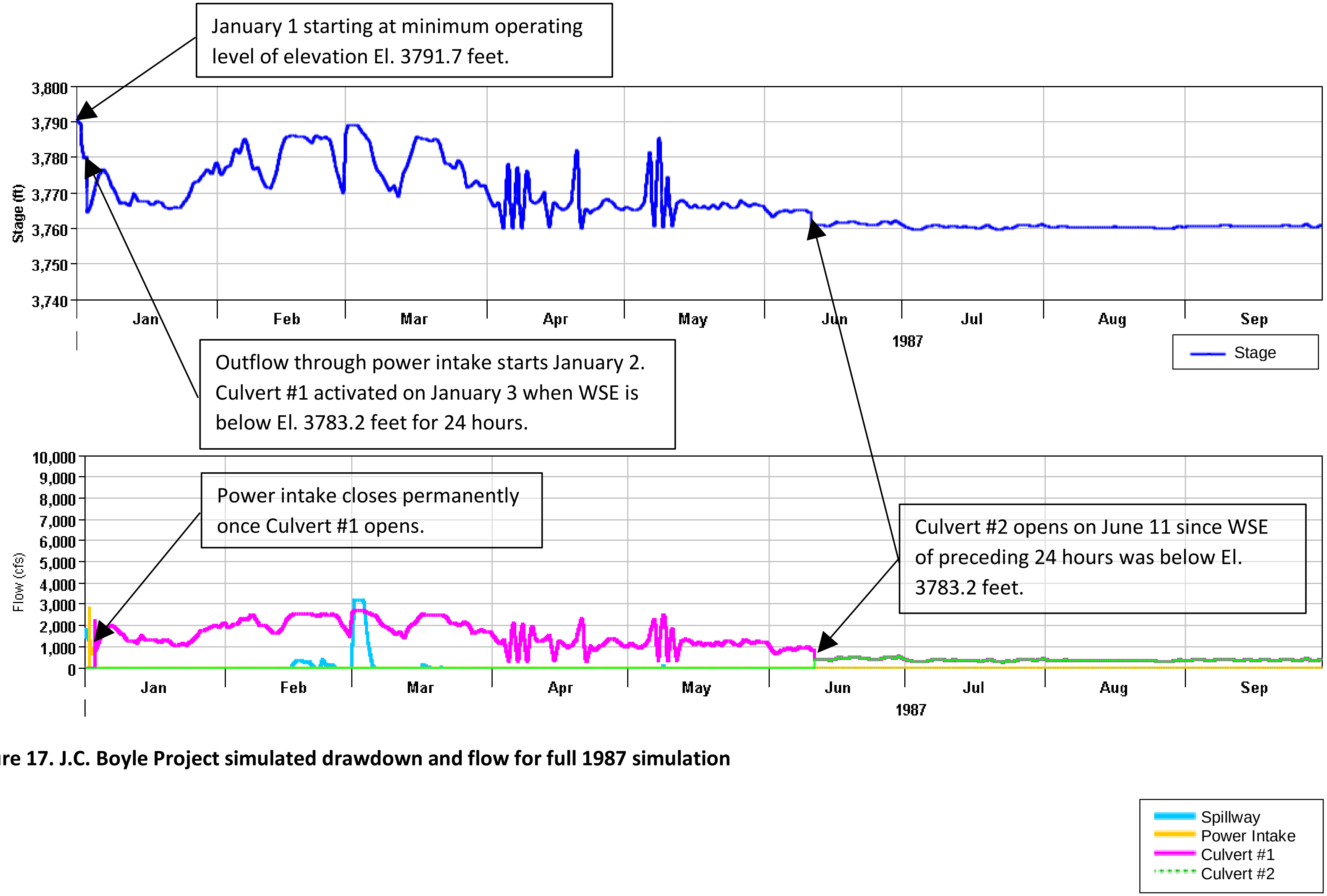

Figure 17. J.C. Boyle Project simulated drawdown and flow for full 1987 simulation

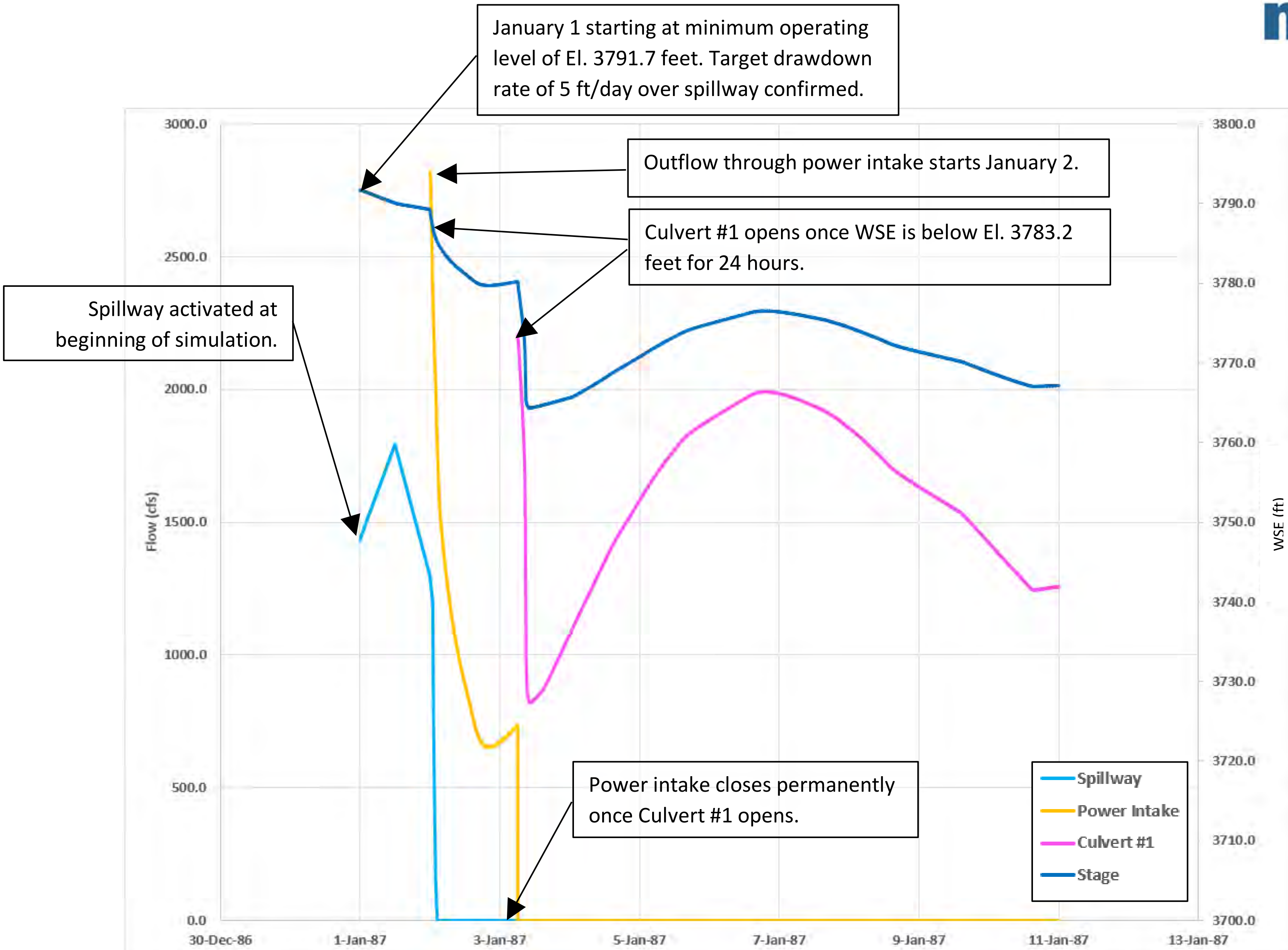

Figure 18. J.C. Boyle Project simulated drawdown and flow for finer resolution 1987 simulation.

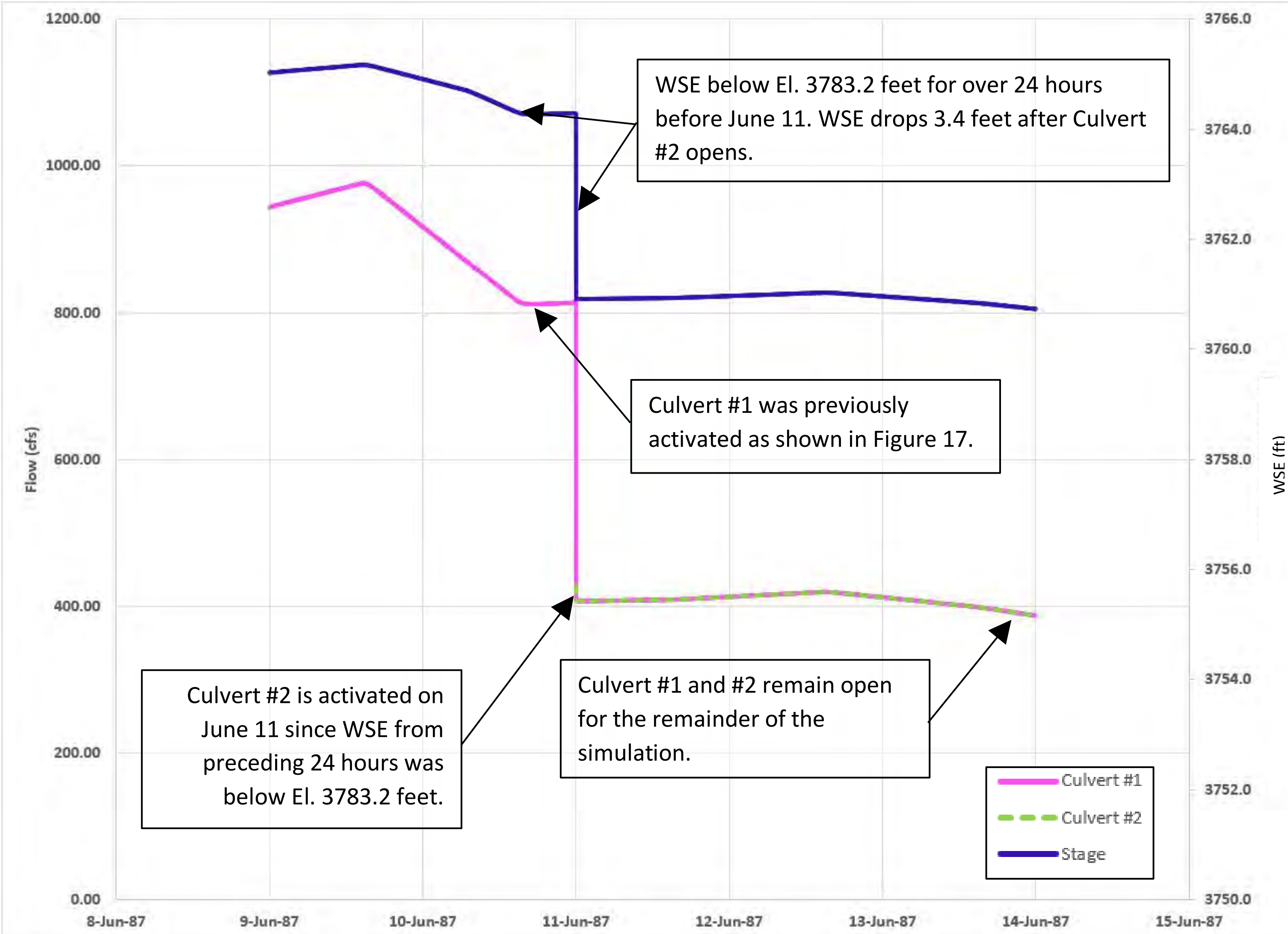

Figure 19. J.C. Boyle Project simulated drawdown and flow for Culvert #2 activation for 1987 simulation.

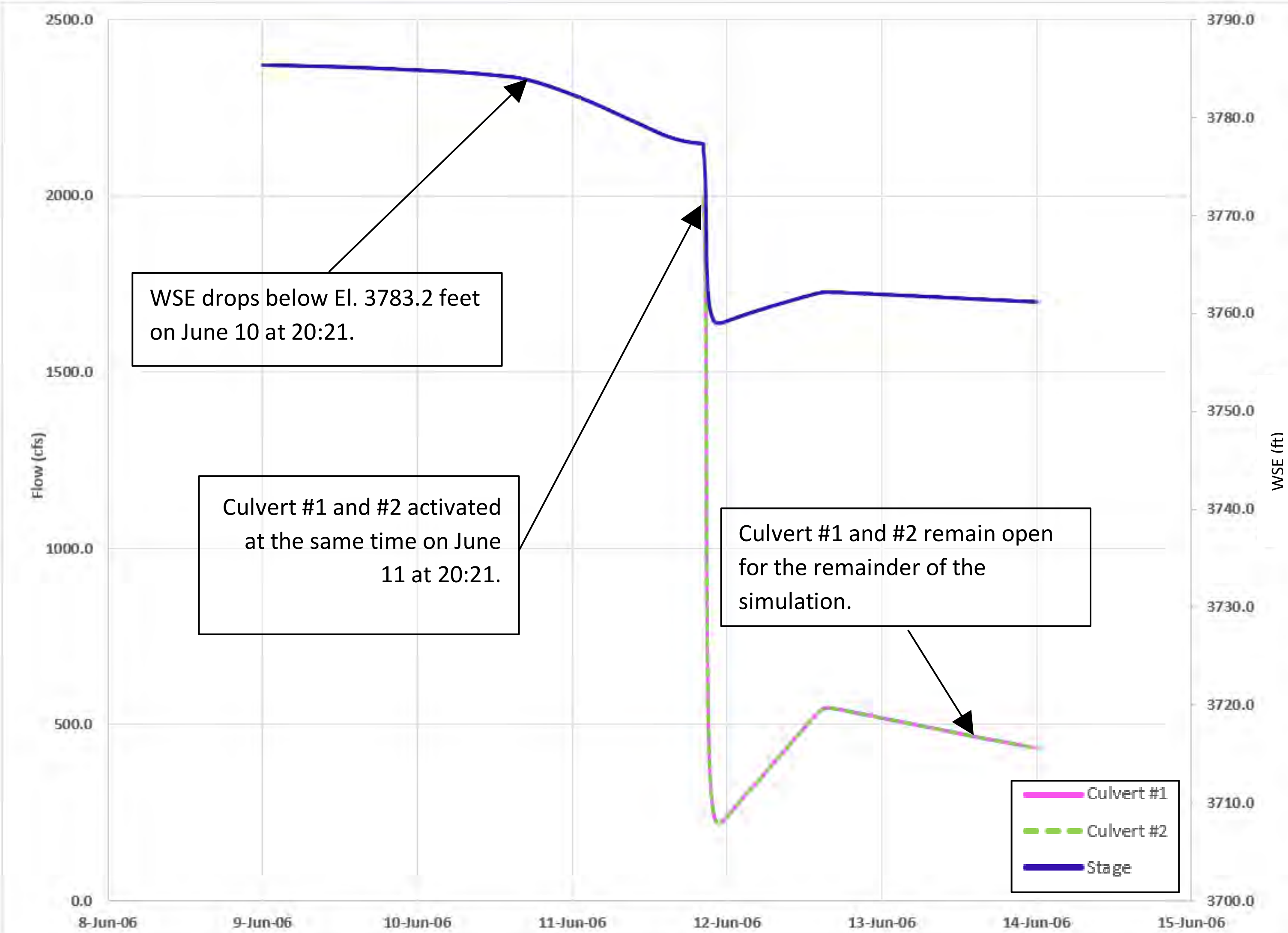

Figure 20. J.C. Boyle Project simulated drawdown and flow for Culvert #2 activation for 2006 simulation.

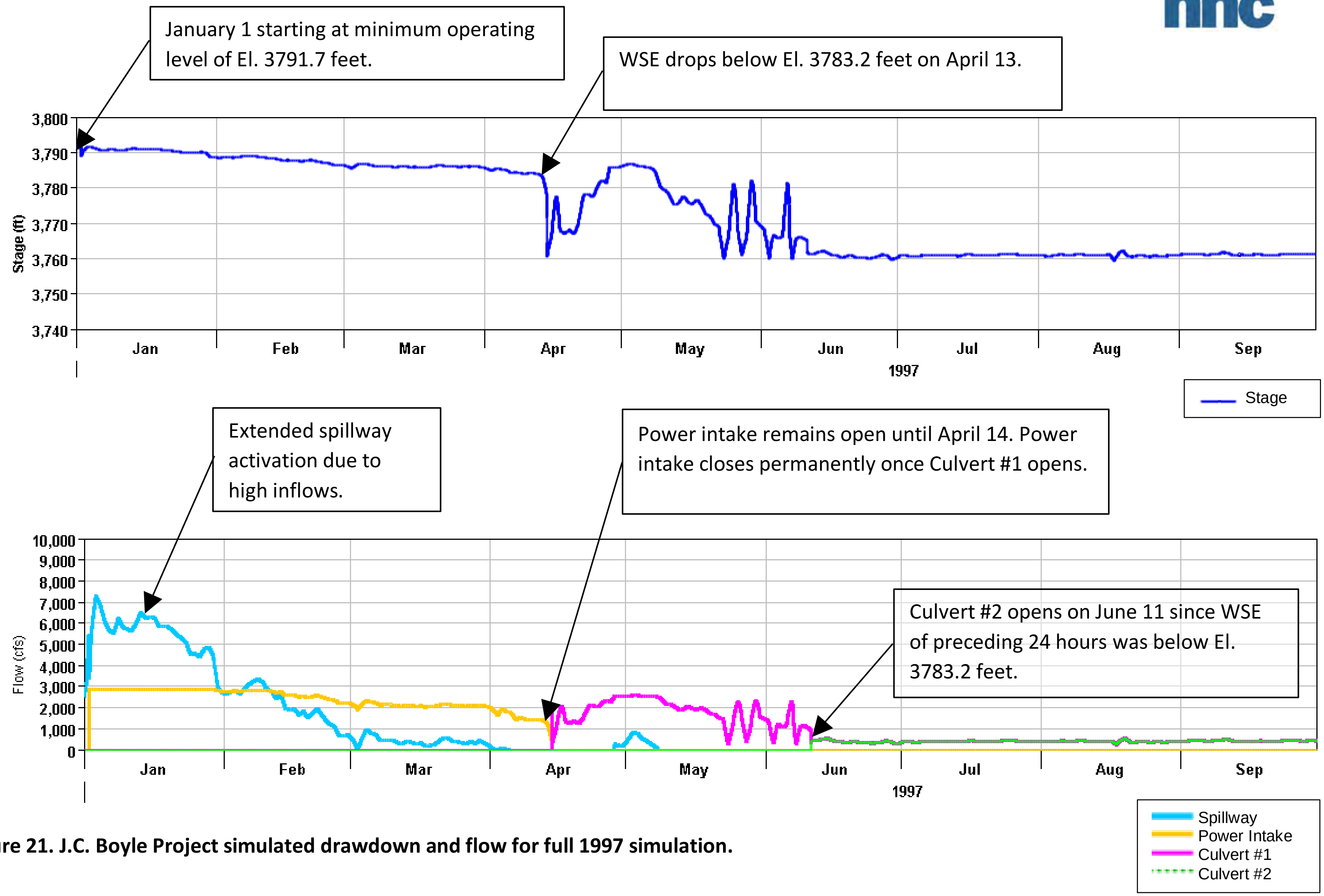

Figure 21. J.C. Boyle Project simulated drawdown and flow for full 1997 simulation.

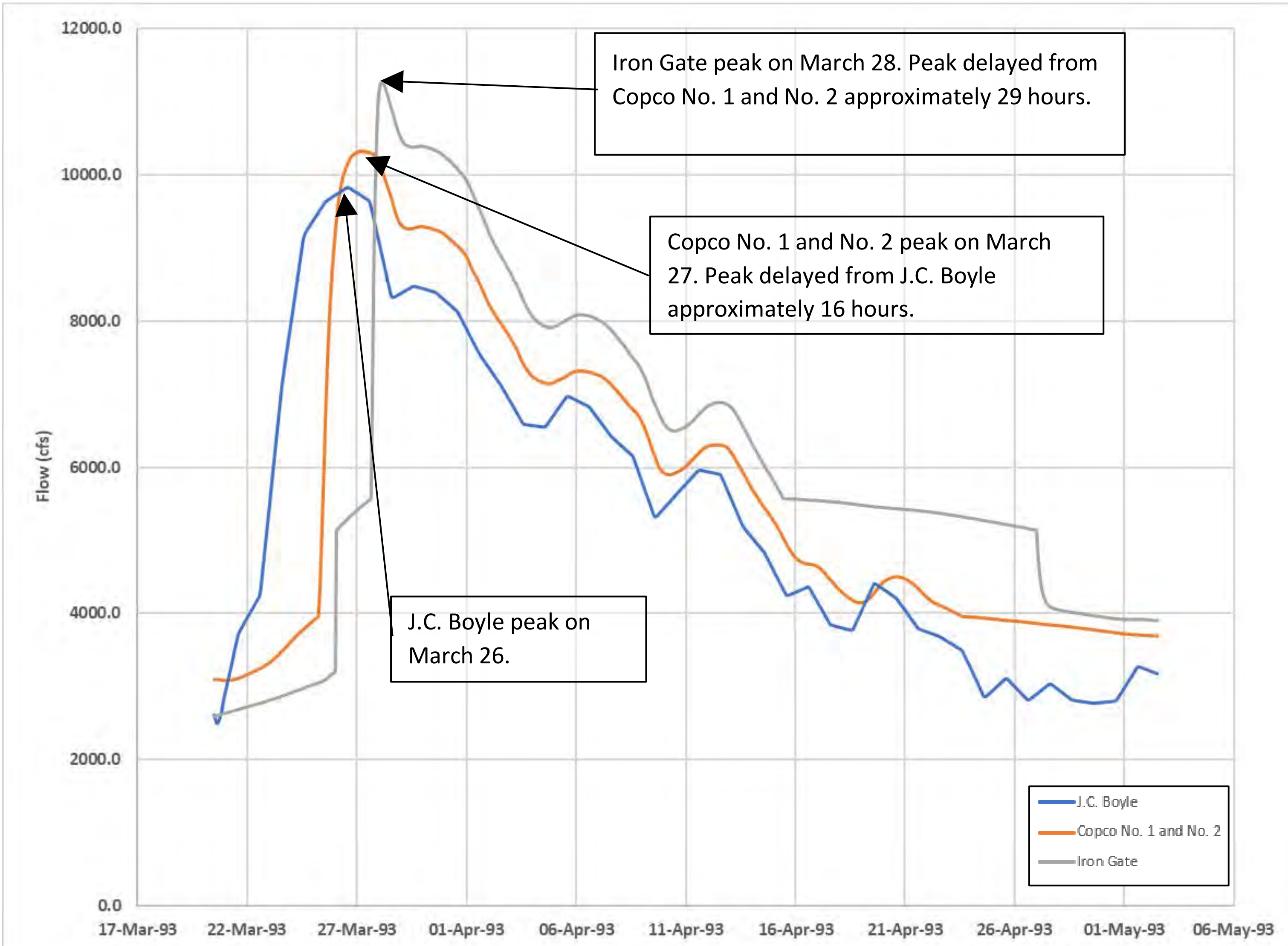

Figure 22. Comparison of inflows for the J.C. Boyle, Copco No. 1 and No.2 and Iron Gate reservoirs for the 1993 simulation.

3.2 Copco No. 1

3.2.1 Simulated Drawdown Results

Stage and flow results for selected Copco No. 1 simulations are provided in Figure 23 through Figure 26. These include simulation years 1984, 1993 and 1997. The 1984 simulation included an extended period of high inflows after the initial opening of the HDT, confirming the time and elevation controls for the HDT. The 1993 simulation displays the rapid drop in the WSE when the HDT is open to 18 feet, and the subsequent inactivation of the Low-Level Outlet. The 1997 simulation highlights the minimum operating level of the reservoir and confirms the proposed operational controls of the HDT due to fluctuating WSE at the beginning of the simulation. Both elevation controls (e.g. Historic Diversion Tunnel (HDT) will not open until WSE drops below El. 2530 feet), and time controls (e.g. HDT will not open until after June 15) are utilized for this reservoir.

A stage and flow profile plot for the full 1997 simulation is provided in Figure 23. A finer resolution profile showing the key operational triggers minimum operating levels is provided in Figure 24 through Figure 26. As show in Figure 23 and Figure 24, the simulation begins at the minimum operating level of El. 2597.0 feet. The Low-Level Outlet is open on January 1 and the spillway is also activated due to high inflows. Despite the WSE dropping below El. 2530.0 feet in April (Figure 23), the HDT will not open until after June 15.

Activation of the HDT is shown in Figure 25 and Figure 26. In Figure 25 (1993 simulation), the WSE is below El. 2530 feet allowing the HDT to open on June 16 to a height of 5 feet. Once the WSE drops below El. 2516 feet, the HDT opens to 18 feet on June 19, causing the WSE to drop 16 feet. This lowers the WSE below the crest of the cofferdam, inactivating the Low-Level Outlet (Figure 25).

Figure 26 (1984 simulation) confirms that both the elevation and time controls are working. The WSE drops below El. 2530 feet on June 21, causing the HDT to open to 5 feet. Opening of the HDT drops the WSE approximately 10 feet. The WSE drops below El. 2516 feet on June 27, causing the HDT to open to 18 feet. Since the HDT did not initially open until after the elevation (WSE below El. 2530) and time (after June 15) requirements, the efficacy of the proposed operational controls for Copco No. 1 are confirmed.

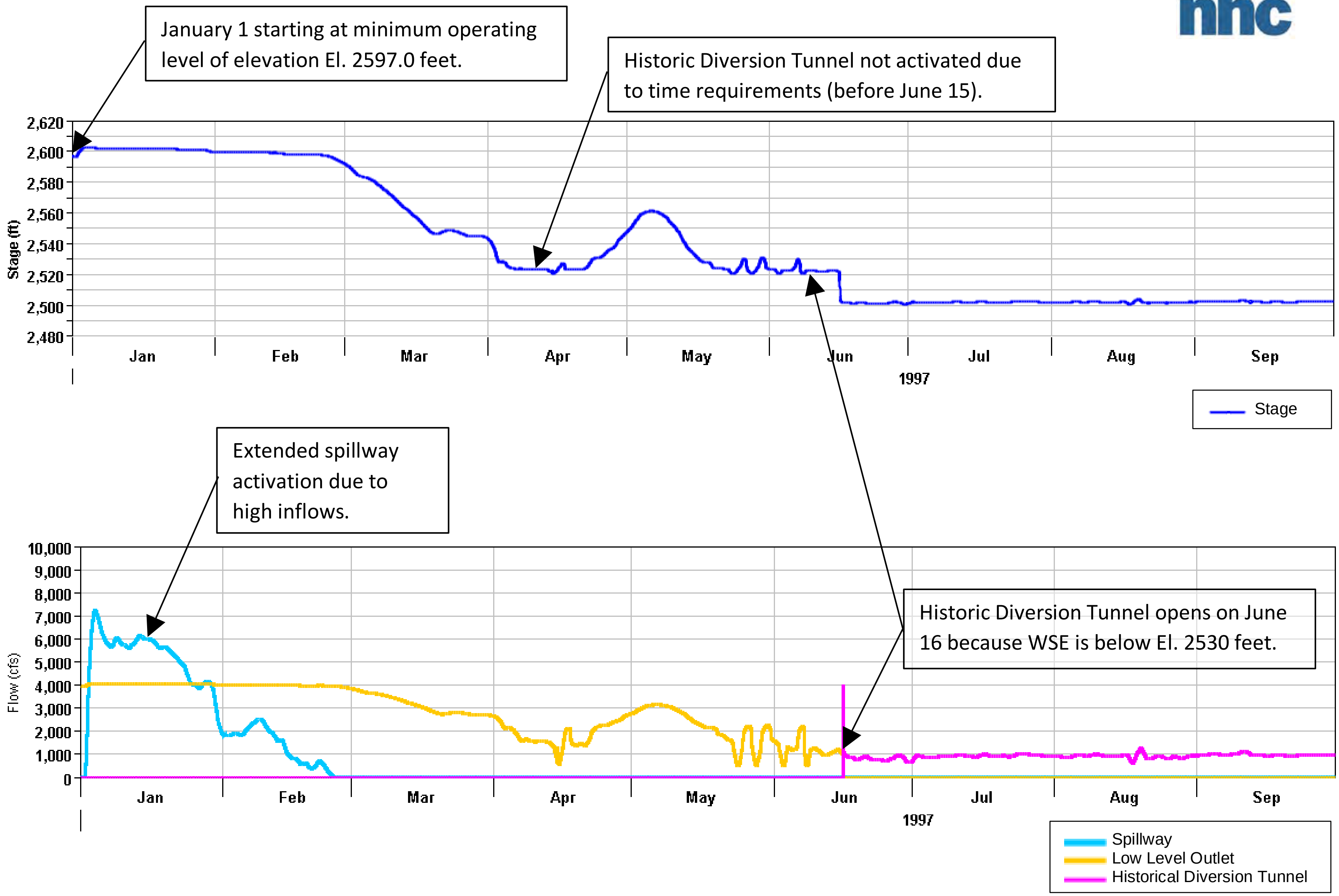

Figure 23. Copco No. 1 simulated drawdown and flow for full 1997 simulation.

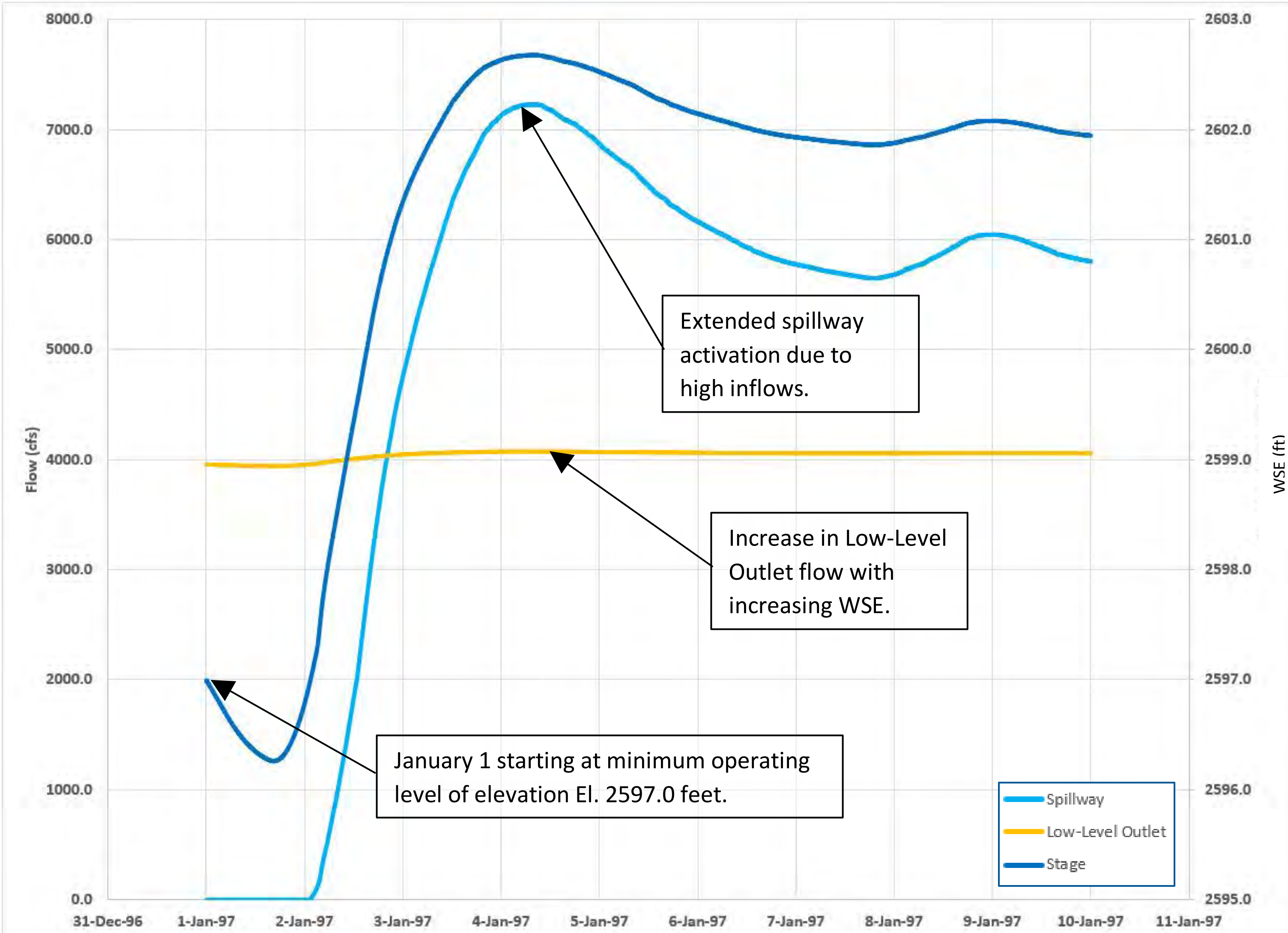

Figure 24. Copco No. 1 Project simulated drawdown and flow for spillway and new low-level outlet for 1997 simulation.

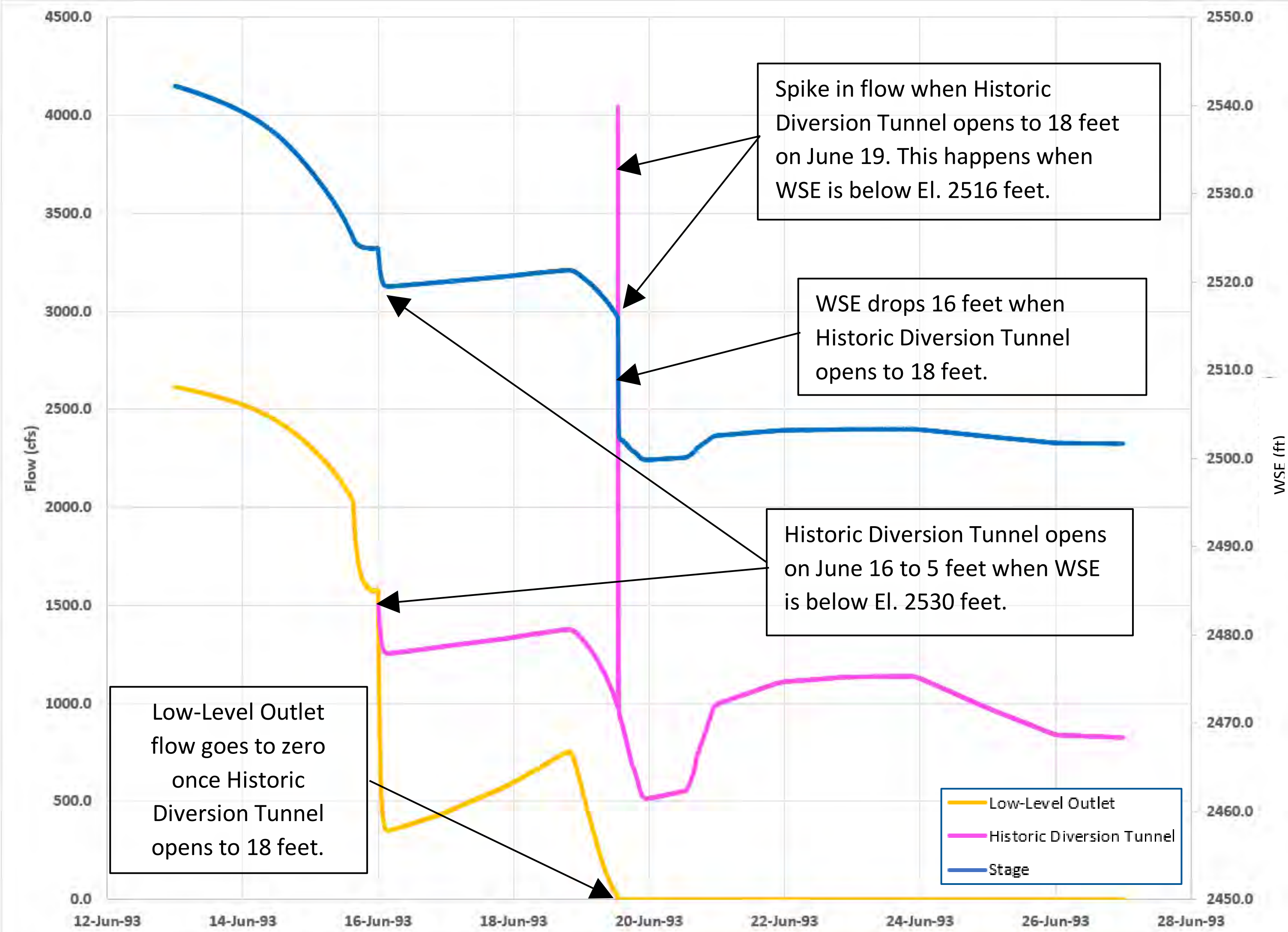

Figure 25. Copco No. 1 Project simulated drawdown and flow for new low-level outlet and HDT for 1993 simulation.

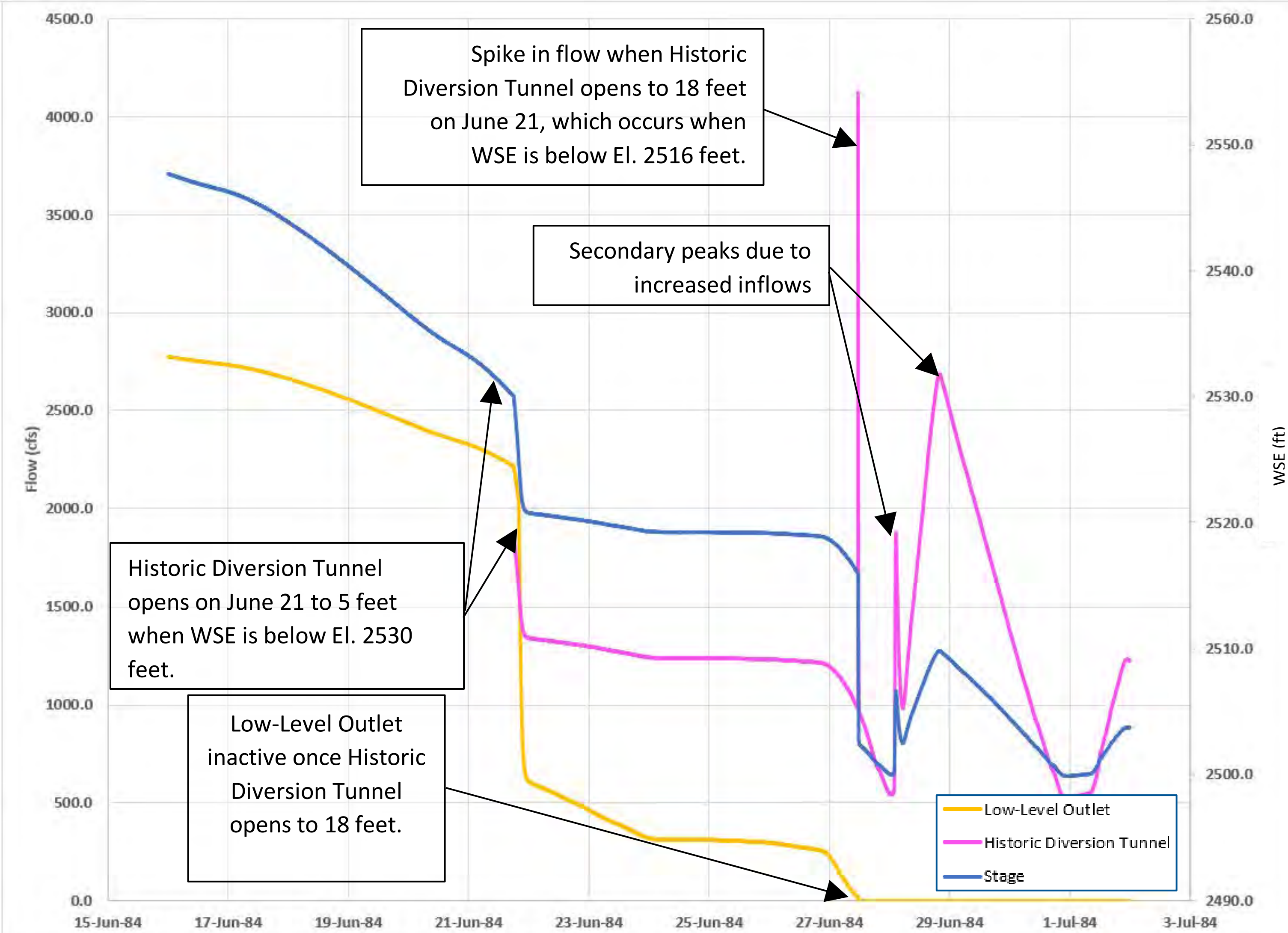

Figure 26. Copco No. 1 Project simulated drawdown and flow for new low-level outlet and HDT for 1984 simulation.

3.3 Copco No. 2

3.3.1 Simulated Drawdown Results

Stage and flow results for the 1997 Copco No. 2 simulation are provided in Figure 27 and Figure 28. The 1997 simulation highlights the connection between Copco No. 1 outflows, and Copco No. 2 inflows. The concrete plug at the spillway of Bay 1 is removed on January 1. No further openings or modifications are made throughout the simulation period at Copco No. 2, thus the outflows and stages in the reservoir are reflections of the rating curves for the remaining spillway gates and the concrete plug removal at Bay 1 only. Since Copco No. 2 functions as run-of-river, the behavior of the Copco No. 2 pond reflects upstream conditions, particularly the conditions at Copco No. 1. Figure 28 provides an example of how strongly correlated the peak flows are between Copco No. 1 and No. 2, typically with minimal attenuation. On June 16, the HDT opens to 5 feet high in Copco No. 1. Approximately 8 minutes later the peak flow reaches Copco No. 2. The HDT opens to 18 feet high at Copco No. 1 four hours after initially opening to 5 feet, and the peak flow reaches Copco No. 2 3 minutes later. The decrease in attenuation between the two peaks can be attributed to increased flow velocities from the rapid opening of the HDT from 5 to 18 feet.

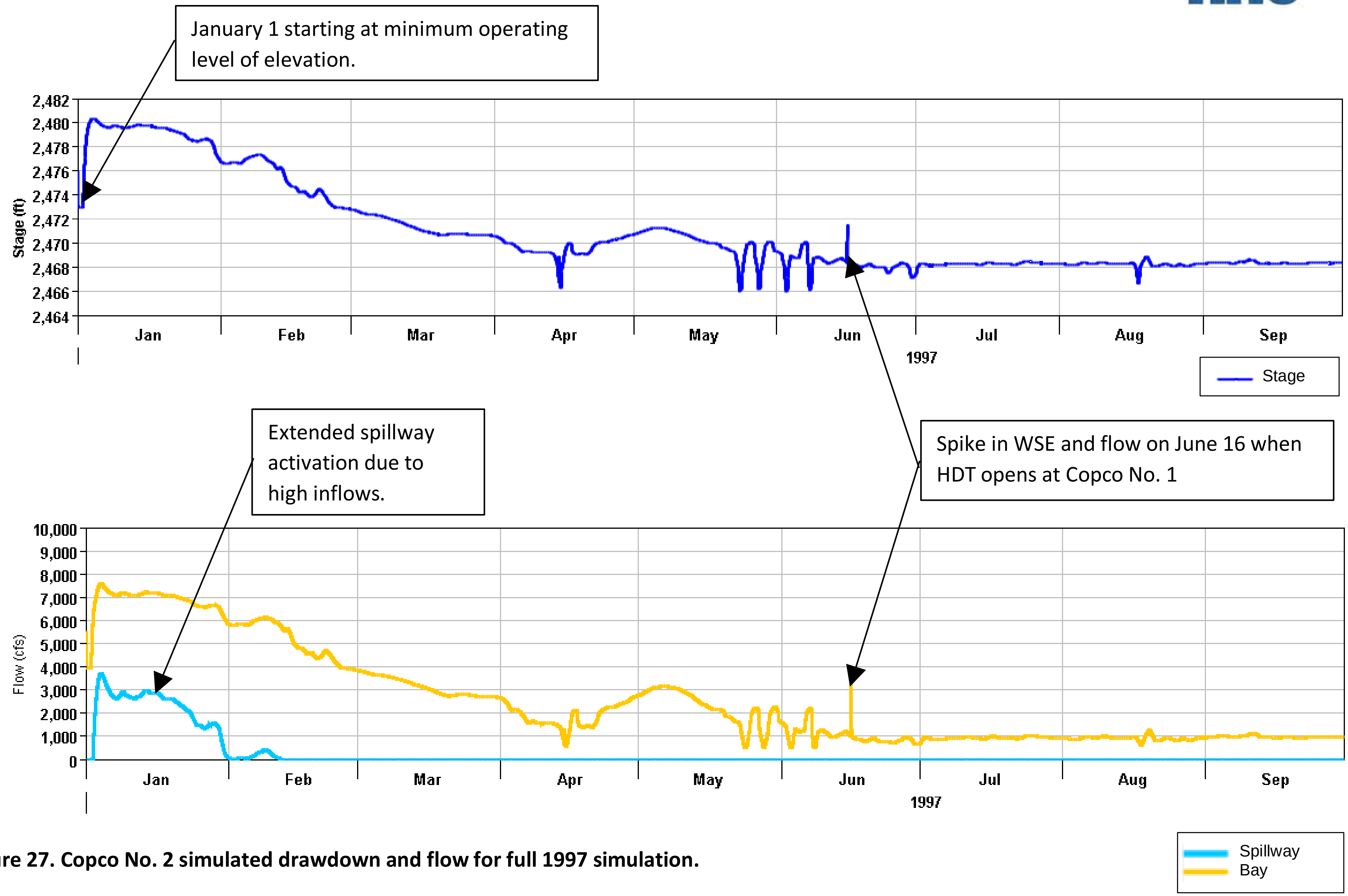

Figure 27. Copco No. 2 simulated drawdown and flow for full 1997 simulation.

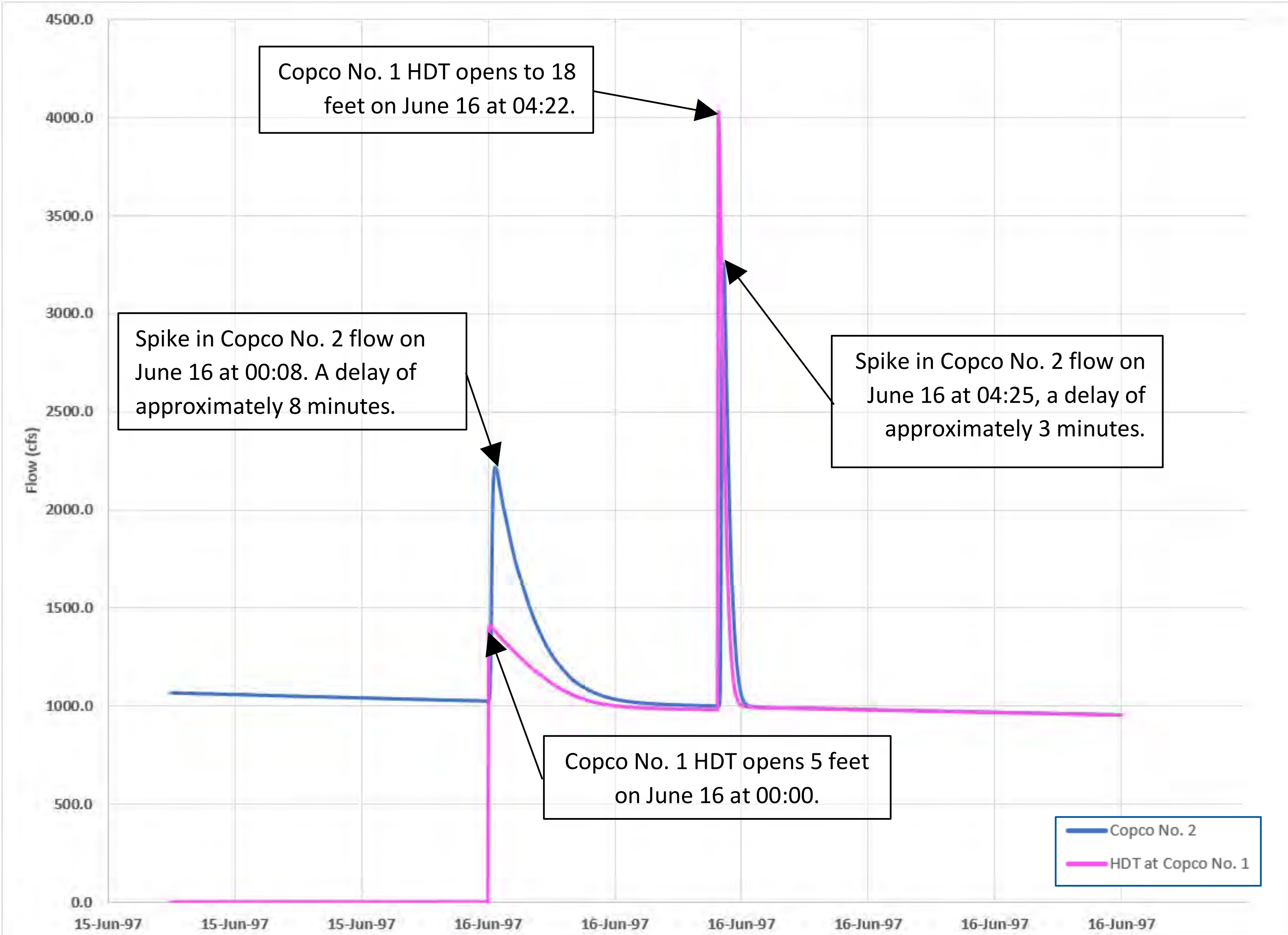

Figure 28. Comparison between Copco No. 1 and No. 2 flows for 1997 simulation.

3.4 Iron Gate

3.4.1 Simulated Drawdown Results

Stage and flow results for selected Iron Gate simulations are provided in Figure 29 through Figure 31. These include the 1997 and 2005 simulations. The 1997 simulation shows extended activation of the bypass valve due to high headwater conditions, and the 2005 simulation provides an example of dramatic stage increases due to spring storm events. Iron Gate is controlled by a spillway, bypass valve, and Historic Diversion Tunnel (HDT).

A stage and flow profile plot for the full 1997 simulation is provided in Figure 29. A finer resolution profile showing the minimum operating levels and initial hydraulic controls is provided in Figure 30. As shown in Figure 29 and Figure 30, the simulation begins at the minimum operating level of El. 2327.3 feet. The bypass valve and HDT are open on January 1 and the gate in the HDT will not be used to regulate flow. Due to high inflows, the bypass valve is utilized until March 17 (Figure 29), when the WSE drops below El. 2305 feet. This is in contrast to the 2005 simulation (Figure 31), where the WSE dropped below the bypass valve invert on January 6.

One notable result in the Iron Gate figures is the significant increase in stage seen in spring due to large inflows from Copco No. 2 and adjacent tributaries, as occurred in 2005 with a stage increase from 2200 to 2300 feet (Figure 31). In general, stage increases were between 40 to 100 feet in the reservoir during these inflows, with the larger increases in the drier years. Outflow from Iron Gate is hydraulically controlled by the regulating gate, which has a capacity of approximately 4000 cfs, providing some attenuation of large inflows within the Iron Gate reservoir. The magnitude of these surges is significant and should be carefully considered when developing plans for nearby work.

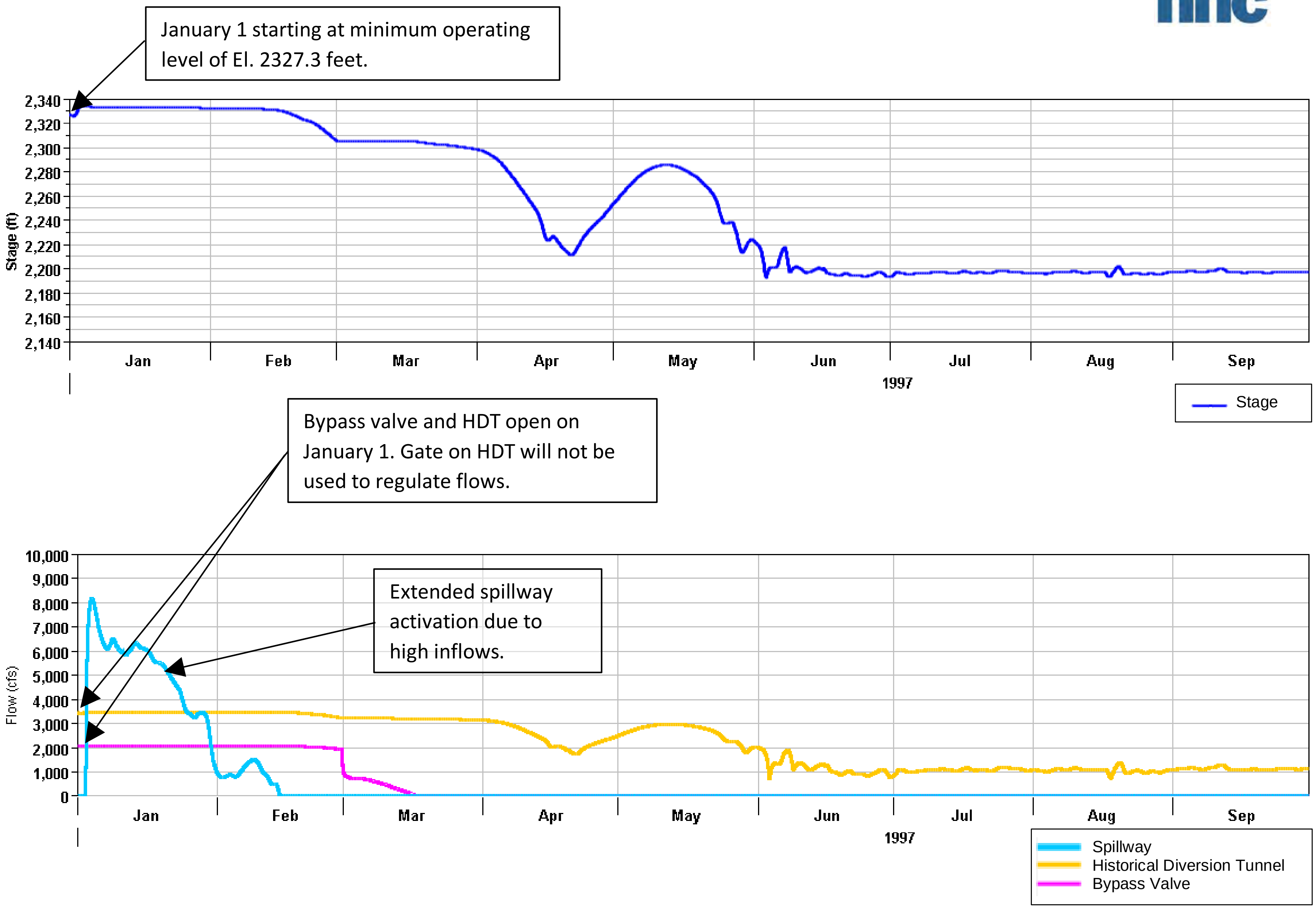

Figure 29. Iron Gate simulated drawdown and flow for full 1997 simulation.

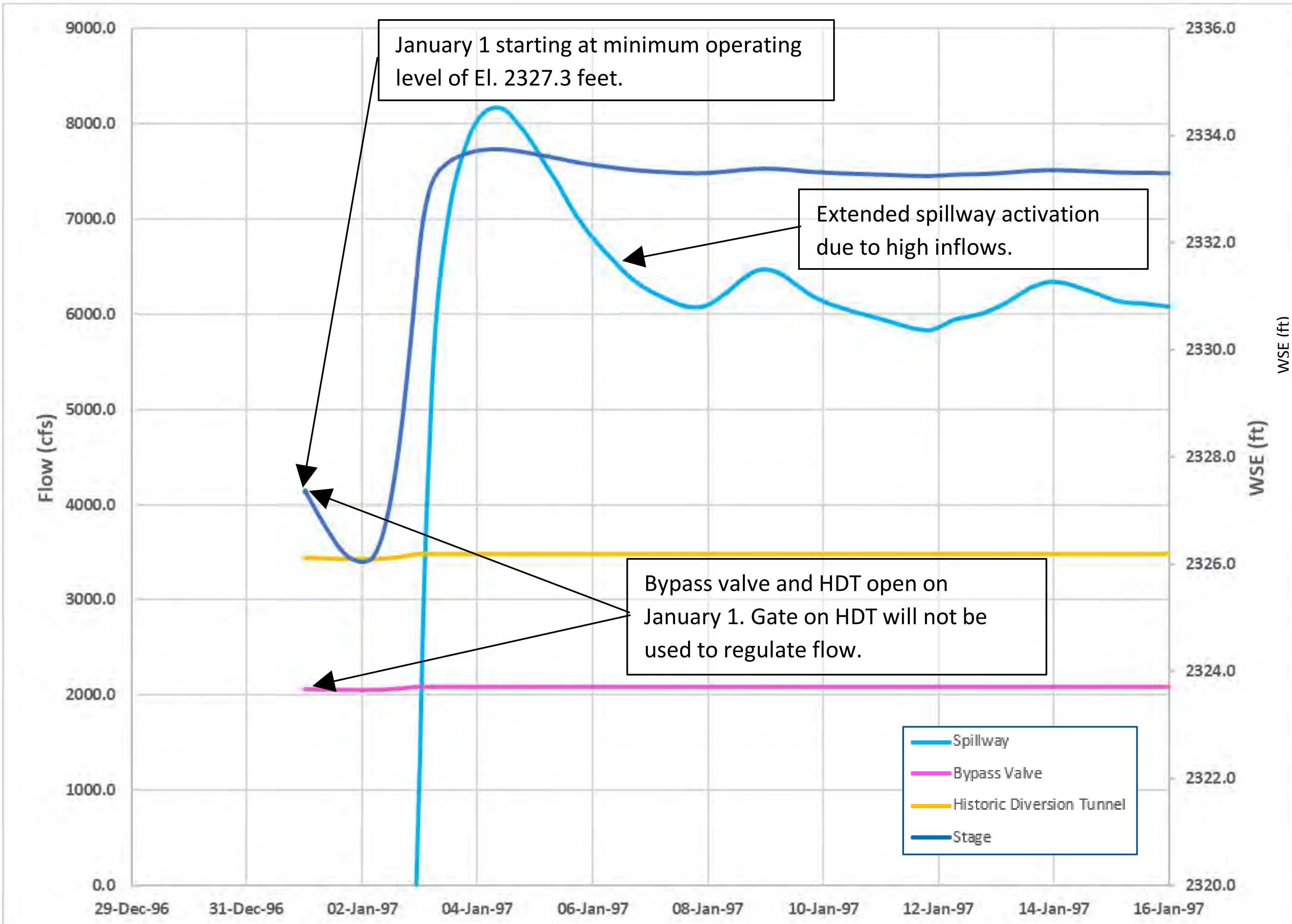

Figure 30. Iron Gate Project simulated drawdown and flow for spillway, HDT, and bypass valve for 1997 simulation.

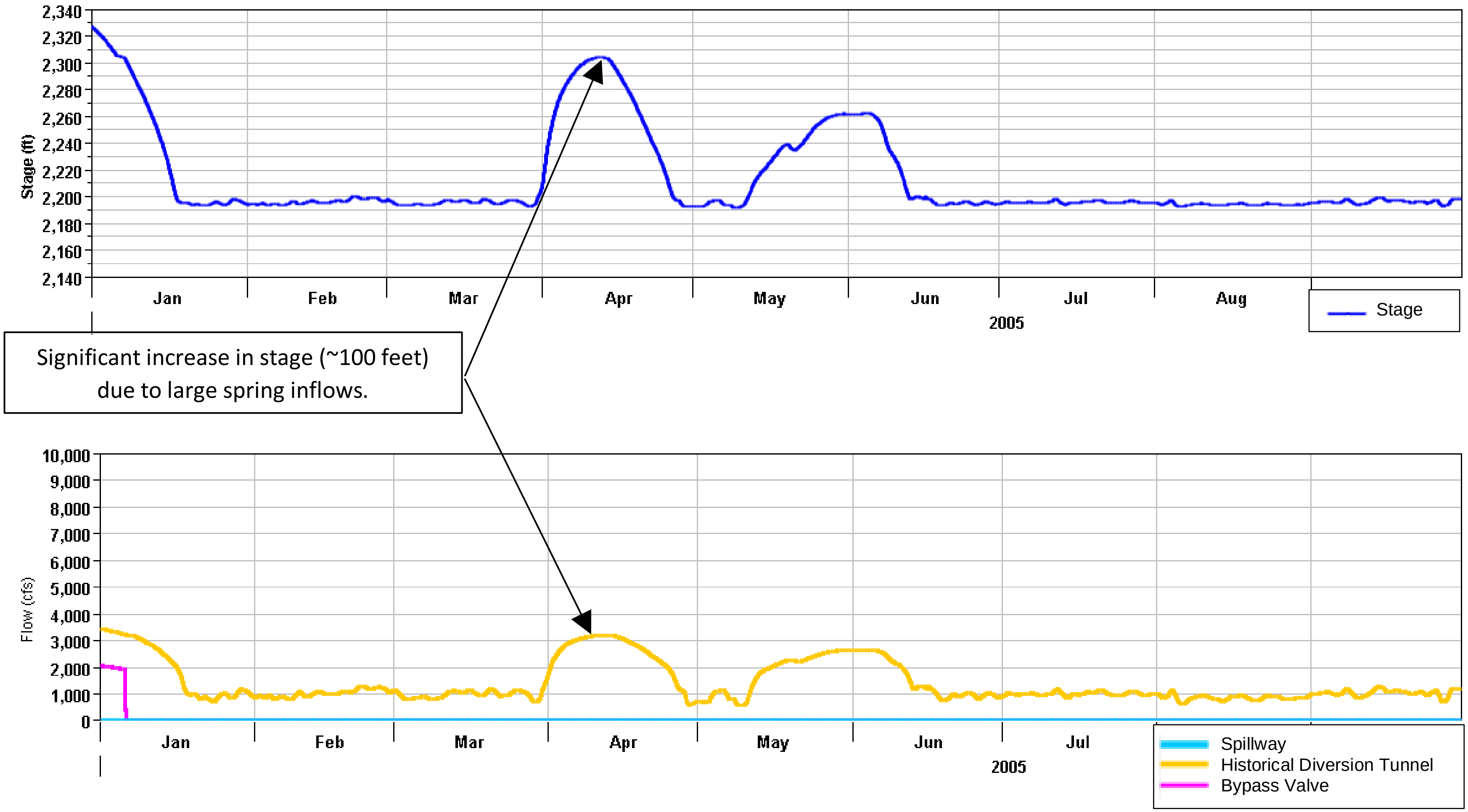

Figure 31. Iron Gate simulated drawdown and flow for full 2005 simulation.

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VERTCON. 2019. US Department of Commerce, and NOAA. VERTCON - North American Vertical Datum Conversion, www.ngs.noaa.gov/TOOLS/Vertcon/vertcon.html.

Appendix A:

Calibration and Sensitivity Analyses for Drawdown Modeling

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

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northwest hydraulic consultants

NHC Ref. No. 2004947 11.2

18 September 2020

Note to File: Klamath River Renewal Project
NHC Hydraulic Memo – 100% Design, Calibration and Sensitivity Analyses for Drawdown Modeling
Prepared by Kayla Kassa, NHC Junior Engineer, Jeremy Payne, NHC Engineer, and Todd Bennett, NHC Principal

Bullet point notes on calibration and sensitivity analyses as part of the 100% Design work for the reservoir drawdown analyses conducted by NHC:

General:

- USGS, PacifiCorp and Bureau of Land Management (BLM) data used for reservoir inflow, outflow and stage.
- Table 1 presents data sources for the reservoir simulations and riverine rating curve calibrations.
- HEC-RAS model developed by NHC used in the reservoir and riverine simulations.
- HEC-RAS model was for two applications:
    - Riverine calibration – to verify the selected Manning's n.
    - Reservoir routing – to verify model properly represented volumes and timing.

Riverine Calibration/Manning's n:

- USGS measured stage and discharge data used for riverine calibration (i.e., Manning's n values).
- For the riverine calibration, a sensitivity analysis was conducted using a range of Manning's n values and comparing the resulting stage/discharge relationships to measured values at the same location on the river. Manning's n values of 0.07 to 0.05 were simulated (using a uniform value for the entire channel width) as well as lower n values if the model numerical solution remained stable (e.g., the simulation became numerically unstable at n values of 0.04).
- A final n value of 0.05 was selected for the main channel based on representing all three conditions (the banks of estimated low flow river channel based on the 2018 bathymetric data was used to define the break between the main channel and the overbanks in the hydraulic model – see Figure 1). A final n value of 0.06 was selected for the overbanks. This yielded a rating curve that looked very similar to using 0.05 across the entire cross-section.
- In very steep sections of the hydraulic model, the n value was raised, and checked against Jarrett's equation (US Army Corps of Engineers Hydrologic Engineering Center, HEC-RAS River Analysis System Hydraulic Reference Manual Version 5, 2016) which substantiates high n values in steep streams.

Station Name	Data Type	Data Source	Period of Record	Time Step	Data Purpose
11509500 Klamath River at Keno, OR	Measured stage/discharge relationship	USGS	10/13/1970 to 06/09/2020	Sporadic Field Measurements	Comparing measured to simulated riverine stage/discharge relationships
	Discharge	USGS	10/01/1987 to Present	Daily	J.C. Boyle reservoir inflow
Historical Generation and Spillway Flow at J.C. Boyle Dam	Discharge	PacifiCorp	01/01/1979 to 12/31/2016	Daily	J.C. Boyle dam outflow Copco No. 1 reservoir inflow
Historical Reservoir Elevations at J.C. Boyle Dam	Stage	PacifiCorp	11/01/1978 to 12/31/2016	Daily	Comparing observed to simulated JC Boyle reservoir stage
11510700 Klamath River Below John C. Boyle PowerPlant, Near Keno, OR	Measured stage/discharge relationship	USGS	10/13/1970 to 06/10/2020	Sporadic Field Measurements	Comparing measured to simulated riverine stage/discharge relationships
Historical Generation and Spillway Flow at Copco No. 1 Dam	Generating and spillway discharge	PacifiCorp	01/01/1979 to 12/31/2016	Daily	Copco Dam No. 1 outflow Iron Gate reservoir inflow
Historical Reservoir Elevations at Copco Dam	Stage	PacifiCorp	11/01/1978 to 12/31/2016	Daily	Comparing observed to simulated Copco No. 1 reservoir stage
1156500 Jenny Creek, CA	Tributary Inflow	BLM	09/01/1998 to 10/01/2013	15-Minute	Tributary flow into Iron Gate reservoir
Historical Generation and Spillway Flow at Iron Gate Dam	Discharge	PacifiCorp	01/01/1979 to 12/31/2016	Daily	Iron Gate Dam outflow
Historical Reservoir Elevations at Iron Gate Dam	Stage	PacifiCorp	01/01/1979 to 12/31/2016	Daily	Comparing observed to simulated Iron Gate reservoir stage
11516530 Klamath River Below Iron Gate Dam, CA	Measured stage/discharge relationship	USGS	12/8/1960 to 06/01/2020	Sporadic Field Measurements	Comparing measured to simulated riverine stage/discharge relationships

Table 1. Data Sources.

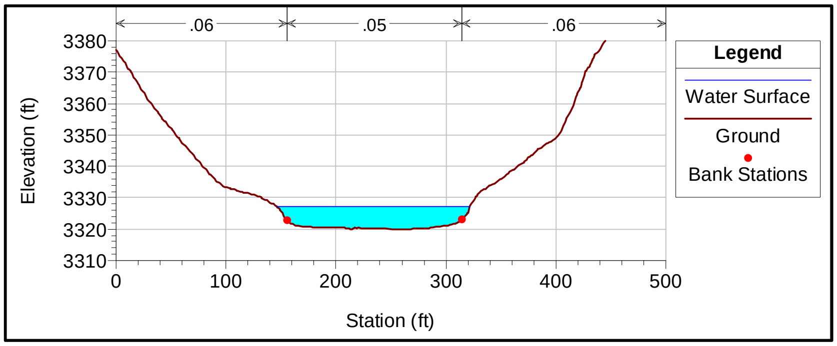

Figure 1. Example Riverine Cross-Section Derived from 2018 Bathymetric Data

Replicating Observed Reservoir Water Stage:

- The HEC-RAS models replicate observed reservoir stage using gaged inflow and outflow, and an estimation for ungaged flows.
- Adding "ungaged flows" in the model (i.e. a water balance calculation) accounts for contributing watershed runoff between the gaged inflow and outflow locations, "ungaged" sources such as groundwater and unreported outflow at the dam, and for potential inaccuracies in the gaged discharge data specified as inflow and outflow in the HEC-RAS model.
- Figure 2 shows an example from J.C. Boyle of reservoir simulations with no ungaged inflow. Using gaged inflow and gaged outflow the simulated reservoir runs dry indicating that additional inflow is needed. The simulation results are similar when ungaged inflows are not added at Copco No. 1 and Iron Gate reservoirs.
- Jenny Creek inflow (from the BLM) was added, in addition to the estimate for ungaged flows, into the Iron Gate reservoir.

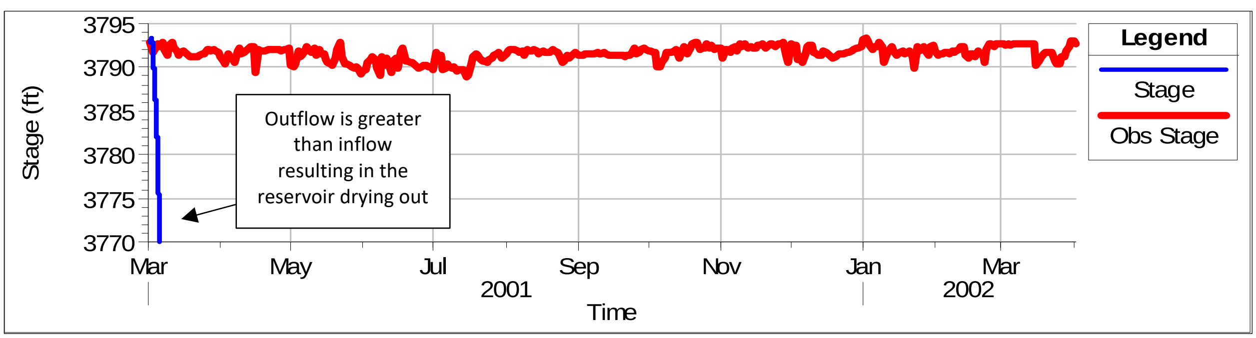

Figure 2. J.C. Boyle Reservoir Simulation Replicating Observed Stage Simulation Assuming No Estimated Ungaged Inflow

Results:

- The simulated to observed values were compared using the Root Mean Square Error (RMSE) method (Table 2 and Table 3). This method measures the differences between values predicted by a model and the values observed. The lower the value, the better performance of the model. The simulations replicating observed reservoir water stage have a RMSE less than 0.3 feet for all three reservoirs. For the simulations replicating the observed riverine condition, the closest to the USGS data uses a Manning's n equal to 0.05 in the channel and 0.06 in the overbanks for all three USGS gaging sites (RMSE less than 0.7 feet for rating curve values).
- Results of the reservoir simulations compared to observed stage, with added ungagged flows, for an example yearlong period are shown in Figure 3, Figure 4, and Figure 5.
- These results show the simulated reservoir stage matches the observed reservoir stage at all three reservoirs for the example year long period within a typical operating range. Given that this required extensive labor effort, and as much of the drawdown simulations for the 100% Design are at lower reservoir elevations, this was deemed a sufficient period for checking model results.
- Figure 6 shows the computed ungaged inflows for the three reservoirs compared to Jenny Creek. The drainage area for these areas are very roughly comparable to Jenny Creek and thus the magnitude of these flows, as expected, are roughly equivalent. For comparison, this figure also shows Klamath River at Keno gaged flow (these are the data used in simulation for gaged, riverine flow into the J.C. Boyle Reservoir) which is larger than the estimated ungaged inflows.
- In conducting this analysis, and evaluating gaged inflows, inconsistencies were noted in the recorded data, such as the reported daily outflow from a dam being greater than the reported outflow at a USGS gage immediately downstream, or changes in recorded reservoir stage not being consistent with changes in reservoir storage (e.g. stage increases but the net reservoir outflow decreases). These discrepancies required using negative ungaged inflow values in some cases (Figure 6).
- Figure 7, Figure 8, and Figure 9 show the results of the riverine simulations for a range of Manning's n values comparing simulated to measured stage/discharge relationships at three USGS station locations.
- Deviations between simulated and observed rating curves are the greatest at the lowest flows. To help explain why, approximate comparisons were made between the LiDAR and USGS measured values of channel width and area at the J.C. Boyle below Powerhouse USGS station. This showed the greatest difference in channel dimensions at the lowest elevations (in some cases the LiDAR data having roughly half the area but twice the width). This indicates poor representation of the channel by the LiDAR data at the lowest stage is limiting the ability to replicate observed water surface elevations at shallow depths (cross-section surveys throughout the entire study area would likely be necessary to ensure the bottom of the channel is accurately represented and improve calibration during the lowest flow).
- Figure 10 shows a time series example of simulated versus observed stage at USGS Station 11510700 upstream of the Copco No. 1 Reservoir for a range of flows

- Additional simulations have been conducted to evaluate different Manning's n values as well as other model parameters. These tests included:
    - o Removing the dams and testing the sensitivity of travel time, attenuation and stage to varying n (in 0.01 increments from 0.04 to 0.07) – to see how the Manning's n values affects simulation results during the drawdown condition.
    - o Through the riverine portion only (with dams in place) testing sensitivity of travel time, attenuation and stage to varying n (0.01 increments from 0.04 to 0.07).
    - o Through the reservoir portion only (with dams in place) testing sensitivity of travel time, attenuation and stage to varying n (n values of 0.03, 0.05 and 0.07).
    - o Varying the computational timestep (5, 15, 30 and 60 second timesteps).
    - o Varying reservoir volume by +/- 10%.
- Sensitivity tests did not show any significant change in simulated stage, travel times or attenuation. These sensitivity tests were conducted for both high and low flows conditions.

Table 2. Root Mean Square Error for Differences in Simulated and Gaged Reservoir Stage

Station Name	RMSE (feet)
Historical Reservoir Elevations at J.C. Boyle Dam	0.20
Historical Reservoir Elevations at Copco No. 1 Dam	0.16
Historical Reservoir Elevations at Iron Gate Dam	0.18

Table 3. Root Mean Square Error, for Differences in Riverine Stage-Discharge Curves, Between USGS Field Measured and Simulated Data Using a Range of Manning's n Values

Station Name	RMSE (feet)			
	Manning's n = 0.05	Manning's n = 0.05/ 0.06	Manning's n = 0.06	Manning's n = 0.07
11509500 Klamath River at Keno, OR	0.72	0.65	0.87	1.08
11510700 Klamath River Below John C. Boyle PowerPlant, Near Keno, OR	0.35	0.35	0.41	0.57
11516530 Klamath River Below Iron Gate Dam, CA	0.40	0.40	0.51	0.83

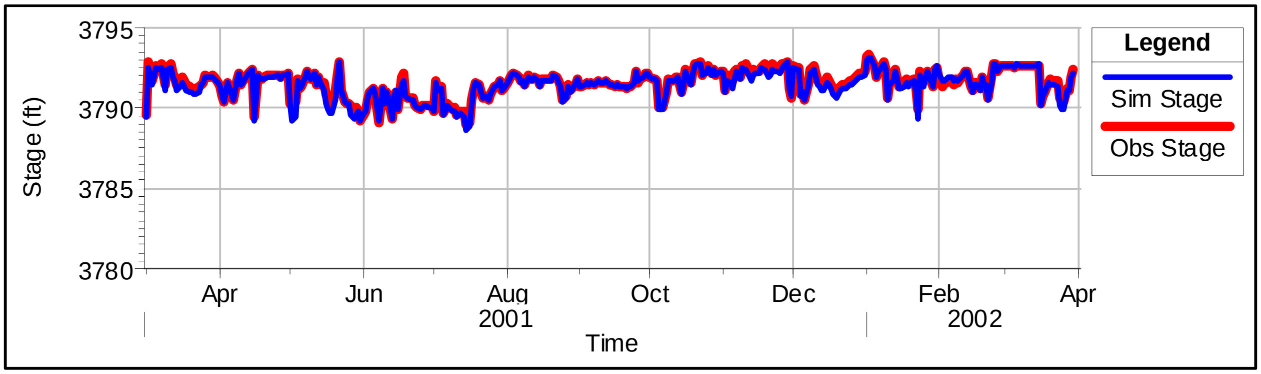

Figure 3. J.C. Boyle Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

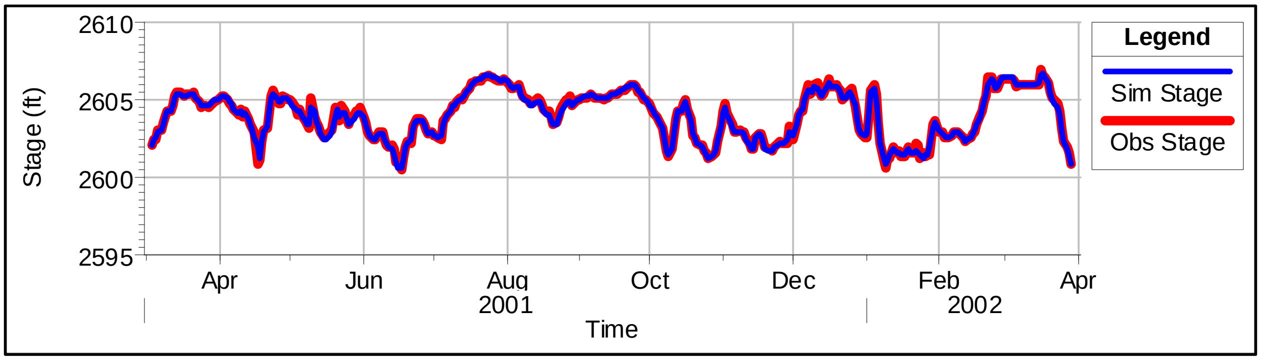

Figure 4. Copco No. 1 Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

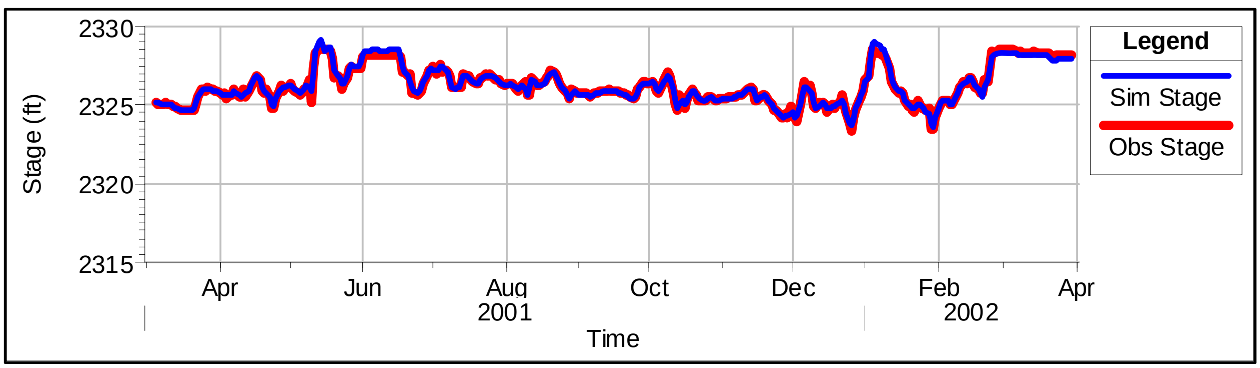

Figure 5. Iron Gate Reservoir Simulation Replicating Observed Stage with Estimated Ungaged Flow

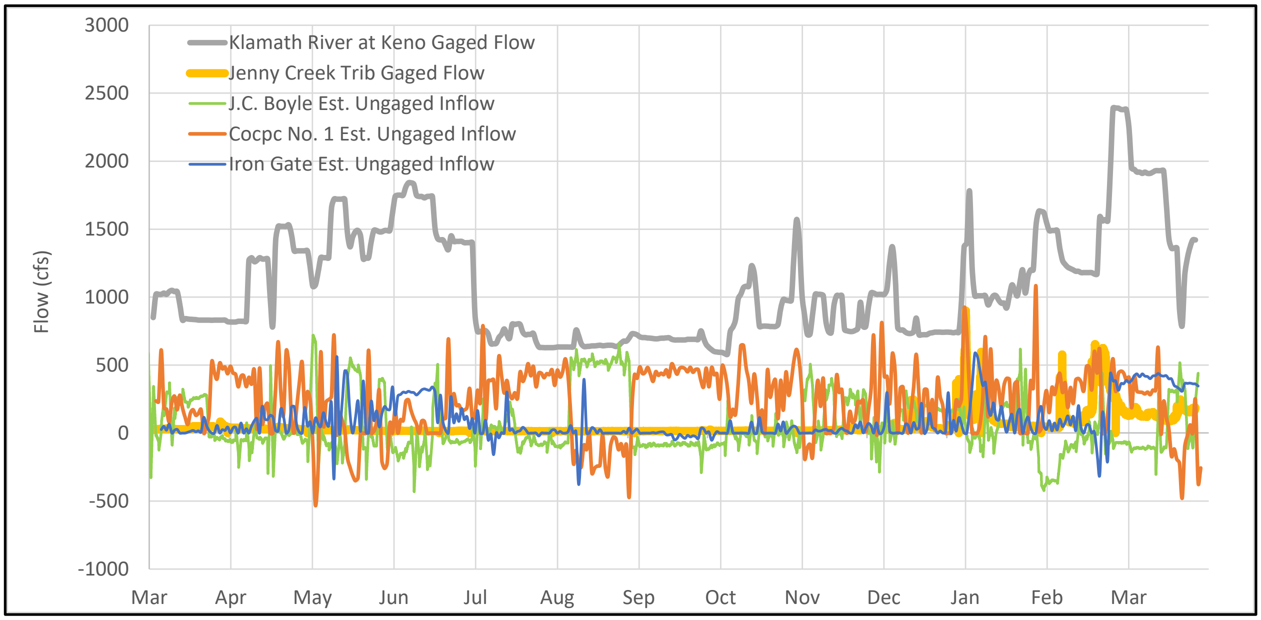

Figure 6. Estimated Ungaged Inflow for J.C. Boyle, Copco No. 1 and Iron Gate Reservoirs Compared to Jenny Creek and Klamath River at Keno Gaged Flows.

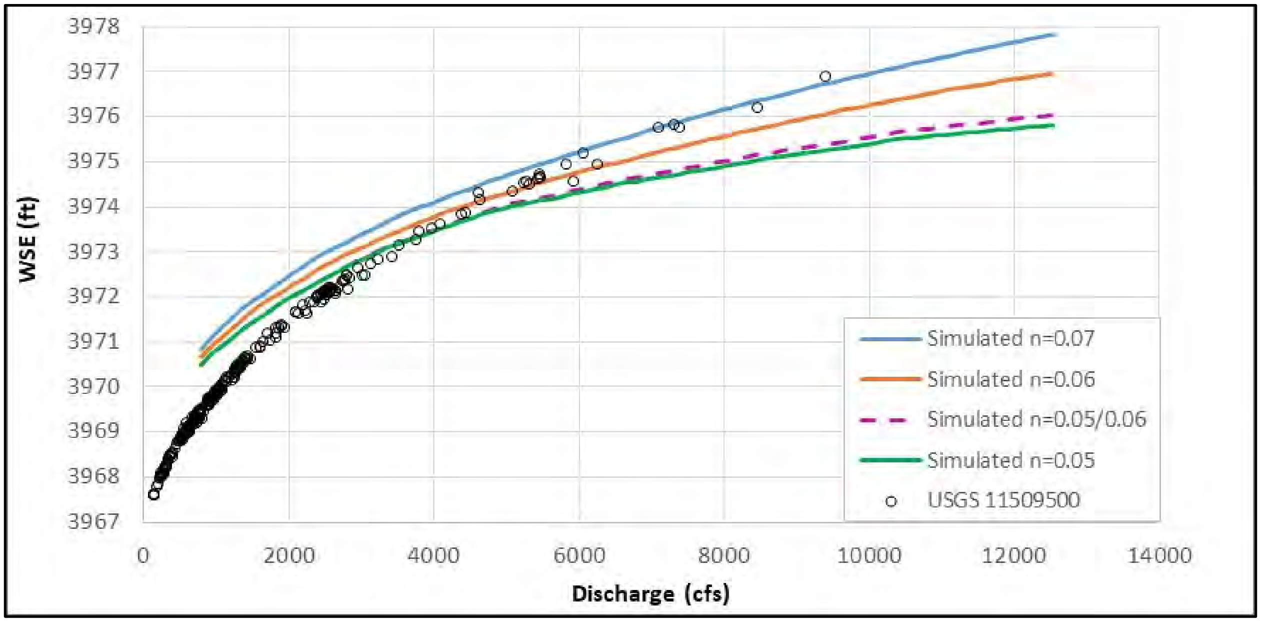

Figure 7. Simulated Stage Discharge Curves for a Range of Manning's n Values Compared to Measured Values at a USGS Station 11509500 Upstream of the J.C. Boyle Reservoir

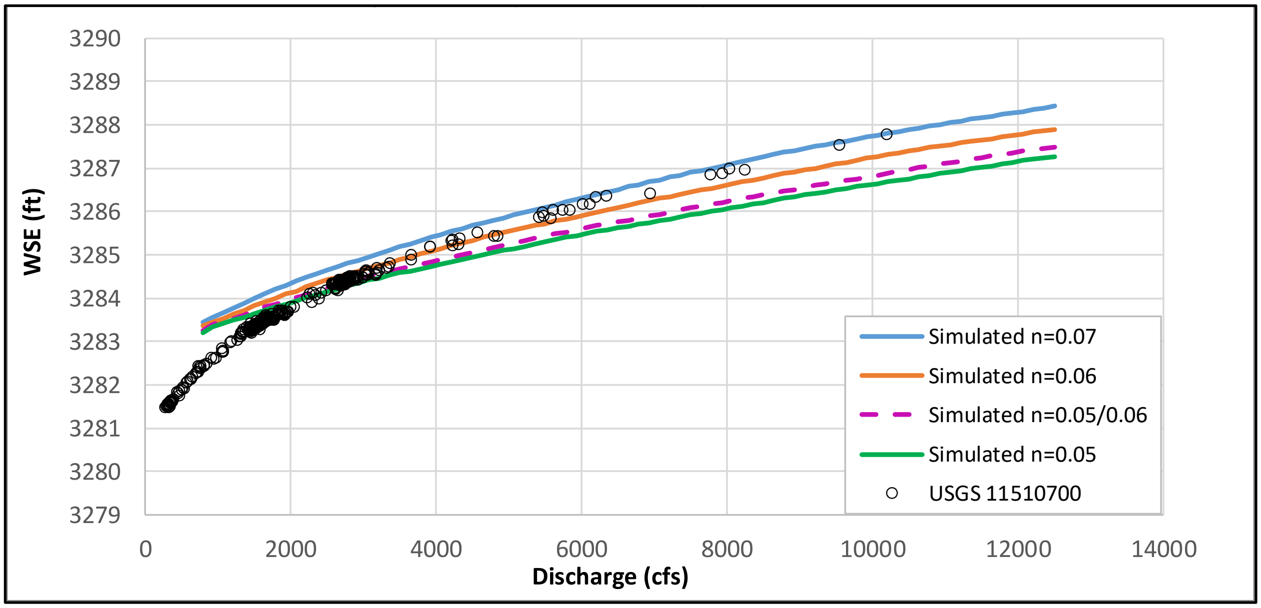

Figure 8. Simulated Stage Discharge Curves for a Range of Manning's n Values Compared to Measured Values at a USGS Station 11510700 Upstream of the Copco No. 1 Reservoir

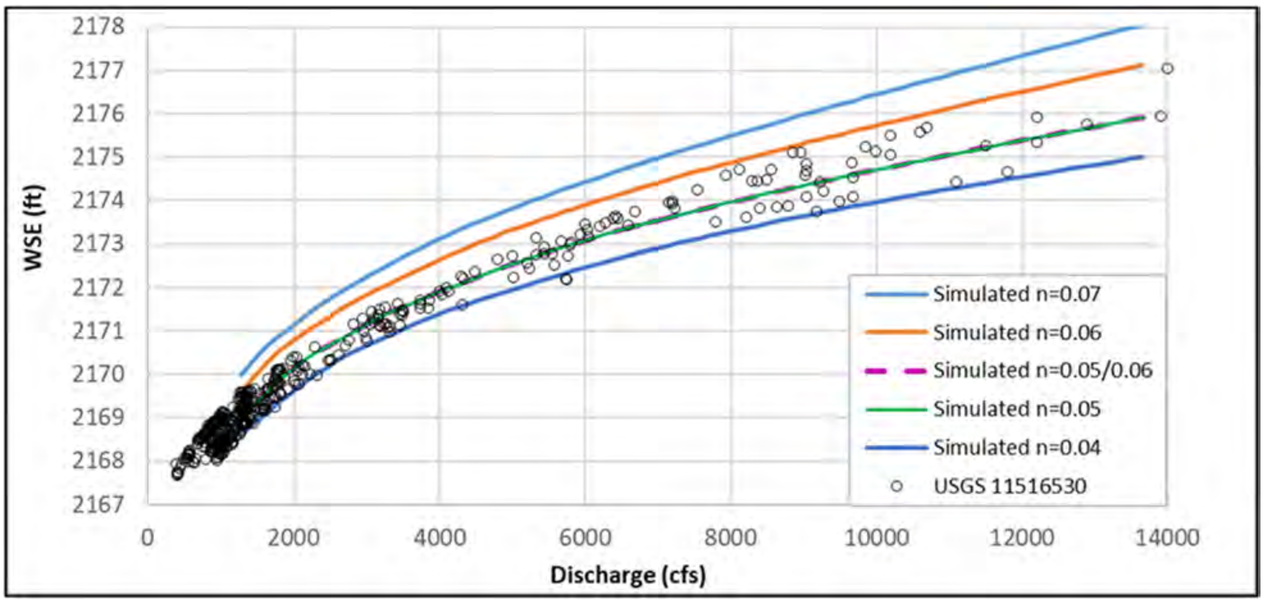

Figure 9. Simulated Stage Discharge Curves for a Range of Manning's n Values Compared to Measured Values at a USGS Station 11516530 Downstream of the Iron Gate Reservoir

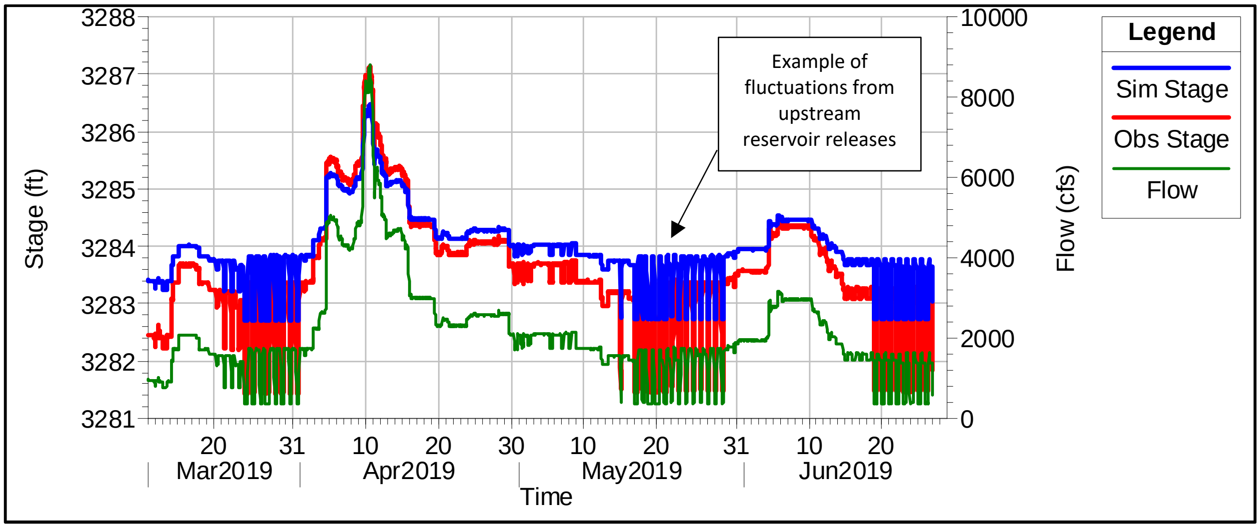

Figure 10. Simulated and Observed Stage for a Range of Flows at a USGS Station 11510700 Upstream of the Copco No. 1 Reservoir

Appendix B:

Drawdown Plots for J.C. Boyle Reservoir

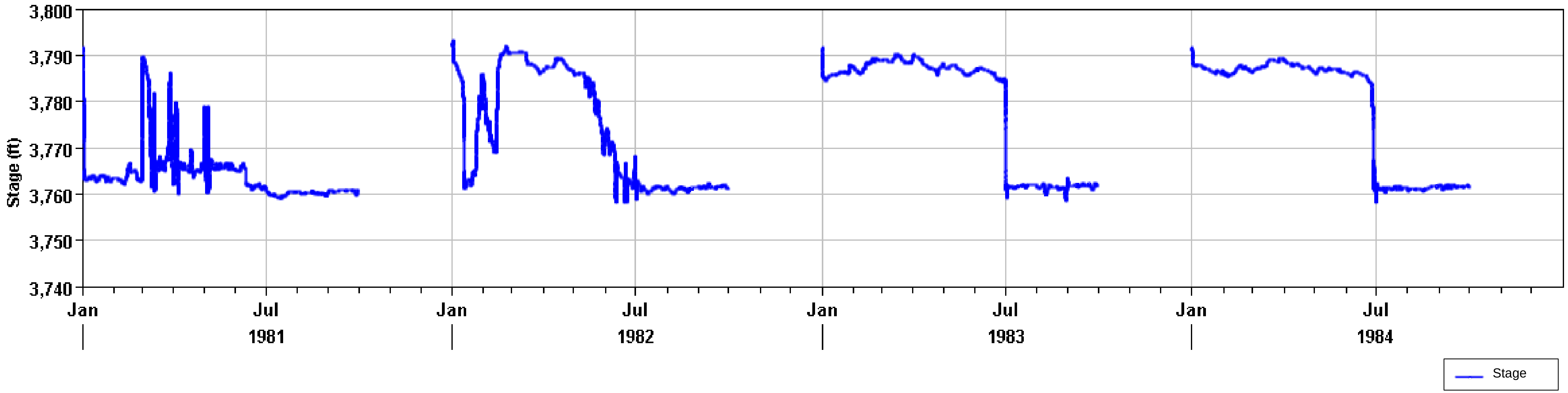

Figure 1: J.C. Boyle Drawdown Stage for years 1981 through 1984

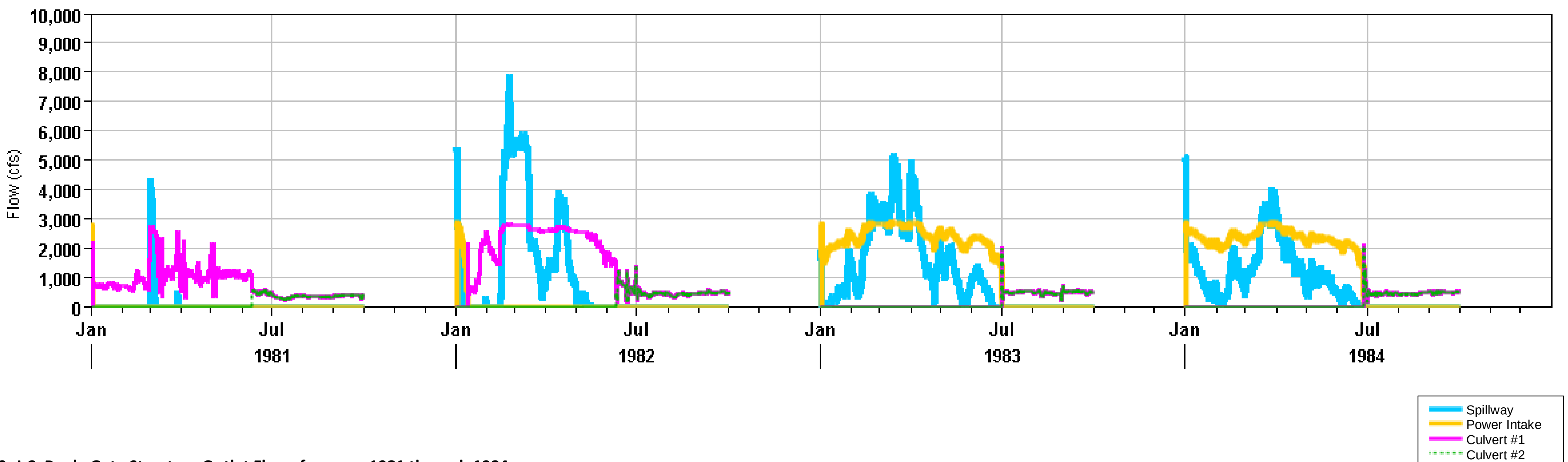

Figure 2: J.C. Boyle Gate Structure Outlet Flows for years 1981 through 1984

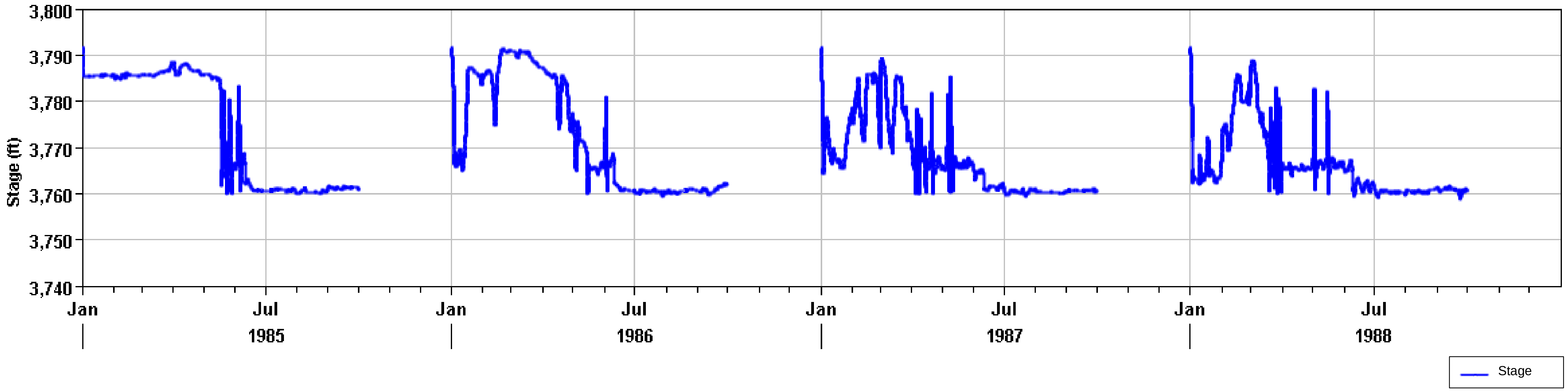

Figure 3: J.C. Boyle Drawdown Stage for years 1985 through 1988

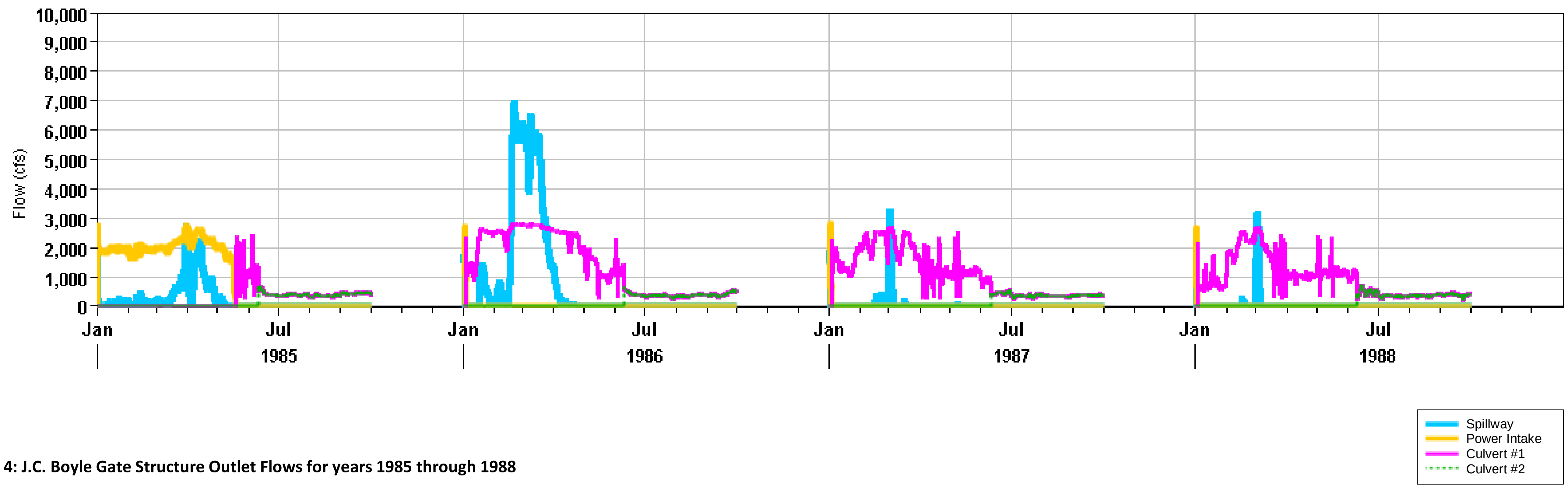

Figure 4: J.C. Boyle Gate Structure Outlet Flows for years 1985 through 1988

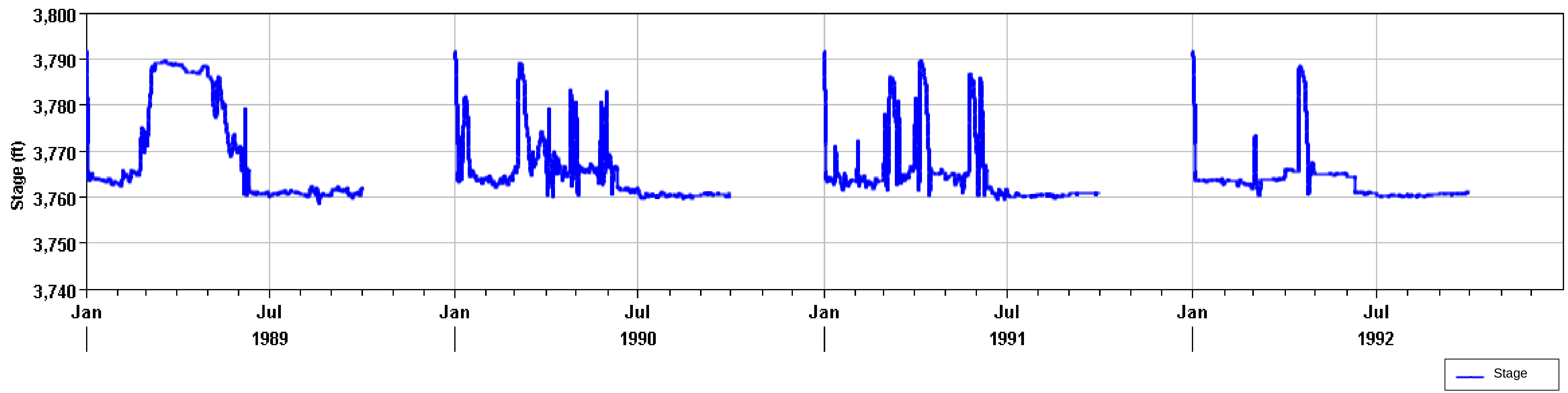

Figure 5: J.C. Boyle Drawdown Stage for years 1989 through 1992

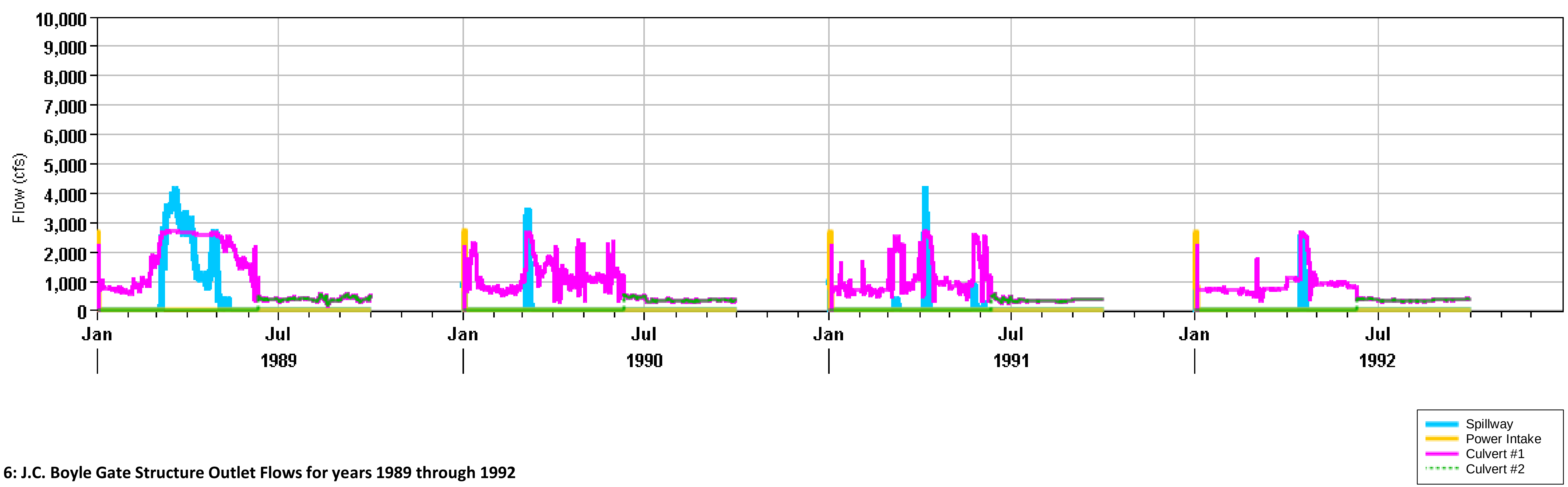

Figure 6: J.C. Boyle Gate Structure Outlet Flows for years 1989 through 1992

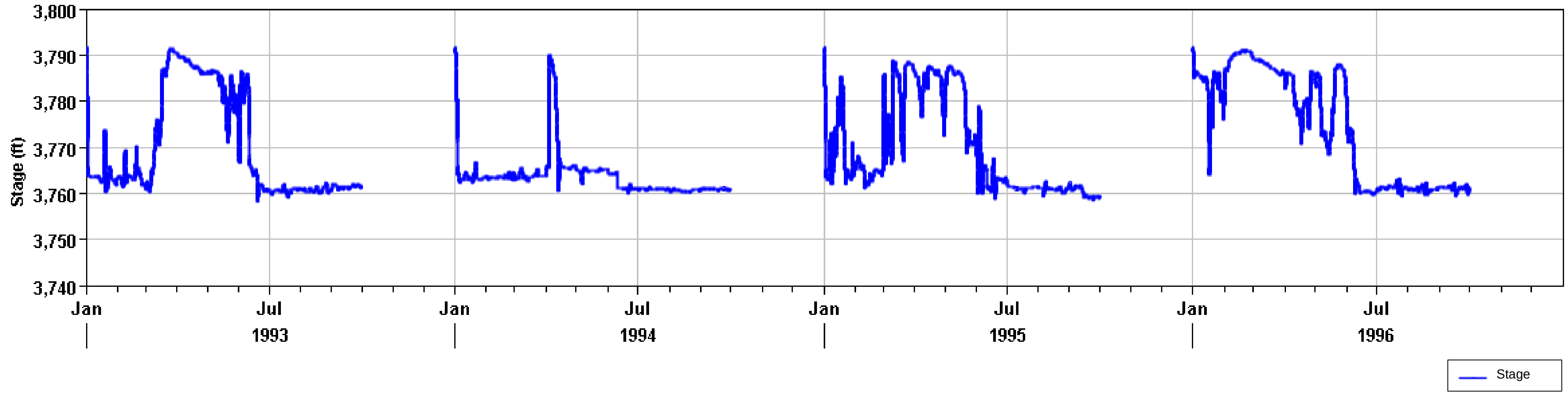

Figure 7: J.C. Boyle Drawdown Stage for years 1993 through 1996

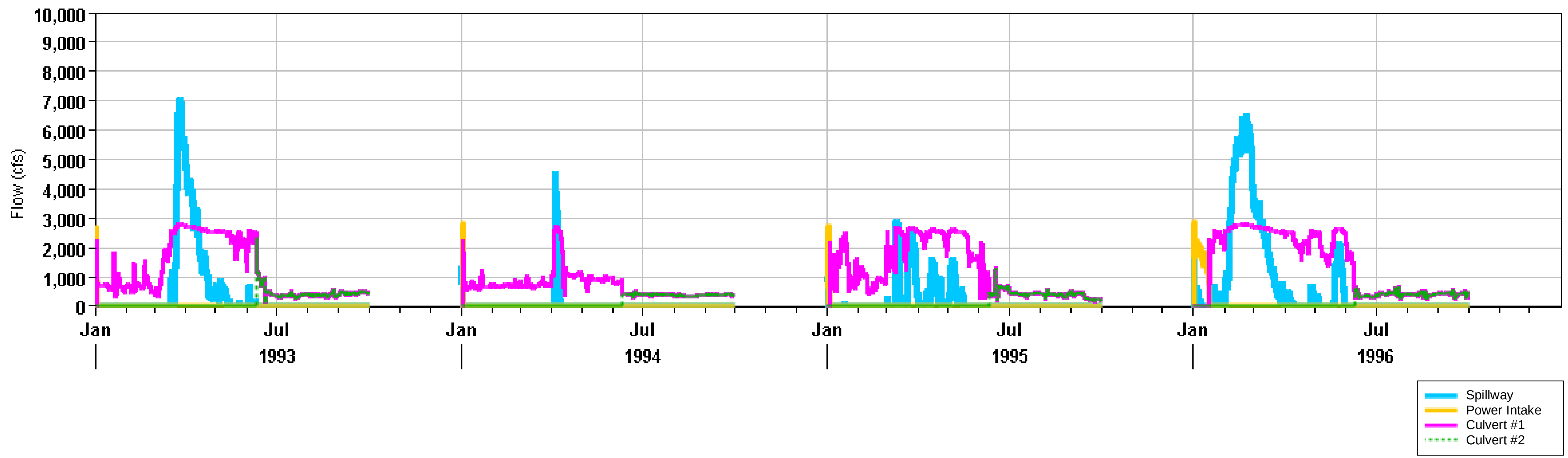

Figure 8: J.C. Boyle Gate Structure Outlet Flows for years 1993 through 1996

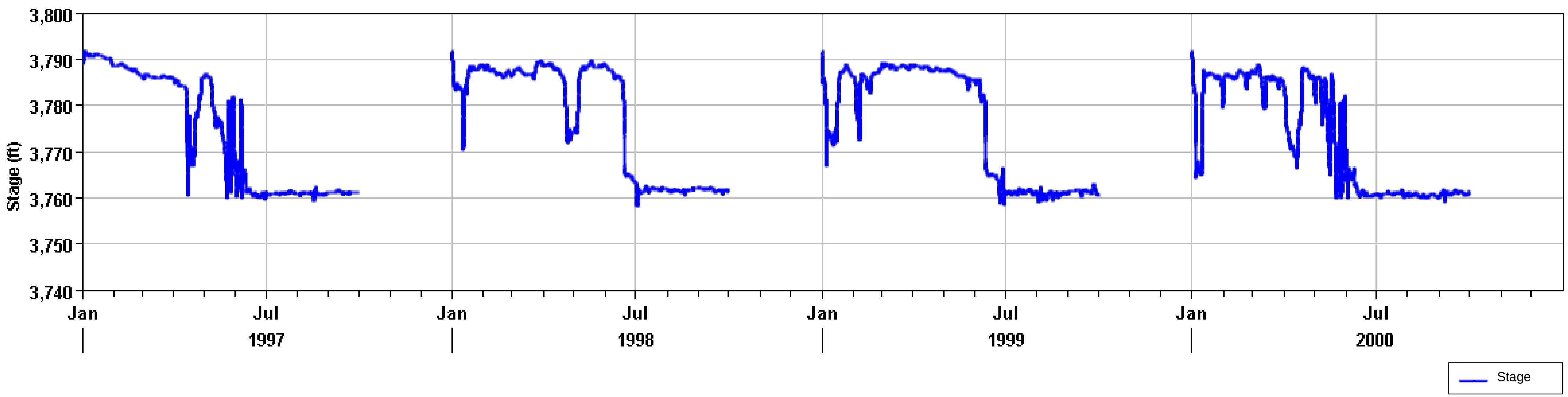

Figure 9: J.C. Boyle Drawdown Stage for years 1997 through 2000

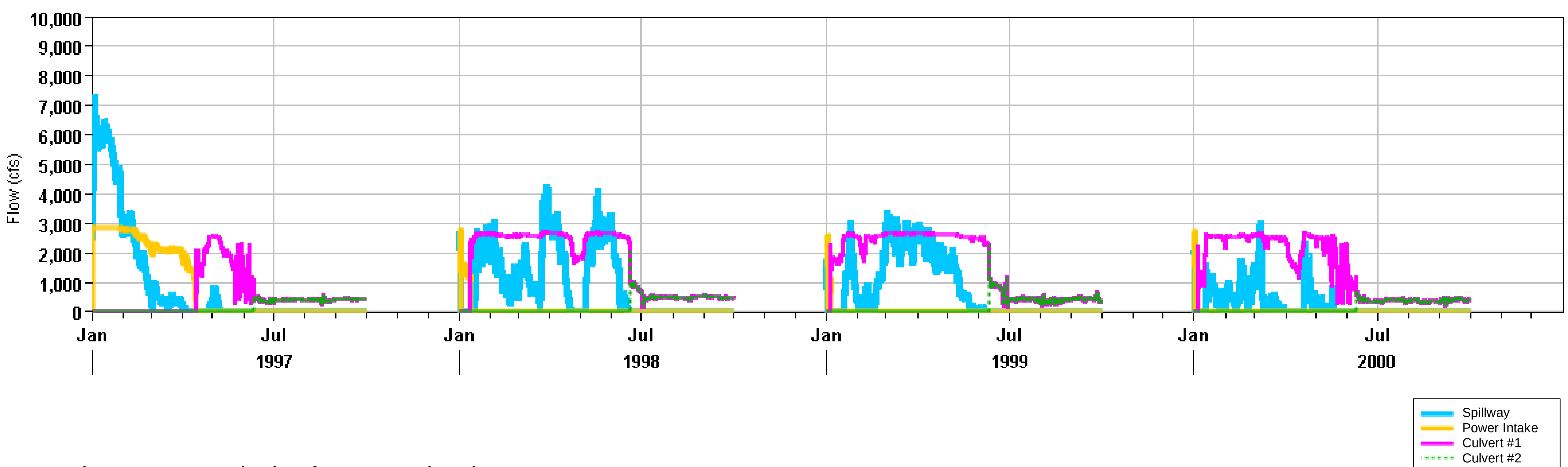

Figure 10: J.C. Boyle Gate Structure Outlet Flows for years 1997 through 2000

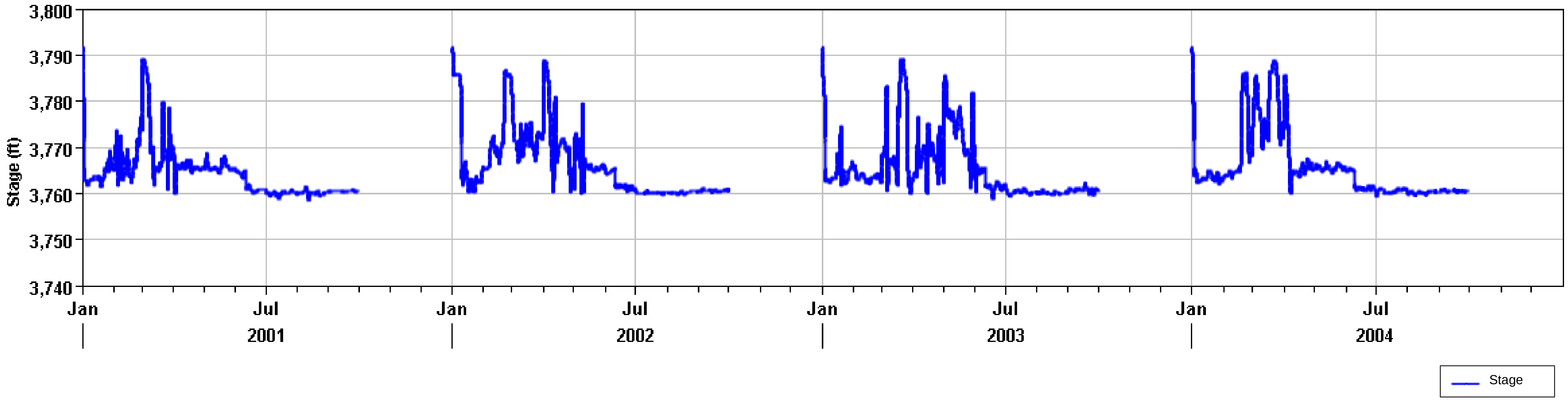

Figure 11: J.C. Boyle Drawdown Stage for years 2001 through 2004

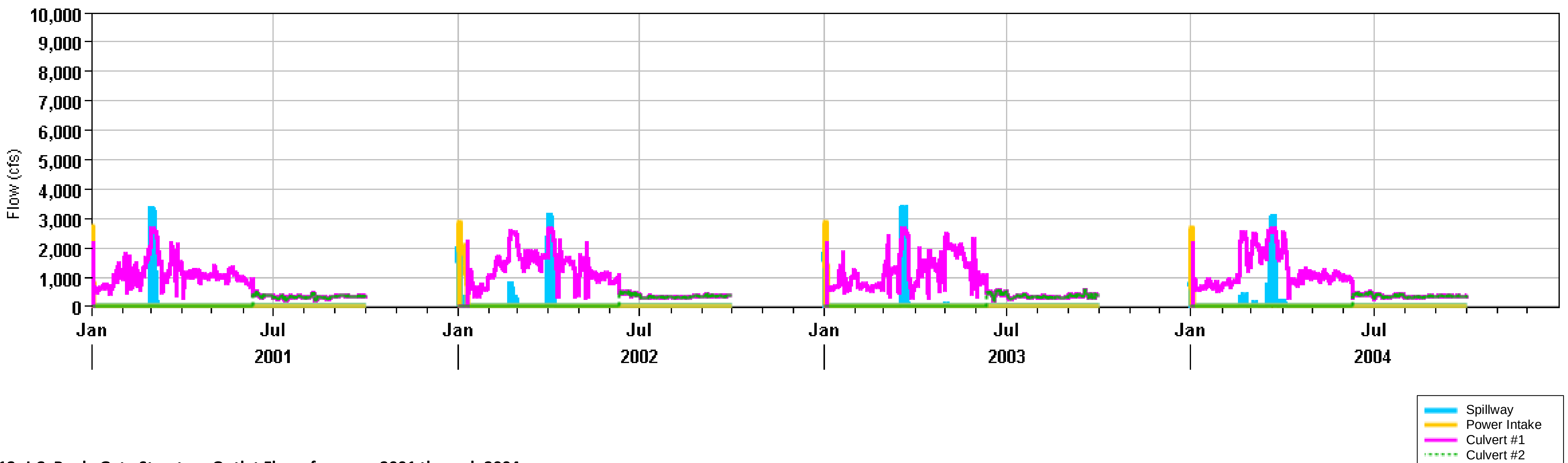

Figure 12: J.C. Boyle Gate Structure Outlet Flows for years 2001 through 2004

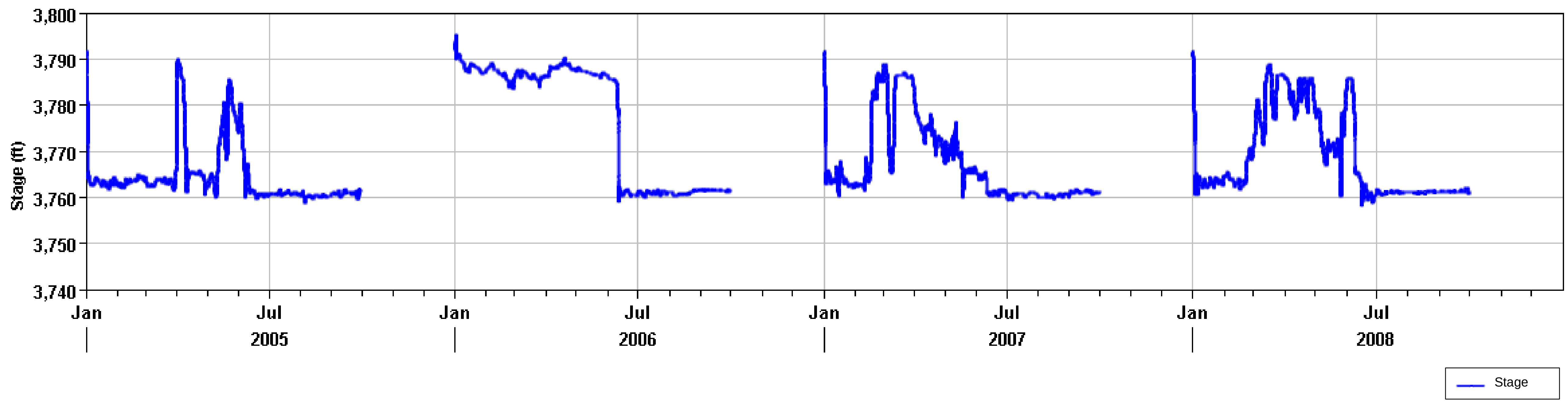

Figure 13: J.C. Boyle Drawdown Stage for years 2005 through 2008

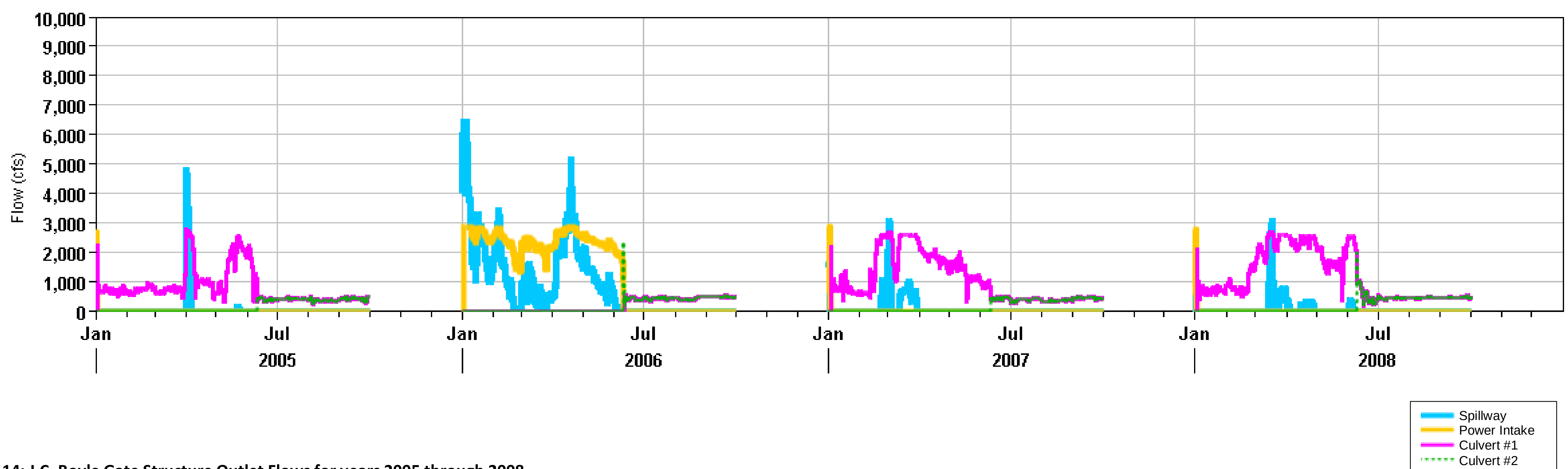

Figure 14: J.C. Boyle Gate Structure Outlet Flows for years 2005 through 2008

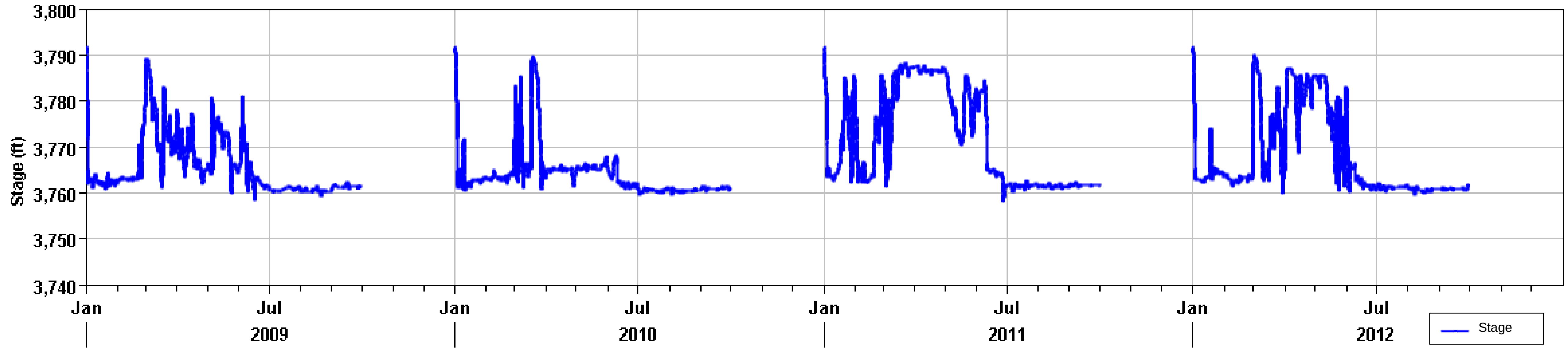

Figure 15: J.C. Boyle Drawdown Stage for years 2009 through 2012

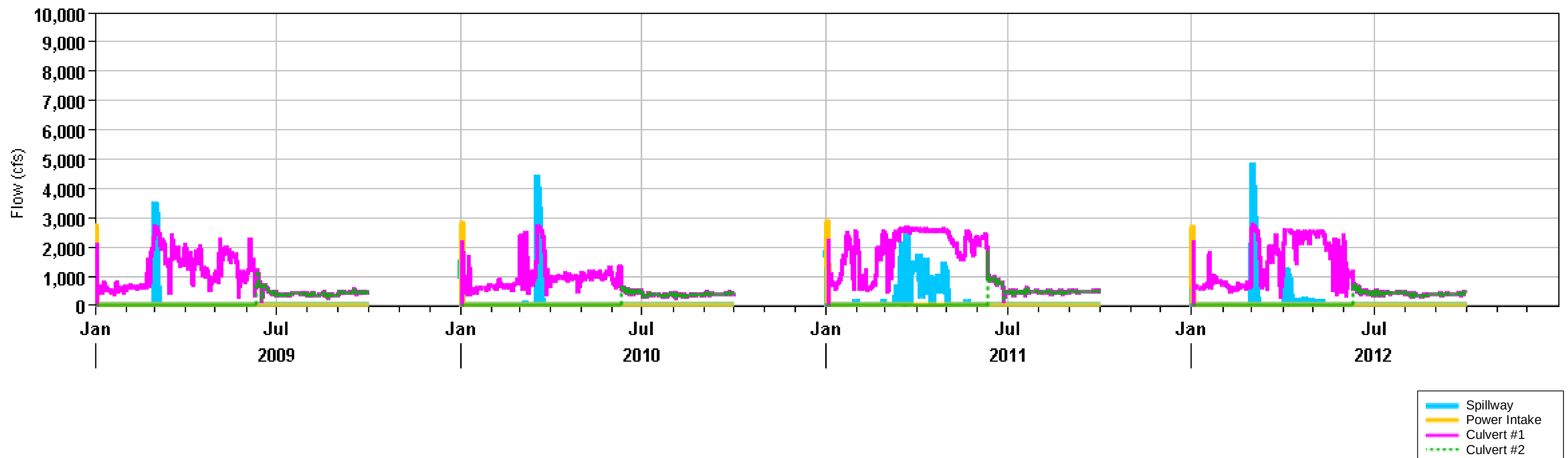

Figure 16: J.C. Boyle Gate Structure Outlet Flows for years 2009 through 2012

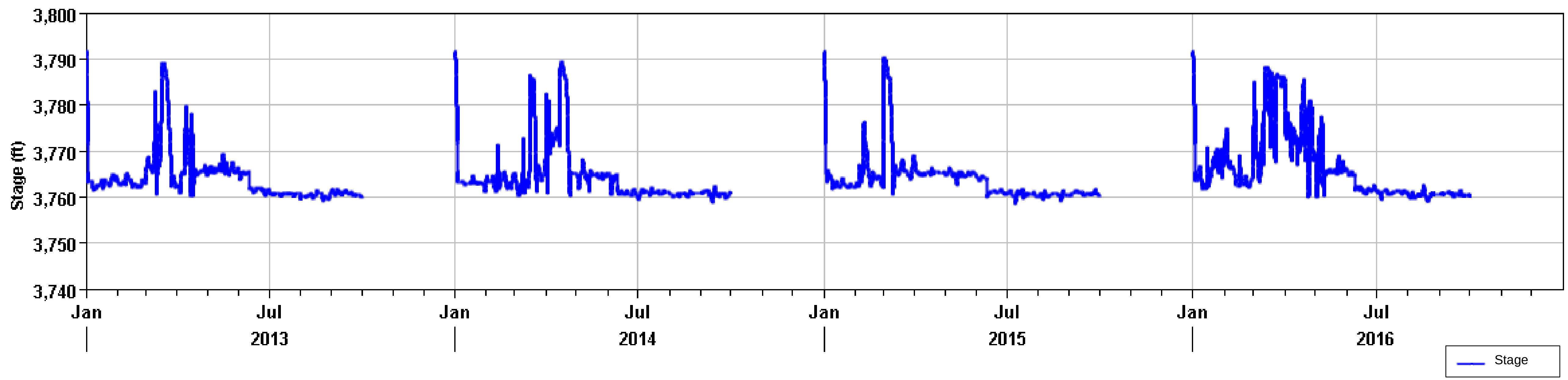

Figure 17: J.C. Boyle Drawdown Stage for years 2013 through 2016

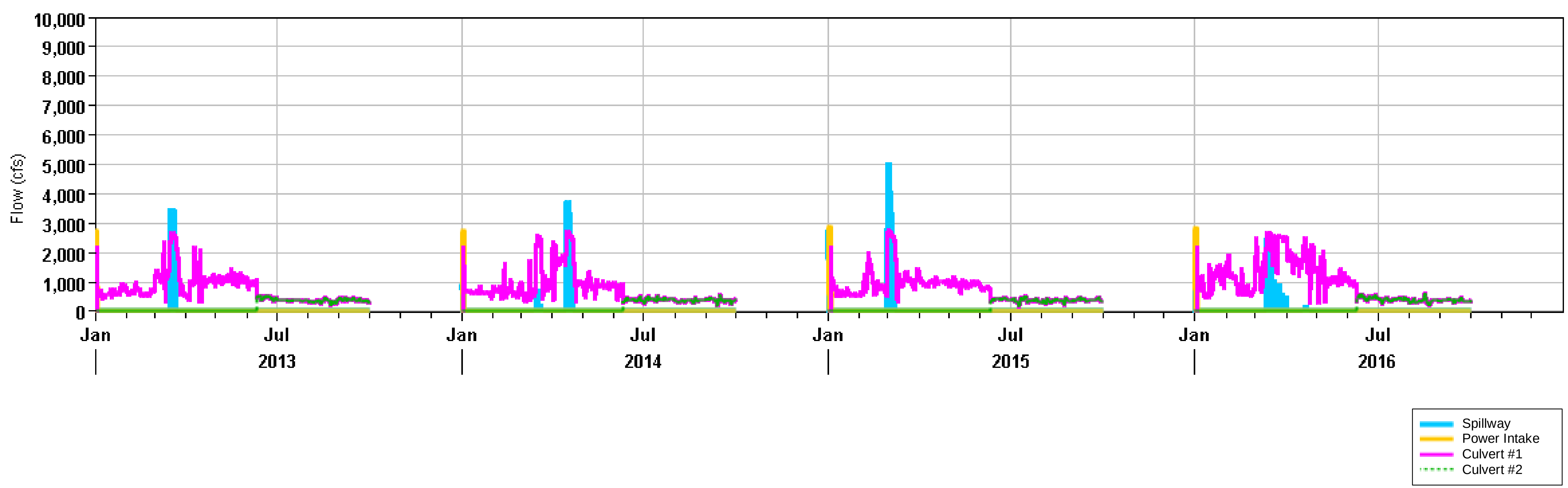

Figure 18: J.C. Boyle Gate Structure Outlet Flows for years 2013 through 2016

Appendix C:

Drawdown Plots for Copco No. 1 Reservoir

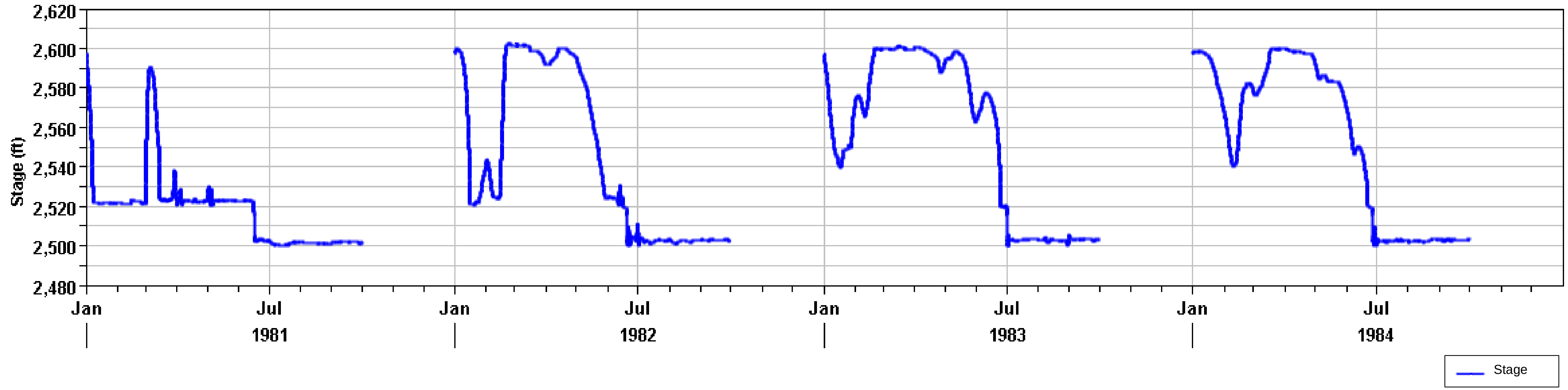

Figure 19: Copco No. 1 Drawdown Stage for years 1981 through 1984

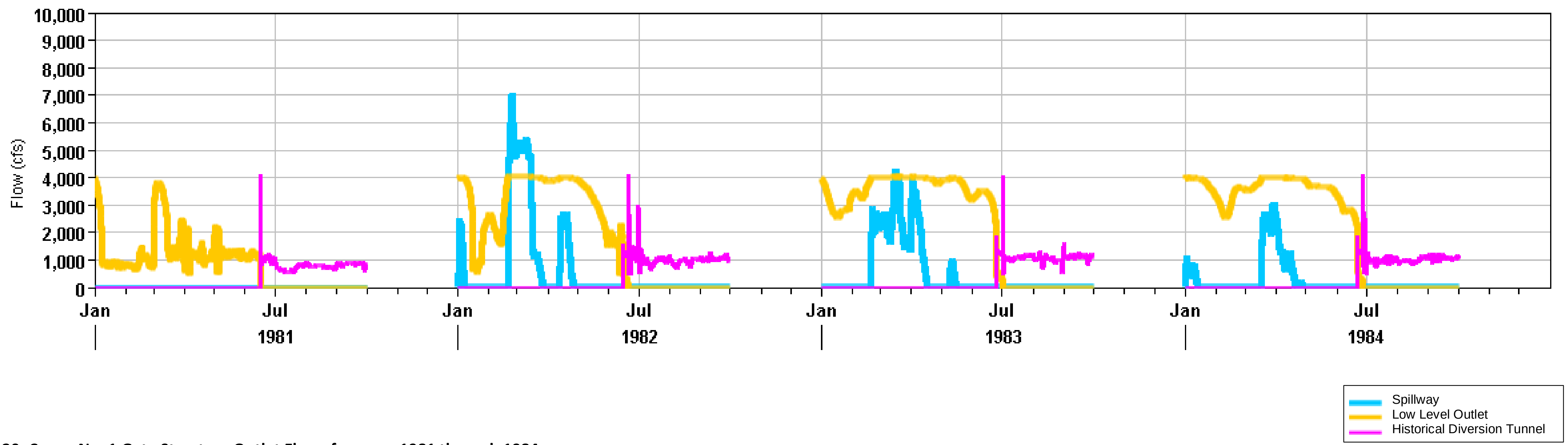

Figure 20: Copco No. 1 Gate Structure Outlet Flows for years 1981 through 1984

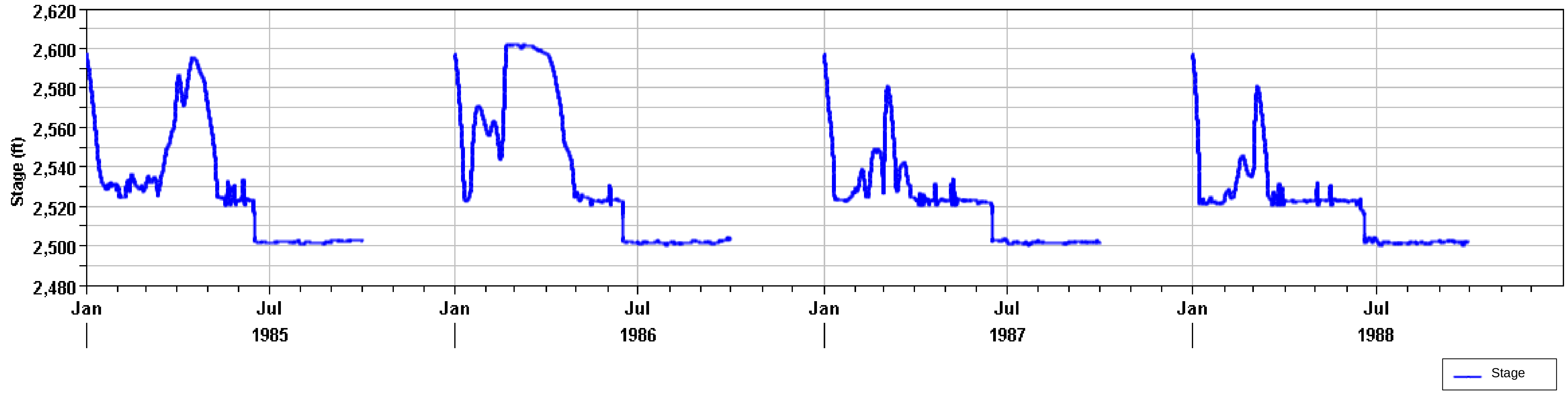

Figure 21: Copco No. 1 Drawdown Stage for years 1985 through 1988

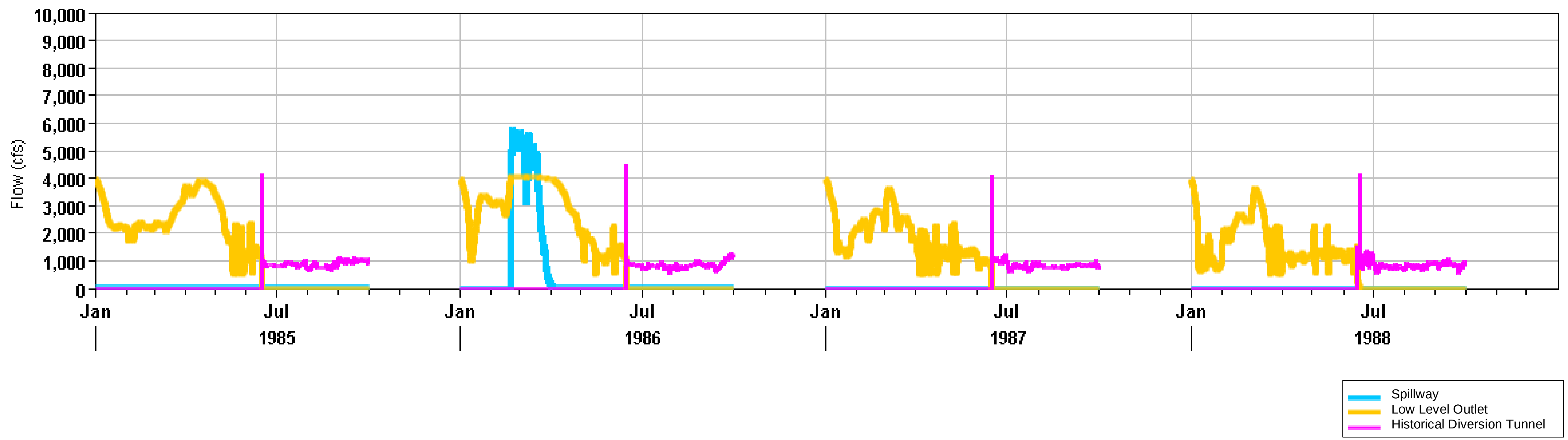

Figure 22: Copco No. 1 Gate Structure Outlet Flows for years 1985 through 1988

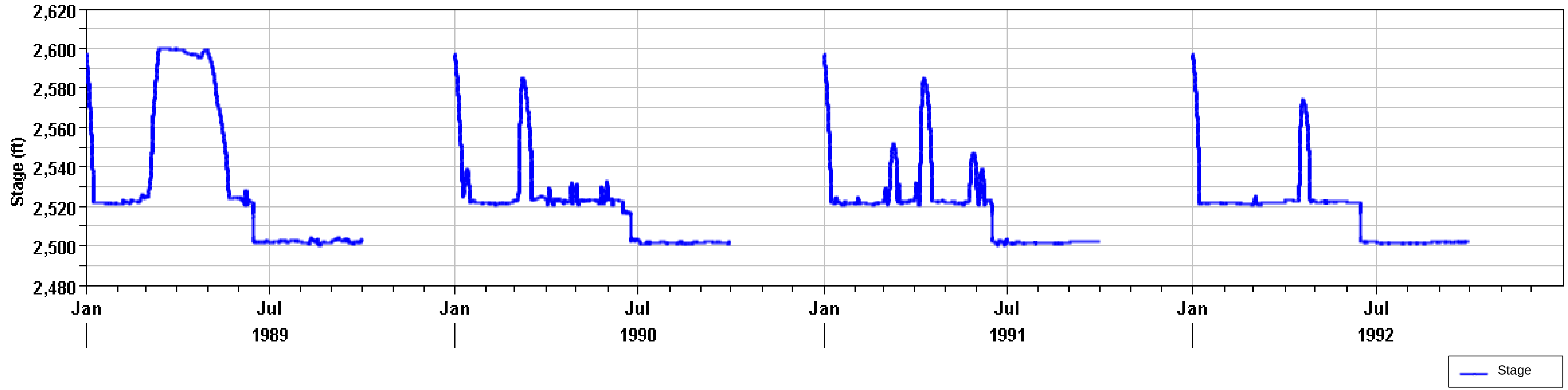

Figure 23: Copco No. 1 Drawdown Stage for years 1989 through 1992

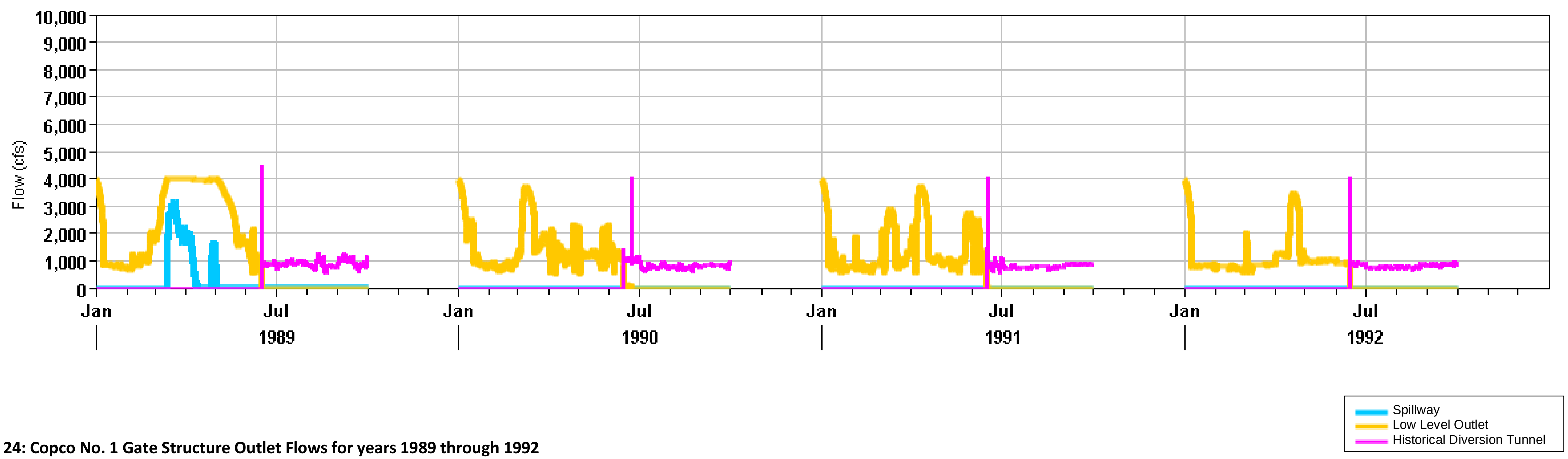

Figure 24: Copco No. 1 Gate Structure Outlet Flows for years 1989 through 1992

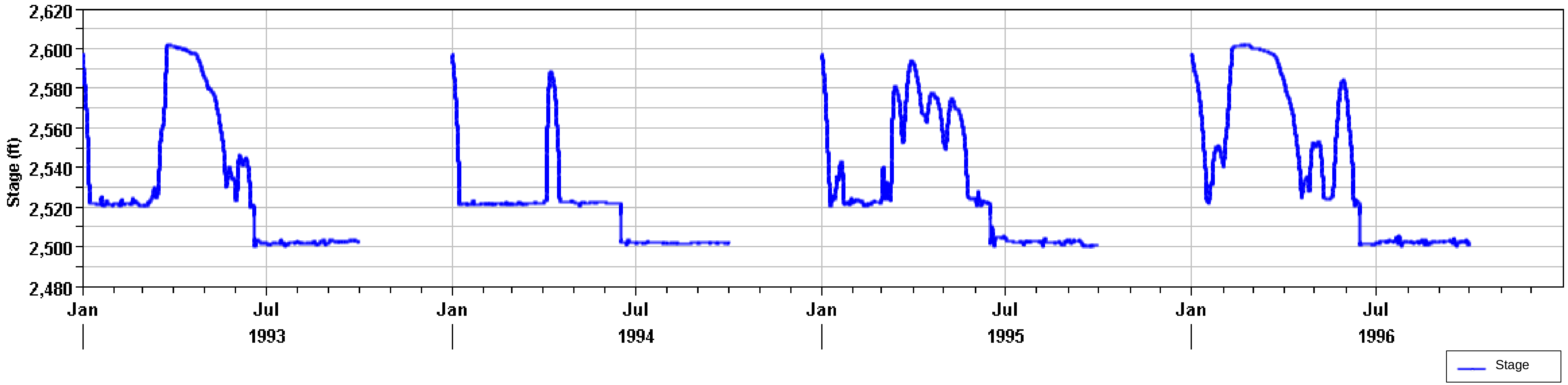

Figure 25: Copco No. 1 Drawdown Stage for years 1993 through 1996

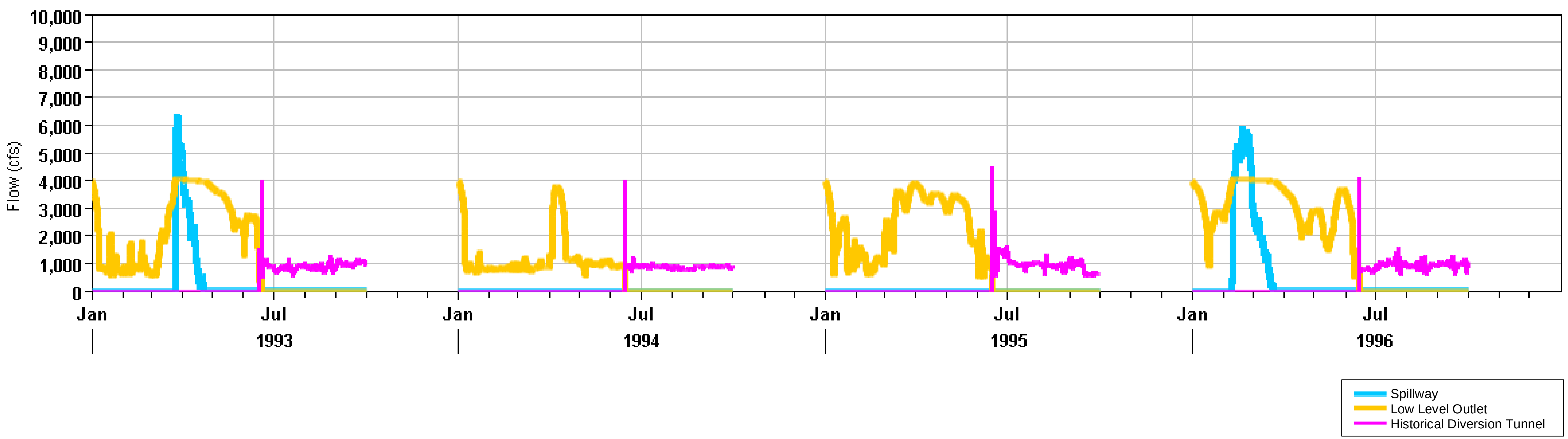

Figure 26: Copco No. 1 Gate Structure Outlet Flows for years 1993 through 1996

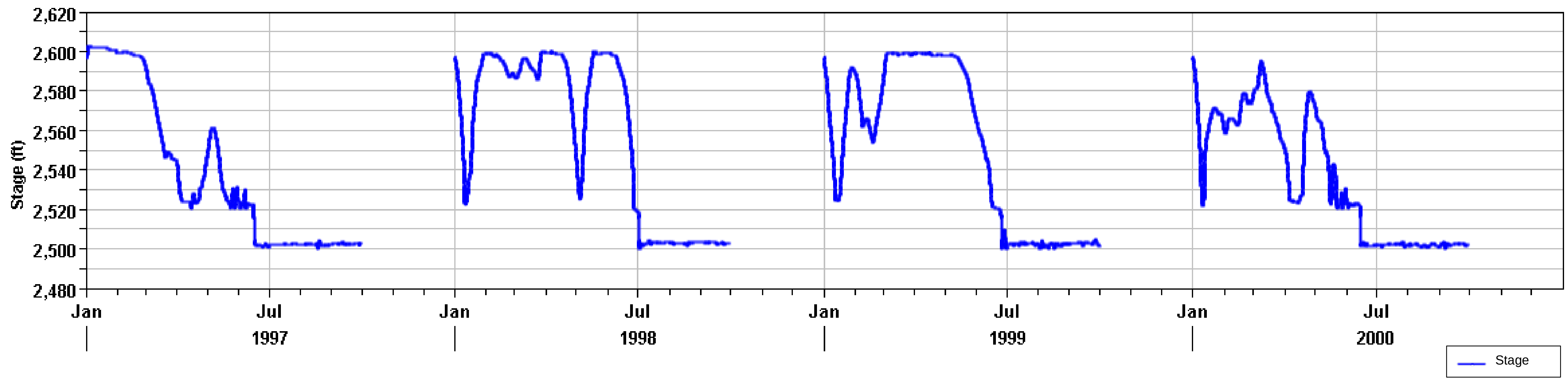

Figure 27: Copco No. 1 Drawdown Stage for years 1997 through 2000

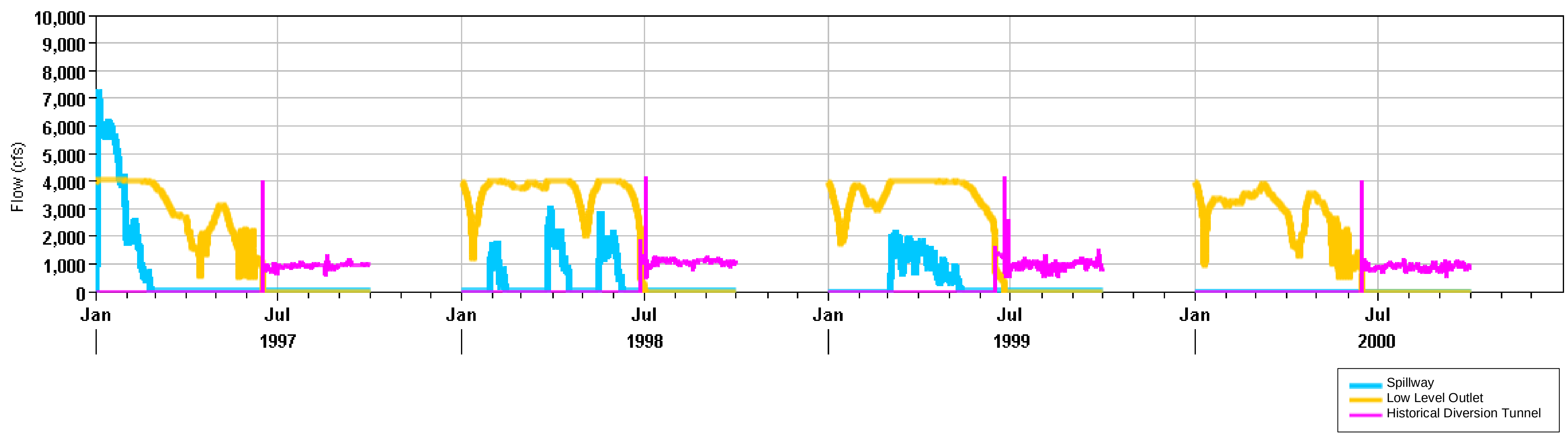

Figure 28: Copco No. 1 Gate Structure Outlet Flows for years 1997 through 2000

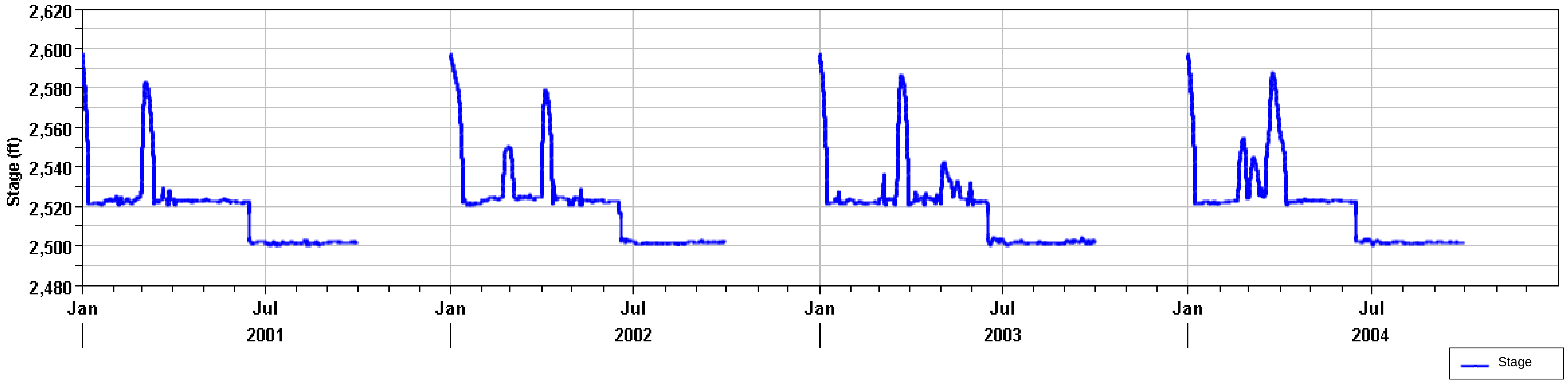

Figure 29: Copco No. 1 Drawdown Stage for years 2001 through 2004

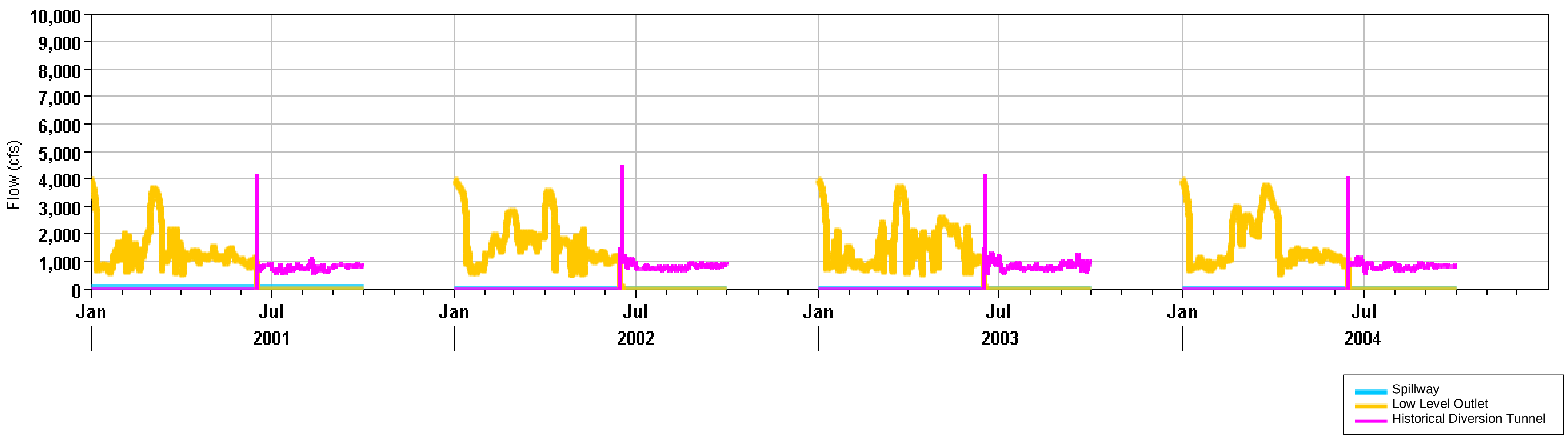

Figure 30: Copco No. 1 Gate Structure Outlet Flows for years 2001 through 2004

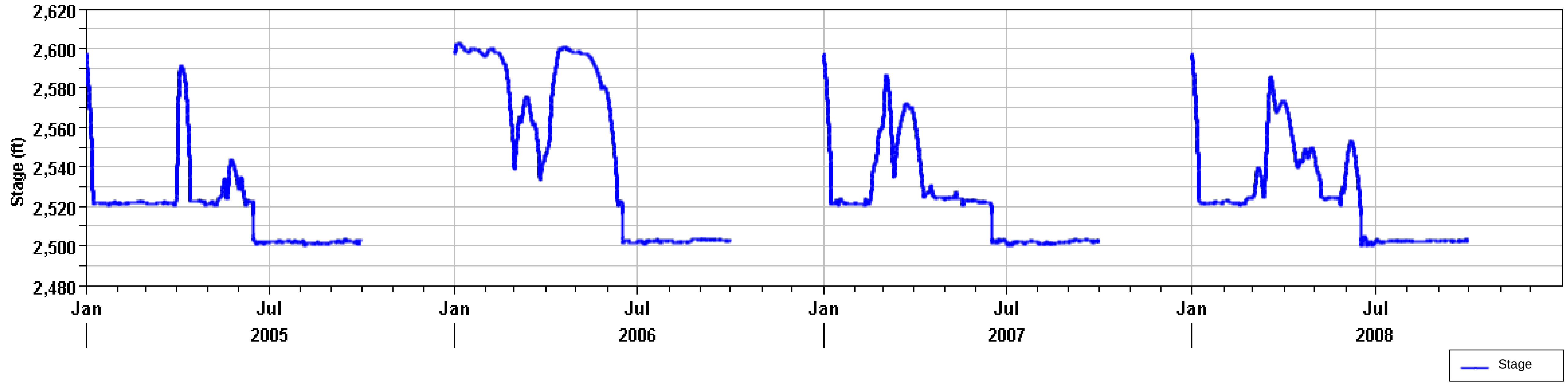

Figure 31: Copco No. 1 Drawdown Stage for years 2005 through 2008

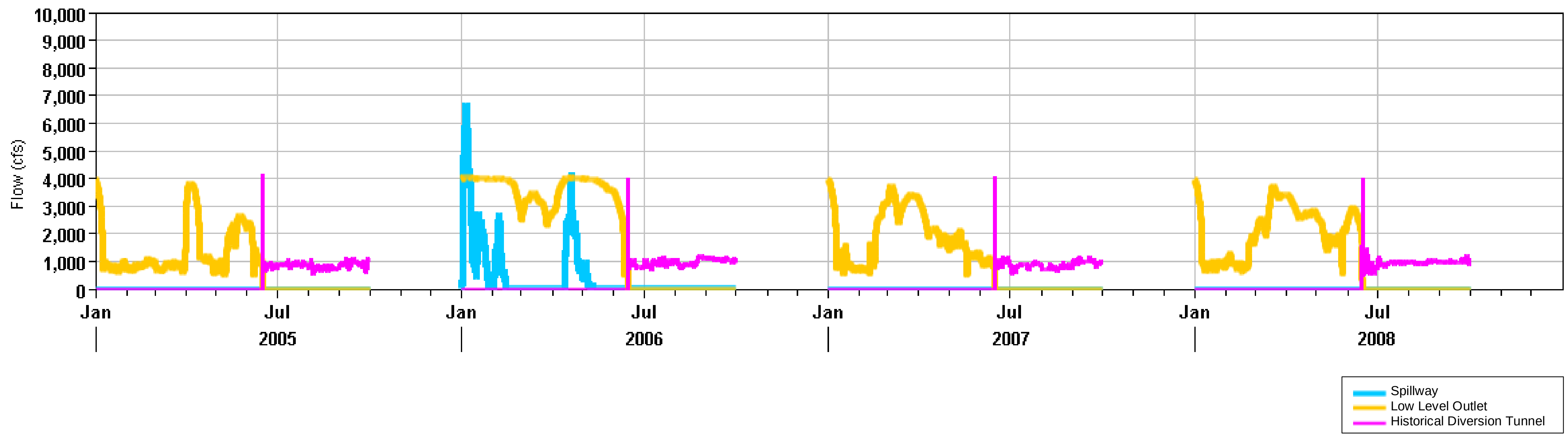

Figure 32: Copco No. 1 Gate Structure Outlet Flows for years 2005 through 2008

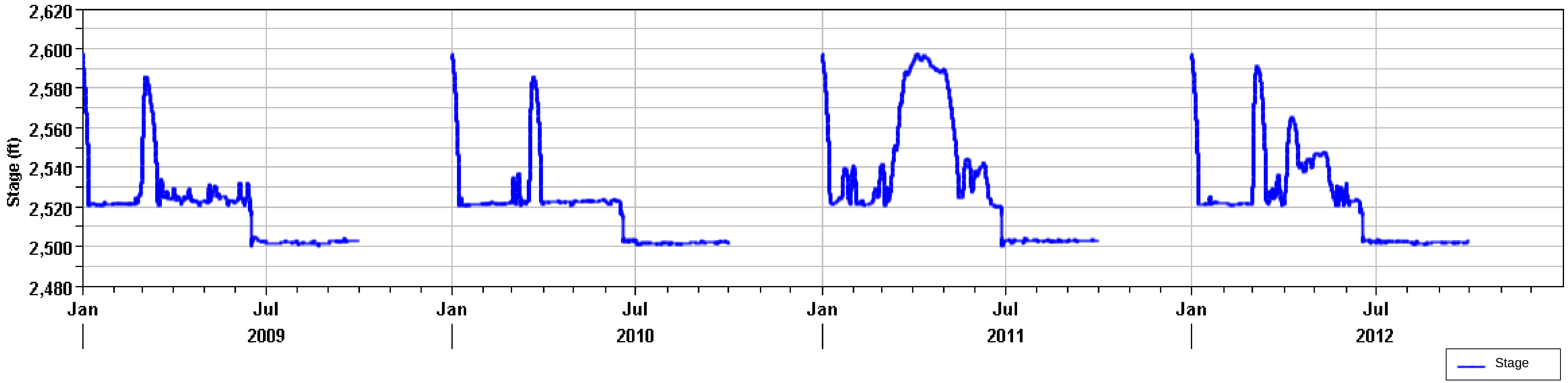

Figure 33: Copco No. 1 Drawdown Stage for years 2009 through 2012

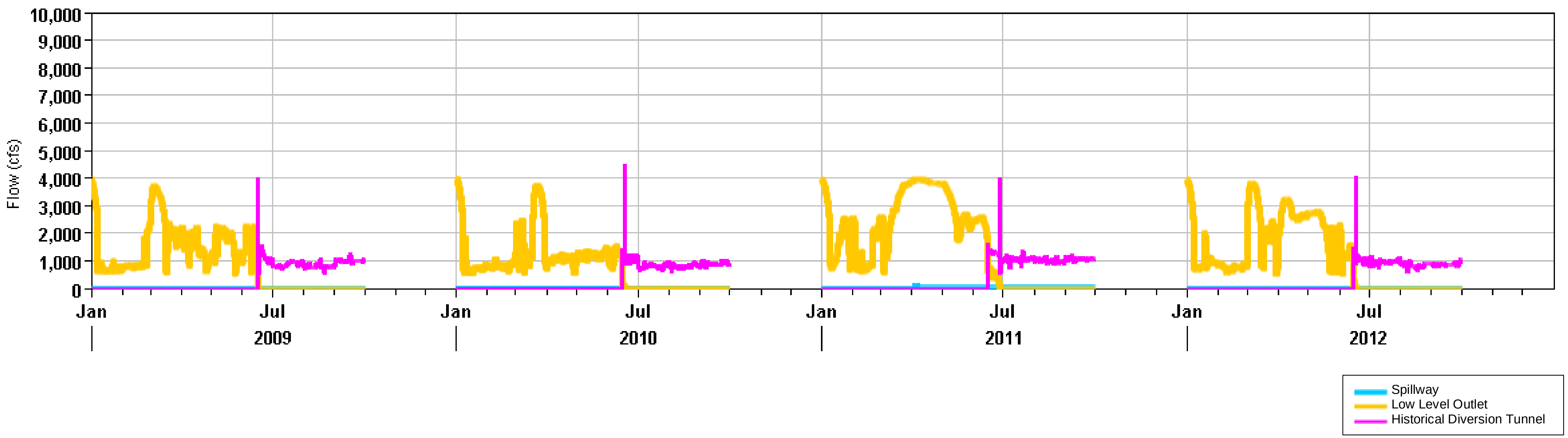

Figure 34: Copco No. 1 Gate Structure Outlet Flows for years 2009 through 2012

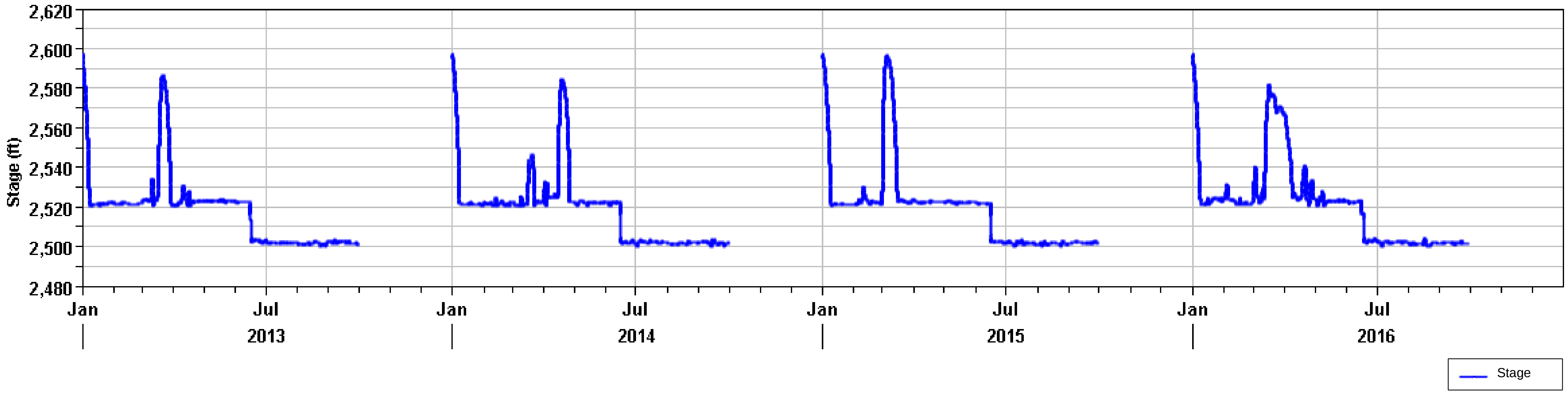

Figure 35: Copco No. 1 Drawdown Stage for years 2013 through 2016

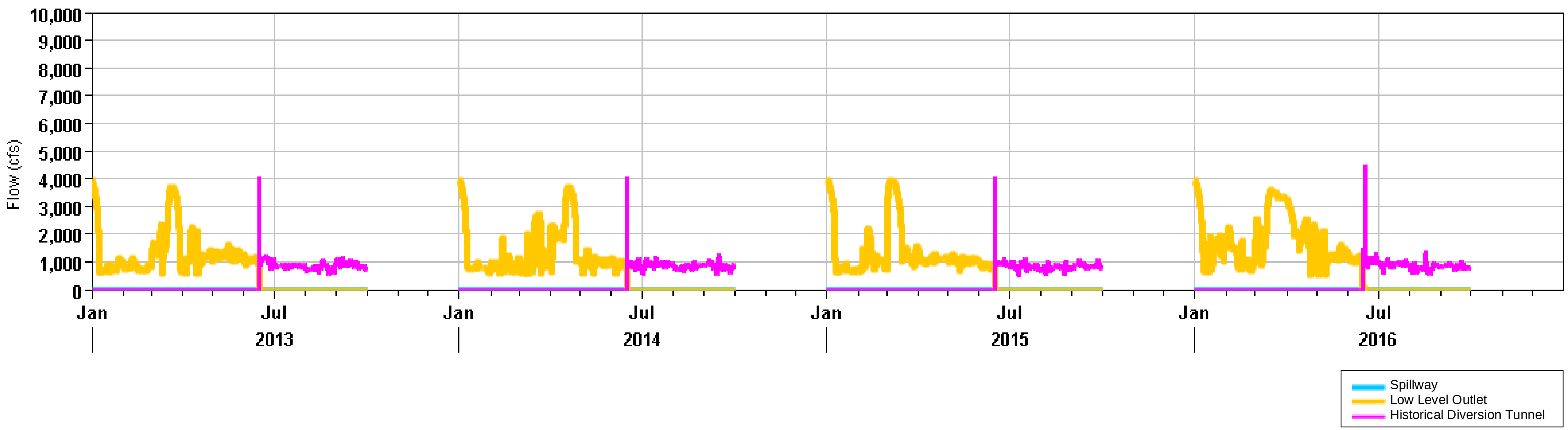

Figure 36: Copco No. 1 Gate Structure Outlet Flows for years 2013 through 2016

Appendix D.

Drawdown Plots for Copco No. 2 Reservoir

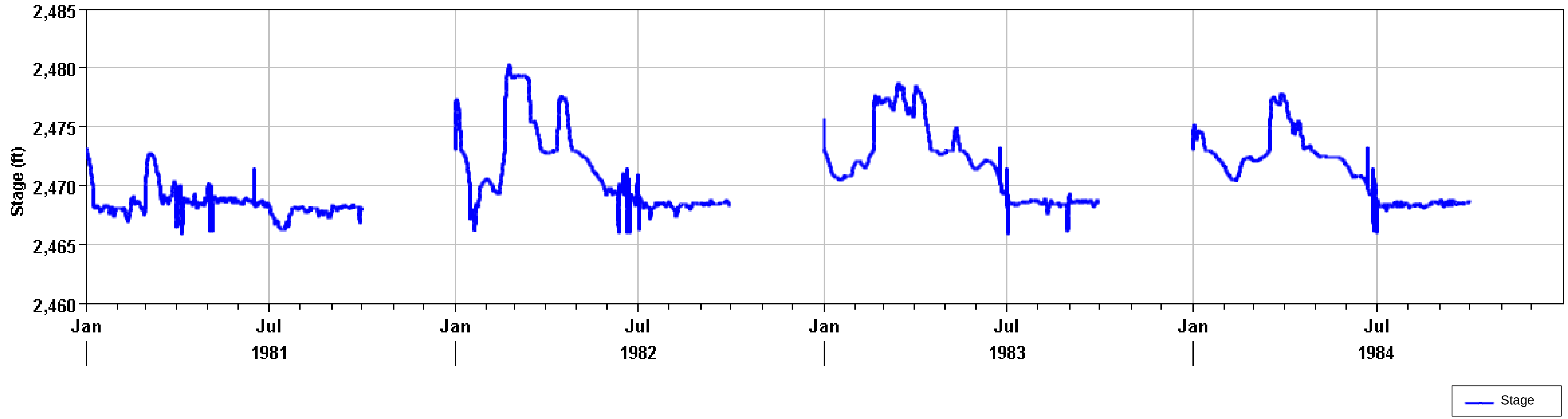

Figure 37: Copco No. 2 Drawdown Stage for years 1981 through 1984

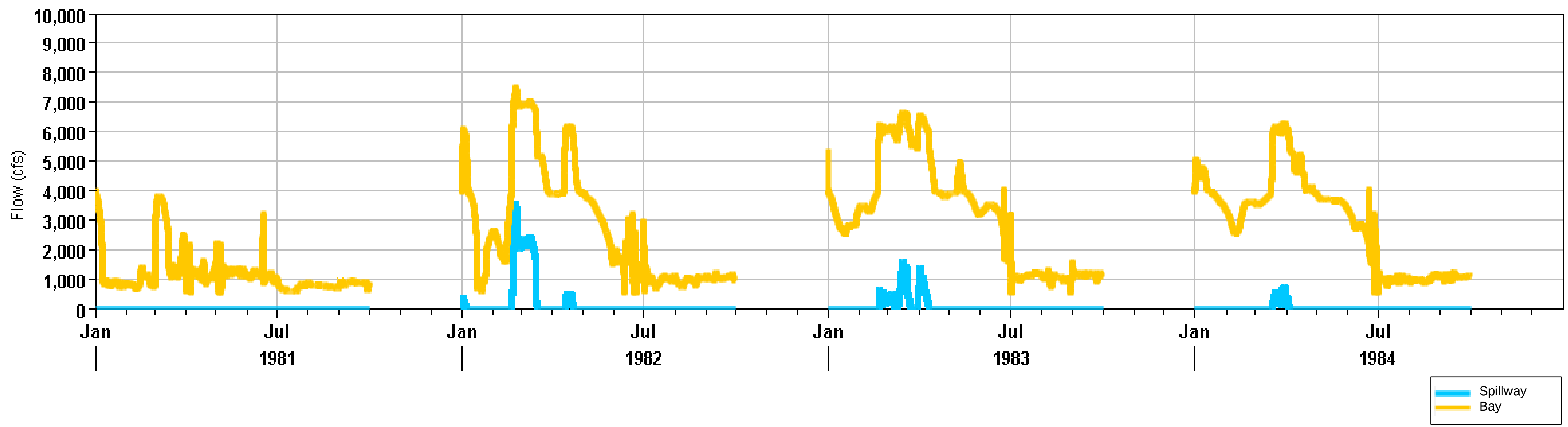

Figure 38: Copco No. 2 Gate Structure Outlet Flows for years 1981 through 1984

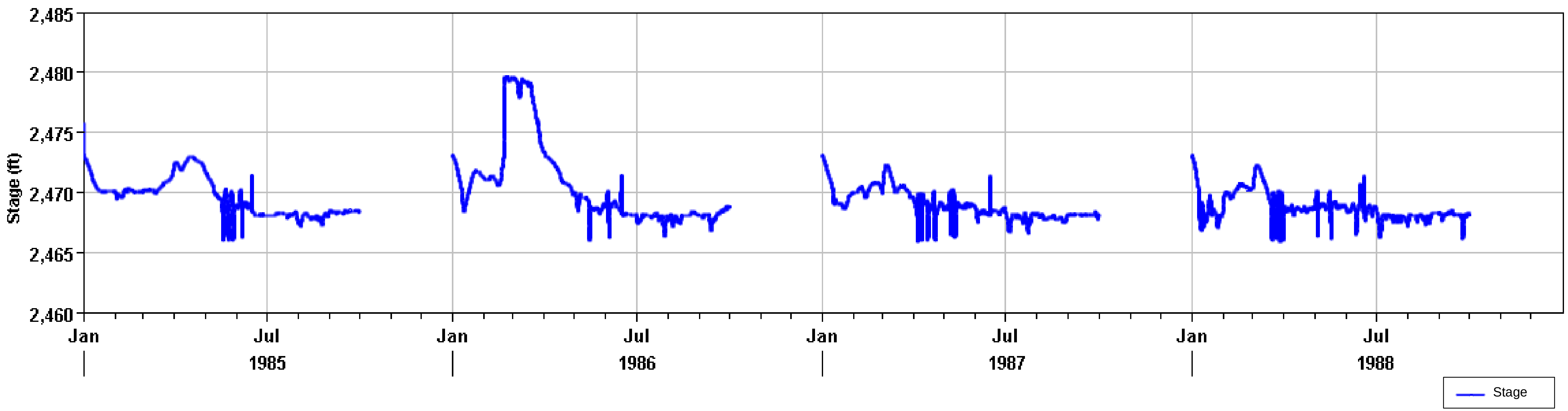

Figure 39: Copco No. 2 Drawdown Stage for years 1985 through 1988

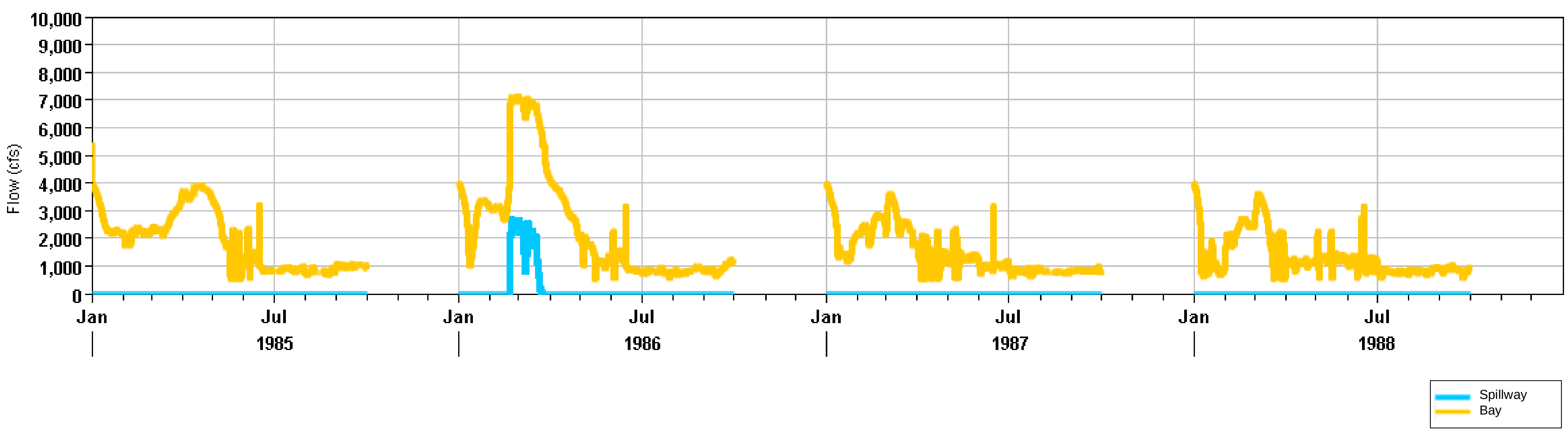

Figure 40: Copco No. 2 Gate Structure Outlet Flows for years 1985 through 1988

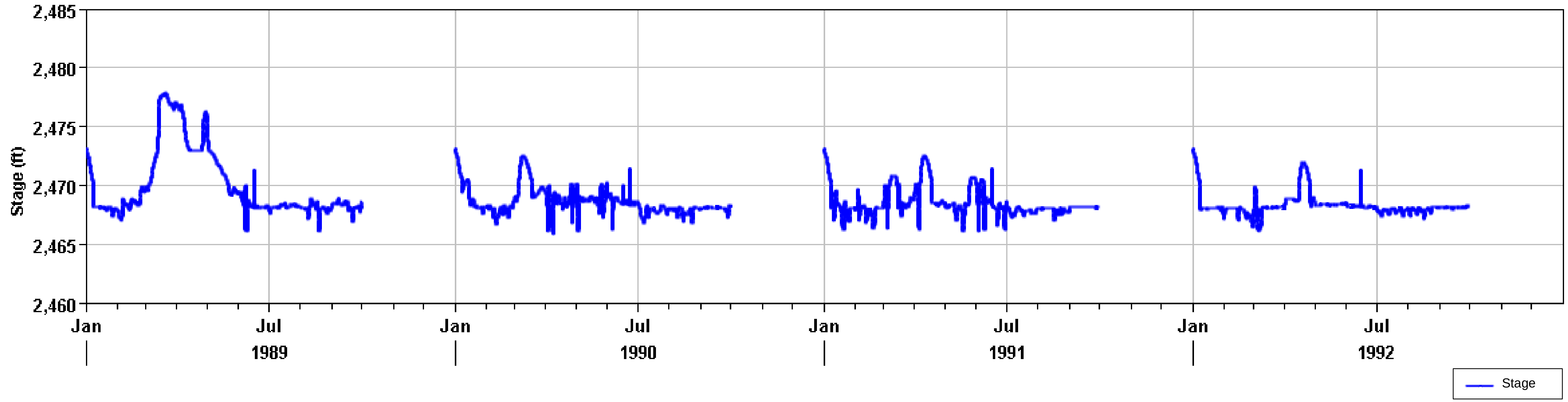

Figure 41: Copco No. 2 Drawdown Stage for years 1989 through 1992

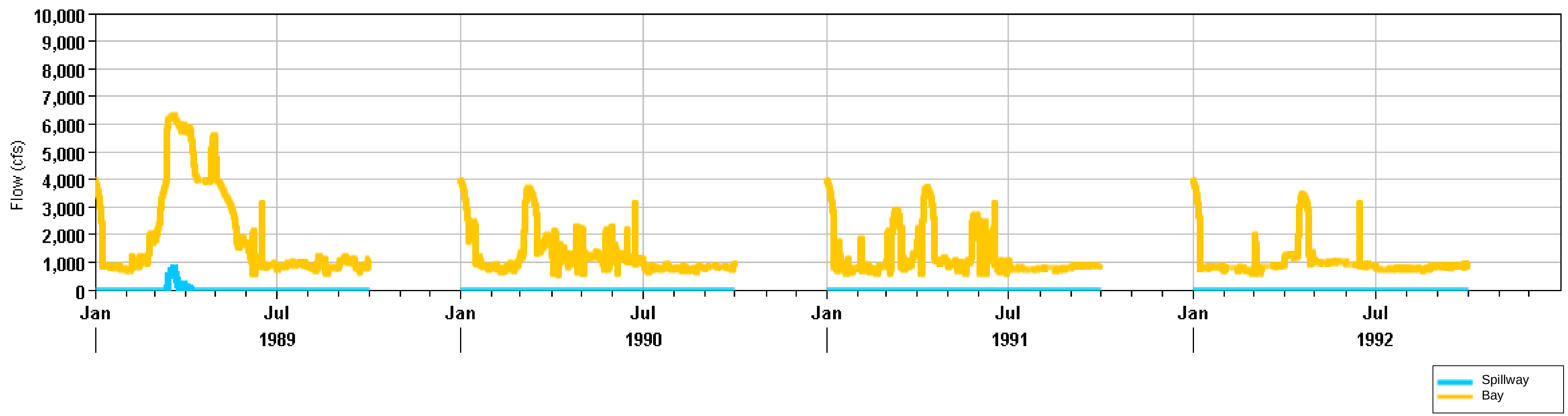

Figure 42: Copco No. 2 Gate Structure Outlet Flows for years 1989 through 1992

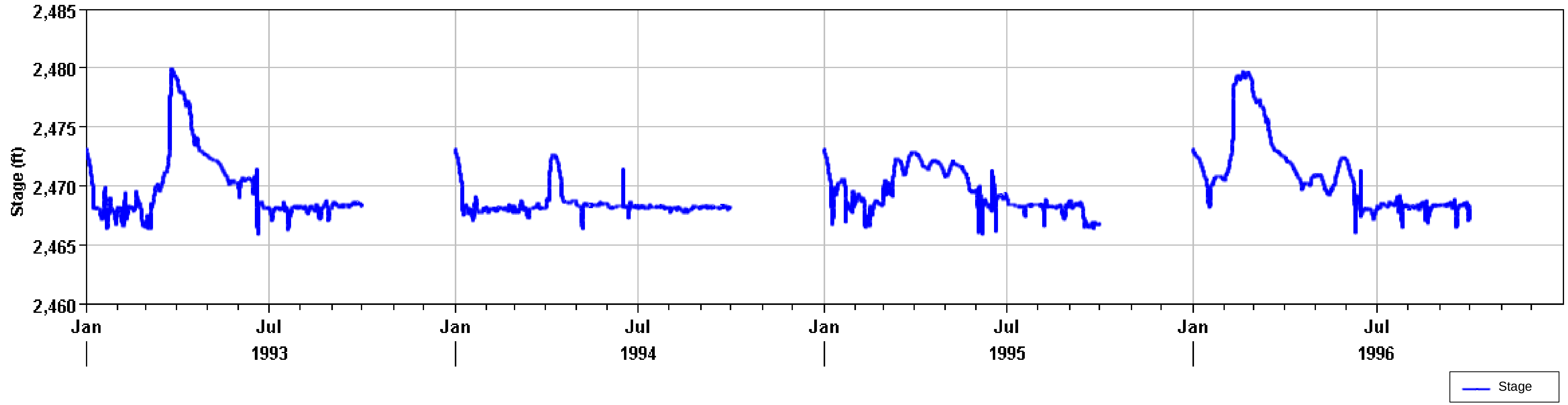

Figure 43: Copco No. 2 Drawdown Stage for years 1993 through 1996

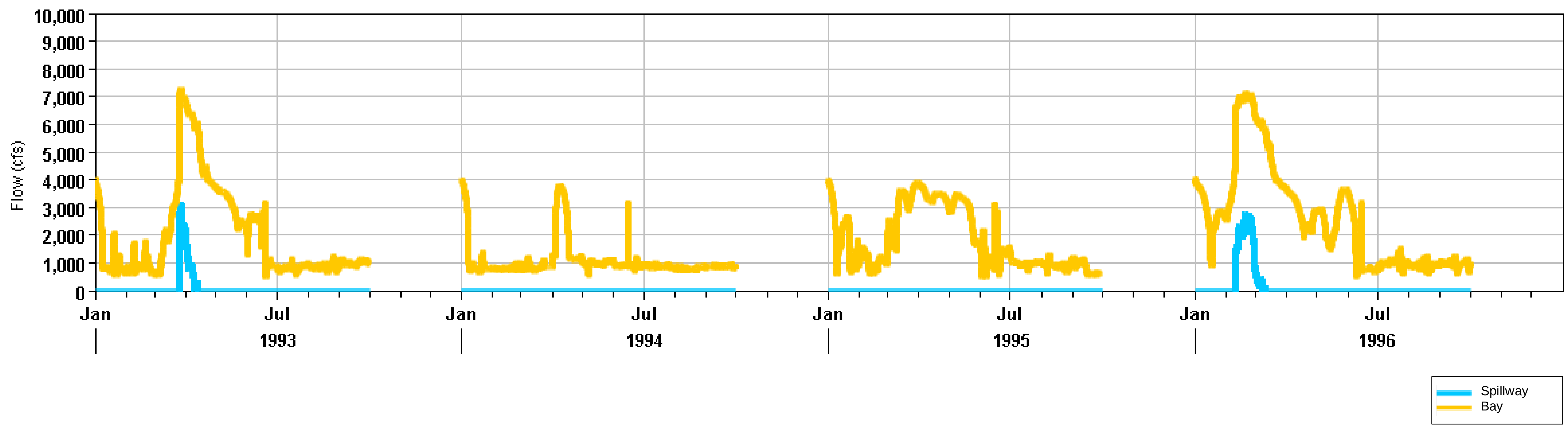

Figure 44: Copco No. 2 Gate Structure Outlet Flows for years 1993 through 1996

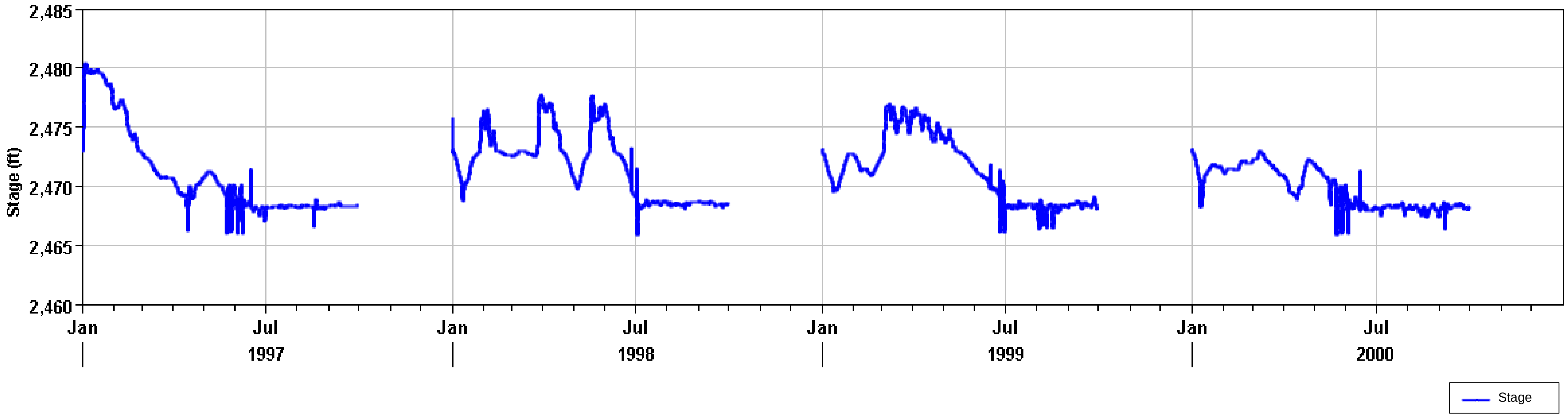

Figure 45: Copco No. 2 Drawdown Stage for years 1997 through 2000

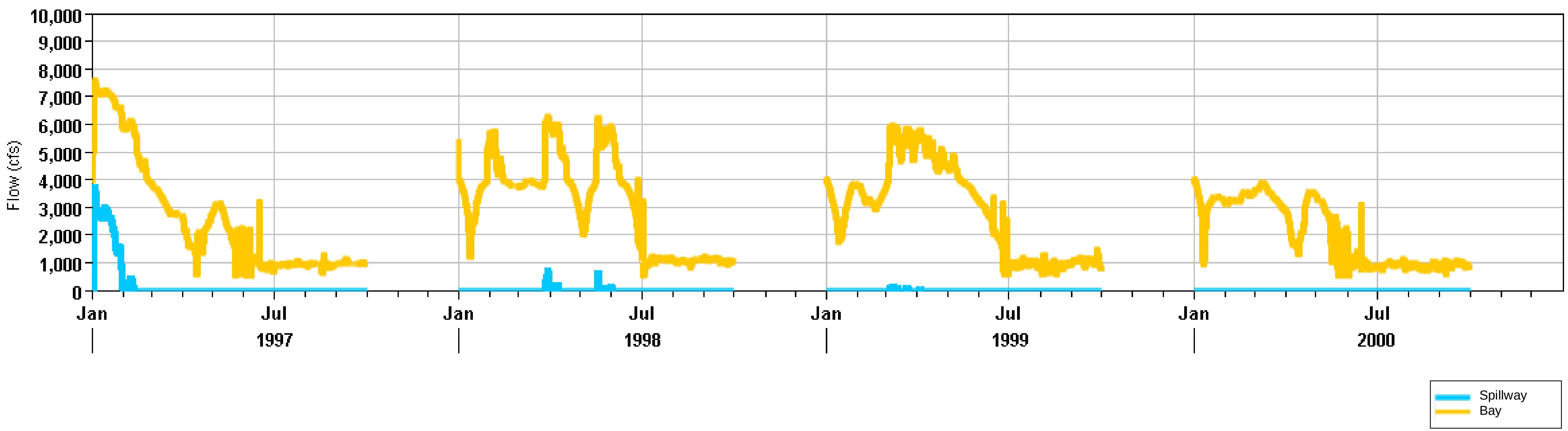

Figure 46: Copco No. 2 Gate Structure Outlet Flows for years 1997 through 2000

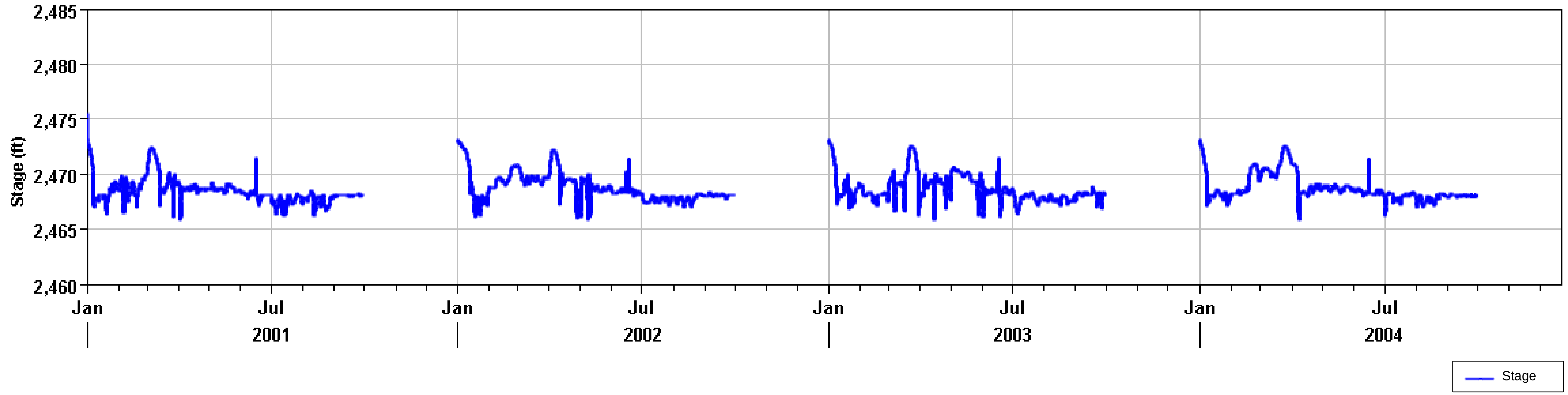

Figure 47: Copco No. 2 Drawdown Stage for years 2001 through 2004

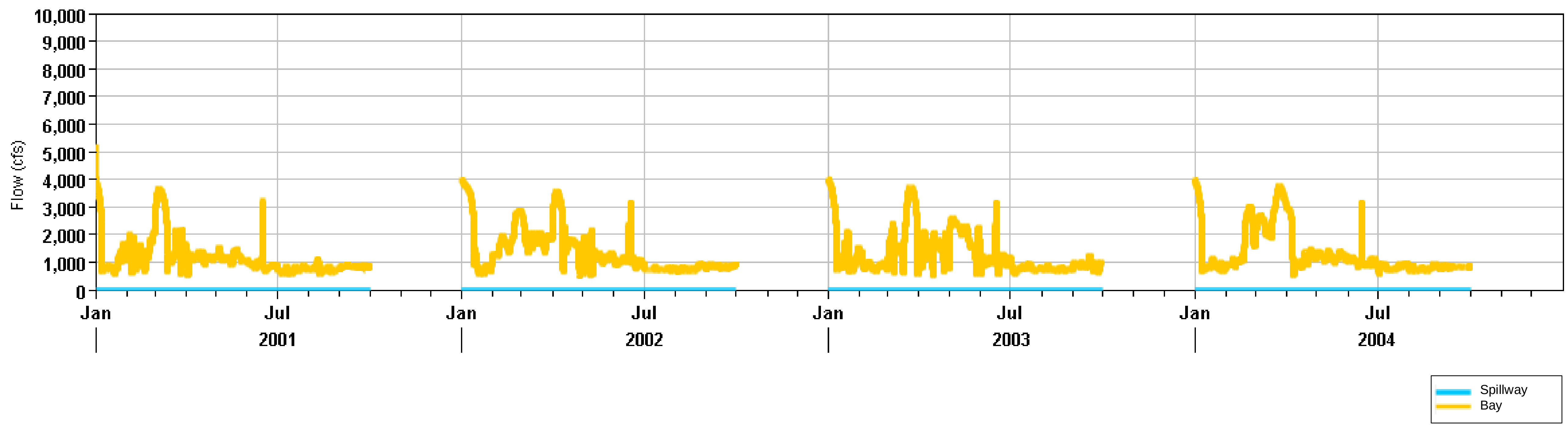

Figure 48: Copco No. 2 Gate Structure Outlet Flows for years 2001 through 2004

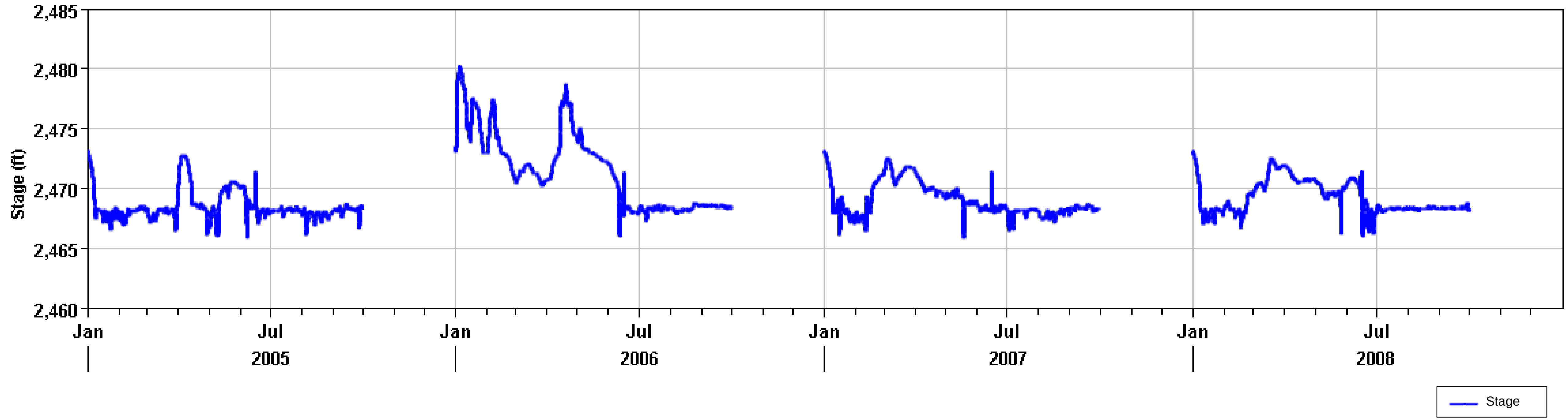

Figure 49: Copco No. 2 Drawdown Stage for years 2005 through 2008

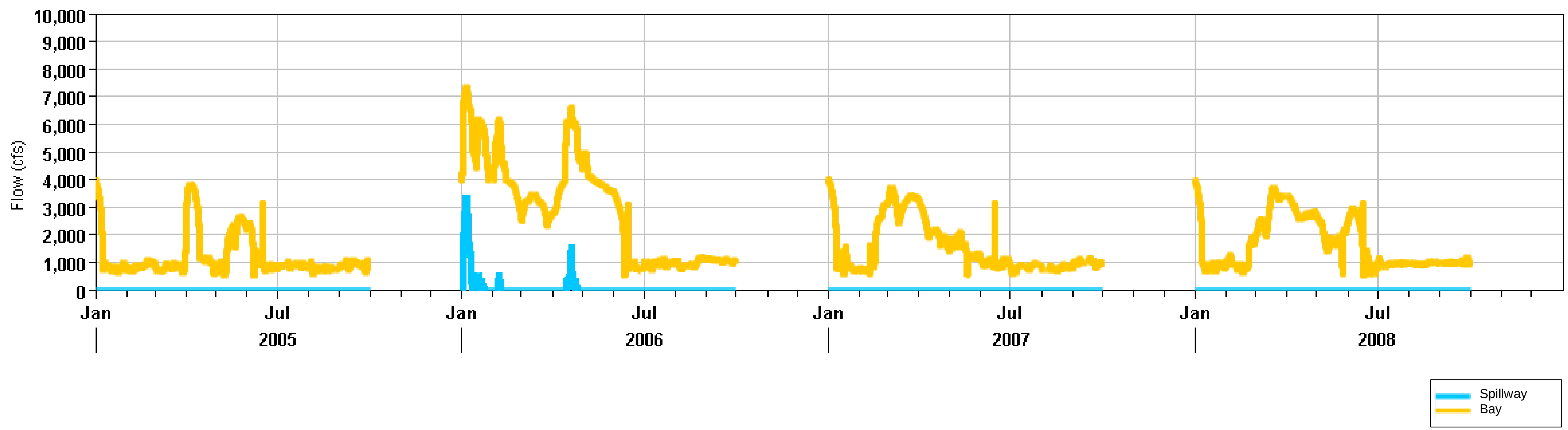

Figure 50: Copco No. 2 Gate Structure Outlet Flows for years 2005 through 2008

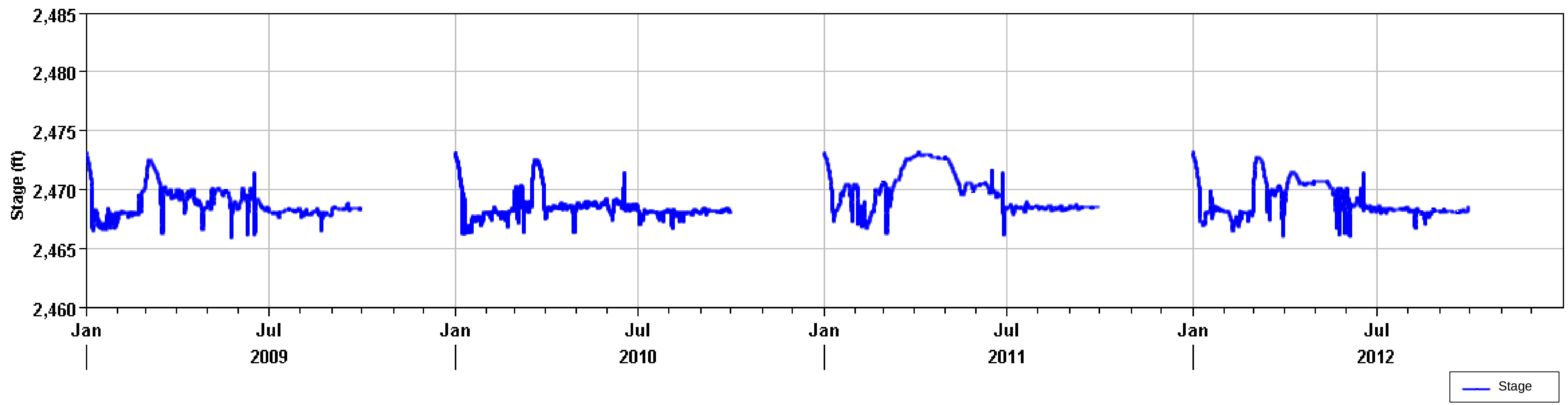

Figure 51: Copco No. 2 Drawdown Stage for years 2009 through 2012

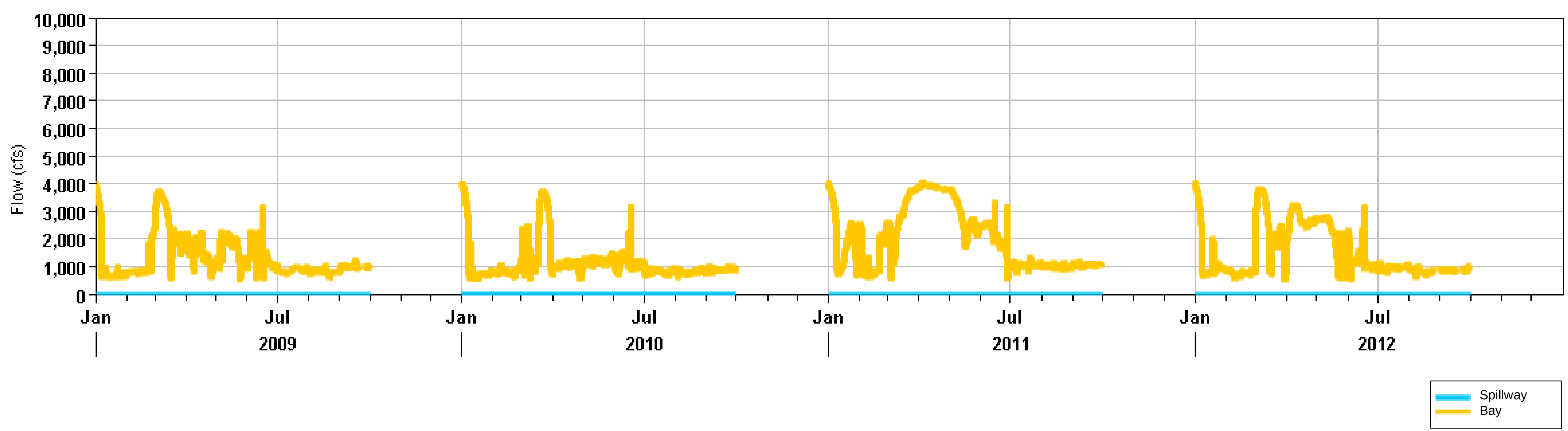

Figure 52: Copco No. 2 Gate Structure Outlet Flows for years 2009 through 2012

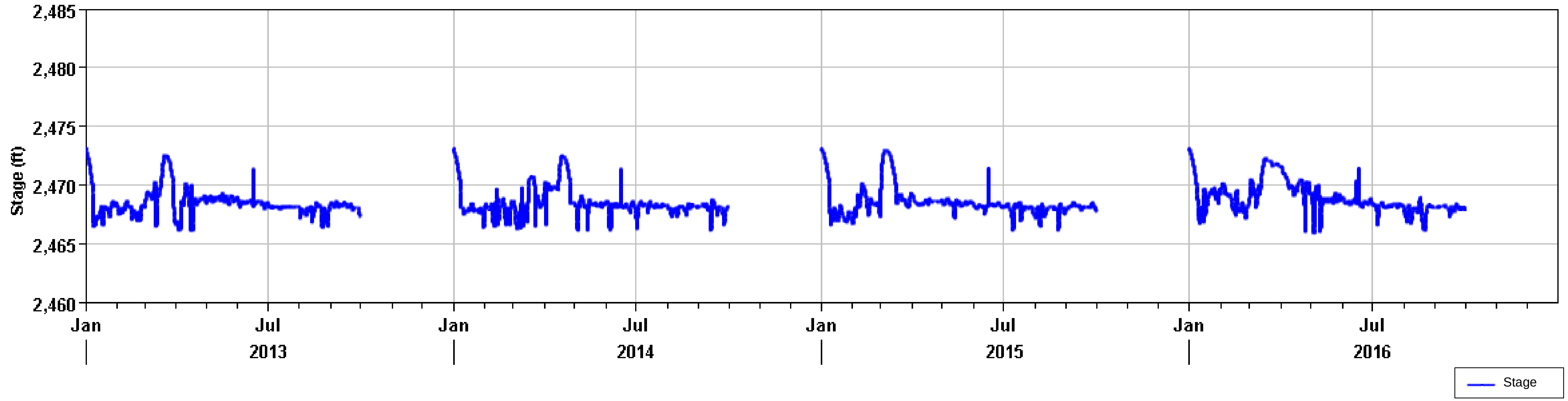

Figure 53: Copco No. 2 Drawdown Stage for years 2013 through 2016

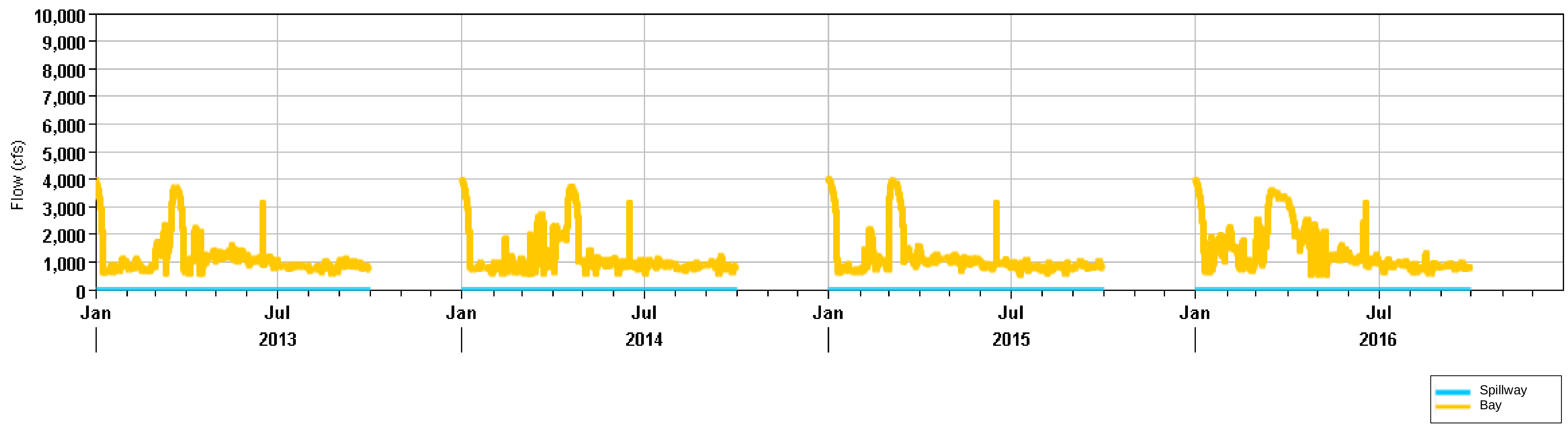

Figure 54: Copco No. 2 Gate Structure Outlet Flows for years 2013 through 2016

Appendix E:

Drawdown Plots for Iron Gate Reservoir

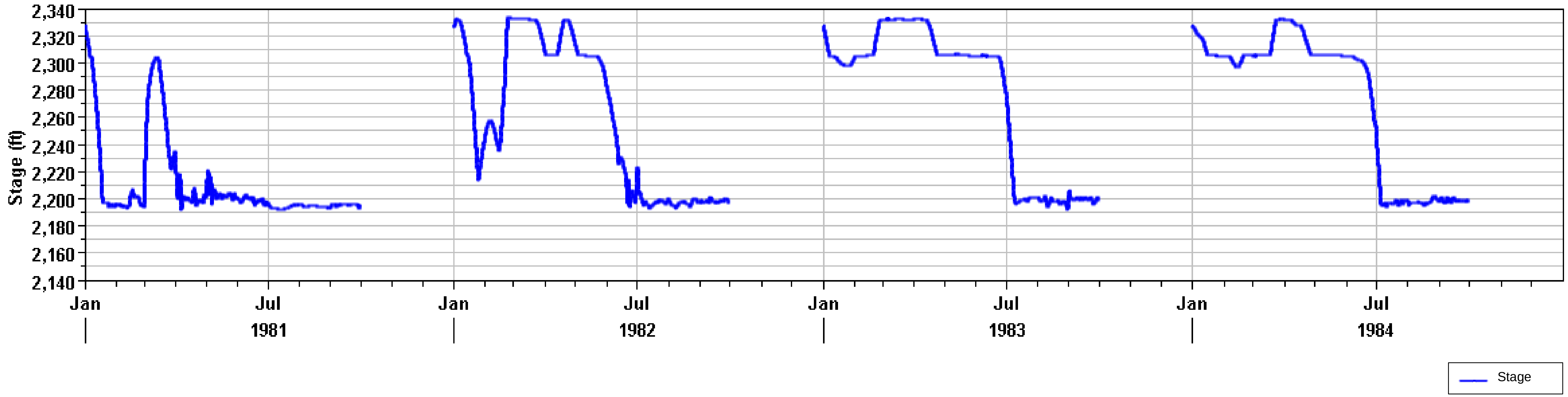

Figure 55: Iron Gate Drawdown Stage for years 1981 through 1984

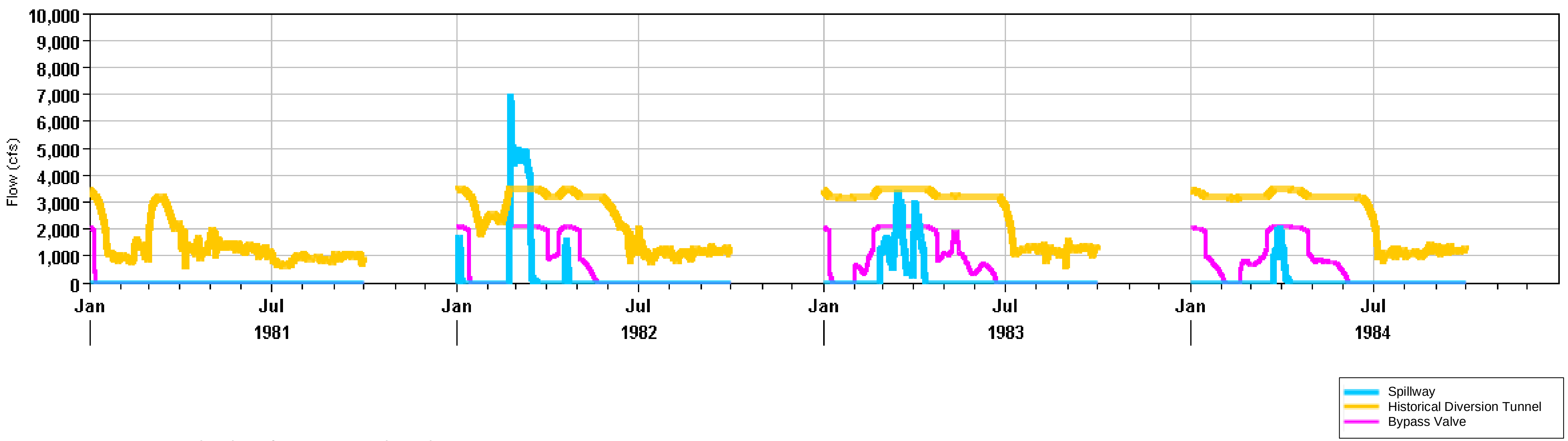

Figure 56: Iron Gate Structure Outlet Flows for years 1981 through 1984

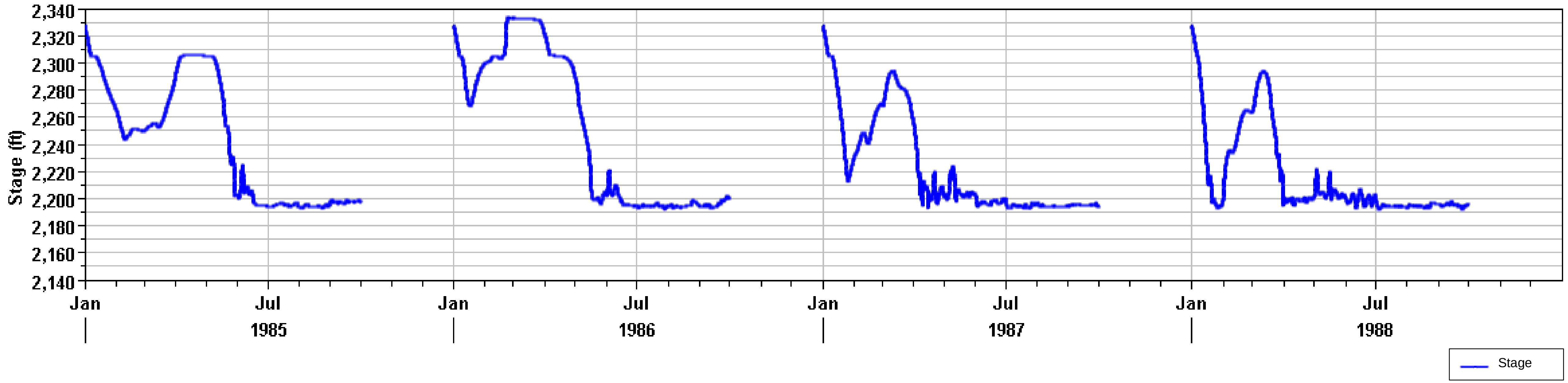

Figure 57: Iron Gate Drawdown Stage for years 1985 through 1988

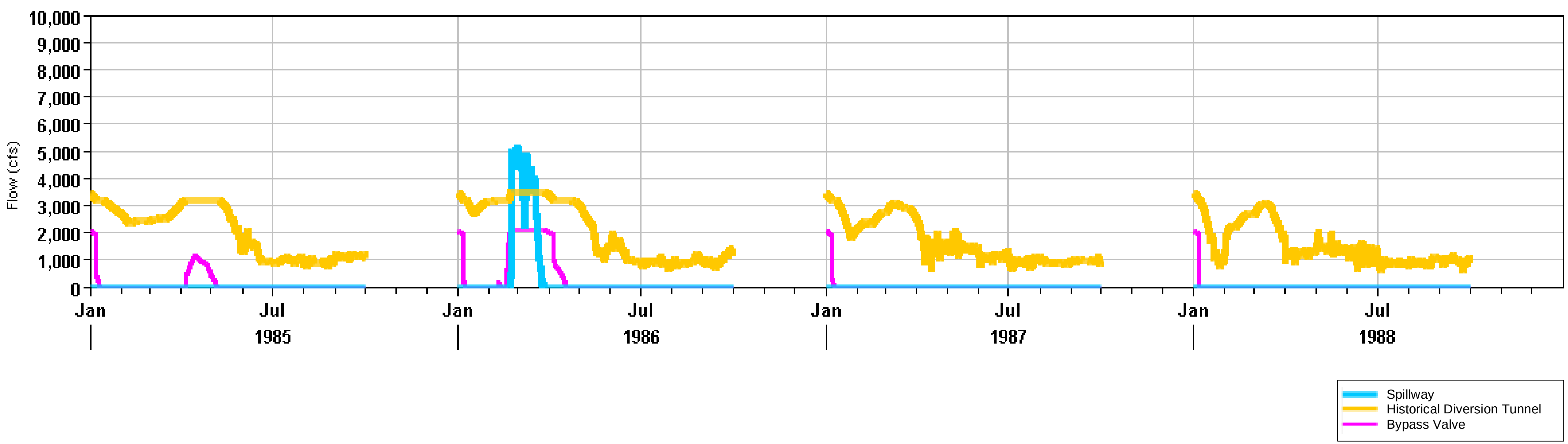

Figure 58: Iron Gate Structure Outlet Flows for years 1985 through 1988

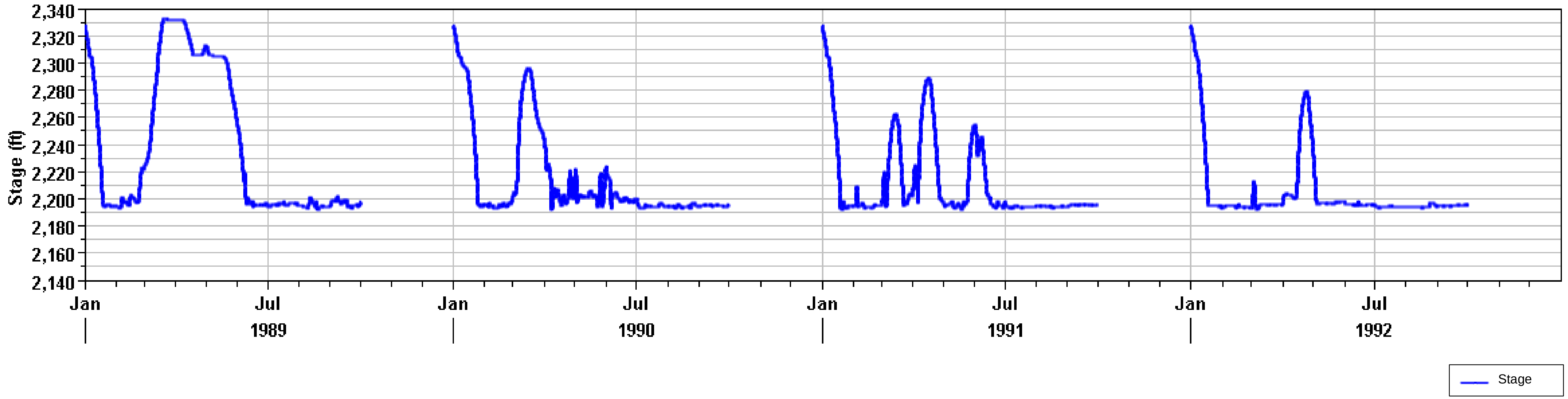

Figure 59: Iron Gate Drawdown Stage for years 1989 through 1992

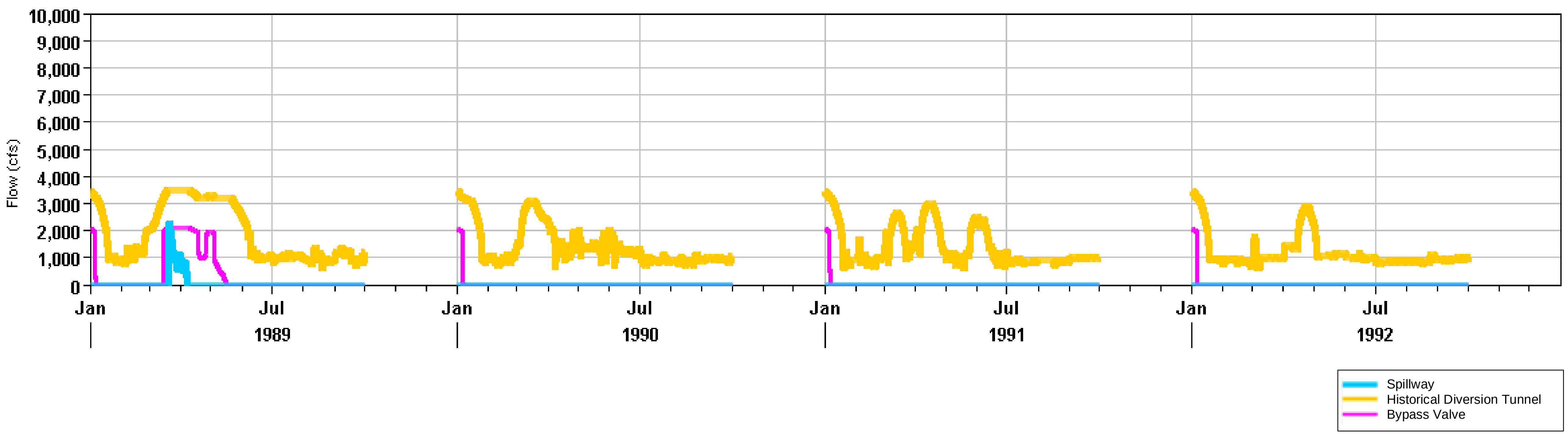

Figure 60 Iron Gate Structure Outlet Flows for years 1989 through 1992

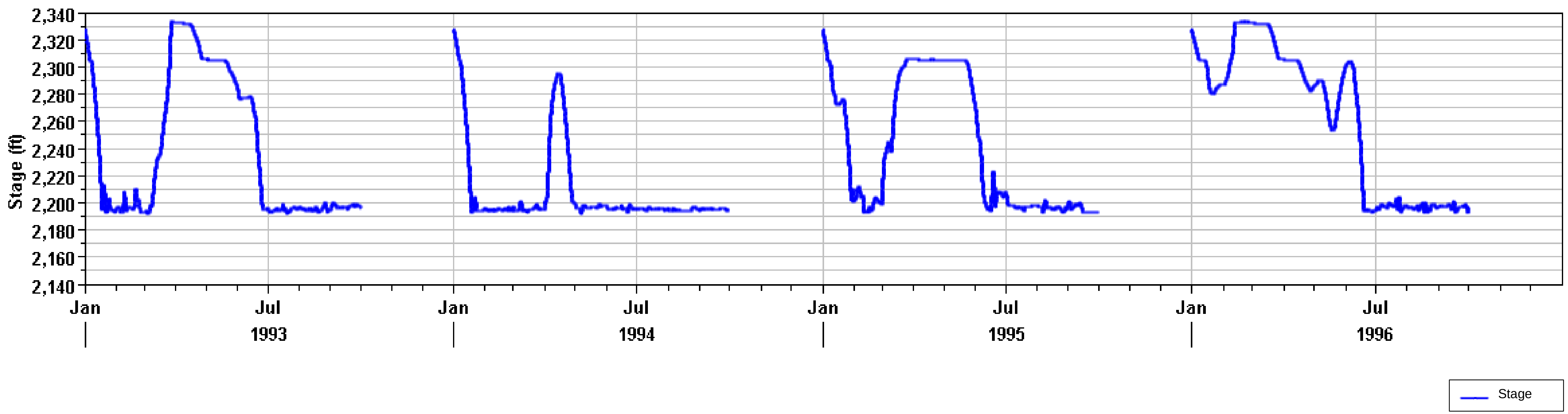

Figure 61: Iron Gate Drawdown Stage for years 1993 through 1996

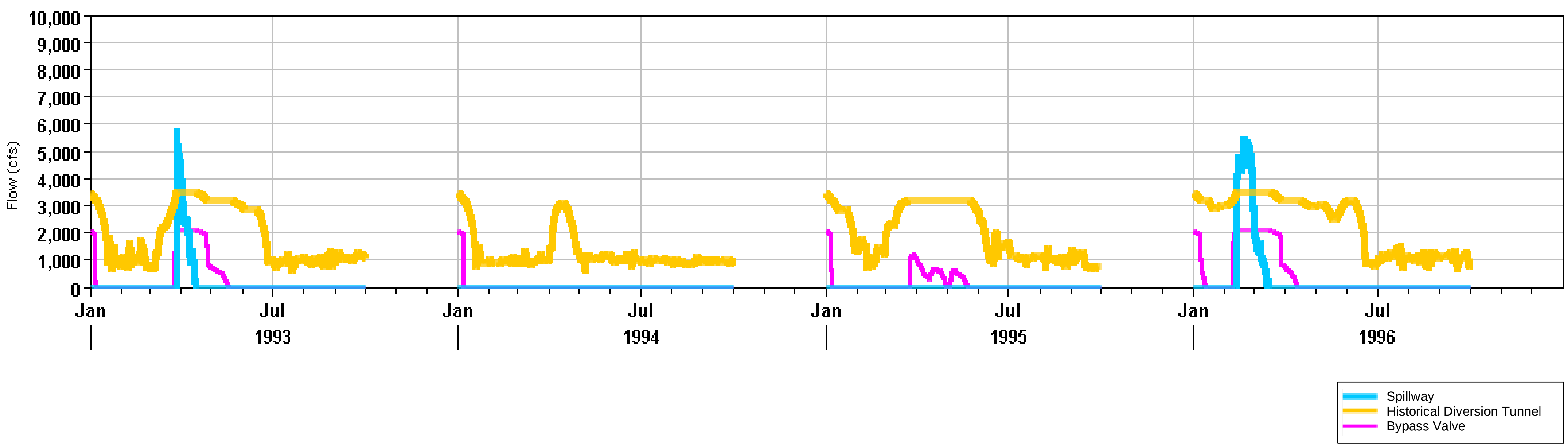

Figure 62: Iron Gate Structure Outlet Flows for years1993 through 1996

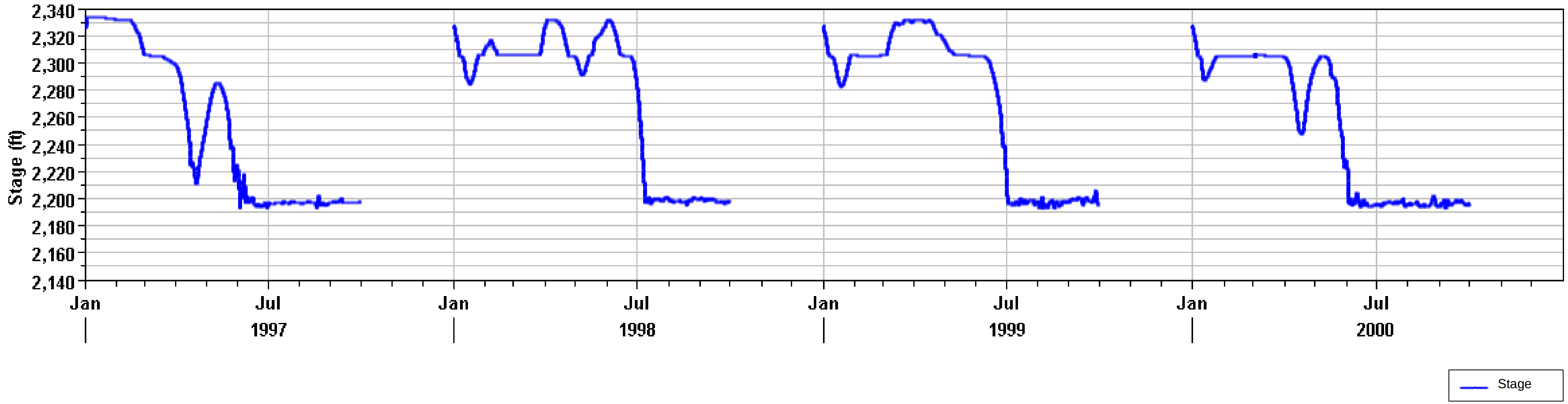

Figure 63: Iron Gate Drawdown Stage for years 1997 through 2000

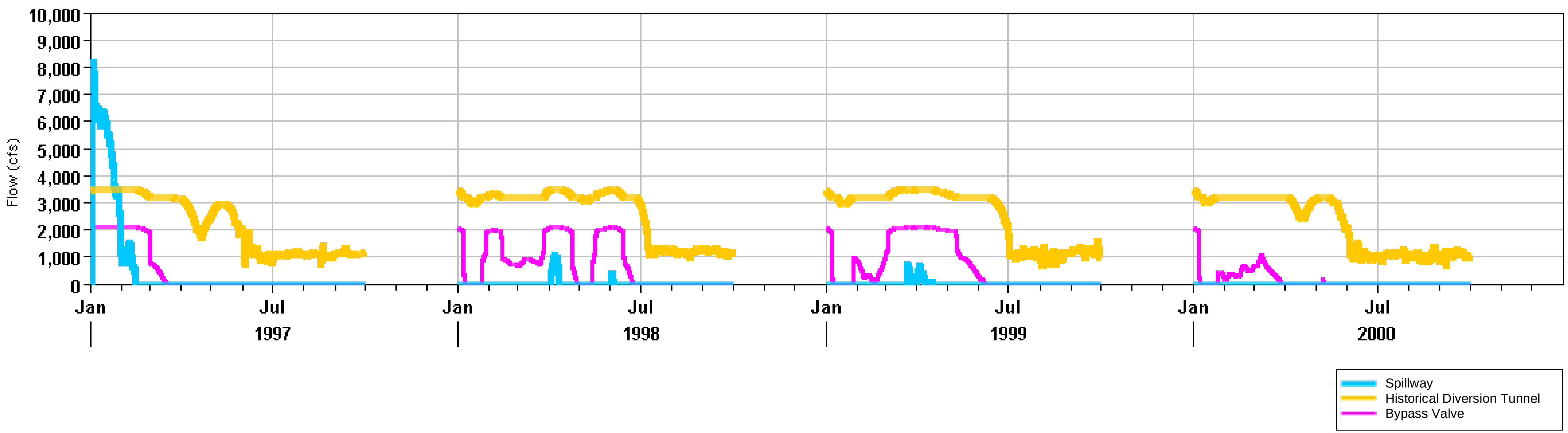

Figure 64: Iron Gate Structure Outlet Flows for years 1997 through 2000

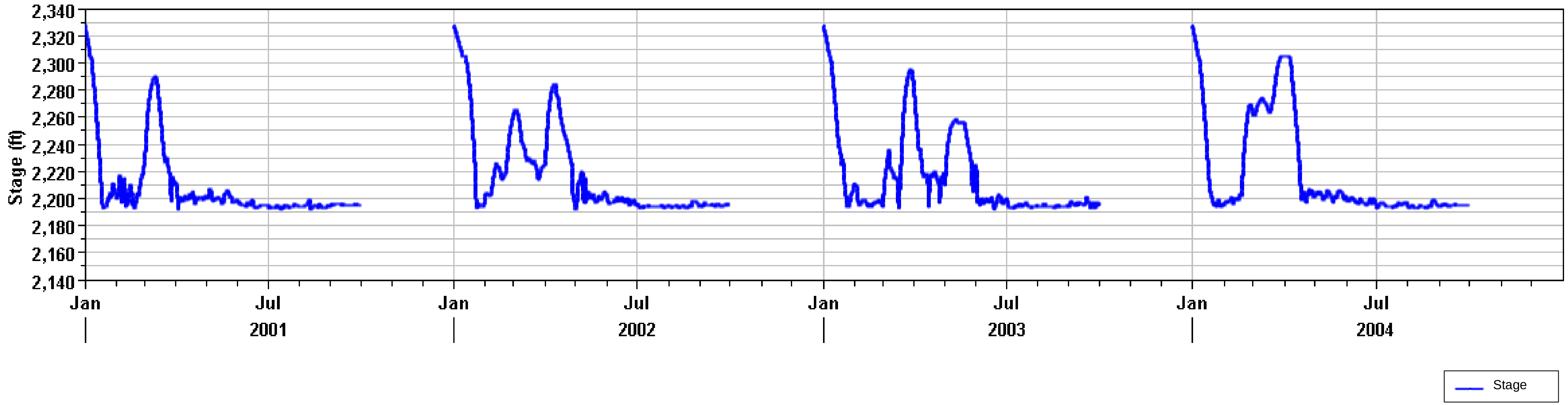

Figure 65: Iron Gate Drawdown Stage for years 2001 through 2004

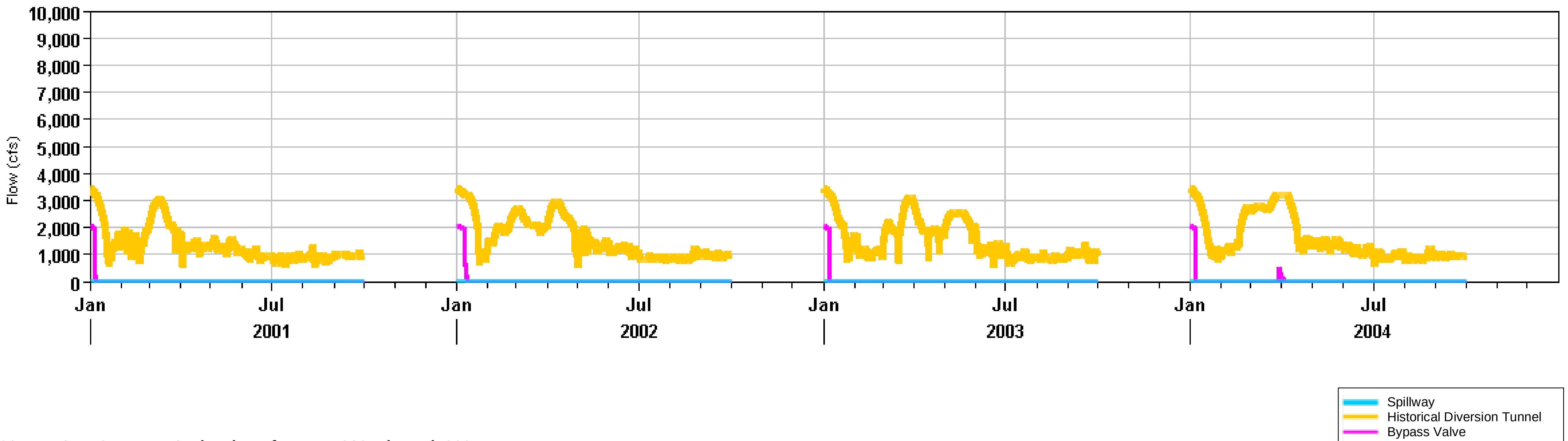

Figure 66: Iron Gate Structure Outlet Flows for years 2001 through 2004

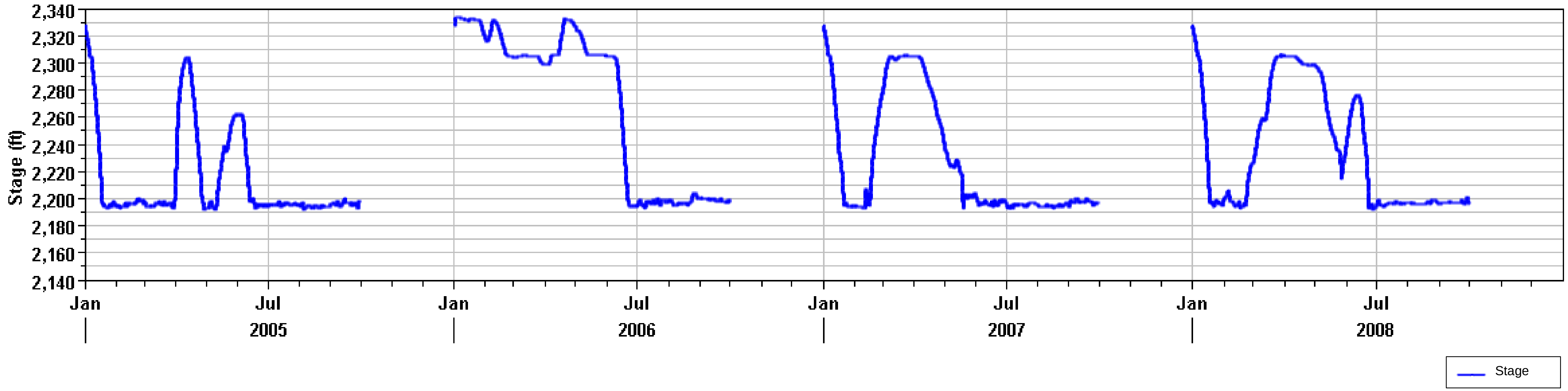

Figure 67: Iron Gate Drawdown Stage for years 2005 through 2008

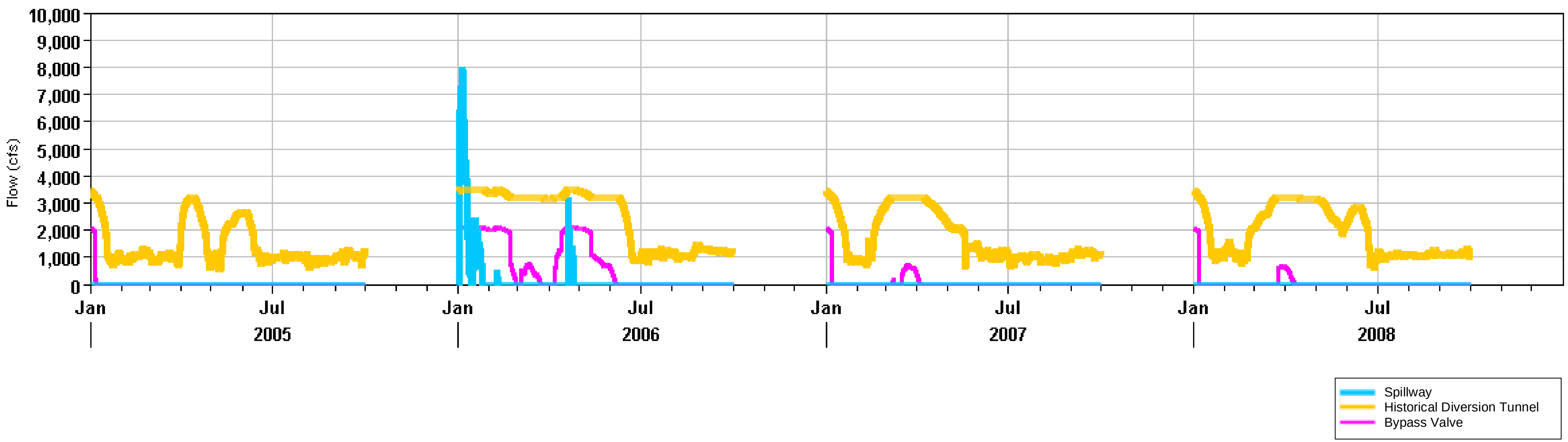

Figure 68: Iron Gate Structure Outlet Flows for years 2005 through 2008

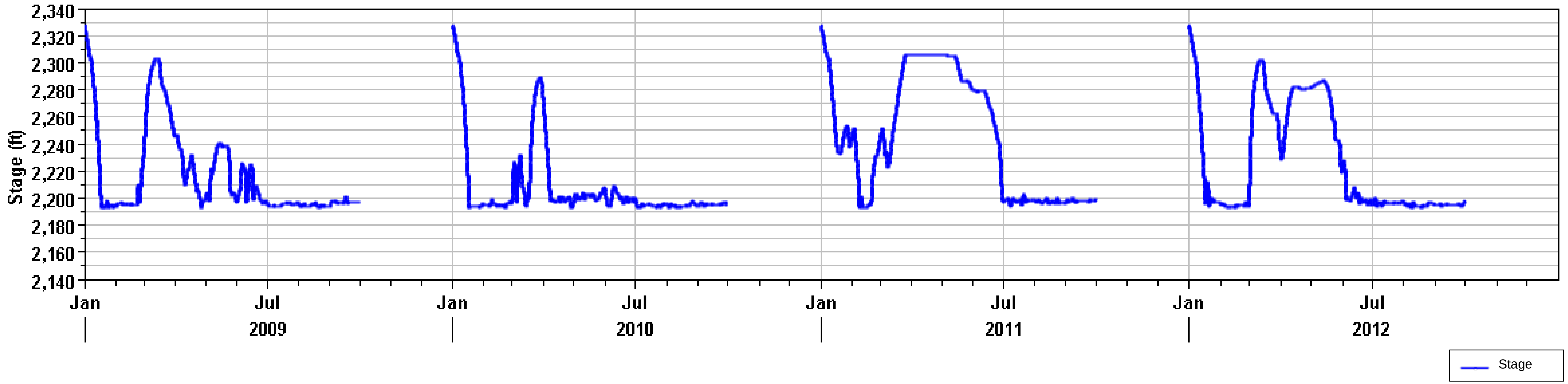

Figure 69: Iron Gate Drawdown Stage for years 2009 through 2012

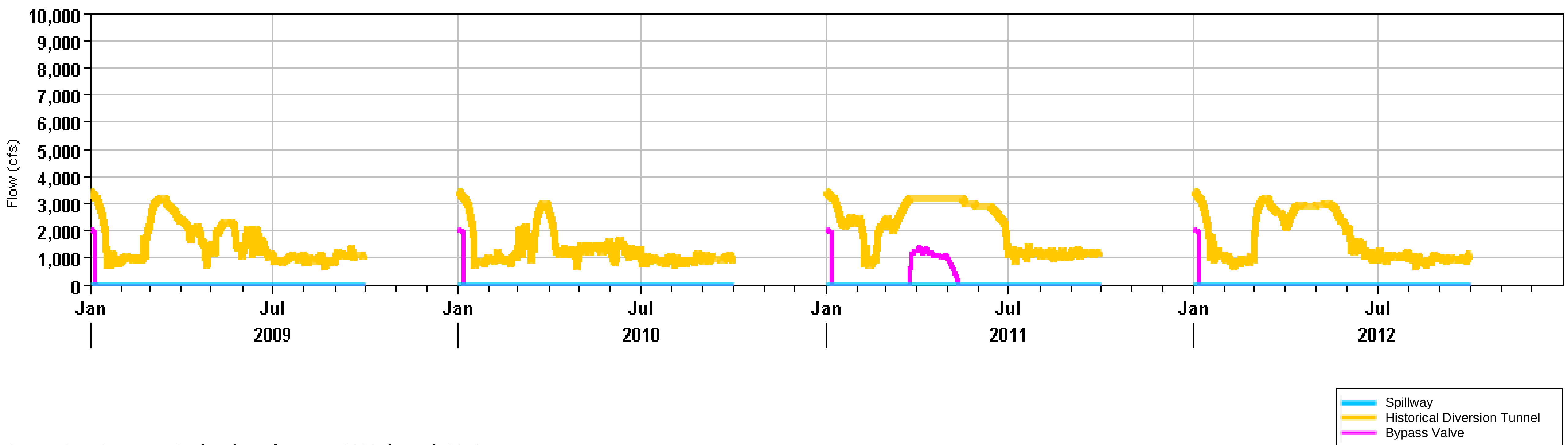

Figure 70: *Iron Gate* Structure Outlet Flows for years 2009 through 2012

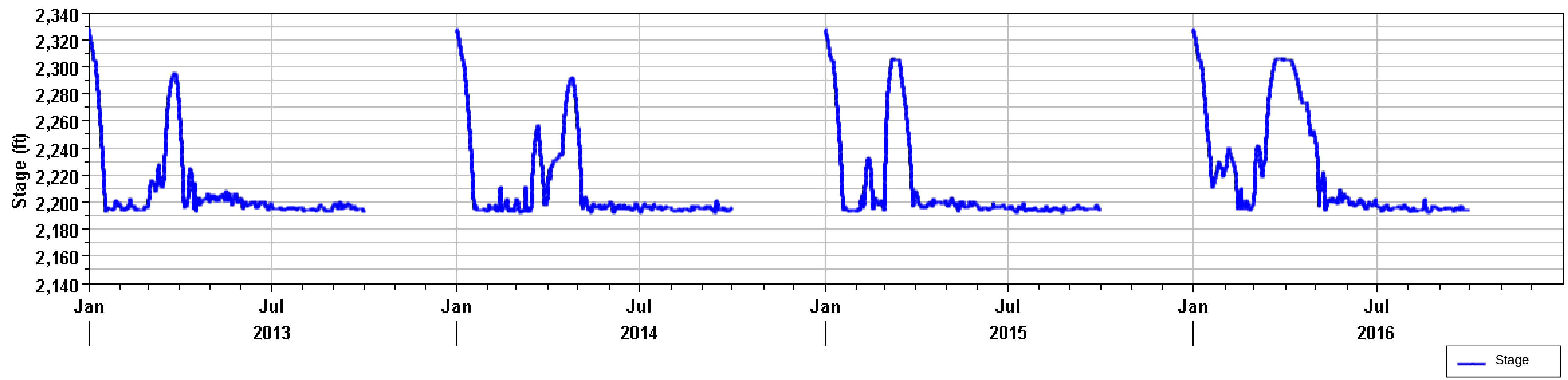

Figure 71: Iron Gate Drawdown Stage for years 2013 through 2016

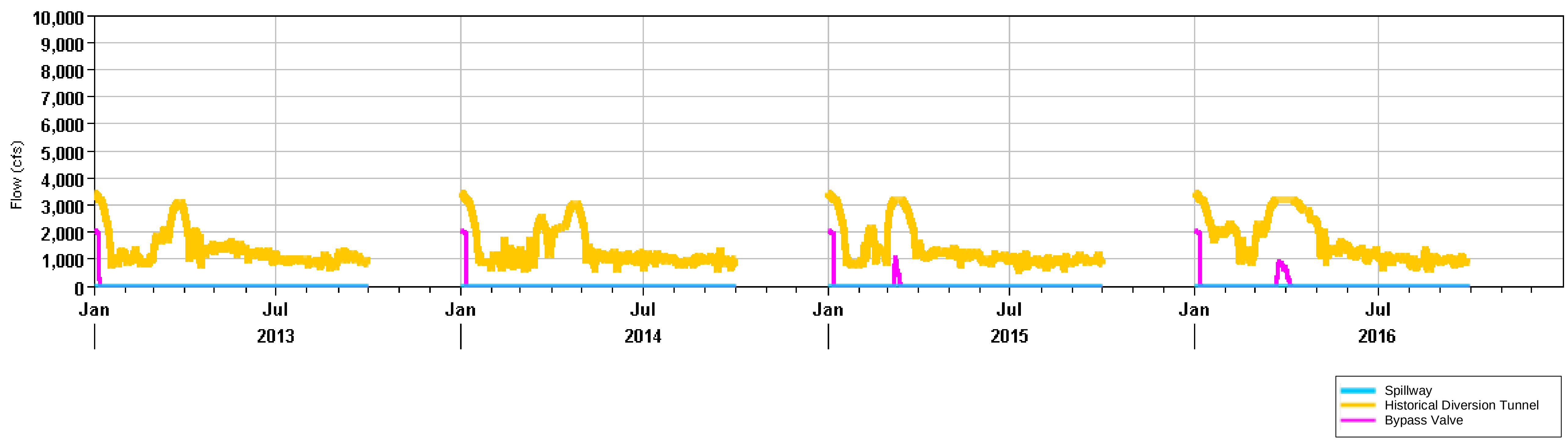

Figure 72: Iron Gate Structure Outlet Flows for years 2013 through 2016

MEMORANDUM

TO: Craig Nistor; Stuart Flett; Larry Buetikofer – Knight Piésold and Co

FROM: Scott Berkebile, PE, QSD/QSP, QISP, ToR – SWPPQueen, Inc.

DATE: May 10, 2022

SUBJECT: Klamath River Renewal Project 100% Design – BMP CGP Compliance Evaluation

1. Purpose

This memorandum provides a summary of temporary and permanent Best Management Practices (BMPs) for erosion and sediment control used for the design of the Klamath River Renewal Project, specifically the dam removal and associated facilities, roads, bridges, and recreation areas. The dams that will be removed include J.C. Boyle Dam in Oregon (OR), Copco No. 1, Copco No. 2, and Iron Gate Dams in California (CA). The JC Boyle Dam is subject to the Oregon Department of Environmental Quality (ODEQ) Stormwater Discharge Permit (Permit No. 1200-C). The Copco No. 1, Copco No. 2, and Iron Gate Dams are subject to the California Construction General Permit (CGP) (Order No. 2009-0009-DWQ as amended by Order No. 2010-2014-DWQ and 2012-0006-DWQ). Temporary and Permanent erosion and sediment control were designed by Knight Piésold for the 100% design plans to comply with the permit associated with each dam's location (state) and are discussed in this memorandum. These BMPs were also selected based on criteria and options outlined in the California Stormwater Quality Association (CASQA) Construction Stormwater BMP Handbook and State of Oregon Department of Environmental Quality (ODEQ) Construction Stormwater Best Management Practices Manual.

2. Temporary and Permanent BMPs – JC Boyle

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the Oregon Permit No. 1200-C and the ODEQ Construction Stormwater BMP Manual.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C1600 **Sheet C1601** **Sheet C1602** **Sheet C1603**	**Sheet C1620** **Sheet C1621** **Sheet C1622** **Sheet C1623** **Sheet C1624**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of very rocky terrain with sparse vegetation and some fines. This native material does not lend itself to staking of straw wattles, silt fencing, and vegetation re-establishment due to the lack of sufficient fines and adequate rainfall. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable ODEQ BMP manual and OR Permit No. 1200-C references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	ODEQ BMP Manual Reference	OR Permit No. 1200-C
Check Dams	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	2.14	2.1, 2.2
Outlet Protection	Erosion and Sediment Controls: Provide energy dissipation at discharge points to prevent scour	2.18	2.1, 2.2
Diversions	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site	2.15	2.1, 2.2
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	2.19	2.1, 2.2

The project will comply with the Permit No. 1200-C Section 2.2.21 Final Stabilization Criteria and Technical Specification 31 25 00. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable Oregon Permit No. 1200-C requirements for Notice of Termination (NOT) and ODEQ BMP references.

Work Area	ESCP Sheet Number	Slope Range	Proposed Final Stabilization	ODEQ BMP Manual Reference	OR Permit No. 1200-C
Disposal Sites	C1620	3.5H:1V to 5H:1V	Hydroseeding Biodegradable Check Dams Outlet Protection Diversions Rock Slope Protection Native Rocky Soils/Cover (<10% Fines)	2.4, 2.11, 2.14, 2.15	Section 2.2.21.a.i, Section 2.2.21.a.iii
Power Canal Burial and Cover	C1621	2%	Outlet Protection Native Rocky Soils/Cover (<10% Fines)	2.18	Section 2.2.21.a.i, Section 2.2.21.a.iii
Forebay Burial and Cover	C1623	1% to 10%	Outlet Protection Diversions Native Rocky Soils/Cover (<10% Fines)	2.15, 2.18	Section 2.2.21.a.i, Section 2.2.21.a.iii
Scour Hole Fill and Cover	C1623	1% to 1.5H:1V	Diversions Outlet Protection Native Rocky Soils/Cover (<10% Fines)	2.15, 2.18	Section 2.2.21.a.i, Section 2.2.21.a.iii
Penstock Concrete Cover and Powerhouse Cover (includes adjacent staging area)	C1624	1.5H:1V	Biodegradable Check Dams Diversions Native Rocky Soils/Cover (<10% Fines)	2.14, 2.15	Section 2.2.21.a.i, Section 2.2.21.a.iii

3. Temporary and Permanent BMPs – Copco 1

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the California CGP and the CASQA Construction Stormwater BMP Handbook.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C2600 **Sheet C2605**	**Sheet C2620** **Sheet C2621**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of very rocky terrain with sparse vegetation and some fines. This native material does not lend itself to staking of straw wattles, silt fencing, and vegetation re-establishment due to the lack of sufficient fines and adequate rainfall. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable CASQA Construction Stormwater BMP Handbook and CA CGP references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	CASQA BMP Handbook Reference	CA CGP
Check Dams	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-4	Attachment E – Section E
Velocity Dissipation Devices	Erosion and Sediment Controls: Provide energy dissipation at discharge points to prevent scour	EC-10	Attachment E – Section D
Earth Dikes and Drainage Swales	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections	EC-9	Attachment E – Section D

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	Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site		
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	TC-1	Attachment E – Section E

The project will utilize the aspect of the CGP Section II.D.3.c that allows for a "Custom Method" with permanent stabilization measures designed to provide long-term protection to underlying soils. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable CGP requirements for Notice of Termination (NOT) and CASQA BMP Handbook references.

Work Area	**ESCP Sheet Number**	**Slope Range**	**Proposed Final Stabilization**	**CASQA BMP Handbook Reference**	**CA CGP**
Disposal Sites	C2620	3H:1V	Biodegradable Check Dams Velocity Dissipation Devices Earth Dikes and Drainage Swales Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	SE-4, EC-9, EC-10, EC-16	Section II.D.3.c
Powerhouse Cover	C2621	1.5H:1V	Biodegradable Check Dams Earth Dikes and Drainage Swales Rock Slope Protection Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	SE-4, EC-9, EC-16	Section II.D.3.c
Powerhouse Access Roads	C2620	1.5H:1V	Biodegradable Check Dams Velocity Dissipation Devices Earth Dikes and Drainage Swales	SE-4, EC-9, EC-10	Section II.D.3.c

Staging Areas	C2620	3H:1V	Biodegradable Check Dams Earth Dikes and Drainage Swales Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	SE-4, EC-9, EC-16	Section II.D.3.c

4. Temporary and Permanent BMPs – Copco 2

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the California CGP and the CASQA Construction Stormwater BMP Handbook.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C3600 **Sheet C3601** **Sheet C3605** **Sheet C3606**	**Sheet C3620** **Sheet C3622** **Sheet C3623** **Sheet C3624**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of very rocky terrain with sparse vegetation and some fines. This native material does not lend itself to staking of straw wattles, silt fencing, and vegetation re-establishment due to the lack of sufficient fines and adequate rainfall. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable CASQA Construction Stormwater BMP Handbook and CA CGP references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	CASQA BMP Handbook Reference	CA CGP
Check Dams	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-4	Attachment E – Section E

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Velocity Dissipation Devices	Erosion and Sediment Controls: Provide energy dissipation at discharge points to prevent scour	EC-10	Attachment E – Section D
Earth Dikes and Drainage Swales	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site	EC-9	Attachment E – Section D
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	TC-1	Attachment E – Section E

The project will utilize the aspect of the CGP Section II.D.3.c that allows for a "Custom Method" with permanent stabilization measures designed to provide long-term protection to underlying soils. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable CGP requirements for Notice of Termination (NOT) and CASQA BMP Handbook references.

Work Area	ESCP Sheet Number	Slope Range	Proposed Final Stabilization	CASQA BMP Handbook Reference	CA CGP
Dam Excavation	C3620	1.5H:1V	Rock Slope Protection Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Wood-Stave Penstock Backfill	C3622	0.5%	Earth Dikes and Drainage Swales Velocity Dissipation Devices Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-9, EC-10, EC-16	Section II.D.3.c

Penstock and Powerhouse Cover	C3623	2.5H:1V	Biodegradable Check Dams Earth Dikes and Drainage Swales Velocity Dissipation Devices Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	SE-4, EC-9, EC-10, EC-16	Section II.D.3.c
Tailrace Fill	C3623	2.5H:1V	Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Dam Access Roads	C3620	1.5H:1V	Biodegradable Check Dams Earth Dikes and Drainage Swales	SE-4, EC-9	Section II.D.3.c
Copco Village	C3624	1.5H:1V	Biodegradable Check Dams Earth Dikes and Drainage Swales	SE-4, EC-9	Section II.D.3.c

5. Temporary and Permanent BMPs – Iron Gate

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the California CGP and the CASQA Construction Stormwater BMP Handbook.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C4600 **Sheet C4601** **Sheet C4605**	**Sheet C4610** **Sheet C4615**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of very rocky terrain with sparse vegetation and some fines. This native material does not lend itself to staking of straw wattles, silt fencing, and vegetation re-establishment due to the lack of sufficient fines and adequate rainfall. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable CASQA Construction Stormwater BMP Handbook and CA CGP references. Some of these

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BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	CASQA BMP Handbook Reference	CA CGP
Check Dams	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-4	Attachment E – Section E
Velocity Dissipation Devices	Erosion and Sediment Controls: Provide energy dissipation at discharge points to prevent scour	EC-10	Attachment E – Section D
Earth Dikes and Drainage Swales	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site	EC-9	Attachment E – Section D
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	TC-1	Attachment E – Section E

The project will utilize the aspect of the CGP Section II.D.3.c that allows for a "Custom Method" with permanent stabilization measures designed to provide long-term protection to underlying soils. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable CGP requirements for Notice of Termination (NOT) and CASQA BMP Handbook references.

Work Area	ESCP Sheet Number	Slope Range	Proposed Final Stabilization	CASQA BMP Handbook Reference	CA CGP
Powerhouse Fill	C4610	2.5H:1V to 5H:1V	Rock Slope Protection	EC-16	Section II.D.3.c

			Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization		
Spillway Fill	C4610	4H:1V	Rock Slope Protection Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Disposal Site #1	C4610	2H:1V to 1%	Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Disposal Site #2	C4610	2H:1V to 1%	Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Disposal Site #3	C4615	2H:1V to 1%	Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	EC-16	Section II.D.3.c
Staging Areas	C4610	2H:1V to 1%	Biodegradable Check Dams Earth Dikes and Drainage Swales Native Rocky Soils/Cover (<10% Fines) – Non-Vegetative Stabilization	SE-4, EC-9, EC-16	Section II.D.3.c

6. Temporary and Permanent BMPs – Access Roads and Bridges

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the California CGP and the CASQA Construction Stormwater BMP Handbook.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C5203 **Sheet C5303**	**Sheet C5204** **Sheet C5304**

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Site reconnaissance and available geotechnical and soils data found that the site primarily consists of very rocky terrain with sparse vegetation and some fines. However, much of the new roadway will use engineered fill material that has 30-40% fines and can be erodible. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable CASQA Construction Stormwater BMP Handbook and CA CGP references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	CASQA BMP Handbook Reference	CA CGP
Check Dams	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-4	Attachment E – Section E
Fiber Rolls	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-5	Attachment E – Section E
Erosion Control Blankets	Erosion Controls: Provide temporary cover to reduce erosion potential	EC-7	Attachment E – Section D
Velocity Dissipation Devices	Erosion and Sediment Controls: Provide energy dissipation at discharge points to prevent scour	EC-10	Attachment E – Section D
Earth Dikes and Drainage Swales	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site	EC-9	Attachment E – Section D
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	TC-1	Attachment E – Section E

The new roadways and bridges outlined below will utilize the aspect of the CGP Section II.D.3.a to establish vegetation growth on the new slopes to 70% coverage. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable CGP requirements for Notice of Termination (NOT) and CASQA BMP Handbook references.

Work Area	ESCP Sheet Number	Slope Range	Proposed Final Stabilization	CASQA BMP Handbook Reference	CA CGP
Camp Creek Road/Culvert	C5204	N/A	Rock Check Dams Earth Dikes and Drainage Swales Biodegradable Fiber Rolls Velocity Dissipation Devices Hydroseeding with Tackifier	SE-4, SE-5, EC-4, EC-9, EC-10	Section II.D.3.a
Scotch Creek Road/Culvert	C5304	N/A	Rock Check Dams Earth Dikes and Drainage Swales Biodegradable Fiber Rolls Velocity Dissipation Devices Hydroseeding with Tackifier	SE-4, SE-5, EC-4, EC-9, EC-10	Section II.D.3.a

7. Temporary and Permanent BMPs – Oregon Recreation Areas

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the Oregon Permit No. 1200-C and the ODEQ Construction Stormwater BMP Manual.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C7000 **Sheet C7005** **Sheet C7010** **Sheet C7015**	**Sheet C7000** **Sheet C7005** **Sheet C7010** **Sheet C7015**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of sparse vegetation, gravel parking areas, and some fines. Recreation areas will be demolished including most of the existing concrete and gravel parking areas will be removed. Buildings and other structures will be removed. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable ODEQ BMP manual and OR Permit No. 1200-C references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

BMP	BMP Purpose	ODEQ BMP Manual Reference	OR Permit No. 1200-C
Sediment (Silt) Fence	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	2.24	2.1, 2.2
Erosion Control Blankets	Erosion Controls: Provide temporary cover to reduce erosion potential	2.6	2.1, 2.2
Diversions	Drainage Controls: Divert run-on and runoff around project work areas to diversion channels and controlled discharge points with outlet protections Erosion and Sediment Controls: Prevent sediment-laden waters from leaving a site and minimizing erosion from the site	2.15	2.1, 2.2
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	2.19	2.1, 2.2

The project will comply with the Permit No. 1200-C Section 2.2.21 Final Stabilization Criteria and Technical Specification 31 25 00. The table below details the final stabilization methods chosen for each specific area of the project site and the applicable Oregon Permit No. 1200-C requirements for Notice of Termination (NOT) and ODEQ BMP references.

Work Area	ESCP Sheet Number	Slope Range	Proposed Final Stabilization	ODEQ BMP Manual Reference	OR Permit No. 1200-C
Gravel Parking Lots	C7005 C7010 C7015	<2%	Hydroseeding Native Rocky Soils/Cover (<10% Fines)	2.3, 2.4	Section 2.2.21.a.i, Section 2.2.21.a.iii
Poles & Signs	C7005 C7010 C7015	<2%	Hydroseeding Native Rocky Soils/Cover (<10% Fines)	2.3, 2.4	Section 2.2.21.a.i, Section 2.2.21.a.iii

8. Temporary and Permanent BMPs – California Recreation Areas

The following 100% design plan sheets were used to evaluate temporary and permanent erosion and sediment control BMPs proposed for the project. BMP types were selected based on the California CGP and the CASQA Construction Stormwater BMP Handbook.

Temporary Erosion and Sediment Control BMPs	Permanent Erosion and Sediment Control BMPs
Sheet C7020	**Sheet C7020**
Sheet C7025	**Sheet C7025**
Sheet C7030	**Sheet C7030**
Sheet C7035	**Sheet C7035**
Sheet C7040	**Sheet C7040**
Sheet C7045	**Sheet C7045**
Sheet C7050	**Sheet C7050**
Sheet C7055	**Sheet C7055**
Sheet C7060	**Sheet C7060**
Sheet C7065	**Sheet C7065**
Sheet C7070	**Sheet C7070**
Sheet C7075	**Sheet C7075**

Site reconnaissance and available geotechnical and soils data found that the site primarily consists of sparse vegetation, gravel parking areas, and some fines. Recreation areas will be demolished including most of the existing concrete and gravel parking areas will be removed. Buildings and other structures will be removed. The table below outlines the proposed temporary structural BMPs selected for the project and the applicable CASQA Construction Stormwater BMP Handbook and CA CGP references. Some of these BMPs will also be used as permanent BMPs given their flexibility and usefulness to fulfill both requirements.

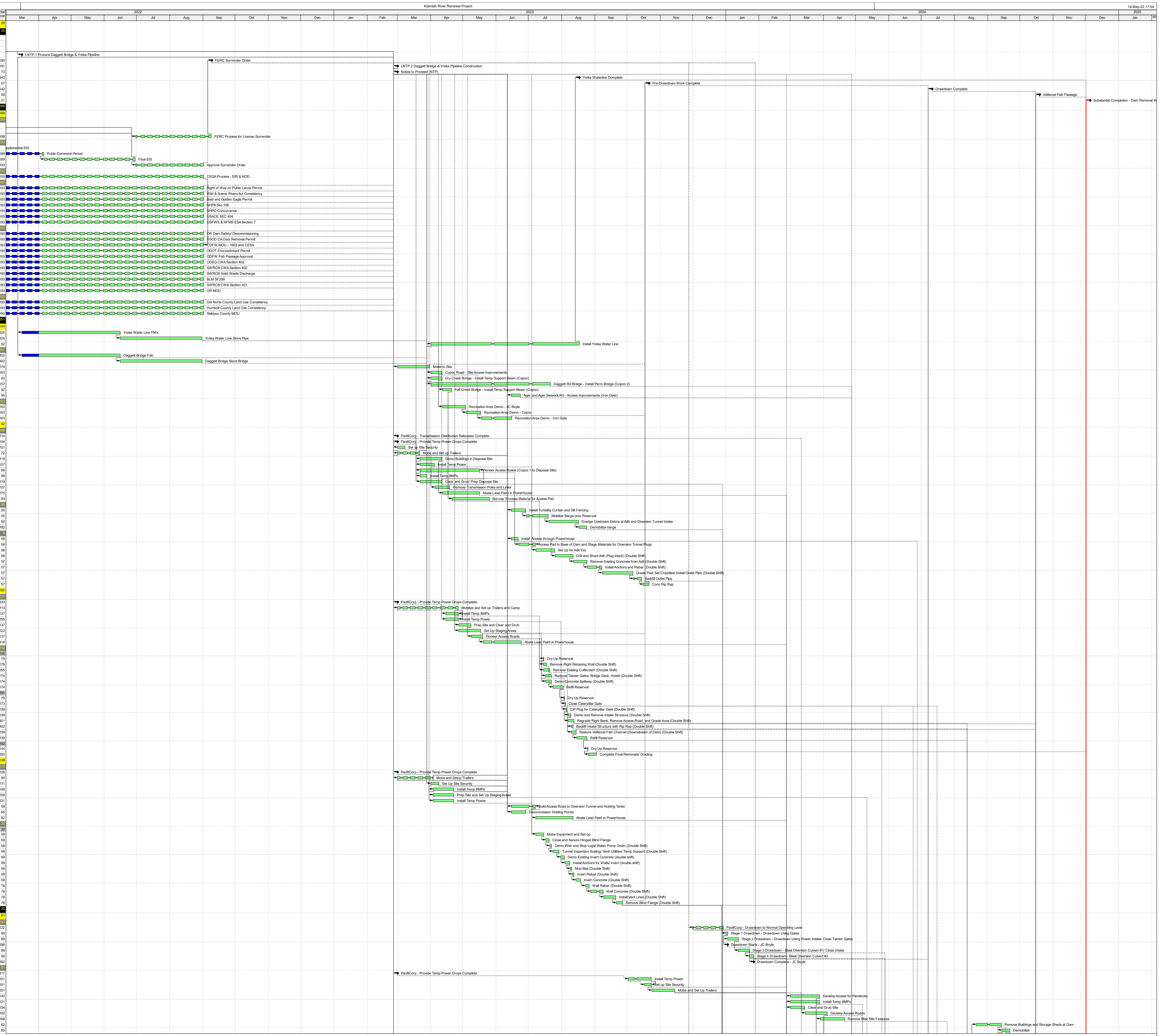
BMP	BMP Purpose	CASQA BMP Handbook Reference	CA CGP
Silt Fence	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-1	Attachment E – Section E
Fiber Rolls	Erosion and Sediment Controls: Reduce channel erosion by restricting flow velocity and minimizing sediment discharges from the site	SE-5	Attachment E – Section E
Hydroseeding	Erosion Controls: Provide temporary or permanent cover to reduce erosion potential	EC-4	Attachment E – Section D
Stabilized Construction Entrance/Exit	Erosion and Sediment Controls: Prevent offsite tracking of sediments by construction vehicles and equipment onto public or private roads	TC-1	Attachment E – Section E

PRIVILEGED AND CONFIDENTIAL

Klamath River Renewal Project

19-May-22 17:54

#	Activity ID	Activity Name	Original Duration	Start	Finish	Total Float
1		**Klamath River Renewal Project - Implementation Work Schedule - 90% GMP**	761	26-Apr-19 A	05-Dec-24	20
2		**MILESTONES**	760	26-Apr-19 A	05-Dec-24	20
3	GEN3030	KIWC Start of Preliminary Services	0	26-Apr-19 A		
4	GEN0011	FERC GMP	0	20-Nov-20 A		
5	GEN0001	GMP Amended - Executed	0	12-Feb-21 A		
6	GEN3010	LNTP 1 Procure Daggett Bridge & Yreka Pipeline	0	16-Mar-22 A		
7	GEN0040	FERC Surrender Order	0	09-Sep-22*		190
8	GEN3020	LNTP 2 Daggett Bridge & Yreka Pipeline Construction	0	01-Mar-23*		191
9	GEN0002	Notice to Proceed (NTP)	0	01-Mar-23*		73
10	GEN0018	Yreka Waterline Complete	0		17-Aug-23	342
11	GEN0006	Pre-Drawdown Work Complete	0		21-Oct-23*	57
12	GEN3000	Drawdown Complete	0		11-Jul-24	142
13	GEN0020	Volitional Fish Passage	0		19-Oct-24	59
14	GEN0008	Substantial Completion - Dam Removal Work (Complete by Dec 31, 2022)	0		05-Dec-24*	21
15		**PRE-CONSTRUCTION**	759	30-Aug-19 A	08-Sep-22	189
16		**Permitting**	759	30-Aug-19 A	08-Sep-22	189
17		**Transfer and Surrender Orders (FERC)**	529	20-Feb-20 A	08-Sep-22	189
18	PERM001	FERC Process for License Transfer	104	20-Feb-20 A	17-Jun-21 A	
19	PERM034	FERC Decommissioning Plan	83	31-Jul-20 A	30-Nov-20 A	
20	PERM003	FERC Process for License Surrender	49	30-Jun-22	08-Sep-22	189
21		**NEPA Process (FERC)**	525	31-Jul-20 A	01-Sep-22	193
22	PERM004	Supplemental EIS	390	31-Jul-20 A	22-Feb-22 A	
23	PERM017	Public Comment Period	30	23-Feb-22 A	05-Apr-22	189
24	PERM018	Final EIS	60	06-Apr-22	29-Jun-22	189
25	PERM019	Approve Surrender Order	45	30-Jun-22	01-Sep-22	193
26		**CEQA Process**	87	30-Aug-19 A	01-Sep-22	193
27	PERM016	CEQA Process - EIR & NOD	87	30-Aug-19 A	01-Sep-22	193
28		**Federal Permits**	396	30-Aug-19 A	01-Sep-22	193
29	PERM022	Right of Way on Public Lands Permit	232	30-Aug-19 A	01-Sep-22	193
30	PERM021	Wild & Scenic Rivers Act Consistency	287	30-Aug-19 A	01-Sep-22	193
31	PERM020	Bald and Golden Eagle Permit	167	30-Aug-19 A	01-Sep-22	193
32	PERM014	NHPA Sec 106	291	30-Aug-19 A	01-Sep-22	193
33	PERM008	SHPO Concurrence	120	30-Aug-19 A	01-Sep-22	193
34	PERM006	USACE SEC 404	396	30-Aug-19 A	01-Sep-22	193
35	PERM005	USFWS & NFMS ESA Section 7	281	30-Aug-19 A	01-Sep-22	193
36		**State Permits**	237	30-Aug-19 A	01-Sep-22	193
37	PERM030	OR Dam Safety/ Decommissioning	146	30-Aug-19 A	01-Sep-22	193
38	PERM029	DSOD CA Dam Removal Permit	146	30-Aug-19 A	01-Sep-22	193
39	PERM028	CDFW MOU - 1602 and CESA	149	30-Aug-19 A	01-Sep-22	193
40	PERM027	ODOT Encroachment Permit	141	30-Aug-19 A	01-Sep-22	193
41	PERM026	ODFW Fish Passage Approval	185	30-Aug-19 A	01-Sep-22	193
42	PERM025	ODEQ CWA Section 402	177	30-Aug-19 A	01-Sep-22	193
43	PERM024	SWRCB CWA Section 402	177	30-Aug-19 A	01-Sep-22	193
44	PERM023	SWRCB Solid Waste Discharge	109	30-Aug-19 A	01-Sep-22	193
45	PERM015	BLM SF299	120	30-Aug-19 A	01-Sep-22	193
46	PERM012	SWRCB CWA Section 401	120	30-Aug-19 A	01-Sep-22	193
47	PERM010	OR MOU	237	30-Aug-19 A	01-Sep-22	193
48		**Local Permits**	170	30-Aug-19 A	01-Sep-22	193
49	PERM032	Del Norte County Land Use Consistency	148	30-Aug-19 A	01-Sep-22	193
50	PERM031	Humbolt County Land Use Consistency	148	30-Aug-19 A	01-Sep-22	193
51	PERM007	Siskiyou County MOU	170	30-Aug-19 A	01-Sep-22	193
52		**PRE-DRAWDOWN YEAR**	348	16-Mar-22 A	21-Oct-23	251
53		**Project Wide**	293	16-Mar-22 A	17-Aug-23	144
54	PW1204	Yreka Water Line PM's	92	16-Mar-22 A	16-Jun-22	325
55	PW1214	Yreka Water Line Store Pipe	76	16-Jun-22	31-Aug-22	325
56	PW1184	Install Yreka Water Line	115	01-Apr-23	17-Aug-23	92
57		**Roads and Bridges**	270	16-Mar-22 A	21-Jul-23	167
58	PW1194	Daggett Bridge Fab	92	16-Mar-22 A	16-Jun-22	402
59	PW1224	Daggett Bridge Store Bridge	76	16-Jun-22	31-Aug-22	402
60	PW1234	Mobe to Site	30	01-Mar-23	31-Mar-23	274
61	PW0044	Copco Road - Site Access Improvements	8	01-Apr-23	11-Apr-23	163
62	PW1001	Dry Creek Bridge - Install Temp Support Beam (Copco)	8	01-Apr-23	11-Apr-23	92
63	PW1004	Daggett Rd Bridge - Install Perm Bridge (Copco 2)	92	01-Apr-23	21-Jul-23	157
64	PW1002	Fall Creek Bridge - Install Temp Support Beam (Copco)	8	12-Apr-23	20-Apr-23	92
65	PW1003	Ager and Ager Beswick Rd - Access Improvements (Iron Gate)	8	15-Jun-23	23-Jun-23	95
66		**Demo Recreation Sites**	55	12-Apr-23	15-Jun-23	163
67	PW1008	Recreation Area Demo - JC Boyle	19	12-Apr-23	03-May-23	163
68	PW1020	Recreation Area Demo - Copco	12	04-May-23	17-May-23	163
69	PW1030	Recreation Area Demo - Iron Gate	24	18-May-23	15-Jun-23	163
70		**Copco 1**	170	01-Mar-23	21-Oct-23	42
71		**Site Prep**	46	01-Mar-23	25-May-23	166
72	CO12222	PacifiCorp - Transmission/ Distribution Relocates Complete	0		01-Mar-23	114
73	CO12240	PacifiCorp - Provide Temp Power Drops Complete	0		01-Mar-23	104
74	CO10290	Set up Site Security	6	01-Mar-23	08-Mar-23	101
75	CO10062	Mobe and Set up Trailers	15	01-Mar-23*	21-Mar-23	72
76	CO10800	Demo Buildings in Disposal Site	18	22-Mar-23	11-Apr-23	119
77	CO10600	Install Temp Power	12	22-Mar-23	04-Apr-23	107
78	CO10021	Pioneer Access Roads (Copco 1 to Disposal Site)	48	22-Mar-23	16-May-23	89
79	CO10040	Install Temp BMPs	6	22-Mar-23	28-Mar-23	89
80	CO10031	Clear and Grub/ Prep Disposal Site	18	22-Mar-23	11-Apr-23	119
81	CO10620	Remove Transmission Poles and Lines	12	05-Apr-23	18-Apr-23	107
82	CO11571	Abate Lead Paint in Powerhouse	30	12-Apr-23	16-May-23	275
83	CO10700	Borrow/ Process Material for Access Pad	30	21-Apr-23	25-May-23	93
84		**Upstream Work**	58	15-Jun-23	24-Aug-23	85
85	CO10030	Install Turbidity Curtain and Silt Fencing	12	15-Jun-23	28-Jun-23	50
86	CO10010	Mobilize Barge onto Reservoir	16	29-Jun-23	19-Jul-23	50
87	CO10210	Dredge Upstream Debris at Adit and Diversion Tunnel Intake	24	20-Jul-23	16-Aug-23	50
88	CO10052	Demobilize barge	6	17-Aug-23	24-Aug-23	182
89		**Downstream Work**	108	15-Jun-23	21-Oct-23	8
90	CO10034	Install Access through Powerhouse	6	15-Jun-23	21-Jun-23	58
91	CO16010	Access Pad to Base of Dam and Stage Materials for Diversion Tunnel Plugs	12	22-Jun-23	07-Jul-23	58
92	CO10380	Set Up for Adit Exc	15	08-Jul-23	25-Jul-23	58
93	CO10071	Drill and Shoot Adit (Plug intact) (Double Shift)	15	26-Jul-23	11-Aug-23	58
94	CO10340	Remove Existing Concrete from Adit (Double Shift)	11	12-Aug-23	24-Aug-23	57
95	CO10360	Install Anchors and Rebar (Double Shift)	11	25-Aug-23	07-Sep-23	57
96	CO10451	Grade Pad/ Set Craddles/ Install Outlet Pipe (Double Shift)	25	08-Sep-23	06-Oct-23	57
97	CO11010	Backfill Outlet Pipe	6	07-Oct-23	14-Oct-23	57
98	CO11561	Conc Rip Rap	6	16-Oct-23	21-Oct-23	57
99		**Copco 2**	129	01-Mar-23	02-Sep-23	292
100		**Access/Site Work**	71	01-Mar-23	24-Jun-23	207
101	CO20521	PacifiCorp - Provide Temp Power Drops Complete	0		01-Mar-23	333
102	CO20491	Mobilize and Set up Trailers and Camp	40	01-Mar-23	26-Apr-23	113
103	CO20011	Install Temp BMPs	10	15-Apr-23	26-Apr-23	137
104	CO20501	Install Temp Power	10	15-Apr-23	26-Apr-23	255
105	CO20013	Prep Site and Clear and Grub	10	27-Apr-23	08-May-23	137
106	CO20012	Set Up Staging Areas	18	27-Apr-23	17-May-23	222
107	CO20014	Pioneer Access Roads	10	09-May-23	19-May-23	137
108	CO20581	Abate Lead Paint in Powerhouse	30	20-May-23	24-Jun-23	316
109		**Dam Removal**	43	15-Jul-23	02-Sep-23	292
110		**Dry Period #1**	16	15-Jul-23	02-Aug-23	245
111	CO2001	Dry Up Reservoir	1	15-Jul-23*	15-Jul-23	79
112	CO2008	Remove Right Retaining Wall (Double Shift)	2	15-Jul-23	18-Jul-23	178
113	CO2026	Remove Existing Cofferdam (Double Shift)	5	15-Jul-23	21-Jul-23	255
114	CO2007	Remove Tainter Gates, Bridge Deck, Hoists (Double Shift)	6	17-Jul-23	22-Jul-23	174
115	CO2011	Demo Concrete Spillway (Double Shift)	6	17-Jul-23	22-Jul-23	174
116	CO20541	Refill Reservoir	9	24-Jul-23	02-Aug-23	174
117		**Dry Period #2**	19	03-Aug-23	24-Aug-23	291
118	CO20481	Dry Up Reservoir	1	03-Aug-23	03-Aug-23	79
119	CO20401	Close Caterpillar Gate	1	04-Aug-23	04-Aug-23	173
120	CO2004	CIP Plug for Caterpillar Gate (Double Shift)	1	05-Aug-23	05-Aug-23	239
121	CO20411	Demo and Remove Intake Structure (Double Shift)	3	07-Aug-23	09-Aug-23	239
122	CO2013	Regrade Right Bank, Remove Access Road, and Grade Area (Double Shift)	6	07-Aug-23	12-Aug-23	301
123	CO2002	Backfill Intake Structure with Rip Rap (Double Shift)	2	10-Aug-23	11-Aug-23	302
124	CO20391	Restore Volitional Fish Channel (Downstream of Dam) (Double Shift)	4	10-Aug-23	14-Aug-23	239
125	CO20551	Refill Reservoir	9	15-Aug-23	24-Aug-23	239
126		**Dry Period #3**	8	25-Aug-23	02-Sep-23	292
127	CO20591	Dry Up Reservoir	1	25-Aug-23	25-Aug-23	144
128	CO20601	Complete Final Removals/ Grading	7	26-Aug-23	02-Sep-23	292
129		**Iron Gate**	148	01-Mar-23	26-Sep-23	138
130		**Access/ Site Work**	110	01-Mar-23	11-Aug-23	176
131	IG4000	PacifiCorp - Provide Temp Power Drops Complete	0		01-Mar-23	126
132	IG0067	Mobe and Setup Trailers	24	01-Mar-23	03-Apr-23	90
133	IG2130	Set Up Site Security	6	01-Apr-23	08-Apr-23	111
134	IG0064	Install Temp BMPs	16	03-Apr-23	22-Apr-23	109
135	IG0065	Prep Site and Set Up Staging Areas	16	03-Apr-23	22-Apr-23	259
136	IG3280	Install Temp Power	16	03-Apr-23	22-Apr-23	121
137	IG0051	Build Access Road to Diversion Tunnel and Holding Tanks	18	15-Jun-23	07-Jul-23	59
138	IG3290	Decommission Holding Ponds	12	15-Jun-23	28-Jun-23	65
139	IG4030	Abate Lead Paint in Powerhouse	30	08-Jul-23	11-Aug-23	82
140		**Pre-Drawdown Modifications**	68	08-Jul-23	26-Sep-23	30
141		**Tunnel Linning**	68	08-Jul-23	26-Sep-23	30
142	IG3050	Mobe Equipment and Set-up	7	08-Jul-23	15-Jul-23	59
143	IG0001	Close and Secure Hinged Blind Flange	4	17-Jul-23	20-Jul-23	59
144	IG3060	Demo Weir and Stop Logs/ Water Pump Down (Double Shift)	2	21-Jul-23	22-Jul-23	59
145	IG3070	Tunnel Inspection Scaling/ Vent/ Utilities/ Temp Support (Double Shift)	6	24-Jul-23	29-Jul-23	59
146	IG3080	Demo Existing Invert Concrete (double shift)	4	31-Jul-23	03-Aug-23	59
147	IG3090	Install Anchors for Walls/ invert (double shift)	4	04-Aug-23	08-Aug-23	59
148	IG3100	Mud Mat (Double Shift)	2	09-Aug-23	10-Aug-23	59
149	IG3110	Invert Rebar (Double Shift)	2	11-Aug-23	12-Aug-23	59
150	IG3120	Invert Concrete (Double Shift)	5	14-Aug-23	18-Aug-23	59
151	IG3130	Wall Rebar (Double Shift)	4	23-Aug-23	26-Aug-23	78
152	IG0013	Wall Concrete (Double Shift)	10	28-Aug-23	08-Sep-23	78
153	IG0028	Install Vent Lines (Double Shift)	10	09-Sep-23	20-Sep-23	78
154	IG3260	Remove Blind Flange (Double Shift)	5	21-Sep-23	26-Sep-23	78
155		**DRAWDOWN YEAR**	402	01-Mar-23	05-Dec-24	20
156		**JC Boyle**	340	01-Mar-23	21-Sep-24	81
157		**Drawdown**	0	01-Dec-23	27-Jan-24	243
158	JCB1180	PacifiCorp - Drawdown to Normal Operating Level	20	01-Dec-23*	29-Dec-23	122
159	JCB0017	Stage 1 Drawdown - Drawdown Using Gates	2	01-Jan-24	03-Jan-24	99
160	JCB1220	Stage 2 Drawdown - Drawdown Using Power Intake/ Close Tainter Gates	10	03-Jan-24	13-Jan-24	99
161	JCB0082	Drawdown Starts - JC Boyle	0	03-Jan-24		286
162	JCB1230	Stage 3 Drawdown - Blast Diversion Culvert #1/ Close Intake	10	13-Jan-24	23-Jan-24	99
163	JCB0019	Stage 4 Drawdown - Blast Diversion Culvert #2	4	23-Jan-24	27-Jan-24	99
164	JCB1050	Drawdown Complete - JC Boyle	0		27-Jan-24	262
165		**Access/ Site Work**	340	01-Mar-23	21-Sep-24	81
166	JCB1270	PacifiCorp - Provide Temp Power Drops Complete	0		01-Mar-23	317
167	JCB1260	Install Temp Power	18	02-Oct-23*	23-Oct-23	201
168	JCB1060	Set up Site Security	6	17-Oct-23	23-Oct-23	201
169	JCB0056	Mobe and Set Up Trailers	18	24-Oct-23	14-Nov-23	201
170	JCB1070	Develop Access for Penstocks	24	01-Mar-24	28-Mar-24	142
171	JCB0051	Install Temp BMPs	24	01-Mar-24	28-Mar-24	121
172	JCB0057	Clear and Grub Site	12	01-Mar-24	14-Mar-24	104
173	JCB0052	Develop Access Roads	18	15-Mar-24	04-Apr-24	102
174	JCB1190	Remove Misc Site Features	20	29-Mar-24	20-Apr-24	168
175	JCB1200	Remove Buildings and Storage Sheds at Dam	20	21-Aug-24	13-Sep-24	82
176	JCB0059	Demobilize	7	14-Sep-24	21-Sep-24	85



Remaining Level of Effort ◆ ◆ Milestone
Actual Level of Effort
Actual Work
Remaining Work
Critical Remaining Work

Klamath River Renewal Project
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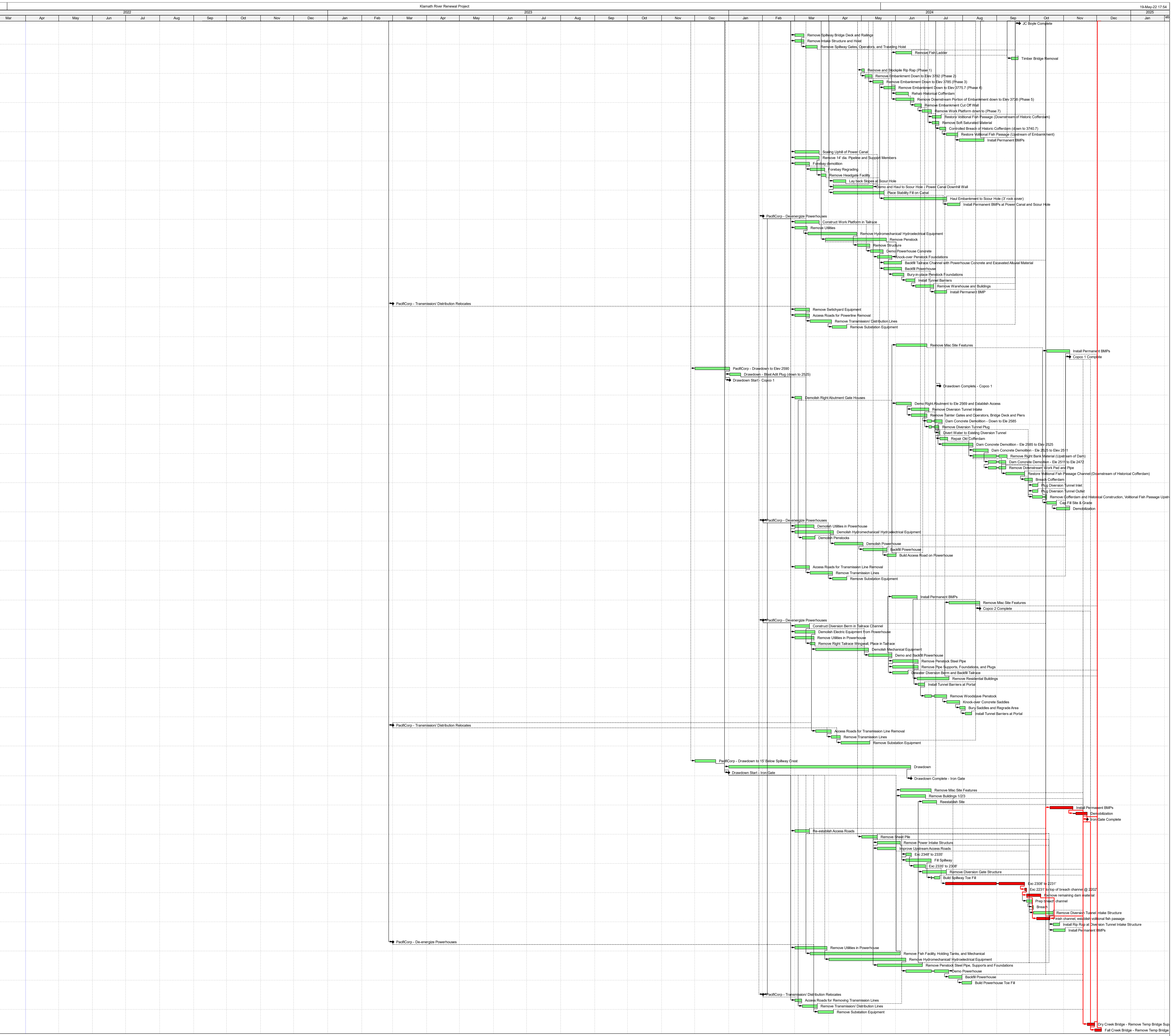
Project Completion : 05-Dec-24
Data Date: 01-Apr-22

#	Activity ID	Activity Name	Original Duration	Start	Finish	Total Float
177	JCB0081	JC Boyle Complete	0		21-Sep-24	81
178		**Dam/Intake/Spillway Removal**	175	01-Mar-24	20-Sep-24	86
179	JCB0014	Remove Spillway Bridge Deck and Railings	8	01-Mar-24	09-Mar-24	235
180	JCB0012	Remove Intake Structure and Hoist	8	01-Mar-24	09-Mar-24	235
181	JCB0015	Remove Spillway Gates, Operators, and Traveling Hoist	10	11-Mar-24	21-Mar-24	235
182	JCB0005	Remove Fish Ladder	13	01-Jun-24	15-Jun-24	163
183	JCB0013	Timber Bridge Removal	6	14-Sep-24	20-Sep-24	86
184		**Embankment Removal**	89	01-May-24	20-Aug-24	108
185	JCB0006	Remove and Stockpile Rip Rap (Phase 1)	3	01-May-24*	03-May-24	84
186	JCB1240	Remove Embankment Down to Elev 3792 (Phase 2)	6	04-May-24	10-May-24	84
187	JCB0020	Remove Embankment Down to Elev 3785 (Phase 3)	8	11-May-24	20-May-24	84
188	JCB1120	Remove Embankment Down to Elev 3775.7 (Phase 4)	10	21-May-24	31-May-24	84
189	JCB0003	Rehab Historical Cofferdam	10	01-Jun-24	12-Jun-24	88
190	JCB1130	Remove Downstream Portion of Embankment down to Elev 3738 (Phase 5)	14	01-Jun-24	17-Jun-24	84
191	JCB0007	Remove Embankment Cut Off Wall	6	18-Jun-24	24-Jun-24	84
192	JCB1170	Remove Work Platform down to (Phase 7)	8	25-Jun-24	03-Jul-24	84
193	JCB1000	Restore Volitional Fish Passage (Downstream of Historic Cofferdam)	8	04-Jul-24	12-Jul-24	146
194	JCB1140	Remove Soft Saturated Material	6	04-Jul-24	10-Jul-24	84
195	JCB0009	Controlled Breach of Historic Cofferdam (down to 3740.7)	5	11-Jul-24	16-Jul-24	84
196	JCB1150	Restore Volitional Fish Passage (Upstream of Embamkment)	10	17-Jul-24	27-Jul-24	84
197	JCB1210	Install Permanent BMPs	20	29-Jul-24	20-Aug-24	82
198		**Power Canal Removal**	129	01-Mar-24	29-Jul-24	132
199	JCB1090	Scaling Uphill of Power Canal	20	01-Mar-24	23-Mar-24	146
200	JCB0024	Remove 14' dia. Pipeline and Support Members	20	01-Mar-24	23-Mar-24	176
201	JCB0029	Forebay demolition	12	01-Mar-24	14-Mar-24	122
202	JCB1010	Forebay Regrading	12	15-Mar-24	28-Mar-24	132
203	JCB0026	Remove Headgate Facility	5	25-Mar-24	29-Mar-24	176
204	JCB0040	Lay back Slopes at Scour Hole	10	05-Apr-24	16-Apr-24	126
205	JCB0027	Demo and Haul to Scour Hole - Power Canal Downhill Wall	32	05-Apr-24	11-May-24	104
206	JCB0028	Place Stability Fill on Canal	40	05-Apr-24	21-May-24	181
207	JCB0031	Haul Embankment to Scour Hole (3' rock cover)	50	21-May-24	17-Jul-24	132
208	JCB1100	Install Permanent BMPs at Power Canal and Scour Hole	10	18-Jul-24	29-Jul-24	132
209		**Powerhouse and Penstock Removal**	106	01-Feb-24	17-Jul-24	137
210	JCB1040	PacifiCorp - De-energize Powerhouses	0		01-Feb-24*	51
211	JCB0034	Construct Work Platform in Tailrace	20	01-Mar-24	23-Mar-24	206
212	JCB0043	Remove Utilities	10	01-Mar-24	12-Mar-24	167
213	JCB0044	Remove Hydromechanical/ Hydroelectrical Equipment	39	13-Mar-24	26-Apr-24	167
214	JCB0022	Remove Penstock	48	29-Mar-24	23-May-24	142
215	JCB1020	Remove Structure	10	27-Apr-24	08-May-24	167
216	JCB0041	Demo Powerhouse Concrete	10	09-May-24	20-May-24	167
217	JCB0023	Knock-over Penstock Foundations	12	15-May-24	28-May-24	142
218	JCB0038	Backfill Tailrace Channel with Powerhouse Concrete and Excavated Alluvial Material	15	21-May-24	06-Jun-24	177
219	JCB0025	Backfill Powerhouse	15	21-May-24	06-Jun-24	167
220	JCB0032	Bury-in-place Penstock Foundations	10	29-May-24	08-Jun-24	142
221	JCB0033	Install Tunnel Barriers	8	10-Jun-24	18-Jun-24	142
222	JCB1030	Remove Warehouse and Buildings	15	19-Jun-24	05-Jul-24	142
223	JCB1250	Install Permanent BMP	10	06-Jul-24	17-Jul-24	142
224		**Transmission/Distribution**	216	01-Mar-23	17-Apr-24	192
225	JCB1110	PacifiCorp - Transmission/ Distribution Relocates	0		01-Mar-23	429
226	JCB0036	Remove Swtichyard Equipment	12	01-Mar-24	14-Mar-24	205
227	JCB0100	Access Roads for Powerline Removal	12	01-Mar-24	14-Mar-24	205
228	JCB0042	Remove Transmission/ Distribution Lines	17	15-Mar-24	03-Apr-24	205
229	JCB0045	Remove Substation Equipment	12	04-Apr-24	17-Apr-24	205
230		**Copco 1**	201	01-Dec-23	06-Nov-24	42
231		**Access/Site Work**	132	01-Jun-24	06-Nov-24	33
232	CO11000	Remove Misc Site Features	24	01-Jun-24	29-Jun-24	123
233	CO11020	Install Permanent BMPs	18	16-Oct-24	06-Nov-24	33
234	CO10064	Copco 1 Complete	0		06-Nov-24	33
235		**Drawdown**	101	01-Dec-23	11-Jul-24	142
236	CO10018	PacifiCorp - Drawdown to Elev 2590	32	01-Dec-23*	01-Jan-24	92
237	CO10003	Drawdown - Blast Adit Plug (down to 2525)	10	01-Jan-24	11-Jan-24	92
238	CO10063	Drawdown Start - Copco 1	0	01-Jan-24		92
239	CO10260	Drawdown Complete - Copco 1	0	11-Jul-24		142
240		**Dam Removal**	201	01-Mar-24	06-Nov-24	33
241	CO10005	Demolish Right Abutment Gate Houses	6	01-Mar-24	07-Mar-24	87
242	CO10004	Demo Right Abutment to Ele 2569 and Establish Access	12	01-Jun-24	15-Jun-24	34
243	CO10072	Remove Diversion Tunnel Intake	13	15-Jun-24	01-Jul-24	60
244	CO10002	Remove Tainter Gates and Operators, Bridge Deck and Piers	12	15-Jun-24	29-Jun-24	34
245	CO10250	Dam Concrete Demolition - Down to Ele 2585	10	29-Jun-24	13-Jul-24	34
246	CO10240	Remove Diversion Tunnel Plug	6	01-Jul-24	10-Jul-24	60
247	CO10331	Divert Water to Existing Diversion Tunnel	1	10-Jul-24	11-Jul-24	60
248	CO10400	Repair Old Cofferdam	6	11-Jul-24	18-Jul-24	75
249	CO11300	Dam Concrete Demolition - Ele 2585 to Elev 2525	24	13-Jul-24	10-Aug-24	34
250	CO10300	Dam Concrete Demolition - Ele 2525 to Elev 2511	12	10-Aug-24	24-Aug-24	34
251	CO10341	Remove Right Bank Material (Upstream of Dam)	25	10-Aug-24	10-Sep-24	55
252	CO11200	Dam Concrete Demolition - Ele 2511 to Ele 2472	12	24-Aug-24	09-Sep-24	34
253	CO10017	Remove Downstream Work Pad and Pipe	12	24-Aug-24	09-Sep-24	64
254	CO10371	Restore Volitional Fish Passage Channel (Downstream of Historical Cofferdam)	15	09-Sep-24	26-Sep-24	34
255	CO11800	Breach Cofferdam	6	26-Sep-24	03-Oct-24	33
256	CO11030	Plug Diversion Tunnel Inlet	4	03-Oct-24	08-Oct-24	39
257	CO10019	Plug Diversion Tunnel Outlet	4	03-Oct-24	08-Oct-24	57
258	CO10351	Remove Cofferdam and Historical Construction, Volitional Fish Passage Upstream	10	03-Oct-24	16-Oct-24	33
259	CO20611	Cap Fill Site & Grade	8	16-Oct-24	25-Oct-24	33
260	CO20621	Demobilization	10	25-Oct-24	06-Nov-24	33
261		**Powerhouse and Penstock Removal**	69	01-Feb-24	01-Jun-24	34
262	CO12230	PacifiCorp - De-energize Powerhouses	0		01-Feb-24	51
263	CO10051	Demolish Utilities in Powerhouse	15	01-Mar-24	18-Mar-24	34
264	CO10053	Demolish Hydromechanical/ Hydroelectrical Equipment	30	01-Mar-24	05-Apr-24	34
265	CO10006	Demolish Penstocks	10	08-Mar-24	19-Mar-24	87
266	CO10007	Demolish Powerhouse	18	06-Apr-24	02-May-24	34
267	CO10022	Backfill Powerhouse	15	02-May-24	24-May-24	34
268	CO10390	Build Access Road on Powerhouse	6	24-May-24	01-Jun-24	34
269		**Transmission/Distribution**	38	01-Mar-24	17-Apr-24	196
270	CO10220	Access Roads for Transmission Line Removal	12	01-Mar-24	14-Mar-24	196
271	CO10054	Remove Transmission Lines	17	15-Mar-24	04-Apr-24	196
272	CO10055	Remove Substation Equipment	9	04-Apr-24	17-Apr-24	196
273		**Copco 2 (Could Happen as Early as Pre-Drawdown Year)**	310	01-Mar-23	16-Aug-24	102
274		**Site**	68	28-May-24	16-Aug-24	101
275	CO20441	Install Permanent BMPs	20	28-May-24	20-Jun-24	101
276	CO20421	Remove Misc Site Features	24	19-Jul-24	16-Aug-24	101
277	CO2081	Copco 2 Complete	0		16-Aug-24	101
278		**Powerhouse and Penstock Removal**	108	01-Feb-24	19-Jul-24	126
279	CO20531	PacifiCorp - De-energize Powerhouses	0		01-Feb-24	111
280	CO20112	Construct Diversion Berm in Tailrace Channel	12	01-Mar-24	14-Mar-24	134
281	CO20051	Demolish Electric Equipment from Powerhouse	16	01-Mar-24	19-Mar-24	99
282	CO20471	Remove Utilities in Powerhouse	15	01-Mar-24	18-Mar-24	100
283	CO20010	Remove Right Tailrace Wingwall, Place in Tailrace	4	15-Mar-24	19-Mar-24	134
284	CO20053	Demolish Mechanical Equipment	35	20-Mar-24	07-May-24	99
285	CO20113	Demo and Backfill Powerhouse	15	07-May-24	28-May-24	99
286	CO20190	Remove Penstock Steel Pipe	20	29-May-24	21-Jun-24	108
287	CO20110	Remove Pipe Supports, Foundations, and Plugs	20	29-May-24	21-Jun-24	108
288	CO20114	Dewater Diversion Berm and Backfill Tailrace	12	29-May-24	12-Jun-24	156
289	CO20431	Remove Residential Buildings	24	20-Jun-24	19-Jul-24	101
290	CO20111	Install Tunnel Barriers at Portal	5	21-Jun-24	27-Jun-24	108
291		**Wood Stave Penstock Removal**	35	27-Jun-24	09-Aug-24	108
292	CO2014	Remove Woodstave Penstock	15	27-Jun-24	17-Jul-24	108
293	CO2016	Knock-over Concrete Saddles	10	17-Jul-24	29-Jul-24	108
294	CO2017	Bury Saddles and Regrade Area	5	29-Jul-24	03-Aug-24	108
295	CO2018	Install Tunnel Barriers at Portal	5	03-Aug-24	09-Aug-24	108
296		**Transmission/Distribution**	230	01-Mar-23	08-May-24	182
297	CO20511	PacifiCorp - Transmission/ Distribution Relocates	0		01-Mar-23	343
298	CO20230	Access Roads for Transmission Line Removal	12	20-Mar-24	03-Apr-24	182
299	CO20052	Remove Transmission Lines	6	03-Apr-24	11-Apr-24	182
300	CO20054	Remove Substation Equipment	18	12-Apr-24	08-May-24	182
301		**Iron Gate**	393	01-Mar-23	22-Nov-24	29
302		**Drawdown**	80	01-Dec-23	15-Jun-24	163
303	IG2000	PacifiCorp - Drawdown to 15' Below Spillway Crest	16	01-Dec-23*	20-Dec-23	147
304	IG0003	Drawdown	166	01-Jan-24	15-Jun-24	199
305	IG0040	Drawdown Start - Iron Gate	0	01-Jan-24		88
306	IG2110	Drawdown Complete - Iron Gate	0		15-Jun-24	164
307		**Access/Site Work**	143	05-Jun-24	22-Nov-24	20
308	IG1000	Remove Misc Site Features	24	05-Jun-24	03-Jul-24	138
309	IG0050	Remove Buildings 1/2/3	20	05-Jun-24	28-Jun-24	124
310	IG3160	Reestablish Site	10	25-Jun-24	08-Jul-24	136
311	IG3170	Install Permanent BMPs	18	19-Oct-24	09-Nov-24	21
312	IG4160	Demobilization	10	09-Nov-24	22-Nov-24	21
313	IG0070	Iron Gate Complete	0		22-Nov-24	21
314		**Embankment Removal**	198	01-Mar-24	02-Nov-24	37
315	IG0033	Re-establish Access Roads	12	01-Mar-24	14-Mar-24	79
316	IG0021	Remove Sheet Pile	10	01-May-24*	15-May-24	35
317	IG3230	Remove Power Intake Structure	15	15-May-24	05-Jun-24	35
318	IG2150	Improve Upstream Access Roads	12	15-May-24	01-Jun-24	42
319	IG4080	Exc 2348' to 2335'	5	10-Jun-24	15-Jun-24	35
320	IG4040	Fill Spillway	20	10-Jun-24	03-Jul-24	134
321	IG4090	Exc 2335' to 2308'	10	17-Jun-24	28-Jun-24	35
322	IG0020	Remove Diversion Gate Structure	18	25-Jun-24	17-Jul-24	118
323	IG4050	Build Spillway Toe Fill	5	03-Jul-24	11-Jul-24	134
324	IG4100	Exc 2308' to 2231'	61	16-Jul-24*	26-Sep-24	22
325	IG4110	Exc 2231' to top of breach channel @ 2202'	2	26-Sep-24	28-Sep-24	22
326	IG4140	Remove remaining dam material	11	28-Sep-24	11-Oct-24	22
327	IG4120	Prep breach channel	4	28-Sep-24	03-Oct-24	24
328	IG4130	Breach	1	03-Oct-24	04-Oct-24	24
329	IG0004	Remove Diversion Tunnel Intake Structure	15	04-Oct-24	22-Oct-24	37
330	IG4150	Finish channel, establish volitional fish passage	11	07-Oct-24	19-Oct-24	22
331	IG0080	Install Rip Rap at Diversion Tunnel Intake Structure	5	22-Oct-24	28-Oct-24	42
332	IG3200	Install Permanent BMPs	10	22-Oct-24	02-Nov-24	37
333		**Powerhouse/ Fish Facility Removal (Could Happen as Early as January Drawdown Year)**	304	01-Mar-23	09-Aug-24	108
334	IG3020	PacifiCorp - De-energize Powerhouses	0		01-Mar-23	351
335	IG0061	Remove Utilities in Powerhouse	26	01-Mar-24	30-Mar-24	108
336	IG0007	Remove Fish Facility, Holding Tanks, and Mechanical	60	15-Mar-24	05-Jun-24	124
337	IG0062	Remove Hydromechanical/ Hydroelectrical Equipment	50	01-Apr-24	10-Jun-24	108
338	IG0031	Remove Penstock Steel Pipe, Supports and Foundations	32	15-May-24	25-Jun-24	119
339	IG0032	Demo Powerhouse	32	10-Jun-24	19-Jul-24	108
340	IG0036	Backfill Powerhouse	10	19-Jul-24	31-Jul-24	108
341	IG4070	Build Powerhouse Toe Fill	8	31-Jul-24	09-Aug-24	108
342		**Transmission/Distribution**	30	01-Feb-24	05-Apr-24	204
343	IG3010	PacifiCorp - Transmission/ Distribution Relocates	0		01-Feb-24	172
344	IG0210	Access Roads for Removing Transmission Lines	6	01-Mar-24	07-Mar-24	166
345	IG0054	Remove Transmission/ Distribution Lines	12	08-Mar-24	21-Mar-24	166
346	IG0063	Remove Substation Equipment	12	22-Mar-24	05-Apr-24	204
347		**Project Wide**	10	22-Nov-24	05-Dec-24	21
348	PW1054	Dry Creek Bridge - Remove Temp Bridge Support (Copco 1)	5	22-Nov-24	29-Nov-24	21
349	PW1053	Fall Creek Bridge - Remove Temp Bridge Support (Copco 1)	5	29-Nov-24	05-Dec-24	21



Remaining Level of Effort
Actual Level of Effort
Actual Work
Remaining Work
Critical Remaining Work
Milestone

Klamath River Renewal Project
Page 2 of 2

Project Completion : 05-Dec-24
Data Date: 01-Apr-22

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TABLE J1.1

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

100% DESIGN REPORT
APPENDIX J1.2 TABLE OF CONTENTS - J.C. BOYLE

Print May/11/22 9:16:36

0	27MAY'22	ISSUED WITH REPORT VA103-00640/01-9	HW	SDR
REV	DATE	DESCRIPTION	PREP'D	RVW'D

EXHIBIT A

100% FINAL Design Report_Appendix J1.2 (June2022) (CEII)
(pages 1 to 339)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

PAGES REDACTED IN ENTIRETY

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TABLE J2.1

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

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REV	DATE	DESCRIPTION	PREP'D	RVW'D

EXHIBIT A

100% FINAL Design Report_Appendix J2.2 (June2022) (CEII)
(pages 1 to 190)

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TABLE J3.1

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

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APPENDIX J3.2 TABLE OF CONTENTS - COPCO NO. 2

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REV	DATE	DESCRIPTION	PREP'D	RVW'D

ATTACHMENT A

100% FINAL Design Report_Appendix J3.2(June2022) (CEII)
(pages 1 to 36)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

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TABLE J4.1

KIEWIT INFRASTRUCTURE WEST CO.
KLAMATH RIVER RENEWAL PROJECT

100% DESIGN REPORT
APPENDIX J4.2 TABLE OF CONTENTS - IRON GATE

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REV	DATE	DESCRIPTION	PREP'D	RVW'D

ATTACHMENT A

100% FINAL Design Report_Appendix J4.2 (June2022) (CEII)
(pages 1 to 305)

CRITICAL ENERGY/ELECTRIC INFRASTRUCTURE INFORMATION (CEII)

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