KLAMATH RIVER RENEWAL CORPORATION

Klamath River Renewal Project

Fall Creek Fish Hatchery—Design Documentation Report

Final Design Submittal

FINAL
Revision No. 02

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October 2020

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Revision Log

Revision No.	Date	Revision Description
0	June 1, 2020	Initial Draft - 50% Design
1	September 2, 2020	90% Design
2	October 28, 2020	Final Design

1.0 Introduction and Background

1.1 Purpose

The purpose of this report is to present the design documentation associated with development of the Fall Creek Fish Hatchery Project.

1.2 Background

1.2.1 Location

The Project is located in Siskiyou County northwest of Iron Gate Dam near Yreka, California. The Project is located at the existing Fall Creek Fish Hatchery site adjacent to Fall Creek.

1.2.2 Project Description

1.2.2.1 Fall Creek Fish Hatchery

The Klamath River Renewal Project includes the removal of four dams along the Klamath River. As part of the overall Project, the existing Iron Gate Fish Hatchery (IGFH) production will be moved to the Fall Creek Hatchery site. The Fall Creek Hatchery site will be modified to upgrade existing facilities and construct new facilities for Coho (*Oncorhynchus kisutch)* and fall-run Chinook salmon (*O. tshawytscha)* production. California-Oregon Power Company (Copco) built the Fall Creek Fish Hatchery (FCFH) in 1919 as compensation for the loss of spawning grounds due to the construction of Copco No. 1 Dam. FCFH was operated by the California Department of Fish and Wildlife (CDFW) to raise approximately 180,000 Chinook salmon yearlings in continuous operation between 1979 and 2003, when it ceased operations and hatchery production on the Klamath River was consolidated at IGFH. The National Marine Fisheries Service (NMFS) and CDFW have determined the priorities for fish production at FCFH under the proposed Fish Hatchery Plan. As a state- and federally listed species in the Klamath River, Southern Oregon Northern California Coastal (SONCC) Coho Distinct Population Segment (DPS) production is the highest priority for NMFS and CDFW, followed by Chinook salmon, which support tribal, sport, and commercial fisheries. Steelhead (*O. mykiss)* production is the lowest priority. Due to limited water availability and rearing capacities at the two facilities, and recent low hatchery steelhead returns, NMFS and CDFW have determined that steelhead production will be discontinued. Table 1-1 summarizes the NMFS/CDFW goals for fish production at FCFH (data compiled from CDFW information).

Table 1-1. Fall Creek Hatchery – Fish Production Goals

Species (Juvenile Life History)	Adult Return*	Incubation Start Date	Incubation Start Number	Target Release Dates	Release Number	Release Size
Coho (Yearling)	Oct. – Dec.	Oct. – Mar.	120,000	Mar. 15 – May 1	75,000	10 fpp
Chinook	Oct. – Dec.	Oct. – Mar.	4.5M**	Pre-Mar. 31	1,250,000	520 fpp

Species (Juvenile Life History)	Adult Return*	Incubation Start Date	Incubation Start Number	Target Release Dates	Release Number	Release Size
(Sub-Yearling)						
Chinook (Sub-Yearling)	Oct. – Dec.	Oct. – Mar.	-	May 1 – June 15	1,750,000	90-100 fpp
Chinook (Yearling)	Oct. – Dec.	Oct. – Mar.	-	Oct. 15 – Nov. 20	250,000	10 fpp

*Adult trapping period from Iron Gate Fish Hatchery data

** Estimated Total Green Egg Requirement at Spawning

fpp = fish per pound

Since ceasing operations in 2003, the FCFH raceways remain and CDFW continues to run water through the raceways. The facility has retained its water rights, but substantial infrastructure improvements will be required to achieve the fish production goals following dam removal. FCFH improvements will occur within the existing facility footprint to minimize environmental and cultural resource disturbances, and the facility must be in operation prior to the drawdown of Iron Gate Reservoir. The water rights and maximum available flow for the Project are set at 10 cubic feet per second (cfs). This water right is non-consumptive and water must be returned to Fall Creek, with final designs addressing National Pollutant Discharge Elimination System (NPDES) water quality permit considerations. The proposed Fish Hatchery Plan requires CDFW to employ Best Management Practices to minimize pollutants and therapeutants being discharged to Fall Creek during hatchery operations.

1.3 Report Organization

This DDR is a record of the design effort for the Project and specifically describes the details of the design process and work effort. The DDR consists of a summary of the design elements, criteria, methods and approach, engineering calculations, and pertinent references. The major report sections and intended purpose are presented in Table 1-2.

Table 1-2. Major Report Sections and Purpose

Section	Description	Purpose
1	Introduction and Background	Presents the authorization, scope, background, a description of the overall Project, and the report organization.
2	Design Criteria	Summarizes the basic design criteria that are used as the basis for the design of the Fall Creek Fish Hatchery.
3	Project Description	Describes the Fall Creek Fish Hatchery Project.
4	Hydraulic Design	Presents the hydraulic analysis of the piping systems, fish ladder, and fish barrier systems.
5	Civil Design	Includes information related to the civil design of the Fall Creek Fish Hatchery and associated access around the site.

Section	Description	Purpose
6	Geotechnical Design	Summarizes the geotechnical design associated with the two borings, B-13 and B-14, summarized in the Geotechnical Data Report prepared by CDM Smith and AECOM Technical Services, Inc.
7	Architectural Design	Includes information related to the architectural design of the FCFH buildings.
8	Structural Design	Includes information related to the structural design of the FCFH buildings, concrete raceways and holding ponds, fish ladder, and barrier.
9	Mechanical Design	Includes information related to the mechanical design of the FCFH facility including supply water, internal building plumbing, and HVAC design.
10	Electrical Design	Includes information related to the electrical design of the FCFH facility.
11	Instrumentation and Controls	Includes information related to the instrumentation and control components of the FCFH facility.
12	Operation	Includes a summary of the anticipated FCFH facility operation.
13	References	Documents the references used in developing the design.
Appendices		
A	Hydraulic Design Calculations	Presents the detailed calculations related to hydraulic design.
B	Civil Design Calculations	Presents the detailed calculations related to civil design.
C	Structural Design Calculations	Presents the detailed calculations related to structural design.
D	Mechanical Design Calculations	Presents the detailed calculations related to mechanical design.
E	Electrical Design Calculations	Presents the detailed calculations related to electrical design.
F	Biological Design Criteria TM	Presents the detailed calculations related to biological design.
G	Water Quality Sampling TM	Summarizes the water quality data collected at the proposed Fall Creek Fish Hatchery intake site.

2.0 Design Criteria

2.1 Pertinent Data

Pertinent data for the Project include the assumed survey datum, topographic mapping, and references as described below.

2.1.1 Survey Datum

The Project data provided by the Klamath River Renewal Corporation (KRRC) were supplied in reference to the North American Vertical Datum of 1988 (NAVD88, Geoid 12B). This is the vertical datum that will be used on all drawings and in all calculations submitted as deliverable for the Project. The horizontal coordinate system is the California Coordinate System of 1983, Zone 1 North American Datum of 1983 (NAD83) in feet.

2.1.2 Topographic Mapping

Topographic data was supplied by CDM Smith and includes (1) Light Detection and Ranging (LiDAR) and sonar survey performed in 2018 by GMA Hydrology, Inc. for the entire site, and (2) a river transect and existing structure survey completed by the River Design Group.

2.2 References and Data Sources

A wide range of data sources and references was used in developing this TM. Specific data related to the conceptual design of the FCFH were obtained from the various technical analyses and memoranda prepared by CDM Smith, which include the following:

- CDM Smith. 2019. Basis of Design Report.
- CDM Smith. 2019. Geotechnical Data Report.
- CDM Smith. 2019. Klamath River Renewal Project Geotechnical Data Report.

Additional data sources, including publicly available aerial imagery, U.S. Geological Survey (USGS) maps, USGS streamflow gaging station data, soils maps, as-constructed drawings, and standard engineering reference documents, were used.

2.3 General Design Criteria and Standards

2.3.1 Standard List of Terms and Abbreviations

ACI	American Concrete Institute
ADM	Aluminum Design Manual
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating and Air-Conditioning Engineers
ASME	American Society of Mechanical Engineers

ASTM	American Society of Testing and Materials
AWS	American Welding Society
CBC	California Building Code
CCOR	California Code of Regulations
CDFW	California Department of Fish and Wildlife
cfs	cubic feet per second
CGP	Construction General Permit
DI	density index
DO	dissolved oxygen
DPS	Distinct Population Segment
ECP	Erosion Control Plan
FCFH	Fall Creek Fish Hatchery
FI	flow index
ft^3	cubic feet
fpp	fish per pound
GBR	Geotechnical Baseline Report
gpm	gallons per minute
HDPE	high-density polyethylene
HEC-RAS	Hydrologic Engineering Center River Analysis System
HMI	Human Machine Interface
hp	horsepower
HVAC	Heating, Ventilation, and Air Conditioning
IBC	International Building Code
IEEE	Institute of Electrical and Electronic Engineers
IESNA	Illuminating Engineering Society of North America
IGFH	Iron Gate Fish Hatchery
ISA	Instrument Society of America
ksf	kips per square foot
KRRC	Klamath River Renewal Corporation
kW	kilowatts
lb/cf/in	pounds of fish per cubic foot of rearing volume per inch of fish length
lbs/ft^3	pounds of fish per cubic foot of rearing space
LED	Light-Emitting Diode
LiDAR	Light Detection and Ranging survey
mA	milliamperes (or milliamps)
MDD	maximum dry density
mg/L	milligrams per liter
ml/L	milliliter per liter
mm	millimeter
mm/ctu/day	millimeters per centigrade temperature unit per day
NAD	North American Datum

NAVD	North American Vertical Datum
nd	no date
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NESC	National Electrical Safety Code
NFPA	National Fire Protection Association
NHC	Northwest Hydraulic Consultants
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
PLC	Programmable Logic Controller
Project	Fall Creek Hatchery Project
pcf	pounds per cubic foot
psf	pounds per square foot
PVC	polyvinyl chloride
RWQCB	Regional Water Quality Control Board
SS	Structural Fill
SONCC	Southern Oregon Northern California Coastal
SCADA	Supervisory Control and Data Acquisition
TM	Technical Memorandum
TSS	total suspended solids
UL	Underwriters Laboratories
USACE	United States Army Corps of Engineers
USACE EMs	United States Army Corps of Engineers Engineer Manuals
USBR	United States Bureau of Reclamation
US DOE	United States Department of Energy
USGS	United States Geological Survey
UV	Ultraviolet
V	Volts (alternating current, if not stated otherwise)
Vac	Volts (alternating current)
Vdc	Volts (direct current)

2.4 Biological

Key biological information used in the development of design criteria are based on a biological program (bioprogram) schedule developed in conjunction with CDFW Fisheries staff. The bioprogram schedule is included with this document as Figure 2-1; biological design criteria addressed below will be discussed in reference to Figure 2-1.

PRELIMINARY BIOPROGRAM AND APPROXIMATE HATCHERY OPERATION SCHEDULE
Fall Creek Hatchery - Coho Yearling / Chinook Sub-Yearling & Yearling Program

9-Mar-20

CHINOOK PRODUCTION
Egg Take - Green to Eyed Egg Period
On Station Incubation - Eyed Eggs xfr in Nov 15 at 400 CTU
Chinook Brood Year Rearing in TBD (~12x4x50 Vats Pond-Rls.) — Mark and Xfr by May 31
~ 250k Chinook Yearlings — Xfr out Nov
Coho BY-A Early Rearing in Vats & Small Raceways — Xfr to Large Ponds
Coho BY-A in Production Raceways/Vats — Xfr out Mid-April
Coho BY-B in Early Rearing Vats & Small Raceways — Xfr to Large Ponds
Coho BY-B in Production Raceways/Vats — Xfr to Large Ponds

				OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT
			(F)	49.0	46.0	43.0	43.0	43.0	46.0	50.0	54.0	54.0	54.5	54.5	50.0	49.0	46.0	43.0	43.0	43.0	46.0	50.0	54.0	54.0	54.5	54.5	50.0	49.0
FC Sub-Yearling Chinook	3,250,000	(Start Inv)						3,250,000	3,250,000	2,000,000	2,000,000	250,000	250,000	250,000	250,000	250,000	250,000			3,250,000	3,250,000	2,000,000	2,000,000	250,000	250,000	250,000	250,000	250,000
Fall Creek Monthly Mean Water Temperature (C)				9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44
Projected Growth Rate (mm/month)	0.05	mm/ctu/day						4.58	11.67	15.00	18.33	18.33	18.75	18.75	15.00	14.16	11.67			4.58	11.67	15.00	18.33	18.33	18.75	18.75	15.00	14.16
Fish Length Inches EOM - Assumes 1200 fpp & .376 g/f @ ponding				L				1.403	1.862	2.453	3.175	3.896	4.634	5.373	5.963	6.521	6.980		L	1.403	1.862	2.453	3.175	3.896	4.634	5.373	5.963	6.521
Fish Weight Grams EOM (Piper Tables; Assumes C3000)				g/f				0.376	0.871	2	4.35	7.98	13.7	20.96	28.94	37.65	45.27		g/f	0.376	0.871	2	4.35	7.98	13.7	20.96	28.94	37.65
Fish Per Pound EOM				fpp				1200 #	521 #	227 #	104#	57#	33 #	22 #	16 #	12 #	10 #		fpp	1200 #	521 #	227 #	104#	57#	33 #	22 #	16 #	12 #
Biomass In Pounds EOM				biom				2,694	6,241	8,819	19,180	4,398	7,551	11,552	15,951	20,751	24,951		biom	2,694	6,241	8,819	19,180	4,398	7,551	11,552	15,951	20,751
Volume Required EOM (cu.ft.)	0.3	DI		cu.ft.				6,401	11,169	11,983	20,139	3,763	5,431	7,167	8,916	10,608	11,915		cu.ft.	6,401	11,169	11,983	20,139	3,763	5,431	7,167	8,916	10,608
Flow Required EOM (gpm)	1.5	FI		680	680	680	680	1,280	2,234	2,397	4,028	753	1,086	1,433	1,783	2,122	2,383			1,280	2,234	2,397	4,028	753	1,086	1,433	1,783	2,122
Assume ~4.5M green; 4,136 green eggs/tray; 1,088 trays = 136 1/2 stacks								680	1.25M Rls End Mar		1.75M Rls End May (post-mark 150/lb)				Incub:	680	680	680	680	680	1.25M Rls End Mar		1.75M Rls End May					680
CDFW Growth Reduction; Days Feed/Month																23 days	15 days	7 days	7 days	7 days	15 days							
									78,400	77,600	77,400	77,200	77,000	76,800	76,600	76,400	76,200	76,000	75,800	75,600	75,400	75,000						
FC Yearling Coho	80,000	(Start Inv)													Ration:	75%	50%	25%	25%	25%	50%	Ap. 15 Rls						
Fall Creek Monthly Mean Water Temperature (C)				9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44
Projected Growth Rate (mm/month)	0.045	mm/ctu/day						4.12	10.50	13.50	16.50	16.50	16.88	16.88	13.50	9.77	5.25	1.92	1.92	1.92	5.25	4.50						
Fish Length Inches EOM - Assumes 1400 fpp & .323 g/f @ ponding				L				1.270	1.684	2.215	2.864	3.514	4.178	4.843	5.374	5.759	5.966	6.041	6.117	6.193	6.400	6.577						
Fish Weight Grams EOM (Piper Tables; Assumes C3500)			Brood Year A	g/f				0.323	0.744	1.72	3.72	6.99	11.7	18.23	24.77	30.2	33.5	35.3	37.5	37.8	41.8	45.4						
Fish Per Pound EOM				fpp				1400 #	610 #	263 #	122#	65 #	39 #	25 #	18 #	15 #	14 #	12.9 #	12.1 #	11.9 #	10.8 #	10 #						
Biomass In Pounds EOM				biom				57	129	294	635	1,190	1,986	3,087	4,183	5,087	5,628	5,915	6,267	6,300	6,948	7,500						
Volume Required EOM (cu.ft.)	0.3	DI		cu.ft.				150	255	443	739	1,129	1,584	2,125	2,595	2,944	3,145	3,263	3,415	3,391	3,619	3,801						
Flow Required EOM (gpm)	1.5	FI		40	40	40	40	30	51	89	148	226	317	425	519	589	629	653	683	678	724	760						40
CDFW Growth Reduction; Days Feed/Month					15 days	7 days	7 days	7 days	15 days																			23 days
					76,200	76,000	75,800	75,600	75,400	75,000											78,400	77,600	77,400	77,200	77,000	76,800	76,600	76,400
FC Yearling Coho	80,000	(Start Inv)		Ration:	50%	25%	25%	25%	50%	Ap. 15 Rls																		
Fall Creek Monthly Mean Water Temperature (C)				9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44	7.78	6.11	6.11	6.11	7.78	10	12.22	12.22	12.5	12.5	10	9.44
Projected Growth Rate (mm/month)	0.045	mm/ctu/day			5.25	1.92	1.92	1.92	5.25	4.50										4.12	10.50	13.50	16.50	16.50	16.88	16.88	13.50	9.77
Fish Length Inches EOM - Assumes 1400 fpp & .323 g/f @ ponding				L	5.966	6.041	6.117	6.193	6.400	6.577									L	1.270	1.684	2.215	2.864	3.514	4.178	4.843	5.374	5.759
Fish Weight Grams EOM (Piper Tables; Assumes C3500)			Brood Year B	g/f	33.5	35.3	37.5	37.8	41.8	45.4									g/f	0.323	0.744	1.72	3.72	6.99	11.7	18.23	24.77	30.2
Fish Per Pound EOM				fpp	14 #	12.9 #	12.1 #	11.9 #	10.8 #	10 #									fpp	1400 #	610 #	263 #	122#	65 #	39 #	25 #	18 #	15 #
Biomass In Pounds EOM				biom	5,628	5,915	6,267	6,300	6,948	7,500									biom	57	129	294	635	1,190	1,986	3,087	4,183	5,087
Volume Required EOM (cu.ft.)	0.3	DI		cu.ft.	3,145	3,263	3,415	3,391	3,619	3,801									cu.ft.	150	255	443	739	1,129	1,584	2,125	2,595	2,944
Flow Required EOM (gpm)	1.5	FI		gpm	629	653	683	678	724	760						40	40	40	40	30	51	89	148	226	317	425	519	589
			GPM	720	1,349	1,373	1,403	2,668	3,009	3,245	4,176	978	1,403	1,858	2,302	3,430	3,732	1,373	1,403	2,668	3,009	3,245	4,176	978	1,403	1,858	2,302	3,430
			CFS	1.6	3.0	3.1	3.1	5.9	6.7	7.2	9.3	2.2	3.1	4.1	5.1	7.6	8.3	3.1	3.1	5.9	6.7	7.2	9.3	2.2	3.1	4.1	5.1	7.6
			Tot. Adult Flow	10.0	10.0	10.0									10.0	10.0	10.0	10.0									10.0	10.0
				OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT

Figure 2-1. Biological Program Schedule – Fall Creek Fish Hatchery

2.4.1 Fish Development Cycle

The colored bars across the top section of Figure 2-1 depict the timing of adult spawning and resulting egg incubation, juvenile fish rearing, and a general approach to fish transfer based on marking and release ("first-feeding" vessels and "grow-out" vessels). The adult holding/spawning process is assumed to mirror current adult holding and spawning at the IGFH and occurs from October through December. Egg/alevin incubation is initiated at the onset of adult spawning and generally runs through March. Egg incubation activities are assumed to be flexible in the initial years of the program as eggs may be sourced from one or more CDFW egg production stations and/or sourced from the most appropriate natural anadromous brood sources. Early rearing will begin as first-feeding fry are ponded, and this period will generally extend until the marking/tagging is completed. The ultimate marking/tagging dates and numbers will be determined after further input from CDFW. Early-rearing tanks/vessels will be designed and sited with consideration for fish collection through the marking trailer, as well as differentiating between marked/tagged and non-marked/tagged groups. Final grow-out rearing will provide adequate rearing space and collection/release methods for fish at release.

2.4.2 Biological Variables

The primary biological variables used in the preparation of the preliminary operations schedule include water temperature, species-specific condition factor/growth rates, fish weight/length targets, and density and flow indices.

2.4.2.1 Water Temperature

Water temperature is a primary determining factor in the development and growth rate of fish. Figure 2-1 (row 2 for each cohort group) provides mean water temperature data that are used to estimate the rate of fish growth, which is also tied to feed rate. Temperature profiles for the Fall Creek source water are considered ideal for the culture of Pacific salmon. CDFW's prior rearing experience at the Fall Creek facility with Chinook salmon demonstrate that rearing conditions are favorable for the production of high-quality juvenile salmon. CDFW-provided mean monthly water temperature data for Fall Creek is presented below in Figure 2-2.

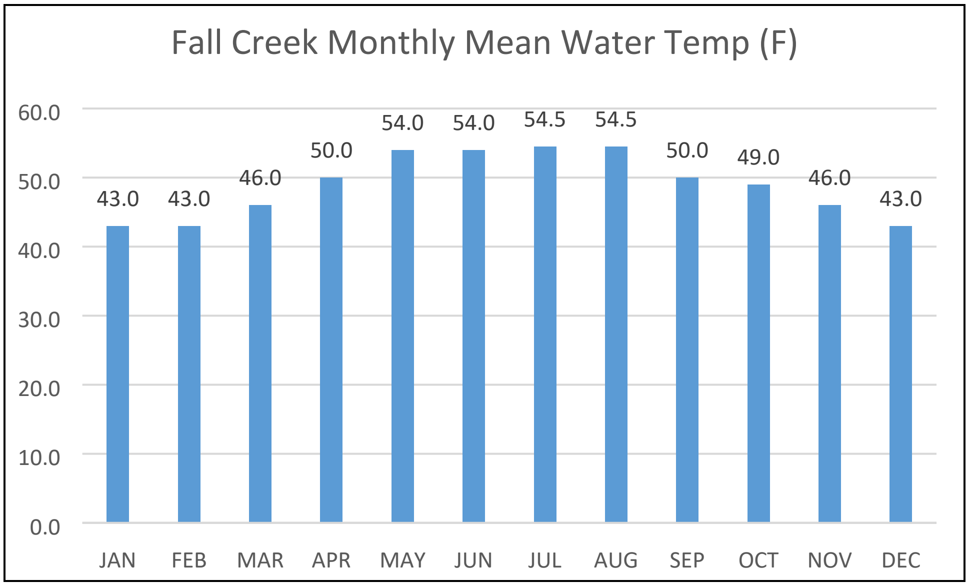

Figure 2-2. Mean Monthly Fall Creek Rearing Temperature Data (Data from L. Radford, CDFW)

2.4.2.2 Expected Growth Rates

The projected monthly growth rate shown in Figure 2-1 (row 3 for each cohort group) is 0.045 and 0.05 millimeters per centigrade temperature unit per day (mm/ctu/day) for Coho and Chinook, respectively. Growth rates are applied to mean water temperatures to develop an estimate of total growth (millimeters per month), which is tied directly to feed rate. Within an ideal water temperature range for salmonids and in the absence of feed modulation, fish will grow faster at higher water temperatures than at lower temperatures (increased daily/monthly growth in millimeters at elevated water temperature range). CDFW does not plan to use chilled water (i.e., water chiller units) for incubation and/or grow-out rearing strategies. For the new facility, CDFW will rely on ambient Fall Creek water temperature profile.

2.4.2.3 Fish Weight and Length

Row 4 of each cohort group shown in Figure 2-1 depicts the cumulative fish length in inches, which is determined by adding the growth per month to the fish length at the end of the preceding month. The mean weight of individual fish in grams is shown in the row below the length (row 5); mean weights are obtained from Piper et al. (1982) Length-Weight Tables for the specific condition factor of fish in culture (Coho C3500, Chinook C3000; $Cx10^{-7}$).

2.4.2.4 Density Index

Density index (DI) is a function of pounds of fish per cubic foot of rearing volume per inch of fish length (lbs fish/cf volume/length [inch]). CDFW staff have agreed to rear fish at a maximum DI of 0.3 for the Coho and Chinook programs at Fall Creek; 0.3 is a conservative DI that is reflective of similar conservation/recovery programs for anadromous Pacific salmon juveniles throughout the Pacific Northwest.

The DI is then used to calculate the total volume of rearing space required in terms of cubic feet. Figure 2-1 (row 8) shows the rearing volume required at the end of each month as fish size increases from left to right. The total volume is then divided by the cubic foot volume of individual rearing tanks/vessels to determine the total number of rearing units required.

2.4.2.5 Flow Index

Flow index (FI) is a function of pounds of fish divided by fish length in inches times flow in gallons per minute (gpm). Flow index is an indication of how much oxygen is available for fish metabolism and is adjusted based on the elevation of the project site and water temperature, both of which affect the amount of oxygen in the water supply at saturation. CDFW staff have agreed to rear fish at a maximum FI of 1.50 for the Coho and Chinook programs at Fall Creek; 1.50 is a conservative FI that is reflective of similar conservation/recovery programs for anadromous Pacific salmon juveniles throughout the Pacific Northwest (at similar elevations and water temperature profiles).

2.4.3 Egg Take and Fish Survival

Current rearing production program scenarios plan for a total of 75,000 Coho salmon and approximately 3.25 million Chinook salmon at various release dates. Mean survival rate estimates provided by CDFW for the IGFH program suggest a green egg to ponding (first-feeding) survival rate of approximately 73 percent. Based on the 73 percent survival estimates, approximately 120,000 green eggs will be required for the Coho program and approximately 4.5 million green eggs will be required for the Chinook program. Acknowledging improved incubation water quality at Fall Creek (vs. poorer Iron Gate water quality) and reduced tray loading densities, survival rates are anticipated to increase as the program develops rearing techniques that favor increased survival.

2.4.4 Incubation and Rearing Facilities

This section provides a brief summary of the incubation and rearing flows, as well as rearing volumes depicted in Figure 2-1.

2.4.4.1 Incubation

Incubation systems currently at IGFH will be used for egg/alevin incubation at Fall Creek. A total of 130 incubation stacks are currently available for future rearing needs. The existing incubation units are vertical stack incubators with a double-stack arrangement with 15 useable trays per stack (full-stack/with the top tray used as sediment tray). Water flow requirements are modeled at 5 gpm, per manufacturer's recommendations, which is an industry standard, regardless of eight-tray or 16-tray configuration.

Early hydraulic modeling efforts indicated that egg incubation systems (vertical stack incubators) would require auxiliary pumping if full-stack arrangements were required (16-tray configuration). In stressing the importance of gravity-flow systems to the extent possible, CDFW staff elected for an eight-tray (half-stack) configuration for all incubation systems at FCFH. Additionally, CDFW staff acknowledge that reducing the tray loading densities for the Chinook program will likely result in increased survival. The current design efforts will assume approximately 50 to 55 ounces of Chinook eggs per tray rather than the approximately 100 ounces/tray currently used at IGFH.

Incubation requirements based on new loading densities for Chinook are approximately 136 half-stack incubators (1,088 trays) requiring approximately 680 gpm. Chinook incubator units are proposed as eight-tray loading with an extra incubation tray on top of the unit acting as a sediment tray (ninth tray without screening used to settle sediment). Incubation requirements for the Coho program are unchanged from the original planning efforts and require six half-stack incubators (approximately 40 trays required) using approximately 30 gpm of water. Coho incubator units have the flexibility (tray space) to accommodate a seven-tray loading configuration with the eighth tray (top) used as a sediment tray.

2.4.4.2 Early Rearing

First-feeding and early-rearing vessel requirements are based on fish size estimates from the bioprogram for the period of ponding through the marking stage of rearing. Maximum bioprogram requirements for rearing space and water flow resulted in approximately 3,850 cubic feet of rearing space and approximately 760 gpm for Coho and approximately 20,200 cubic feet and 4,050 gpm for Chinook. Acknowledging the maximum space and flow required at peak production for each species, the estimated rearing space required for early-rearing through marking phases are identified below:

- Coho Early-Rearing: Total rearing required at mark size of about 150 fish per pound (fpp) – 650 ft^3
- Chinook Early-Rearing: Total rearing required at mark size of about 150 fpp – 16,000 ft^3

Total early-rearing space provided for Coho is approximately 825 ft^3 of fiberglass vat rearing and an additional 1,200 ft^3 available in renovated concrete raceways; the renovation of the concrete raceways provides a total of eight individual rearing containers that can be used to maximize the population compartmentalization of the listed Coho stock. Total early-rearing space provided for Chinook is approximately 20,200 ft^3 and provides maximum compartmentalization for cohort groups of between 204,000 (16 rearing units) and 408,000 (eight rearing units) fish, depending on mean fish size.

The maximum production/flows for Coho occur at mid-April release and the maximum biomass/flows for Chinook occur at late-May release, as shown in Figure 2-1. Coho brood cohorts (first-feeding fry and smolt program) will overlap from early-ponding through smolt release; Coho production for the second cohort is assumed to require approximately 650 ft^3 of rearing space (the four fiberglass vats) and 90 gpm from first-feeding through late-April transfer to larger production ponds (post-smolt release).

2.4.4.3 Juvenile Rearing

Grow-out vessel requirements based on Figure 2-1 result in a maximum grow-out rearing need of 3,800 ft^3 of Coho rearing space (April release) and approximately 20,200 ft^3 of Chinook rearing space (May release) based upon the bioprogram. Total rearing volume provided in the facility design is 4,190 ft^3 for Coho and 23,040 ft^3 for Chinook. Raceway drains for both Coho and Chinook units have been designed to allow for volitional emigration of fish directly to Fall Creek; volitional water supply routing is described later in this document.

2.4.4.4 Adult Holding

Adult holding and spawning ponds have been designed per CDFW recommendations and align with NOAA guidelines for anadromous adults as closely as possible. The existing raceway series currently on-site (south of Copco Road) will be retained, renovated, and will provide sufficient space to hold the requested 100 Coho and 200 Chinook pre-spawn adults. One of the four existing raceways will act as a primary trapping and handling pond, with two ponds renovated to act as longer-term holding for pre-spawn Coho and Chinook adults. The remaining pond will be used as a settling pond and is described later in the report. All non-cleaning (effluent) flow, which will be a maximum of 10 cfs, will be routed to the adult ponds and used for adult holding and fish ladder attraction flows when required, which is assumed between September and December.

The three adult rearing ponds will be renovated with screen and stoplog keyways (and adequate quiescent zones; effluent collection) to allow for the potential short-term rearing of juvenile Chinook that would have otherwise been released early because of space limitations in the Chinook rearing raceway complex. Flow to the holding ponds is second-pass, untreated water from the Coho and Chinook rearing facilities. However, the second pass water should be of sufficient quality and oxygen levels for surplus juvenile Chinook because of the conservative density and flow indices used in the biological program. Assuming three raceways with approximately 2,500 ft^3 of vacant space per unit (12.5'W x 50'L x 4'D useable space; 7,500 ft^3 total), serial reuse flows from the upper production units, and using a 0.3 density index, the maximum permissible weight of 3.175-inch fish (about 104 fpp) would be approximately 7,100 pounds (about 740,000 fish at 104 fpp). Drains have been designed to provide volitional emigration of fish to Fall Creek; volitional water supply routing from this series is described later in this document.

2.4.5 Peak Water Demand

Appendix A provides a water budget for an entire calendar year with a peak water demand of 9.3 cfs projected for May of each year immediately prior to Chinook sub-yearling releases and when juvenile Coho are in early rearing containers. The projected annual water budget by month is also provided below in Table 2-1.

Table 2-1. Fall Creek FH Water Requirements – Full Production

Month:	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Total Juv. CFS	3.1	5.9	6.7	7.2	9.3	2.2	3.1	4.1	5.1	7.6	8.3	3.1
Total Ladder CFS	-	-	-	-	-	-	-	-	10.0	10.0	10.0	10.0

2.5 Civil

2.5.1 Erosion Control Plan

The contractor will be required to obtain a Construction Storm Water General Permit from the California State Water Resources Control Board prior to construction. Construction General Permits (CGPs) are required for construction projects that result in greater than 1 acre of soil disturbance. The CGP requires temporary and post-construction Best Management Practices to prevent erosion and reduce sediment discharges from construction sites.

Prior to permit issuance by Siskiyou County, submittal of an Erosion Control Plan (ECP) to the appropriate Director at Siskiyou County is required. The ECP shall include methods for controlling runoff, erosion, and sediment movement.

2.5.2 Hatchery Effluent Discharge

The California Regional Water Quality Control Board (RWQCB) requires hatchery facilities that discharge effluent to obtain an NPDES permit to regulate the hatchery effluent discharge. It is assumed that the waste stream from FCFH will be required to meet effluent limitations included in the California Regional Water Quality Control Order No. R1-2015-0009, General NPDES CAG131015, Waste Discharge Requirements for Cold Water Concentrated Aquatic Animal Production Facility Discharges to Surface Waters.

2.5.3 Stormwater Control

The federal Clean Water Act requires facilities that discharge stormwater runoff to obtain an NPDES permit to regulate the discharge of stormwater into surface waters such as Fall Creek. The design of the FCFH site will minimize the addition of impervious areas. The addition of impervious areas will be limited to rooftops and gravel surfacing around the site. The drainage from new impervious areas will be routed to a storm drain system that will provide treatment before discharging water to Fall Creek. The storm drain system was sized to treat a water quality storm event of 0.2 inches per hour (in/hr) per California Stormwater Quality Association recommendations and local rain data, and to withstand the 100-year rainfall event without ponding on the road surfaces.

2.5.4 Grading

According to the California Building Code adopted by the County of Siskiyou design standards, slopes shall be no steeper than 2 horizontal (H) to 1 vertical (V). Steeper slopes may be allowed if the Building Official determines they will be stable or if a geotechnical engineer certifies that the site has been investigated and that the proposed deviation will be and will remain structurally stable.

2.5.5 Site Access

Modeling to simulate site access conditions was performed using AutoTurn software and the following design vehicles:

- Standard pickup truck (2019 Ford F-450, Crew Cab).
- Marking and tagging trailer for access and egress from the Coho and Chinook rearing ponds (43.0-foot-long Newmar X-Aire 2009, on a 21.85-foot-long design truck, based on typical marking trailers used by the U.S. Fish and Wildlife Service).
- Septic pump truck for access and egress from the settling pond, storm drain hydrodynamic separators, and vault toilet (33.6-foot-long design truck).

2.6 Hydraulic

The proposed hydraulic engineering criteria are presented in the tables below. A brief description of the contents of each table is as follows:

- **Table 2-2.** Hydraulic Standards, References, and Standards of Practice
- **Table 2-3.** Governing Hydrological Criteria for Adult Salmon Facilities
- **Table 2-4.** Inlet Structure Hydraulic Criteria
- **Table 2-5.** Supply Piping Hydraulic Criteria
- **Table 2-6.** Drain Piping Hydraulic Criteria
- **Table 2-7.** Volitional Fish Release Pipe Hydraulic Criteria
- **Table 2-8.** Coho Rearing Hydraulic Criteria
- **Table 2-9.** Chinook Rearing Hydraulic Criteria
- **Table 2-10.** Adult Holding Hydraulic Criteria
- **Table 2-11.** General NPDES CAG131015 Effluent Limitations
- **Table 2-12.** Settling Pond Hydraulic Criteria
- **Table 2-13.** Fish Ladder Hydraulic Criteria
- **Table 2-14.** Fish Barrier Hydraulic Criteria

2.6.1 Applicable Codes and Standards

The following codes, standards, and specifications will serve as the general design criteria for the hydraulic design of the FCFH facilities.

Table 2-2. Hydraulic Standards, References, and Standards of Practice

Standard	Reference
ASCE, 1975	American Society of Civil Engineers (ASCE). 1975. *Pipeline Design for Water and Wastewater*. ASCE: New York, NY.
CDFW, 2004	California Department of Fish and Wildlife (CDFW). 2004. *California Salmonid Stream Habitat Restoration Manual.* March 2004.
Chow, 1959	Chow, V.T. 1959. *Open Channel Hydraulics*. McGraw-Hill Book Company: New York, NY.
Idaho DEQ, nd	Idaho Department of Environmental Quality. nd. *Idaho Waste Management Guidelines for Aquaculture Operations.*
Lindeburg, 2014	Lindeburg, M.R. 2014. *Civil Engineering Reference Manual, Fourteenth Edition.* Professional Publications, Inc.: Belmont, CA.
Miller, 1990	Miller, D.S. 1990. *Internal Flow Systems.* The Fluid Engineering Centre, BHRA: Cranfield, UK.
NMFS, 2011	National Marine Fisheries Service (NMFS). 2011. *Anadromous Salmonid Passage Facility Design*. National Oceanic and Atmospheric Administration, NMFS, Northwest Region: Portland, OR.
NOAA Atlas 14	National Oceanic and Atmospheric Administration (NOAA). 2014. *Precipitation-Frequency Atlas of the United States, Volume 6 Version 2.3: California.* NOAA, National Weather Service: Silver Spring, MD.

Standard	Reference
Rossman, 2000	Rossman, L.A. 2000. EPANET2, User's Manual. U.S. Environmental Protection Agency (USEPA), Office of Research and Development, National Risk Management Research Laboratory: Cincinnati, OH.
Tullis, 1989	Tullis, J.P. 1989. *Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients*. John Wiley & Sons, Inc.
USFWS, 2017	U.S. Fish and Wildlife Service (USFWS). 2017. *Fish Passage Engineering Design Criteria*. USFWS, Northeast Region RG, Hadley, MA.
USBR, 1987	U.S. Bureau of Reclamation (USBR). 1987. *Design of Small Dams*. U.S. Department of the Interior, USBR: Washington, D.C.

2.6.2 Fall Creek Hydrology

USGS Gage Station No. 11512000 was used to estimate the hydrology of Fall Creek near the proposed FCFH site. This gage station is located approximately two-thirds of a mile downstream from the existing lower raceway bank at the site, and therefore provides the best representation of flows at the site. The data record consists of daily average discharge, and extends from 1933 to 1959, and then from 2003 to 2005. Table 2-3 below presents the governing hydrological criteria used as the basis of the design for adult collection facilities at FCFH.

Table 2-3. Governing Hydrological Criteria for Adult Salmon Facilities

Criteria	Units	Value	Comments
Period of Anadromous Fish Present at Site	-	Oct – Dec	See Bioprogram
95% Exceedance Streamflow (Fish Passage Low Flow)	cfs	23.4	NMFS, 2011; for period when anadromous fish are present at the site
50% Exceedance Streamflow (Fish Passage Typical Flow)	cfs	30.1	NMFS, 2011; for period when anadromous fish are present at the site
5% Exceedance Streamflow (Fish Passage High Flow)	cfs	46.8	NMFS, 2011; for period when anadromous fish are present at the site
1% Exceedance Streamflow (Fish Passage High Flow)	cfs	71.9	CDFW, 2004; alternative high flow definition, for period when anadromous fish are present at the site
1% Exceedance Streamflow (Juvenile High Flow)	cfs	76.9	High flow for maximum flow month during juvenile release (March)
2-year Flood Event Streamflow	cfs	115.3	Adjusted from downstream USGS Gage 11512000
100-year Flood Event Streamflow	cfs	756.2	Adjusted from downstream USGS Gage 11512000
2-year, 24-hour Precipitation Depth	in	1.94	NOAA Atlas 14, Volume 6, Version 2
10-year, 24-hour Precipitation Depth	in	2.88	NOAA Atlas 14, Volume 6, Version 2
100-year, 24-hour Precipitation Depth	in	4.43	NOAA Atlas 14, Volume 6, Version 2

2.6.3 Fall Creek Intake Structure

A non-consumptive water diversion from Fall Creek will support hatchery operations by construction of a new intake structure at Dam A. Water demand for facility operations will vary to meet biological criteria for various life stages of fish development. Table 2-4 below summarizes the design criteria used to support the design of the intake structure at Dam A on Fall Creek.

Table 2-4. Intake Structure Hydraulic Criteria

Criteria	Units	Value	Comments
Design Flow	cfs	10	FCFH Water Right and Proposed Maximum Diversion Flow from Fall Creek to Project Site
Design Water Surface Elevation	ft	2510.4	Elevation of Dam A at crest
Trash Rack Percent Open Area	%	50	Typical, subject to screen manufacturer specifications
Maximum Allowable Trash Rack Occlusion	%	40	Assumed, conservative for an automatically cleaned screen
Pipe Entrance Loss Coefficient, K_e	-	0.7	USBR, 1987; Maximum for open pipe with downstream isolation valve
Screen Cleaning System	-	See Comment	Automatic active water spray bar system.

2.6.4 Supply Piping

The supply piping network was analyzed using EPANET2 software (Rossman, 2000) to determine the head at the design locations, and to size the water supply pipes in the network. The supply piping consisted of four main distribution networks: (1) the Coho building distribution piping, (2) the Chinook raceway distribution piping, (3) the Chinook Incubation Building distribution piping, and (4) the adult holding pond distribution piping. These constituted four separate models in the EPANET2 software. Table 2-5 below summarizes the supply piping initial hydraulic criteria used to develop the EPANET2 model. For a full discussion of the supply piping scenarios modeled (and associated conditions and coefficients, see Section 4.1).

Table 2-5. Supply Piping Hydraulic Criteria

Criteria	Units	Value	Comments
Pipe Hazen-Williams Coefficient	-	120	ASCE 1975; Small diameter of good workmanship or large diameter of ordinary workmanship. Schedule 80 PVC material.
Minor Loss Coefficient – 90° Bend	-	0.24	Tullis, 1989
Minor Loss Coefficient – 45° Bend	-	0.10	Tullis, 1989
Minor Loss Coefficient – 22.5° Bend	-	0.06	Tullis, 1989
Minor Loss Coefficient – Butterfly Valve (Open)	-	0.2	Tullis, 1989

Criteria	Units	Value	Comments
Minor Loss Coefficient – Tee (Branch Flow)	-	1.0	Miller, 1990; Approx. 60%-40% Flow Split
Minor Loss Coefficient - Tee (Line Flow)	-	0.2	Miller, 1990; Approx. 60%-40% Flow Split
Minor Loss Coefficient - Reducer	-	See Comment	Calculated based on relative pipe size according to Tullis 1989

2.6.5 Drain Piping

The online drain pipeline will convey effluent from the rearing vessels to the adult holding ponds and will ultimately be discharged into Fall Creek via the new fish ladder. All outlet pipes and trunk lines were sized to maintain open-channel flow with the exception of pipe risers into the adult holding ponds (see Section 4.2). Table 2-6 below summarizes the drain piping hydraulic criteria used to develop the drain piping hydraulic calculations.

Table 2-6. Drain Piping Hydraulic Criteria

Criteria	Units	Value	Comments
Gravity Flow – Maximum Flow Depth	%	75	Prevent pressurizing of pipe for presence of waves, etc. Generally less than 70%
Minimum Self-Cleaning Velocity	ft/s	1.5	Typical, Sewer Design
Typical Self-Cleaning Velocity	ft/s	2.0	Typical, Sewer Design
Gravity Flow Pipe Manning's Roughness Coefficient, n	-	0.013	Maximum; Plastic Pipe
Pressure Pipe Relative Roughness	in	6.0×10^{-5}	Lindeburg, 2014; Plastic Pipe
Minor Loss Coefficient – 90° Bend	-	0.24	Tullis, 1989
Minor Loss Coefficient – 45° Bend	-	0.10	Tullis, 1989
Minor Loss Coefficient – Tee (Branch Flow)	-	1.0	Miller, 1990; Approx. 60%-40% Flow Split
Minor Loss Coefficient - Tee (Line Flow)	-	0.2	Miller, 1990; Approx. 60%-40% Flow Split
Orifice Discharge Coefficient	-	0.62	Lindeburg, 2014; Sharp-Edge

2.6.6 Volitional Fish Release Pipes

The volitional fish release pipes will convey juvenile fish from the rearing raceways to various discharge points in Fall Creek. Pipe design was subject to design criteria from NMFS (2011) for fish bypass pipes. Table 2-7 below summarizes the fish release piping hydraulic design criteria.

Table 2-7. Volitional Fish Release Pipe Hydraulic Criteria

Criteria	Units	Value	Comments
Gravity Flow – Maximum Flow Depth	%	75	Prevent pressurizing of pipe for presence of waves, etc. NMFS, 2011; Section 11.9.3.2 Generally less than 70%
Gravity Flow – Minimum Flow Depth	%	40	NMFS, 2011; Section 11.9.3.9
Minimum Bend Radius R/D	-	5.0	NMFS, 2011; Section 11.9.3.4 Greater for supercritical flows; Bend radius 5 times the pipe diameter
Typical Access Port Spacing	ft	150	NMFS, 2011; Section 11.9.3.5
Maximum Pipe Velocity	ft/s	12.0	NMFS, 2011; Section 11.9.3.8
Minimum Pipe Velocity	ft/s	6.0	NMFS, 2011; Section 11.9.3.8 Generally less than 6.0 ft/s, absolute minimum of 2.0 ft/s
Minimum Pipe Diameter	in	10	NMFS, 2011; Table 11-1
Plunge Pool Maximum Impact Velocity	ft/s	25.0	NMFS, 2011; Section 11.9.4.2
Plunge Pool Minimum Depth	ft	4.0	USFWS, 2017; Reference Plate 9-2 Up to an equivalent drop height of 16’, then ¼ of the equivalent drop height

2.6.7 Rearing Facilities

Based upon the biological design criteria summarized above, Table 2-8, Table 2-9, and Table 2-10 below summarize the hydraulic criteria, flow, and volume requirements for each of the rearing facilities at FCFH.

Table 2-8. Coho Rearing Hydraulic Criteria

Criteria	Units	Value	Comments
Maximum Rearing Volume Requirement	ft^3	3,850	See Bioprogram
Maximum Flow Requirement	gpm	765	See Bioprogram; Flow to rearing raceways only, additional flow to first-feeding vessels
Cleaning Method	-	See Comment	Vessels to be cleaned using vacuum system
Cleaning Maximum Flow	gpm	200	Assumed. Two vessels cleaned at one time. Intermittent flow.

Table 2-9. Chinook Rearing Hydraulic Criteria

Criteria	Units	Value	Comments
Maximum Rearing Volume Requirement	ft^3	20,190	See Bioprogram
Maximum Flow Requirement	gpm	4,040	See Bioprogram
Cleaning Method	-	See Comment	Vessels to be cleaned using vacuum system
Cleaning Maximum Flow	gpm	200	Assumed

Table 2-10. Adult Holding Hydraulic Criteria

Criteria	Units	Value	Comments
Chinook Holding Capacity	#	200	See Bioprogram
Coho Holding Capacity	#	100	See Bioprogram
Adult Chinook Weight	lbs	12	Estimated, CDFW
Adult Coho Weight	lbs	8	Estimated, CDFW
Minimum Holding Volume	ft^3/lb-biomass	0.75	NMFS, 2011; long-term holding: Holding > 72 hours, 0.75 x Weight of Fish: If temperature exceeds 50°F, reduce pounds of fish by 5% for each degree over 50°F
Minimum Adult Holding Flow	gpm/fish	2 (long-term holding)	NMFS, 2011; 0.67 gpm per fish for short-term holding. Increase three times for fish held over 72 hours.
Jump Protection Height	ft	5.0	NMFS, 2011; to meet jump minimization criterion, alternatively nets, coverings, or sprinklers may be used

2.6.8 FCFH Wastewater Treatment

Flow-through water through the rearing facilities will be discharged to the adult holding ponds and ultimately through the fish ladder without treatment. Wastewater flows consisting of solids collected through vacuuming rearing vessels and flows treated with therapeutants will be discharged to a new settling pond for treatment. The downstream end of the settling pond will be equipped with an overflow structure that will divert overflows into the fish ladder to be mixed with the adult holding pond overflows and ultimately to Fall Creek.

The east-most pond in the existing lower concrete raceway bank will be repurposed as a settling pond that will be used to settle out any biosolids or other solid waste from cleaning of the upstream facilities. This pond will be refurbished and parsed into two distinct chambers such that solids can be dried in one chamber while the other is in use. It is assumed that the waste stream from FCFH will be required to meet effluent limitations included in the California Regional Water Quality Control Order No. R1-2015-0009, General NPDES CAG131015, and Waste Discharge Requirements for Cold Water Concentrated Aquatic Animal Production Facility Discharges to Surface Waters. The General NPDES CAG131015 effluent limitations and the hydraulic criteria used to design the settling basin are summarized in Table 2-11 and Table 2-12 below.

Table 2-11. General NPDES CAG131015 Effluent Limitations

Criteria	Units	Value	Comments
Average Monthly Total Suspended Solids (TSS)	mg/L	8	Net Increase Over Influent Limitations
Maximum Daily TSS	mg/L	15	Net Increase Over Influent Limitations
Average Monthly Settleable Solids	ml/L	0.1	Net Increase Over Influent Limitations
Maximum Daily Settleable Solids	ml/L	0.2	Net Increase Over Influent Limitations
pH	-	7 to 8.5	Receiving water shall not be depressed below or above the pH values identified. If the influent exceeds a pH of 8.5, the pH of the effluent shall not exceed the pH of the influent.
Receiving Water Dissolved Oxygen (DO) Non-Spawning	mg/L	≥7.0	Effluent shall not cause the dissolved oxygen (DO) of the receiving water to be depressed below 7.0 mg/L during non-spawning and egg incubation periods.
Receiving Water DO during Critical Spawning and Egg Incubation Periods	mg/L	≥9.0	Effluent shall not cause the DO of the receiving water to be depressed below 7.0 mg/L during spawning and egg incubation periods.
Turbidity	%	20	Effluent shall not cause receiving waters to be increased more than 20% above naturally occurring background levels.
Temperature	ºF	≤5	Net Increase above natural temperature of receiving water.

Table 2-12. Settling Pond Hydraulic Criteria

Criteria	Units	Value	Comments
Design Discharge	gpm	200	Only water used during vacuum cleaning routed through the settling pond. Intermittent flow.
Design Settling Velocity	ft/s	1.51×10^{-3}	Idaho DEQ, nd; Settling velocity is the maximum overflow rate from the settling pond
Overflow Weir Discharge Coefficient	-	3.33	Assumed

2.6.9 FCFH Fish Ladder

A concrete fish ladder will be constructed from Fall Creek up to the existing concrete outlet structure at the lower raceway bank. The ladder will terminate at the finger weir at the downstream end of the trapping and sorting pond and will convey fish into the pond for sorting. The fish ladder will be of the Denil steep pass type as described in the NMFS (2011) guidelines and will have two pools separated by a weir at the top for turning into the pond structure. The design criteria used to design the fish ladder, so that the fish ladder is passable to the target fish with available flow, are included in Table 2-13 below.

Table 2-13. Fish Ladder Hydraulic Criteria

Criteria	Units	Value	Comments
Fish Ladder Type	-	See Comment	Denil Steep pass
Design Discharge	cfs	10	Full water right
Minimum Attraction Flow	cfs	4.7	NMFS, 2011; Section 4.2.2.3; 10% Fish Passage High Flow
High Tailwater Elevation	ft	2,484.77	Modeled in HEC-RAS
Typical Tailwater Elevation	ft	2,484.27	Modeled in HEC-RAS
Low Tailwater Elevation	ft	2,484.12	Modeled in HEC-RAS
Debris Characterization		See Comment	NMFS, 2011; Section 4.10.2.1; Very little debris is expected as this is the downstream extents of the facility and water will have been screened multiple times
Maximum Slope	%	20	NMFS, 2011; Section 4.10.2.1
Maximum Average Chute Velocity	ft/s	5	NMFS, 2011; Section 4.10.2.1
Maximum Horiz. Distance between Rest Pools	ft	25	NMFS, 2011; Section 4.10.2.1
Minimum Flow Depth	ft	2	NMFS, 2011; Section 4.10.2.1
Minimum Flow Depth over Weir	ft	1.0	NMFS, 2011; Section 4.5.3.2
Energy Dissipation Factor	ft-lbs/s/ft^3	4.0	NMFS, 2011; Section 4.5.3.5

2.6.10 FCFH Fish Barriers

A system of fish exclusion barriers will be constructed that will (1) exclude adult and juvenile fish passage upstream of existing Dams A and B year-round, and (2) direct adult fish into the fish ladder during the trapping season. The fish barrier system will consist of three components: (1) a high-velocity concrete apron on the downstream side of Dam A, (2) a high-velocity concrete apron on the downstream side of Dam B, and (3) a set of removable picket panels on a concrete apron immediately upstream of the fish ladder. The NMFS requirements and design criteria for both velocity barriers at Dams A and B, and for a picket barrier at the fishway entrance are presented in Table 2-14 below.

Table 2-14. Fish Barrier Hydraulic Criteria

Criteria	Units	Value	Comments
Fishway Entrance (Trapping Only)			
Fish Barrier Type	-	-	Picket Barrier
Adult Fish Passage High Flow	ft^3/s	71.9	1% Exceedance during months of October - December
Adult Fish Passage Low Flow	ft^3/s	23.4	95% Exceedance during months of October - December
Juvenile Fish Passage High Flow	ft^3/s	76.9	1% Exceedance during March (max release month)
Juvenile Fish Passage Low Flow	ft^3/s	23.4	95% Exceedance during May (min release month)
Maximum Picket Clear Spacing	in	1.0	NMFS, 2011; Section 5.3.2.1
Maximum Average Velocity Through Barrier	ft/s	1.0	NMFS, 2011; Section 5.3.2.2; Discharge evenly distributed over gross wetted area
Maximum Head Differential (over clean picket condition)	ft	0.3	NMFS, 2011; Section 5.3.2.3
Minimum Picket Freeboard on Fish Passage High Flow	ft	2.0	NMFS, 2011; Section 5.3.2.6
Minimum Submerged Depth at Fish Passage Low Flow	ft	2.0	NMFS, 2011; Section 5.3.2.7; often relaxed in smaller drainages such as this
Minimum Picket Porosity	%	40	NMFS, 2011; Section 5.3.2.8
Sill/Apron Construction	-	See Comment	Picket barrier sill shall consist of a concrete sill with cutoff walls
Dams A & B (Year-Round)			
Fish Barrier Type	-	-	Velocity Barrier
Dam A High Flow	ft^3/s	50.0	Maximum powerhouse discharge
Dam A Low Flow	ft^3/s	15.0	Minimum flow requirement downstream of Dam A
Dam B Juvenile High Flow	ft^3/s	62.1	1% Exceedance during March (max release month); adjusted to Dam B reach
Dam B Fish Passage High Flow	ft^3/s	56.9	1% Exceedance during months of October – December; adjusted to Dam B reach
Dam B Fish Passage Low Flow	ft^3/s	8.4	95% Exceedance during months of October – December; adjusted to Dam B reach
Minimum Weir Height	ft	3.5	NMFS, 2011; Section 5.4.2.1
Minimum Apron Length	ft	16	NMFS, 2011; Section 5.4.2.2
Minimum Apron Slope	ft/ft	1 / 16	NMFS, 2011; Section 5.4.2.3
Maximum Weir Head	ft	2.0	NMFS, 2011; Section 5.4.2.4

Criteria	Units	Value	Comments
Downstream Apron Elevation	-	-	Above fish passage high flow tailwater

2.7 Geotechnical

To support final engineering efforts, the following geotechnical criteria will be required:

- Soil Bearing Pressure
- Water Table Height
- Active/Passive Lateral Earth Pressure
- Passive Soil Pressure (Lateral)
- Soil Weight
- Soil Friction Factor
- Site Class as Defined by ASCE 7-16 Table 3.13
- Frost Depth
- Minimum Footing Bearing Depth
- Minimum Footing Width
- Anticipated Total Settlement
- Anticipated Differential Settlement

CDM Smith and AECOM Technical Services, Inc. prepared a Geotechnical Data Report for KRRC in June 2019. Two borings, B-13 and B-14, were drilled near Fall Creek Bridge by Gregg Drilling between September 25 and October 18, 2019, with a truck-mounted Mobile B-53 drill rig. The borings reached depths of 21 feet (B-13) and 29 feet (B-14) below ground surface.

The Project site is mapped as Quaternary (Qv) and Tertiary (Tv) volcanic rock with nearby landslide deposits (Qls) associated with steep slopes on the east side of Fall Creek and just south of the Project site. Cobble- and boulder-sized rocks were observed on the ground surface at the proposed hatchery site and will likely need to be cleared to support construction. The borings advanced in the Project vicinity indicate approximately 18 inches of fill (road base) overlying slightly to completely weathered basalt. Based on the presence of sand, clay, and root structures at depth, we interpreted the deposit to be colluvium consisting of cobbles and boulders within a clay/sand matrix. Colluvium was interpreted to extend to the depths explored in boring B-13 and to a depth of 13 feet in boring B-14. Highly weathered andesite was observed below the colluvium in boring B-14 and extended to the depth explored (29 feet).

2.8 Architectural

2.8.1 Applicable Codes and Standards

The following references will serve as the basis for preparation of the architectural design elements specific to the currently adopted codes of the County of Siskiyou, California:

- 2019 California Building Code, Title 24, Volumes 1 & 2, Part 2
- 2019 California Energy Code, Title 24, Part 6
- 2019 California Fire Code, Title 24, Part 9

2.8.2 Building Summary

With respect to the Coho Building, the Chinook Incubation Building and the Spawning Building, all will be constructed utilizing a pre-engineered metal building system. A specific pre-engineered metal building manufacturer has not been identified and will be open to a competitive bid process. Documents submitted illustrate a typical basis-of-design and final building packages will be subject to the proprietary components and detailing of the awarded vendor. The vendor will be required to provide the necessary shop drawings and engineering to validate compliance with the design intent. Structural design requirements are further discussed in Section 2.9 of this document.

2.8.3 Energy Code Compliance

All the above-mentioned building envelopes are considered "Processed Spaces" based on the nature of their use. As such, Title 24, Part 6 exempts these building envelopes from meeting energy compliance requirements that would normally apply to a pre-engineered metal building. Exemption is based on the following conditions:

- Process space is a nonresidential space that is designed to be thermostatically controlled to maintain a process environment temperature less than 55F or to maintain a process environment temperature greater than 90F for the whole space that the system serves, or that is a space with space-conditioning system designed and controlled to be incapable of operating at temperatures above 55F or incapable of operating at temperatures below 90F at design conditions.

While all buildings meet the exemption requirements, they will be clad with insulated metal wall and roof panels that meet the prescriptive energy compliance mandates per Title 24, Part 6. In addition, all buildings will be daylit by means of Solatube daylighting devices. These devices will also meet the minimum energy code requirements.

2.9 Structural

The design criteria apply to all design procedures to be implemented during the Project design phase. Structural design considerations listed in this section—including detailing of structural components, material selection, and design requirements—are intended to be incorporated into Project design. The structural facilities consists of 11 main systems: (1) the intake structure, (2) the dam A velocity barrier, (3) the dam B velocity barrier, (4) the coho building, (5) the chinook raceways, (6) the chinook incubation building, (7) the spawning building, (8) the adult holding ponds, (9) the meter vault, (10) the fish ladder, (11) the temporary picket barrier, and (12) the fish release pipe support.

2.9.1 Applicable Codes and Standards

The following codes, standards, and specifications will serve as the general design criteria for the structural design of the facilities. The applicable version of each document is the latest edition in force unless noted otherwise. References to the specific codes and standards will be included in the applicable technical specifications as the final design documents are prepared.

The structural design, engineering, materials, equipment, and construction will conform to the codes and standards listed in Table 2-15.

Table 2-15. Structural Codes and Standards

Code	Standard
2018 IBC	2018 International Building Code
2019 CBC	2019 California Building Code
SEI/ASCE 7-16	Minimum Design Loads for Buildings and Other Structures, 2016 Edition
ANSI/AISC 360-16	Specification for Structural Steel Buildings, 2016 Edition
AISC 341-16	Seismic Provisions for Structural Steel Buildings, 2016 Edition
ACI 318-14	Building Code Requirements for Structural Concrete
ACI 350-06	Code requirements for Environmental Engineering Concrete Structures
ACI 350.4R-04	Design Considerations for Environmental Engineering Concrete Structures
ADM1-2015	Aluminum Design Manual, 2015 Edition
AWS D1.1-2020	Structural Welding Code – Steel, 2020 Edition
AWS D1.2-16	Structural Welding Code – Aluminum, 2016 Edition
AWS D1.6	Structural Welding Code – Stainless Steel

The following references are used in development of the structural design elements of the Project:

- American Institute of Steel Construction (AISC) (2017). "Steel Construction Manual," Fifteenth Edition.
- County of Siskiyou Building Code – Design Information, https://www.co.siskiyou.ca.us/building/page/design-information.

2.9.2 Materials

The material properties assumed for preparation of the design and engineering are listed in Table 2-16.

Table 2-16. Structural Material Properties

Structural Stainless Steel	
Bars and Shapes	ASTM A240, Type S31600
Plates	ASTM A240, Type S31600
Hollow Sections	ASTM A312, Type S31600
Structural bolts	ASTM F593 Type 316
Nuts and washers	ASTM F593 Type 316
Anchor bolts	ASTM F593 Type 316
Structural Mild Steel	
Wide Flanges	ASTM A992, Gr. 50

Other Shapes, Plates, Angles, and Bars	ASTM A36
Pipe	ASTM A53, Gr. B
Hollow Structural Sections (HSS)	ASTM A500. Gr. B
Structural Weathering Steel	
Wide Flanges	ASTM A588, Gr. 50
Rectangular and Square HSS	ASTM A847, Gr. 50
Other Shapes, Plates, and Bars	ASTM A588, Gr. 50
Miscellaneous	
Grating	Fiberglass reinforced plastic (FRP)
Stair Treads	Fiberglass reinforced plastic (FRP)
Handrails	Fiberglass reinforced plastic (FRP)
Ladders	Fiberglass reinforced plastic (FRP)
Aluminum alloy shapes	6061-T6
Aluminum alloy plates	5052-H32
Concrete	
Concrete	4,500 psi normal weight
Rebar	ASTM A615, Grade 60

2.9.3 Design Loads

The general loads considered in the design of the facilities are summarized in this section. All loads will be combined per the requirements of ASCE 7 for the various loading conditions to assess factors of safety. The actual design loads for each structure are included on the structural drawings.

2.9.3.1 Dead Load

The structural system for all Project elements will be designed and constructed to support all dead loads, permanent or temporary, including but not limited to self-weight, pipe systems, fixed mechanical and electrical equipment, stairs, walkways, and railings.

2.9.3.2 Live Load

Live loads during construction and operation consist of workers on the structures, temporary stored materials or equipment on the Project elements, impact, and construction equipment and vehicles. Live loads on the access stairways will be superimposed as per the CBC codes.

2.9.3.3 External Hydrostatic Loads

A triangular distribution of static water pressure is assumed to act normal to the upstream faces of all screen panels, stop logs, and gate structures.

2.9.3.4 Buoyancy Loads

Structures will be designed to resist upward hydrostatic pressures from high groundwater or river levels. Design factors of safety follow ACI 350.4R Section 3.1 guidelines recommending a factor of safety of 1.1 for groundwater to the top of wall, not considering soil, and 1.25 considering soil and groundwater elevations below the top of wall.

2.9.3.5 Earthquake Loads

Earthquake loads have been selected based on the IBC related maps and tables. S_s=0.584g, S_1=0.304g. The buildings will be designed for Risk Category II with an importance factor of 1.0 and assuming Site Class D or worse. Using Site Class D: S_{DS}=0.519g, C_V=1.089. The Seismic Design Category classification for the Project is D.

2.9.3.6 Earth Loads

Below-grade structures and water-holding basins will be designed for worst-case load combinations of full height of backfill plus a minimum 2-foot soil surcharge. Additional surcharge loads will be applied to account for unique conditions due to adjacent structure proximity and traffic or equipment loading.

2.9.3.7 Snow Loads

The structures will be designed to carry the applicable snow load. The flat roof snow load at this site is 40 pounds per square foot (psf) in accordance with the County of Siskiyou Building Code. Design snow loads include effects from drift surcharge loads and unbalance snow load requirements. Grating area will be treated as impervious surface with no reductions applied for the open area of the grating surface.

2.9.3.8 Wind Loads

Wind loads will be applied in the design of the buildings and elevated structures. For structures, wind loads will be computed per the IBC using an ultimate design wind speed of 115 miles per hour and a minimum design wind pressure of 20 psf, exposure category C, Risk Category II, and an importance factor of 1.0. Wind loads will be compared to the earthquake forces and the controlling load will be used.

2.9.3.9 Temperature Loads

Temperature changes for expansion and contraction will be considered based on the site location.

2.9.4 Frost Depth

The design minimum frost depth is 12 inches in accordance with the County of Siskiyou Building Code.

2.10 Mechanical

2.10.1 Applicable Codes and Standards

The following references will serve as the basis for preparation of the mechanical design elements:

- American Society of Testing and Material (ASTM)

- American National Standards Institute (ANSI)
- American Society of Mechanical Engineers (ASME)
- American Welding Society (AWS)
- American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE)
- National Fire Protection Association (NFPA)

2.10.2 Materials

The material properties assumed for preparation of the preliminary design are listed in Table 2-17. Yellow metals and galvanized systems that would come in contact with fish production water supply will not be allowed.

Table 2-17. Mechanical Materials

Component	Materials
Gates	Cast iron, Aluminum, Stainless Steel
Buried Piping	PVC, Ductile Iron
Exposed Piping	PVC, Carbon Steel, Ductile Iron
Valves	PVC, Ductile Iron
Hardware	Stainless, PVC
Ductwork	Galvanized Sheet Metal, Aluminum for high humidity areas
Transport Flumes	Aluminum
Fish Transport Pipes	HDPE
Intake Fish Screens	Stainless steel, Mild Steel
Incubation Trays	Fiberglass, Plastic
Feeding Vessels	Fiberglass

2.10.3 Design Loads

The mechanical loads are listed in Table 2-18.

Table 2-18. Mechanical Loads

Load	Description
Pump Loads	Net Positive Suction Head Required and Net Positive Suction Head Available will be determined to size all pumps to prevent cavitation.
Piping Loads	Piping and fittings will be designed to the working pressure of the fluid and the pipe wall thickness will be designed for a sufficient bursting pressure.
Gate Loads	Load calculations for deflection for gates at the maximum expected head.

Load	Description
Valve Loads	Valves will be designed for expected maximum pressure and expected maximum differential pressure.
Debris Screens	Debris screens will be designed for a maximum differential pressure of 3-ft of water across the upstream and downstream faces.
Building Cooling	Cooling will not be provided; air circulation will be provided by large high-volume wall mount fans to allow airflow across the building space. The ventilation system will be designed based on a maximum summer ambient temperature of 97°F.
Building Heating	The heating system will be designed to maintain building space temperature above freezing (40°F). Heating system will be designed based on a minimum winter ambient temperature of 15.9°F.

2.10.4 HVAC

Heating and ventilation will be provided to the Coho Rearing Building, Chinook Incubation Building, and the Spawning Building. Heating in all buildings will be provided by wall- or ceiling-mounted electric unit heaters. Cooling will not be provided.

2.10.5 Plumbing

No sanitary waste collection system or potable water distribution system is included in the project. An outdoor vault toilet with a sealed inground tank will be provided on site.

2.10.6 Fire Protection

Automatic fire sprinklers are not required. A fire extinguisher will be provided according to applicable building codes and NEPA standards at all buildings.

2.11 Electrical

The electrical design criteria apply to all design procedures to be implemented during the Project design phase. Electrical design considerations listed in this section, including detailing of electrical components, material selection, and design requirements, are intended to be incorporated into Project design.

2.11.1 Applicable Codes and Standards

The following references and design standards will serve as the general design criteria for the electrical design of the Project. The applicable version of each document is the latest edition enforced, unless noted otherwise. References to the specific codes and standards are included in the applicable technical specifications. The electrical design, materials, equipment, and construction will conform to the codes and standards listed in Table 2-19.

Table 2-19. Electrical Codes and Standards

Code	Standard
ANSI	American National Standards Association

Code	Standard
CARB	California Air Resources Board
CCOR Title 24	California Code of Regulations
CPUC GO 128	California Public Utilities Commission – General Order No. 128: Construction of Underground Electric Supply and Communication Systems
IEEE	Institute of Electrical and Electronics Engineers
IESNA	Illuminating Engineering Society of North America – Lighting Application Handbook
ISA	Instrument Society of America
NEMA	National Electrical Manufacturers Association
NETA ATS	International Electrical Testing Association Acceptance Testing Specifications
NFPA 70	National Electrical Code (NEC)
NFPA 70E	Standard for Electrical Safety in the Workplace
NFPA 101	Life Safety Code
NFPA 110	Standard for Emergency and Standby Power Systems
OSHA	Occupational Safety and Health Act
UL	Underwriters Laboratory

2.11.2 Materials

The materials assumed for preparation of the preliminary design and applicable for engineering of the Project are listed in Table 2-20.

Table 2-20. Electrical Materials

Material	Standard
Panelboards	NEMA PB 1, UL 67
Transformers, Dry Type	NEMA ST 1, UL 1561, 10 CFR – Part 431 DOE 2016
Circuit Breakers	NEMA AB 1, UL 489
Switches	NEMA KS 1, UL 98
PLCs	NEMA ICS 1, UL 508
Terminal Blocks	UL 1059
Instrumentation Cable: THWN Copper	ASTM B8, NEMA WC 57, UL 13, UL 83, UL 1277
Power Conductors/Cable: THWN Copper; XHHW-2 Copper	ASTM B3, ASTM B8, ASTM B496, NEMA WC 70, UL 83

Material	Standard
Splices, Connectors, and Terminations	UL 486A-486B, UL 486C, UL 510
Grounding: Copper	UL 467
Boxes and Enclosures: NEMA 1, 12, 3R, & 4	NEMA 250, UL 514A
Raceway: Rigid Galvanized Steel; Intermediate Metal Conduit; PVC Schedule 80; Liquid-tight Flexible Metal Conduit	NEMA C80.1, NEMA C80.6, NEMA RN 1, UL 6, UL 360, UL 514B, UL 651, UL 1242
Transfer Switches	NEMA ICS 1, NEMA ICS 2, UL 1008
Motors: TEFC or submersible	IEEE 112, NEMA MG 1, UL 2111
Motor Controls	NEMA ICS 2
Wiring Devices	NEMA WD 1, NEMA WD 6
Luminaires: LED	IESNA HB-9, IESNA LM-80, IEEE C62.41.1, UL 1598, UL 2108, UL 8750, U.S. DOE Energy Star
Surge Protective Devices	UL 1449

2.11.3 Design Loads

All currently anticipated electrical loads are summarized in Table 2-21.

Table 2-21. Electrical Loads

Load	Description
Booster Pump Skids	480V, 3-phase, 3 hp, 3 ea.
Intake Traveling Screens	480V, 3-phase, 1 hp, 2 ea.
Intake Screen Spray Pumps	480V, 3-phase, 2 hp, 2 ea.
Existing Conveyor Belt	208V, single-phase, 1.5 hp
Existing Fish Crowder and Lift	480V, 3-phase, 1.5 hp (crowder drive) 1.5 hp (crowder lift)
Existing Electro-Anesthesia Tank	120V, single-phase, 1.92 kVA
Existing Electro-Anesthesia Tank Hydraulic Hoist	120V, single-phase, 2 hp
Electro-Anesthesia Tank Fill Pump	480V, 3-phase, 2 hp
Existing UV Lamp Ballast Outlet	120V, single-phase, 75 VA

Load	Description
Waste Drain Wet Well Sump Pumps	480V, 3-phase, 2 hp, 2 ea.
Coho Building Unit Heaters	480V, 3-phase, 5 kW, 5 ea.
Chinook Incubation Building Unit Heaters	480V, 3-phase, 5 kW, 5 ea.
Spawning Building Unit Heater	480V, 3-phase, 10 kW
Coho Building Radiant Heaters	208V, 3-phase, 3 kW, 2 ea.
Chinook Incubation Building Radiant Heaters	208V, 3-phase, 3 kW, 2 ea.
Spawning Building Radiant Heaters	480V, 3-phase, 4.5 kW, 1 ea.; 208V, 3-phase, 3 kW, 2 ea.
Electrical Room Split AC Unit	208V, single-phase, 2.08 kVA
Building Exhaust Fans	120V, single-phase, 3/4 hp, 2 ea., 1/2 hp, 3 ea., 1/6 hp, 2 ea., 1/20 hp, 1 ea.
Meter Vault Exhaust Fan	120V, single-phase, 170 VA
Motorized Dampers	120V, single-phase, 100 VA, 8 ea.
Meter Vault Sump Pump	120V, single-phase, 1/2 hp
Tagging Trailer Receptacles, 100A	240V, single-phase, 19.2 kVA, 2 ea.
Tagging Trailer – Fish Pump Receptacles, 60A	240V, single-phase, 11.5 kVA, 2 ea.
Waste Drain Pump Receptacles, 20A	208V, single-phase, 3.33 kVA, 7 ea.
Lighting, LED	120V, single-phase, 4.27 kVA
Sky Light Dimmers	120V, single-phase
Convenience Receptacles	120V, single-phase, 180 VA, 47 ea.
Standby Generator Loads	208V, single-phase, 2.50 kVA (block heater); 120V, single-phase, 400 VA (battery heater), 100 VA (battery charger)
SCADA Panel	120V, single-phase, 400 VA
Autodialer	120V, single-phase, 36 VA
Instrumentation	120V, single-phase or 24 Vdc, 4-20 mA
Intrusion Detection	120V, single-phase

2.12 Instrumentation and Controls

2.12.1 Applicable Codes and Standards

The following references and design standards will serve as the general design criteria for the instrumentation and control design of the Project. The applicable version of each document is the latest edition enforced, unless noted otherwise. References to the specific codes and standards are included in the applicable technical specifications. The instrumentation and control design, materials, equipment, and construction will conform to the codes and standards listed in Table 2-22.

Table 2-22. Instrumentation and Control Codes and Standards

Code	Standard
IEEE	Institute of Electrical and Electronics Engineers
ISA 5.1	Instrumentation Symbols and Identification
NEMA	National Electrical Manufacturers Association
NFPA 70	National Electrical Code (NEC)
UL	Underwriters Laboratory

3.0 Project Description

3.1 General Description

The general site layout is depicted in Figure 3-2, with the major components of the layout summarized in Table 3-1, as well as in the following sections.

3.2 Intake Structure and Meter Vault

A hatchery intake structure will be located along the southeast bank of Fall Creek directly adjacent to Dam A and opposite the City of Yreka intake structure (see Figure 3-1). The intake will be constructed of concrete and will divert flows up to 10 cfs from Fall Creek. A buried 24-inch-diameter pipe will supply the site and will divide flows into four buried water supply pipes to deliver flow to the various hatchery facilities. A debris screening system will be added at the entrance to the new intake structure to prevent large sediment, detritus, and other debris from entering the intake chamber. The debris screening system will be equipped with an automated screen-cleaning system that will operate at regular intervals or based on an acceptable head differential across the screen. Behind each screen will be stop log guide slots for isolation of the pipeline, or closure of one of the screen slots for general maintenance.

Inside the intake structure, the 24-inch-diameter supply line will be set in the concrete wall at a sufficient depth to preclude significant air entrainment at the pipe entrance. After the flow split, the four hatchery facility supply pipelines will be equipped with magnetic flow meters and isolation valves located in a concrete vault that will transmit flow rates to a programmable logic controller (PLC) located in the electrical room connected to the Chinook Incubation Building (see below). The intake will also be equipped with a sediment sluiceway outside of the intake chamber, for bypassing sediment and bedload that may accumulate at the toe of the intake screens.

Figure 3-1. Intake Structure Location and City of Yreka Intake (Source: McMillen Jacobs)

Table 3-1. Major Facilities Schedule

Facility	Species	Required Capacity / Volume	Rearing Volume Provided	Flow Requirement	Total Dimensions (Rearing Dimensions)	Comments
Intake Structure	-	-	-	10 ft³/s	8' (W) x 8.9' (L) x 8.5' (H)	Concrete Structure
Meter Vault	-	-	-	-	13' (W) x 15' (L) x 6.4' (H)	Concrete In-Ground Vault
Coho Building	Coho	-	-	-	53' (W) x 65' (L)	Pre-engineered Metal Building
Incubators	Coho	48 trays	48 trays	40 gpm	25" (W) x 25" (L) x 34.5" (H) (per stack)	Existing, from IGFH
Incubation Working Vessel	Coho	150 ft³	150 ft³	30 gpm	(2) 2' (W) x 15' (L) x 3' (H)	Existing, from IGFH
First-Feeding Vessel	Coho	750 ft³	825 ft³	150 gpm	(2) 4' (W) x 16' (L) x 3' (H), Existing (3' W x 15' L x 2.5' Depth) Existing	Existing, from IGFH
					(2) 6' (W) x 21' (L) x 4' (H), New (5' W x 20' L x 3' Depth) New	Fiberglass Vat
Rearing Ponds	Coho	3,850 ft³	5,400 ft³	764 gpm	(2) 11' (W) x 40' (L) x 3.8' (H), Existing (11' W x ~38' L x 3' Depth) Existing	Existing Concrete Raceway
					(2) 12.0' (W) x 34.8' (L) x 5' (H), New (12.0' W x 30' L x 4' Depth) New	Concrete Raceway
Chinook Incubation Building	Chinook	-	-	-	50' (W) x 60' (L)	Pre-engineered Metal Building
Incubators	Chinook	1,088 trays	1,088 trays	680 gpm	25" (W) x 25" (L) x 34.5" (H) (per stack)	Existing, from IGFH
Incubation Working Vessel	Chinook	290 ft³	290 ft³	60 gpm	(4) 2.5' (W) x 14.5' (L) x 2.5' (H)	Existing, from IGFH
Chinook Rearing Ponds	Chinook	20,200 ft³	23,040 ft³	4,040 gpm	(8) 12' (W) x 64.8' (L) x 5' (H) (12' x 60' L x 4' Depth)	Concrete Raceway
Trapping/Sorting Pond	Coho/Chinook	3,350 ft³	3,350 ft³	200 gpm	12.6' (W) x 66.3' (L) x 5' (H)	Concrete Raceway (1495 gpm provided)
Chinook Adult Holding Pond	Chinook	1,800 ft³	3,350 ft³	400 gpm	12.6' (W) x 66.3' (L) x 5' (H)	Concrete Raceway (1495 gpm provided)
Coho Adult Holding Pond	Coho	600 ft³	3,350 ft³	200 gpm	12.6' (W) x 66.3' (L) x 5' (H)	Concrete Raceway 1495 gpm provided
Spawning Building	Coho/Chinook	-	-	-	25' (W) x 35' (L)	Pre-engineered Metal Building
Settling Pond	-	3,200 ft³	3,200 ft³	-	(2) 12.6' (W) x 31.8' (L) x 5' (H)	Concrete Pond (2 Bays)
Fish Ladder	Coho/Chinook	-	-	10 ft³/s	2.5' (W) x 24.6' (L)	Denil Type (Concrete)
Fish Barrier (Dam A)	Coho/Chinook	-	-	-	29' (W) x 16' (L)	Velocity Apron (Concrete)
Fish Barrier (Dam B)	Coho/Chinook	-	-	-	11.5' (W) x 20' (L)	Velocity Apron (Concrete)
Fish Barrier (Fishway)	Coho/Chinook	-	-	-	17.3' (W) x 8' (L) x 4.5' (H)	Picket Panels on Concrete Sill

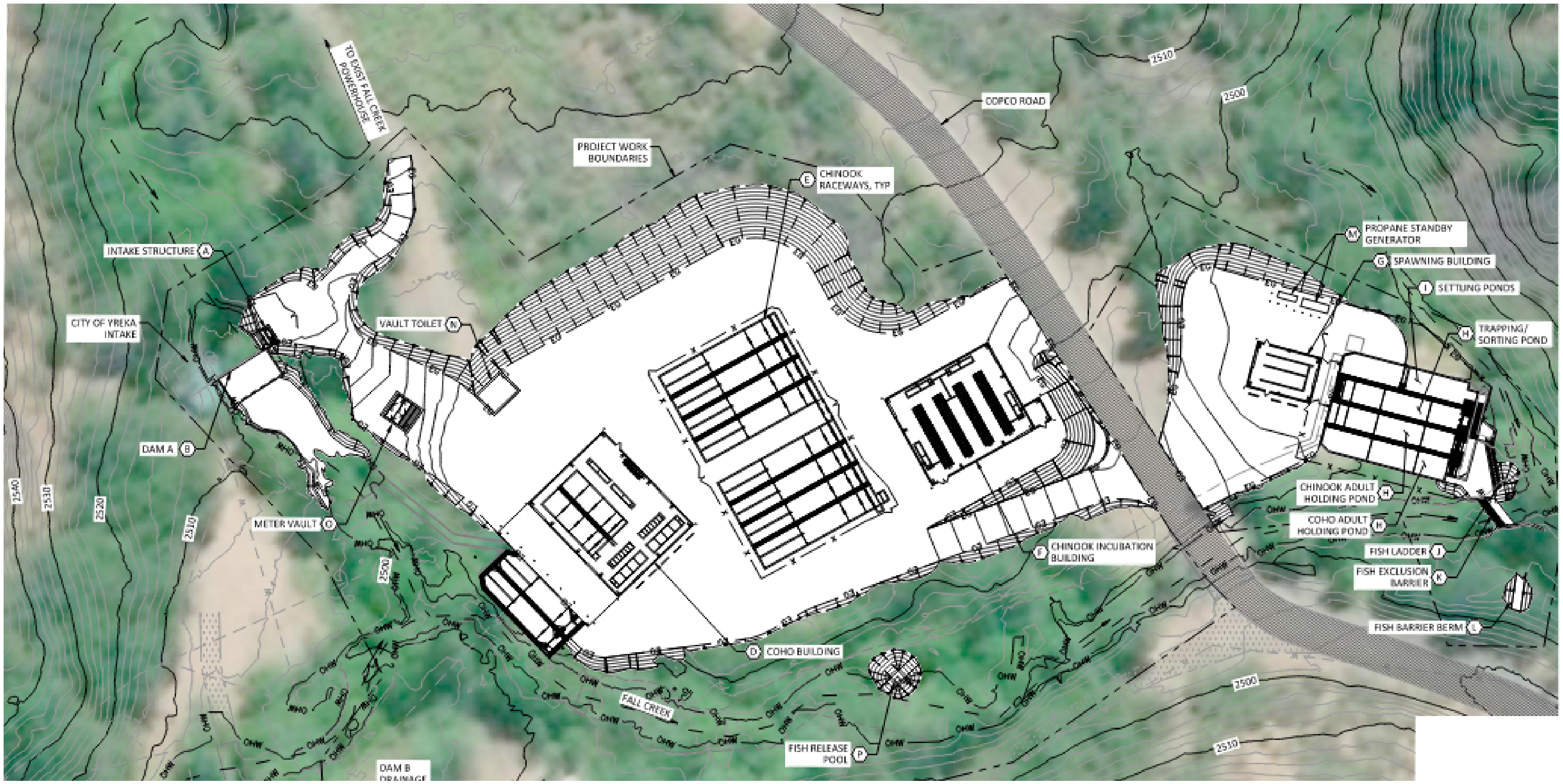

Figure 3-2. General Site Layout

3.3 Coho Building

The Coho Building will be located at the north end of the Project site at pad elevation 2503.0 (North American Vertical Datum [NAVD] 88), and will house all Coho incubation, grow-out, and rearing infrastructure Coho production facilities. The Coho Building will be a pre-engineered metal building with interior dimensions of 53 feet wide by 66.5 feet long.

Existing incubation stacks and trays will be reused from IGFH (see Figure 3-3), and will be configured in a row of six half-stacks (i.e., eight trays per stack) along the southwest wall. This will accommodate the 120,000 Coho green eggs discussed in the bioprogram at 2,500 eggs per tray. A water flow rate of 5 gpm will be provided to each of the incubation stacks via a head tank located above the stacks. The intent of a head tank design is to protect against any potential flow interruption. Water will flow downward through the stacks to a floor drain that discharges to a production drain system, with flows diverted to one of two systems (adult ponds as online flow; effluent ponds as effluent flow). The incubation stacks will be supplemented with two working vessels (egg picking, enumeration) that will be reused from IGFH (see Figure 3-3).

Figure 3-3. Existing IGFH Incubators (Left) and Working Vessels (Right) (Source: McMillen Jacobs)

Four first-feeding vessels will be provided for initial ponding of the Coho fry consisting of two existing vats from IGFH and two new fiberglass aquaculture vats, providing a total of 825 ft^3 of ponding volume. First-feeding vessels will be equipped with screen guides, such that a quiescent zone can be maintained at the downstream end of the vessel. These vessels will operate in a flow-through condition with a 150-gpm (total) renewal rate, and online overflows will pass through a standpipe in the quiescent zone that flows into the drain system and then routed to the adult holding ponds; effluent will be conveyed to the effluent pond (or holding tanks if designed) via an effluent standpipe adjacent to the vats in the floor, which will discharge to the effluent drain system.

Grow-out and rearing space will be provided in part in the existing upper raceway bank (see Figure 3-4). There are two existing concrete raceways (approximately 11 feet wide by 40 feet long by 3.8 feet deep) adjacent to Fall Creek that will be just outside of the Coho building. These will be rehabilitated with a surficial mortar layer and resurfaced with an epoxy liner for use in Coho grow-out and rearing. This raceway bank will be covered with a roof above and predator netting and fencing provided along the sides of the site. The existing flume that feeds these raceways will be demolished and replaced with pipe manifolds that provide a maximum of 210 gpm to each of the existing raceways. The raceways will be further subdivided by two 20-foot-long pony walls, equipped with dam boards and fish screen slots. This will provide approximately 1,300 ft^3 of early rearing volume for use prior to fish tagging/marking. After fish have been tagged/marked, the dam boards and fish screens can be removed, allowing the full 2,500 ft^3 of rearing space to be used.

Figure 3-4. Existing Upper Raceway Bank (Source: McMillen Jacobs)

At the downstream end of the existing raceways, dam boards and fish screens will be installed upstream of the outlet works. Additionally, a set of dam boards will be installed in the existing concrete outlet flume, and pond overflow will be directed into a production drainpipe that will convey flow to the adult holding ponds. When fish are to be released from these raceways, a gate will be closed on the production drainpipe, and dam boards will be lowered in the existing concrete flume to allow fish to pass over the dam boards and directly into Fall Creek.

Further rearing space will be provided by two additional constructed concrete raceways 12 feet wide by about 35 feet long by 5 feet deep, located approximately 20 feet from the existing raceways inside the Coho Building. A roadway will pass under the roof structure between the existing and the new. At tagging and marking, the trailer will pull between the existing and new raceways and the roll-up doors on the Coho Building will be opened. Newly tagged/marked fish can then be distributed among the four raceways as required by rearing volume.

Overflow from the new concrete raceways will discharge to an approximately 2-foot-wide exit channel that will direct flows to a production drainpipe in the concrete wall. In addition, there will be in the exit channel a 2-foot by 2-foot box behind a set of dam boards leading to the volitional fish release pipe. If it is desired that fish be volitionally released from these ponds, the gate on the production drainpipe can be closed and dam boards can be removed at the volitional fish release box. Fish will volitionally go over the dam boards and enter a 10-inch-diameter fish release pipe that will convey fish to the existing concrete flume on the discharge end of the existing Coho rearing raceways, and ultimately out to Fall Creek.

Finally, because production periods will overlap and all Coho infrastructure, with the exception of the existing upper raceways, will be housed in the same building, biosecurity will be maintained by curtain systems between the respective areas of the Coho Building (e.g., incubation, first-feeding, rearing/grow-out).

3.4 Chinook Incubation Building

The Chinook Incubation Building will be located immediately north of Copco Road at pad elevation 2,503.0 (NAVD 88) and will house only the Chinook egg incubation operations. The Chinook Incubation Building will be a pre-engineered metal building with interior dimensions of 50 feet wide by 60 feet long.

Existing incubation stacks and trays will be reused from IGFH and will be configured in eight rows of 17 half-stacks, for a total of 136 stacks or 1,088 trays. Incubation trays will accommodate the 4.5 million Chinook green eggs discussed in the bioprogram at an approximate loading density of 4,150 eggs per tray. Rows of incubation stacks will maintain a 7.5-foot buffer on other rows to mitigate any cross-contamination from splashing. A flow of 5 gpm will be routed to each of the incubation half-stacks via head tank above, as in the Coho Building, and water will flow to the drain system in the floor.

Four incubation working vessels will be reused from IGFH and will be positioned around the inside perimeter of the building for hatchery operations.

3.5 Chinook Raceways

Eight concrete raceways will be constructed in two raceway banks north of the Chinook Incubation Building at pad elevation 2,503.0 (NAVD 88), with the pond invert set 3 feet below the pad elevation (2,500.0 NAVD 88). Raceways will be constructed with 26-foot-long pony walls and fish screen guide slots and stop log slots at intervals along the length of the structure, such that ponding volumes can be incremented based on fish development. The eight raceways provide a total rearing volume of 23,040 ft^3. Bioprogram requirements for tagging and marking assume Chinook will be marked at 150 fpp with a required rearing volume of 16,045 ft^3. CDFW staff have indicated that Chinook sub-yearling cohort releases will begin immediately after marking has been completed. If required, the total rearing volume available (23,040 ft^3) provides adequate rearing flexibility for CDFW staff to rear fish up until approximately 104 fpp before approaching the recommended 0.3 density index maximum.

Chinook rearing raceways will be operated in a flow-through condition, with manifolds at the upstream end of the pond supplying a maximum of 500 gpm to each of the ponds, and dam board overflows draining to a sloped concrete exit channel that connects the two raceway banks. The concrete exit channel will be equipped with two open concrete boxes at the southwest end of the channel containing the production drainpipe and the volitional fish release pipe, respectively. During normal operations, dam boards will be in place to isolate the volitional fish release pipe, such that all water is directed to the production drainpipe and on to the adult holding ponds.

During volitional fish release, it is anticipated that the adult holding ponds may be used for raising fish on second-pass water, and therefore flow through the Chinook raceways will need to be divided between the production drain system and the volitional fish release pipe. At volitional fish release, fish screens in each of the raceways will be removed and a fish screen will be installed in front of the production drain box. Dam boards in front of both pipe boxes will be adjusted for the desired distribution between the two pipes, while maintaining a pool in the exit channel for fish that volitionally leave the raceways. Fish will be contained in the exit channel until they volitionally pass over the dam boards into the volitional fish release pipe. The volitional fish release pipe will convey fish entrained flows in an open channel condition to a constructed plunge pool adjacent to Fall Creek, approximately 150 feet upstream of the existing Copco Road bridge.

Predator netting and security fencing will be supplied to protect the Chinook rearing raceways. Predator netting will be connected to an exterior security fence with a metal frame structure that will allow personnel to stand and move around in the enclosure for access to the ponds. The security fence will generally be maintained 3 foot from edge of concrete, such that personnel will have access to all sides of the ponds from inside the fence enclosure. The security fence will be equipped with man gates and double-leaf gates between the raceway banks such that vehicles could access the 12-foot-wide center aisle between the raceway banks. At tagging/marking, it is anticipated that the tagging/marking trailer will pull along the north end of the ponds for access to electrical outlets and easy access through the double-leaf gates to the ponds.

3.6 Adult Holding Ponds

The existing lower concrete pond bank consists of four ponds approximately 12.5 feet wide by 70 feet long, with a concrete outlet structure at the downstream end (see Figure 3-5). Three of these ponds will be refurbished for use as adult holding ponds: one for trapping and sorting, one for Coho holding, and one for Chinook holding. Existing pond concrete walls are in poor structural condition and will require demolition and reconstruction. Reconstructed walls will be equipped with walkways between each of the ponds and sprinkler systems to mitigate fish jumping.

Based on estimates of holding 200 Chinook and 100 Coho at any given time and estimated adult weights (Chinook – 12 lbs, Coho – 8 lbs), NMFS guidance (2011) dictates a minimum of 1,800 ft^3 of pond volume for Chinook and 600 ft^3 of storage for Coho. Each individual pond is estimated to have approximately 3,350 ft^3 of storage, which provides ample capacity for adult holding. Because of the available capacity in the reconstituted ponds, these ponds may additionally be used for raising fish on second-pass water at the option of CDFW. Therefore, the ponds will be retrofitted with fish screen slots for partitioning, as needed operationally.

The adult holding ponds will be fed by a supply pipe from the intake structure, but will also be fed by the fish production drain system, such that at any given time (aside from nominal losses to cleaning) the adult ponds will be fed with the full water right of 10 cfs. In the Coho and Chinook holding ponds, during normal operations, the water supply will flow over a set of dam boards at the downstream end and through a floor diffuser into the fish ladder. The trapping-and-sorting pond will be equipped with an adjustable finger weir at the downstream end through which pond outflow will be routed. This will then serve as the trap at the end of the fish ladder. As fish go over the weir, they will remain in the trapping-and-sorting pond until they are transferred into their respective holding ponds. The trapping-and-sorting pond will be equipped with the existing Iron Gate Hatchery fish crowder and lift to aid in sorting and transfer of the respective species.

The adult holding ponds have been designed with fish screen keyways that will allow for culture and effluent collection for a limited number of Chinook juveniles during the periods when adult Coho and Chinook are not present. Acknowledging that the water source will be serial reuse from upper facility fish rearing systems (Coho and Chinook production raceways), the conservative density and flow indices used in the program should provide second-pass water of sufficient quality and oxygen levels to support serial reuse for a limited number of surplus juvenile Chinook. If juvenile fish are to be raised in these ponds, the Coho and Chinook holding pond outflow can be isolated from the fish ladder with a set of dam boards to full height. A fish release pipe with another set of dam boards in the exit channel provides the option of volitional release from these ponds. The fish release pipe will convey fish to the pool at the toe of the fish ladder. It should be noted, however, that the fish release pipe is not designed for concurrent use with the Denil fishway (i.e. during adult trapping), and the plunging flow from the release pipe could inhibit adults from accessing the fishway. It is not the design intent that juveniles would be released from these ponds while adult trapping is occurring. Finally, the adult holding ponds will be connected by dam boards that may be removed such that fish can be directed into any of the three ponds.

Figure 3-5. Existing Lower Raceway Bank Ponds (Source: McMillen Jacobs)

The lower raceway bank will be surrounded by an enclosure of perimeter fencing. Sufficient clearance to the perimeter fencing will be maintained around the ponds, such that personnel will be able to access the ponds and associated infrastructure. Security fencing will tie into the Spawning Building at the north end of the pond.

3.7 Spawning Building

Immediately north of the adult holding ponds at pad elevation 2491.5 (NAVD 88) will be the Spawning Building. The Spawning Building will be a pre-engineered metal building with interior dimensions of 25 feet wide by 35 feet long and will house equipment relocated from IGFH. A roll-up working door will be located on the southeastern wall of the building, providing direct access to the head of the sorting/trapping raceway. Within the sorting/trapping raceway, the existing fish crowder and lift will be provided to transfer fish from the raceway to the existing IGFH electro-anesthesia tank for fish sedation or euthanasia. A sorting table will be placed immediately outside of the roll-up door to sort and transfer sedated fish into the Spawning Building through removable troughs.

Within the Spawning Building, a holding table and air spawning table are provided for egg retrieval. The existing egg rinsing table and water hardening table will be relocated from IGFH for egg processing prior to incubation. A conveyor belt will be provided for transferring fish carcasses to a collection bin located outdoors. Additional return pipes are to be provided along the southeastern wall of the building for returning fish to either the trapping/sorting pond or the Chinook holding pond.

Excess space is provided within this structure for storage of hatchery supplies, as needed. Additional workspace is provided for any collaborator activities.

3.8 Settling Pond

The final pond in the existing lower concrete raceway bank (eastern-most pond) will be used as a settling pond to settle out any biosolids or other solid waste from cleaning of the upstream facilities discharged to a waste drain. The effluent treatment is discussed in greater detail in Section 10.4. This pond will be refurbished and parsed into a wet well and two distinct bays such that solids can be dried and removed as necessary over the life of the facility, while the waste drain system remains in operation.

When cleaning of the settling pond is required, a septic pump truck will access the pond from the adjacent pad, and the solids can be vacuumed out of the pond.

The downstream end of each of the settling pond bays will be equipped with an overflow structure that will divert flow-through water into the fish ladder (see below) for mixing with the adult holding pond flows and release to Fall Creek. At the interval required by the NPDES permit, measurements of the settling pond effluent flow rate will be performed by hand measurements using a staff gage at the overflow weir, and a calibrated discharge relationship to determine the flow rate from the facility. This will be added to measurements taken at the upstream end of the fish ladder to determine a total discharge rate.

3.9 Fish Ladder

The fishway is a baffled chute which is a type of roughened chute designed to meet the NMFS criteria. The baffled chute type is a Denil fishway. The Denil fishway is 2.5-foot-wide by approximately 25-foot-long. The entrance to the fishway will be located just downstream of the picket barrier at the upstream terminus to maximize fish passage efficiency. The fishway will ascend to the constructed concrete outlet structure at the lower raceway bank and will terminate at the finger weir at the downstream end of the trapping and sorting pond to convey fish into the pond for sorting. The fish ladder will consist of 14 standard baffles in total and will be of the Denil-type, as described in the NMFS (2011) guidelines (see Figure 3-6). At the top of the Denil ladder will be a pool for fish to turn into the constructed outlet structure. This turning/resting pool is sized to provide adequate energy dissipation characteristics and will be equipped with a dam board weir for fish to enter the constructed outlet structure. This pool, and the upstream overflow weir, will serve as the location for measuring flow rates from the ponds into the Denil fishway. At the interval required by the NPDES permit, hatchery personnel will perform hand measurements using a staff gage at the weir, and a calibrated discharge relationship to determine the flow rate into the Denil fishway. This will be added to the effluent flow rate determined at the settling ponds to determine an overall flow rate out of the hatchery.

The uppermost pool in the constructed outlet structure will be fed by the flow over the finger weir, and by flow from the Coho and Chinook holding ponds through a floor diffuser. The finger weir is sized according to recommendations from the U.S. Army Corps of Engineers *Fisheries Handbook* (Bell, 1991), and maintains approximately 4 inches above the fingers of the finger weir.

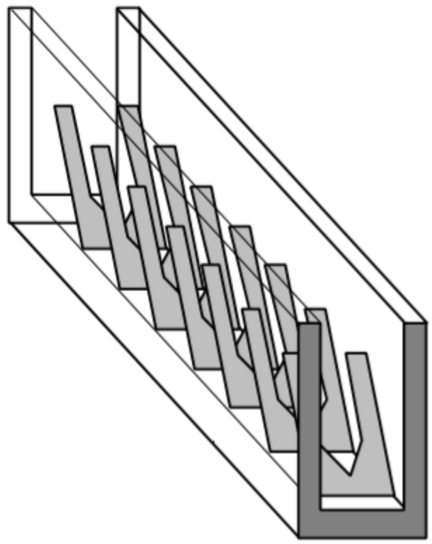

Figure 3-6. Perspective of Denil-Type Fish Ladder with Single-Plane Baffles (Source: NRCS, 2007)

During seasons when the fish ladder is not in operation, a bar screen will be installed at the downstream end of the ladder and baffles will be removed to exclude both adult and juvenile native fish populations from entrainment in the fish trap.

3.10 Fishway Picket Fish Barrier

A removable fish exclusion picket barrier will be constructed with the fish ladder that will guide fish to the fish ladder entrance pool and ultimately up to the trap. The fish barrier will consist of a set of aluminum pickets with 1-inch-maximum clear spacing that will be installed on a permanent concrete sill and removed each year at the beginning and end of the trapping season. The sill will have side walls and a 6-inch-tall curb across the bottom that the picket panels will be able to seal against, forming a continuous barrier across the stream. The sill and removable pickets will be oriented at an angle of approximately 30 degrees to the stream transect, such that an anadromous fish moving upstream will encounter the barrier and be directed toward the stream’s east bank, where the fish ladder entrance pool is situated. The typical fish ladder flow of 10 cfs will act as an attraction flow to the anadromous fish. NMFS (2011) recommendations for attraction flow in smaller streams are typically greater than 10 percent of the design high flow during the fish passage season. In this case, 10 cfs is approximately 20 percent of the design high flow and will provide effective attraction flow. The orientation of the picket barrier will also aid in reducing approach velocities at the barrier.

The picket framing will consist of ultra-high molecular weight (UHMW) stringer bars with penetrations for the aluminum pickets to slide in. UHMW stringer bars will be overlapped at installation to tie the individual picket panels together. These picket panels will rest at the bottom against the concrete sill, with a 6-inch-tall curb to prevent fish from passing underneath the panels. The picket panels will then be connected to a stand that will be secured to the concrete sill. A small walkway will be cantilevered from the framing/stringer bars above the high water level, such that access may be maintained to the whole length of the barrier without entering the stream (see Figure 3-7).

When debris or bedload accumulates on the pickets, the pickets will need to be manually cleaned to ensure that less than 0.3 feet of additional headloss from the clean picket condition is maintained (per NMFS, 2011). This can be performed with the use of a simple hand-rake or stiff-bristled broom, or by raising and lowering individual pickets through the stringer bars to allow the accumulated debris or bedload to be washed downstream. This will be performed from the small access way and will only need to be performed during the trapping season, as the pickets will be removed from the creek at all other times.

Figure 3-7. Temporary Picket Barrier for Adult Fish Trap (Source: McMillen Jacobs)

3.11 Dam A Velocity Barrier

Immediately downstream of existing Dam A, a 16-foot-long by 29-foot-wide sloped concrete apron will be constructed from the downstream face of Dam A. The apron will be sloped at 16H:1V (about 6.3 percent), resulting in high velocities and shallow flow depths. The combined high-velocity apron and the jump required to pass upstream of Dam A will effectively bar passage to both juvenile and adult anadromous fish for the anticipated creek flow range expected during juvenile fish release, adult migration, and up to larger flood events. This barrier follows design guidance from NMFS (2011).

3.12 Dam B Velocity Barrier

Immediately downstream of existing Dam B, a 16-foot-long by 11.5-foot-wide sloped concrete apron will serve as a similar velocity barrier to preclude fish from approaching the Dam B reservoir and exclude juvenile fish passage upstream. In order to prevent significant downstream modifications to the stream corridor inside of the ordinary high-water mark (OHWM), the Dam B concrete apron will primarily be constructed above grade. This will result in some demolition of the existing pier and sill for construction of the concrete apron. The existing sill where the stop logs are located will be raised approximately 1-foot 10-inches and new aluminum stop logs will be fabricated to fit the existing stop log slots.

Because of the limited height of Dam B, the stop logs will have insufficient height above the new concrete apron to meet the NMFS (2011) weir conditions for a standard velocity barrier. Therefore, the stop logs will be fitted with a newly fabricated nappe extension piece that will push the nappe overflow approximately 3.0 feet downstream of the aluminum stop logs, making for more difficult jump conditions for upstream migrating fish. This method has proven effective for similar conditions excluding anadromous salmonids in McMillen Jacobs Associates previous project experience.

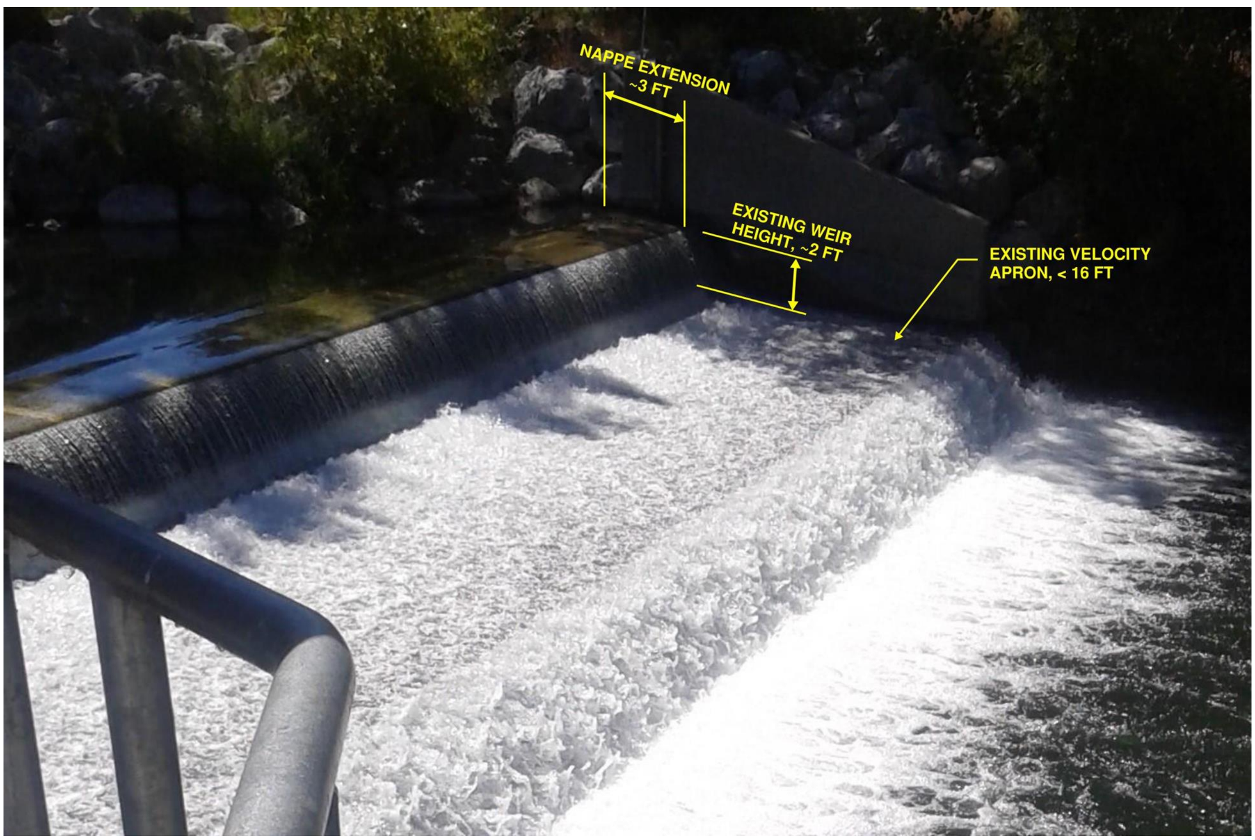

Figure 3-8. Nappe Extension Retrofit (Source; McMillen Jacobs)

In all other regards, the barrier follows design guidance from NMFS (2011). A sluicing gate and pipe will pass underneath the velocity barrier to allow flushing of accumulated sediment upstream of Dam B.

4.0 Hydraulic Design

The facility hydraulic design consists of four main piping systems:

1. Water supply piping system
2. Production drain system
3. Waste drain system
4. Volitional fish release pipes

The design also includes three fish passage/trapping elements:

1. Fish Ladder
2. Finger Weir
3. Fish Barriers

The design also includes the effluent treatment system. Hydraulic calculations for each of these elements can be found in Appendix A of this DDR, and each is discussed in detail below.

4.1 Supply Piping System

The supply piping system consists of four primary pipelines from the intake structure to the major production facilities, which include: (1) the Coho Building, (2) the Chinook rearing raceways, (3) the Chinook Incubation Building, and (4) the adult holding ponds. All pipes were assumed to be schedule 80 PVC, which are typical in hatchery applications, and present considerable cost savings over alternatives. The site is relatively constrained in terms of hydraulic head. The assumed water surface at the intake structure is at elevation 2,510.4 (NAVD 88), and the pad for the majority of the site is at elevation 2,503.0 (NAVD 88), providing only about 7.4 feet of hydraulic head across much of the site. For this reason, pipes were conservatively sized to minimize dynamic head losses through the piping system. At the same time, pipes were sized to maintain a minimum velocity of 1.5 feet per second (ft/s) and a typical velocity of approximately 2.0 ft/s such that they would be self-cleaning, and would not settle out any sediment, detritus, or other material in suspension that may pass the upstream traveling screen.

Modeling of the supply piping system using EPANET software (Appendix A) was performed for a series of 5 scenarios to ensure that water supply would be available under a number of contingency conditions. Scenarios that were modeled are described below:

1. **Scenario 0, Base Case:** The base case scenario evaluates the pipe flow under normal conditions, at the time in the bioprogram when demands on the supply lines are the greatest. Pipes were assumed to be in a clean, new condition (Hazen-Williams coefficient 140), and the minor loss coefficients as enumerated in Section 2.6.4 were applied.

2. **Scenario 1, Pipe Degradation:** Scenario 1 evaluates the condition where the pipes have degraded over time, either through accumulation of biomass or through a failure of the screen

leading to introduction of sediment, debris, or detritus to the pipeline. The friction loss coefficient was adjusted for this case while still being appropriate to plastic pipe (Hazen-Williams coefficient 120).

3. **Scenario 2, Operational Change & Pipe Degradation:** Scenario 2 evaluates the same degraded pipe condition as Scenario 1, but for an operational change that requires the maximum bioprogram flow to all design points (incubation stacks, working vessels, raceways, etc.) on each supply line simultaneously.

4. **Scenario 3, Intake Loss Contingency, Operational Change, & Pipe Degradation:** Scenario 3 builds on Scenario 2 by adding contingency losses at the intake structure, due to a traveling screen being taken out of operation, or excessive blockage, or some other additional head losses being introduced at the intake structure.

5. **Scenario 4, Minor Loss Contingency & Pipe Degradation:** Scenario 4 retains the pipe degradation condition from Scenario 1 and uses much more conservative estimates of minor losses. For this condition, the highest water demand from the bioprogram was used.

For all of the above scenarios modeled, the pipe system was found to meet the demand at each of the demand nodes with positive driving head, including critical locations such as the incubation head tanks.

Table 4-1. Available Head at Demand Nodes

Supply Line	Critical Location	Available Head (ft)				
		Scenario 0	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Coho Building	Incubation Head Tank	1.27	1.09	0.64	0.35	0.73
Chinook Rearing	Final Raceway Manifold	2.48	2.24	2.24	1.95	0.61
Chinook Incubation	Incubation Head Tank	1.07	0.88	0.37	0.08	0.19
Adult Holding	Chinook Holding Pond Manifold	15.89	15.85	13.71	13.42	14.71

It is noteworthy, however, that hydraulic head is limited and therefore infrastructure was kept as low as possible, including the use of half-stacks for incubation. In addition, pressurized cleanouts are provided at intervals along the supply pipelines such that water may be blown out and pipes cleaned if fouling of the pipe or accumulation of fine sediments occurs. The supply pipes will be screened at the upstream end, and these cleanouts are provided as a contingency feature to ensure that the hydraulic head is not impacted over time. Pipe sizes are shown in the Drawing package accompanying this document.

4.2 Production Drain System

The production drain system is the primary drain system for all hatchery infrastructure and drains to the adult holding ponds and out to Fall Creek through the fish ladder. The production drain system consists of lateral lines that convey flow from individual hatchery elements to larger trunk lines that collect and convey flows to their terminus. The system was designed to convey flows primarily in a gravity flow regime, such that pipes would not pressurize and hydraulically connect the ponds. Pipes were sized such that at maximum flow rates the pipes would flow at most 75 percent full, which is typical for the design of open channel drain piping.

In the lower portion of the production drain system, riser pipes distribute flows into the three adult holding ponds, and therefore, the trunk line in the lower portion of the site will pressurize. Calculations demonstrate that this lower pressurization of the pipe occurs well below the invert elevation of all the upstream pond and raceway systems, and therefore no impacts will be conveyed to those design elements. This transition from gravity flow to pressure pipe flow will require the pipe to have adequate venting to provide the necessary air flow into the pipe to accommodate the transition.

While the production drain system is expected to have minimal solids content due to the outlet configurations of the upstream ponds, the pipes were designed to maintain minimum self-cleaning velocities such that accumulation of biosolids or suspended sediment would not occur in the pipeline. Thus, it is expected that biofouling will occur over the 8-year life of the facility. Regularly spaced cleanouts are provided to the ground surface such that these pipes can be cleaned at intervals and operations are not inhibited. Calculations in support of the production drain system hydraulics, including vent pipes, can be found in Appendix A, and pipe sizing information can be found on the Drawings accompanying this document.

4.3 Waste Drain System

The waste drain system will be used when cleaning the facilities, and significant content of biosolids is anticipated in the effluent. The waste drain system conveys biosolid-laden flows from each of the hatchery vessels or raceways to the settling pond located adjacent to the adult holding ponds. At each of the hatchery vessels or raceways, a riser pipe will be provided to the ground surface with a cam-lock fitting on the end. When cleaning the ponds or vessels, hatchery operators will vacuum waste to these riser pipes that will then discharge to the waste drain system. Because this system is fed by vacuum cleaning flows only, the system has a uniform design flow of approximately 200 gpm, under the assumption that only one to two of the raceways or vats will be cleaned simultaneously.

The waste drain system was designed similar to the production drain system to operate in a gravity flow regime, and pipes were sized to flow at most 70 percent full at the maximum design flow. These pipes, however, will maintain an open channel regime all the way to their outlet at the settling pond wet well. Vent pipes are provided at locations of steep slopes, such that sufficient airflow is maintained in the pipe. The waste drain system will have cleanouts to grade at regular intervals for cleaning, as necessary. Calculations associated with the waste drain system, including vent pipes, are provided in Appendix A, and pipe sizes are summarized in the Drawings accompanying this document.

4.4 Volitional Fish Release Pipes

The volitional fish release pipes are provided from the Coho rearing raceways, the Chinook rearing raceways, and from the adult holding ponds, where there is potential for raising juvenile fish, to various outlet points in Fall Creek. Volitional fish release pipes were subject to more stringent criteria than the other pipe systems, because of the entrained fish in the flow. Design criteria are summarized in Section 2.6 above and follow guidance from NMFS (2011) for fish bypass pipes. All volitional fish release pipes will be butt-welded HDPE and will have any internal weld beads or burrs removed for fish safety.

For the Coho rearing raceways, flow-through rates were limited, and therefore at volitional release the entirety of the flow is to be directed through the volitional release pipe to the existing concrete flume and ultimately out to Fall Creek. This location appears to have been previously used for fish release, and therefore was deemed appropriate and the most cost-effective solution due to the proximity of the existing raceways to Fall Creek. The drop into Fall Creek is relatively limited, and therefore impact velocities will be well below the maximum threshold recommended by NMFS. Because fish are released in a juvenile state, and generally not during the trapping period, fish released to Fall Creek will have free egress down from the hatchery site to the lower reaches of Fall Creek and into the Klamath River.

For the Chinook rearing raceways, the majority of the hatchery water right will be flowing through the Chinook raceways at volitional release, and therefore, the flow needs to be distributed between the volitional release pipe and the production drain system that supplies water to the lower raceway bank. Due to the constraints on the volitional release pipe (depth in pipe greater than 40 percent full, but less than 70 percent full), the pipe will only be able to accommodate a limited range of flows. A flow range from 2.6 cfs to 4.5 cfs (about 25 to 50 percent of the Chinook pond outflow) was selected for the volitional release pipe, allowing a majority of the water to supply the lower site. Outside of the defined flow range, the volitional release pipe will not operate as intended. The fish ladder is not anticipated to be in operation during volitional fish release. Juveniles reared in the adult ponds, if utilized, will need to be transferred off station and/or volitionally released *prior to* release of the primary Chinook production ponds; this is an important operational consideration as surplus juveniles reared in the adult ponds are on second-pass water and biomass estimates for surplus production assume 100% of the flow coming from the Chinook ponds (this will not be the case when production Chinook in the upper rearing ponds are being released volitionally).

The Chinook volitional release pipe will convey fish to a constructed plunge pool in the east overbank area adjacent to Fall Creek, approximately 150 feet upstream of the existing Copco Road bridge. The pipe invert at the plunge pool will be approximately 1.7 feet above the high tailwater level in Fall Creek, and approximately 2.3 feet above the low tailwater level. The plunge pool will be excavated such that it is approximately 4.5 feet deep at high tailwater and 4.0 feet deep at low tailwater. This results in impact velocities at the low water surface of approximately 13 ft/s and at the bottom of the pool of approximately 17 ft/s. Both of these values are within the 25 ft/s recommended by NMFS (2011), and the plunge pool was deemed appropriate.

Finally, the adult holding volitional release pipe will convey the entirety of the flows through the Coho and Chinook adult holding ponds, and possibly the flow through the sorting/trapping pond, as well. This results in a design flow range from 6.7 cfs to 10 cfs. The adult holding volitional release pipe is located less than 20 ft from the fish ladder entrance pool, and therefore will only convey fish a short distance.

Further details regarding the design of the volitional fish release pipes and the plunge pools can be found in the calculations in Appendix A. Pipe design and sizing are summarized in the Drawing package accompanying this report.

4.5 Fish Ladder

The Denil fish ladder was designed according to standard Denil geometry, as provided by USFWS (2017), and according to the guidance provided by NMFS (2011). It was assumed that during the trapping season, when the fish ladder is in operation, the full water right (10 cfs) would be directed to the adult holding ponds (either through the production drain system or the supply pipe) and out through the fish ladder, with only occasional, minimal losses to cleaning and utility water. The slope of the fish ladder was selected to minimize the slope and resultant turbulence in the ladder, while avoiding the introduction of turns and rest pools. It was found that at the design flow, a 2.5-foot-wide ladder at 18 percent slope would result in flow depths in excess of 2.0 feet and cross-section average velocities less than 2.0 ft/s. This was within guidance for these structures and provided flow characteristics that would be passable to both adult Chinook and Coho. The rating curve calculated in association with the designed fishway is presented in Figure 4-1.

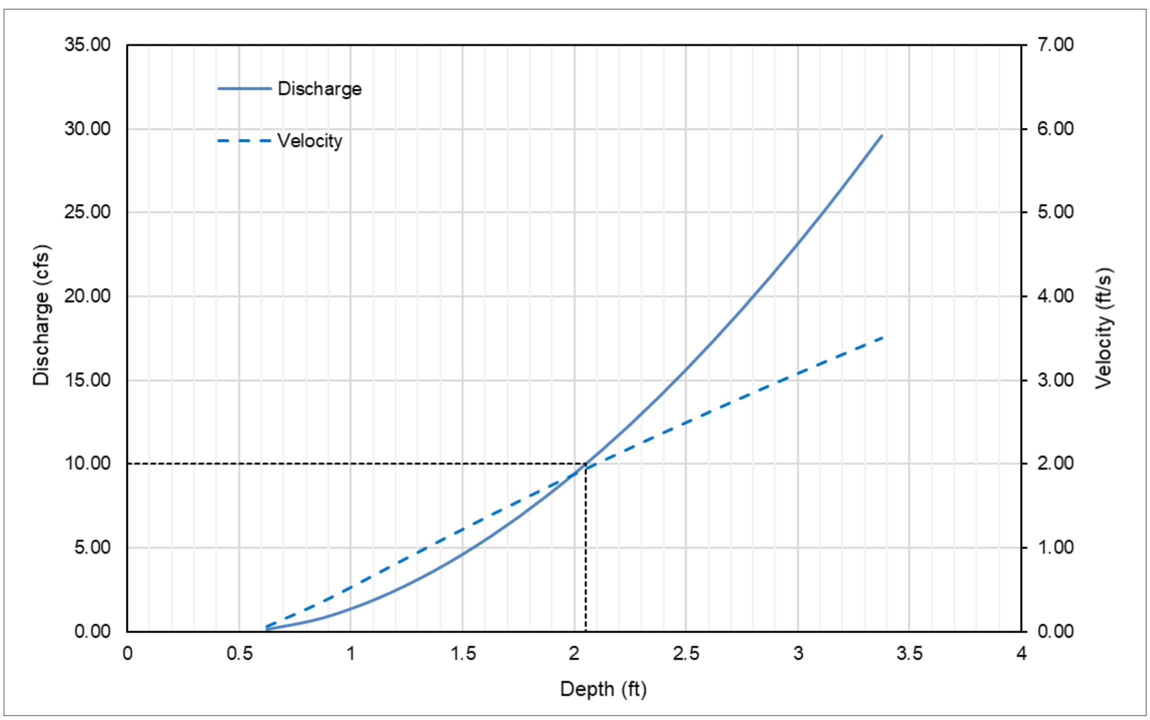

Figure 4-1. Denil Fish Ladder Rating Curve

At the top of the Denil fish ladder will be a resting and turning pool with a set of dam boards that will allow fish to pass into the adult holding raceway outlet structure and on to the finger weir. The turning and resting pool provides an energy dissipation factor of 2.8 ft-lbs/s-ft^3, which is below the maximum value recommended by NMFS (2011) of 4.0 ft-lbs/s-ft^3.

4.6 Finger Weir

After passing the fish ladder, a 1-foot drop will be maintained across a finger weir coming out of the trapping and sorting pond. The finger weir was designed according to the hydraulic guidance provided by the U.S. Army Corps of Engineers (Bell, 1991), to maintain 2 to 6 inches of water depth above the fingers of the weir. The finger weir will be attached to a gate that will allow for raising and lowering of the weir based on the desired water surface level in the pond. This water surface will need to be coordinated with the downstream set of dam boards, such that the hydraulic control in the pond is maintained at the finger weir.

4.7 Fish Barrier

The fish barrier system consists of three components. Dam A and Dam B will be modified to serve as permanent velocity barriers to preclude both juvenile and adult fish passage to the impoundments above the dams. At the fishway, a removable picket barrier with a concrete sill will be installed to direct adult fish to the fishway during the trapping season. The hydraulic design of each of these barriers is discussed below.

4.7.1 Dam A and Dam B Velocity Barriers

NMFS (2011) recommended velocity barriers consist of two components: (1) a downstream high-velocity apron, and (2) an upstream weir. The combination of these two components produces a shallow flow depth and a high velocity on the apron, which makes the jump for an adult anadromous fish impassable over the weir. The design of the Dam A and Dam B velocity barriers use the existing dams as the weir portion of the barrier and are amended with a downstream steep concrete apron to form an impassable barrier to adult fish.

Downstream aprons were provided in accordance with NMFS (2011) recommendations and maintain a minimum length of 16 feet and a slope of about 6.3 percent (16H:1V). Open-channel flow calculations with an assumed Manning's roughness of 0.015 (concrete, float finish; Chow, 1959) were performed for the flows on the aprons to ensure flows were shallow and fast such that the jump over the dams would be impassable. Table 4-2 summarizes the calculated depths and velocities.

Table 4-2. Velocity Apron Depths and Velocities

Location	Flow Condition	Flow (cfs)	Depth (in)	Velocity (ft/s)
Dam A	High Flow	50.0	2.4	8.5
	Low Flow	15.0	1.2	5.3
Dam B	Juvenile High Flow	62.1	4.9	13.1
	Adult High Flow	56.9	4.7	12.7
	Adult Low Flow	8.4	1.5	6.0

The Dam B velocity barrier, as described above in Section 3.12, is unable to meet the NMFS (2011) recommended weir height of 3.5 feet. In order to pass the fish passage high flows without overtopping the dam, the crest of the stop logs will be set approximately 2-feet 2-inches above the invert at the top of the velocity apron. Therefore, the stop logs at this location will have a nappe extension fitting that will be placed over the aluminum stop logs and will push the nappe approximately 3.0 feet away from the stop logs making for more difficult jump conditions for upstream migrating fish. This extension is based on McMillen Jacobs' successful past experience retrofitting sites that did not meet NMFS (2011) criteria.

The velocity barriers will also be equipped with vent pipes located under the overflow nappe with risers built onto/into the concrete walls. The pipe risers will be open to the atmosphere above the high-water elevation at the weir overflow. These vent pipes will ensure an aerated nappe which decreases upstream water surface elevations and minimizes the potential for fish jumping past the barrier.

4.7.2 Removable Picket Barrier

The removable picket barrier to be installed yearly at the beginning of the trapping period was designed according to typical guidance from NMFS (2011) for picket barrier systems. Through picket velocities were calculated for the pickets based on the gross area of picket panels and adjusted for the rotation about the stream transect and the rotation about vertical. Table 4-3 summarizes the calculations through the picket barrier.

Table 4-3. Picket Barrier Flow Characteristics

Flow Condition	Flow (cfs)	Depth (ft)	Through Picket Velocity (ft/s)	Head Loss Across Pickets (in)
Fish Passage High Flow	71.9	1.7	2.0	4.0
Fish Passage Low Flow	23.4	1.1	1.0	1.0

The picket barrier is not able to meet the through picket velocity criterion of 1 ft/s for the design high flow. Meeting the 1 ft/s picket velocity criterion, however, has proven challenging in the setting of small mountain streams across the Pacific Northwest, such as Fall Creek. It is not anticipated that the 1 ft/s picket velocity criterion will be met by this design; however, it is not expected that the picket barrier will pose a fish impingement concern for the following reasons:

1. The fish habitat above this barrier is very limited, and fish (especially anadromous fish) are not anticipated upstream of the picket barrier where impingement could occur.
2. The exposure window when the pickets will be in place is limited to the period of trapping. At all other times, the pickets will be removed, and the stream will flow through naturally.
3. The screen is oriented at an angle to the stream transverse, increasing the wetted area of the picket panels and decreasing average velocities through the pickets to the greatest degree possible.

4. Natural flow velocities in the stream around this location are as high as 4.5 ft/s under high-flow conditions. The flow through the pickets will be much less than the natural surrounding stream, due to the orientation of the barrier, and effects of the sill on the stream hydraulics. In addition, the angle of the picket will guide the fish to the entrance of the fishway and the attraction flows emanating from the fishway.

Likewise, it may be observed that the minimum submerged picket depth at the barrier of 2 feet is not attained under any of the design flows. This is to be expected as the natural flow depth in this portion of the stream is only about 9 inches at low flow. Meeting the minimum submerged picket depths would require significant deviation about the natural channel flows. Therefore, the current design meets the intent of the picket barrier guidelines and criteria, though, like many other sites on small mountainous streams, it is unable to meet the values specified.

4.8 Effluent Treatment

Primary effluent concerns for the FCFH will be settleable solids (see *TM 002 – Design Criteria* for a complete listing of NPDES requirements), and particularly biosolids produced in the hatchery vessels. As discussed above, biosolids will be cleaned from all vessels and ponds via vacuum to the waste drain system, where they will be deposited in the settling pond. Idaho DEQ (nd), which has been widely used in aquaculture applications across the Pacific Northwest, recommends that a settling pond be sized based on a settling velocity of 0.00151 ft/s, such that the overflow velocity is less than the settling velocity ($V_o < V_s$). It was found that the existing pond in the lower raceway bank provided approximately 2.6 times the surface area required for settling of the biosolids, or if the pond is split into two chambers, each would maintain approximately 1.3 times the surface area required.

The other effluent concern for the facility will be the use of therapeutants or inorganics that could occasionally be required for treatment of fish. Use of such therapeutants is not anticipated due to the high quality of the intake water and the short design life of the facility. If it is determined that therapeutants will be required, the use of therapeutants used for fish treatments can be addressed operationally by using the 3,200 ft^3 of effluent holding provided by the effluent pond. While use would depend on flow rates supplied to each individual rearing unit, the effluent ponds provide short-term storage of up to 24,000 gallons of therapeutant laden flow that could then be pumped to appropriate storage tanks and transferred to approved off-site disposal areas, or discharged to Fall Creek after a prescriptive residence time.

5.0 Civil Design

5.1 General Description

This section presents the civil design elements at each of the Project structures and summarizes the design of the overall site layout.

5.2 Erosion and Sediment Control

The Contractor is required to install, monitor, and maintain erosion and sediment control measures as identified within the Project Drawings, and prepare the required documents discussed in Section 2.5 as determined by the various regulatory agencies. The erosion control measures shall be maintained for the duration of the construction project.

The Contractor will be required to install specified permanent post-construction measures as required for the Project. The permanent measures are designed to protect the exposed slopes until the vegetation is fully established. Following construction, the disturbed areas of the Project site will be revegetated with native plant mixes. The Contractor will be required to submit a Notice of Termination (NOT) to the State Water Resources Control Board (SWRCB) after completing the Project. This is required to be relieved from the Construction General Permit requirements. Final soil stabilization throughout the proposed Project area must be achieved prior to the SWRCB approval of the NOT.

5.3 North Site

The North Site, or the Project site north of existing Copco Road, consists of a pad at approximate elevation 2503 (NAVD88) that was designed to support the Coho Building and infrastructure, the Chinook raceways, the Chinook Incubation Building and supporting infrastructure, and the vault toilet. The pad elevation was selected such that sufficient hydraulic head would be maintained from the intake structure at elevation 2510.4 (NAVD88) to the design elements, while minimizing earthworks quantities.

Pad limits were determined to maintain a footprint within previous work boundaries, to the extent possible. The pad maintains sufficient space for access and egress around structures such that the whole site is accessible via standard pickup truck. The site layout also maintains access for an assumed tagging and marking trailer to locations near the Coho rearing raceways and the Chinook rearing raceways. Additionally, access is maintained for an assumed, design pump truck to the new vault toilet, and the storm drain system hydrodynamic separator. A swept path analysis was performed to ensure site access, and discussion of design vehicles, clearances, and swept path results can be found in Appendix B.

5.3.1 Fencing

Per direction from CDFW, perimeter fencing around the entirety of the North Site will not be required. Fencing will be required, however, around the Chinook rearing raceways as part of the predator exclusion system. Fencing will be 8-foot-tall chain link fence with three strands of barbed wire oriented at 45 degrees outward to prevent larger predators from climbing over the fence. The fencing layout will be as indicated on the drawings and will have man-gates and vehicular access double-leaf gates in the locations indicated.

5.3.2 Grading

Site grading at the North Site will generally be a flat pad at elevation 2503 (NAVD88) but will be graded at slopes (0.02 ft/ft) away from all buildings and structures. Cut-and-fill slopes will be graded at a maximum slope of 2H:1V in accordance with the Project civil design criteria. Locally steepened slopes, as indicated on the Drawings, may be required in some locations, provided they meet approval of the project geotechnical engineer. The pad will be surfaced with a 4-inch-thick ¾-inch-maximum Type Granular Fill per specifications, and an 8-inch-thick Type Aggregate Subbase material per specifications beneath.

5.3.2.1 Site Drainage

The majority of the site drainage from impervious areas, including rooftops, will be directed to a series of CalTrans standard catch basins distributed about the site. From these catch basins, storm drainpipes will convey flows to the north site hydrodynamic separator that will be sized to treat a water quality storm (WQS) event of 0.33 cfs, prior to discharging back to Fall Creek in the Chinook volitional release plunge pool. The north site hydrodynamic separator will be a proprietary system that will treat both suspended solids entrained in the site runoff, and any oils, grease, or hydrocarbons from the roadway and parking areas on site.

A small portion of the north site, around the Chinook incubation building, will drain to a drain rock sump adjacent to Copco road. The drain rock sump will be below grade and will be lined with an impervious geomembrane that will contain any accumulated runoff. A perforated PVC pipe in the drain rock will collect flows and convey them to the south site storm drain system for treatment and release.

The drain rock sump and outlet pipe were sized such that the sump could contain runoff from the 2-year storm event. Runoff volumes in excess of the 2-year event will overflow the sump area across the driveway and will drain directly to Fall Creek. Pollutant load in runoff is typically taken up by the initial stages of runoff, prior to arrival of peak flow, which will report to the drainage sump first and will pass into the storm drain system for treatment and release.

Calculations supporting the design of the site drainage system, including the runoff modeling, pipe system sizing, hydrodynamic separator selection, and drainage sump sizing are provided in Appendix B.

5.3.3 Intake Structure and Dam A Velocity Barrier Modifications

5.3.3.1 Cofferdam and Dewatering

It is anticipated that a cofferdam will be required to aid construction of the intake and Dam A velocity barrier modifications and will need to be staged with construction. The Contractor will review the hydrology and hydraulics of the powerhouse canal (Specification 01 12 00) and determine the elevations required for any cofferdam system. Dewatering pumps will be placed inside the cofferdam and the intake construction area to collect seepage and pump it over the cofferdam to the Dam A impoundment. The Contractor will be responsible to treat flows in accordance with their CGP, prior to discharge in the impoundment. Staging of the cofferdam must always maintain water to the City of Yreka intake . Therefore, it is expected that the cofferdam will be in place around the working area on the southeast bank of the powerhouse canal and will extend across the upstream face of the existing Dam A, such that the intake screens for the City of Yreka are not obstructed. Flows in excess of the City of Yreka withdrawals will need to be bypassed downstream past the work area. Staging and design of the cofferdam system will ultimately be the responsibility of the Contractor.

5.3.3.2 Excavation and Backfill

Around the intake structure, a pad at elevation 2512.4 (NAVD88) will be constructed to exclude water behind the intake. The pad will be constructed from available on-site fill materials, in accordance with the specifications, and will be lined with riprap available from the North Site pad grading excavation. A cutoff wall will be installed down to bedrock along the north and east faces of the intake structure extending 30-feet into the overbank area to mitigate any seepage that may occur from the Dam A impoundment.

Under the intake, a 6-inch-thick layer Type Drain Rock, Graded (DRG) will be placed to mitigate any pore water pressure that may develop on the bottom of the structure.

The Dam A concrete velocity apron will likewise be constructed over a 6-inch-thick layer of free-draining graded drain rock and will have French drains on either side of the apron to relieve any pressure that may build up on the bottom of the slab. French drains will consist of a coarse drain rock backfill, surrounding a perforated pipe that will outlet to the powerhouse channel immediately downstream of the velocity barrier.

5.3.3.3 Fencing

Fencing will be provided around the intake structure for safety and for protection of equipment such as the traveling screens and gates from theft or vandalism. The intake structure enclosure will be accessed through a double leaf gate such that vehicles can access the structure for maintenance or for hauling away accumulated debris from the traveling screens. Fencing will be 8-foot-tall chain link fence with three strands of barbed wire oriented at 45 degrees outward.

5.3.4 Dam B Velocity Barrier Modifications

5.3.4.1 Cofferdam and Dewatering

It is anticipated that a cofferdam will be required to aid construction of the Dam B velocity barrier modifications. The Contractor will review the hydrology and hydraulics of Fall Creek (Specification 01 12 00) and determine the elevations required for any cofferdam system. Dewatering pumps will be placed inside the cofferdam and construction area to collect seepage and pump it downstream into Fall Creek beyond the limits of construction. The Contractor will be responsible to treat flows in accordance with their CGP, prior to discharge in the creek. The Dam B velocity barrier modifications will span a portion of the creek at this location, but will maintain flows to the City of Yreka Dam B intake. A bypass pipe will need to be installed to maintain flows past the construction area.

5.3.4.2 Excavation and Backfill

The concrete velocity apron will be constructed above grade on the downstream side of Dam B. After clearing and grubbing, and scarifying and recompacting the subgrade, the concrete subgrade will be built up on Type Structural Fill (SF) compacted to 95 percent maximum dry density as determined by ASTM D 1557, to 6 inches below the bottom of the concrete, as depicted on the Drawings. The structural fill will be overlaid with a 6-inch-thick layer of Type DRG fill, per specifications, that will drain to French drains on either side of the concrete velocity apron.

Above the French drains, a 2.0-foot thick layer of approximately 12-inch (D_{50}) riprap will be placed to mitigate any potential erosion that would arise from dam overtopping during extreme events. This riprap layer will be placed against the valley side walls, which are expected to consist of bedrock (per the City of Yreka as-built information). Downstream of this structure, any in-stream disturbance will be replaced with natural cobbles removed during clearing and grubbing of the site.

5.3.5 Coho Building

The Coho Building will be located at the northern extent of the North Site pad grading. The pre-engineered metal building will consist of one room that houses Coho infrastructure from incubation, through first-feeding, and grow-out. The building will be accessible via man-door on the south side of the building, or through one of three roll-up doors (two on the north side of the building, one on the south side). To the north of the building, the concrete slab will extend approximately 22 feet from the outside face of the building to the two existing Coho rearing raceways. The roof from the building will extend out over the existing rearing raceways, and predator netting connected to the roof will form an enclosure around the outdoor rearing raceways. Bollards will be located at all building corners, and along the length of the existing raceways at 10-foot spacing to ensure that a 5-foot offset is maintained by vehicles at all times.

5.3.5.1 Excavation and Backfill

In order to provide a consistent subgrade below the Coho Building, the subgrade will be over-excavated to a minimum of 18 inches under all footings and 6 inches under all slabs The subgrade will be scarified to a depth of 6-inches and compacted per the specifications. The areas will be back-filled with Type SF material per specifications, which is a readily compacted, crushed rock with 1.5-inch-maximum aggregate. The Type SF fill should extend a minimum of 18 inches beyond the edge of the footings. The structural fill should be compacted to 95 percent maximum dry density as determined by ASTM D 1557.

5.3.6 Chinook Raceways

The Chinook raceways will be outdoors and will consist of two banks of four ponds. These raceways will all discharge to a common exit channel, and the exit channels between the two raceway banks will be connected by a 2.5-foot-wide by 3.0-foot-tall buried box culvert. The two raceway banks will have a 12-foot center aisle running between them for vehicular access. The ponds will be surrounded by fencing and predator netting (see Section 5.3.1 above) that will maintain a minimum 3.0-foot offset from the pond concrete, such that personnel can access all sides of the ponds from the inside of the enclosure.

The pond inverts will be located at elevation 2500 (NAVD88) and the pond walls will extend 2 feet above grade to elevation 2505 (NAVD88).

5.3.6.1 Excavation and Backfill

The ponds will be excavated 3 ft below the pad elevation (2503 NAVD88) and will be over-excavated an additional 6 inches. The subgrade shall be scarified and recompacted, and a 6-inch layer of Type DRG, per specifications, will be placed and compacted to form a suitable subgrade for the ponds.

5.3.7 Chinook Incubation Building

The Chinook Incubation Building is located at the southern extent of the North Site adjacent to the existing Copco Road. The pre-engineered metal building will house all Chinook incubation infrastructure, including incubation stacks and working vessels. The building will be accessed on the west side through a set of double doors, or through one of roll-up doors (north, south, and west building faces) for equipment access.

Along the southern edge of the building, a separate room will house the site's electrical infrastructure. The electrical room will be accessed through a man-door on the west side of the building. Around the outside of the building, the building corners will be protected by bollards.

5.3.7.1 Excavation and Backfill

In order to provide a consistent subgrade below the Chinook Incubation Building, the subgrade will be over-excavated to a minimum of 18 inches under all footings and 6 inches under all slabs and will be back-filled with Type SF material per specifications, which is a readily compacted, crushed rock with 1.5-inch-maximum aggregate. The Type SF fill should extend a minimum of 18 inches beyond the edge of the footings. The structural fill should be compacted to 95 percent maximum dry density as determined by ASTM D 1557.

5.4 South Site

The South Site, or the Project site south of existing Copco Road, consists of a pad extending down from the existing road to elevation 2491.5 (NAVD88) designed to support the Spawning Building. In addition, the South Site contains the genset and propane tank, the adult holding ponds, the settling pond, the fish ladder, and the removable fish barrier.

The South Site was designed to provide vehicular access to the Spawning Building and to the settling pond by the design vehicles. A swept path analysis was performed for this area, and the design vehicles have access and egress to the design points. The swept path analysis is summarized in Appendix B.

5.4.1 Fencing

Fencing is provided around the majority of the South Site, to preclude unhindered access to the Spawning Building equipment, the holding ponds, and the settling pond. Fencing will be 8-foot-tall chain-link fence with three strands of barbed wire oriented at 45 degrees outward to prevent larger predators from climbing over the fence. The fencing layout will be as indicated on the Drawings and will have man-gates and vehicular access double-leaf gates in the locations indicated.

5.4.2 Grading

Grading of the area was primarily driven by the elevation of the Spawning Building and existing concrete raceways and the elevation of Copco Road. Grades were maintained from Copco Road (approx. elevation 2496 [NAVD88]) down to this lower site (approx. elevation 2491.5 [NAVD88]) at no greater than 8 percent for vehicular access. At elevation 2491.5 (NAVD88), the pad flattens out and remains at or slightly below that elevation. The pad is primarily in cut, and maximum cut slopes of 2H:1V were maintained.

The pad will be surfaced with a 4-inch-thick ¾-inch-maximum Type Granular Fill per specifications, and an 8-inch thick Type Aggregate Subbase material per specifications beneath.

5.4.2.1 Site Drainage

Due to the grading constraints, the pad is naturally graded toward the Spawning Building. Concrete swales will collect water around the Spawning Building and will direct any surface runoff to catch basins located around the South Site pad grading. Catch basins will direct flows through the storm drain system to the south site hydrodynamic separator. The hydrodynamic separator will be sized for a WQS event of 0.36 cfs, and will treat suspended sediment loads and oil, grease, and hydrocarbons from parking and driveway areas, prior to discharge into Fall Creek.

5.4.3 Spawning Building

The Spawning Building is located at the north end of the existing lower raceway bank, approximately 10 feet 3 inches from the outside face of the concrete. The pre-engineered metal building will house all infrastructure necessary for spawning activities, including the egg-rinsing table, water hardening table, holding table, air spawning table, fish chutes, fish conveyors, collection bins, etc. To the south, the Spawning Building will have an awning that will be used to keep personnel out of the elements during spawning activities and collection of fish from the adult holding ponds.

The Spawning Building will have access from the east and the west by man-doors and will have roll-up doors to the north and south for equipment access. A parking area will be maintained on the west side of the building, and all building corners will be protected by bollards.

5.4.3.1 Excavation and Backfill

In order to provide a consistent subgrade below the Spawning Building, the subgrade will be over-excavated to a minimum of 18 inches under all footings and 6 inches under all slabs with the subgrade being scarified to a depth of 6 inches and compacted per the earthwork specifications. The area will be back-filled with Type SF material per specifications, which is a readily compacted, crushed rock with 1.5-inch-maximum aggregate. The Type SF fill should extend a minimum of 18 inches beyond the edge of the footings. The structural fill should be compacted to 95 percent maximum dry density as determined by ASTM D 1557.

5.4.4 Fish Ladder and Temporary Picket Barrier

The fish ladder and temporary picket barrier will be located at the southern end of the existing raceway bank, and in the adjacent stretch of Fall Creek. The temporary picket barrier will be placed yearly at the beginning of the trapping period; however, a concrete sill and walls will be permanently in the stream. Both the fish ladder and the sill will be concrete structures, as depicted in the plans. In addition, some localized grading will be provided around these structures.

5.4.4.1 Cofferdam and Dewatering

It is anticipated that a cofferdam will be required to aid construction of both the fish ladder and the temporary picket barrier sill. The Contractor will review the hydrology and hydraulics of Fall Creek (Specification 01 12 00) and determine the elevations required for any cofferdam system. Dewatering pumps will be placed inside the cofferdam and construction area to collect seepage and pump it downstream into Fall Creek beyond the limits of construction. The Contractor will be responsible to treat flows in accordance with their CGP, prior to discharge in the creek. The concrete sill will span the entire creek at this location, and therefore a bypass pipe will need to be installed to maintain flows past the construction area.

5.4.4.2 Excavation and Backfill

After the area is cleared and grubbed and topsoil is stripped from the site, the fishway will be excavated into the eastern bank of Fall Creek. The fish ladder will be over-excavated an additional 6 inches and after the subgrade is scarified and recompacted, a 6-inch layer of Type DRG material per specifications will be placed and compacted to form a suitable subgrade for the concrete construction.

For the concrete sill, a similar process will be performed with a 6-inch-thick layer of Type DRG material underlaying the concrete construction. Following completion of the concrete work in this area, the natural creek bed will be restored with any material or cobbles that were removed during the initial clearing of the site.

6.0 Geotechnical Design

6.1 Engineering Soil Properties

Engineering soil properties were selected based on the subsurface conditions described in the Geotechnical Data Report. Anticipated ranges in soil properties are provided below.

Table 6-1. Soil Properties

Soil Unit	Total Unit Weight (pcf)	Friction Angle, ϕ (deg)	Cohesion, c (psf)
Existing Fill	140	38	0
Colluvium	115-120	26-30	50 - 200
Alluvium	120	28-32	0

6.2 Shallow Foundations

The Coho Building, Hatchery Building, and Chinook Raceways will be supported on shallow foundations. Recommendations for shallow foundations are provided in the following sections.

6.2.1 Bearing Surface Preparation

Based on available geotechnical data, structures will bear primarily within colluvium soils. Footings bearing in colluvium should be supported on an 18-inch to 24-inch section of imported structural fill (SF) foundation base material. The bearing surface should be inspected prior to placement of SF and should be clear of deleterious material and standing water. If soft, pumping soils are observed at the bearing elevation, an additional 6- to 12-inches of colluvium should be removed from below the footing. A non-woven geotextile consisting of Mirafi RS280i or equivalent, should be placed at the base of the footing excavation for added stability.

Structural fill should be placed in loose lifts of 6- to 8-inches and compacted to 95 percent of maximum dry density (MDD).

6.2.2 Bearing Resistance

Structures bearing on soils prepared as outlined in the previous section may be design using an allowable bearing resistance of 2 kips per square foot (ksf). This allowable bearing resistance applies to the total of dead and long-term live lads and may be increased by up to one-third for wind or seismic loads.

6.2.3 Lateral Resistance

Lateral forces on shallow foundation may be resisted by passive resistance on the side of footings and by friction on the base of the footings. Frictional resistance may be computed using an allowable coefficient of friction of 0.49 for cast-in-place foundations and 0.39 for precast concrete foundations applied to vertical dead load forces.

Passive pressure acting at the side of the shallow foundation can be estimated using an equivalent fluid density of 400 pounds per cubic foot (pcf) (triangular distribution).

The above coefficients of friction and passive equivalent fluid density values incorporate a FS of 1.5.

6.3 Lateral Earth Pressures

Lateral earth pressures are needed for design of the raceways and adult holding ponds. The raceways and holding ponds are restrained against deflection; therefore, at-rest earth pressures are recommended for use in design. At-Rest earth pressure coefficients are presented below.

Table 6-2. At-Rest Earth Pressure Coefficients

Soil Unit	At-Rest, K_O	At Rest + Seismic, K_{OE}
Colluvium	0.53	0.91

7.0 Architectural Design

7.1 General Description

Architectural design for the Project was largely driven by building use and occupancy. The scope of work included the coordination of the pre-engineered metal buildings and associated doors, skylights and accessories. Each building is a stand-alone structure and is comprised of primary frame members, end wall columns, horizontal wall girts and roof purlins. Cross bracing, moment frames and/or portal frames are provided as necessary. Each building is clad with 42-inch wide and 3-inch thick insulated standing seam metal roof panels and 42-inch wide and 2-inch thick insulated metal wall panels. Overhead sectional doors, overhead coiling doors and man doors are provided based on use.

7.2 Coho Building

The Coho Building has a covered roof area of 6,960 sf with an enclosed area of 3,635 sf and an overhang comprised of 3,325 sf. The roof overhang protects existing raceways that will receive some minor upgrades and be enclosed with predator netting. Within the confines of the building envelope, there are two new raceways and various other components that will be separated with ceiling hung biosecurity curtains. This building will have three (3) overhead coiling doors in lieu of sectional doors to avoid conflicts with the biosecurity curtains and one (1) man door to access the space. Natural light will be provided within the space by means of 12 solar tube directional skylights.

7.3 Chinook Incubation Building

The Chinook Incubation Building is fully enclosed and has a floor area of 3,227 sf. The floor plan is divided into the Chinook Incubation space and a smaller adjacent Electrical Room. Within the confines of the building envelope, there are multiple rows of incubational tanks and holding tanks. There is a network of floor trenches and sump drains that cater to the wet environment within the building. The Chinook Incubation space will have three (3) overhead sectional doors and one (1) double man door to access the space. The adjacent Electrical Room is accessed via its own man door from the exterior of the building. Natural light will be provided within the space by means of 12 solar tube directional skylights.

7.4 Spawning Building

The Spawning Building is the smallest building on site and has a covered roof area of 1,089 sf with an enclosed area of 812 sf and an overhang comprised of 277 sf. The roof overhang protects personnel when completing spawning activities outside of the building. Within the confines of the building envelope, the floor plan is wide open and the slab on grade, unlike the other buildings this building does not have raceways or incubation trays. There is a network of floor trenches and sump drains that cater to the wet environment within the building. This building will have two (2) overhead sectional doors and two (2) man door to access the space. Natural light will be provided within the space by means of four (4) solar tube directional skylights.

8.0 Structural Design

8.1 General Description

The structural facilities consists of 11 main systems: (1) the intake structure, (2) the dam A velocity barrier, (3) the dam B velocity barrier, (4) the coho building, (5) the chinook raceways, (6) the chinook incubation building, (7) the spawning building, (8) the adult holding ponds, (9) the meter vault, (10) the fish ladder, (11) the temporary picket barrier, and (12) the fish release pipe support. Structural calculations for these systems can be found in Appendix D of this DDR.

8.2 Intake Structure

The intake structure measuring approximately 10 feet by 10 feet is situated at the south end of dam A. Portions of the existing dam will need to be demolished in order to construct the intake structure, as the bottom of the intake structure extends below the bottom of the dam. The dam would therefore be undermined during the construction of the intake structure. Since a portion of the dam will be removed, a cutoff wall will be constructed below the intake structure and extend to the end of the previous southern end of dam A. The cutoff wall will tie into the existing cutoff wall at dam A and provide a continuous cutoff to the extent that is currently provided at the existing dam.

The intake structure is composed of reinforced concrete walls with a concrete wingwall measuring 8 feet long, travelling screens with stainless steel support system, and FRP grating across the top providing access to the screens. The new intake structure walls and slab will tie into the existing dam A at the interface with drilled epoxy dowels. Retrofit waterstops will be provided at all joints between new and existing concrete. An oversized travelling screen support column will also provide stop logs slots for dewatering the intake behind the travelling screens.

The new intake structure improves the overall stability of dam A. The intake structure consists of a considerable amount of additional concrete, increasing the overall weight and base width of the structure. This will increase the factor of safety of the dam due to sliding and overturning.

8.3 Dam A Velocity Barrier Modifications

In addition to the demolition work at the south end of the dam, the toe of the dam for the entire width of the proposed downstream velocity barrier apron will need to be demolished. The velocity barrier apron consists of a reinforced concrete apron slab measuring approximately 29 feet wide by 16 feet long with vertical retaining walls at both canal banks. The apron and retaining walls will tie into the existing dam A concrete with drilled epoxy dowels. Retrofit waterstops will be provided at all joints between new and existing concrete. The functionality and condition of the existing dam cutoff wall is unknown; therefore, it is conceivable that uplift pressures could be generated underneath the new velocity barrier. To combat this, a drainage layer with parallel french drains will be provided in the apron slab to allow any buildup of water pressure.

The new velocity barrier also improves the overall stability of dam A. The velocity barrier consists of a considerable amount of additional concrete, increasing the overall weight and base width of the structure.

This will increase the factor of safety of the dam due to sliding and overturning, while also reducing bearing pressures at the toe.

8.4 Dam B Velocity Barrier Modifications

The velocity barrier apron consists of a reinforced concrete apron slab measuring approximately 11.5 feet wide by 16 feet long with vertical retaining walls at both canal banks. The apron and retaining walls will tie into the existing dam B concrete with drilled epoxy dowels. The existing stoplog slots will be replaced with shorter slots on top of a concrete platform, effectively raising the sill elevation of the stoplogs. Retrofit waterstops will be provided at all joints between new and existing concrete. The functionality and condition of the existing dam cutoff wall is unknown; therefore, it is conceivable that uplift pressures could be generated underneath the new velocity barrier. To combat this, a drainage layer with parallel french drains will be provided in the apron slab to allow any buildup of water pressure.

The entire center pier will need to be demolished in order to install the flush drain and ventilation piping. A new steel supported FRP walkway will span across the full width just downstream of the piers. New aluminum stoplogs will be fabricated with a custom top piece that will seal against a nappe extension. The nappe extension is fabricated of aluminum plate reinforced with transverse angle stiffeners. Four (4) threaded eye-bolts provide ample lifting points on the top side of the nappe extension. The extension is set into place by aligning the male side of the four threaded eye-bolts with the holes located on the horizontal flange of the support angle. This prevents the nappe-extensions from lateral movement once in place. The weight of the water on top of the nappe extensions prevents it from lifting out of the holes. The nappe extension is independent of the dam boards, so that the dam boards can be removed prior to removing the nappe extension. This facilitates removal of the system with two laborers.

The new velocity barrier also improves the overall stability of dam B. The velocity barrier consists of a considerable amount of additional concrete, increasing the overall weight and base width of the structure. This will increase the factor of safety of the dam due to sliding and overturning, while also reducing bearing pressures at the toe.

8.5 Coho Building

The Coho Building is the largest of three buildings on the Project. The building consists of a fully enclosed portion measuring approximately 54 feet by 66 feet, and a roof-only portion measuring approximately 50 feet by 66 feet. The roof of the fully enclosed building continues over the roof-only portion for a seamless transition. The building itself is a pre-engineered metal building with insulated metal panels. All exposed steel surfaces of the building will be hot dip galvanized. Flooring will consist of a 6-inch concrete slab. The foundation system consists of cast-in-place (CIP) reinforced concrete stem walls and spread footings for the enclosed portion and four individual column footings for the roof-only portion. The interior column loads are transferred to the soil through square spread footings.

The enclosed portion of the building houses new concrete Coho raceways and various incubation and feeding vessels. The raceways will consist of two ponds measuring approximately 38 feet by 12 feet each. The ponds will consist of 8-inch cast-in-place reinforced concrete walls with embedded stainless guide slots for the existing aluminum fish screens and new aluminum dam boards, and a 2-foot-wide FRP walkway on top of the interior wall. Hinged sections of grating allow access to the guide slots underneath.

Directly adjacent to the building under the roof only portion will be a 20-foot-wide concrete drive-through area for the fish tagging and marking trailer. This area is designed for a 250 psf uniform vehicular surcharge pressure.

The existing concrete raceways will also be under the roof of this structure, directly adjacent to the drive-through. The existing raceway walls and slabs will remain in place, while all of the walls will be raised to finish-floor elevation. The new wall extensions will be tied to the existing walls with drilled epoxy dowels. The existing raceways will be retrofitted with new reinforced concrete pony walls, stainless steel guide slots, FRP walkways, aluminum dam boards and fish screens, and a fish-friendly polyurethane coating. Hinged sections of grating allow access to the guide slots underneath. Predator netting extending down from the roof framing to grade will protect the Coho ponds from birds of prey, namely kingfishers.

8.6 Chinook Raceways

The new Chinook raceways are located just south-east of the Coho Building. The raceways will consist of two banks of four ponds each, with a 12-foot drive-through between the two. Each pond measures approximately 70 feet by 12 feet. The ponds will consist of 8-inch cast-in-place reinforced concrete walls with embedded stainless guide slots for the existing aluminum fish screens and new aluminum dam boards, and a 2-foot-wide FRP walkway on top of all interior walls. Hinged sections of grating allow access to the guide slots underneath. The two bays of fish screen piers will be removeable, with a pipe welded to the bottom of the steel pier that slides and locks into an embedded pipe in the concrete slab.

Chain-link fencing around the perimeter of the Chinook raceways will prevent large predators from entering. A predator netting support structure consisting of weathering steel framing and cable wire-rope will surround the ponds. The netting will run across the top of the support structure and connect to the chain-link fencing to provide complete protection from birds of prey. The netting and cable system has a design sag of 5’ under its self-weight. A counter-weight system will be provided which allows the netting to deflect in the event of a snow load greater than 1.25 psf. The system is designed so that when the counter-weight rises, the net sags and hits the water surface which thereby knocks off any accumulated snow causing the net to again rise to its original position of 5’ of sag. A situation in which the accumulated snow does not become dislodged by the counter-weight system could arise. In this situation, operations staff will need to physically knock this material off with a pole.

8.7 Chinook Incubation Building

The Chinook Incubation Building is fully enclosed, measuring approximately 63 feet by 53 feet with a 12-foot by 10-foot electrical room attached to the south corner. The main building and electrical room both have an eave height of 15 feet. The building is a pre-engineered metal building with insulated metal panels. All exposed steel surfaces of the building will be hot dip galvanized. The building houses incubation vessels and tray storage. Flooring will consist of a 6-inch concrete slab. The foundation system consists of cast-in-place (CIP) reinforced concrete stem walls and spread footings. The interior column loads are transferred to the soil through square spread footings.

8.8 Spawning Building

The Spawning Building is the smallest of three buildings on the Project. The building consists of a fully enclosed portion measuring approximately 37 feet by 27 feet and a roof-only portion measuring approximately 10 feet by 27 feet. The roof of the fully enclosed building continues over the roof-only portion for a seamless transition. The enclosed portion of the building houses various worktables used for collecting eggs from adult salmon. Flooring will consist of a 6-inch concrete slab. The foundation system consists of cast-in-place (CIP) reinforced concrete stem walls and spread footings for the enclosed portion, and two individual column footings for the roof-only portion. The interior column loads are transferred to the soil through square spread footings. The roof-only portion will exhibit a limestone surfacing and provide shelter for the electro-anesthesia (EA) tank and hatchery workers.

8.9 Adult Holding Ponds

The adult holding ponds are located directly adjacent to the roof-only portion of the Spawning Building. The holding ponds will consist of four ponds measuring approximately 70 feet by 12 feet. The ponds will consist of 8-inch cast-in-place reinforced concrete walls with embedded stainless guide slots for new aluminum fish screens and new aluminum dam boards, and a 2-foot-wide FRP walkway on top of all interior walls. Hinged sections of grating allow access to the guide slots underneath. Jump prevention netting will be provided at all interior walls along the walkway to prevent fish from jumping between ponds. Floor diffusers located at the north end of the ponds provide an obstacle-free path on that side of the ponds. For egg collection, hatchery workers can crown the fish to the north end of the sorting pond into a hoist that will lift the fish into the EA tank.

Chain-link fencing around the perimeter of the adult holding ponds ties into the Spawning Building and will prevent large predators from entering. A predator netting support structure consisting of stainless steel HSS and cable wire-rope will be mounted to the top of the exterior walls. The netting will run across the top of the support structure and connect to a cable running along the top of the walls to provide protection from birds of prey. There will be some small openings in the netting along the southern side where the netting crosses the ponds.

8.10 Meter Vault

The meter vault will house various flow meters and mechanical valves for the intake piping for the Project. The vault will consist of cast-in-place reinforced concrete slab, walls, and roof with an aluminum access hatch measuring 8 feet 13 feet. The inside dimensions of the vault are approximately 13 feet by 15 feet. Due to the close proximity to Fall Creek, the meter vault will need to be designed to resist buoyant forces due to water pressure beneath the slab. This will be accomplished with a thickened slab to add weight to the overall structure.

8.11 Fish Ladder

The fish ladder structure connects the adult holding ponds to Fall Creek downstream of the facility. Adult salmon will travel up the fish ladder and be sorted into the various ponds during spawning season. The fish ladder consists of CIP reinforced concrete with timber Denil-style baffle sections which slide into plain concrete guides. The guides extend to the top of the walls so that the baffles can be placed from the top of the ladder walls. Embedded stainless steel guides at the entry of the fish ladder will house a removable aluminum bar screen which can be installed to prevent fish from entering the facility during certain times of the year.

8.12 Temporary Picket Barrier

The temporary picket barrier prevents fish from travelling farther upstream Fall Creek and directs the fish into the Denil fish ladder. The barrier is removeable and will only be in place during spawning season. The barrier consists of three separate panels weighing approximately 60 lbs each. Each panel consists of an aluminum HSS A-frame which is bolted to concrete embed tabs. Forty-six (46) aluminum rods are then individually placed into pre-cut holes in the A-frame and rest on the concrete apron. The panels can be set in place in their location in the channel in a relatively short amount of time due to their light weight and simple design. A CIP reinforced concrete apron measuring approximately 8 feet by 17 feet will serve as a uniform sill surface for the temporary barrier to sit on. The apron will span between CIP reinforced concrete retaining walls at each bank.

8.13 The Fish Release Pipe Support

The volitional release of fish from the Chinook Raceways occurs through a 14” PVC pipe that discharges into Fall Creek. The pipe extends approximately 16 ft – 6 inches from its daylight points out into the canal. A concrete piling pipe support has been designed to support the pipe in the canal. The piling consists of a 1 ft – 6 inch diameter concrete piling with a 4 ft square footing. The pipe rests on a UHMW saddle which allows temperature expansion/contraction of the PVC piping.

9.0 Mechanical Design

9.1 General Description

This section presents a narrative description of the mechanical elements at each of the Project facilities and provides details on the mechanical design of each component.

9.2 Intake Structure

The mechanical components of the intake structure include debris screens and pumps, a sluicing gate, isolation valves, vacuum breaker valves, and flow meters. The design, sizing, and operation of these components are discussed in the following subsections.

9.2.1 Debris Screens

The debris screens at the intake of the hatchery will consist of two vertically oriented traveling screens located in guide slots immediately upstream of the hatchery supply piping inlet. The debris screens will serve to filter out larger debris and detritus from entering the facility to minimize the risk of clogging small piping and valves. The screens will have 1-inch clear openings and will be mobilized such that any debris captured on the upstream face is lifted out of the water to a spray wash system, where any material caught on the screen will be dislodged and fall into a debris trough. The debris trough will rest on the operator's platform atop the intake structure and will be cleaned out periodically by operations and maintenance staff.

The screen and spray wash system can have three different modes of operation:

- The screen and spray wash may be set to automatically operate at time intervals defined by hatchery personnel, based on site experience.
- The screen and spray wash may be set to automatically operate when a set head differential is measured across the screen by the surrounding level sensors.
- The screen and spray wash may be set by manual actuation, as necessary, by hatchery personnel.

The spray wash will consist of a pump and piping system that draws water from the downstream side of the screen and conveys it to a spray bar with nozzles that will extend across the screen above the debris trough. It is expected that when the spray wash system is engaged, there will be some minor losses to evaporation and aberrant sprays, but these losses are expected to be minimal.

9.2.2 Intake Sluice Gate

As flow passes over the concrete lip at the entrance of the intake structure, some debris is anticipated to settle out of the flow immediately upstream of the debris screens. A cast iron sluice gate with self-contained frame will be located on the upstream face of Dam A, intended to discharge any collected debris from the intake structure though a new 1-foot square penetration through the dam. This gate is anticipated to be normally closed and opened via a handwheel-actuated rising stem by hatchery personnel as part of routine maintenance activities.

9.2.3 Isolation Valves

Immediately downstream of the intake structure the intake piping branches into four individual supply pipes and enters a metering vault. Within this vault, each pipe will be provided an isolation gate valve to allow shutting off flow to any of the structures within the hatchery. The valves are anticipated to be normally open and are intended to be closed during major maintenance activities or whenever a complete dewatering of the facility is required. Each valve will be a flanged, ductile iron, resilient seated gate valve with a manual 2-inch square nut actuator.

9.2.4 Air/Vacuum Valves

An air/vacuum valve will be located downstream of the isolation valves within the valve vault on each supply pipeline. These valves will allow air to be released from the pipeline during initial filling and prevent vacuum formation within the line during a dewatering event. The combination air release/vacuum breaker valve is anticipated to be 2-inch diameter, of cast iron construction, and located at the crown of each supply pipeline. Each Air/Vacuum Valve will be equipped with an isolation ball valve to shut off the air valve if the pipe is being dewatered or cleaned using pressurized air.

9.2.5 Flow Meters

Each supply line will be equipped with an inline magnetic flowmeter for reliable flow measurement to each structure in the hatchery. The flowmeters will be located a sufficient distance upstream of the isolation valves to minimize flow disturbance and ensure accurate flow measurement readings. Each meter will be of steel or cast-iron construction and contain a polyurethane liner. The flow meters will be sized based on the design criteria shown in Table 9-1.

Table 9-1. Flow Meter Design Criteria

Equipment ID	Description	Flow Range (GPM)	Accuracy
FE-200	Coho Building Supply	0 - 1000	±5%
FE-201	Adulting Holding Pond Supply	0 - 4500	±5%
FE-202	Chinook Rearing Supply	0 - 4500	±5%
FE-203	Chinook Incubation Supply	0 - 750	±5%

9.2.6 Vault Sump

To allow for collection and removal of any leakage or infiltration of water into the metering vault, a sump will be provided with single sump pump. The sump pump will be actuated based on a level sensor within the vault, operating periodically to remove any water accumulation.

9.3 Dam B

The mechanical components at Dam B include the sluicing pipe, shear gate, and flap gate. The design, sizing, and operation of these components are discussed in the following subsections.

9.3.1 Sluicing Pipe

To allow for the flushing of any accumulated debris/sediment upstream of Dam B, a sluice pipe will be located through the dam and velocity apron foundation. This pipe will be 8” diameter and contain a cast iron shear gate on the upstream face. To prevent fish from entering the sluice pipe downstream of the velocity barrier, a cast iron flap gate will be located to allow free flow during discharge operations while preventing entry when the pipe is not in use.

9.4 Coho Building

The mechanical components within the Coho Building include the rearing raceway banks, incubation head tank, incubation working vessels, feeding vessels, waste drain system, plumbing system, and building HVAC. The design, sizing, and operation of these components are discussed in the following subsections.

9.4.1 Rearing Raceways

Two sets of raceways exist within the Coho Building:

- A pair of existing raceways, located outdoors underneath the building awning, and;
- A pair of new raceways located within the building structure

Each raceway will contain segmented bays for varying the allocated space requirement of the juvenile Coho salmon. The bays will be separated by the removable aluminum fish screens currently in use at the Iron Gate Hatchery facility. To facilitate use of the existing fish screens, piers will be installed down the centerline of each raceway allowing for two 5 foot -3/8-inch screens to be inserted and removed by hatchery personnel.

At the head of each raceway, flow is controlled with a 4-inch PVC ball valve, manually throttled to achieve the desired flow rate. At the downstream end of each raceway, flows pass over a dam board weir, set to a height required to achieve necessary flow depth for fish rearing. An aluminum stop gate is located at the inlet to the drainage piping, which shall be installed to divert flow through the fish release pipe during volitional fish releases to Fall Creek.

9.4.2 Coho Incubation Head Tank

Incubation stacks will be re-used from the Iron Gate Hatchery to facilitate Coho egg incubation. The incubation head tank/stack design will consist of an aluminum tray stand with adjustable feet supporting six stacks of eight trays. Approximately 5 gpm will be supplied to each stack through a head trough, with a 1-inch PVC ball valve at each stack used for flow regulation and isolation purposes. The head trough will be supported from the wall of the Coho Building and will be equipped with an overflow standpipe, providing a constant head for easier adjustment of the flow rate into each stack.

9.4.3 Coho Incubation Working Vessels

Existing fiberglass tanks will be re-used from the Iron Gate Hatchery as working vessels for the Coho incubation area. These vessels are anticipated to be used for egg picking and enumeration purposes. A 3-inch ball valve will be provided at the head of each working vessel for flow regulation and isolation purposes. Flow will be drained through a removable standpipe at the downstream end of each vessel.

9.4.4 Coho Feeding Vessels

Four feeding vessels will be located within the Coho building, two of which are re-used from the Iron Gate Hatchery, and two will be newly fabricated for the Fall Creek Hatchery. The new feeding vessels will be of fiberglass construction with a width of 5 feet 1 inch and a length of 20 feet. The feeding vessels will be segmented into quarters, with fish screen slots to facilitate insertion of the existing aluminum fish screens from the Iron Gate Hatchery. Flow will be regulated by a 3-inch PVC ball valve at the upstream end and drained by a removable standpipe at the downstream end.

9.4.5 Waste Drain System

A waste drain system will be provided within the Coho Building and adjacent to the outdoor raceways to facilitate removal of fish fecal matter and uneaten food from the ponds. The waste drain system will consist of 2-inch-diameter pipe protrusions from the floor with a stainless-steel cam locking-type quick disconnect for attaching a waste removal vacuum attachment during regular cleaning cycles. All waste will be conveyed through this piping to the settling pond, where it will be collected and removed from the facility. Note that the maximum capacity of the settling pond is approximately 200 gpm. The waste vacuum systems are designed such that this will not be exceeded, however if flow from the Chinook Incubation Stacks (170 gpm) is being diverted to the settling pond, waste drain vacuuming should not occur simultaneously.

9.4.6 Plumbing System

Non-potable utility water will be provided within the Coho Building to supply washdown water through numerous hose bibs located internally and externally throughout the structure. A booster pump will tap off the adult holding pond supply line to fill and pressurize two 80-gallon hydropneumatic tanks located at the eastern corner of the building. The hydropneumatic tanks are anticipated to provide a flow at a relatively constant pressure to the hose bib system located throughout the building.

9.5 Chinook Rearing Area

Mechanical design elements at the Chinook rearing area consist of components within the Chinook rearing raceways and the waste drain system.

9.5.1 Rearing Raceways

Eight raceways are provided for the rearing of Chinook salmon. Each raceway will contain segmented bays for varying the allocated space requirement of the juvenile fish. The bays will be separated by the removable aluminum fish screens currently in use at the Iron Gate Hatchery facility. To facilitate use of the existing fish screens, piers will be installed down the centerline of each raceway allowing for two 5 foot-3/8-inch screens to be inserted and removed by hatchery personnel.

At the head of each raceway, flow is controlled with a 6-inch butterfly valve, manually throttled to achieve the desired flow rate. At the downstream end of each raceway, flow passes over a dam board weir, set to a height required to achieve necessary flow depth for fish rearing purposes. Additional dam board slots are provided upstream of the fish release and drain pipelines for diversion of flow during volitional release operations.

9.5.2 Waste Drain System

A waste drain system will be provided around the Chinook rearing raceways to facilitate removal of fish fecal matter and uneaten food from the ponds. The waste drain system will consist of 2-inch-diameter pipe protrusions from the floor with a stainless-steel cam locking-type quick disconnect for attaching a waste removal vacuum attachment during regular cleaning cycles. All waste will be conveyed through this piping to the settling pond, where it will be collected and removed from the facility.

9.6 Chinook Incubation Building

The mechanical components within the Chinook Incubation Building include the incubation head tanks, incubation working vessels, plumbing system and building HVAC. The design, sizing, and operation of these components are discussed in the following subsections.

9.6.1 Chinook Incubation Head Tank

Incubation stacks will be reused from the Iron Gate Hatchery to facilitate Chinook egg incubation. The incubation head tank/stack design will consist of an aluminum tray stand with adjustable feet supporting 17 stacks of eight trays. Approximately 5 gpm will be supplied to each stack through a head trough feeding back to back rows of incubation trays (34 stacks total), with a 1-inch PVC ball valve at each stack used for flow regulation and isolation purposes. The head trough will be equipped with an overflow standpipe, providing a constant head for easier adjustment of the flow rate into each stack. The Chinook Incubation Building will house four back-to-back rows of incubation trays, for a total of 136 incubation tray stacks.

Each tray will discharge into a drainage trench located within the concrete underneath the centerline of each head tank. The end of the drainage trench will contain two 8-inch-diameter standpipes, one leading to the adult holding ponds (drain) and the other leading to the settling ponds (waste drain). During normal operations, the water will be directed into the drain directing flow to the adult holding ponds. Hatchery personnel will have the option of pulling the waste drain standpipe under one row of 34 stacks at a time and diverting all flow to the settling pond during cleaning operations. Note that if more than one row is diverted through the waste drain simultaneously, the capacity of the settling ponds will be exceeded.

9.6.2 Chinook Incubation Working Vessels

Existing fiberglass tanks will be reused from the Iron Gate Hatchery as working vessels for the Chinook Incubation Building. These vessels are anticipated to be used for egg picking and enumeration purposes. A 3-inch ball valve will be provided at the head of each working vessel for flow regulation and isolation purposes. Flow will be drained through a removable standpipe at the downstream end of each vessel.

9.6.3 Plumbing System

Non-potable utility water will be provided within the Chinook Incubation Building to supply washdown water through numerous hose bibs located internally and externally throughout the structure. A booster pump will tap off the adult holding pond supply line to fill and pressurize two 80-gallon hydropneumatic tanks located at the southern corner of the building. The hydropneumatic tanks are anticipated to provide a flow at a relatively constant pressure to the hose bib system located throughout the building.

9.7 Spawning Building

Mechanical design elements within the Spawning Building include the electro-anesthesia tank, egg rinse/water hardening stations, conveyor belt, and building plumbing.

9.7.1 Electro-Anesthesia System

An electro-anesthesia system will be located at the head of the trapping/sorting pond for the purposes of anesthetizing fish for sorting and spawning purposes. This device is an existing element that will be reused from the Iron Gate Hatchery. Fish are deposited into the electro-anesthesia tank from the existing fish crowder on the trapping and sorting pond, where they are sedated or euthanized, depending on the operation being performed. The electro-anesthesia tank is additionally equipped with a separate hydraulic hoist where fish are raised and deposited on a sorting table for further processing.

9.7.2 Egg Rinse/Water Hardening Station

An existing egg rinsing table and water hardening table will be relocated from the Iron Gate Hatchery to the Spawning Building. Both units will be located against the northeastern wall of the structure and provided with water from the adult holding ponds supply line. Water is discharged through the tables into a drainage trench where it is drained to the settling pond.

9.7.3 Conveyor Belt

The existing motorized conveyor belt at the Iron Gate Hatchery will be relocated to the Spawning Building. The conveyor belt contains multiple sections and may be connected to an approximate 100-foot length. This system is primarily intended to be used for transporting fish carcasses to a collection bin located outside the northern wall of the structure.

9.7.4 Plumbing System

Non-potable utility water will be provided within the Spawning Building to supply washdown water through numerous hose bibs located internally and externally throughout the structure. A booster pump will tap off the adult holding pond supply line to fill and pressurize two 80-gallon hydropneumatic tanks located at the eastern corner of the building. The hydropneumatic tanks are anticipated to provide a flow at a relatively constant pressure to the hose bib system located throughout the building. One hose bib shall be located on a retractable hose reel above the holding table to provide washdown water and a wetted surface during fish sorting/spawning operations.

9.8 HVAC Design

9.8.1 Winter Heating

The Coho Building, Chinook Incubation Building, and Spawning Building heating systems will consist of four wall mounted electric resistance heaters and single downflow electric unit heater located in the middle of the building. The downflow electric heater will be used to provide additional heating if the four wall mounted heaters are unable to meeting the heating requirements of the space. The downflow heater will also be utilized during power outages to provide on demand heating to the building space as needed by the operator. During a Standby Power condition time relays in the 4 wall mounted unit heaters will sequence the startup of each individual heater so that one heater will power up at a time so that only one heater is started at a time. Supplemental spot heating will be provided by electric radiant heaters at the locations recommended for personnel comfort.

9.8.2 Building Fresh Air Requirements

Fresh air ventilation will be provided by the use a single inline fresh air fan and louver in each building. The fan will provide continuous ventilation through the year. Wall mounted occupancy sensor will trigger the fresh air fans on during building operation. During unoccupied mode the fresh air fans will turn off to conserve energy. A wall mounted pressure relief damper will allow excess air pressure to escape the building and prevent over pressurization. The fresh air requirements for each building will be per American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) 62.1-2019.

9.8.3 Summer Cooling

The Coho Building and Chinook Incubation Building cooling systems will consist of two wall-mount propeller fans with two fresh air louvers that will provide free air cooling. The fan flow rate is designed for six air changes per hour to minimize condensation build-up and provide air circulation through the building space. The wall-mount fans will be controlled by a wall mounted speed controller to allow the operator to select between a low and high setting fan speed.

The Spawning Building's cooling system will consist of a single wall-mount propeller fans with a fresh air louver that will provide free air cooling. The fan flow rate is designed for six air changes per hour to minimize condensation build-up and provide air circulation through the building space. The wall-mount fans will be controlled by a wall mounted speed controller to allow the operator to select between a low and high setting fan speed.

The electrical room located within a separate room attached to the Chinook Incubation building will require cooling. The cooling system will consist of a 1-ton mini split wall-mount unit and condenser unit. The condenser unit will be mounted on a small support stand to protect it from snow and water build-up. The electrical equipment heat output in the room is anticipated to be 2.5 kW. Mechanical heating will not be required due to the high heat output of the electrical equipment in the room.

9.8.4 California title 24 Energy Compliance Requirements

The electrical resistance heaters are compliant with the California energy compliance title 24 requirements due to Exception 5 to section 140.4(g). Exception 5 section 140.4(g) states: Where an electric resistance heating system serves an entire building that is not a high-rise residential or hotel/motel building; has no mechanical cooling; and is in an area where natural gas is not currently available.

9.9 Sorting/Trapping Ponds

Mechanical design elements at the Sorting and Trapping Pond include the finger weir trap and fish crowder and lift system.

9.9.1 Fish Crowder/Lift

The existing fish crowder/lift will be transferred from the Iron Gate Hatchery for use at the sorting and trapping pond at Fall Creek Hatchery. ASCE 25-lb rails will be provided atop the walls of the raceway for guiding the unit. Modifications to the crowder will be performed by the contractor prior to transferring the device to the new facility to ensure it will suit the dimensions of the ponds at the Fall Creek Facility.

9.9.2 Finger Weir

At the discharge of the trapping and sorting pond, a finger weir will be installed on a new aluminum weir gate for trapping adult salmon that have traveled up the fish ladder and into the pond. This finger weir gate will allow for adjustable head in the pond, controllable by a handwheel actuator atop the pond wall. The finger weir contains adjustable fingers for modifying the angle to optimize fish trapping or prevent fish from entering the pond.

10.0 Electrical Design

10.1 Utility Power Service

Power from a locally available source will need to be conveyed to the site. The nearest power source would be from the three-phase power utility lines to the east owned by PacifiCorp. The distance from the existing utility lines to the proposed site is approximately 520 feet. The installation contractor will coordinate with PacifiCorp to provide a new power utility service drop for the site location. The service voltage required is 480 volt, three-phase power, connected in wye-ground configuration. Load calculations place the service transformer size at a minimum of 225 kVA. Service equipment will be located at the Chinook Incubation Building due to its proximity to the utility power alignment, with the utility metering equipment installed on the exterior wall of the electrical room for ease of access.

10.2 Facility Power Distribution

Due to the presence of splashing and spraying water in the incubation and spawning rooms at each building during normal operations, electrical equipment has been located either in the electrical room of the Chinook Incubation Building or exterior to each building. The Chinook Incubation Building will house the majority of electrical equipment in a dry electrical room. The Chinook Incubation Building will subfeed the Coho Building and Spawning Building, with the Intake Structure and Meter Vault subfed from the Coho Building. The general distribution arrangement for the majority of loads at each building will consist of a 480V, three-phase panel, a step-down transformer, and a 208V/120Y, three-phase panel. The 480V panelboards will serve the large motor loads and HVAC equipment, while the 208/120V panelboard will serve lights, convenience receptacles, instrumentation, SCADA, and small HVAC and motor loads as required. Detailed load calculations are included in the panel schedules on the drawings. Local disconnects will be provided in accordance with the NEC for equipment that is not within sight of the panelboard feeding it, either integral to control equipment or with a dedicated safety switch. Additionally, a step-down transformer and 240/120V, single-phase panel will be provided to feed the tagging trailer and fish pump power receptacles, which require 240V to operate at 100A and 60A respectively.

As stated above, two sets of power receptacles will be provided near the Coho and Chinook raceways for use by the CDFW-owned tagging trailer and fish pump. Power distribution is sized assuming that only one tagging trailer and one fish pump can be used at a time, as discussed with CDFW. Power receptacles have also been provided around the Coho and Chinook raceways and the holding ponds to provide power to a portable waste drain pump. Similarly to the other power receptacles, power distribution is sized assuming that only two portable waste drain pumps would be in use throughout the facility at a time. No power receptacle has been provided for settling ponds, as those will be pumped periodically using a truck.

Starters will be provided for each exhaust fan in each building. Fan starters will either be manual or magnetic type as shown, have a red pilot light, and open the motorized louver related to the fan when the fan is turned on. The duct ventilation fan starters will have provisions for automatic control by an occupancy switch supplied by the HVAC contractor. The walls fans will have a simple adjustable speed controller supplied by the fan supplier. All heaters will be controlled by thermostats and time delay relays supplied by the HVAC contractor. The split unit in the electrical room will be controlled by a thermostat supplied by the HVAC contractor. The meter vault exhaust fan will be controlled by an on-off switch.

10.3 Propane Standby Generator

The existing 100 kW Kohler generator set has been assessed for reuse at this facility to provide standby power to all critical loads for the facility. Generator sizing calculations were performed using Kohler's generator sizing software, and are included in Appendix E. As the existing generator is a recent model, this sizing calculation represents a reasonable approximation of its characteristics. This design includes methods for automatic load-stepping of equipment starting and disabling/ignoring non-essential loads during a power outage to avoid procuring a larger generator. In order to meet this requirement, a manual transfer switch will be provided, and the below operation sequence for power outages in Table 10-1 is anticipated.

Table 10-1. Anticipated Standby Power Operation Sequence

Step No.	Description
(1)	Upon power outage, travel to site.
(2)	Disable all Spawning Building equipment that may have been active at the time of the outage (e.g. electro-anesthesia unit, conveyor, fish crowder, etc.), if any.
(3)	Disable the waste drain wet well sump control panel.
(4)	Disable all radiant heater loads for all buildings. Unit heaters and exhaust fans are not required to be disabled.
(5)	Start generator from transfer switch. When generator is ready, initiate manual transfer of facility to standby power.
(6)	The traveling screen system, SCADA cabinet, split unit, lighting, and outlets will turn on immediately. Verify traveling screens are operating. Inline duct fans will operate as usual. Exhaust fans will turn on if left in on position.
(7)	Unit heaters will automatically heat to the t-stat setting after a pre-set time delay. The meter vault sump pump will operate normally.

The generator will be designed to run on liquid propane (LP) stored in an on-site tank. This design anticipates reusing the existing propane tank currently suppling the existing generator above. It is assumed that this tank will provide a minimum of 24 hours' worth of power.

10.4 Lighting Design

High bay lighting on dimmer switches will be provided at each building, and switched lights will be provided above building exterior doors and at the intake structure for maintenance purposes. Lighting will conform to the requirements of the California Energy Code (CEC) and will be exclusively LED-based fixtures. Excluding the electrical room, interior lighting will be provided primarily by skylight refraction tubes during the day, with high bay fixtures providing supplementary illumination to each building during night operations and other times when natural light is limited. The electrical room lighting will be provided with dimming control down to 5% to meet CEC requirements. Lighting level calculations for each room have been provided under Appendix E.

The underlying design assumption for each building is that high intensities of light (88 ft-c and greater) will act as a lethal agent to Coho and Chinook salmon eggs, as found by Eisler (1958). Further, dimmable lighting levels may be desirable to the facility operators to limit adult and juvenile salmon exposure to light to a natural, circadian schedule. Under those assumptions, both the skylight refraction tubes and high bay fixtures will be controlled by manual dimmer switches to allow the operators to dim lighting as much as necessary to prevent premature egg mortality, but also provide lighting necessary for natural salmon growth rates. Preliminary lighting levels for the Coho and Chinook Incubation Buildings are designed to provide 40 ft-c on average from skylight refraction tubes and 20 ft-c on average from high bay lighting. For the Spawning Building, both skylight refraction tubes and high bay lighting levels are designed to provide 20 ft-c on average. The lighting fixtures as specified will allow dimming down to 10 percent illumination for the high bays, and 2 percent for the skylight refraction tubes. Options for further dimming are available, but not currently included in the design. No lighting occupancy sensors, photocell control, or other intelligent lighting control are planned for the facility.

11.0 Instrumentation and Controls

11.1 General Description

All instrumentation and controls will be mustered to a single SCADA cabinet located in the Chinook Incubation Building electrical room. The SCADA cabinet will house a PLC, an HMI, a UPS, an ethernet switch, and alarms, relays, terminal blocks, and other components required for a complete system. There will be no remote control of the facility; all subsystems will be controlled locally through manual or sensor-based actuation.

PLCs used in the Project will be Emerson, Allen Bradley, Schneider Electric, or equal models. The SCADA cabinet will have a UPS to maintain operability of critical monitoring functions at the fish hatchery for a short duration, with the on-site standby generator providing up to 24 hours of backup power to the facility. In the event of a primary PLC failure, the facility will alert operators of the loss through SCADA.

Telemetry communication for system visibility to the operators will be achieved using an automatic cellular alarm dialer (autodialer). The autodialer will call site operators when an alarm occurs and will allow for multiple sequential alarm dial-out numbers and alarm acknowledgement from remote phones. The autodialer will be equipped with automatic battery backup, in addition to being backed up by the standby generator. Additionally, an alarm siren will provide local annunciation of alarm conditions to the site. Provisions for interconnecting future communications with remote systems have been included in the design.

The water surface elevation sensors will be submersible pressure transducers in stilling wells. The raw water flowmeters will be magnetic, inline type, as described above in Section 9.2.5. The level switches will be the float type. Intrusion switches will be standard magnetic type with normally closed contacts.

11.2 Intake Structure

The traveling screen vendor will supply a control panel for control of the screens and screen spray pumps. The traveling screens and spray wash pumps will be controlled locally from the vendor-supplied control panel only, either automatically or manually as described above in Section 9.2.1. The control panel will include a main breaker, a microcontroller, provisions for level control and remote level annunciation, and alarms, relays, terminal blocks, and other components required for a complete system.

A sump pump control panel will be provided by the meter vault sump pump supplier for pump control. It will include a lockable, outdoor enclosure, an alarm beacon, and provisions for automatic level control and remote level alarm annunciation.

Instrumentation at the Intake Structure will consist of intake water surface elevation sensors (for measurement of differential pressure across the screen), raw water supply piping flowmeters located in a vault, level switches in the meter vault, and a vault intrusion detection switch.

11.3 Coho Building

Instrumentation at the Coho Building will consist of a level switch in the incubator head tank and door intrusion detection switches. Status I/O points will be sent to SCADA from each of the switches. No other process instrumentation and control are planned for this building.

11.4 Chinook Raceways

Process instrumentation and control are not planned for this feature.

11.5 Chinook Incubation Building

The facility SCADA cabinet and the autodialer will be installed in the electrical room, and will act as the main monitoring equipment for facility operations. This SCADA cabinet will be supplied by the installing contractor. A detailed panel layout, bill of materials, and schematic diagrams have been provided in the drawings for use in the panel fabrication process. The SCADA cabinet will have an HMI that allows status and alarm monitoring – but not control – of plant processes, and LED pilot lights for "general alarm" and "SCADA control power on". Additional alarms and statuses originating inside the SCADA cabinet include "PLC fault", "surge protection fault", "UPS battery on", "UPS low battery", "UPS fault", "ethernet switch fault", and "loss of dc power". A set of spare conduits to the SCADA cabinet and an exterior-located junction box will be provided for future communication connection.

The autodialer will receive a specific set of critical alarms for annunciation over a cellular antenna. These consist of "general system fault", "traveling screen fault", "meter vault flood alarm", "intrusion alarm", "incubation tanks water low", and "wet well high level alarm". A cellular service will need to be set up in order for this system to function properly.

The alarm siren control station will be located on the east exterior corner of the main building and aimed east for alerting the area east of the plant, which is planned for future use as an operator living area. The alarm siren control station will have an audible alarm and silence pushbutton. These will be connected directly to the SCADA cabinet, and all alarm logic for this siren will be programmed at the PLC.

Instrumentation at the Chinook Incubation Building will consist of a level switch in each of the incubator head tanks and door intrusion detection switches. Status I/O points will be sent to SCADA from each of the switches. No other process instrumentation and control are planned for this building.

11.6 Spawning Building

Instrumentation at the Spawning Building will consist of door intrusion detection switches. Status I/O points will be sent to SCADA from each of the intrusion switches. No other process instrumentation and control are planned for this building; however, see the holding and settling ponds section below for additional instrumentation and control requirements in this area.

11.7 Adult Holding and Settling Ponds

CDFW has informed McMillen Jacobs that it is desirable to retain the existing controls and instrumentation for the spawning building and holding pond areas from the existing Iron Gate Fish

Hatchery facility for reuse at Fall Creek. The following controls and instrumentation will be relocated for reuse:

- The spawning area control station, to be installed at the sorting table, and controls the fish crowder drive and lift, the electro-anesthesia hydraulic tank hoist, and the tank fill pump;
- The electro anesthesia unit, control panels, shock paddles, cabling and other associated appurtenances;
- The foot-pedal safety switch for the electro-anesthesia system, to be installed below the sorting table;
- All other controls and instrumentation related to the electro anesthesia tank hydraulic hoist system;
- The motor starter panels for the crowder lift, crowder drive, and conveyor belt;
- Four limit switches for stopping fish crowder lift and drive movement, and;
- The conveyor belt control pendant and instrumentation integrated into the conveyor mechanism;

As the electro anesthesia tank fill pump is being replaced, a new starter control panel will be provided as well. The starter control panel will have an outdoor enclosure, an external disconnect, a combination full-voltage non-reversible magnetic starter, a motor circuit protector, an on-off pushbutton, and a red pilot light. This panel will interconnect with the existing spawning area control station to provide pump control at the sorting table.

The waste drain wet well pumps will be controlled by a local pump control panel supplied by installing contractor. For this control panel, a detailed panel layout, bill of materials, and schematic diagrams have been provided in the drawings for use in the panel fabrication process. The waste drain wet well pumps will be controlled locally from the control panel only, either manually or automatically based on a lead/lag level switch alternating pump control scheme. The pump control panel will contain a main breaker and disconnect, two combination full-voltage non-reversible magnetic starters, two motor temperature and leak protection relays, a phase monitoring relay, a duplex alternating lead-lag relay, and alarms, relays, terminal blocks, and other components required for a complete system.

Instrumentation at the holding and settling ponds will consist of the foot-pedal safety switch for the electro-anesthesia unit and waste drain wet well level switches. Status I/O points will be sent to SCADA from the wet well pump control panel and the wet well alarm high level switch. The safety switch will be used for local shutoff of the electro-anesthesia unit only. No other process instrumentation and control are planned for this feature.

12.0 Operations

12.1 General Description

The following subsections discuss general operations of the Fall Creek Hatchery. The information is intended to be high-level for this design phase and will be further defined through discussions with KRRC and CDFW in future design phases.

12.2 Water Distribution and Collection Systems

The intake located at Dam A for the Project is intended to operate autonomously, with self-cleaning screens set to initiate a cleaning cycle based on pre-set head differential or time interval. Debris removed from the screens will be collected in a trough, which will require occasional removal by hatchery personnel. The isolation valves on each of the four (4) supply pipelines are intended to be normally open, with all flow being controlled in the downstream distribution systems.

Supply piping will generally be operated by valves located at each of the raceways, vessels, or working spaces. Flows through each of the supply pipelines will be monitored by the flow meters located in a below grade vault with flow rate estimates transmitted to the PLC. To maintain the 10 cfs water right, the PLC will be programmed to alert hatchery personnel if the water right is exceeded. There has been a 0.5 cfs contingency built within the FCFH bioprogram to ensure that the water right is not exceeded while hatchery production goals are achieved.

Flow to individual rearing raceways or vessels will be adjusted by operating the supply manifold valve and estimating flow at the overflow discharge. The production drain piping system will simply convey the rearing raceway and vessel drain flows to the adult holding ponds. There are no control valves on the drain piping system. Clean-outs have been provided on all pipelines throughout the facility to allow hatchery staff to flush the pipelines, as needed, if flow disturbances are observed.

Under typical operations, water will return to Fall Creek after being routed through the drain piping system, through the adult holding ponds and ultimately through the fish ladder downstream of the adult holding ponds.

During times of fish release, water can also return through any of the three (3) volitional release pipes located at the Coho Raceways, Chinook Raceways, or the adult holding pond discharge channel. Stop gates or dam boards shall be placed in front of the raceway drain, diverting all flow through the fish release piping after those respective dam boards have been removed. The volitional release pipes will only be in operation when hatchery staff release fish to Fall Creek throughout the year.

12.3 Waste Management

Waste management will be performed with a vacuum system that discharges to the waste drain system. Quiescent zones will be maintained near the downstream end of the raceways and rearing vessels, where biosolids will settle. Vacuums, as depicted in Figure 12-1, will be used to suction out the solids, and discharge into the waste drain system. The waste drain system will discharge the solids with a transport water flow to the settling pond.

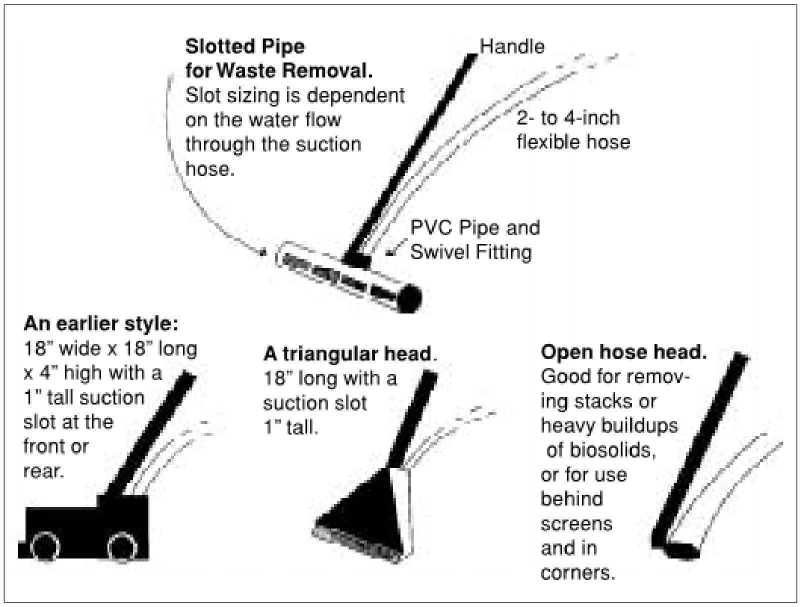

Figure 12-1. Typical Vacuum Removal of Solids (Source: Idaho DEQ, nd)

The settling pond will be partitioned into two sections with the flow from the waste drain system directed to one or the other of these partitions by a valve. One of these subdivisions will collect flows from the upstream cleaning of the ponds, while the water content in the other is allowed to evaporate. Once the drying partition is sufficiently dry, biosolids will be removed and disposed of. The valve will be adjusted to direct flows to the now empty partition, and the water content in the other partition will be allowed to evaporate.

The downstream end of each of the settling pond bays will be equipped with an overflow structure that will divert flow-through water into a pipe that discharges into the fish ladder. The fish ladder will be the primary outfall from the hatchery.

12.3.1 NPDES Sampling

Water quality samples will be required to be sampled at fish ladder downstream of the settling pond discharge location to verify the effluent is within the allowable parameters set by the NPDES permit. CDFW is in the process of negotiating the NPDES permit for the Project. At this design phase, it is assumed that the waste stream from FCFH will be required to meet effluent limitations included in the California Regional Water Quality Control Order No. R1-2015-0009, General NPDES CAG131015, and Waste Discharge Requirements for Cold Water Concentrated Aquatic Animal Production Facility Discharges to Surface Waters. The General NPDES CAG131015 effluent limitations are summarized in Table 2-11. This NPDES design criteria for the Project will be updated once an NPDES permit has been issued for the site.

12.3.2 Treatment of Therapeutants

Another effluent concern for the facility will be the use of therapeutants or inorganics that could occasionally be required for treatment of fish. Use of such therapeutants is not anticipated due to the high quality of the intake water and the short design life of the facility. However, if therapeutants are used for treatment of fish operationally hatchery staff can isolate and direct the flow to the waste drain system and utilize the 3,200 ft^3 of effluent holding provided by the effluent settling pond. While use would be dependent on flow rates supplied to each individual rearing unit, the effluent settling ponds provide short-term storage of up to 24,000 gallons of therapeutant laden flow that could then be pumped to appropriate storage tanks and transferred to approved off-site disposal areas, or discharged to Fall Creek after the required residence time.

12.4 Adult Holding and Spawning

12.4.1 Trapping/Sorting

Adult salmon will be guided to the base of the fish ladder by the fish exclusion picket barrier located adjacent to the holding ponds on Fall Creek. At the head of the fish ladder, adult salmon will pass over a dam board weir and enter the holding pond outflow structure where attractant flows will guide them over a finger weir trap into the sorting/trapping pond. A manual crowding screen will be placed by hatchery personnel to guide fish to the head of the pond and into the fish lift, where they may be hoisted into the electro-anesthesia tank for temporary sedation. Sedated fish will be raised to a sorting table, where adult Chinook are placed in their respective pond through a removable pipe and adult Coho are processed and placed in a separate pond by hatchery personnel.

12.4.2 Spawning

During Chinook spawning operations, the dam boards separating the Chinook holding pond from the sorting/trapping pond will be removed, and a fish screen will be installed in the upper quarter of the trapping pond. The manual fish crowder will be placed by hatchery personnel in the Chinook pond to guide fish into the sorting pond and into the fish lift, where they may be hoisted into the electro-anesthesia tank for sedation. At the sorting table, males and females will be separated and transferred to the holding table within the spawning building. Female salmon eggs will be gathered on the air spawning table, where they will be rinsed, water hardened, and prepared for incubation. If male salmon are to be used more than once during the spawn season, stripped males will be manually returned to their respective rearing containers (raceways for Chinook and spawning tubes for Coho). Fish carcasses will be placed on the conveyor belt and deposited in a collection bin outside, where they will be periodically gathered and processed by hatchery personnel.

12.5 Incubation

Incubation trays are provided in the Coho and Chinook buildings for egg/alevin incubation within the hatchery. Multiple ½-stack incubators (8 trays per stack) are provided in both buildings and hold eggs during incubation, with the water supply provided by a constant head tank feeding each row. Hatchery personnel will be required to perform periodic cleaning of the trays during the incubation period, and working vessels are provided for egg picking and enumeration purposes.

12.6 Juvenile Rearing

Rearing of juvenile salmonids is anticipated to take place in the Coho and Chinook raceway banks. Additionally, the adult holding ponds are provided with dam boards and fish screen slots to allow for juvenile rearing if elected by hatchery personnel. Each raceway contains segmented bays, with the total rearing volume configurable by insertion of removable fish screens. A final screened bay shall be used for initial settling of waste, to be periodically cleaned by hatchery personnel through the waste drain system.

Each raceway bank is equipped with a volitional release piping system, returning juvenile salmon to Fall Creek at the end of the rearing season. Stop gates or dam boards shall be placed in front of the raceway drain, diverting all flow through the fish release piping after those respective dam boards have been removed.

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Appendix A
Hydraulic Design Calculations

Calculation Cover Sheet

Project: Fall Creek Hatchery

Client: Klamath River Renewal Corporation **Proj. No.** 20-024

Title: Hydraulic Calculations - 100% Design

Prepared By, Name: Andrew Leman

Prepared By, Signature: **Date:** 10/28/2020

Peer Reviewed By, Name: Vincent Autier, P.E.; Nathan Cox, P.E.

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10/28/2020

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier/N. Cox
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Hydraulic Calculations - 100% Design	**PROJECT NO.:** 20-024	

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SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Streamflow	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to identify design streamflows throughout the site.

References

• *Gotvald, A.J., Barth, N.A., Veilleux, A.G., and Parrett, Charles, 2012, Methods for determining magnitude and frequency of floods in California, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2012–5113, 38 p., 1 pl., available online only at http://pubs.usgs.gov/sir/2012/5113/.*

• *FERC (Federal Energy Regulatory Commission). 2007. Klamath Hydroelectric Project, FERC Project No. 2080-027, Oregon and California: Environmental Impact Statement. U.S. Dept of Energy: FERC. Washington, D.C.*

• *NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.*

• *CDFW (California Department of Fish and Wildlife). 2004. California Salmonid Stream Habitat Restoration Manual, Vol. II: Fish Passage Evaluation at Stream Crossings. State of California, California Dept. of Fish and Game, Wildlife and Fisheries Division. March 2004.*

• *USGS (U.S. Geological Survey). 2019. Guidelines for Determining Flood Flow Frequency: Bulletin 17C. Version 1.1. U.S. Dept. of the Interior, U.S. Geological Survey, Washington, D.C. May 2019.*

Method

The following design streamflows were identified as necessary for the design of Fall Creek hatchery and appurtenant facilities:

1. **100-year flood -** This information will be used to ensure that facilities are protected against large storm events, and outside of the floodway.

2. **2-year flood -** The 2-year flood is often associated with the bankfull flow condition in natural streams and rivers. This information will be collected for reference in determining bank locations. This also provides a more frequent flooding event that is very likely to be encountered during the life of the facility.

3. **Fish Passage 95% Exceedance -** This is designated as a design flow by NMFS (2011), and represents a low design flow during the period that the barrier, fish ladder, and trap are in operation, and anadromous fish are present at the site.

4. **Fish Passage 50% Exceedance -** This information is collected as a reference value for what would be expected as a typical flow at the site during the period that the barrier, fish ladder, and trap are in operation, and anadromous fish are present at the site.

5. **Fish Passage 5% Exceedance -** This is designated as a design flow by NMFS (2011), and represents a high design flow during the period that the barrier, fish ladder, and trap are in operation, and anadromous fish are present at the site.

6. **Fish Passage 1% Exceedance -** This is designated as the high design flow by CDFW (2004) for stream crossings, and was applied as the high flow design criteria for consistency with other elements of the project as a whole.

7. **Juvenile Release 1% Exceedance -** This was selected as the peak flow month (March) in which juveniles would be released from the hatchery. While it is not typical behavior for them to migrate upstream, the barriers at Dam A and Dam B were designed to preclude passage based on this design flow. The 1% exceedance probability was selected based on CDFW criteria for fish passage (see above).

The following locations of streamflow were identified as necessary for modeling flows in Fall Creek:

1. **Powerhouse Channel -** This reach is fed by flows diverted to the upstream powerhouse and will be the location of the intake for the hatchery as well as the intake for the City of Yreka, at Dam A.

2. **Upper Reach -** This reach is the main branch of Fall Creek, and is fed by a waterfall upstream of Dam B (not shown on Figure 1).

3. **Middle Reach -** Downstream of the confluence of the penstock channel and the upper reach, will be the reach that flows past much of the site including the Copco road bridge, and the fish barrier and trap.

4. **Unnamed Drainage -** This drainage flows toward the southwest past the existing lower pond battery and combines with the main stream of Fall Creek. This is the drainage into which the existing lower raceway battery currently discharges.

5. **Lower Reach -** Downstream of the confluence of the middle reach and the unnamed drainage is the lower reach of Fall Creek that continues on to the Klamath River.

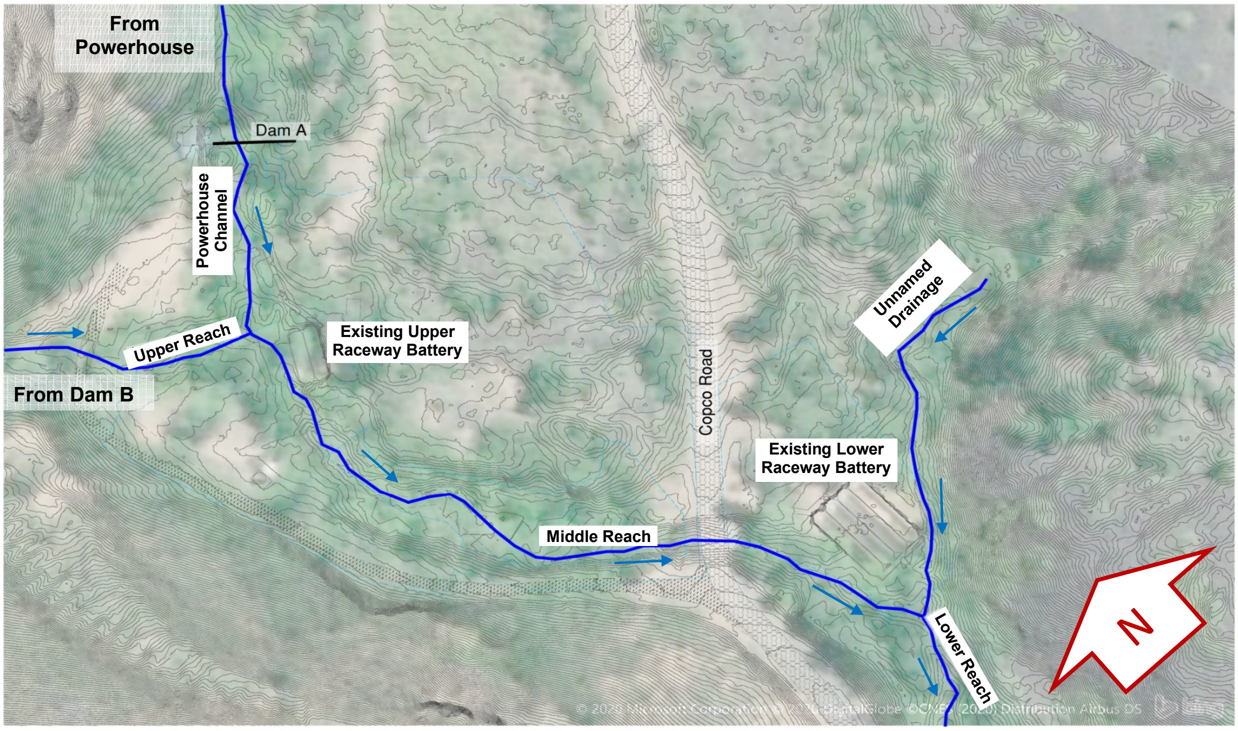

Figure 1. Stream Network Schematic

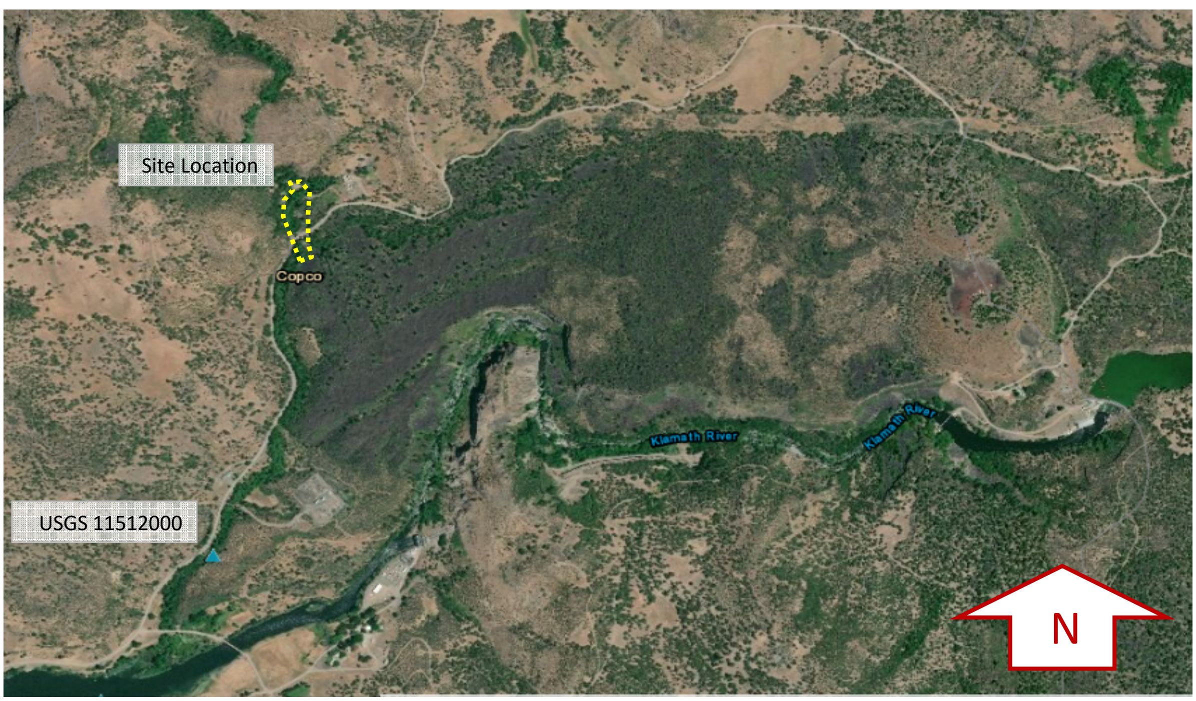

Figure 2. USGS Gage Location Map

The following data sources were identified for evaluation of streamflows at the above locations:

1. USGS Gage Station 11512000 - This gage station is located approximately 2/3 mile downstream from the existing lower raceway bank (see Figure 2), and therefore provides the best representation of flows at the site. The data record consists of daily averaged discharge, and extends from 1933 to 1959, and then from 2003 to 2005. While this does not represent the most recent 25 years (per NMFS, 2011), it is the best available data and does represent a 28 year record.

2. Gotvald et al, 2012 - This report from the USGS provides regional regression relationships by which streamflow can be estimated for ungaged stream locations. This is the method employed by the USGS StreamStats software in the state of California.

3. USGS StreamStats Software - The drainage areas at the points of interest were delineated using the USGS StreamStats software which utilizes the USGS 3DEP (3D Elevation Program) topography.

4. FERC Environmental Impact Statement (2007) - The flows diverted to the Fall Creek powerhouse from Spring Creek and Fall Creek were collected from the FERC environmental impact statement for the Klamath Hydroelectric Project.

The method employed in these calculations will be as follows:

Fish Passage Flows

1. Develop a flow exceedance curve for the downstream gage station 11512000 during the months when fish are present at the site (adults: October - December; juveniles: Mar - May).

2. Determine the fish passage and juvenile design criteria flows (1%, 5%, 50%, and 95% exceedance) from the flow exceedance curve.

3. Adjust the flow rates at the USGS gage to the locations of interest.

a. The regression relationships of Gotvald et al (2012) identify three primary variables of interest to the streamflow: (1) drainage area, (2) precipitation, and (3) elevation. Because of the proximity of the USGS gage to the project site, both precipitation and elevation are expected to be similar. Therefore, the adjustment from the USGS gage station to the project site can be performed based on the ratio of drainage areas. Therefore, the adjustment from the USGS gage station to the project site will follow the equation:

$$Q_{site} = Q_{USGS}\left(\frac{A_{site}}{A_{USGS}}\right)$$

b. In the case of the powerhouse channel, flows are dictated by the diversion to the powerhouse and therefore are human-influenced more than based on a natural regime. Furthermore, the withdrawals by the City of Yreka will be variable and unknown.

c. Therefore, an estimation of the division of the middle branch flows is required between the upper reach and powerhouse channel flows. A constant flow was applied to the powerhouse channel that is equal to the minimum flow requirement (15 cfs) downstream of Dam A. The following should be noted when considering this assumption:

i. There is relatively little contributing area to upper reach drainage and it will therefore be primarily human-influenced.

ii. The barrier located at Dam A will be designed for the full range of anticipated powerhouse flows (15 cfs - 50 cfs). All other in-stream design points are either in the adjacent drainage or well downstream of this point, and impacts to the stream model from this assumption will be limited.

iii. For flooding evaluation, the remainder of the flow will be contributed from the Upper Reach of Fall Creek, which meets up with the powerhouse channel near the existing upper pond battery. There will be no infrastructure (with the exception of the intake) upstream of this location, and therefore the flooding limits will not be unduly influenced by this assumption.

Flooding Flows

1. Collect peak flow statistics from the USGS StreamStats online software for the USGS gaging station 11512000.

2. Adjust the flow rates to the project location based on drainage area, according to the drainage area scaling discussed above.

a. The same assumption with respect to the Fall Creek upper reach will be made as for the fish passage flows.

Calculations

Fish Passage Flows

Data collected from USGS Station 11512000 was processed to eliminate all data that was not approved for published use, and was limited to the months of October through December (adult fish present at the site). This is summarized in the exceedance curve below:

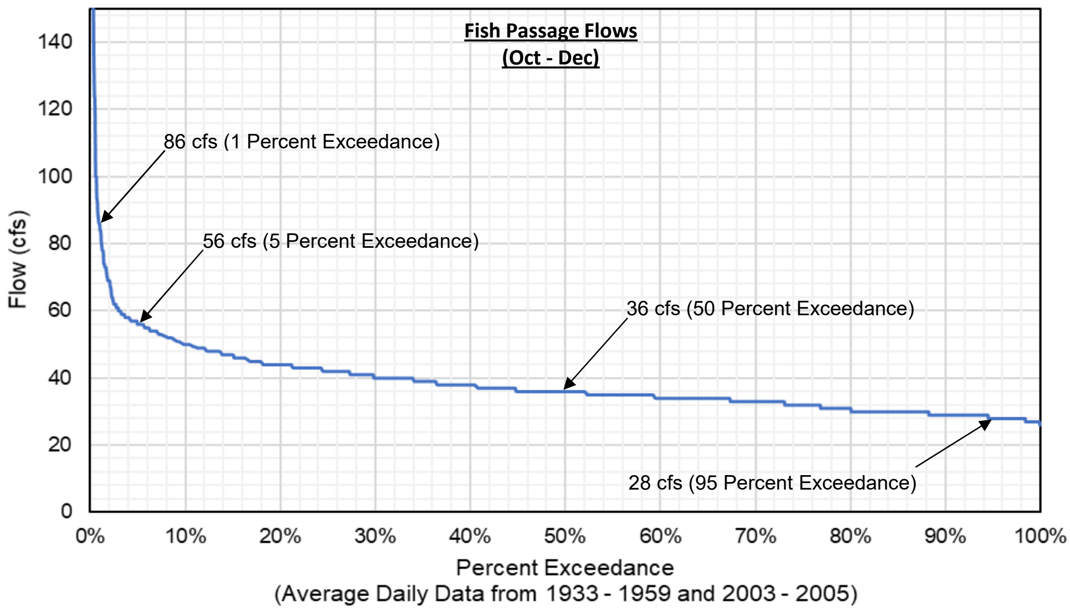

Figure 2. Exceedance Curve for USGS Station 11512000 (October - December)

Exceedance Criterion	Flow (cfs)
1% Exceedance	86
5% Exceedance	56
50% Exceedance	36
95% Exceedance	28

Drainage areas were collected from StreamStats for each of the points of interest and for the USGS gage station:

Location	Drainage Area
	mi^2
USGS Gage Station	14.6
Powerhouse Channel	0.1
Upper Reach	12.1
Middle Reach	12.2
Unnamed Drainage	2.2
Lower Reach	14.4

From which the adjusted fish passage flows could be calculated:

Location	95% cfs	50% cfs	5% cfs	1% cfs
Powerhouse Channel	15.0	15.0	15.0	15.0
Upper Reach	8.4	15.1	31.8	56.9
Middle Reach	23.4	30.1	46.8	71.9
Unnamed Drainage	4.2	5.4	8.4	13.0
Lower Reach	27.6	35.5	55.2	84.8

Juvenile Flows

Data collected from USGS Station 11512000 was processed to eliminate all data that was not approved for published use, and was limited to the month of March, the peak month when fish will be released from the site. This is summarized in the exceedance curve below:

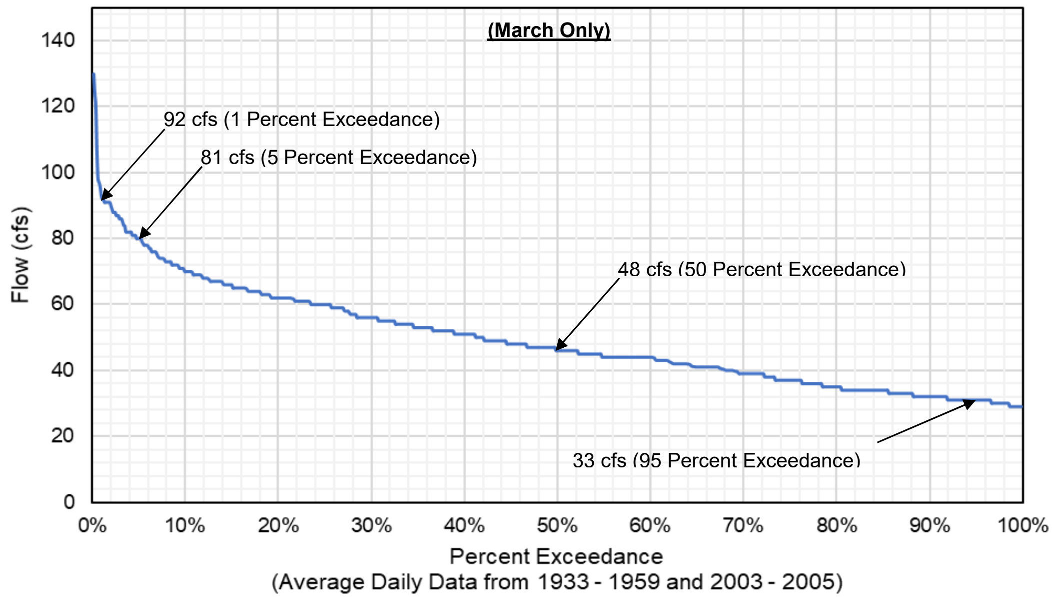

Figure 3. Exceedance Curve for USGS Station 11512000 (March Only)

Exceedance Criterion	Flow (cfs)
1% Exceedance	92
5% Exceedance	81
50% Exceedance	48
95% Exceedance	33

The juvenile design flow was then determined using the drainage area weighting as discussed above to determine the juvenile design high flow:

Location	1% cfs
Powerhouse Channel	15.0
Upper Reach	61.9
Middle Reach	76.9
Unnamed Drainage	13.9
Lower Reach	90.7

Flood Flows

The flood flows for the USGS gaging station were collected from the USGS StreamStats online software.

Return Period	Flow (cfs)
2-yr Flood	138
100-yr Flood	905

These values were checked against the methods of Bulletin 17C (USGS, 2019), and were found to be within 2% of each other, with the reported values slightly higher than those calculated by the methods of Bulletin 17C. Therefore, the reported values were accepted.

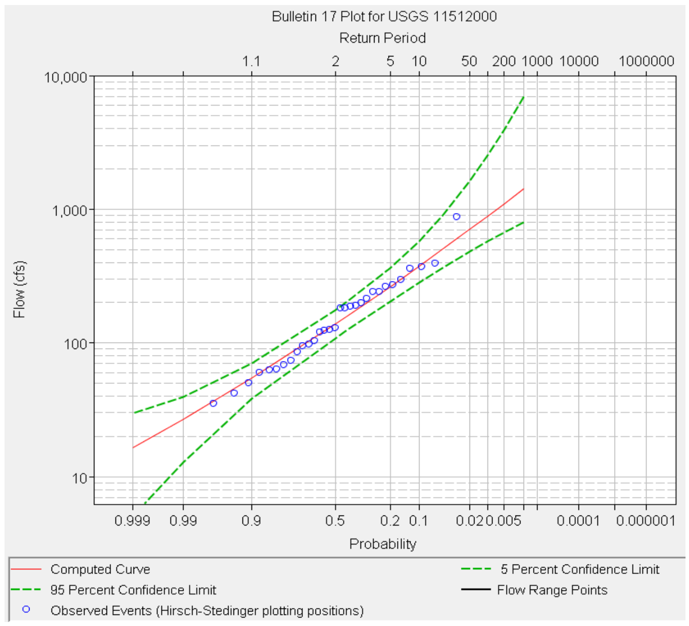

Figure 4. Frequency Analysis Results (Bulletin 17C)

These were then adjusted to the project site according to the drainage area scaling:

Location	2-yr cfs	100-yr cfs
Powerhouse Channel	15.0	15.0
Upper Reach	100.3	741.2
Middle Reach	115.3	756.2
Unnamed Drainage	20.8	136.4
Lower Reach	136.1	892.6

Conclusions

The streamflows for Fall Creek were determined from nearby USGS gage station 11512000 and adjusted to the site based on the relative drainage areas at each location. The streamflows are summarized below, and will serve as boundary conditions for the hydraulic model (see Tailwater calculations):

Location	Adult Fish Passage				Juvenile	Extreme Events	
	95% cfs	50% cfs	5% cfs	1% cfs	1% cfs	2-yr cfs	100-yr cfs
Powerhouse Channel	15.0	15.0	15.0	15.0	15.0	15.0	15.0
Upper Reach	8.4	15.1	31.8	56.9	61.9	100.3	741.2
Middle Reach	23.4	30.1	46.8	71.9	76.9	115.3	756.2
Unnamed Drainage	4.2	5.4	8.4	13.0	13.9	20.8	136.4
Lower Reach	27.6	35.5	55.2	84.8	90.7	136.1	892.6

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SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Tailwater	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to demonstrate the calculations of water surface elevations along the length of Fall Creek.

References

• *Chow, V.T. 1959. Open Channel Flow. McGraw Hill: New York.*

• *Gotvald, A.J., Barth, N.A., Veilleux, A.G., and Parrett, Charles, 2012, Methods for determining magnitude and frequency of floods in California, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2012–5113, 38 p., 1 pl., available online only at http://pubs.usgs.gov/sir/2012/5113/.*

• *Hydrologic Engineering Center (HEC). 2016. HEC-RAS: River Analysis System Hydraulic Reference Manual, Version 5.0. U.S. Dept. of the Army, Army Corps of Engineers, Hydrologic Engineering Center: Davis, CA. February 2016.*

Method

The tailwater elevation at the fishway entrance was calculated by 1-dimensional HEC-RAS modeling along Fall Creek. Model characteristics are summarized below:

Geometry

• Model geometry was collected from surveyed transects including both ground shots and stream bathymetry at approximately 50' spacing.
• Channel banks were surveyed as part of the transects, and were used to differentiate channel and overbank regions and their associated hydraulic roughness and conveyance.
• Manning's roughness coefficients of 0.035 were assigned uniformly to the channel, consistent with mountain streams with gravel bottoms (Chow, 1959).
• Manning's roughness coefficients of 0.060 were assigned to the overbank regions, consistent with floodplains with moderate brush (Chow, 1959).
• Levees were introduced at locations to contain flows within the channel in locations of depressions in the overbank areas and where there would be no upstream/downstream connectivity of the depression in the floodplain.
• Ineffective areas were introduced at locations of depression in the overbank areas where there is upstream/downstream connectivity, however the depression would not add to the cross-section conveyance (i.e. storage only).
• A flat section was introduced as a temporary measure at the fishway and exclusion barrier, and the roughness was adjusted to 0.015 for the concrete sill and abutments.
• Cross-sections were interpolated at 5-ft spacing according to the default HEC-RAS algorithm to ensure that changes in the energy grade line would be small and minimize errors in the calculations.

Hydrology

• See "Streamflow" calculations for assumptions regarding hydrology and flow boundary conditions. Seven flow conditions were evaluated:
  • Fish passage low flow (95% exceedance)
  • Fish passage typical flow (50% exceedance)
  • Fish passage high flow (NMFS Definition, 5% exceedance)
  • Fish passage high flow (CDFW Definition, 1% exceedance)
  • Juvenile high flow (1% exceedance, March only)
  • Flooding Flow - 2 year
  • Flooding Flow - 100 year

Boundary Conditions

• The boundary condition at Dam A was assumed to be critical.
• The boundary conditions in the two tributaries and at the downstream of the model extents was assumed to be normal flow with local bed slopes measured from the transect data or the LiDAR data as appropriate to the location.

Modeling Assumptions

• HEC-RAS solves the energy equation for each cross-section using the iterative process of the standard step method (HEC, 2016).
• The model was run as a steady model ($dQ/dt = 0$) at the peak discharge for each of the flow conditions listed above.
• The model was run for mixed regime, in order to allow for variations between subcritical and supercritical flow.
• Junctions were modeled using the energy equation, as is the HEC-RAS default, as the energy loss across the junction was not expected to be significant.

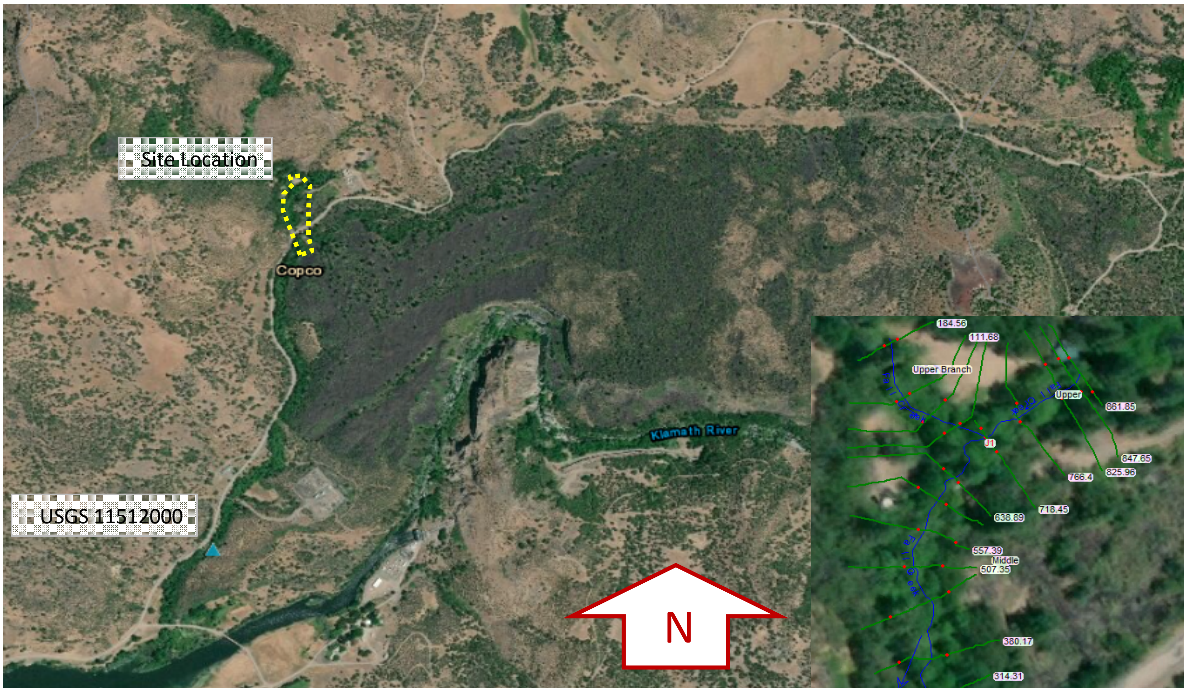


Figure 1. Gage Location

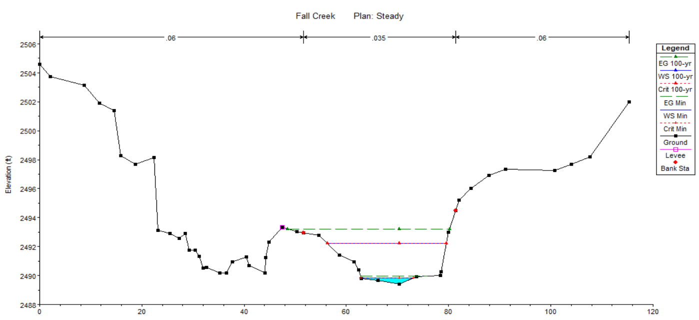


Figure 3. Typical Cross-Section

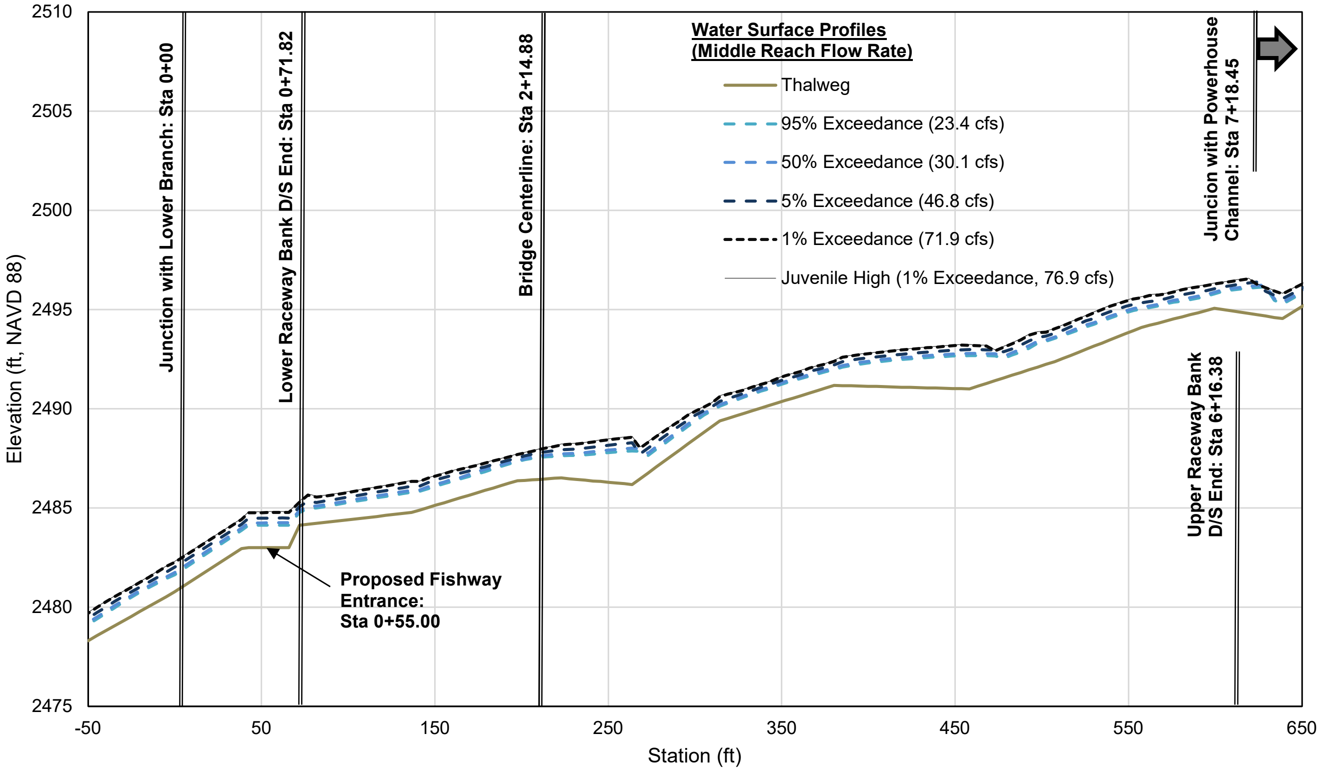
Figure 2. Model Geometry

Flow Change Location	Low Flow cfs	Typical cfs	High Flow (5%) cfs	High Flow (1%) cfs	Juvenile High cfs	2-yr cfs	100-yr cfs
Powerhouse Channel	15	15	15	15	15	15	15
Upper Reach	8	15	32	57	62	100	741
Middle Reach	23	30	47	72	77	115	756
Unnamed Drainage	4	5	8	13	14	21	136
Lower Reach	28	36	55	85	91	136	893

Table 1. Flow Change Locations
(Reference Streamflow Calculations)

Results

The results of the HEC-RAS modeling for the juvenile and adult fish passage flows are summarized in the longitudinal profile along Fall Creek, in Figure 4 below:



Bas

Figure 4. Longitudinal Profile

Conclusions

Water surface profiles in Fall Creek were calculated for each of the design flows using a 1-dimensional HEC-RAS model and available topography and bathymetry surveyed at the site. These water surface profiles were used in the design of in-stream structures, as well as to determine flooding extents and elevations for extreme event design flows. One location of critical interest to the site, was the proposed fishway entrance and temporary barrier, for fish trapping. The table below summarizes water surface elevations and depths at this location. Other locations were queried from the model, directly.

Flow Condition	Flow cfs	WSEL ft msl	Depth ft
Low - 95% Exceedance	23.40	2484.12	1.12
Typ - 50% Exceedance	30.08	2484.24	1.24
High - 5% Exceedance	46.79	2484.48	1.48
High - 1% Exceedance	71.86	2484.77	1.77
Juvenile Hi - 1% Exc.	76.88	2484.82	1.82
2-year	115.32	2485.13	2.13
100-year	756.23	2487.21	4.21

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SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** N. Cox
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Intake Losses	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to determine hydraulic head losses through the intake.

References

- *Tullis, J. Paul. (1989). Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients. New York: John Wiley & Sons.*
- *U.S. Bureau of Reclamation (USBR). 1987. Design of Small Dams. Third Edition. U.S. Dept. of the Interior, Bureau of Reclamation: Washington, D.C.*

Method

The head losses through the intake structure were considered to consist of two components: (1) debris screen losses and (2) pipe entrance losses. Elsewhere, the velocity is to be maintained 1 ft/s or less and therefore minor losses and friction losses were considered negligible.

Debris Screen
USBR, 1987; Section 10.15, Eq 11

Debris screen losses are evaluated according to the equation presented in the *Design of Small Dams* (USBR, 1987; see also Creager & Justin, 1963). The losses through the debris screen are a function of the percent opening (net screened area divided by gross area):

$$K_s = 1.45 - 0.45\frac{A_n}{A_g} - \left(\frac{A_n}{A_g}\right)^2$$

$$A_n = (1 - R_D)R_0 A_g$$

$$h_s = K_s\left(\frac{v_n^2}{2g}\right)$$

where:
$K_s =$ Screen loss coefficient
$h_s =$ Screen head losses, ft
$A_n =$ Net screen area (less screen and occlusions), ft^2
$A_g =$ Gross screen area, ft^2
$v_n =$ Net velocity (through net screen area), ft/s
$g =$ Gravitational constant, 32.2 ft/s^2
$R_D =$ Ratio of debris coverage
$R_o =$ Ratio of open area (clean bars)

Pipe Entrance Losses
Tullis, 1989; Table 1.4 and USBR, 1987; Table 10.1

Entrance loss coefficients have been tabulated by a number of sources, including Tullis (1989) and the USBR (1987). The USBR provides a range of coefficients based on a survey of texts and technical papers.

$$h_e = K_e\left(\frac{v_p^2}{2g}\right)$$

where:
$h_e =$ Entrance head losses, ft
$K_e =$ Entrance loss coefficient
$v_p =$ Pipe velocity, ft/s
<Other parameters as previously defined>

TABLE 1.4 Minor Loss Coefficients

Item	K_l Typical Value	K_l Typical Range
Pipe inlets		
Inward projecting pipe	0.78	0.5 to 0.9
Sharp corner—flush	0.50	
Slightly rounded	0.20	0.04 to 0.5
Bell mouth	0.04	0.03 to 0.1

FIGURE 1. Typical Entrance Loss Coefficients (Tullis, 1989)

	Discharge coefficient, *C*			Loss coefficient, *A*		
	Max.	*Min.*	*Avg.*	*Max.*	*Min.*	*Avg.*
(a) Gate in thin wall – unsuppressed contraction	0.70	0.60	0.63	1.80	1.00	1.50
(b) Gate in thin wall – bottom and sides suppressed	.81	.68	.70	1.20	0.50	1.00
(c) Gate in thin wall – corners rounded	.95	.71	.82	1.00	.10	0.50
(d) Square-cornered entrances	.85	.77	.82	0.70	.40	.50
(e) Slightly rounded entrances	.92	.79	.90	.60	.18	.23
(f) Fully rounded entrances ($r/D \geq 0.15$)	.96	.88	.95	.27	.08	.10
(g) Circular bellmouth entrances	.98	.95	.98	.10	.04	.05
(h) Square bellmouth entrances	.97	.91	.93	.20	.07	.16
(i) Inward projecting entrances	.80	.72	.75	.93	.56	.80

FIGURE 2. USBR Entrance Loss Coefficients (USBR, 1987)

Inputs

Geometric

The geometric inputs are summarized below:

Intake

Min. WSE:	2510.4	ft msl	[Dam A crest elevation]
Intake Bottom El:	2506.3	ft msl	[Design value, per City of Yreka sluice gate invert]
Intake Width:	6.0	ft	[2 x 3.0' wide screens]
Intake Min. Depth:	4.10	ft	
Open Area Ratio, R_o:	50%		[Assumed, subject to screen manufacturer]

Pipe

Prelim. Nom Dia:	24.0	in	
Inner Dia:	21.418	in	[Sched 80 PVC]
	1.78	ft	

Hydraulic

The hydraulic inputs are summarized below:

Max Screen Occlusion:	50%		[Max recommended by USBR, 1987]
Typ/Max Demand:	10	cfs	

Calculations

Debris Screen Losses

Percent Occluded, R_D	Ratio of Open Area, R_o	Gross Area, A_g (ft^2)	Net Area, A_n (ft^2)	Ratio of Net to Gross Area, A_n/A_g	Loss Coeff, K_s	Net Velocity, V_n (ft/s)	Velocity Head, h_v (ft)	Head Loss, h_s (ft)
0%	50%	24.6	12.30	50%	0.98	0.81	0.01	0.01
5%	50%	24.6	11.68	48%	1.01	0.86	0.01	0.01
10%	50%	24.6	11.07	45%	1.05	0.90	0.01	0.01
15%	50%	24.6	10.45	43%	1.08	0.96	0.01	0.02
20%	50%	24.6	9.84	40%	1.11	1.02	0.02	0.02
25%	50%	24.6	9.22	38%	1.14	1.08	0.02	0.02
30%	50%	24.6	8.61	35%	1.17	1.16	0.02	0.02
35%	50%	24.6	7.99	33%	1.20	1.25	0.02	0.03
40%	**50%**	**24.6**	**7.38**	**30%**	**1.23**	**1.36**	**0.03**	**0.03**
45%	50%	24.6	6.76	28%	1.25	1.48	0.03	0.04
50%	50%	24.6	6.15	25%	1.28	1.63	0.04	0.05

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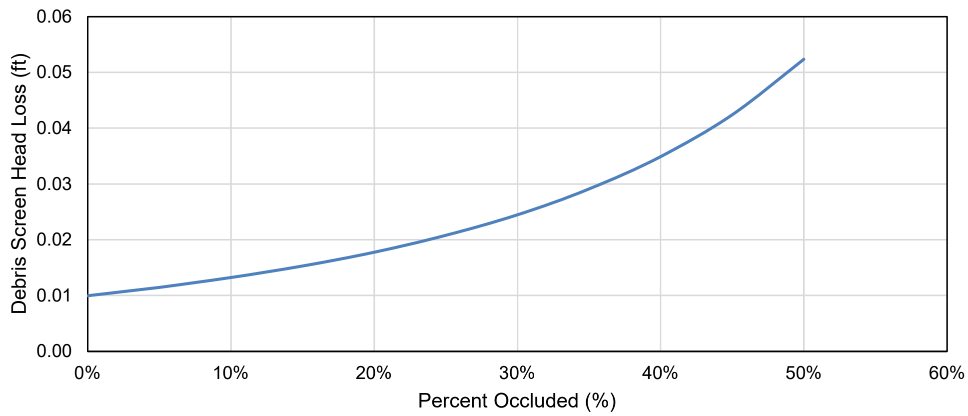


Entrance Losses

Pipe entrance losses were calculated for a variety of conditions, for use in the design process. It was ultimately elected that no gate would be present at the intake structure, but rather isolation would be performed using a downstream isolation valve in the meter vault. Therefore, the open pipe values were used.

Entrance	Condition	Pipe Nom. Dia, D (in)	Pipe Inner Dia, D_i (in)	Pipe Velocity, V_p (ft/s)	Velocity Head, h_v (ft)	Loss Coeff, K_e	Head Loss, h_e (ft)
Gate	Max (unsuppressed gate)	24.0	21.418	4.00	0.25	1.8	0.45
	Avg (unsuppressed gate)	24.0	21.418	4.00	0.25	1.5	0.37
	Min (unsuppressed gate)	24.0	21.418	4.00	0.25	1.0	0.25
	Improved (corners round)	24.0	21.418	4.00	0.25	0.5	0.12
Open Pipe (D/S Isolation Valve)	**Max (square corners)**	**24.0**	**21.418**	**4.00**	**0.25**	**0.7**	**0.17**
	Avg (square corners)	24.0	21.418	4.00	0.25	0.5	0.12
	Min (square corners)	24.0	21.418	4.00	0.25	0.4	0.10
	Improved (slightly round)	24.0	21.418	4.00	0.25	0.23	0.06

Conclusions

The above calculations demonstrate that the head losses through the intake under worst case conditions, i.e. 50% screen occlusion and unimproved entrance conditions at the pipe, would be approximately 0.22 ft (2.6 in). This is not expected to be the case, however, as the screens will be actively cleaned and it is not expected that occlusion will reach 50%. As a design value, a conservative screen occlusion of 40% was assumed, however, resulting in a maximum loss through the intake of 0.21 ft. This value was used as a boundary condition to the head modeling performed for the supply piping (see "Supply Hydraulics" calculations).

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Supply Hydraulics

BY: A. Leman CHK'D BY: N. Cox
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to demonstrate the hydraulic calculations associated with the supply piping.

References

- *Miller, D.S. 1990. Internal Flow Systems, Second Edition. Cranfield, UK: BHRA, The Fluid Engineering Centre.*
- *Rossman, L.A. 2000. EPANET2, User's Manual. U.S. Environmental Protection Agency (USEPA), Office of Research and Development, National Risk Management Research Laboratory: Cincinnati, OH.*
- *Tullis, J. Paul. 1989. Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients. New York: John Wiley & Sons.*
- *U.S. Bureau of Reclamation (USBR). 1951. Turbines and Pumps, Design Supplement No. 6. U.S. Dept. of the Interior, USBR: Washington, D.C.*
- *U.S. Bureau of Reclamation (USBR). 1987. Design of Small Dams, Third Edition. U.S. Dept. of the Interior, USBR: Washington, D.C.*
- *Walski, T.M., J.D. Edwards, and V.M. Hearne. 1989. Loss of Carrying Capacity in Pipes. Environmental Engineering Proceedings of the 1989 Specialty Conference, Austin, TX, July 10-12. Published by ASCE.*

Method

The supply piping network was analyzed using EPANET2 software (Rossman, 2000), which calculates hydraulic head conditions based on the following equations for head loss. Calculations were compared at various stages of the design process with manual calculations performed by the engineer. Given the low gradient across the site it is anticipated that the driving head will be limited at certain design points. These calculations seek to confirm that the driving head is sufficient to meet the hatchery needs.

Friction Losses

Friction losses were calculated according to the Hazen-Williams equation:

$$h_{f,ft} = \frac{10.44 L_{ft} Q_{gpm}^{1.85}}{C^{1.85} d_{in}^{4.87}}$$

where:
$h_{f,ft} =$ Friction head losses, ft
$L_{ft} =$ Length of pipe run, ft
$Q_{gpm} =$ Discharge, gpm
$C =$ Hazen-Williams coefficient
$d_{in} =$ Pipe diameter, in

Minor Losses

Minor losses were calculated according to the standard minor loss formulation:

$$h_L = K\left(\frac{V^2}{2g}\right)$$

where:
$h_L =$ Minor head losses, ft
$K =$ Composite minor loss coefficient
$V =$ Pipe average velocity, ft/s
$g =$ Gravitational constant, 32.2 ft/s^2

Inputs

Demand Nodes

Demand at the design point model nodes were based on variable operational scenarios as discussed in the Scenarios section below. A detailed description of each of the cases is provided below.

Pipe Velocities/Sizes

Pipe sizes were selected to maintain velocities within the desired range of 1.5 feet per second (fps) to 5.0 fps, such that pipes would be self-cleaning (lower bound), but head losses would not be excessive and abrasion potential would be mitigated (upper bound). At locations where velocities were allowed to drop below 2 ft/s a pressurized cleanout is provided in the vicinity such that maintenance could be performed as necessary.

Piping Configuration

Pipes were configured in the model according to the current stage of design, and all lengths, fittings, and elevations were derived from this piping configuration. Significant field construction adjustments would require further evaluation.

Upstream Boundary Condition

For the upstream boundary condition, a conservative case for which the water surface elevation is at the Dam A crest elevation was used. Losses were accounted for in the 'Intake Losses' calculations, and have been reported here. The upstream boundary condition for the model is located inside the pipe just downstream of the intake structure. An extreme boundary condition is also reported below, for an assumed 0.5 ft head loss through the intake structure. The calculated head loss through the intake structure is already a conservative condition, which uses the maximum recommended USBR (1987) loss coefficient through a square-edged pipe entrance (0.7, cf. EPANet recommendation - 0.5) and an assumed 40% occluded screen. This "extreme condition" represents a worst case scenario, if for instance one of the screens were to be taken out of operation, and the other screen were highly occluded. This is provided for reference, and is not expected to occur.

U/S Boundary Condition	Crest El	Head Loss	U/S Boundary
	ft	ft	ft
Calculated	2510.4	0.21	2510.19
"Extreme"	2510.4	0.50	2509.90

Minor Loss Coefficients

There is some variation between sources for recommended minor loss coefficients due to broad sampling of laboratory and field studies, different approaches to factors of safety for engineering design, and difrerences in Reynolds number for pipelines of varying sizes. The following three sets of minor loss coefficients represent different samples. For this analysis, the coefficients of Tullis (1989) and Miller (1990) will be used alongside the coefficients of Rossman (2000) for comparison of the different approaches.

	90° Bends	45° Bends	22.5° Bends	Gate Valve (Open)	Butterfly Valve (Open)	Tee (Branch)	Tee (Line)	Reducer (Contract)	Sources
Coefficient K	0.24	0.1	0.06	0.15	0.2	1	0.2		*Tullis, 1989 and Miller, 1990* [1,2]
	0.23	0.16	0.09	0.19	0.15	1.25	0.03	0.5	*USBR, 1987 and USBR,1951* [3]
	0.9	0.4	0.2	0.2	0.27	1.8	0.6		*Rossman, 2000* [4,5,6] *(EPANet Manual)*

[1] *All values taken from Tullis, 1989 except for tees, for which the reader is referred to the work of Miller, 1990. Tee values were assumed based on a dividing flow with a flow distribution of 75% in the branch and 25% in the main stem for a sharp-edged 90 degree tee. The flow distribution will vary based on the operational conditions. These values are conservative, as typically the majority of the flow will remain in the main branch, yielding lower coefficient values. See Figure below.*

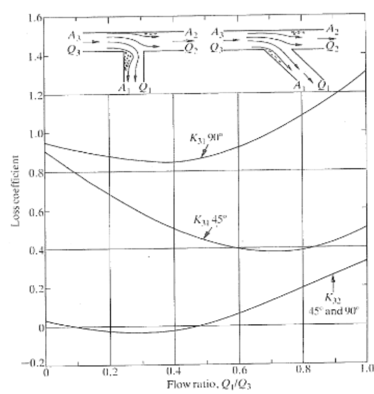


Figure 1. Dividing Flow Tee Loss Coefficients (Miller, 1990)

[2] *Reducer losses were calculated based on the following equation for contractions:* $K = \left(1/C_c - 1\right)^2$

A_2/A_1	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
C_c	0.624	0.632	0.643	0.659	0.681	0.712	0.755	0.813	0.892
K	0.363	0.339	0.308	0.268	0.219	0.164	0.105	0.053	0.015

and the following equation for expansions: $K = \left(1 - \left(\frac{A_1}{A_2}\right)^2\right)$

[3] *All values were taken from USBR (1987) Design of Small Dams, except for tees which were collected from USBR (1951) Turbines and Pumps Design Supplement No. 6 .*

[4] *EPANet User's Manual does not provide a value for 22.5° bends. A value of 1/2 times the 45° bend was used as an estimate.*

[5] *EPANet User's Manual does not provide a value for butterfly valves, so the gate valve coefficient was scaled according to the ratio of coefficients in Tullis, 1989.*

[6] *The reducer coefficient was not reported in the EPANet User's Manual (Rossman, 2000), so the method of Tullis, 1989 (see note 2 above) was utilized.*

Friction Loss Coefficients

Friction loss coefficients were selected based on PVC schedule 80 pipe. The pipe will be installed in new condition throughout the site, however it is expected that there may be some potential for deposition of fine material in the pipe which would have to pass the intake screens (velocities will also be maintained to ensure resuspension of fine sediments). It is also anticipated that there could be some biological accumulation (biofilm) along the pipe walls that would deviate from the new pipe condition over the life of the facility. Potential values for Hazen-Williams coefficients associated with these conditions are summarized below:

Condition of Pipe	Recommended H-W Coefficient
New or in excellent condition cast-iron and steel pipe with cement, or bituminous linings centrifugally spun, cement-asbestos pipe, copper tubing, brass pipe, **plastic pipe**, and glass pipe	140
Older pipe listed above in good condition, and/or pipes, cement mortar lined in place with good workmanship, larger than 24 in. in diameter	130
Cement mortar lined pipe in place, small diameter with good workmanship or large diameter with ordinary workmanship: work stave; tar-dipped cast-iron pipe new and/or old in inactive water	120
Old unlined or tar-dipped cast-iron pipe in good condition	100
Old cast-iron pipe severely tuberculated or any pipe with heavy deposits	40-80

** Table reproduced from Tullis, 1989*

Walski, et al. (1989) have esimated a linear relationship of pipe degradation, in terms of a Hazen-Williams coefficient, against the age of the pipe in water distribution systems in Austin, TX. Their relationship is given by:

$$C = 145 - 1.8t$$

where: $C =$ Hazen-Williams coefficient
$t =$ Pipe age, yrs

According to this approach, after approximately 8 years the Hazen-Williams coefficient would be about 130, and after about 12 years the coefficient would be about 120. Given occasional cleaning of the pipes, the smooth condition of these pipes could be extended much longer.

Other Inputs

Gravitational Constant 32.2 ft/s^2

Scenarios

A number of scenarios were evaluated for the site, anticipating operational changes, contingencies, and degradation over time. A description of each of the scenarios is provided below, and they are summarized in the table that follows:

Scenario 0, Base Case - Scenario 0 evaluates the pipe under normal conditions, at the time in the bioprogram when the water demand on the supply line is greatest. Pipes are assumed to be in a clean, new condition, and the minor loss coefficients of Tullis & Miller (see above) are applied. The highest water demand according to the bioprogram for each of the facilities (and supply lines) is listed below:

Coho Building - The critical timing occurs in January when the incubation stacks are still in full operation (40 gpm), but the previous brood year is still occupying the raceways (683 gpm). At the same time, the incubation working vessels will be used for picking (30 gpm).

Chinook Raceways - The critical timing for the Chinook raceways occurs in May, when there will be approximately 2,000,000 fish at 104 fish per pound (fpp) in the Chinook raceways. At this time, the raceways will require 4,028 gpm.

Chinook Incubation Building - The critical timing for the Chinook incubation building occurs in January when egg picking will require the full use of the incubation stacks (680 gpm) and the 4 working vessels (60 gpm).

Adult Holding - The critical timing for the adult holding will be in December when trapping occurs and other flows around the site are at a minimum. To get 10 cfs to the fish ladder, 3100 gpm will be required through the adult holding supply line.

Scenario 1, Pipe Degradation - Scenario 1 evaluates the condition where the pipes have degraded over time, either through accumulation of biomass or through a failure of the screen leading to introduction of sediment, debris, or detritus to the pipeline. For this scenario, the Hazen-Williams coefficient will be adjusted to 120, to account for the additional friction losses in the pipe.

Scenario 2, Operational Change & Pipe Degradation - Scenario 2 evaluates the same degraded pipe condition of Scenario 1, but for an operational change that requires the maximum bioprogram flow to all design points (incubation stacks, working vessels, raceways, etc.) within a building at a single time. These demands are summarized in the table below.

Scenario 3, Intake Loss Contingency, Operational Change & Pipe Degradation - Scenario 3 builds on Scenario 2 by adding contingency losses at the intake structure, due to a traveling screen being taken out of operation, or excessive blockage, or some other additional head losses being introduced at the intake structure.

Scenario 4, Minor Loss Contingency & Pipe Degradation - Scenario 4 retains the pipe degradation from Scenario 1, and uses the much more conservative minor loss accounting given by the EPANet User's Manual (Rossman, 2000; see above for details). This scenario uses the highest water demand from the bioprogram, as discussed in the base case above.

Scenario	Demand Condition	Friction Loss Coefficients	Minor Loss Coefficients	U/S Boundary Condition
Scenario 0 *(Base Case)*	**<u>Coho</u>** Incubation Stacks - 40 gpm Working Vessels - 30 gpm Raceways - 683 gpm **<u>Chinook Raceways</u>** Raceways - 4,028 gpm **<u>Chinook Incubation</u>** Incubation Stacks - 680 gpm Working Vessels - 60 gpm **<u>Adult Holding</u>** Holding Ponds - 3,100 gpm	140 (new pipe)	Tullis & Miller	2510.19
Scenario 1 *(Pipe Degradation)*	See Base Case Above	120	Tullis & Miller	2510.19
Scenario 2 *(Operational Change +* *Pipe Degradation)*	**<u>Coho</u>** Incubation Stacks - 40 gpm Working Vessels - 30 gpm Raceways - 764 gpm First-Feeding - 150 gpm **<u>Chinook Raceways</u>** Raceways - 4,028 gpm **<u>Chinook Incubation</u>** Incubation Stacks - 816 gpm Working Vessels - 60 gpm **<u>Adult Holding</u>** Holding Ponds - 4,480 gpm	120	Tullis & Miller	2510.19
Scenario 3 *(Intake Contingency +* *Operational Change +* *Pipe Degradation)*	See Scenario 2 Above	120	Tullis & Miller	2509.90
Scenario 4 *(Minor Loss Conting. +* *Pipe Degradation)*	See Base Case Above	120	Rossman (EPANet)	2510.19

Calculations

The following tables summarize the pipe inputs into the models including lengths, inner diameters, Hazen-Williams coefficients, and composite minor losses after summing up all of the fittings, valves, and other losses.

Supply Line 1 - Coho Building

Pipe I.D.	Length	Inner Diameter	Scenario 0		Scenario 1		Scenario 2		Scenario 3		Scenario 4	
	ft	in	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff
P1	175	14.213	140	1.579	120	1.579	120	1.579	120	1.579	120	2.598
P2	11.5	7.565	140	1.201	120	1.201	120	1.201	120	1.201	120	2.001
P3	9	7.565	140	0.000	120	0.000	120	0.000	120	0.000	120	0.000
P4	1	7.565	140	0.000	120	0.000	120	0.000	120	0.000	120	0.000
P5	4	7.565	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P6	8.5	7.565	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P7	4	7.565	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P8	10.5	3.786	140	2.899	120	2.899	120	2.899	120	2.899	120	5.086
P9	10.5	3.786	140	2.899	120	2.899	120	2.899	120	2.899	120	5.086
P10	10.5	3.786	140	2.899	120	2.899	120	2.899	120	2.899	120	5.086
P11	10.5	3.786	140	2.899	120	2.899	120	2.899	120	2.899	120	5.086
P12	23	7.565	140	1.201	120	1.201	120	1.201	120	1.201	120	2.001
P13	17	3.786	140	1.659	120	1.659	120	1.659	120	1.659	120	3.186
P14	4.5	3.786	140	2.000	120	2.000	120	2.000	120	2.000	120	2.800
P15	4.5	3.786	140	2.000	120	2.000	120	2.000	120	2.000	120	2.800
P16	13	7.565	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P17	17	3.786	140	1.659	120	1.659	120	1.659	120	1.659	120	3.186
P18	4.5	3.786	140	2.000	120	2.000	120	2.000	120	2.000	120	2.800
P19	4.5	3.786	140	2.000	120	2.000	120	2.000	120	2.000	120	2.800
P20	10	5.709	140	0.277	120	0.277	120	0.277	120	0.277	120	0.677
P21	18	2.864	140	0.418	120	0.418	120	0.418	120	0.418	120	0.818
P22	20.5	2.864	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P23	13	2.864	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P24	25	5.709	140	1.000	120	1.000	120	1.000	120	1.000	120	1.800
P25	5	2.864	140	1.458	120	1.458	120	1.458	120	1.458	120	2.918
P26	10	2.864	140	2.200	120	2.200	120	2.200	120	2.200	120	3.067
P27	10	2.864	140	2.200	120	2.200	120	2.200	120	2.200	120	3.067
P28	27	5.709	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P29	5.5	5.709	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P30	3	5.709	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P31	8	2.864	140	2.898	120	2.898	120	2.898	120	2.898	120	5.085
P32	8	2.864	140	2.898	120	2.898	120	2.898	120	2.898	120	5.085
P33	8	5.709	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P34	8	2.864	140	2.898	120	2.898	120	2.898	120	2.898	120	5.085
P35	8	2.864	140	2.898	120	2.898	120	2.898	120	2.898	120	5.085

Supply Line 2 - Chinook Rearing

Pipe I.D.	Length	Inner Diameter	Scenario 0		Scenario 1		Scenario 2		Scenario 3		Scenario 4	
	ft	in	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff
P1	269	21.418	140	2.421	120	2.421	120	2.421	120	2.421	120	4.894
P2	9	21.418	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P3	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P4	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P5	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P6	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P7	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P8	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P9	6.3	21.418	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P10	19	14.213	140	0.327	120	0.327	120	0.327	120	0.327	120	0.727
P11	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P12	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P13	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P14	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P15	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P16	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P17	6.3	14.213	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P18	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P19	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P20	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P21	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P22	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P23	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P24	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P25	12	5.709	140	2.999	120	2.999	120	2.999	120	2.999	120	5.185
P26	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P27	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P28	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P29	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P30	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P31	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P32	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134
P33	12	5.709	140	2.947	120	2.947	120	2.947	120	2.947	120	5.134

Supply Line 3 - Chinook Incubation Building

Pipe I.D.	Length	Inner Diameter	Scenario 0		Scenario 1		Scenario 2		Scenario 3		Scenario 4	
	ft	in	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff	H-W Coeff	Composite Minor Loss Coeff
P0	452	14.213	140	2.716	120	2.716	120	2.716	120	2.716	120	6.915
P1	6.75	9.493	140	0.124	120	0.124	120	0.124	120	0.124	120	0.124
P2	18.5	2.864	140	1.308	120	1.308	120	1.308	120	1.308	120	2.108
P3	3	2.864	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P4	10	2.864	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P5	3.5	9.493	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P6	4.5	2.864	140	1.308	120	1.308	120	1.308	120	1.308	120	2.108
P7	10	2.864	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P8	3	9.493	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P9	1.5	5.709	140	1.163	120	1.163	120	1.163	120	1.163	120	1.963
P10	11	5.709	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P11	12	9.493	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P12	1.5	5.709	140	1.163	120	1.163	120	1.163	120	1.163	120	1.963
P13	11	5.709	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P14	12	5.709	140	0.363	120	0.363	120	0.363	120	0.363	120	0.763
P15	1.5	5.709	140	1.000	120	1.000	120	1.000	120	1.000	120	1.800
P16	11	5.709	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P17	12	5.709	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P18	1.5	5.709	140	1.000	120	1.000	120	1.000	120	1.000	120	1.800
P19	11	5.709	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P20	2.5	5.709	140	0.200	120	0.200	120	0.200	120	0.200	120	0.600
P21	4.5	2.864	140	1.218	120	1.218	120	1.218	120	1.218	120	2.018
P22	10	2.864	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967
P23	3.5	2.864	140	0.418	120	0.418	120	0.418	120	0.418	120	0.818
P24	19	2.864	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P25	3	2.864	140	0.240	120	0.240	120	0.240	120	0.240	120	0.900
P26	10	2.864	140	1.920	120	1.920	120	1.920	120	1.920	120	3.967

Supply Line 4 - Adult Holding

Pipe I.D.	Length	Inner Diameter	Scenario 0		Scenario 1		Scenario 2		Scenario 3		Scenario 4	
	ft	in	H-W Coeff	Composite Minor Loss	H-W Coeff	Composite Minor Loss	H-W Coeff	Composite Minor Loss	H-W Coeff	Composite Minor Loss	H-W Coeff	Composite Minor Loss
P1	730	21.418	140	3.361	120	3.361	120	3.361	120	3.361	120	8.794
P2	50	3.786	140	3.039	120	3.039	120	3.039	120	3.039	120	5.819
P3	13.5	14.213	140	0.327	120	0.327	120	0.327	120	0.327	120	0.727
P4	13.5	11.294	140	0.456	120	0.456	120	0.456	120	0.456	120	0.922
P5	18	11.294	140	2.884	120	2.884	120	2.884	120	2.884	120	5.071
P6	18	11.294	140	2.736	120	2.736	120	2.736	120	2.736	120	4.922
P7	3.5	11.294	140	1.240	120	1.240	120	1.240	120	1.240	120	1.900

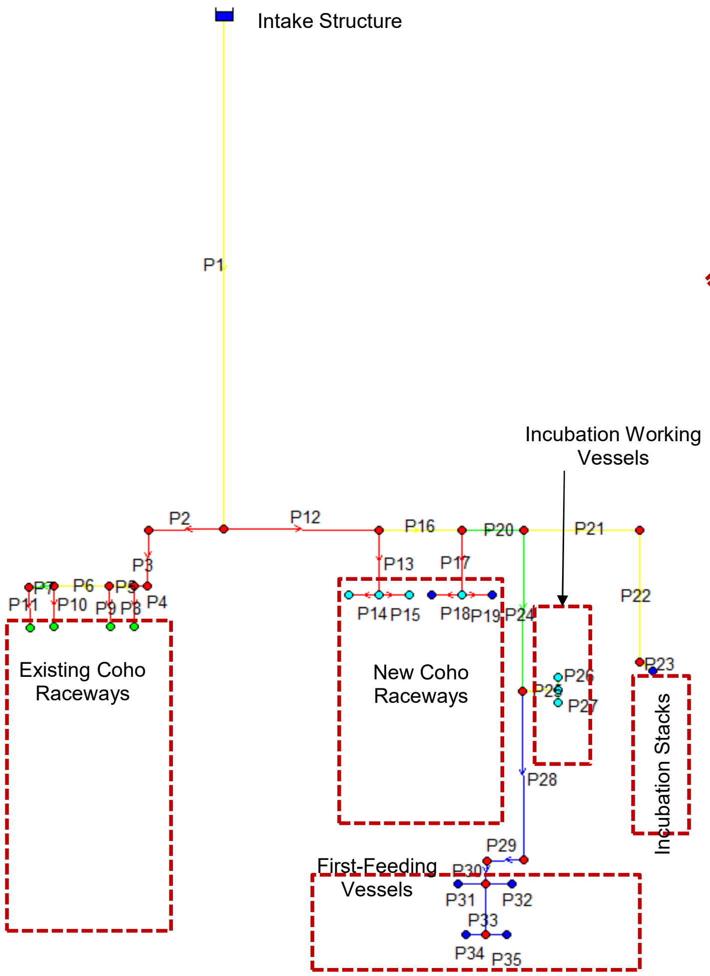


Figure 2. Pipe Naming Convention (Coho Building)

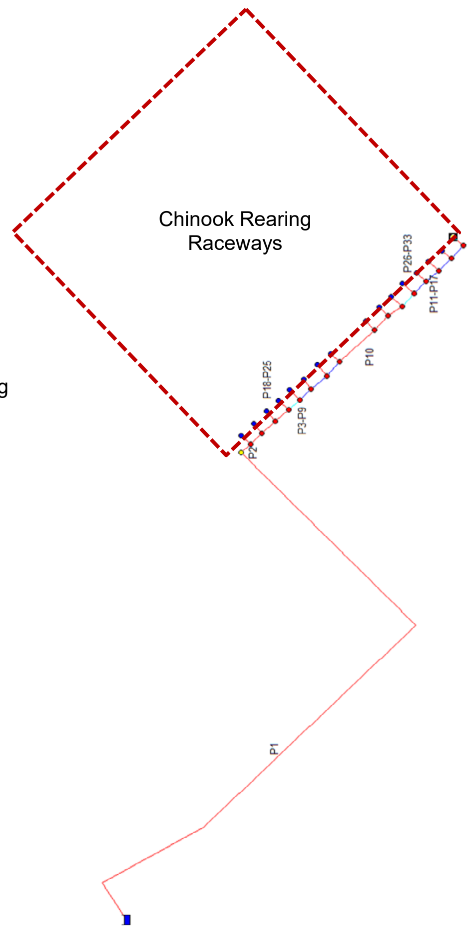


Figure 3. Pipe Naming Convention (Chinook Rearing)

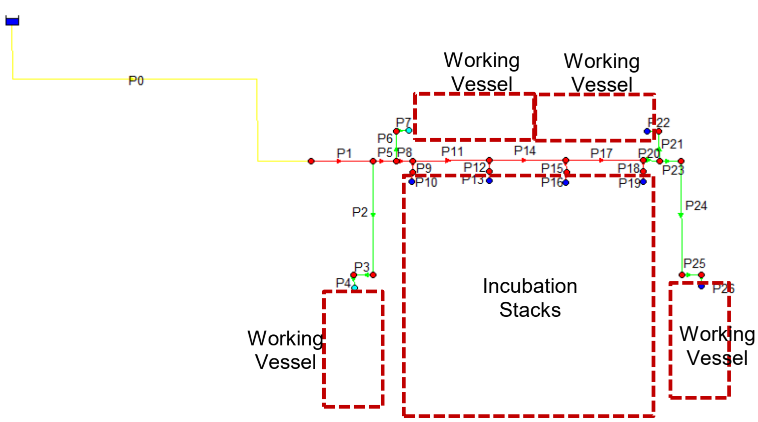


Figure 4. Pipe Naming Convention (Chinook Incubation Building)

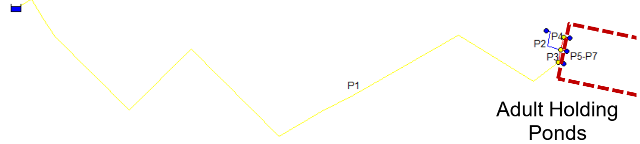


Figure 5. Pipe Naming Convention (Adult Holding)

Results

Supply Line 1 - Coho Building

Location	Pipe CL Elevation ft	Scenario 0 Tot. Head ft	Scenario 0 Avail. Head ft	Scenario 0 Pressure psig
Pipe split (under concrete slab)	2500.50	2510.03	9.53	4.13
Existing Rearing Raceways	2503.66	2509.5	5.84	2.53
New Rearing Raceways	2505.33	2508.53	3.20	1.39
First-Feeding Vessels	2507.83	2509.74	1.91	0.83
Incubation Working Vessels	2506.83	2509.64	2.81	1.22
Incubation Head Tank	2508.00	2509.27	1.27	0.55

Location	Pipe CL	Scenario 1			Scenario 2		
	Elevation	Tot. Head	Avail. Head	Pressure	Head	Avail. Head	Pressure
	ft	ft	ft	psig	ft	ft	psig
Pipe split (under concrete slab)	2500.50	2510.00	9.50	4.12	2509.92	9.42	4.08
Existing Rearing Raceways	2503.66	2509.42	5.76	2.50	2509.34	5.68	2.46
New Rearing Raceways	2505.33	2508.33	3.00	1.30	2507.96	2.63	1.14
First-Feeding Vessels	2507.83	2509.66	1.83	0.79	2508.64	0.81	0.35
Incubation Working Vessels	2506.83	2509.56	2.73	1.18	2508.92	2.09	0.91
Incubation Head Tank	2508.00	2509.09	1.09	0.47	2508.64	0.64	0.28

Location	Pipe CL	Scenario 3			Scenario 4		
	Elevation	Head	Avail. Head	Pressure	Head	Avail. Head	Pressure
	ft	ft	ft	psig	ft	ft	psig
Pipe split (under concrete slab)	2500.50	2509.63	9.13	3.96	2509.96	9.46	4.10
Existing Rearing Raceways	2503.66	2509.05	5.39	2.34	2509.07	5.41	2.34
New Rearing Raceways	2505.33	2507.67	2.34	1.01	2507.53	2.20	0.95
First-Feeding Vessels	2507.83	2508.35	0.52	0.23	2509.49	1.66	0.72
Incubation Working Vessels	2506.83	2508.63	1.80	0.78	2509.33	2.50	1.08
Incubation Head Tank	2508.00	2508.35	0.35	0.15	2508.73	0.73	0.32

Supply Line 2 - Chinook Rearing

Location	Pipe CL Elevation ft	Scenario 0 Tot. Head ft	Scenario 0 Avail. Head ft	Scenario 0 Pressure psig
Final Raceway	2505.50	2507.98	2.48	1.07

Location	Pipe CL	Scenario 1			Scenario 2		
	Elevation	Tot. Head	Avail. Head	Pressure	Head	Avail. Head	Pressure
	ft	ft	ft	psig	ft	ft	psig
Final Raceway	2505.50	2507.74	2.24	0.97	2507.74	2.24	0.97

Location	Pipe CL	Scenario 3			Scenario 4		
	Elevation	Head	Avail. Head	Pressure	Head	Avail. Head	Pressure
	ft	ft	ft	psig	ft	ft	psig
Final Raceway	2505.50	2507.45	1.95	0.84	2506.11	0.61	0.26

Supply Line 3 - Chinook Incubation Building

Location	Pipe CL Elevation ft	Scenario 0 Tot. Head ft	Scenario 0 Avail. Head ft	Scenario 0 Pressure psig
Pipe Entrance to Building	2499.00	2509.8	10.80	4.68
Incubation Stacks (Southmost)	2508.00	2509.07	1.07	0.46
Working Vessel (South Wall)	2506.33	2509.25	2.92	1.27

Location	Pipe CL Elevation ft	Scenario 1 Tot. Head ft	Scenario 1 Avail. Head ft	Scenario 1 Pressure psig	Scenario 2 Head ft	Scenario 2 Avail. Head ft	Scenario 2 Pressure psig
Pipe Entrance to Building	2499.00	2509.7	10.70	4.64	2509.52	10.52	4.56
Incubation Stacks (Southmost)	2508	2508.88	0.88	0.38	2508.37	0.37	0.16
Working Vessel (South Wall)	2506.33	2509.06	2.73	1.18	2508.67	2.34	1.01

Location	Pipe CL Elevation ft	Scenario 3 Head ft	Scenario 3 Avail. Head ft	Scenario 3 Pressure psig	Scenario 4 Head ft	Scenario 4 Avail. Head ft	Scenario 4 Pressure psig
Pipe Entrance to Building	2499.00	2509.23	10.23	4.43	2509.56	10.56	4.58
Incubation Stacks (Southmost)	2508.00	2508.08	0.08	0.03	2508.19	0.19	0.08
Working Vessel (South Wall)	2506.33	2508.38	2.05	0.89	2508.54	2.21	0.96

Supply Line 4 - Adult Holding

Location	Pipe CL Elevation ft	Scenario 0 Tot. Head ft	Scenario 0 Avail. Head ft	Scenario 0 Pressure psig
Chinook Holding Pond	2492.20	2508.09	15.89	6.89
Spawning Building	2492.00	2508.18	16.18	7.01

Location	Pipe CL Elevation ft	Scenario 1 Tot. Head ft	Scenario 1 Avail. Head ft	Scenario 1 Pressure psig	Scenario 2 Head ft	Scenario 2 Avail. Head ft	Scenario 2 Pressure psig
Chinook Holding Pond	2492.20	2508.05	15.85	6.87	2505.91	13.71	5.94
Spawning Building	2492	2508.09	16.09	6.97	2506.52	14.52	6.29

Location	Pipe CL Elevation ft	Scenario 3 Head ft	Scenario 3 Avail. Head ft	Scenario 3 Pressure psig	Scenario 4 Head ft	Scenario 4 Avail. Head ft	Scenario 4 Pressure psig
Chinook Holding Pond	2492.20	2505.62	13.42	5.82	2506.91	14.71	6.37
Spawning Building	2492.00	2506.23	14.23	6.17	2507.13	15.13	6.56

Conclusions

The above calculations document the head calculations for the four supply lines at the FCFH hatchery. The supply lines were run under various contingency conditions to determine the resilience of the pipelines to (1) pipe degradation, (2) operational changes, (3) intake shocks, and (4) minor loss calculation methods. It was found that for all cases, the available head was sufficient, though margins on the extreme contingency conditions were thin in some cases. The following tables summarize the critical locations for each of the scenarios.

Supply Line / Critical Location	Available Head (ft) Scenario 0 Base Case	Scenario 1 Degraded Pipe	Scenario 2 Degrade Pipe + Op Change	Scenario 3 Degraded Pipe + Op Change + Intake Cont.	Scenario 4 Degraded Pipe + Minor Loss Contin.
Coho Building / Incubation Head Tank	1.27	1.09	0.64	0.35	0.73
Chinook Rearing / Final Raceway	2.48	2.24	2.24	1.95	0.61
Chinook Incubation Building / Incubation Stacks	1.07	0.88	0.37	0.08	0.19
Adult Holding / Chinook Holding Pond	15.89	15.85	13.71	13.42	14.71

Supply Line / Critical Location	Pressure (psig) Scenario 0 Base Case	Scenario 1 Degraded Pipe	Scenario 2 Degrade Pipe + Op Change	Scenario 3 Degraded Pipe + Op Change + Intake Cont.	Scenario 4 Degraded Pipe + Minor Loss Contin.
Coho Building / Incubation Head Tank	0.55	0.47	0.28	0.15	0.32
Chinook Rearing / Final Raceway	1.07	0.97	0.97	0.84	0.26
Chinook Incubation Building / Incubation Stacks	0.46	0.38	0.16	0.03	0.08
Adult Holding / Chinook Holding Pond	6.89	6.87	5.94	5.82	6.37

SUBJECT: Klamath River Renewal Corporation | BY: A. Leman | CHK'D BY: N. Cox
Fall Creek Hatchery | DATE: 10/28/2020
Drain Hydraulics | PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to determine the hydraulics of the drain piping system.

References

- *Lindeburg, Michael R. 2014. Civil Engineering Reference Manual, Fourteenth Edition. Professional Publications, Inc. Belmont, CA.*

- *FHWA (Federal Highway Administration). 2012. Hydraulic Design Series Number 5, Hydraulic Design of Highway Culverts, Third Edition. U.S. Department of Transportation, FHWA. Washington, D.C. January 2012.*

Method

The drain pipeline will convey effluent from the ponds and vats to the adult holding ponds. All outlet pipes and trunk lines will be sized to maintain open-channel flow. Open channel flow calculations followed the equations below (Lindeburg, 2014), and were calculated iteratively using a Newton-Raphson iterating scheme:

$$\theta_{deg} = 2\cos^{-1}\left(\frac{\frac{D}{2} - d}{\frac{D}{2}}\right) \qquad R_h = \frac{A}{P} \qquad \frac{n}{n_{full}} = 1 + \left(\frac{d}{D}\right)^{0.54} - \left(\frac{d}{D}\right)^{1.2}$$

$$A = \left(\frac{D}{2}\right)^2 \frac{\theta_{rad} - \sin\theta_{deg}}{2} \qquad V = \left(\frac{1.486}{n}\right) R_h^{2/3} S^{1/2}$$

$$P = \frac{D\theta_{rad}}{2} \qquad Q = AV$$

where:

$\theta =$ Internal angle of water surface
$D =$ Pipe inner diameter, ft
$d =$ Flow depth, ft
$A =$ Flow area, ft^2
$P =$ Wetted perimeter, ft
$R_h =$ Hydraulic radius, ft
$V =$ Average flow velocity, ft/s
$n =$ Manning's roughness coefficient
$S =$ Pipe bed slope, ft/ft
$Q =$ Discharge, cfs
$n_{full} =$ Pipe-full roughness coefficient

At the adult holding ponds, the orifices will cause the pipe to pressurize such that sufficient head is built up to convey the flow into the ponds. The design head on the orifice will be calculated according to the orifice equation:

$$Q = C_D A_0 \sqrt{2gh}$$

$$h = \left(\frac{Q}{C_D A_0}\right)^2 \Big/ 2g$$

where:

$Q =$ Design discharge, cfs
$C_D =$ Discharge coefficient
$A_0 =$ Orifice aperture, ft^2
$g =$ Gravitational constant, 32.2 ft/s^2
$h =$ Orifice head, ft

In addition to the design head on the orifice, head losses in the pressure pipe must be accounted for. Friction losses will be calculated according to the Darcy equation:

$$h_f = f\frac{L}{D}\frac{V^2}{2g}$$

where:

$h_f =$ Friction head losses, ft
$f =$ Friction factor
$L =$ Length of full pipe run, ft
$D =$ Pipe inner diameter, ft
$V =$ Pipe average velocity, ft/s
<all other values as previously defined>

The friction factor is calculated according to the Colebrook-White equation:

$$\frac{1}{\sqrt{f}} = -2\log_{10}\left(\frac{\frac{\epsilon}{D}}{3.7} + \frac{2.51}{Re\sqrt{f}}\right)$$

where:

$\epsilon =$ Surface roughness, ft
$Re =$ Reynolds Number, VD/ν
$\nu =$ Kinematic viscosity, ft^2/s
<all other values as previously defined>

Minor losses are also accounted for in the headloss, according to the equation:

$$h_L = K\left(\frac{V^2}{2g}\right)$$

where:
$h_L =$ Minor head losses, ft
$K =$ Composite minor loss coefficient
<all other values as previously defined>

The location that the pipe starts to flow full pressure is at the elevation of the orifice plus the orifice head and all friction and minor losses:

$$z_{press} = z_o + h + h_f + h_L$$

where:
$z_{press} =$ Elevation pressure flow begins, ft
$z_o =$ Orifice elevation (free discharge), ft
<all other values as previously defined>

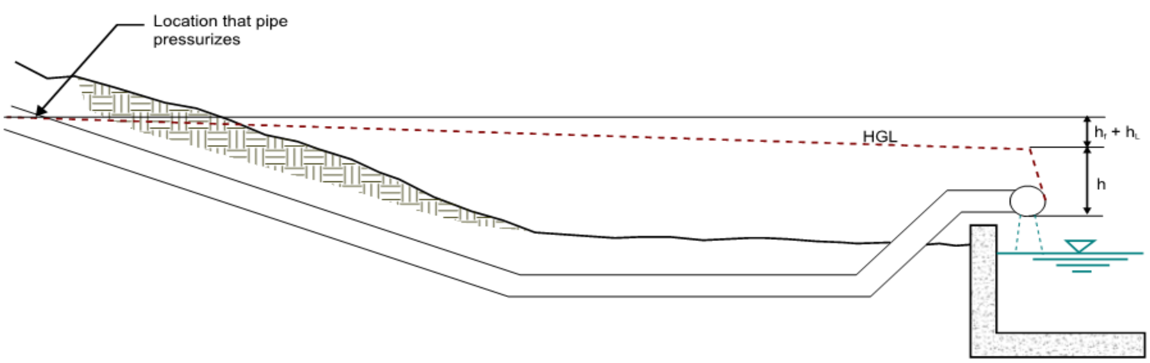


Figure 1. Pipe Downstream Schematic

Finally, the inlets were checked at the three major drain locations to determine the headwater condition at the upstream end of the pipe. Headwater depth was calculated according to Equations A.1 and A.3 from Appendix A of the FHWA Hydraulic Design Series Number 5 (HDS5; 2012), with the constants enumerated in Appendix A.

unsubmerged, circular; A.1
$$\frac{HW}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M + K_s S$$

submerged, circular; A.3
$$\frac{HW}{D} = c\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y + K_s S$$

where:
$HW =$ Headwater, ft
$D =$ Pipe inner diameter, ft
$H_c =$ Specific energy at critical depth, ft
$A =$ Culvert (full) barrel area, ft^2
$S =$ Culvert slope, ft/ft
$K_u =$ Unit conversion, 1.0 for USCS units
$K_s =$ Slope correction, -0.5
$K, M, c, Y =$ Constants, based on entrance conditions
<all other values as previously defined>

Assumptions

The following assumptions are made in these calculations:

(1) In order to allow for sufficient airflow, and to prevent periodic pressurization of the pipe where unintended, the pipe size is designed to convey the flow in an open-channel condition with the depth less than 70% of the inner diameter of the pipe, and a maximum of 75% full.

(2) The pipe is assumed to be plastic or some other smooth interior pipe, and non-profile wall pipe. Accordingly, a conservative roughness coefficient of 0.013 was applied.

(3) Based on standard sewer design, the pipe is considered self-cleaning if the velocity is greater than 2.0 ft/s. Above 1.5 ft/s is acceptable if occasional flushing flows are expected. The pipes were designed to meet this criterion.

Inputs

General Parameters

Gravitational constant, g	32.2	ft/s^2	
Kinematic Viscosity, ν	1.41E-05	ft^2/s	[@ 50 F]
Orifice Discharge Coefficient, C_D	0.62		[Lindeburg, 2014; sharp-edged, conservative]

Orifice Data

Orifice Diameter, D_o	12	in	
Orifice Diameter, D_o	1.00	ft	
Number of Ponds, N_p	3		
Number of Orifices per Pond, N	1		
Total Number of Orifices, N_0	3		
Orifice Elevation, z_o	2491.75	ft	[T.O.C. plus 3 inches]

Calculations

Gravity Pipeline

Location I.D.	Description	Discharge, Q gpm	Pipe Nom. Diameter in	Pipe Inner Diameter ft	Slope ft/ft	Roughness Coeff, *n*	Flow Depth, d ft	<70% Full?
DR1	Trunk Drain - Reach 1	420	12	0.94	0.005	0.013	0.49	53%
DR2	Trunk Drain - Reach 2	420	18	1.33	0.005	0.013	0.43	32%
DR3	Trunk Drain - Reach 3	805	18	1.33	0.005	0.013	0.60	45%
DR4	Trunk Drain - Reach 4	850	18	1.33	0.005	0.013	0.62	46%
CH1	Chinook Drain - Reach 1	4040	24	1.78	0.041	0.013	0.72	40%
DR5	Trunk Drain - Reach 5	4190	24	1.78	0.005	0.013	1.33	75%
DR6	Trunk Drain - Reach 6	4190	24	1.78	0.005	0.013	1.33	75%
DR7	Trunk Drain - Reach 7	4190	24	1.78	0.005	0.013	1.33	75%
DR8	Trunk Drain - Reach 8	4190	24	1.78	0.200	0.013	0.48	27%
DR9	Trunk Drain - Reach 9	4190	24	1.78	0.055	0.013	0.68	38%

Location I.D.	Description	Internal Angle, θ deg	Flow Area, A ft^2	Top Width, T ft	Flow Velocity, V ft/s	Self-Cleaning?	Froude Number
DR1	Trunk Drain - Reach 1	186	0.37	0.47	2.53	OK	0.50
DR2	Trunk Drain - Reach 2	138	0.39	0.62	2.43	OK	0.54
DR3	Trunk Drain - Reach 3	169	0.61	0.66	2.93	OK	0.54
DR4	Trunk Drain - Reach 4	172	0.64	0.67	2.98	OK	0.54
CH1	Chinook Drain - Reach 1	157	0.94	0.88	9.57	OK	1.63
DR5	Trunk Drain - Reach 5	239	2.01	0.78	4.65	OK	0.51
DR6	Trunk Drain - Reach 6	239	2.01	0.78	4.65	OK	0.51
DR7	Trunk Drain - Reach 7	239	2.01	0.78	4.65	OK	0.51
DR8	Trunk Drain - Reach 8	125	0.55	0.79	17.04	OK	3.61
DR9	Trunk Drain - Reach 9	152	0.87	0.87	10.77	OK	1.90

Orifice Head/Pressure Pipe

While the anticipated flow rate through the drain pipe system is equal to that of Trunk Drain Reach 6 above, the pressure pipe portion was designed for the full water right of 10 cfs, as it is critical that the pressure section not attain the elevation of the upstream ponds. Therefore, the following calculations were performed using a design discharge of 10 cfs.

Discharge, Q cfs	Orifice Aperture, A_0 ft^2	Number of Orifices, N_0	Discharge Coefficient, C_D	Head Req'ment, h ft	HGL ft
10	0.79	3	0.62	0.73	2492.48

Piping Losses

Discharge, Q cfs	Pipe Nom. Diameter in	Pipe Inner Diameter[1] in	Pipe Full Area ft^2	Velocity ft/s	Velocity Head ft	Reynolds Number	Surface Roughness in	Friction Factor[2], f
10	24	21.418	2.50	4.00	0.25	5.06E+05	6.00E-05	0.0132

Pipe Length[3] ft	Composite Minor Loss Coefficient[4] K	Major Losses ft	Minor Losses ft	Total Losses ft	HGL ft	
200	2.28	0.37	0.57	0.93	**2493.41**	<----- Location of pipe full

[1] Pipe inner diameter and surface roughness based on Schedule 80 PVC pipe.
[2] Friction factor calculated according to the Colebrook-White Equation.
[3] Pipe length is the length of pipe flowing full, based on the orifice head. This was rounded up to the nearest 100 ft based on the pipe alignment and profile.
[4] Composite minor loss coefficient was based on drain pipe layout, and includes (2) x 90 bends, (2) x 45 bend, (2) x tee (line flow), (1) x tee (branch flow), and (1) x open valve.

Inlet Control?

Location I.D.	Description	Discharge, Q gpm	Discharge, Q cfs	Nominal Diameter in	Inner Diameter ft	Culvert Barrel Area, A ft^2	Culvert Barrel Slope, S ft/ft
C1	Existing Coho	420	0.9	12	0.94	0.70	0.005
C2	Coho Raceway Bank 2	345	0.8	12	0.94	0.70	0.005
CH1	Chinook Raceways	4040	9.0	24	1.78	2.50	0.022

Location I.D.	Description	Critical Depth, d_c ft	Critical Spec Energy, H_c ft	Unit Conversion K_u	Slope Correction K_s	Constant[1] K	Constant[1] M	Constant[1] c	Constant[1] Y
C1	Existing Coho	0.41	0.62	1	-0.5	0.0098	2.0	0.0398	0.67
C2	Coho Raceway Bank 2	0.37	0.56	1	-0.5	0.0098	2.0	0.0398	0.67
CH1	Chinook Raceways	1.11	1.66	1	-0.5	0.0098	2.0	0.0398	0.67

Location I.D.	Description	Headwater Ratio, HW/D	Sub-merged?	>70%?	Sub-merged HW/D
C1	Existing Coho	67%	NO	NO	-
C2	Coho Raceway Bank 2	60%	NO	NO	-
CH1	Chinook Raceways	99%	NO	YES	-

[1] Constants taken from HDS-5 Appendix A, Table A.1 based on circular pipe in headwall.

Conclusions

The above calculations provide a set of flow, slope, and pipe size conditions that will maintain gravity flow in the drain pipes. It is likewise found that the orifice is expected to back flow up to elevation 2496.37, which is well below the lowest pond elevation and should not pose a concern for backing up the ponds. This elevation also provides an expected location upstream of which venting of the drain pipe will be required.

Finally, the entrance conditions were checked at the three major inlets to the drain system. It was found that the headwater was less than 70% of the pipe diameter for the Coho inlets, and therefore no modifications would be required. The Chinook raceways, on the other hand, have a headwater nearly equal to the pipe diameter, and therefore a vent pipe will be needed downstream if the pipe to provide adequate airflow downstream of the entrance condition.

SUBJECT: Klamath River Renewal Corporation | **BY:** A. Leman | **CHK'D BY:** N. Cox
Fall Creek Hatchery | **DATE:** 10/28/2020
Waste Drain Hydraulics | **PROJECT NO.:** 20-024

Purpose

The purpose of this calculation sheet is to determine the hydraulics of the waste drain piping system.

References

- *Lindeburg, Michael R. 2014. Civil Engineering Reference Manual, Fourteenth Edition. Professional Publications, Inc. Belmont, CA.*

Method

The waste stream pipeline will convey flushing flows from the ponds and vats to the settling pond in the existing lower raceway bank. All outlet pipes will be sized to maintain open-channel flow. Open channel flow calculations followed the equations below (Lindeburg, 2014), and were calculated iteratively using a Newton-Raphson iterating scheme:

$$\theta_{deg} = 2\cos^{-1}\left(\frac{\frac{D}{2} - d}{\frac{D}{2}}\right)$$

$$R_h = \frac{A}{P}$$

$$\frac{n}{n_{full}} = 1 + \left(\frac{d}{D}\right)^{0.54} - \left(\frac{d}{D}\right)^{1.2}$$

$$A = \left(\frac{D}{2}\right)^2 \frac{\theta_{rad} - \sin\theta_{deg}}{2}$$

$$V = \left(\frac{1.486}{n}\right) R_h^{2/3} S^{1/2}$$

$$P = \frac{D\theta_{rad}}{2}$$

$$Q = AV$$

where:

$\theta =$ Internal angle of water surface
$D =$ Pipe inner diameter, ft
$d =$ Flow depth, ft
$A =$ Flow area, ft^2
$P =$ Wetted perimeter, ft
$R_h =$ Hydraulic radius, ft
$V =$ Average flow velocity, ft/s
$n =$ Manning's roughness coefficient
$S =$ Pipe bed slope, ft/ft
$Q =$ Discharge, cfs
$n_{full} =$ Pipe-full roughness coefficient

Assumptions

The following assumptions are made in these calculations:

(1) In order to allow for sufficient airflow, and to prevent periodic pressurization of the pipe where unintended, the pipe size is designed to convey the flow in an open-channel condition with the depth less than 70% of the inner diameter of the pipe, and a maximum of 75% full.

(2) The pipe is assumed to be plastic or some other smooth interior pipe, and non-profile wall pipe. Accordingly, a conservative roughness coefficient of 0.013 was applied.

(3) Based on standard sewer design, the pipe is considered self-cleaning if the velocity is greater than 2.0 ft/s. Above 1.5 ft/s is acceptable if occasional flushing flows are expected. The pipes were designed to meet this criterion.

Inputs

It is assumed that each raceway/pond/vat will be cleaned using a vacuum system that will connect to a riser pipe for each of the design points, via cam-lock. As such, the maximum flow in any pipe (outlet or trunk line) at any given time will be 200 gpm.

Design Discharge, Q	200	gpm
	0.45	cfs

Calculations

Because the design discharge is the same for all of the pipes, design pipe sizes were determined as a function of the slope condition, such that the drain pipe sizing could be calculated for any given location:

Description	Discharge, Q gpm	Pipe Nom. Diameter in	Pipe Inner Diameter ft	Slope ft/ft	Roughness Coeff, *n*	Flow Depth, d ft	<70% Full?
0.5% Slope	200	8	0.630	0.005	0.013	0.40	63%
1.0% Slope	200	8	0.630	0.010	0.013	0.33	52%
1.5% Slope	200	8	0.630	0.015	0.013	0.29	46%
2.0% Slope	200	8	0.630	0.020	0.013	0.27	43%
2.5% Slope	200	8	0.630	0.025	0.013	0.26	41%
3.0% Slope	200	8	0.630	0.030	0.013	0.24	39%
4.0% Slope	200	8	0.630	0.040	0.013	0.23	36%
5.0% Slope	200	8	0.630	0.050	0.013	0.21	34%
10.0% Slope	200	8	0.630	0.100	0.013	0.18	28%

Description	Internal Angle, θ deg	Flow Area, A ft^2	Flow Velocity, V ft/s	Self-Cleaning?	Top Width, T ft	Froude Number
0.5% Slope	211	0.21	2.14	OK	0.61	0.64
1.0% Slope	185	0.16	2.72	OK	0.63	0.94
1.5% Slope	172	0.14	3.13	OK	0.63	1.16
2.0% Slope	164	0.13	3.47	OK	0.62	1.35
2.5% Slope	158	0.12	3.75	OK	0.62	1.51
3.0% Slope	154	0.11	4.00	OK	0.61	1.66
4.0% Slope	147	0.10	4.44	OK	0.60	1.92
5.0% Slope	142	0.09	4.80	OK	0.60	2.15
10.0% Slope	128	0.07	6.16	OK	0.57	3.04

Conclusions

The above pipe sizes were calculated for the waste drain pipes used for cleaning the ponds and vats which report to the settling pond in the lower bank of existing raceways. Appropriate pipe sizes that maintain gravity flow and are self-cleaning, were calculated for slopes from 0.5% to 10% as a design aid for sizing the drain pipes based on profile requirements.

SUBJECT:	Klamath River Renewal Corporation	BY:	A. Leman	CHK'D BY:	V. Autier
	Fall Creek Hatchery	DATE:	10/28/2020		
	Volitional Release Pipes	PROJECT NO.:	20-024		

Purpose

The purpose of this calculation sheet is to document the design of the three (3) fish volitional release pipes.

References

- *NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.*
- *USFWS (U.S. Fish and Wildlife Service). 2017. Fish Passage Engineering Design Criteria. USFWS, Northeast Region R5, Hadley, MA.*

Design Criteria

The NMFS (2011) criteria for a fish bypass pipe are summarized below:

NMFS Guidelines	Value		Comments
Flow Regime	Open-Channel		NMFS 11.9.3.2 and 11.9.3.3
	No Hydraulic Jump		NMFS 11.9.3.12
Minimum Bend Radius (R/D)	5.0		NMFS 11.9.3.4 (greater for super-critical velocities)
Minimum Pipe Diameter	10.0	in	NMFS Table 11-1
Typical Access Port Spacing	150	ft	NMFS 11.9.3.5
Minimum Bypass Flow	5%		NMFS 11.9.3.7 (5% of diverted flow)
Maximum Pipe Velocity	12	ft/s	NMFS 11.9.3.8
Minimum Pipe Velocity	2	ft/s	NMFS 11.9.3.8 (6 ft/s recommended, 2 ft/s absolute where sedimentation is a concern)
Minimum Depth (d/D)	40%		NMFS 11.9.3.9 (percentage of pipe diameter); absolute > 2 in
Valves	None		NMFS 11.9.3.10

The NMFS (2011) criteria for a bypass outfall are summarized below:

NMFS Guidelines	Value		Comments
Location	Minimizes Predation		NMFS 11.9.4.1
	No eddies, reverse flow, predators		NMFS 11.9.4.1
Minimum Ambient River Velocities	4	ft/s	NMFS 11.9.4.1
Pool Depth	Not impact bottom		NMFS 11.9.4.1
Maximum Impact Velocity	25	ft/s	NMFS 11.9.4.2
Must be designed to avoid adult attraction			NMFS 11.9.4.3

Method

Open Channel Hydraulics

Fish pipe hydraulics were calculated according to standard open channel flow equations in a circular pipe:

$$\theta_{deg} = 2\cos^{-1}\left(\frac{\frac{D}{2}-d}{\frac{D}{2}}\right)$$

$$\frac{n}{n_{full}} = 1 + \left(\frac{d}{D}\right)^{0.54} - \left(\frac{d}{D}\right)^{1.2}$$

$$A = \left(\frac{D}{2}\right)^2 \frac{\theta_{rad} - \sin\theta_{deg}}{2}$$

$$P = \frac{D\theta_{rad}}{2}$$

$$R_h = \frac{A}{P}$$

$$V = \left(\frac{1.486}{n}\right) R_h^{2/3} S^{1/2}$$

$$Q = AV$$

where:

$\theta =$	Internal angle of water surface
$D =$	Pipe inner diameter, ft
$d =$	Flow depth, ft
$A =$	Flow area, ft^2
$P =$	Wetted perimeter, ft
$R_h =$	Hydraulic radius, ft
$V =$	Average flow velocity, ft/s
$n =$	Manning's roughness coefficient
$S =$	Pipe bed slope, ft/ft
$Q =$	Discharge, cfs
$n_{full} =$	Pipe-full roughness coefficient

Calculations were performed iteratively using a Newton-Raphson iterating scheme.

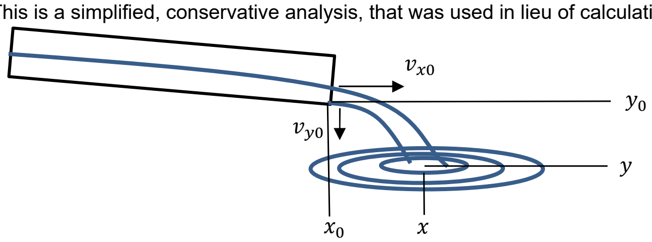
Fish Bypass Pipe

• The fish bypass pipe was sized to meet the minimum depth criterion (40% of the inner diameter), while also ensuring that the pipe would not pressurize. In order to ensure open channel flow, the water surface was generally maintained less than 70% of the pipe diameter, and strictly less than 75%.
• The Coho fish release pipes have a much smaller flow-through discharge and therefore, it was assumed that the full discharge through the Coho raceways would be directed to Fall Creek at volitional fish release.
• The Chinook fish release pipes will be operated while still maintaining flow down to the adult holding ponds at volitional fish release. Therefore, an operational flow range was selected that would be diverted to fish release, and the remainder will be directed to adult holding ponds, based on the placement/removal of stoplogs (see "Chinook Outlet" calculations). The operational flow range was maintained within the same 40% - 75% of the pipe inner diameter for volitional release.

• The adult holding fish release pipe will be operated to drain the Coho and Chinook holding ponds. These can be hydraulically connected to the trapping and sorting pond, and therefore could see a range of flows from 6.6 cfs - 10 cfs. This is considered the operational range for the volitional release pipe. The operational flow range was maintained within the same 40% - 75% of the pipe inner diameter for volitional release.
• Velocities were subsequently checked to ensure that they are maintained within the NMFS guidelines for fish bypass pipes.

Plunge Pool

• The plunge pool impact velocity was calculated according to basic kinematic equations. The impact velocity was calculated at the water surface, and at the bottom of the pool. If both of these locations are less than the critical impact velocity, it was deemed that the criterion was met. This is a simplified, conservative analysis, that was used in lieu of calculating hydraulics of the jet in the plunge pool.



$$y = y_0 + v_{y0} t_i + \frac{1}{2} a_y t_i^2$$
$$x = x_0 + v_{x0} t_i$$

where:
$a_y =$ Acceleration in y-direction, 32.2 ft/s^2
$t_i =$ Time to impact, s

Inlet Control

The inlets were checked at the three fish release pipe locations to determine the headwater condition at the upstream end of the pipe. Headwater depth was calculated according to Equations A.1 and A.3 from Appendix A of the FHWA Hydraulic Design Series Number 5 (HDS5; 2012), with the constants enumerated in Appendix A.

unsubmerged, circular; A.1 $$\frac{HW}{D} = \frac{H_c}{D} + K\left[\frac{K_u Q}{AD^{0.5}}\right]^M + K_s S$$

submerged, circular; A.3 $$\frac{HW}{D} = c\left[\frac{K_u Q}{AD^{0.5}}\right]^2 + Y + K_s S$$

where:
$HW =$ Headwater, ft
$D =$ Pipe inner diameter, ft
$H_c =$ Specific energy at critical depth, ft
$A =$ Culvert (full) barrel area, ft^2
$S =$ Culvert slope, ft/ft
$K_u =$ Unit conversion, 1.0 for USCS units
$K_s =$ Slope correction, -0.5
$K, M, c, Y =$ Constants, based on entrance conditions
<all other values as previously defined>

Inputs

The following inputs were used for the design of the fish bypass pipe and outfall:

Inputs (Chinook)	Value		Comments
Maximum outflow	4.5	cfs	50% of the Chinook pond outflow
Minimum outflow	2.6	cfs	~25% of the Chinook pond outflow
Outfall Pipe Invert Elevation	2495.4	ft	Selected, based on pipe routing and 100-year TW
Pool Bottom Elevation	2489.4	ft	Selected, Min pool depth 3.0'
100-year Tailwater Elevation	2494.5	ft	HEC-RAS Model
High Tailwater Elevation	2492.9	ft	March 1% Exceedance Flow
Low Tailwater Elevation	2492.4	ft	May 95% Exceedance Flow
Pipe Material	HDPE		butt welded for smooth interior
Pipe Dimension Ratio	26		From Civil Calculations
Gravitational Constant	32.2	ft/s^2	

Inputs (Coho)	Value		Comments
Outflow (New ponds)	0.77	cfs	2 ponds x 172 gpm/pond
Outflow (New ponds + Exist)	1.70	cfs	New ponds + 2 ponds x 210 gpm/pond
Existing Conc Flume Width	4	ft	Measured in survey
Pool Bottom Elevation	2494.93	ft	Measured in survey
100-year Tailwater Elevation	2498.26	ft	HEC-RAS Model
High Tailwater Elevation	2496.46		March 1% Exceedance Flow
Low Tailwater Elevation	2495.98		May 95% Exceedance Flow
Pipe Material	HDPE		butt welded for smooth interior
Pipe Dimension Ratio	26		From Civil Calculations

Inputs (Adult Holding)	Value		Comments
Maximum outflow	10	cfs	Full flow - 3 ponds
Minimum outflow	6.6	cfs	Full flow - 2 ponds
100-year Tailwater Elevation	2487.21	ft	HEC-RAS Model
High Tailwater Elevation	2484.77	ft	March 1% Exceedance Flow
Low Tailwater Elevation	2484.12	ft	May 95% Exceedance Flow = Oct-Dec Fish Passage Low Flow
Pool Bottom Elevation	2482.07	ft	See Denil Fishway Calculations
Pipe Inlet Elevation	2486.5	ft	See Denil Fishway Calculations
Pipe Outlet Elevation	2486.25	ft	Input
Pipe Material	HDPE		butt welded for smooth interior
Pipe Dimension Ratio	26		From Civil Calculations

Calculations

Chinook Fish Release

Bypass Pipe Calculations

The following table was used as a design aid for the fish release pipe design:

Pipe Nominal Diameter in	Pipe Inner Diameter ft	Manning's Rough Coefficient	Discharge cfs	Slope ft/ft	Flow Depth ft	% Full	Flow Velocity ft/s	Froude Number
20	1.54	0.013	4.5	0.005	0.94	61%	3.80	0.75
20	1.54	0.013	2.6	0.005	0.69	45%	3.22	0.78
16	1.23	0.013	4.5	0.01	0.87	71%	5.00	0.98
16	1.23	0.013	2.6	0.01	0.63	51%	4.22	1.05
16	1.23	0.013	4.5	0.015	0.77	63%	5.75	1.25
16	1.23	0.013	2.6	0.015	0.57	46%	4.87	1.30
14	**1.08**	**0.013**	**4.5**	**0.02**	**0.77**	**71%**	**6.49**	**1.36**
14	**1.08**	**0.013**	**2.6**	**0.02**	**0.56**	**52%**	**5.48**	**1.45**
14	1.08	0.013	4.5	0.03	0.68	63%	7.46	1.73
14	1.08	0.013	2.6	0.03	0.50	46%	6.31	1.80
12	0.98	0.013	4.5	0.04	0.66	67%	8.36	1.93
12	0.98	0.013	2.6	0.04	0.48	49%	7.07	2.03
12	**0.98**	**0.013**	**4.5**	**0.06**	**0.58**	**59%**	**9.62**	**2.43**
12	**0.98**	**0.013**	**2.6**	**0.06**	**0.43**	**44%**	**8.15**	**2.51**
12	0.98	0.013	4.5	0.07	0.56	57%	10.14	2.65
12	0.98	0.013	2.6	0.07	0.41	42%	8.61	2.72
10	0.83	0.013	4.5	0.1	0.55	67%	11.79	2.97
10	0.83	0.013	2.6	0.1	0.40	49%	9.97	3.13
10	0.83	0.013	4.5	0.15	0.49	59%	13.56	3.74
10	0.83	0.013	2.6	0.15	0.36	44%	11.49	3.86
10	0.83	0.013	4.5	0.2	0.45	55%	14.98	4.37
10	0.83	0.013	2.6	0.2	0.34	41%	12.73	4.47

** red indicates outside of 40% - 70% full range, and only occurs where standard pipe sizes above the minimum cannot accommodate the operational flow range within those recommended water depths.*

McMILLEN JACOBS ASSOCIATES

Plunge Pool Calculations

Scenario	Pipe Outfall Velocity, V (ft/s)	Initial Velocity, V_x (ft/s)	Initial Velocity, V_y (ft/s)	Pipe Elevation (ft)	Tailwater Elevation (ft)	Drop Height (ft)	Drop to Bottom of Pool (ft)
Lo Release, Lo TW	5.48	5.48	0.11	2495.4	2492.4	3.0	6.0
Lo Release, Hi TW	5.48	5.48	0.11	2495.4	2492.9	2.5	6.0
Hi Release, Lo TW	6.49	6.49	0.13	2495.4	2492.4	3.0	6.0
Hi Release, Hi TW	6.49	6.49	0.13	2495.4	2492.9	2.5	6.0

Scenario	Pool Depth (ft)	Min Pool Depth	Pool Depth Factor of Safety	Time to Impact WSEL (s)	Time to Impact Bottom* (s)	Impact Velocity at WSEL (ft/s)	Impact Velocity at Bottom* (ft/s)	x-distance to WSEL Impact (ft)
Lo Release, Lo TW	3.0	0.76	4.0	0.4	0.6	15.00	20.45	2.36
Lo Release, Hi TW	3.5	0.63	5.5	0.4	0.6	13.89	20.45	2.15
Hi Release, Lo TW	3.0	0.76	4.0	0.4	0.6	15.40	20.75	2.79
Hi Release, Hi TW	3.5	0.63	5.5	0.4	0.6	14.32	20.75	2.55

**Note: impact velocity calculated at the bottom of the pool as the maximum possible impact velocity. It is demonstrated, that the bypass flow does not impact the bottom, but rather the water surface a minimum of 3.0' above the pool bottom.*

Inlet Control Calculations

Condition	Discharge, Q (gpm)	Discharge, Q (cfs)	Nominal Diameter (in)	Inner Diameter (ft)	Culvert Barrel Area, A (ft^2)	Culvert Barrel Slope, S (ft/ft)	Critical Depth, d_c (ft)	Critical Spec Energy, H_c (ft)
Low Flow	1,167	2.6	24	1.85	2.68	0.06	0.574	0.86
Hi Flow	2,020	4.5	24	1.85	2.68	0.06	0.763	1.14

Condition	Unit Conversion K_u	Slope Correction K_s	Constant[1] K	Constant[1] M	Constant[1] c	Constant[1] Y	Headwater Ratio, HW/D	Sub-merged?	>70%?	Sub-merged HW/D
Low Flow	1.0	-0.5	0.0098	2.0	0.0398	0.67	0.47	NO	47%	-
Hi Flow	1.0	-0.5	0.0098	2.0	0.0398	0.67	0.63	NO	63%	-

[1] *Constants taken from HDS-5 Appendix A, Table A.1 based on circular pipe in headwall.*

The above calculations were performed to determine the pipe size at which the HDPE pipe is no longer inlet controlled, and flow can remain open-channel. The pipe will then reduce to the nominal pipeline diameter selected above.

Coho Fish Release

Bypass Pipe Calculations

The following table was used as a design aid for the fish release pipe design:

Pipe Nominal Diameter in	Pipe Inner Diameter ft	Manning's Rough Coefficient	Discharge cfs	Slope ft/ft	Flow Depth ft	% Full	Flow Velocity ft/s	Froude Number
10	0.83	0.013	0.77	0.005	0.47	57%	2.42	0.69
10	**0.83**	**0.013**	**0.77**	**0.01**	**0.39**	**47%**	**3.09**	**0.99**
10	0.83	0.013	0.77	0.015	0.35	42%	3.56	1.22
10	0.83	0.013	0.77	0.02	0.32	39%	3.94	1.42
10	0.83	0.013	0.77	0.025	0.30	37%	4.27	1.59
10	0.83	0.013	0.77	0.04	0.27	33%	5.05	2.01
10	0.83	0.013	0.77	0.06	0.24	29%	5.83	2.46

** red indicates outside of 40% - 70% full range, and only occurs where standard pipe sizes above the minimum cannot accommodate the operational flow range within those recommended water depths.*

The bypass pipe will terminate in the existing concrete outlet flume on the existing upper concrete raceways, which will convey fish to Fall Creek. The water surfaces of interest in this area are as follows:

Existing Conc Flume Invert	2498.4	ft
Pipe Invert Elevation	2499.61	ft
100-year Flood Elevation	2498.26	ft
Dam Board Normal Elevation	2502.2	ft
Dam Board Vol Release Elevation	2499.35	ft

Plunge Pool Calculations

The release to the stream will be at the location of existing fish release from the existing facility. No constructed plunge pool is expected for this site.

Inlet Control Calculations

Condition	Discharge, Q gpm	Discharge, Q cfs	Nominal Diameter in	Inner Diameter ft	Culvert Barrel Area, A ft^2	Culvert Barrel Slope, S ft/ft	Critical Depth, d_c ft	Critical Spec Energy, H_c ft
Low Flow	344	0.77	10	0.83	0.54	0.01	0.387	0.58

Condition	Unit Conversion K_u	Slope Correction K_s	Constant[1] K	Constant[1] M	Constant[1] c	Constant[1] Y	Headwater Ratio, HW/D	Sub-merged?	>70%?	Sub-merged HW/D
Low Flow	1.0	-0.5	0.0098	2.0	0.0398	0.67	0.73	NO	73%	-

[1] Constants taken from HDS-5 Appendix A, Table A.1 based on circular pipe in headwall.

Because the pipe diameter is able to accommodate the flow with more than 25% of the pipe diameter open for air flow, and because the pipe length is so short with an open end, it is deemed appropriate that no ventilation of this pipe is required.

Adult Holding Fish Release

Bypass Pipe Calculations

The following table was used as a design aid for the fish release pipe design:

Pipe Nominal Diameter in	Pipe Inner Diameter ft	Manning's Rough Coefficient	Discharge cfs	Slope ft/ft	Flow Depth ft	% Full	Flow Velocity ft/s	Froude Number
30	2.31	0.013	6.60	0.005	0.96	41%	4.03	0.84
30	2.31	0.013	10.00	0.005	1.20	52%	4.56	0.82
24	1.85	0.013	6.60	0.01	0.87	47%	5.29	1.13
24	1.85	0.013	10.00	0.01	1.10	60%	6.00	1.10
24	1.85	0.013	6.60	0.015	0.78	42%	6.10	1.40
24	1.85	0.013	10.00	0.015	0.98	53%	6.91	1.37
20	1.54	0.013	6.60	0.02	0.79	51%	6.91	1.54
20	1.54	0.013	10.00	0.02	1.00	65%	7.85	1.48
18	1.38	0.013	6.60	0.03	0.74	53%	8.07	1.85
18	1.38	0.013	10.00	0.03	0.94	68%	9.18	1.76
18	1.38	0.013	6.60	0.04	0.68	49%	8.93	2.15
18	1.38	0.013	10.00	0.04	0.86	62%	10.13	2.08
18	1.38	0.013	6.60	0.06	0.61	44%	10.30	2.66
18	1.38	0.013	10.00	0.06	0.77	55%	11.66	2.60
18	1.38	0.013	6.60	0.07	0.59	42%	10.87	2.88
18	1.38	0.013	10.00	0.07	0.74	53%	12.30	2.83
16	1.23	0.013	6.60	0.1	0.56	46%	12.50	3.36
16	1.23	0.013	10.00	0.1	0.71	57%	14.17	3.28
14	1.08	0.013	6.60	0.15	0.53	50%	14.66	4.00
14	1.08	0.013	10.00	0.15	0.67	63%	16.64	3.86
14	1.08	0.013	6.60	0.2	0.49	46%	16.22	4.64
14	1.08	0.013	10.00	0.2	0.62	58%	18.38	4.53

** red indicates outside of 40% - 70% full range, and only occurs where standard pipe sizes above the minimum cannot accommodate the operational flow range within those recommended water depths.*

Ultimately, based on a sensitivity analysis, a 30" dia. pipe at slope 1.6% was selected, such that pipes would not be submerged, but a minimum of 6.0 ft/s could be maintained in the pipe:

30	2.31	0.013	6.60	0.016	0.70	31%	6.10	1.51
30	2.31	0.013	10.00	0.016	0.88	38%	6.88	1.50

It is recognized that the 40% full criterion is not met, however, the minimum 6.0 ft/s velocity criterion was deemed more important, such that fish cannot hold in the release pipe.

Plunge Pool

The adult holding fish release pipe will discharge to the entrance pool at the toe of the Denil fishway. The following calculations are performed for the impact velocity at this location.

Scenario	Pipe Outfall Velocity, V ft/s	Initial Velocity, V_x ft/s	Initial Velocity, V_y ft/s	Pipe Elevation ft	Tailwater Elevation ft	Drop Height ft	Drop to Bottom of Pool ft
Lo Release, Lo TW	6.10	6.10	0.12	2486.25	2484.12	2.1	4.2
Lo Release, Hi TW	6.10	6.10	0.12	2486.25	2484.77	1.5	4.2
Hi Release, Lo TW	6.88	6.88	0.14	2486.25	2484.12	2.1	4.2
Hi Release, Hi TW	6.88	6.88	0.14	2486.25	2484.77	1.5	4.2

Scenario	Pool Depth ft	Min Pool Depth	Pool Depth Factor of Safety	Time to Impact WSEL s	Time to Impact Bottom* s	Impact Velocity at WSEL ft/s	Impact Velocity at Bottom* ft/s	x-distance to WSEL Impact ft
Lo Release, Lo TW	2.05	0.53	3.85	0.4	0.5	13.21	17.51	2.20
Lo Release, Hi TW	2.70	0.37	7.30	0.3	0.5	11.51	17.51	1.83
Hi Release, Lo TW	2.05	0.53	3.85	0.4	0.5	13.58	17.79	2.47
Hi Release, Hi TW	2.70	0.37	7.30	0.3	0.5	11.94	17.79	2.06

Inlet Control Calculations

Condition	Discharge, Q gpm	Discharge, Q cfs	Nominal Diameter in	Inner Diameter ft	Culvert Barrel Area, A ft^2	Culvert Barrel Slope, S ft/ft	Critical Depth, d_c ft	Critical Spec Energy, H_c ft
Low Flow	2,962	6.60	30	2.31	4.18	0.016	0.871	1.31
Hi Flow	4,488	10.00	30	2.31	4.18	0.016	1.082	1.62

Condition	Unit Conversion K_u	Slope Correction K_s	Constant[1] K	Constant[1] M	Constant[1] c	Constant[1] Y	Headwater Ratio, HW/D	Sub-merged?	>70%?	Sub-merged HW/D
Low Flow	1.0	-0.5	0.0098	2.0	0.0398	0.67	0.58	NO	58%	-
Hi Flow	1.0	-0.5	0.0098	2.0	0.0398	0.67	0.73	NO	73%	-

[1] *Constants taken from HDS-5 Appendix A, Table A.1 based on circular pipe in headwall.*

The inlet control conditions were calculated for the 30" diameter HDPE pipe, and it was found that the pipe inlet would not submerge. At the full 10 cfs, it was found that the pipe at the inlet would flow just barely above 70%, and it was deemed that the 30" diameter pipe was appropriate without any necessary venting.

Conclusions

The above calculations document the design of the fish release pipes and plunge pools in Fall Creek, and demonstrate that the fish release pipes follow recommendations/guidelines from NMFS. It should be noted, however, that both the Chinook volitional release pipe and the adult holding volitional release pipe were designed for a specific flow range, and should only be operated within those parameters at fish release.

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Volitional Release Pipe Sensitivity Analysis	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to determine the sensitivity of the fish release pipes to the slope and roughness coefficient.

Method

These calculations will show the sensitivity of the fish release pipe hydraulics, particularly the pipe velocity, to the slope and to the selected Manning's roughness coefficient:

1. **Slope** - Due to potential for field deviations in elevation, the pipe slopes were varied by allowing the downstream pipe elevation to vary by +/- 3 inches. This would be equivalent to the upstream pipe velocity varying by the same amount. Hydraulic calculations were performed for the whole range of values to determine key hydraulic characteristics.

2. **Manning's Roughness** - In the initial pipe sizing, pipes were assigned an unadjusted roughness coefficient of 0.013 and were then adjusted iteratively with the flow depth in the pipe. The unadjusted roughness coefficient of 0.013 represents a conservative estimate after the pipe has already undergone some degree of fouling or other degradation. The Manning's roughness coefficient in this sensitivity analysis will be allowed to vary from 0.009 to 0.015 (typical limits for PE pipe) to see the effects on the pipe hydraulics.

Inputs

Inputs (Chinook)	Value		Comments
Maximum outflow	4.5	cfs	50% of the Chinook pond outflow
Minimum outflow	2.6	cfs	~25% of the Chinook pond outflow
Pipe 1			
Nominal Diameter	12	in	See original pipe design calculations
Pipe DR	26		See original pipe design calculations
Pipe O.D.	12.75	in	Standard size
Pipe I.D.	11.77	in	Calculated
Pipe U/S Elevation	2498.93	ft	Pipe design
Pipe D/S Elevation	2497.07	ft	Pipe design
Pipe Length	30.8	ft	Pipe design
Pipe Slope	6.0%		Calculated
Pipe 2			
Nominal Diameter	14	in	See original pipe design calculations
Pipe DR	26		See original pipe design calculations
Pipe O.D.	14	in	Standard size
Pipe I.D.	12.92	in	Calculated
Pipe U/S Elevation	2497.07	ft	Pipe design
Pipe D/S Elevation	2495.24	ft	Pipe design
Pipe Length	91	ft	Pipe design
Pipe Slope	2.0%		Calculated

Inputs (Coho)	Value		Comments
Outflow (New ponds)	0.77	cfs	2 ponds x 172 gpm/pond
Nominal Diameter	10	in	See original pipe design calculations
Pipe DR	26		See original pipe design calculations
Pipe O.D.	10.75	in	Standard size
Pipe I.D.	9.92	in	Calculated
Pipe U/S Elevation	2500.5	ft	Pipe design
Pipe D/S Elevation	2500.16	ft	Pipe design
Pipe Length	38.63	ft	Pipe design
Pipe Slope	0.9%		Calculated

Inputs (Adult Holding)	Value		Comments	
Maximum outflow	10	cfs	Full flow - 3 ponds	
Minimum outflow	6.6	cfs	Full flow - 2 ponds	
Nominal Diameter	30	in	See original pipe design calculations	
Pipe DR	26		See original pipe design calculations	
Pipe O.D.	30	in	Standard size	
Pipe I.D.	27.69	in	Calculated	
Pipe U/S Elevation	2486.5	ft	Pipe design	2486.3402
Pipe D/S Elevation	2486.25	ft	Pipe design	2486.43
Pipe Length	15.98	ft	Pipe design	
Pipe Slope	1.6%		Calculated	

McMILLEN JACOBS ASSOCIATES

Calculations

Chinook Pipes (Slope Change)

Chinook, Pipe 1, Max Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2496.82	0.069	4.5	0.98	0.56	10.07	57%	2.61
- 2"	2496.90	0.066	4.5	0.98	0.57	9.93	58%	2.56
- 1"	2496.99	0.063	4.5	0.98	0.57	9.78	59%	2.50
As Designed	2497.07	0.060	4.5	0.98	0.58	9.64	59%	2.44
+1"	2497.15	0.058	4.5	0.98	0.59	9.49	60%	2.38
+ 2"	2497.24	0.055	4.5	0.98	0.60	9.33	61%	2.32
+3"	2497.32	0.052	4.5	0.98	0.61	9.17	62%	2.25

Chinook, Pipe 1, Min Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2496.82	0.069	2.6	0.98	0.42	8.54	42%	2.69
- 2"	2496.90	0.066	2.6	0.98	0.42	8.42	43%	2.63
- 1"	2496.99	0.063	2.6	0.98	0.42	8.30	43%	2.58
As Designed	2497.07	0.060	2.6	0.98	0.43	8.17	44%	2.52
+1"	2497.15	0.058	2.6	0.98	0.43	8.04	44%	2.46
+ 2"	2497.24	0.055	2.6	0.98	0.44	7.91	45%	2.40
+3"	2497.32	0.052	2.6	0.98	0.45	7.77	46%	2.34

Chinook, Pipe 2, Max Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2494.99	0.023	4.5	1.08	0.74	6.79	68%	1.47
- 2"	2495.07	0.022	4.5	1.08	0.74	6.70	69%	1.44
- 1"	2495.16	0.021	4.5	1.08	0.75	6.60	70%	1.40
As Designed	2495.24	0.020	4.5	1.08	0.77	6.50	71%	1.36
+1"	2495.32	0.019	4.5	1.08	0.78	6.40	72%	1.32
+ 2"	2495.41	0.018	4.5	1.08	0.79	6.29	73%	1.28
+3"	2495.49	0.017	4.5	1.08	0.80	6.18	75%	1.24

Chinook, Pipe 2, Min Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2494.99	0.023	2.6	1.08	0.54	5.74	50%	1.56
- 2"	2495.07	0.022	2.6	1.08	0.54	5.66	50%	1.53
- 1"	2495.16	0.021	2.6	1.08	0.55	5.57	51%	1.49
As Designed	2495.24	0.020	2.6	1.08	0.56	5.49	52%	1.46
+1"	2495.32	0.019	2.6	1.08	0.56	5.40	52%	1.42
+ 2"	2495.41	0.018	2.6	1.08	0.57	5.31	53%	1.39
+3"	2495.49	0.017	2.6	1.08	0.58	5.21	54%	1.35

Chinook Pipes (Manning's Coeff Change)

Chinook, Pipe 1, Max Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.060	4.5	0.98	0.47	12.45	48%	3.61
0.010	0.010	0.060	4.5	0.98	0.50	11.57	51%	3.24
0.011	0.011	0.060	4.5	0.98	0.53	10.82	54%	2.92
0.012	0.012	0.060	4.5	0.98	0.56	10.19	57%	2.66
As Designed (0.013)	0.013	0.060	4.5	0.98	0.58	9.64	59%	2.44
0.014	0.014	0.060	4.5	0.98	0.61	9.16	62%	2.25
0.015	0.015	0.060	4.5	0.98	0.63	8.73	65%	2.08

Chinook, Pipe 1, Min Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.060	2.6	0.98	0.35	10.61	36%	3.66
0.010	0.010	0.060	2.6	0.98	0.37	9.84	38%	3.29
0.011	0.011	0.060	2.6	0.98	0.39	9.20	40%	2.99
0.012	0.012	0.060	2.6	0.98	0.41	8.65	42%	2.73
As Designed (0.013)	0.013	0.060	2.6	0.98	0.43	8.17	44%	2.52
0.014	0.014	0.060	2.6	0.98	0.45	7.76	46%	2.33
0.015	0.015	0.060	2.6	0.98	0.46	7.39	47%	2.17

Chinook, Pipe 2, Max Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.020	4.5	1.08	0.61	8.37	57%	2.08
0.010	0.010	0.020	4.5	1.08	0.65	7.79	61%	1.85
0.011	0.011	0.020	4.5	1.08	0.69	7.29	64%	1.66
0.012	0.012	0.020	4.5	1.08	0.73	6.87	68%	1.50
As Designed (0.013)	0.013	0.020	4.5	1.08	0.77	6.50	71%	1.36
0.014	0.014	0.020	4.5	1.08	0.80	6.18	75%	1.24
0.015	0.015	0.020	4.5	1.08	0.84	5.89	78%	1.12

Chinook, Pipe 2, Min Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.020	2.6	1.08	0.45	7.11	42%	2.14
0.010	0.010	0.020	2.6	1.08	0.48	6.60	45%	1.92
0.011	0.011	0.020	2.6	1.08	0.51	6.17	47%	1.74
0.012	0.012	0.020	2.6	1.08	0.53	5.80	49%	1.59
As Designed (0.013)	0.013	0.020	2.6	1.08	0.56	5.49	52%	1.46
0.014	0.014	0.020	2.6	1.08	0.58	5.21	54%	1.35
0.015	0.015	0.020	2.6	1.08	0.60	4.97	56%	1.25

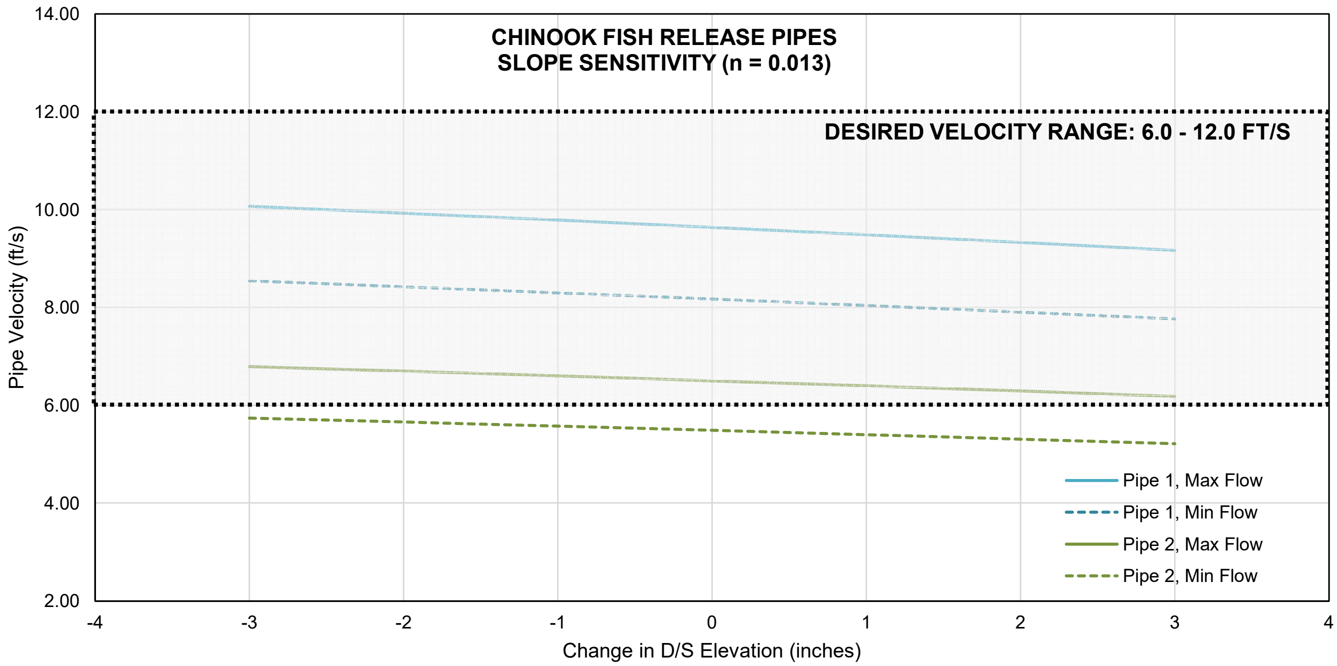


Figure 1. Chinook Release Pipe Slope Sensitivity

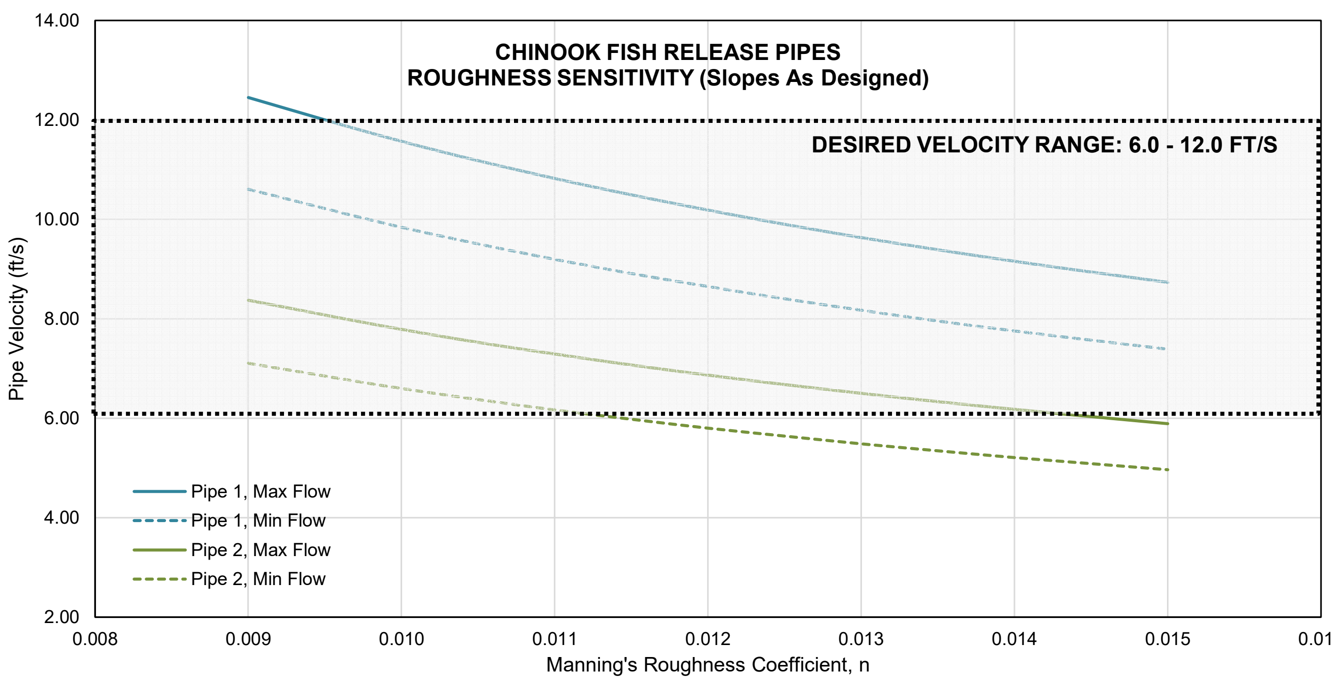


Figure 2. Chinook Release Pipe Roughness Sensitivity

Coho Pipe (Slope Change)

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2499.91	0.015	0.77	0.83	0.35	3.58	42%	1.23
- 2"	2499.99	0.013	0.77	0.83	0.36	3.40	44%	1.14
- 1"	2500.08	0.011	0.77	0.83	0.38	3.19	46%	1.04
As Designed	2500.16	0.009	0.77	0.83	0.40	2.95	49%	0.93
+1"	2500.24	0.007	0.77	0.83	0.44	2.67	53%	0.80
+ 2"	2500.33	0.004	0.77	0.83	0.49	2.33	59%	0.65
+3"	2500.41	0.002	0.77	0.83	0.59	1.86	72%	0.44

Coho Pipe (Manning's Coeff Change)

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.01	0.77	0.83	0.33	3.83	40%	1.36
0.010	0.010	0.01	0.77	0.83	0.35	3.55	42%	1.22
0.011	0.011	0.01	0.77	0.83	0.37	3.32	44%	1.10
0.012	0.012	0.01	0.77	0.83	0.39	3.12	47%	1.01
As Designed (0.013)	0.013	0.01	0.77	0.83	0.40	2.95	49%	0.93
0.014	0.014	0.01	0.77	0.83	0.42	2.80	51%	0.86
0.015	0.015	0.01	0.77	0.83	0.44	2.67	53%	0.80

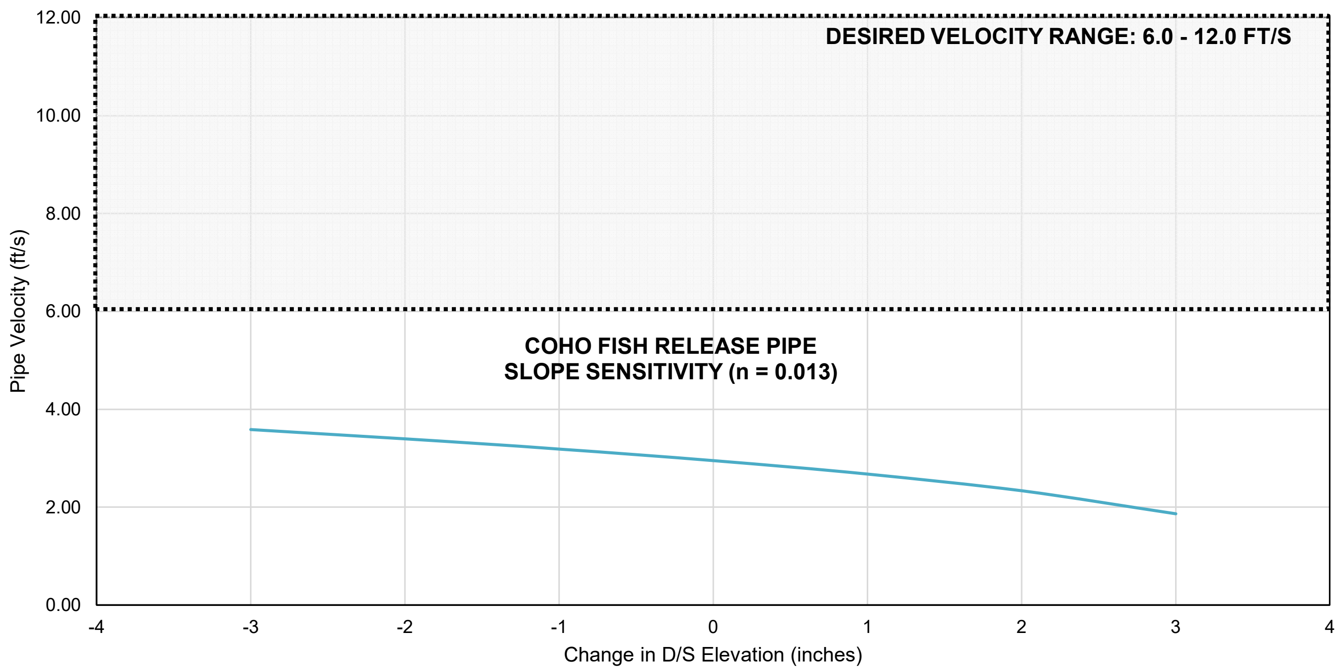


Figure 3. Coho Release Pipe Slope Sensitivity

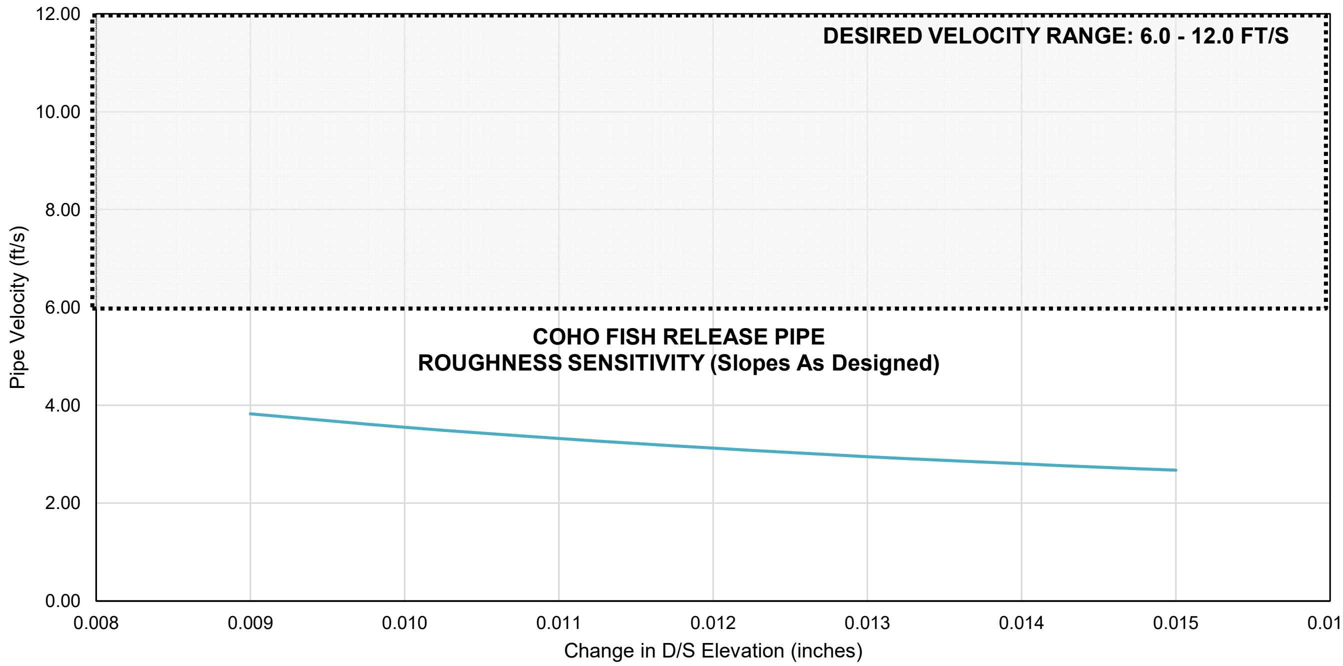


Figure 4. Coho Release Pipe Roughness Sensitivity

Adult Holding Release Pipe (Slope Change)

Max Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2486.00	0.031	10	2.31	0.73	8.73	32%	2.11
- 2"	2486.08	0.026	10	2.31	0.77	8.18	33%	1.92
- 1"	2486.17	0.021	10	2.31	0.82	7.56	35%	1.72
As Designed	2486.25	0.016	10	2.31	0.88	6.82	38%	1.49
+1"	2486.33	0.010	10	2.31	0.98	5.91	42%	1.21
+ 2"	2486.42	0.005	10	2.31	1.18	4.63	51%	0.84
+3"	NO SLOPE!							

Min Flow

Scenario	D/S Elev. ft	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
- 3"	2486.00	0.031	6.6	2.31	0.59	7.77	26%	2.11
- 2"	2486.08	0.026	6.6	2.31	0.62	7.27	27%	1.93
- 1"	2486.17	0.021	6.6	2.31	0.66	6.71	29%	1.72
As Designed	2486.25	0.016	6.6	2.31	0.71	6.06	31%	1.49
+1"	2486.33	0.010	6.6	2.31	0.79	5.24	34%	1.22
+ 2"	2486.42	0.005	6.6	2.31	0.94	4.09	41%	0.86
+3"	NO SLOPE!							

Adult Holding Release Pipe (Manning's Coeff Change)

Max Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.016	10	2.31	0.73	8.87	31%	2.15
0.010	0.010	0.016	10	2.31	0.77	8.22	33%	1.94
0.011	0.011	0.016	10	2.31	0.81	7.68	35%	1.76
0.012	0.012	0.016	10	2.31	0.84	7.22	37%	1.61
As Designed (0.013)	0.013	0.016	10	2.31	0.88	6.82	38%	1.49
0.014	0.014	0.016	10	2.31	0.92	6.47	40%	1.38
0.015	0.015	0.016	10	2.31	0.95	6.16	41%	1.29

Min Flow

Scenario	Manning's Coeff	Pipe Slope ft/ft	Flow Rate cfs	Pipe I.D. ft	Flow Depth ft	Flow Velocity ft/s	% Full	Froude Number
0.009	0.009	0.016	6.6	2.31	0.59	7.88	25%	2.15
0.010	0.010	0.016	6.6	2.31	0.62	7.31	27%	1.94
0.011	0.011	0.016	6.6	2.31	0.65	6.83	28%	1.76
0.012	0.012	0.016	6.6	2.31	0.68	6.41	29%	1.62
As Designed (0.013)	0.013	0.016	6.6	2.31	0.71	6.06	31%	1.49
0.014	0.014	0.016	6.6	2.31	0.74	5.74	32%	1.38
0.015	0.015	0.016	6.6	2.31	0.76	5.47	33%	1.29

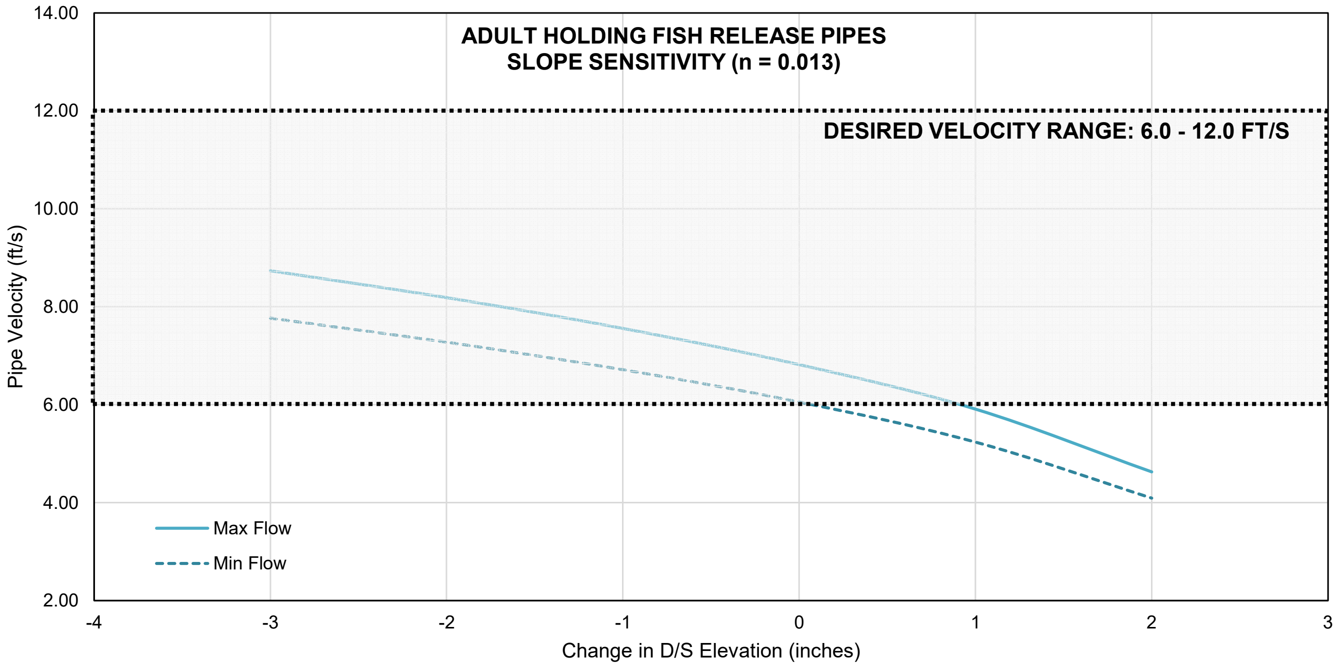


Figure 5. Adult Holding Release Pipe Slope Senstivity

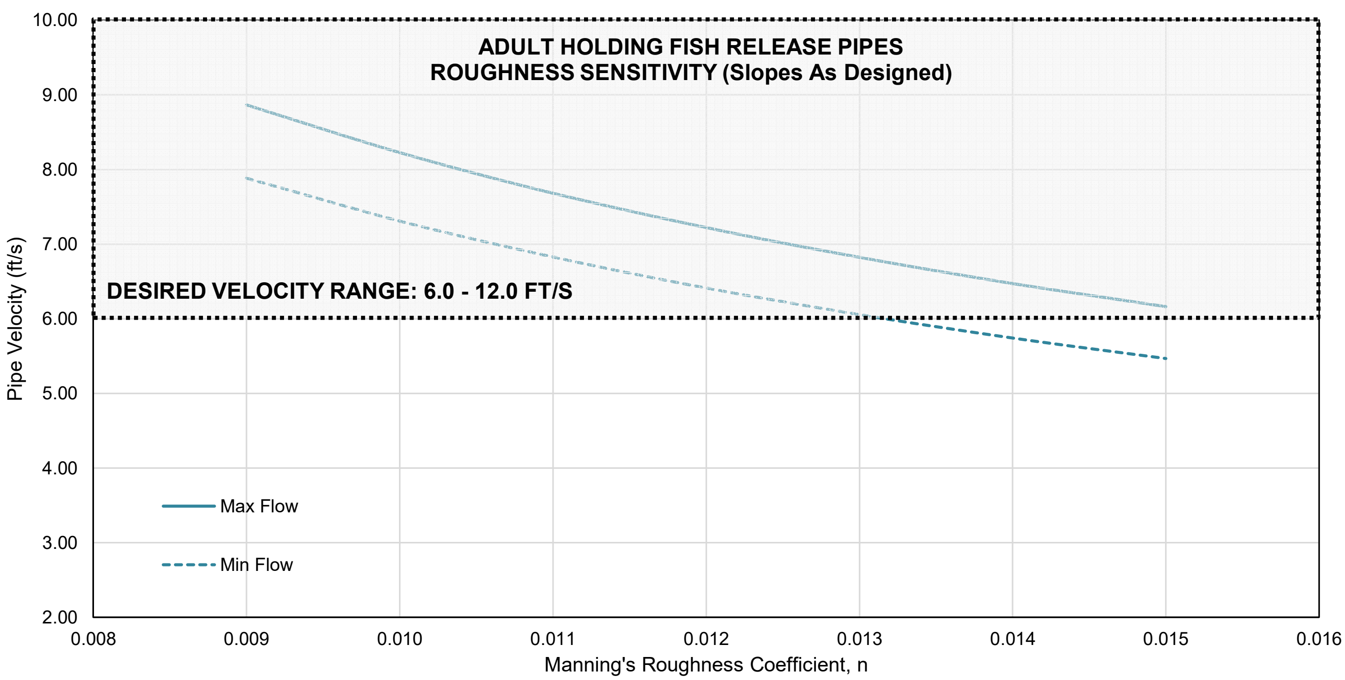


Figure 6. Adult Holding Release Pipe Roughness Senstivity

Conclusions

The following conclusions may be drawn from the above sensitivity analysis:

Chinook Fish Release Pipes

The Chinook release pipes were designed for a range of flows from 1/4 of the total Chinook raceway outflow to 1/2 of the total Chinook raceway outflow. Because this represents a range of approximately 2 cfs, it is not expected that one pipe will be able to meet the ideal hydraulic characteristics for the a range of roughness coefficients and slope deviations. That being said, the following was found:

1. Flow remained supercritical for all cases, with a minimum Froude number of 1.12, and therefore it is not expected that a hydraulic jump would occur within the pipe. While a Froude number of 1.12 is close to the range where flow will become unstable, this provides some buffer even at the worst case condition against the formation of a hydraulic jump.

2. Pipe flows generally remained within the 40% to 75% full flow range, which ensures that flows will neither be too shallow, nor will they fill the pipe and change to a pressure flow regime. For brand new pipe (n = 0.009, 0.010) at the minimum flows, the flow depths did shallow slightly below the 40% threshold (36% full, 38% full) for the steeper pipe run (6.0%). This approximates the 40% threshold and represents a worst case scenario at the minimum allowable flow rate. Conversely, for very degraded pipe (n = 0.015), the maximum flow rate does fill to about 78% full, which cuts into some of the allowance for waves and air flow. It is not expected, however, that the pipe will reach such a degraded condition in the short life over which it will be operated.

3. Velocities within the 6.0 ft/s to 12.0 ft/s range are desirable for fish bypass pipes. The graphs above demonstrate that it is challenging to meet the velocity criterion for the entire range of flow rates and for a range of pipe slopes or roughness coefficients. The pipes as designed with a roughness coefficient of 0.013 were maintained within the pipe velocity range for all cases, except for the minimum flow in pipe 2, which fell just below the 6.0 ft/s criterion. This being said, the roughness of 0.013 represents a high roughness associated with significant pipe degradation. This would be unlikely over the short facility life, and it is expected that the roughness would fall closer to the 0.010 - 0.011 range. It can be seen from Figure 2 that the entire flow range for both pipe reach 1 and pipe reach 2 fall within the desired velocity range for roughness coefficients of 0.010 and 0.011. Higher roughness coefficients will still maintain velocities within the desired range, however, the minimum flow rate will start to fall below the 6.0 ft/s threshold. If this is the case, and fish are found to hold in the pipe, the upstream dam boards can be adjusted such that a larger proportion of the flow reports to the bypass pipe and fish are moved into Fall Creek.

4. For brand new pipe (n = 0.009), the maximum flow exceeds the 12.0 ft/s threshold slightly in the upper pipe (12.45 ft/s). This is a minor exceedance of the desired velocity range, and can be adjusted by the upstream dam boards if absolutely necessary. It is not expected however that the pipe will stay in this condition for long. Some minor changes to the pipe roughness will bring these flow rates into the desired range.

5. It is noteworthy that the fish release pipes have significant enough slope that the pipes will be fairly self-cleaning. So, while they will not remain in new condition, flushing flows will produce velocities and shear stresses in the pipe that will maintain a relatively smooth condition. Therefore, it is expected that the pipes will have a roughness of approximately 0.010 - 0.012, which is ideal for maintaining good hydraulic conditions for fish passage over the whole range of flows.

Coho Fish Release Pipe

The Coho release pipe was constrained by the minimum pipe size (10 inch diameter, nominal) and the elevations of the new rearing ponds and the existing concrete outlet flume, such that velocities in the desired 6.0 ft/s - 12.0 ft/s range were not attainable. Velocities were below the desired range in the pipe, and there is potential for the fish to hold in the pipe. This being said, however, the pipe run is only 40 ft long and there will be a simple drop into the existing outlet flume such that fish that volitionally leave down the pipe will not be able to get back up into the pipe. Therefore, the short run of pipe was deemed acceptable to the current application. The design flow rate for all conditions was found to be within the 40% - 75% full depth range.

Adult Holding Fish Release Pipe

The adult holding fish release pipe was sized at 30" diameter, such that the pipe would not submerge at the inlet. This run of pipe is very short, however, (approx. 15' long), and therefore it was deemed unnecessary that the pipe should reduce to a smaller pipe size. The 30" diameter was maintained for the entire length of the fish release pipe. Therefore, in order to maintain velocities in the 6.0 ft/s to 12.0 ft/s range, the flow depth was allowed to dip below the 40% full flow depth criterion. Given the size of the pipe, however, this should provide ample water column for the limited time fish will be in the release pipe. The findings of the sensitivity analysis for this pipe are summarized below:

1. Due to the short length of the fish release pipe, the pipe is very sensitive to the elevation of the inlet and outlet. If the outlet were raised by 3", then the pipe would have no drop and consequently no slope to the outlet. Therefore, it will be critical that the pipe maintains at least the 1.6% slope to the outlet in order to maintain velocities in the desired range. If the slope is increased, on the other hand, velocities will be higher, but flows will also be shallower. In order to achieve a balance of the flow depth criterion and the velocity criterion, the 1.6% slope is maintained. It is expected that this could be easily achieved over such a short run of pipe. If it the slope is shallower, the pipe run is very short and large diameter, and fish holding in the pipe could easily be encouraged to move downstream. If the slope is steeper, then fish will easily be passed downstream by the velocity in the pipe.

2. The adult holding fish release pipe was found to be within the desired velocity range for all roughness coefficients less than or equal to 0.013 for the entire flow range. It is expected that the roughness coefficient will be in the lower end of this range due to the short life of the facility. If it is found, however, that a biofilm is forming that is adversely affecting the hydraulics, there is ample access to this pipe and the pipe run is so short, such that the pipe can be easily cleaned to improve the hydraulic properties within the pipe.

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Volitional Release Pipe Support

BY: A. Leman **CHK'D BY:** N. Cox/V. Autier
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to determine the support and deflection conditions for the Chinook Fish Release Pipe at the plunge pool.

References

- *PPI (Plastic Pipe Institute). Handbook of Polyethylene Pipe, 2nd Edition. 2008.*
- *Munshi, S.R., Modi, V.J., and Yokomizo, T. Fluid Dynamics of Flat Plates and Rectangular Prisms in the Presence of Moving Surface Boundary-Layer Control. Journal of Wind Engineering and Industrial Aerodynamics, Vol. 79, Iss. 1-2, pp. 37-60. January 1999.*

Method

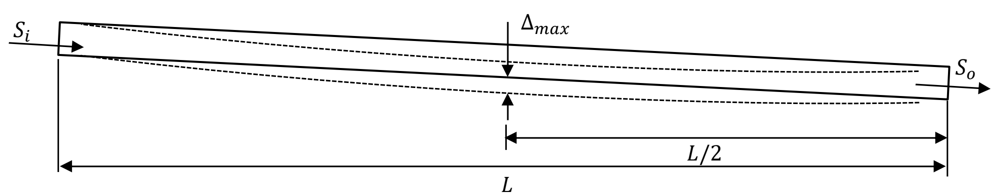
These calculations will determine the following:

1. The allowable deflection such that a change to the hydraulic regime (hydraulic jump) is avoided.
2. The allowable span between the buried pipe and the support based on an allowable pipe deflection.
3. The allowable overhang from the pipe support to the pipe outlet.
4. The lateral loads on the pipe support associated with the 100-year event.

The calculations will proceed according to the following methods:

Allowable Deflection

The allowable deflection was calculated such that the slope change on the downstream half of the pipe would not yield a Froude number less than 1.1. Open channel flow calculations are depicted in the fish release pipe hydraulic calculations.



$$S_o = \frac{S_i L - \Delta_{max}}{L/2} \implies \Delta_{max} = S_i L - S_o\Big|_{\mathrm{F}=1.1}\left(\frac{L}{2}\right)$$

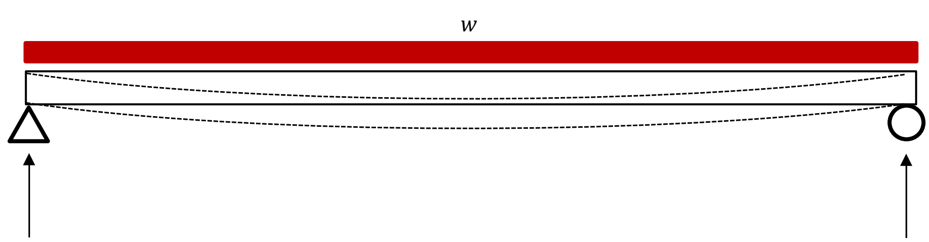
where:
$\mathrm{F} =$ Froude number, evaluated at 1.1

Allowable Span

The allowable span between the buried pipe and the support was calculated according to a simple beam analysis (see PPI, 2008) with pinned end connections (free rotation), where the maximum deflection (at the center of the span) is equal to:

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

where:
$w =$ Uniform distributed load, lb/in
$L =$ Span length, in
$E =$ Modulus of elasticity, HDPE, psi
$I =$ Moment of inertia, in



The temperature effects will be small for the length of unsupported pipe (40deg F change from service temp, 20' length of pipe, ~0.05 in additional deflection), and are neglected in this analysis. This is within the conservatism associated with neglecting the additional length of overhanging pipe, and pinned end connections. The allowable span, therefore, can be determined from the following equation, with an additional safety factor added to the load:

$$L_{max} = \sqrt[4]{\frac{384\Delta_{allow}EI}{5(FS)w}}$$

where:
$L_{max} =$ Maximum span length, in
$\Delta_{allow} =$ Allowable deflection, 1/2 in (PPI, 2008)
$FS =$ Factor of Safety
<other variables as previously defined>

Pipe Overhang

The allowable pipe overhang was based on the equation for deflection of a uniformly loaded cantilever beam with fixed end. It is recognized that the fixed end assumption is a simplification, however, the uniform load on the adjacent span would naturally result in a rotation upward (see above) of the cantilevered section and ultimately reduce the deflection. Therefore, the fixed end assumption is conservative. The equation for deflection of a uniformly distributed load on a cantilevered beam is:

$$\Delta_{max} = \frac{wL^4}{8EI}$$

where:
<all variables as previously defined>

100-year Lateral Loads

The lateral loads on the concrete pipe support was calculated based on the results of the HEC-RAS model for free-stream velocity, and the equation for drag force:

$$F_D = \frac{C_D A \rho V^2}{2g}$$

where:
$F_D =$ Drag force, lbs
$C_D =$ Drag coefficient
$A =$ Projected Area, ft^2
$\rho =$ Density water, 62.4 lbs/ft^3
$V =$ Free-stream velocity, ft/s
$g =$ Gravitational constant, 32.2 ft/s^2

The drag coefficient associated with a projected long rectangular member was assumed to be approximately 2.05 for design. This will vary with dimension ratio of the pier and Reynolds number, but is a conservative value for design.

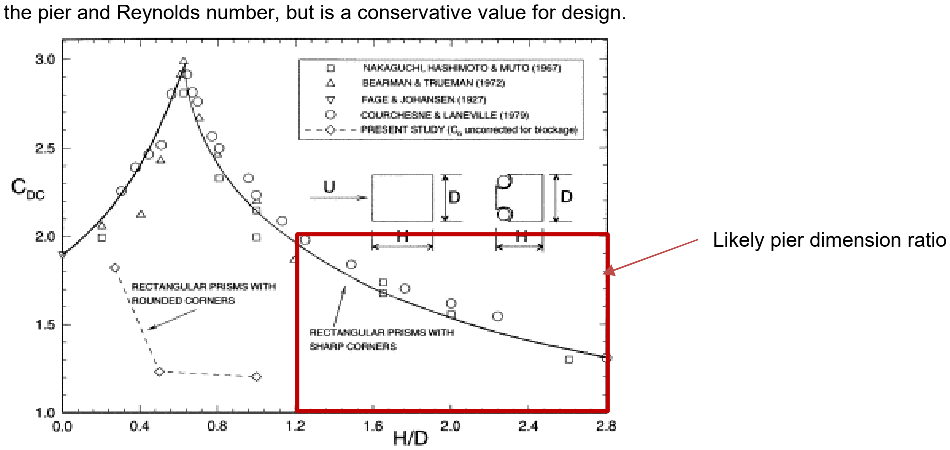


FIGURE 1. Rectangular Pier Drag Coefficients (Source; Munshi et al, 1999)

Inputs

The following inputs were used in this analysis:

Pipe Inputs	Value	Comments
Modulus of Elasticity, E	342,000 psi	60deg F, load duration of 50-year
Pipe Dimension Ratio, DR	26	
Nominal Diameter	14 in	See Fish Release Pipe hydraulic calculations
Outer Diameter	14 in	
Inner Diameter	12.92 in	
Moment of Inertia, I	516.3 in^4	
Pipe Weight	0.83 lb/in	Manufacturer data
Water Weight	4.74 lb/in	Assumed full, for case of blockage
Initial (Undeflected) Slope, S_i	0.02 ft/ft	See Fish Release Pipe hydraulic calculations
Initial (Assumed) Free Span	14 ft	Estimate (iterated)
Cantilevered Length	2.5 ft	Measured from CAD

Other Inputs	Value	Comments
Allowable Deflection	0.5 in	PPI, 2008; subject to allowable deflection calculations
Water Unit Weight	62.4 lb/ft^3	50 deg F
Factor of Safety	1.5	Assumed
Gravitational Constant	32.2 ft/s^2	

Calculations

Allowable Deflection

Flow Rate, Q_w cfs	Pipe Inner Dia. ft	Outlet Slope, S_o ft/ft	Manning's Coeff	Flow Depth ft	Internal Angle deg	Flow Area ft^2	Top Width ft	Flow Velocity ft/s	Froude Number
2.6	1.077	0.0120	0.013	0.64	202	0.57	1.06	4.59	1.10
4.5	1.077	0.0148	0.013	0.85	250	0.77	0.88	5.85	1.10

Span Length ft	Initial Slope, S_i ft/ft	Outlet Slope, S_o ft/ft	Maximum Deflection, Δ_{max} ft	Maximum Deflection, Δ_{max} in
14	0.02	0.0148	0.18	2.12

Allowable Span

Uniform Dist Load, w lb/in	Modulus of Elast, E psi	Moment of Inertia, I in^4	Maximum Deflection[1], Δ_{max} in	FOS	Allowable Span in	Allowable Span ft
5.57	342,000	516.3	0.50	1.5	169	14.1

[1] Maximum deflection is the minimum of the 0.5 inches dictated by PPI and the deflection resulting in a Froude number of 1.1 (above).

Cantilevered Section Deflection

Cantilever Length, L in	Uniform Dist Load, w lb/in	Modulus of Elast, E psi	Moment of Inertia, I in^4	Maximum Deflection, Δ_{max} in	Calculated FOS
30	5.57	342,000	516.3	0.003	157

100-year Lateral Loads

100-year WSEL ft	Bottom of Pool El ft	Assumed Pier Width ft	Flow Depth, d ft	Avg Flow Velocity[1], V ft/s	Drag Coefficient C_D	Projected Area, A ft^2	Factor of Safety, SF	Drag Force, F_D lbs	Distributed Drag Force, w_D lbs/ft
2494.50	2488.40	1.00	6.10	8.49	2.05	6.1	1.5	1310	215

[1] Note that the average cross-sectional velocity is used here, which is conservative. In actuality the pier will be close to the bank which will provide some shear resistance resulting in reduced velocities at the bank. Further definition on the velocity would require 2D modeling, which is unwarranted for this application.

Conclusions

Loading of the Fish Release pipe support was calculated to determine the maximum allowable span according to the methods of the plastic pipe institute (PPI) with a check to ensure that the pipe deflection does not produce a hydraulic jump. It was found that the maximum allowable span was 14-feet with a factor of safety of 1.5.

Downstream of the pipe support, the pipe will overhang (cantilever) by approximately 2'-6". Based on the deflection equations for a uniformly loaded cantilever beam, the maximum deflection will be less than 1/100th of an inch, well within the 0.5 inch maximum deflection requirements of PPI.

100-year flood event lateral loads were calculated at the pipe support pier, and it was found that a distributed load of ~215 lbs/ft is applied to the concrete support. This conservatively assumes the average channel velocity encounters the pier, and a conservative drag coefficient of 2.05 was applied.

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Pipe Vent Sizing

BY: A. Leman CHK'D BY: N. Cox
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to size the vent pipes and air/vacuum valves for all of the pipelines on site..

References

• *Falvey, H.T. 1980. Air-Water Flow in Hydraulic Structures: Engineering Monograph No. 41. U.S. Department of the Interior, U.S. Bureau of Reclamation, Water and Power Resources Eservice Engineering and Research Center: Denver, CO. December 1980.*

• *Kalinske, A.A., Roberston, J.M. 1943. Closed Conduit Flow. Trans. ASCE, Vol. 108, pp. 1431-1516.*

• *Sikora, A. 1965. Air Entrainment in Shaft Spillways [Czechoslovakia]. 1965.*

• *Val-Matic Valve & Mfg. Corp. (Val-Matic). 2018. White Paper: Theory, Application, and Sizing of Air Valves. Accessed at www.valmatic.com.*

Method

Vent pipes were sized according to the methods of the U.S. Bureau of Reclamation (Falvey, 1980) . The vent pipes were sized for two conditions in the waste drain and drain pipelines:

1. Open channel flow where there is no upstream path for air to enter the pipeline (e.g. submerged inlets).

2. Transitions in the pipeline from open channel flow to pressure pipe flow, where there is an air demand from a hydraulic jump.

The following equations were used for determining the air demand for the two conditions open channel conditions.

Sikora, 1965; as reported in Falvey, 1980
Equations (75), (76), and (78) - Open Channel Conduit

The air demand in the open channel flow will be dependent upon the development of the air-side boundary layer downstream of the point at which the flow transitions to open channel. The following equations give an estimate of the boundary layer thickness above the air-water interface, and assume a 1/7th power law for the velocity distribution within the boundary layer. This velocity power law was then numerically integrated over the area of the boundary layer to determine an air demand. If it was found that the boundary layer attained the top of the pipe, the conservative equation of Sikora (1965) was applied, which assumes that the air attains the velocity of the water through the entire open area of the pipe. This is a very conservative condition, and therefore was only applied where the boundary layer development was sufficient to attain the top of the pipe and Poiseulle flow conditions would prevail.

$$\delta = 0.01L$$

$$u(y) = V_w \left(\frac{y_b}{\delta}\right)^{1/7}$$

$$Q_a = \int V\,dA = 2V_w \delta^{-1/7} \int_{R-d}^{R-d-\delta} y_b^{1/7} \sqrt{R^2 - y^2}\,dy$$

$$y_b = -y - R + d + \delta$$

$$\left(\frac{Q_a}{Q_w}\right)_{max} = \frac{A_d}{A} - 1$$ [if boundary layer attains top of pipe]

where:
$\delta =$ Boundary layer thickness, ft
$L =$ Boundary layer development length, ft
$u =$ Local streamwise velocity, ft/s
$V_w =$ Water velocity (at surface), ft/s
$y_b =$ Vertical boundary layer coordinate (above W.S.), ft
$y =$ Vertical coordinate about pipe center, ft
$R =$ Pipe inner radius, ft
$d =$ Water flow depth, ft
$Q_a =$ Air flow rate, cfs
$Q_w =$ Water flow rate, cfs
$A_d =$ Cross-sectional area of conduit, ft^2
$A =$ Max cross-sectional area of water flow, ft^2

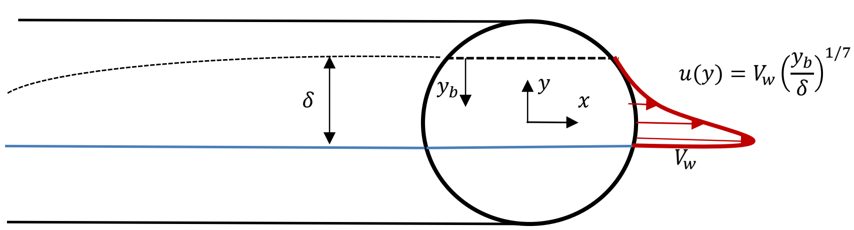


Figure 1. Definition Sketch Air-Water Interface Boundary Layer

Kalinske and Robertson, 1943; as reported in Falvey, 1980
Equation (79) - Hydraulic Jump Filling the Conduit

The following equation was developed from dimensional analysis and model studies. This only applies to cases where no "blowback" of air pockets occurs. Conditions of air bubbles downstream of the pipe filling will be checked in subsequent calculations (see below).

$$\left(\frac{Q_a}{Q_w}\right) = 0.0066(\mathrm{F}-1)^{1.4}$$

where:
$F =$ Froude number upstream of hydraulic jump
<other values as previously defined>

Once the air demand is calculated, an air velocity criterion was applied to size the vent pipes. As a best practice, it was assumed that the air velocity would be less than 40 ft/s, which would yield velocity head of 0.03 ft of water column, and would limit the pressure differential from atmospheric to the inside of the pipe. This is more stringent than the 50 ft/s cited by Falvey (1980) for which loose objects could be swept up or held on the air vent louvers. The equations to size the pipe are given below:

Pipe Sizing Equations
Assumed Velocity Criterion (see Falvey, 1980 for Noise, Safety, and Other Concerns)

$$V < V_{max} = 40\ \mathrm{ft/s}$$

$$V = \frac{Q_a}{\frac{\pi}{4} d_v^2}$$

where:
$V =$ Air velocity in the vent, ft/s
$V_{max} =$ Assumed max velocity criterion, ft/s
$d_v =$ Vent pipe inner diameter, ft
<other values as previously defined>

Finally, for the case where a hydraulic jump fills the pipe, it should be considered what the ultimate fate of the entrained air will be. Falvey (1980) provides the following chart for evaluating the bubble response to the pipe slopes and flowrate. The intent would be to direct any accumulated air to the vents for release without accumulation that could induce blow-back damage to the pipe.

Bubble Motion in Closed Conduits Flowing Full
Figure 29, Falvey 1980

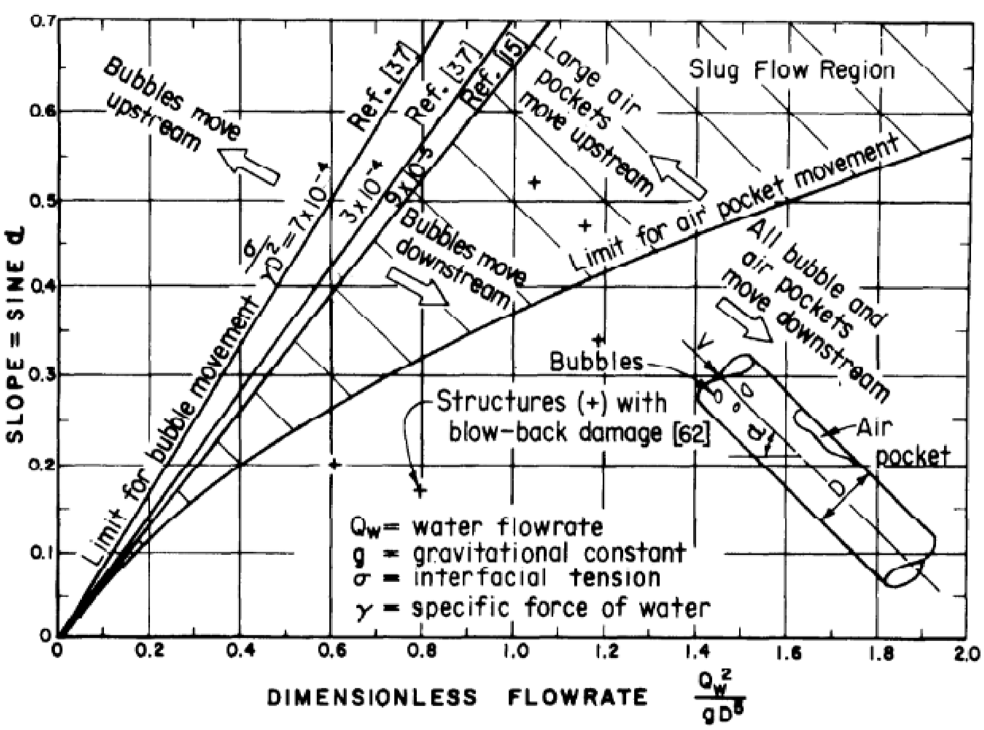


Figure 2. Air Movement Regime Chart (Source; Falvey, 1980)

For the pressure pipes, the air release, air/vacuum release, and combination air valves were sized according to the Val-Matic (2018) White Paper on theory, application, and sizing of air valves.

Air/Vacuum Release Valve Sizing
Equation 5

The following equation is used for sizing the air/vacuum release valve upstream of a steep slope. This performs two functions: (1) it allows the release of large amounts of air at filling so that there are no large trapped air pockets in the pipe, and (2) it releases any vacuum that may form at the top of the slope when draining the pipe. It should be noted that for the 24" Schedule 80 PVC pipe, the critical collapse pressure (150 psi) is able to withstand full vacuum pressure, and furthermore it is not anticipated that the pressure pipes will require frequent draining. That being said, however, at filling there is potential for large pockets of air to get trapped at the top of the slope leading down to Copco Road. Based on the bubble regime chart given above by Falvey, at the full 10 cfs flow through the supply pipe, on a 20% grade, it is expected that bubbles will move upstream and will not be carried downstream by the flow. This presents the potential for bubbles to permanently accumulate at the top of the slope, if unvented, and potentially impact the hydraulics of the pipe. Therefore, either an air-release valve or a air/vacuum release valve will be provided at the top of the slope. In either case, the required air flow will be calculated according to the Val-Matic equation 5:

$$Q_a = 678\gamma d_o^2 C_D \left(\frac{\Delta p\, p_1}{T_1 S_g}\right)^{1/2}$$

where:
$Q_a =$ Air capacity of valve, SCFM
$\gamma =$ Expansion Factor, .93 for exhaust at 2 psi
$d_o =$ Diameter of valve, in
$C_D =$ Orifice discharge coefficient, 0.6 for sharp corners
$\Delta p =$ 2 psi for exhaust sizing w/out slow closure
$p_1 =$ Inlet pressure, 16.7 psia for 2 psi differential
$T_1 =$ Inlet temperature, 520 Rankine
$S_g =$ Specific Gravity, 1 for air

The air capacity of the valve should be equal to the fill rate, as the valve will need to evacuate the volume of air (at 5 psi differential pressure) that is being replaced by the water fill rate. It is assumed that the fill rate will be 2.5 cfs, or the maximum water right divided equally among the 4 pipelines. This could potentially be exceeded, however, if an air release valve is used, then the valve will continue to vent trapped air after the line has achieved pressure and this is not a major concern.

Air Release Valve Sizing
Equation 6 (?)

The air release valve sizing was calculated according to the equation given in the Val-Matic (2018) White Paper, which is a variation on Equation 5 above, but considering the line pressure:

$$Q_a = \frac{330.7 d_o^2 C_D p_1}{\sqrt{T_1 S_g}}$$

where:
$p_1 =$ Pipeline pressure, psia (NOTE, absolute pressure)
<other values as previously defined>

It is furthermore recommended in the White Paper, that the venting "demand" be estimated by assuming a 2% air concentration in the flow:

$$Q_d = 0.02 Q_{w,gpm} \left(\frac{0.13\, ft^3}{gal}\right)$$

where:
$Q_d =$ Capacity requirement, SCFM
$Q_w =$ Design flow rate in pipe, gpm

By setting the capacity equal to the demand, a design air valve diameter can be calculated.

Locations

The following locations were identified for necessary vent pipes and valves. See 'Drain Hydraulics' Calculations for inlet control conditions and calculation of potential submergence. For the waste drains, it was assumed that venting of the inlet pipes would be provided inside of the buildings.

I.D.	Pipeline	Location	Type of Venting
VT1	Drain Pipe	Chinook Rearing	Open Channel (D/S of submerged inlet)
VT2	Drain Pipe	Incubation Building	Transition to Pressure Pipe (Hydraulic Jump)
VT3	Waste Drain Pipe	Chinook Rearing	Open Channel (U/S of steep drop)
VT4	Waste Drain Pipe	Chinook Rearing	Open Channel (Start of Chinook Raceways Waste Drain)
VT5	Waste Drain Pipe	Incubation Building	Open Channel (U/S of steep drop)
VT6	Supply Pipe - Adult Hold	Chinook Rearing	Air Release (D/S of pipe "sag")
VT7	Supply Pipe - Adult Hold	Incubation Building	Air Release (U/S of steep drop)

Calculations

Open Channel Vent Locations

Location I.D.	D/S Pipe Nom. Dia.	D/S Pipe Inner Dia.	Flow Rate, Q_w	Flow Rate, Q_w	Boundary Layer Length, L	D/S Flow Depth, y	D/S Flow Area, A	Total Pipe Flow Area, A_d	Air Demand, Q_a
	in	ft	gpm	cfs	ft	ft	ft^2	ft^2	cfs
VT1	24	1.78	4040	9.0	25	0.72	0.94	2.50	3.7
VT3	8	0.63	200	0.4	40	0.21	0.09	0.31	0.9
VT4	8	0.63	200	0.4	100	0.24	0.11	0.31	0.8
VT5	8	0.63	200	0.4	50	0.18	0.07	0.31	1.5

Location I.D.	Nom. Vent Pipe Size	Vent Pipe Inner Dia.	Vent Pipe Area	Air Velocity, V	< 40 ft/s?
	in	ft	ft^2	ft/s	
VT1	6	0.48	0.18	21	YES
VT3	3	0.24	0.04	21	YES
VT4	3	0.24	0.04	18	YES
VT5	3	0.24	0.04	33	YES

Hydraulic Jump Vent Locations

Location I.D.	D/S Pipe Nom. Dia.	D/S Pipe Inner Dia.	Flow Rate, Q_w	Flow Rate, Q_w	Froude Number, F	Air Demand, Q_a	Slope	Dim'less Flow Rate Q_w^2/gD^5	Air Flow Regime*
	in	ft	gpm	cfs		cfs	ft/ft		
VT2	24	1.78	4190	9.34	3.61	0.24	0.20	0.15	U/S Mvmt

Location I.D.	Nom. Vent Pipe Size	Vent Pipe Inner Dia.	Vent Pipe Area	Air Velocity, V	< 40 ft/s?
	in	ft	ft^2	ft/s	
VT2	3	0.24	0.04	5.3	YES

* 'U/S Mvmt' indicates that bubbles move upstream; 'D/S Mvmt' indicates that all bubble and air pockets move downstream; 'Slug Flow' indicates that bubbles move downstream, but large air pockets move upstream.

Pressure Pipe Valve Sizing

Location I.D.	Water Flow Rate, Q_w	Air Demand, Q_d	Valve Size, d_o	Expansion Factor, γ	Orifice Coeff, C_D	Diff. Pressure, Δp	Inlet Pressure*, p_1	Valve Capacity[†], Q_a	Sufficient?
	gpm	SCFM	in			psi	psia	SCFM	
VT6	4480	12	0.5	-	0.6	-	20.2	44	OK
Filling[‡] ------ -------------->	1120	150	2	0.93	0.6	2	16.7	384	OK
VT7	1120	150	2	0.93	0.6	2	16.7	384	OK

* Inlet pressure in psia, assumes 14.7 psi atmospheric

[†] Note, equations are different for two rows, as the applications are different. Do not drag down.

[‡] Filling is a concern for VT6, as well, and therefore it was sized for the more conservative of the two cases.

Conclusions

Seven locations were identified that would require air vents either because (1) shear at the air-water interface in open channel flow created an air demand, (2) a hydraulic jump that fills the pipe created an air demand, (3) trapped air would accumulate in pressure lines, or (4) air would be trapped at pipe filling and would not be flushed by maximum flows in the pipeline. For case 1, boundary layer development equations according to USBR Monograph 41 (Falvey, 1980) were applied to determine the air demand based on air velocities in the pipe. For case 2, an empirical equation from the USBR Monograph 41 (Falvey, 1980) was applied to determine the air demand at the hydraulic jump. The air flow regime was furthermore determined for this case to ensure that slug flow and consequent blow-back would not induce damage to the pipe. The vent pipes were then sized for cases 1 and 2 such that air velocities through the pipe would be maintained less than 40 ft/s. For cases 3 and 4 the methods outlined by Val-Matic (2018) were used to size air-release or air/vacuum release valves. A summary for each of the identified locations is provided in the table below:

Location I.D.	Nom. Vent/Valve Size (in)	Air Demand, Q_a (cfs)	Air Flow Regime
VT1	6	3.72	-
VT2	3	0.24	Bubbles move upstream
VT3	3	0.93	-
VT4	3	0.80	-
VT5	3	1.48	-
VT6	2	2.5	-
VT7	2	2.5	Bubbles move upstream

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Chinook Outlet

BY: A. Leman CHK'D BY: V. Autier
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this sheet is to document the design of the Chinook outlet for splitting flows to the volitional release pipe and the production drain.

References

- *NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.*

Method

The outlet of the Chinook raceways will feed a single exit channel, that will typically be operated to direct flows to the production drain system. During volitional fish release, however, flows will need to be diverted to both the production drain system (and on to the adult holding ponds, as "second pass" water) and to the volitional release pipe. The calculations below document the following:

Overflow Dam Boards

These calculations determine the weir overflow depth, and consequently the elevation of the dam boards at the end of the Chinook raceways. Calculations are based on the weir equation with pier contractions as given in HDC 111-3 (USACE, 1977). The discharge coefficient was determined according to the Rehbock equation:

$$Q = \frac{2}{3} C_1 \sqrt{2g} \left(L' - 2(NK_p)H_e\right) H_e^{3/2}$$

$$C_1 = \left(0.6035 + 0.0813 \frac{H_e}{Y} + \frac{0.000295}{Y}\right)\left(1 + \frac{0.00361}{H_e}\right)^{3/2}$$

where:
$Q =$ Discharge, cfs
$C_1 =$ Discharge coefficient
$L' =$ Net Length of crest, ft
$N =$ Number of piers
$K_p =$ Pier contraction coefficient, ft
$H_e =$ Energy head, ft
$Y =$ Weir height, ft

Volitional Release Dam Boards

These calculations determine the elevation at which the volitional release dam boards need to be set to maintain a minimum pool depth, such that fish that drop into the exit channel do not drop onto concrete. These calculations will also set the water surface in the exit channel for determining the flow split between the production drain and the volitional release pipe.

$$Q = \frac{2}{3} C_1 \sqrt{2g} L H_e^{3/2}$$

where:
$L =$ Crest length, ft

Volitional Release Pipe

Volitional release pipe calculations are performed on the "Volitional Pipe Release" sheets.

Fish Screen

During volitional release, the production drain will have a fish screen in place to prevent fish from being entrained in the production drain system. The fish screen will be brought over from IGFH and will be of the type that is currently in use by CDFW. The fish screen was sized such that approach velocities would be less than 0.4 ft/s per NMFS 11.6.1.1. Active screen values were used as this is not in the stream, but is downstream of the ponds and has already been screened multiple times before this point. There will also be significant sweeping velocities along the length of the screen from the draw at the volitional release dam boards.

$$A = W \times d$$

$$V_a = \frac{Q}{A}$$

where:
$W =$ Screen width, ft
$A =$ Screen area, ft^2
$V_a =$ Approach velocity, ft/s

Production Drain

The production drain will be operated, during volitional release by another set of dam boards. These will be placed to direct the remainder of the flow (not going to the volitional release pipe) to the production drain system.

McMILLEN JACOBS ASSOCIATES

Inputs

Parameter	Units	Value	Description
Total Flow	cfs	9	
Flow per Pond	cfs	1.125	Total, divided by 8 ponds
Volitional Release Min Flow	cfs	2.6	see "Volitional Release Pipes" calculations
Volitional Release Max Flow	cfs	4.5	see "Volitional Release Pipes" calculations
Pond Floor Elevation	ft	2500	
Pond Water Surface Elevation	ft	2504	
Pond Depth	ft	4	Design Value
Pond Width	ft	12	Design Value
Exit Channel Width	ft	2.5	Design Value
Exit Channel Floor Elevation (@ Volitional Rel)	ft	2498.93	Design Value
Volitional Release Min Pool Depth	ft	3	Design Value
Pier Width	ft	1.5	Design Value
Number of Piers per pond		1	
Pier Contraction Coefficient, K_p		0.1	Assumed, conservative
Gravitational Constant	ft/s^2	32.2	

Calculations

Overflow Dam Boards

Q cfs	H_e ft	Y ft	L' ft	C_1	Q_{calc} cfs	Goal Seek to 1.0
1.125	0.10	3.90	10.5	0.64	1.126	1.00

Overflow dam board crest elevation: 2503.90 ft

Volitional Release Dam Boards

Q cfs	H_e ft	Y ft	L ft	C_1	Q_{calc} cfs	Goal Seek to 1.0
9	1.08	1.92	2.5	0.65	9.006	1.00
6.4	0.89	2.11	2.5	0.64	6.647	1.04
4.5	0.68	2.32	2.5	0.63	4.502	1.00

Discharge to Production Drain cfs	Discharge to Volitional Release cfs	Production Drain Dam Boards Crest El ft	Volitional Release Dam Boards Crest El ft	WSEL ft
2.6	6.4	2501.64	2501.04	2501.93
4.5	4.5	2501.51	2501.25	2501.93

Volitional Release Pipe

See "Volitional Release Pipe" calculations.

The Chinook volitional release pipe was sized for a flow range from:

$Q_{max} =$	4.5	cfs	[50% total flow]
$Q_{min} =$	2.6	cfs	[~25% total flow]

Fish Screen

Q cfs	d ft	W ft	A ft^2	V_a ft/s
4.5	3.0	5	15	0.30
6.4	3.2	5	16	0.40

**use 5.0' b/c of existing screens at IGFH*

Conclusions

The above calculations document the design of the Chinook outlet channel for diverting water to the production drain and the volitional release pipe. During normal operations, the dam boards at the volitional release pipe will be full height, and all water will be drained to the production drain system. During volitional release, a 3.0' deep pool will be maintained in the exit channel, based on the crest elevation of the volitional release pipe dam boards. The production drain will have a fish screen that meets NMFS criteria for a range of flows from 4.5 cfs to 6.4 cfs. Behind the fish screen will be another set of dam boards that will control the amount of flow diverted to the production drain system. See the drawings for details.

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Fish Barrier	**PROJECT NO.:** 20-024	

Purpose

The purpose of this sheet is to design the fish exclusion system in Fall Creek.

References

- *Brater, E.F., King, H.W., Lindell, J.E., Wei, C.Y. 1976. Handbook of Hydraulics, 7th Edition* . McGraw-Hill.
- *NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.*

Method

The fish exclusion system in Fall Creek is intended for several main purposes:

(1) To exclude anadromous adults from the upstream reaches above Dam A and Dam B where they can pose a concern for the intake structures and for disease to the hatchery water supply.

(2) To exclude juvenile hatchery fish being released in the Spring from the same areas.

(3) To direct anadromous adults toward the fishway entrance and ultimately to the fish trap.

During the design process it was identified by NOAA that the habitat between Dam A/Dam B and the fishway is to be maintained. Therefore, in order to provide a barrier during trapping that will direct fish into the fishway, but will remain open during other seasons or after the closure of the hatchery, a 3-part barrier system is provided.

(1) **Lower Barrier** - In the lower portion of the site, adjacent to the fishway and trap, a removable picket barrier will be provided which will be placed at the start of each trapping season on a concrete sill. The pickets can then be removed at the end of the trapping season to allow unimpeded passage. The lower barrier sill will be oriented at an angle to the natural channel direction, such that fish will be directed toward the fishway entrance pool.

(2) **Dam A Barrier -** In order to prevent fish from accessing the reach containing the hatchery intake structure and City of Yreka intake building, Dam A will be modified with a steep apron to constitute a NMFS standard velocity barrier. This steep apron will convey natural Dam A overflows at shallow depths and high velocities into the stream below, such that an anadromous fish could not swim up the apron, or if it did, depths would not be sufficient for the fish to jump over Dam A.

(3) **Dam B Barrier -** In order to prevent fish from accessing the reach containing the City of Yreka intake structure in the Dam B reach, Dam B will likewise be modified with a steep apron to constitute a NMFS standard velocity barrier.

The design of each of the barrier systems is described below.

Criteria

The NMFS (2011) criteria for the two barrier types under consideration are summarized below:

NMFS Guidelines (Pickets)	**Value**		**Comments**
Picket Clear Spacing	1	in	NMFS 5.3.2.1, max
Maximum River Velocity	1.25	ft/s	NMFS 5.3.2.2
Average River Velocity	1	ft/s	NMFS 5.3.2.2, gross picket area
Maximum Head Differential	0.3	ft	NMFS 5.3.2.3, on the clean picket condition
Debris and Sediment	-		NMFS 5.3.2.4, debris and sediment removal must be considered
Picket Barrier Orientation	-		NMFS 5.3.2.5, direct fish toward fishway
Minimum Picket Freeboard	2	ft	NMFS 5.3.2.6 (during fish passage)
Minimum Submerged Depth	2	ft	NMFS 5.3.2.7, for 10% of cross-section; low design flow
Minimum Percent Open	40%		NMFS 5.3.2.8
Picket Materials	-		NMFS 5.3.2.9, Flat or round, steel, aluminum, or durable plastic
Picket Sill	-		NMFS 5.3.2.10, Uniform concrete sill

NMFS Guidelines (Velocity)	**Value**		**Comments**
Minimum Weir Height	3.5	ft	NMFS 5.4.2.1, relative to maximum apron elevation
Minimum Apron Length	16.0	ft	NMFS 5.4.2.2
Minimum Apron Slope	0.06	ft/ft	NMFS 5.4.2.3, 16H:1V
Maximum Weir Head	2.0	ft	NMFS 5.4.2.4
Downstream Apron Elevation	-		NMFS 5.4.2.5, must be greater than tailwater at high design flow
Flow Ventilation	-		NMFS 5.4.2.6, fully ventilated nappe flow

Inputs

Hydrologic Inputs

Barrier 1 (Lower)	Value		Comments
Adult Fish Passage High Flow	71.86	ft^3/s	1% Exceedance Probability for Oct - Dec (CDFW Definition)
Adult Fish Passage Low Flow	23.40	ft^3/s	95% Exceedance Probability for Oct - Dec
Extreme Event: 2-year Flood	115.32	ft^3/s	See "Streamflow" Calculations
Extreme Event: 100-year Flood	756.23	ft^3/s	See "Streamflow" Calculations

Barrier 2 (Dam A)	Value		Comments
Powerhouse High Flow*	50.00	ft^3/s	Klamath Hydroelectric Project, EIS 2007
Powerhouse Low Flow*	15.00	ft^3/s	Klamath Hydroelectric Project, EIS 2007

**Note: Flows in the Dam A drainage are predominantly anthropogenic, from the powerhouse. The drainage area reporting to this area is very limited, and these two design flows will be representative of the flow regime in the Dam A drainage.*

Barrier 3 (Dam B)	Value		Comments
Juvenile High Flow	62.14	ft^3/s	1% Exceedance Probability for the peak month of juvenile release (Mar)
Adult Fish Passage High Flow	56.86	ft^3/s	1% Exceedance Probability for Oct - Dec
Adult Fish Passage Low Flow	8.40	ft^3/s	95% Exceedance Probability for Oct - Dec
Extreme Event: 2-year Flood	100.32	ft^3/s	See "Streamflow" Calculations
Extreme Event: 100-year Flood	741.23	ft^3/s	See "Streamflow" Calculations

Other Inputs

Barrier 1 (Lower)	Value		Comments
Natural Channel Width	15.00	ft	Measured from upstream and downstream transects
Broad-Crested Weir Coefficient	2.65		Brater et al., 1976; 5.0-ft wide crest; ~ 1.0 - 2.0 overflow
Floodplain Weir Elevation	2488.00	ft	
Floodplain Weir Crest Length	30.00	ft	Measured in CAD
Sill Crest Elevation	2483.00	ft	
Screen Angle to Horiz	60.00	deg	
Adult High Flow WSEL	2484.77	ft	See 'Tailwater' Calculations
Adult Low Flow WSEL	2484.12	ft	See 'Tailwater' Calculations
2-year Flood WSEL	2485.13	ft	See 'Tailwater' Calculations
100-year Flood WSEL	2487.21	ft	See 'Tailwater' Calculations

Barrier 2 (Dam A)	Value		Comments
Apron Width	29.00	ft	City of Yreka Intake Bldg to Hatchery Intake

Barrier 3 (Dam B)	Value		Comments
Apron Width	10.00	ft	Estimated from photograph of existing Dam B

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Calculations

Barrier 1 (Lower) Calculations

Picket Flow Depths & Velocities

The flow depths through the pickets were calculated from the backwater HEC-RAS calculations. These flow depths were then used to determine velocities by rotation angle about the stream transect and the vertical angle of the screens. Only adult fish passage flows were used, as this barrier will only be in operation during trapping periods.

Rotation Angle about Stream	Adult High Flow			Adult Low Flow		
	Discharge	Flow Depth	Flow Velocity	Discharge	Flow Depth	Flow Velocity
(°)	cfs	ft	ft/s	cfs	ft	ft/s
0	71.86	1.77	2.34	23.40	1.12	1.21
5	71.86	1.77	2.34	23.40	1.12	1.20
10	71.86	1.77	2.31	23.40	1.12	1.19
15	71.86	1.77	2.26	23.40	1.12	1.17
20	71.86	1.77	2.20	23.40	1.12	1.13
25	71.86	1.77	2.12	23.40	1.12	1.09
30	**71.86**	**1.77**	**2.03**	**23.40**	**1.12**	**1.04**

Upstream Water Surface Elevation / Head Loss

Water surface elevations at the fish barrier were calculated in HEC-RAS via backwater calculations. These calculations, however, do not include the additional head losses accounting for the picket barrier. Therefore head losses were calculated across the barrier using the screen head loss equations (USBR, 1987):

$$K_s = 1.45 - 0.45\frac{A_n}{A_g} - \left(\frac{A_n}{A_g}\right)^2$$

$$A_n = (1 - R_D)R_0 A_g$$

$$h_s = K_s\left(\frac{v_n^2}{2g}\right)$$

where:

$K_s =$ Screen loss coefficient
$h_s =$ Screen head losses, ft
$A_n =$ Net screen area (less screen and occlusions), ft^2
$A_g =$ Gross screen area, ft^2
$v_n =$ Net velocity (through net screen area), ft/s
$g =$ Gravitational constant, 32.2 ft/s^2
$R_D =$ Ratio of debris coverage
$R_o =$ Ratio of open area (clean bars)

It is assumed that the removable pickets will maintain 2.0' of freeboard above the upstream elevation of the fish passage high flow water surface with an additional 0.3' for screen occlusions.

Event	Discharge	Backwater Elevation	Gross Screened Area	% Open	Net Screened Area	Ratio An/Ag
	cfs	ft	ft^2		ft^2	
Adult Fish Passage High Flow	71.86	2484.77	30.7	50%	15.3	50%
Adult Fish Passage Low Flow	23.40	2484.12	19.4	50%	9.7	50%
Extreme Event: 2-year Flood	115.32	2485.13	36.9	50%	18.4	50%

Event	Loss Coeff	Net Velocity	Net Velocity Head	Head Loss	Clean Picket U/S Elev	Occluded Screen U/S Elev	Top of Picket Elevation
		ft/s	ft	ft	ft	ft	ft
Adult Fish Passage High Flow	0.975	4.69	0.34	0.33	2485.10	2485.4	**2487.40**
Adult Fish Passage Low Flow	0.975	2.41	0.09	0.09	2484.21	2484.5	-
Extreme Event: 2-year Flood	0.975	6.25	0.61	0.59	2485.72	2486.0	-

100-year Flood Elevation

It is conservatively assumed that for the 100-year flood, the pickets are in place and not able to be removed. They furthermore are assumed to be fully occluded with debris. Thus all flows will act as weir flow over the occluded pickets and the overflow weir in the floodplain. Calculations of the weir flow at the 100-year flood are provided below for setting the grade on the east bank of the stream.

Event	WSEL ft	Depth @ OF Weir ft	Length of OF Weir ft	OF Weir Discharge Coeff	OF Weir Discharge cfs
Extreme Event: 100-year Flood	2490.26	2.26	30.00	2.65	461

Event	Depth over occluded barrier ft	Length of occluded barrier ft	Height of Occluded Barrier ft	Rehbock Discharge Coeff	Barrier Discharge cfs	OF Weir Discharge cfs	Total Discharge cfs
Extreme Event: 100-year Flood	2.86	17.32	4.40	3.52	295	461	756

Given the conservative assumptions of the barrier remaining in place and being fully occluded by debris, a 7" freeboard was maintained on all walls, and 4" of freeboard was maintained on the elevation at either bank.

Wall Elevation	2490.85
Bank Elevation	2490.60

Barrier 2 (Dam A) Calculations

Apron Depths & Velocities

The depths and flow velocities on the Dam A high velocity apron were calculated according to a normal flow assumption. The aim of the high velocity apron is to provide a section that will be too shallow and too fast for an adult to jump from over Dam A. Velocities and flow depths were calculated for powerhouse high and low flows.

Design Flow cfs	Slope ft/ft	Width ft	Roughness Coeff, n	Normal Flow Depth in	Velocity ft/s	Apron Length ft	Drop ft
50.00	0.0625	29.00	0.015	2.4	8.48	16	1
15.00	0.0625	29.00	0.015	1.2	5.26	16	1

Tailwater

Tailwater was calculated to set the elevation of the downstream end of the apron. It was assumed that the apron would be in cut, and therefore some channel regrading would be required on the downstream side of the apron. The constructed channel slope was assumed to be a nominal slope of 0.5% to ensure that the cut would attain grade before the confluence. Channel dimensions were measured from the LiDAR surface in CAD.

Design Flow cfs	Slope ft/ft	Width ft	Side Slope, Z1 H:1V	Side Slope, Z2 H:1V	Roughness Coeff, n	Normal Flow Depth ft	Normal Flow Depth in	Velocity ft/s
50.00	0.005	35.00	2.00	2.00	0.030	0.58	7.0	2.4
15.00	0.005	35.00	2.00	2.00	0.030	0.28	3.4	1.5

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Barrier 3 (Dam B) Calculations

Apron Depths & Velocities

The depths and flow velocities on the Dam B high velocity apron were calculated according to a normal flow assumption. The aim of the high velocity apron is to provide a section that will be too shallow and too fast for an adult to jump from over Dam B. Velocities and flow depths were calculated for juvenile high flows and adult high and low flows.

Design Flow cfs	Slope ft/ft	Width ft	Roughness Coeff, n	Normal Flow Depth in	Velocity ft/s	Apron Length ft	Drop ft
62.14	0.0625	11.50	0.015	4.9	13.10	16	1
56.86	0.0625	11.50	0.015	4.7	12.66	16	1
8.40	0.0625	11.50	0.015	1.5	6.00	16	1

Tailwater

Tailwater was calculated to set the elevation of the downstream end of the apron. It was assumed that the apron would be in cut, and therefore some channel regrading would be required on the downstream side of the apron. The constructed channel slope was assumed to minimize tailwater, while also seeking to minimize the distance the slope would chase the natural 3.0% grade of the stream. Channel dimensions were measured from the LiDAR surface in CAD.

Design Flow cfs	Slope ft/ft	Width ft	Side Slope, Z1 H:1V	Side Slope, Z2 H:1V	Roughness Coeff, n	Normal Flow Depth ft	Normal Flow Depth in	Velocity ft/s
62.14	0.01	15.30	1.00	1.50	0.035	0.97	11.7	3.9
56.86	0.01	15.30	1.00	1.50	0.035	0.92	11.1	3.8

Discussion

Based on the foregoing calculations, there remain two guidelines/criteria that are unmet by the design of the lower picket barrier (Barrier 1). These will be discussed in turn:

NMFS 5.3.2.2 - Picket Velocities

High picket velocities can pose a concern for impingement of fish upstream of the barrier on screens or picket panels. Meeting the 1 ft/s picket velocity criterion, however, has proven challenging in the setting of small mountain streams across the Pacific Northwest, such as Fall Creek. It is not anticipated that the 1 ft/s picket velocity criterion will be met by this design. However, it is not expected that the picket barrier will pose a fish impingement concern, because of the following mitigating factors:

- The fish habitat above the FCFH exclusion barrier is very limited, and fish are not anticipated upstream of the picket barrier where impingement could occur.
- The exposure window when the pickets will be in place is limited to the period of trapping. At all other times the pickets will be removed, and streamflow will flow through naturally.
- The screen will be oriented at an angle to the stream transverse, increasing the wetted area of the picket panels and decreasing the average velocities through the pickets.
- Natural flow velocities in the stream around this location are as high as 4.5 ft/s under high flow conditions. The flow through the pickets will be much less than the natural surrounding stream, due to the orientation of the barrier, the backwater caused by the picket head losses, and the local shallowing of the slope for the concrete sill.
- In the language of the NMFS guidelines, this is not a "criterion" but is meant to serve as a "guideline." Given all of the site-specific mitigating factors above, it is expected that the current design is within the spirit of the guideline.

NMFS 5.3.2.7 - Minimum Submerged Picket Depth

The minimum submerged depth at the picket barrier is a criterion that is also challenging to meet in the setting of the FCFH barrier, and in other similar locations across the Pacific Northwest. It is not anticipated that this criterion will be met for the FCFH exclusion barrier. Similar reasons for relaxation of this criterion apply as those given above. In addition, it may be noted:

- The natural flow depth through this region is only about 9 inches deep at low flow. Meeting the minimum submerged picket depths would require significant deviation about the natural channel flows.
- The current design will cause a backwater that will raise the water surface elevations as high as possible. Further modifications would require drastic alteration of the natural stream environment.
- No alternative locations at the site are anticipated to be significantly more confined than the location selected, and therefore the water surface elevations at other locations about the site should not show much improvement in meeting this criterion.

It is therefore deemed that, while these represent exceptions to the NMFS guidelines and/or criteria, these are common exceptions required in small stream/tributary settings such as this one. The design meets the spirit of the NMFS (2011) guidelines to the extent possible in such a setting.

Conclusions

The above calculations and discussion detail the design of the exclusion barrier system at the FCFH site. It was elected that 3-part barrier system be constructed, with a temporary picket barrier system that is used for trapping of adults only, and a velocity barrier system at Dam A and Dam B that uses existing infrastructure to the greatest possible extent. As is the case with many sites on small streams, such as Fall Creek, some of the NMFS criteria are unattainable due to site specific constraints. These are discussed in detail above.

SUBJECT: Klamath River Renewal Corporation | BY: A. Leman | CHK'D BY: V. Autier
Fall Creek Hatchery | DATE: 10/28/2020
Fish Barrier Nappe Aeration Pipes | PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to determine the location and size of the nappe aeration pipes at the velocity barriers.

References

- *Blaisdell, F.W. 1954. Equation of the Free-Falling Nappe. Proceedings, ASCE, vol. 80, separate no. 482, 16pp. August 1954.*
- *Bos, M.G. Ed. 1989. Discharge Measurement Structures. International Institute for Land Reclamation and Improvement. Wageningen, Netherlands.*
- *Chow, V.T. 1959. Open-Channel Hydraulics. McGraw Hill, New York, NY.*

Method

The sizing and location of the nappe aeration pipe will follow the procedure below:

1. The nappe aeration pipe will be sized according to the equations of Bos (1989) for air demand in the lower nappe subspace.

2. The location of the nappe aeration pipe will be determined by calculating the nappe profile on the downstream side of the weir overflow, according to the equations given by Chow (1959; among others).

Equations used in this analysis are summarized below:

Bos (1989)
"The Aeration Demand of Weirs"

The method of Bos begins by calculating the air demand from the weir, which he derives from the data of Howe (1955). Then a maximum allowable percent increase in flow rate is assumed, and the below relationship is used to determine the resultant gauge pressure in the lower nappe subspace. Once the pressure has been determined, the diameter of the air vent pipe can be calculated assuming that the pressure under the nappe is low enough that the density of air can be considered constant. Then ordinary hydrynamic equations for losses in the pipe are used to determine the pipe size that would produce the end pressure previously calculated. A summary of all equations and variable definitions are provided below:

Air Demand

$$Q_{air} = 0.1\left(\frac{h_1}{y_p}\right)^{3/2} Q_u$$

$$y_p = \Delta z\left(\frac{q^2}{g\Delta z^3}\right)^{0.22}$$

Percent increase in flow rate

$$X_Q = 20\left(\frac{-p_{sn}}{h_1}\right)^{0.92}$$

Air pipe capacity

$$p_{sn} = \frac{\rho_{air}}{\rho_w}\left[K_e + \frac{fL}{D_p} + K_b + K_{ex}\right]\frac{v_{air}^2}{2g}$$

$$Q_{air} = \frac{1}{4}\pi D_p^2 v_{air}$$

where:
$Q_{air} =$ Maximum air demand, cfs
$Q_u =$ Unsubmerged weir discharge, cfs
$h_1 =$ Weir hydraulic head, ft
$y_p =$ Depth of pool between the weir and the nappe, ft
$\Delta z =$ Weir drop height, ft
$q =$ Weir specific discharge, ft2/s
$X_Q =$ Percent increase in flow rate, %
$p_{sn} =$ Gauge pressure within the nappe subspace, ft
$K_e =$ Air pipe entrance loss coeff, 0.5
$K_b =$ Air pipe bend loss coeff, 1.1
$K_{ex} =$ Air pipe exit loss coeff, 1.0
$f =$ Darcy-Weisbach friction loss coeff, assume 0.02 (per Bos)
$L =$ Length of air pipe, ft
$D_p =$ Pipe inner diameter, ft
$v_{air} =$ Velocity of air in pipe, ft/s
$\rho_{air}/\rho_w =$ Ratio of air density to water, ~1/830

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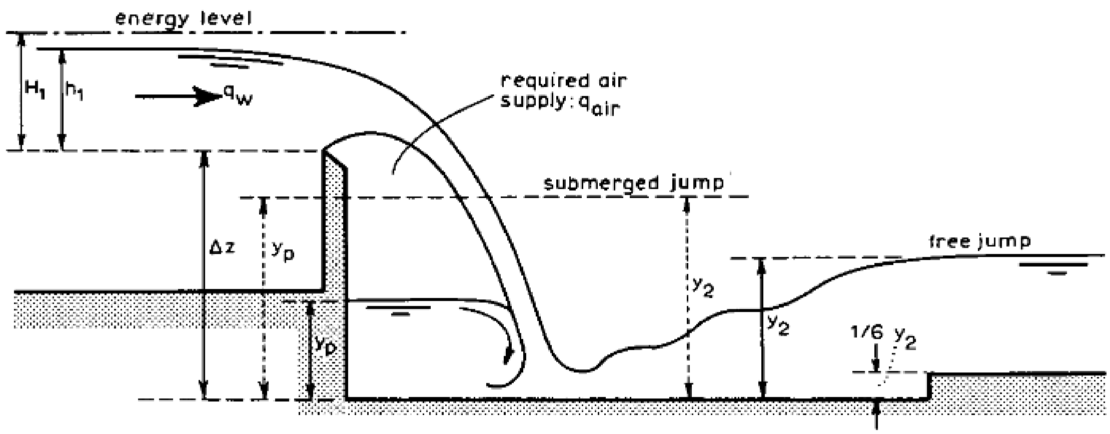

Figure 1. Definition Sketch for Air Demand Calculations (Source; Bos, 1989)

Chow (1959)
Sharp Crested Weir Nappe Profiles

The lower nappe profile is theoretically parabolic in shape. Coefficients were proposed by Blaisdell (1954) based on data from the USBR, Hinds, Creager, and Justin, and from Ippen. The equations used to calculate the lower nappe profile are summarized below with all variable definitions.

Lower Nappe

$$\frac{y}{H} = A\left(\frac{x}{H}\right)^2 + B\frac{x}{H} + C$$

$$A = -0.425 + 0.25\frac{h_v}{H}$$

$$B = -0.411 - 1.603\frac{h_v}{H} - \sqrt{1.568\left(\frac{h_v}{H}\right)^2 - 0.892\frac{h_v}{H} + 0.127}$$

$$C = 0.150 - 0.45\frac{h_v}{H}$$

where:

$y =$ Vertical distance (downward positive) from the crest, ft
$H =$ Weir energy head, ft
$x =$ Horizontal distance from the crest, ft
$h_v =$ Velocity head of the approach flow, ft/s

Francis Weir Equation &
Rehbock Weir Coefficient

In support of the above calculations, the weir energy head needs to be calculated. Weir energy head was calculated according to the Francis Weir Equation, using the Rehbock Weir Coefficient. The equations and variable definitions are provided below:

$$Q_u = \frac{2}{3}C_1\sqrt{2g}LH^{3/2}$$

$$C_1 = \left(0.6035 + 0.0813\frac{H}{Y} + \frac{0.000295}{Y}\right)\left(1 + \frac{0.00361}{H}\right)^{3/2}$$

where:

$Y =$ Upstream height of the weir, ft
<other values as previously defined>

Inputs

The following inputs were used for each of the 2 velocity fish barriers:

Inputs	Units	Dam A	Dam B	Description
Weir Drop Height, Δz	ft	4.08	1.50	Dam A based on crest elev.; Dam B assumed 8" FB
Weir Upstream Height, Y	ft	4.00	4.35	See Drawings
Unsubmerged Weir Discharge, Q_u	cfs	50.00	62.14	See "Fish Barrier" Calculations
Weir Length	ft	18	10	Total length of overflow
Specific Discharge, q	ft^2/s	2.78	6.21	Discharge divided by length

Calculations

Air Demand/Pipe Sizing

Location	Guess Energy Head ft	Rehbock Weir Coeff, C_1	Discharge Coeff, C_D	Weir Upstream Height, Y ft	Weir Length, L ft	Discharge, Q cfs	Energy Head, H_1 ft	Goal Seek to 1.0
Dam A	0.88	0.63	3.35	4.00	18	50.00	0.88	1.00
Dam B	1.76	0.64	2.65	4.35	10	62.14	1.76	1.00

Location	Guess U/S[§] Hydraulic Head ft	Flow Area, A ft^2	Velocity, V ft/s	Velocity Head, h_v ft/s	Calc U/S Hydraulic Head ft	Goal Seek to 1.0	Hydraulic Head, h_1 ft
Dam A	4.87	87.72	0.57	0.01	4.88	1.00	0.88
Dam B	6.10	60.99	1.02	0.02	6.10	1.00	1.75

Location	Specific Discharge, q ft^2/s	Weir Drop Heigh, dz ft	Pool Depth, y_p ft	Air Demand, Q_{air} cfs	Permiss Increase in Q, X_Q %	Pressure under Nappe, p_{sn} ft*
Dam A	2.78	4.08	1.18	3.22	1.0%	0.0340
Dam B	6.21	1.50	1.19	11.01	2.0%	0.1445

Location	ρ_{air}/ρ_w	Entrance Loss Coeff, K_e	Bend Loss Coeff[†], K_b	Exit Loss Coeff, K_{ex}	Friction Coeff, *f*	Length of Vent Pipe ft
Dam A	0.001	0.5	3.3	1	0.02	20
Dam B	0.001	0.5	3.3	1	0.02	2.5

Location	Guess Pipe (Inner) Diameter in	Air Velocity, v_{air} ft/s	Number of Pipes[‡]	Calculated Pipe I.D., D_p in	Goal Seek to 1.0	
Dam A	4.1	17.5	2	**4.1**	1.0	<---- 6-inch pipes
Dam B	5.0	39.6	2	**5.0**	1.0	<----- 8-inch pipe (splits into two)

[§] *U/S hydraulic head evaluated where indicated on the definition sketch, where the velocity uses the entire flow area.*
** All pressures expressed in feet are in feet of water column*
[†] *Assumes 3 bends: 1 in wall, 2 above wall in "gooseneck" configuration*
[‡] *Assumes air demand taken by pipes on either side of the overflow.*

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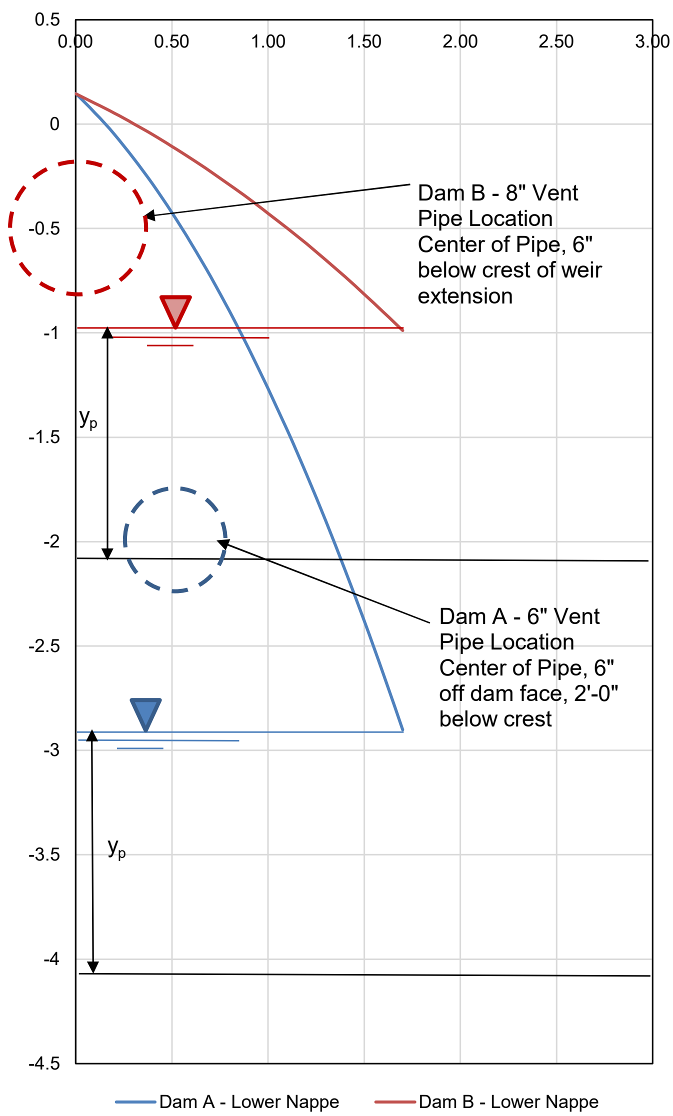
Nappe Profiles / Pipe Location

Location	Velocity Head, h_v ft/s	Energy Head, H_1 ft	Coeff A	Coeff B	Coeff C
Dam A	0.01	0.88	-0.424	-0.769	0.147
Dam B	0.02	1.76	-0.423	-0.771	0.146

x ft	Dam A y ft	Dam B y ft
0.00	0.14743039	0.14589009
0.10	0.05491004	0.10087265
0.20	-0.0484656	0.05314115
0.30	-0.1626966	0.0026956
0.40	-0.2877828	-0.050464
0.50	-0.4237243	-0.1063377
0.60	-0.5705212	-0.1649254
0.70	-0.7281733	-0.2262271
0.80	-0.8966808	-0.290243
0.90	-1.0760435	-0.3569728
1.00	-1.2662616	-0.4264168
1.10	-1.467335	-0.4985747
1.20	-1.6792636	-0.5734468
1.30	-1.9020476	-0.6510329
1.40	-2.1356868	-0.731333
1.50	-2.3801814	-0.8143472
1.60	-2.6355312	-0.9000755
1.70	-2.9017364	-0.9885178



Conclusions

Vent pipes were sized and lower nappe profiles were calculated to locate the vent pipe under the nappe, according to the air demand method of Box (1989) and the nappe profiles outlined in Chow (1959). The following table summarizes the design of the vent pipes:

Location	Number of Pipes	Nom Pipe Size in	Pipe Type*	Pipe CL Location Below Crest ft-in	Pipe CL Location Off Wall ft-in	Config-uration	Additional Information
Dam A	2	6	Sched. 80 PVC	2'-0"	0'-6"	Gooseneck to atmos	Located either side of the concrete apron; riser outside of the apron structure
Dam B	2	8	Sched 80 PVC	0'-8"	0'-6"	Gooseneck to atmos	Located either side of the stoplogs; pipe in the flow path of the lower nappe. See drawings for details.

** Because the pipe will be cast into the concrete, Sched 80 PVC was used to withstand any additional stresses from the construction method.*

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 4/2/2020	
	Denil Fishway	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to size the Denil fishway for the design flow.

References

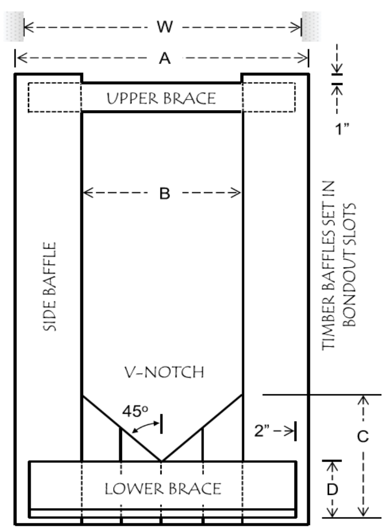
- *NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.*
- *NRCS (Natural Resources Conservation Service). 2007. Technical Supplement 14N: Fish Passage and Screening Design. National Engineering Handbook. USDA: NRCS. August 2007.*
- *Odeh, M. 2003. Discharge Rating Equation and Hydraulic Characteristics of Standard Denil Fishways. Journal of Hydraulic Engineering, 129(5), 341-348.*
- *Slatick, E. 1975. Laboratory Evaluation of a Denil-Type Steeppass Fishway with Various Entrance and Exit Conditions for Passage of Adult Salmonids and American Shad. Marine Fisheries Review, 37.*
- *USFWS (U.S. Fish and Wildlife Service). 2017. Fish Passage Engineering Design Criteria. USFWS, Northeast Region R5, Hadley, MA.*

Design Criteria

The NMFS (2011) criteria for a Denil fishway are summarized below:

NMFS Guidelines	Value		Comments
Debris Characterization	-		Must be low/no debris accumulation, NMFS 4.10.2.1
Maximum Slope	20%		NMFS 4.10.2.1
Maximum Avg. Chute Velocity	5	ft/s	NMFS 4.10.2.1
Max Horiz. Distance b/w Rest Pools	25	ft	NMFS 4.10.2.1
Minimum Flow Depth	2	ft	NMFS 4.10.2.1

Standard Denil baffle sizes used by the USFWS Region 5 (Northeast; 2017) were used for reference:



STANDARD DENIL GEOMETRY *

W **	A	B	C	D	S ***
4' - 0"	4' - 3"	2' - 4"	2' - 0"	1' - 0"	2' - 6"
3' - 6"	3' - 9"	2' - 0"	1' - 9"	10½"	2' - 4"
3' - 0"	3' - 3"	1' - 9"	1' - 6"	9"	2' - 0"
2' - 6"	2' - 9"	1' - 5½"	1' - 3"	7½"	1' - 8"
2' - 0"	2' - 3"	1' - 2"	1' - 0"	6"	1' - 4"

* U.S. Fish and Wildlife Service criteria
** Denil channel width denoted by W; typically inside width of concrete channel
*** Horizontal (longitudinal) spacing of baffles in channel denoted by S

No standard design guidance or requirements were found from CDFW, or USFWS Region 8.

Method

A rating curve will be calculated to determine appropriate geometries of a Denil fishway, according to the equations of Odeh (2003):

$$Q = (1.34 - 1.84S_0)h_u^{1.75}B^{1.75}\sqrt{gS_0}$$

$$h_u = H - D\sin(45° + \tan^{-1}(S_0))$$

where:
- $Q =$ Design discharge, cfs
- $S_0 =$ Bed slope, ft/ft
- $h_u =$ Depth above V-notch, ft
- $B =$ Width through baffle, ft
- $g =$ Gravitational constant, 32.2 ft/s^2
- $H =$ Depth above invert, ft
- $D =$ Height of V-notch above invert, ft

This rating curve can then be converted to an average velocity basis (for comparison with NMFS criterion), by dividing the flow rate by the flow area:

$$V_{avg} = \frac{Q}{WH}$$

where:
- $W =$ Chute width, ft

<all other values as previously defined>

This was calculated on the gross chute area because it is called an "average chute design velocity" in the NMFS (2011) criteria. As flows pass down the chute, the angled baffles will result in variable flow areas along the entire length.

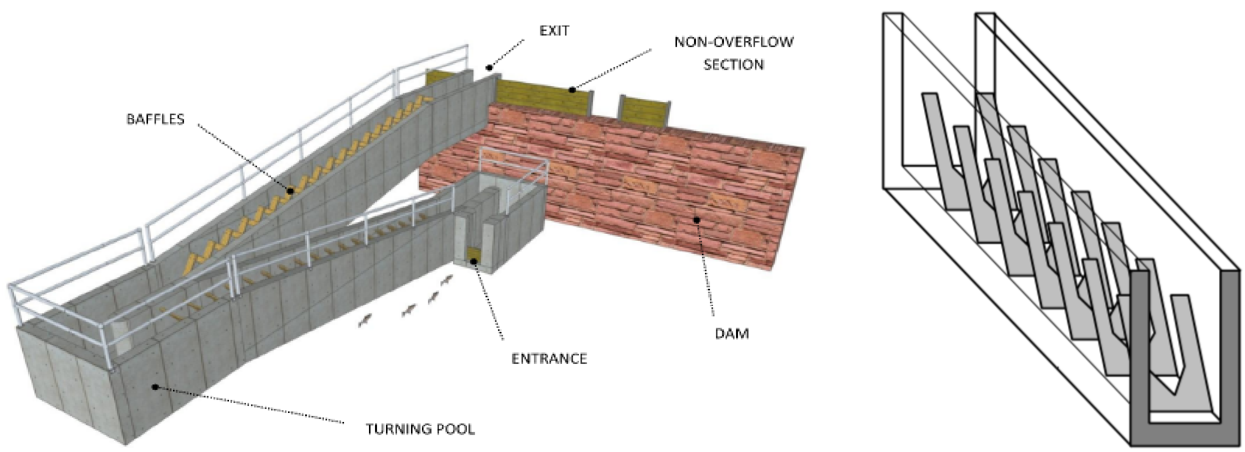


Figure 1. Denil Fishway Schematics (Left Source: USFWS, 2017; Right Source: NRCS, 2007)

Finally, a bar screen will be designed for the downstream extents of the Denil fishway, that will be lowered into place in "non-trapping" seasons to prevent entrainment of resident, non-anadromous fish. Because the flow is out of the Denil fishway, and no fish are expected upstream of the screen, impingement of fish is not a concern for this screen. Therefore, bars were spaced at a clear spacing of 1-inch such that resident adult fish would be excluded from the fishway, and the majority of debris and detritus could pass through the bar screen openings. For smaller fish, the baffles in the Denil fishway will be pulled in the "non-trapping" season making the fishway impassable to smaller fish. The head differential across the screen, in support of structural design, was calculated according to the USBR (1987) *Design of Small Dams* equation for screen losses:

$$K_s = 1.45 - 0.45\frac{A_n}{A_g} - \left(\frac{A_n}{A_g}\right)^2$$

$$A_n = (1 - R_D)R_0A_g$$

$$h_s = K_s\left(\frac{v_n^2}{2g}\right)$$

where:
- $K_s =$ Screen loss coefficient
- $h_s =$ Screen head losses, ft
- $A_n =$ Net screen area (less screen and occlusions), ft^2
- $A_g =$ Gross screen area, ft^2
- $v_n =$ Net velocity (through net screen area), ft/s
- $g =$ Gravitational constant, 32.2 ft/s^2
- $R_D =$ Ratio of debris coverage
- $R_o =$ Ratio of open area (clean bars)

This was performed for a maximum occlusion of the screen of 50%, assuming that the screen would not be actively managed, but that for large debris accumulations, the hatchery staff would dislodge and remove accumulated debris.

It was furthermore assumed that the screen would provide about 6-inches of freeboard on the 2-year extreme event with the calculated head loss. This will prevent fish from getting entrained in the Denil for regular events, and the screen will only be overtopped by extreme (rare) events.

Inputs

The following inputs were used for calculation of the Denil fishway rating curve:

Hydraulic Parameters	Value		Comments
Design Discharge	10	cfs	Typical for operation of the fish ladder

Tailwater Parameters	Value		Comments
High Tailwater	2484.77	ft msl	from Tailwater calculations
Typical Tailwater	2484.24	ft msl	from Tailwater calculations
Low Tailwater	2484.12	ft msl	from Tailwater calculations
Streambed Elevation	2483.00	ft msl	from Tailwater calculations

Upper Pool Parameters	Value		Comments
Denil Crest Elevation	2486.50	ft msl	Based on desired water surface

Fishway Parameters (User Inputs)	Value		Comments
Fishway Width, W	2.5	ft	Sized for for flow using standard Denil sizes
Baffle Inner Width, B	1.4583	ft	Standard, W = 2.5
Baffle V-Notch Bottom Height, D	0.625	ft	Standard, W = 2.5
Baffle Spacing, S	1.67	ft	Standard, W = 2.5
Bed Slope, S_0	0.18	ft/ft	Determined to meet depth requirements
Baffle Angle, α	45	deg	Standard

Screen Parameters	Value		Comments
Screen Exposed Width	2.5	ft	Same width as fishway
Min. WSEL	2483.00	ft msl	Streambed Elevation
Max. Design WSEL	2485.13	ft msl	2-year Event Elevation (see 'Tailwater' calculations)
Screen Bottom Elevation	2482.07	ft msl	From Fishway calculations (see below)
Min. Depth	0.93	ft	Calculated
Max. Depth	3.06	ft	Calculated
Bar Clear Spacing	1	in	Assumed, see 'Method' above
Bar Diameter	0.5	in	Assumed
Max. Screen Occlusion	50%		Assumed, see 'Method' above

Calculations

Rating Curves

	Total Depth, H	Depth Over Baffle, h_u	Discharge, Q	Avg Velocity, V
	ft	ft	cfs	ft/s
	0.625	0.11	0.10	0.06
	0.88	0.36	0.79	0.36
	1.13	0.61	1.99	0.71
	1.38	0.86	3.62	1.05
	1.63	1.11	5.66	1.39
	1.88	1.36	8.07	1.72
	2.13	1.61	10.84	2.04
	2.38	1.86	13.95	2.35
	2.63	2.11	17.39	2.65
	2.88	2.36	21.15	2.94
	3.13	2.61	25.22	3.23
	3.38	2.86	29.59	3.51
DESIGN	2.05	1.54	10.00	1.95

Velocity	1.95	**< 5 ft/s**
Depth	2.05	**> 2 ft**

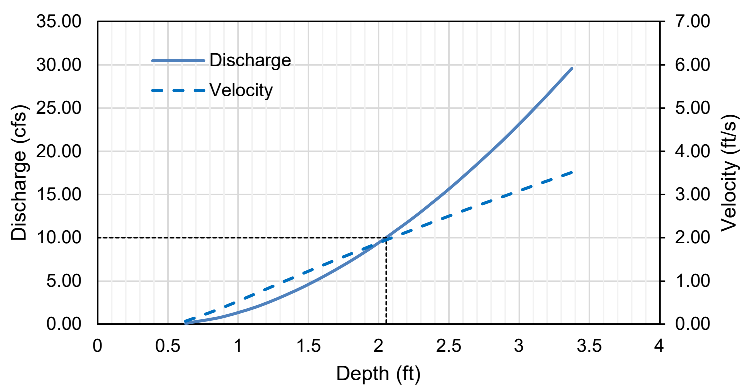


Figure 2. Denil Fishway Rating Curve

Fishway Length

Denil Crest El	2486.50	ft msl	
Denil Bottom El	2482.07	ft msl	[Low Tailwater less calculated flow depth]
Elevation Difference	4.43	ft	
Slope	0.180	ft/ft	
Required Length	**24.6**	**ft**	
Intermediate Rest Pools?	0	#	
Number of Baffles	14	#	

Screen Losses

Scenario	Guess Head Loss (ft)	Gross Area, A_g (ft^2)	Net Area*, A_n (ft^2)	Screen Loss Coeff, K_s	Net Velocity†, V_n (ft/s)	Head Loss, h_s (ft)	Goal Seek to 1.0
Low Flow	0.9	4.6	1.4	1.21	6.9	0.9	1.0
Max Flow	0.3	8.3	2.6	1.21	3.8	0.3	1.0

** Net area was calculated based on the bar spacing. Cross-bracing in the lateral direction will be subject to the structural design, and therefore a 5% deduction of area was assumed for the lateral supports.*

† Note, velocities represent the two extreme conditions. It is not expected that the low flow condition would occur unless there was no flow in the stream, and 50% occlusion on the screen. This will be a conservative estimate for structural design of the screen.

Conclusions

A Denil fishway is designed above for conveyance of Chinook and Coho to the trap. It is found that adequate hydraulics (per NMFS, 2011 criteria) can be provided for a bedslope of 0.18 ft/ft and with the baffle geometry summarized below in Figures 3 and 4. Given the steepness of the structure and the small vertical distance that needs to be traversed, the Denil fishway could maintain a single run with no intermediate resting pools.

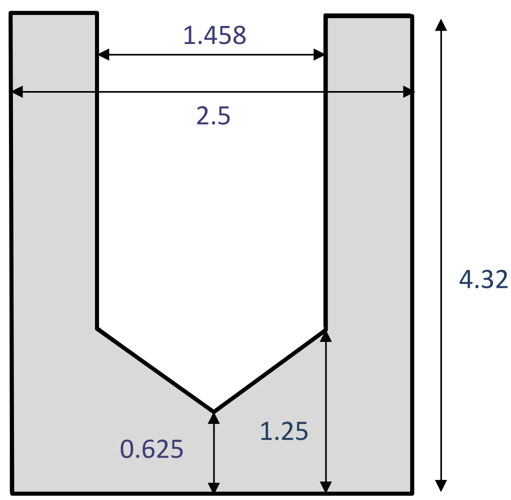


Figure 3. Baffle Geometry Summary

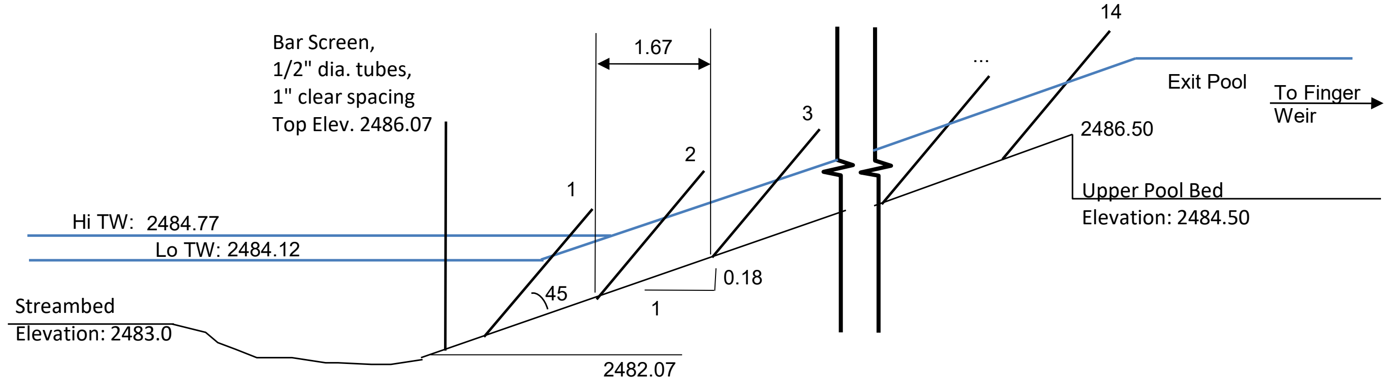


Figure 4: Denil Fishway Profile Summary

In addition, a bar screen was evaluated for the bottom of the fishway, for use in "non-trapping" seasons. It was found that the maximum head differential across the screen was 0.9 ft at the extreme low-flow condition.

SUBJECT:	Klamath River Renewal Corporation	**BY:** ASL	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** ########	
	Finger Weir Design	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to size the length of the finger weir.

References

- *American Society of Civil Engineers (ASCE). 1965. Factors Influencing Flow in Large Conduits. Proc. ASCE, Journal of the Hydraulics Division, Vol 91, No. HY6, November 1965.*
- *Bell, M. 1991. Fisheries handbook of engineering requirements and biological criteria. U.S. Dept. of the Army, Army Corps of Engineers, North Pacific Division, Fish Passage Development and Evaluation Program.*
- *Tullis, J. Paul. 1989. Hydraulics of Pipelines, Pumps, Valves, Cavitation, Transients. New York: John Wiley & Sons.*

Method

The finger weir will be mounted so as to adjust the height of the weir to provide 2 to 6 inches of flow depth over the fingers per the fisheries handbook (Bell, 1991).

Weir Flow

The flow over the weir will be calculated according to the equation:

$$Q = C_w C_v L H_w^{1.5}$$

where:
$Q =$ Design discharge, cfs
$C_w =$ Weir discharge coefficient
$C_v =$ Villemonte submerged weir coefficient
$L =$ Weir crest length, ft
$H_w =$ Weir head, ft

Discharge Coefficient

The discharge coefficient will be calculated according to the following equation:

$$C_w = C_c \frac{2}{3}\sqrt{2g}$$

where:
$C_c =$ Sharp crested weir coefficient, 0.62
$g =$ Gravitational constant, 32.2 ft/s^2

This is modified for the rounded crest of the finger weir, by applying a factor from Miller (1968) for rounded edge orifices:

$$K_s = \frac{1}{C_c^2}$$ [Tullis, 1989; Eq 4.7] for free discharge valves

$$K_r = K_s C_{rad}$$ [Miller, 1968]

$$C_{c,r} = \sqrt{\frac{1}{K_r}}$$

where:
$K_s =$ Sharp crest loss coefficient
$K_r =$ Rounded crest loss coefficient
$C_{rad} =$ Rounded edge coefficient
$C_{c,r} =$ Rounded crest weir coefficient

Submerged Weir Discharge Coefficient

The coefficient for submerged weir flow is calculated as follows:

$$C_v = \left(1 - \left(\frac{H_d}{H_w}\right)^{3/2}\right)^{0.385}$$

where:
$H_d =$ Downstream head on weir, ft

Head Loss Through Fingers

The head on the weir is equal to the head upstream of the weir and fingers less the head losses through the finger slots:

$$H_w = H_u - h_L$$

where:
$H_w =$ Head at the weir, ft
$H_u =$ Head upstream of weir and fingers, ft
$h_L =$ Head loss through finger slots, ft

And the head loss through the finger slots can be calculated as:

$$h_L = K_f \frac{(PQ/A)^2}{2g}$$

where:
$K_f =$ Finger slot loss coefficient, ft
$P =$ Proportion of flow through the finger slots, %
(i.e. not the 2-6 inches over the top)
$A =$ Flow area through the finger slots, ft^2

The flow area through the finger slots can be calculated as:

$$A = LB\cos\theta$$

where:
$B =$ Chord length of fingers, ft
$\theta =$ Angle of finger chord to vertical, degree

And lastly, the finger slot loss coefficient was determined based on the following chart for flows through bar racks:

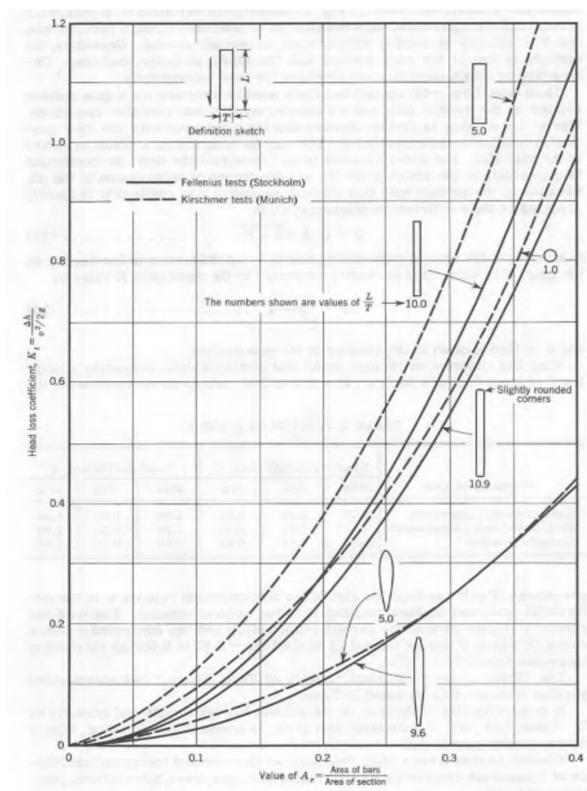


FIG. 2

Figure 1. Loss Coefficients for Flows through Bar Racks (Source; ASCE, 1965)

Inputs

The following parameters were adopted for these calculations

Parameter	Units	Value	Description
Design discharge	cfs	**3.33**	Water right, divided equally to 3 ponds
(MIN) -15%	cfs	2.8	
(MAX) +15%	cfs	3.8	
Sharp Crested Weir Coeff, Cc		**0.62**	from Rouse
Rounded Edge Coeff, Crad		**1.00**	No rounded edge
Finger Loss Coefficient, Kf		**0.42**	See chart above; assume 1/2" dia. bars, 1-1/2" clear
Proportion of Flow thru Fingers, P		**87.5%**	Assumed
Chord Length of Fingers, B	ft	1.00	Assumed, to produce 2" - 6" over fingers
Finger Chord Angle to Vert, θ	deg	67.5	Assumed
Gravitational Constant, g	ft/s^2	32.2	
Upstream Head, Hu	ft	0.66	Assumed, 8"
Downstream Head, Hd	ft	0.0	

Calculations

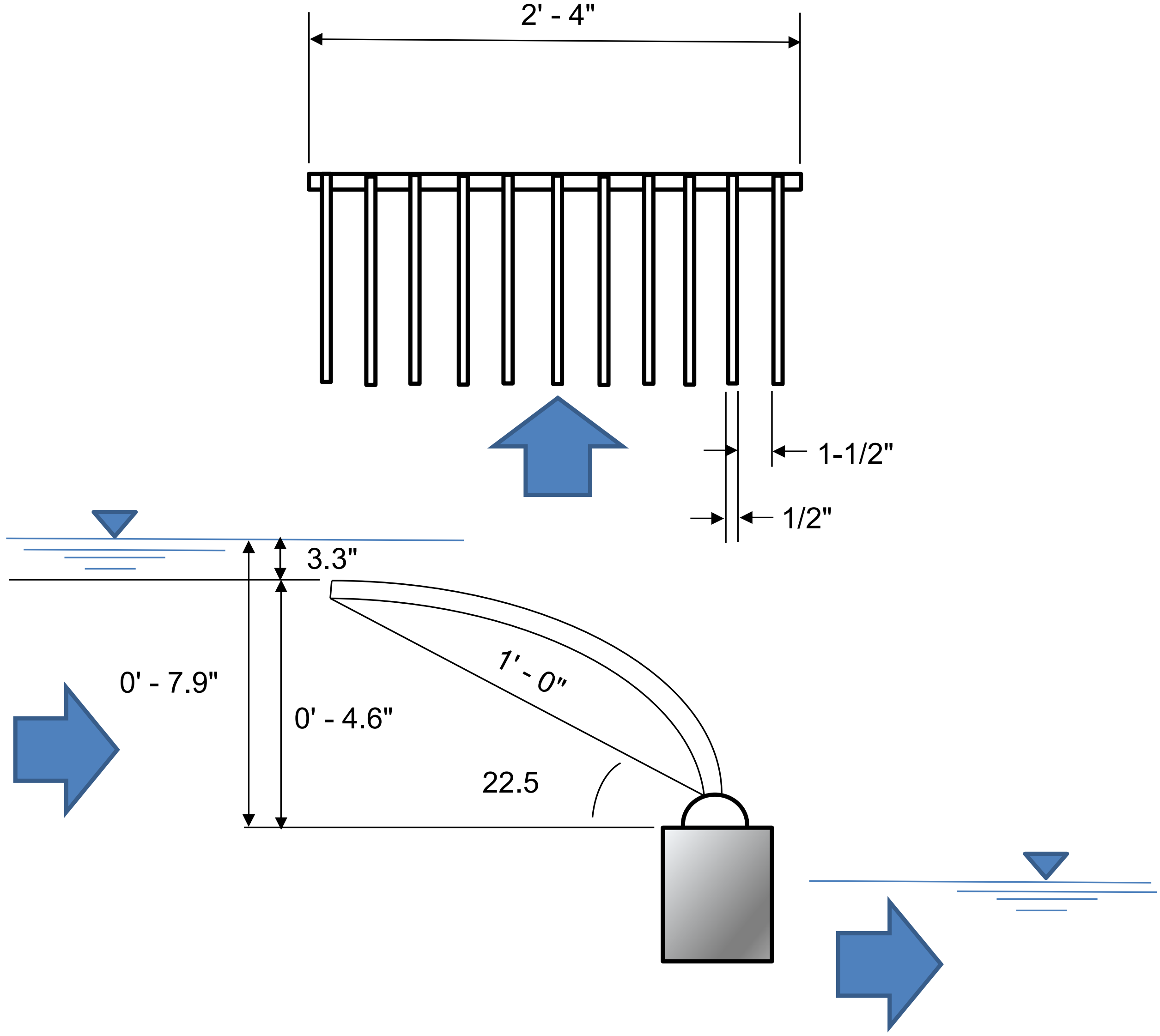
The required weir length was calculated iteratively according to the equations above. The following scenarios were run:

1. Normal - calculates the required weir length, based on the design upstream head.
2. Rounded - calculates the upstream head based on the weir length to a rounded value.
3. Flow sensitivity (low) - calculates the upstream head based on a low flow (-15%).
4. Flow sensitivity (high) - calculates the upstream head based on a high flow (+15%).
5. Coefficient sensitivity (low) - calculates the upstream head based on a low weir coefficient (-10%).
6. Coefficient sensitivity (high) - calculates the upstream head based on a high weir coefficient (+10%).

Scenario	Q cfs	L ft	H_u ft	$C_{c,r}$	C_w	A ft^2	h_L ft	H_w ft	Q_{calc} cfs	Depth above Fingers in
Normal	3.33	2.32	0.66	0.620	3.317	0.89	0.070	0.590	3.49	3.3
Rounded	3.33	2.33	0.66	0.620	3.317	0.89	0.069	0.588	3.49	3.3
Q - 15%	2.8	2.33	0.57	0.620	3.317	0.89	0.049	0.520	2.90	2.2
Q + 15%	3.8	2.33	0.74	0.620	3.317	0.89	0.090	0.645	4.01	4.2
Cw - 10%	3.33	2.33	0.70	0.620	2.985	0.89	0.069	0.629	3.47	3.8
Cw + 1'0%	3.33	2.33	0.62	0.620	3.649	0.89	0.069	0.552	3.49	2.9

Conclusions

The finger weir crest length and finger orientation were sized such that the recommended depth of 2-6 inches would be maintained above the fingers for the design flow. The orientation is summarized below:



These above orientation was subjected to sensitivity analysis on both the flow over the finger weir and the weir coefficient. It was found that for low flows, some nominal depth would be maintained over the fingers, however the fingers would remain submerged. This was deemed acceptable given that there will be control of the flow through the ponds via valves at the head of the ponds.

For high flows, it was found that the 6 inch recommendation was exceeded by about 2.5 inches. This is not expected to result in any escapement, however, if this becomes a concern the flow to the pond may be adjusted. It is not expected that more than 3.3 cfs will report to this pond.

If the weir coefficient is found to be overestimating by 10%, the depth above the fingers are found to be less than 1/2 inch above the recommended range. This could be controlled via flow through the pond, as in the case above, or by allowing the fingers to rotate such that the desired depths above the fingers are attained.

Therefore, the finger weir orientation depicted above is expected to meet the design intent.

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** N. Cox
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Settling Pond	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to check the size of the settling pond meets typical criteria for settling solids.

References

- *Idaho Department of Environmental Quality (Idaho DEQ), nd. Idaho Waste Management Guidelines for Aquaculture Operations. Published online: https://www.deq.idaho.gov/media/488801-aquaculture_guidelines.pdf, Accessed March 2020.*

- *Lindeburg, Michael R. 2014. Civil Engineering Reference Manual, Fourteenth Edition. Professional Publications, Inc. Belmont, CA.*

Method

This sheet will check that the overflow rate is less than the accepted values of settling velocity for aquaculture waste (Idaho DEQ, nd). The overflow rate is defined as:

$$V_o = \frac{Q}{A_s} < V_s$$

where:
$V_s =$ Settling velocity, ft/s
$V_o =$ Overflow velocity, ft/s
$A_s =$ Settling pond surface area, ft^2
$Q =$ Discharge, cfs

These calculations will also determine the weir elevation for setting the water surface through the settling pond according to the equation:

$$Q_w = C_D B \sqrt{2g} h^{3/2}$$

where:
$Q_w =$ Weir overflow, cfs
$C_D =$ Discharge coefficient
$B =$ Weir length, ft
$g =$ Gravitational constant, 32.2 ft/s^2
$h =$ Head over the weir, ft

Finally, these calculations will determine the head on the discharge pipe required to push the design overflow from the settling pond into the Denil. This will be performed by determining the required head from the orifice equation, and adding any additional losses in the pipe from both friction and minor losses:

$$Q_o = C_D A_0 \sqrt{2gh_o}$$

$$h_L = \frac{fL}{2Dg}\left(\frac{Q_o}{A_0}\right)^2 + \frac{K_L}{2g}\left(\frac{Q_o}{A_0}\right)^2$$

$$h_{SP} = h_D + h_o + h_L$$

where:
$Q_o =$ Orifice flow rate, cfs
$A_0 =$ Pipe flow area, ft^2
$h_o =$ Orifice head, ft
$f =$ Darcy Friction Factor
$L =$ Pipe length, ft
$D =$ Pipe inner diameter, ft
$K_L =$ Composite minor loss coefficient
$h_{SP} =$ Energy head at the settling pond overflow, ft
$h_D =$ Hydraulic head at the Denil, ft

McMILLEN JACOBS ASSOCIATES

Assumptions

The above formulation for settling is standard calculation for wastewater settling basins, and is based on a plug flow assumption through the basin.

Inputs

General Parameters	Value		Comments
Gravitational Constant	32.2	ft/s^2	
Settling Velocity	0.00151	ft/s	Idaho DEQ, nd; minimum
Kinematic Viscosity of Water	1.41E-05	ft^2/s	@ 50 F

Hydraulic Parameters	Value		Comments
Design Discharge, Q	200	gpm	
Weir Discharge Coefficient	3.33		Typical
Orifice Discharge Coefficient	0.72		For discharge from a long pipe (Lindeburg, 2014)

Settling Pond Parameters	Value		Comments
Pond Width	12.5	ft	Client supplied CAD linework
Pond Bay Length	29.4	ft	2 bays, less wet well width of 4 feet, less wall widths
Pond Bottom Elevation	2486.5	ft	X-Section Survey
Pond Depth	3.5	ft	Idaho DEQ, nd; recommended for monthly cleanout
Weir Length	5.0	ft	

Pipe Parameters	Value		Comments
Pipe Length	93.0	ft	Measured
Pipe Inner Diameter	0.63	ft	8" Nominal Sch 80 PVC Pipe
Pipe Flow Area (Full)	0.31	ft^2	Calculated
Pipe Roughness Height	0.00006	in	Typical for PVC pipe, 0.0015 mm
Composite minor loss coefficient	2.3		1 x entrance (0.5), 1 x exit (1.0), 5 x 45deg bend (0.5), 1 x 22.5deg bend (0.06), 1 x line flow tee (0.2)
Hydraulic Grade Line at Denil	2587.1	ft	Invert + 2.05' (see Denil fishway calculations)

Calculations

Settling Velocity

Discharge, Q cfs	Settling Pond Area, A_s ft^2	Settling Velocity, V_s ft/s	Overflow Velocity, V_o ft/s	Ratio V_s/V_o
0.45	367.708333	0.00151	0.00121	1.25

Overflow Weir

Discharge, Q cfs	Weir Length, B ft	Discharge Coefficient, C_D	Weir Head, h ft	Weir Crest Elevation ft
0.45	5.00	3.33	0.09	2489.91

Head at Discharge Pipe

Discharge, Q cfs	Friction Factor, f	Orifice Head, h_o ft	Friction Head Loss, h_f ft	Minor Head Loss, h_m ft	Denil Fishway HGL, h_D ft	Head at Settling Pond Discharge, h_{SP} ft
0.45	0.0198	0.06	0.09	0.07	2587.12	2587.35

Conclusions

It was found that the pond in the existing lower battery of raceways provides sufficient area per Idaho DEQ standards for aquaculture solid waste management when divided into 2 bays. The two bays will allow for drying of one of the bays, while keeping the waste drain system online. Additionally, overflow weir calculations were performed and the head required to discharge the 200 gpm to the Denil were calculated.

Appendix B
Civil Design Calculations

Calculation Cover Sheet

Project: Fall Creek Hatchery

Client: Klamath River Renewal Corporation **Proj. No.** 20-024

Title: Civil Calculations - 100% Design

Prepared By, Name: Andrew Leman

Prepared By, Signature: **Date:** 10/28/2020

Peer Reviewed By, Name: Vincent Autier, P.E.

Peer Reviewed, Signature: **Date:** 10/28/2020

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Civil Calculations - 100% Design	**PROJECT NO.:** 20-024	

Table of Content

Civil **Page**

Vehicle Tracking 3

- *Identify design vehicles and determine swept path through the facility.*

Earthworks 9

- *Document the earthworks associated with the current pad layout.*

Pipe Crushing 12

- *Determine whether sufficient cover is maintained on the buried pipelines for HS20 traffic loads.*

Thrust Blocks 16

- *Calculates the thrust at various fittings throughout the site.*

Stormwater Management Plan 17

- *Documents the stormwater management plan for the site.*

Riprap Sizing 29

- *Size the riprap to mitigate scour potential on-site.*

SUBJECT:	Klamath River Renewal Corporation	BY:	A. Leman	CHK'D BY:	V. Autier
	Fall Creek Hatchery	DATE:	10/28/2020		
	Vehicle Tracking	PROJECT NO.:	20-024		

Purpose

The purpose of this calculation sheet is to document the design vehicles for the site and determine the swept path for facility layout.

References

- *Transoft Solutions. 2020. Autoturn Online [software]. Online at https://www.autoturnonline.com, Accessed February 2020.*

- *U.S. Fish and Wildlife Service (USFWS). 2013. Great Lakes Mass Marking Program. Published online at https://www.fws.gov/midwest/greenbayfisheries/documents/Mass-Marking2013.pdf, Accessed February 2020.*

Method

The swept path analysis was performed using AutoTurn online software and the site layout. The site layout was developed iteratively with the swept path analysis. Where possible (or not otherwise constrained) the site sought to maintain a 2.0 ft (min.) buffer on the swept path to any structures, ponds, buildings, etc.

Inputs

Design Vehicles

Marking/Tagging Trailer

The marking and tagging trailer was the largest of the design vehicles for the site, and needed access and egress from both the Coho rearing ponds and the Chinook rearing ponds. The design vehicle used for the swept path analysis was a 43.0-ft long Newmar X-Aire 2009, on a 21.85-ft long design truck. This selection was based on typical marking trailers used by the U.S. Fish and Wildlife Service (see Figures 1 and 2).

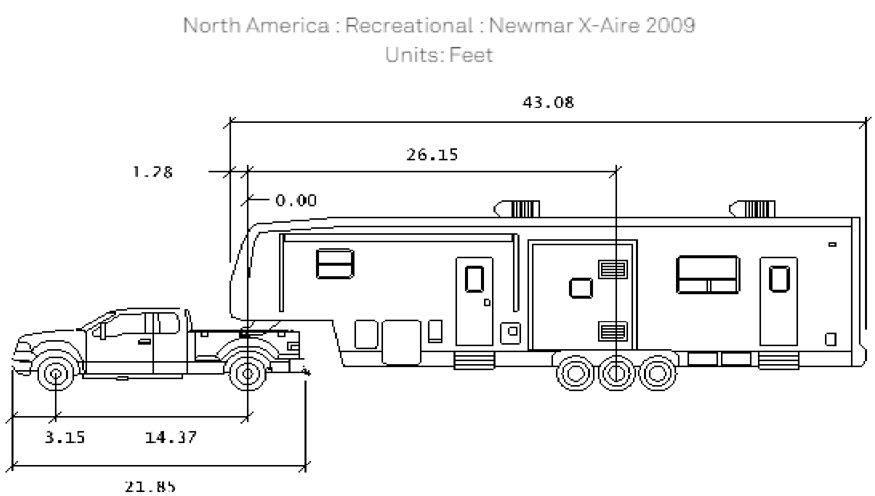


Figure 1. Design Marking/Tagging Trailer (Transoft Solutions, 2020)

Figure 2. U.S. Fish and Wildlife Tagging and Marking Trailer (USFWS, 2013)

Standard Pickup Truck

A standard pickup truck was treated as the design vehicle for typical use at the site, and therefore would be required to access every portic

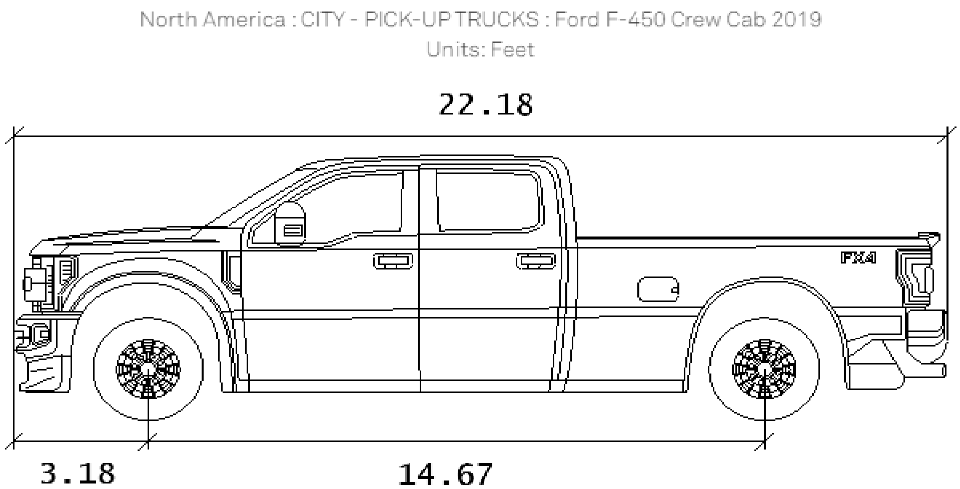


Figure 3. Ford F-450 Dimensions (Transoft Solutions, 2020)

Pump Truck

A pump truck will be required to access the settling pond, vault toilet, and hydrodynamic separators for removal of accumulated waste. No pump truck was available in the AutoTurn online vehicle library, so a truck of comparable size, number of axles, configuration, etc. was used.

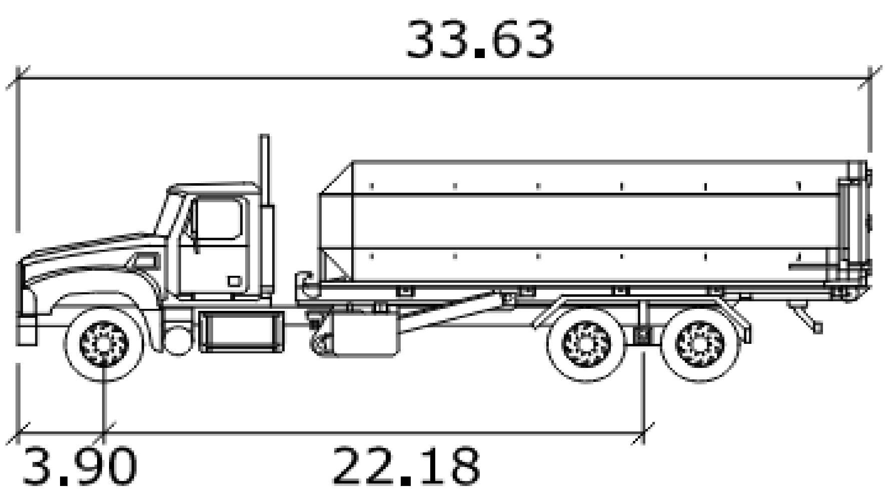


Figure 4. Pump Truck (Similar) Dimensions (Transoft Solutions, 2020)

Site Layout

The site layout that was utilized represents the site layout as defined in the current design phase.

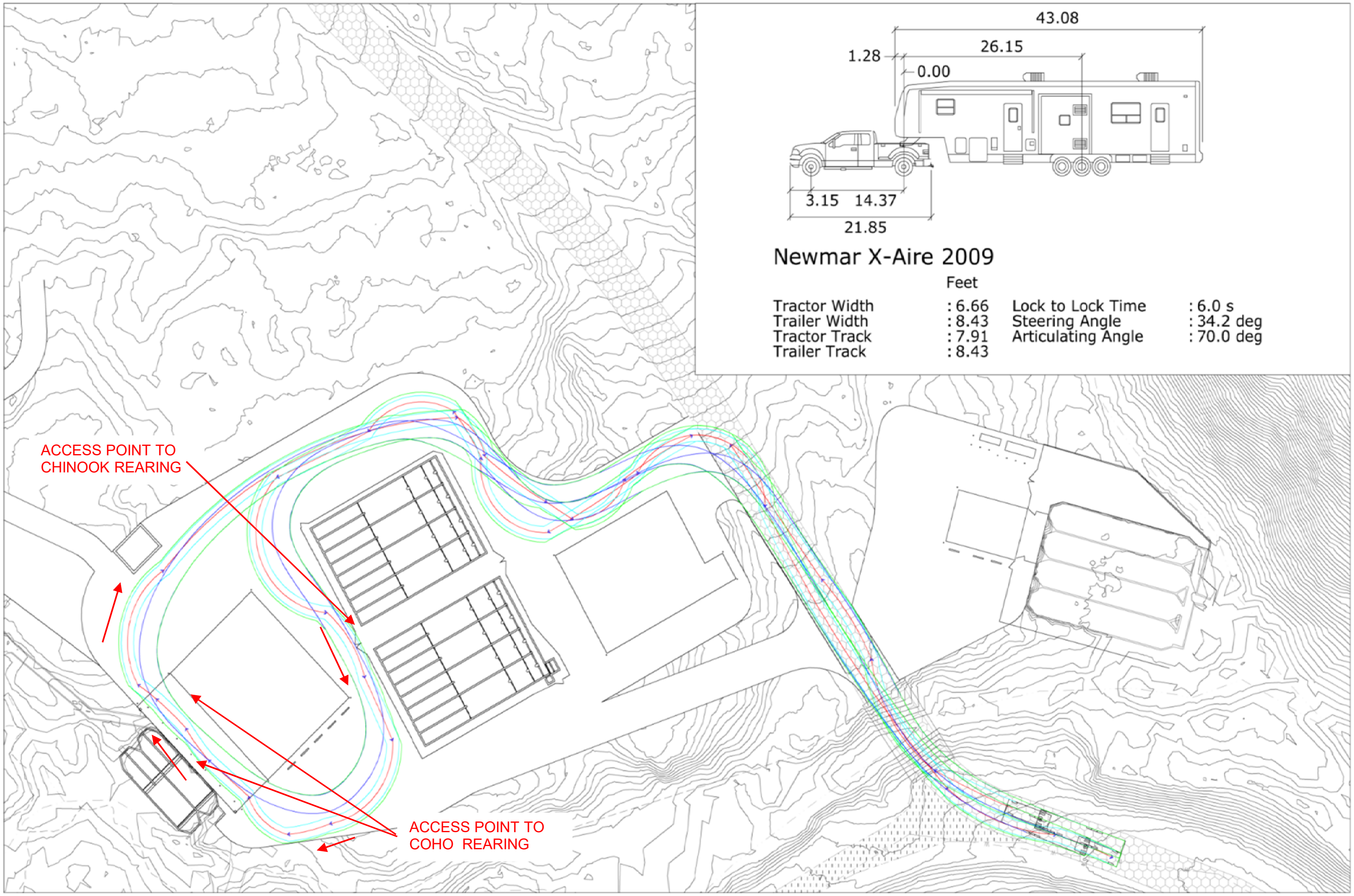


Figure 5. Marking/Tagging Trailer Swept Path

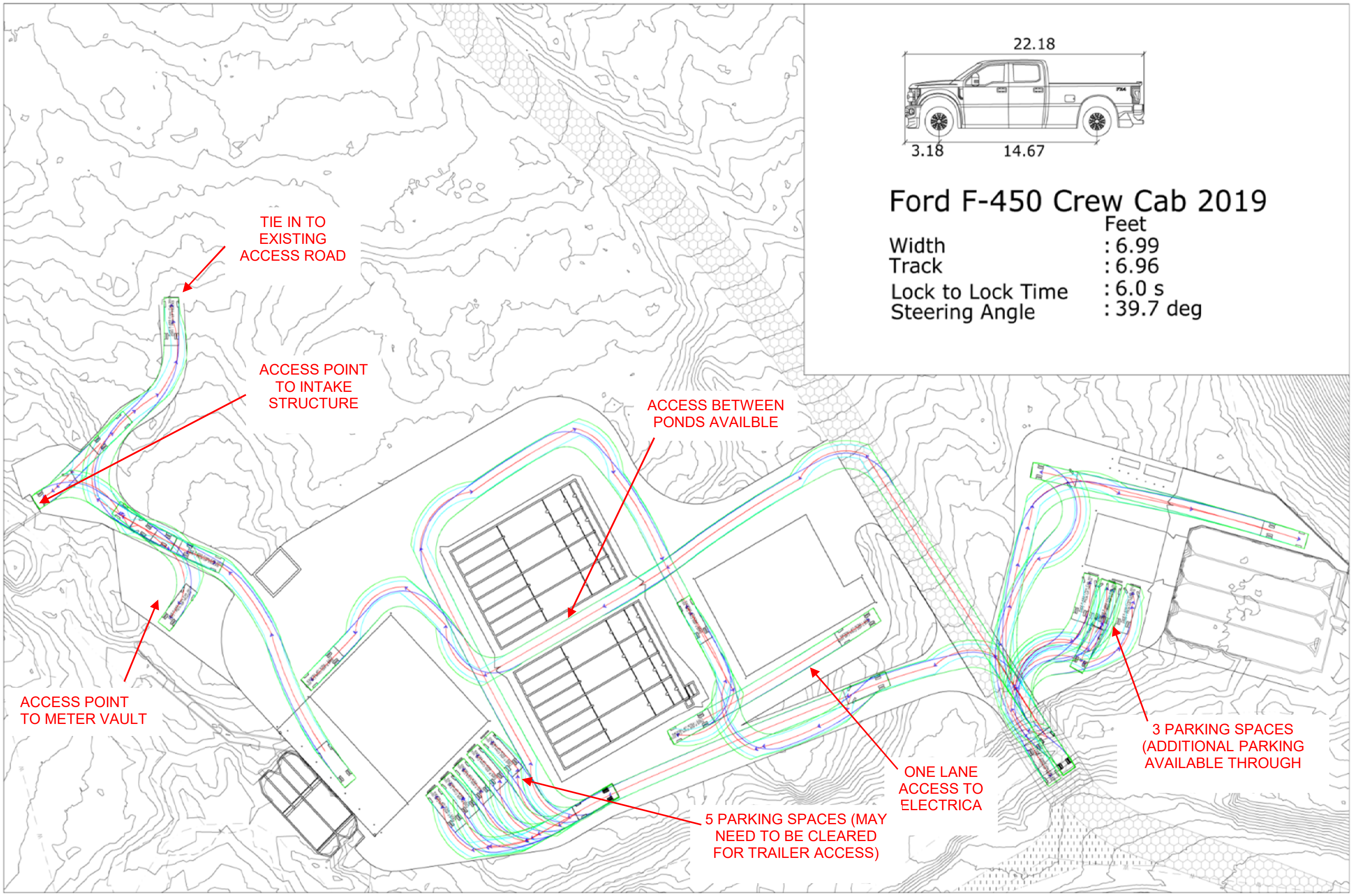


Figure 6. Design Pickup Truck Swept Paths

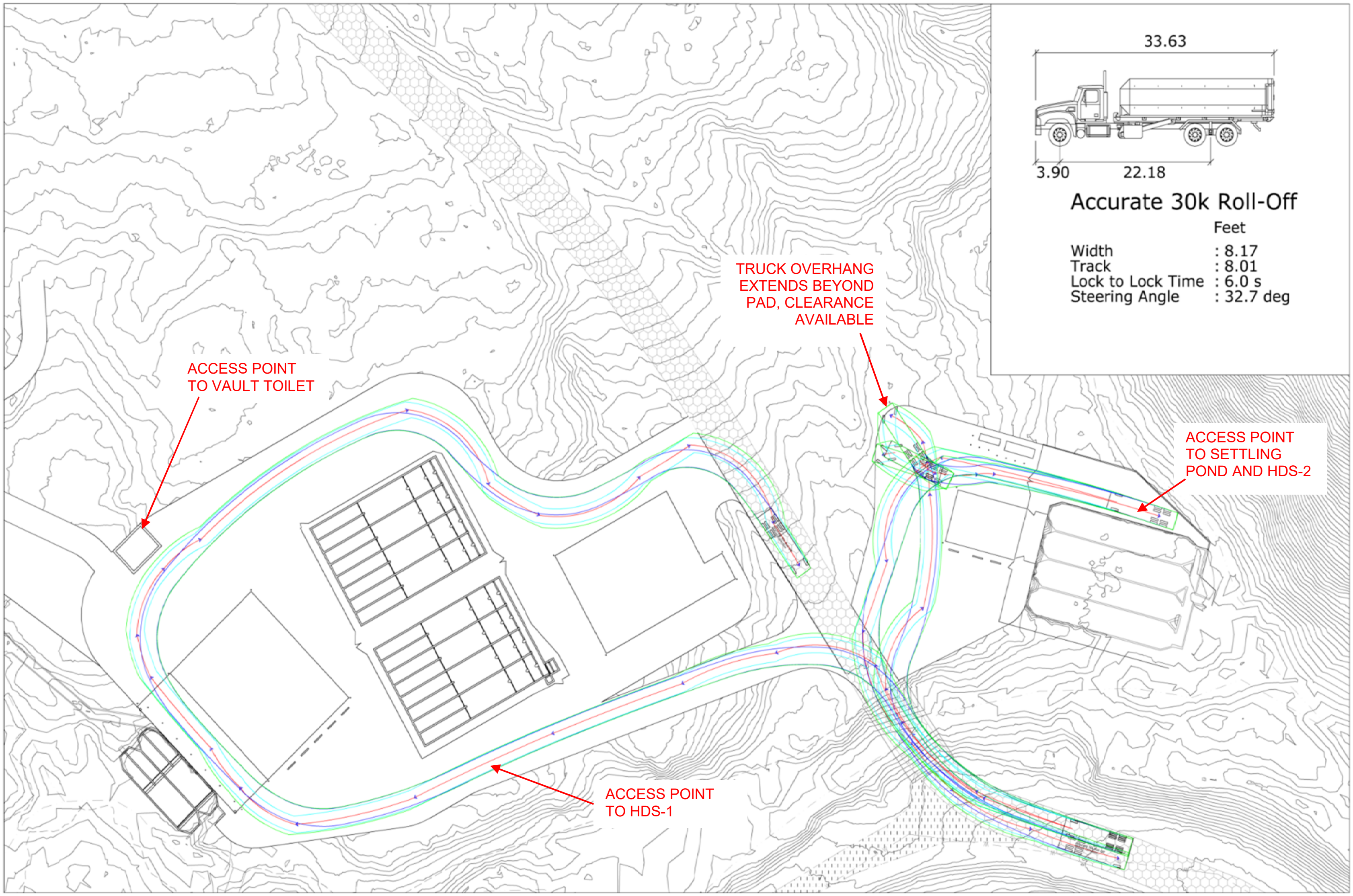


Figure 7. Design Pump Truck Swept Path

Conclusion

A swept path analysis has been run to ensure site access and egress is maintained on this relatively constrained site. Three (3) design vehicles were used for the swept path analysis: (1) a tagging and marking trailer that will need access and egress to the Coho and Chinook rearing ponds, (2) a design pickup truck that will need access to the majority of the site, (3) and a pump truck (similar) that will need to access the settling ponds, vault toilet, and hydrodynamic separators. It was found that the site layout maintained sufficient space that all of the design vehicle requirements could be met, however, in some cases with relatively small margin. This is due to the constrained nature of the site, and was primarily a problem for the less frequently used tagging and marking trailer. Therefore, the current layout is deemed sufficient given the short design life of the facility.

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	Earthworks	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to document the earthworks for the current pad layout.

References

- *Autodesk. 2018. AutoCAD Civil 3D 2018 [software]. Autodesk, Inc. San Rafael, CA.*

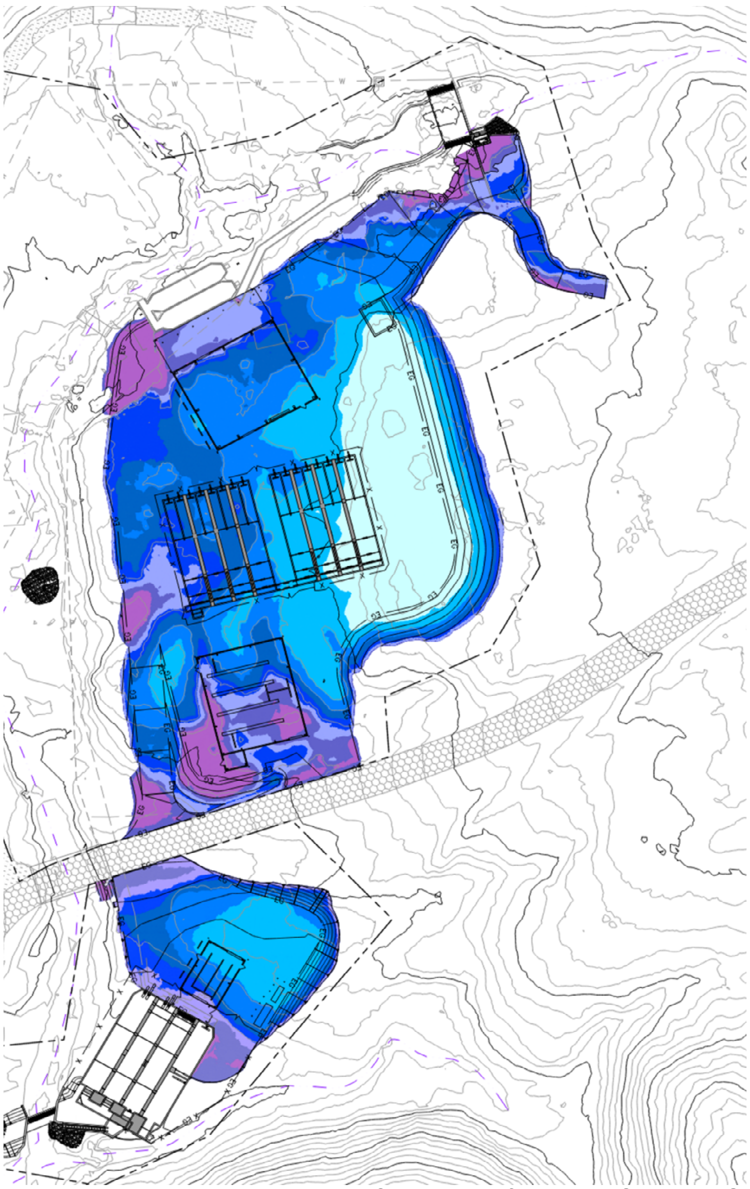
- *CDM Smith. 2019. Klamath River Renewal Project Geotechnical Data Report. Prepared for Klamath River Renewal Corp.*

Information - Input

Pad grading for earthwork volumes was based on the layout of the facility as represented in the current design phase. Pad grading was compared against a composite existing ground triangular irregular network (TIN) consisting of the following in order of precedence (greatest precedence to least):

- *Site structure and ground shot survey*
- *River transect survey*
- *LiDAR and Sonar prepared by GMA Hydrology, Inc. (2018)*

Figure 1 presents a map of the cut and fill locations. The pad grading is almost exclusively in cut.



MIN		MAX	
-13.667	to	-7.87	ft
-7.870	to	-4.721	ft
-4.721	to	-2.769	ft
-2.769	to	-1.531	ft
-1.531	to	-0.667	ft
-0.667	to	-0.009	ft
-0.009	to	0.615	ft
0.615	to	4.198	ft

Figure 1. Cut-Fill Map of North and South Pad Grading

Geotechnical data available for the preliminary analysis consists of two borings located near the Copco Road bridge (CDM Smith, 2019):

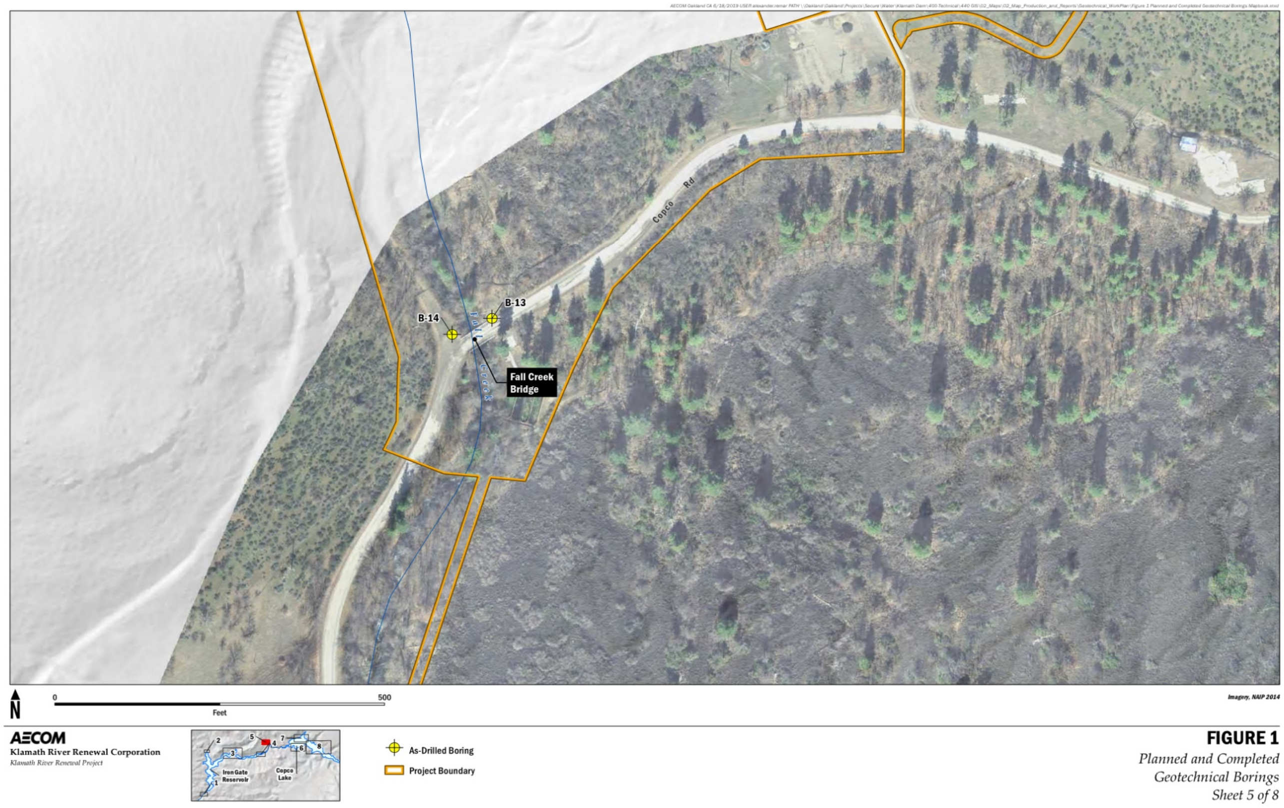


Figure 2. Boring Locations (Source; CDM Smith, 2019)

Boring data was derived from the same source:

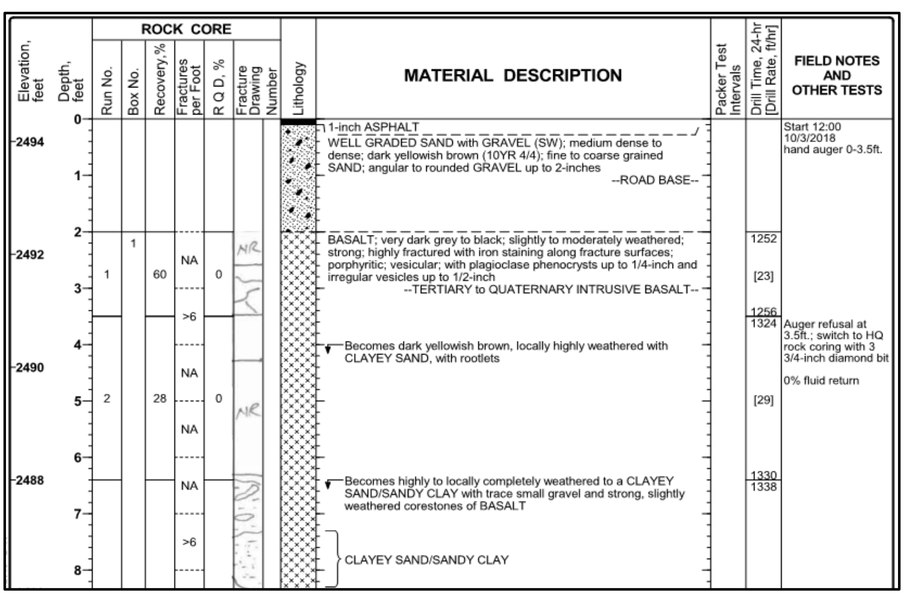


Figure 3. Boring B-13 Log (Source; CDM Smith, 2019)

The boring reached hand auger refusal at approximate elevation 2491 ft (NAVD 88). Both pads were kept above this elevation, however further geotechnical information may be required to determine whether there will be significant rock excavation associated with the current arrangement.

Calculation

Cut and fill volumes were determined using AutoCAD Civil 3D 2018 (Autodesk, 2018). All volumes are reported **in bank condition**. The following table summarizes the cut and fill volumes associated with the preliminary design.

Location	Description	Cut (yd^3)	Fill (yd^3)	Net (yd^3)
North Pad	Pad Grading North of Copco Road	9,370	345	9,025
South Pad	Pad Grading South of Copco Road	1,257	16	1,241

Conclusion

Cut and fill quantities were determined for the pad grading at the Fall Creek Fish Hatchery. Quantities were determined from AutoCAD Civil 3D 2018 and were based on a composite existing ground surface consisting of ground survey, LiDAR, and Sonar. It was found that a total net excavation of approximately 10,000 cubic yards (bank) is required for the current pad configuration. Limited geotechnical boring information suggests that bedrock is below the pads, however the bedrock elevation could fluctuate significantly across the site, and further geotechnical information would support decisions and cost estimating related to rock excavation.

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Pipe Crushing

BY: A. Leman **CHK'D BY:** V. Autier
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to determine whether sufficient cover is maintained on the buried pipelines for HS20 traffic loads.

References

- PPI (Plastics Pipe Institute), 2019. *Handbook of PE Pipe, 2nd Edition* . Published online at https://plasticpipe.org/publications/pe-handbook.html. Accessed Sept. 2019.

- American Lifelines Alliance. 2001. *Guidelines for the Design of Buried Steel Pipe.* American Society of Civil Engineers (ASCE) and Federal Emergency Management Agency (FEMA).

- Spangler, M.G. 1941. The Structural Design of Flexible Pipe Culverts, Bulletin 153, Iowa Engineering Experiment Station, Ames, IA.

Information - Input

The following parameters were used in the development of the pipe crushing calculations.

General Parameters	Value	Units	Comments
Backfill Dry Unit Weight	140	lb/ft^3	Conservative
Unit Weight of Water	62.4	lb/ft^3	Standard, T = 50 F
Bedding Factor, K_{bed}	0.1		Typical
Deflection Lag Factor, L_{DL}	1.25		Typically, 1.0-1.5 (Spangler, 1941)
Modulus of Soil Rxn, E'	1000	psi	Assume Type SC @ 90% Compaction, see Tables below
Trench Width Ratio, B/D_o	2		Maintain one radius either side of pipe
Native Modulus of Soil Reaction, E'_N	700	psi	Assume soft cohesive, conservative
Soil Support Factor, F_s	0.85		See Tables below

PVC Pipe Parameters	Value	Units	Comments
PVC Modulus of Elasticity, E	280,000	psi	@ 73 F, reduced ~20% for long term
Pipe Nominal Diameter	24	in	Maximum pipe size used at site, limiting case
Pipe Pressure Rating	Sched 80		

HDPE Pipe Parameters	Value	Units	Comments
HDPE Modulus of Elasticity, E	28,000	psi	@73 F, for 50-year sustained load, PE3608
Pipe Nominal Diameter	14	in	Case of interest, largest pipe size under road
Pipe Pressure Rating	Determined in analysis below		

Method

Calculations were performed according to the Handbook of PE Pipe, 2nd Edition, using data associated with PVC pipe. The Handbook of PE Pipe method follows Spangler's modified Iowa equation for pipe deflection, which is typical for PVC pipe as well as HDPE pipe.

Live Load
HS20 Soil Pressure Table (Table 3-4)

The live load was determined from Table 3-4 of the Plastic Pipe Institute (2019) Handbook of PE pipe. This is applicable to PVC pipe as well as PE pipe, and represents an unpaved or flexible pavement condition. The tabulated values do not include an impact factor, which will be applied in subsequent calculations based on the cover condition.

Unpaved or Flexible Pavement		
Depth of Cover	Soil Pressure	
ft	psf	psi
1.5	2000	13.9
2	1340	9.3
2.5	1000	6.9
3	710	4.9
3.5	660	4.6
4	600	4.2
6	310	2.2
8	200	1.4
10	140	1.0

Dead Load
Soil Prism (Eq 3-1)

Dead load was calculated according to a modification on the standard soil prism equation, to account for the water table above the pipe crown (American Lifelines Alliance, 2001). This is summarized below:

$$\sigma_{DL} = \gamma_d H\left(1 - \frac{1}{3}\frac{h_W}{H}\right) + \gamma_W H$$

where:
$\sigma_{DL} =$ Dead load pressure, psf
$\gamma_d =$ Dry weight of soil, lb/ft^3
$\gamma_W =$ Unit weight of water, lb/ft^3
$H =$ Cover over pipe crown, ft
$h_w =$ Height of water table above crown, ft

Pipe Deflection / Ovality
Modified Iowa Equation (Eq 3-10)

The pipe deflection/ovality was calculated according to the modified Iowa equation (PPI, 2019), following the work of Spangler (1941).

$$\frac{\Delta y}{D_M} = \frac{K_{bed}L_{DL}\sigma_{DL} + K_{bed}\sigma_{LL}}{\frac{2E}{3}\left(\frac{1}{D_o/_t - 1}\right)^3 + 0.061F_sE'}$$

where:
$\Delta y =$ Vertical deflection
$D_M =$ Mean pipe diameter
$D_o =$ Outside pipe diameter
$K_{bed} =$ Bedding factor
$L_{DL} =$ Lag deflection factor
$E =$ Pipe modulus of elasticity, psi
$t =$ Pipe wall thickness, in
$F_s =$ Soil Support Factor
$E' =$ Modulus of Soil Reaction, psi
<other values as previously defined>

Tables for selecting soil values are summarized below:

Modulus of Soil Reaction					
Type of Soil	Depth of Cover ft	Modulus of Soil Reaction, E' <85%	90%	95%	100%
Fine-grained soils with < 25% sand content (CL, ML, CL-ML)	0	500	700	1000	1500
	5	600	1000	1400	2000
	10	700	1200	1600	2300
	15	800	1300	1800	2600
Coarse-grained soils with fines (SM, SC)	0	600	1000	1200	1900
	5	900	1400	1800	2700
	10	1000	1500	2100	3200
	15	1100	1600	2400	3700
Coarse-grained soils with little or no fines (SP, SW, GP, GW)	0	700	1000	1600	2500
	5	1000	1500	2200	3300
	10	1050	1600	2400	3600
	15	1100	1700	2500	3800

Native Soil Modulus of Soil Reaction				
Granular		Cohesive		
Std. Penetration ASTM D1586	Description	Unconf. Compress. Strength (tsf)	Description	E'_N (psi)
>0 -1	v. v. loose	>0 - 0.125	v. v. soft	50
1-2	very loose	0.125 - 0.25	very soft	200
2-4	very loose	0.25 - 0.50	soft	700
4-8	loose	0.50 - 1.00	medium	1,500
8-15	slight.comp.	1.00 - 2.00	stiff	3,000
15-30	compact	2.00 - 4.00	very stiff	5,000
30-50	dense	4.00 - 6.00	hard	10,000
> 50	very dense	> 6.00	very hard	20,000
Rock	-	-	-	50,000

E'$_N$/E'	Soil Support Factor Ratio of Trench Width to Pipe Outer Diameter, 1.5	2.0	2.5	3.0	4.0	5.0
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
5.0	2.00	1.60	1.40	1.25	1.10	1.00

Safe Deflection Limits - Pressure Pipe DR	%
7.3	3.00%
9.0	4.00%
13.5	6.00%
17.0	6.00%
21.0	7.50%
26.0	7.50%
32.5	7.50%

Pipe Wall Buckling
Luscher Equation (Eq 3-15)

The pipe wall buckling contraint is calculated according to Luscher's equation for constrained pipe wall buckling:

$$\sigma_{b,allow} = \left(\frac{1}{SF}\right)\sqrt{\frac{32RB'E'E}{12\left(D_0/_t - 1\right)^3}}$$

$$B' = \frac{1}{1 + 4e^{-0.065H}}$$

$$R = \left(1 - \frac{1}{3}\frac{h_W}{H}\right)$$

where:
$\sigma_{b,allow} =$ Allowable constrained buckling pressure, psi
$SF =$ Safety Factor, >2 recommended
$R =$ Buoyancy Reduction Factor
$B' =$ Soil Support Factor
<other values as previously defined>

Calculations

The following calculations demonstrate that at 2.0' of cover above the crown of the pipe, the pipes are adequately protected against ovality and pipe wall buckling for HS20 traffic loads.

PIPE Pipe Material	Pressure Rating	Nominal Pipe Diameter (in)	Wall Thickness (in)	Pipe Outer Diameter (in)	Pipe Mean Diameter (in)	Pipe Inner Diameter (in)	Pipe Moment of Inertia (in^4/in)	Pipe Modulus of Elast., E (psi)
PVC	Sched 80	24	1.218	24	22.782	21.564	0.1506	280,000
HDPE	DR26.	14	0.538	14	13.462	12.924	0.0130	28,000

LOADS Pipe Material	Pressure Rating	Burial Depth (to Crown), H (ft)	Backfill Dry Unit Weight, γ_d (lb/ft^3)	Height of Water Table above Pipe Crown, h_w (ft)	Live Load Type	Impact Factor, F'	Dead Load Pressure, σ_{DL} (psi)	Live Load Pressure, σ_{LL} (psi)	Total Pressure, σ_T (psi)
PVC	Sched 80	2.0	140	0	HS20	1.35	1.94	12.56	14.51
HDPE	DR26.	2.0	140	0	HS20	1.35	1.94	12.56	14.51

Deflection Pipe Material	Pressure Rating	Bedding Factor, K_{bed}	Deflection Lag Factor, L_{DL}	Modulus of Soil Rxn, E' (psi)	Soil Support Factor, F_s	% Deflection, $\Delta y/D_m$	Acceptable Deflection %	Deflection OK?
PVC	Sched 80	0.1	1.25	1000	0.85	1.87%	7.01%	OK!
HDPE	DR26.	0.1	1.25	1000	0.85	2.83%	7.50%	OK!

Pipe Wall Buckling							
Pipe Material	Pressure Rating	Soil Support Factor, B'	Buoyancy Reduction Factor, R	Allowable Buckling Press, σ_b (FS = 2) psi	Actual Pressure	Calculated FS	Buckling OK?
PVC	Sched 80	0.22	1.00	79.5	14.51	11.0	OK!
HDPE	DR26.	0.22	1.00	16.2	14.51	2.2	OK!

Conclusion

Calculations demonstrate that a 24" nominal diameter Schedule 80 PVC pipe with 2.0' of cover above the crown of the pipe is well within the limits for acceptable ovality and pipe wall buckling. Similar preliminary calculations show that acceptable factors of safety are available for ring thrust and through-wall bending as well. Therefore, a minimum cover of 2.0' will be applied to all pipes across the site, as this is the limiting case. Where pipes are buried less than 1 diameter below finished grade in traffic rated areas, controlled low-strength material, or some alternative engineered solution will be used to protect the pipes against crushing.

SUBJECT: Klamath River Renewal Corporation
Fall Creek Hatchery
Thrust Blocks

BY: A. Leman CHK'D BY: V. Autier
DATE: 10/28/2020
PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to calculate the thrust at various fittings throughout the site for sizing of thrust blocks.

References

- *Lindeburg, Michael. 2014. Civil Engineering Reference Manual for the PE Exam, Fourteenth Edition. Professional Publications, Inc. Belmont, CA.*

Method

Thrust loads at pipe bends can be calculated based on the superposition of the resultant pressure vector and the resultant dynamic force vector. Because they are acting ver the same angle, the magnitudes may be added. The following equations give the resultant forces:

$$R_p = \sqrt{R_{p,x}^2 - R_{p,y}^2} = \sqrt{\left[p\pi\left(\frac{d}{2}\right)^2(1-\cos\beta)\right]^2 + \left[p\pi\left(\frac{d}{2}\right)^2(\sin\beta)\right]^2} = \frac{1}{2}p\pi d^2 \sin\frac{\beta}{2}$$

$$R_v = \sqrt{R_{v,x}^2 - R_{v,y}^2} = \sqrt{\left[\rho\pi\left(\frac{d}{2}\right)^2 v^2(1-\cos\beta)\right]^2 + \left[\rho\pi\left(\frac{d}{2}\right)^2 v^2(\sin\beta)\right]^2} = \frac{1}{2}\rho\pi d^2 v^2 \sin\frac{\beta}{2}$$

$$R_{tot} = R_p + R_v$$

where:
$R_p =$ Pressure resultant force, lbs
$R_v =$ Dynamic resultant force, lbs
$p =$ Pressure, psi
$\rho =$ Density of water, lb/ft^3 (x ft^2/144in^2)
$d =$ Pipe inner diameter, in
$v =$ Average flow velocity, ft/s
$\beta =$ Deflection angle, degrees
$R_{tot} =$ Resultant thrust, lbs

This can be further simplified, by the fact that average velocities are typically low enough that they do not produce a significant component of thrust. Moreover, the critical cases for pressure (static pressure, hydrostatic pressure testing) are cases where velocities are not a factor. Therefore, thrust will be calculated strictly based on the pressure in the pipeline. This will be calculated on a per psig of pressure basis, such that the Contractor can field calculate the required bearing area for any location on any pipeline.

Calculation

Pressure Pipeline

Nominal Pipe Size (in)	Pipe Inner Diameter, d (in)	Pressure*, p (psig)	Safety Factor, SF	Fittings					
				11.25° Bend	22.5° Bend	30° Bend	45° Bend	90° Bend	Dead End / Tee
4	3.786	1	1.5	3	7	9	13	24	24
6	5.709	1	1.5	8	15	20	29	54	54
8	7.565	1	1.5	13	26	35	52	95	95
10	9.493	1	1.5	21	41	55	81	150	150
12	11.294	1	1.5	29	59	78	115	213	213
14	12.41	1	1.5	36	71	94	139	257	257
16	14.213	1	1.5	47	93	123	182	337	337
18	16.014	1	1.5	59	118	156	231	427	427
20	17.814	1	1.5	73	146	194	286	529	529
24	21.418	1	1.5	106	211	280	414	764	764

** Thrust calculated on a per psig bases, and can be multiplied by the max pressure (hydrostatic testing, static pressure, surge, etc.) to arrive at the total thrust*

Conclusion

Calculations of the thrust resultant vector were calculated for the pressure in the pressure pipelines. Calculations were performed on a per psig basis, such that values were dependent on the pipe size and fitting type only, and could be multiplied by the maximum design pressure to arrive at a total thrust load to be resisted. These values were used to populate detail C605.

SUBJECT:	Klamath River Renewal Corporation	**BY:** A. Leman	**CHK'D BY:** V. Autier
	Fall Creek Hatchery	**DATE:** 10/28/2020	
	SW Management Plan	**PROJECT NO.:** 20-024	

Purpose

The purpose of this calculation sheet is to document the stormwater management plan for the Fall Creek Hatchery.

References

- *California Stormwater Quality Association (CASQA). 2003. California Stormwater Best Management Paractices (BMP) Handbook: New Development and Redevelopment. Accessed online at https://www.casqa.org/resources/bmp-handbooks/new-development-redevelopment-bmp-handbook, June 2020.*

- *California North Coast Regional Water Quality Control Board (Water Board). 2020. NPDES Stormwater [website]. Accessed at https://www.waterboards.ca.gov/northcoast/water_issues/programs/npdes_stormwater/, July 2020.*

- *California State Water Resources Control Board (Water Board). 2011. Runoff Coefficient Fact Sheet 5.1.3. Accessed online at https://www.waterboards.ca.gov/water_issues/programs/swamp/docs/cwt/guidance/513.pdf, July 2020.*

- *Lindeburg, Michael R. 2014. Civil Engineering Reference Manual, Fourteenth Edition. Professional Publications, Inc. Belmont, CA.*

- *Natural Resources Conservation Service (NRCS). 1986. Urban Hydrology for Small Watersheds: Technical Release 55 (TR-55). U.S. Department of Agriculture, NRCS, Conservation Engineering Division, June 1986.*

- *Natural Resources Conservation Service (NRCS). 2020. Web Soil Survey [online software]. Accessed at https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx, June 2020.*

- *National Oceanic and Atmospheric Administration (NOAA). 2014. Precipitation-Frequency Atlas of the United States, Volume 5, Version 2.3: California. NOAA, National Weather Service: Silver Spring, MD.*

Strategy

The strategy developed for the stormwater system at FCFH is based on Sections 2.2 and 2.3 of the CASQA BMP Handbook for new development and redevelopment (2003), and follows the following steps:

1. Determine Permit Requirements - The site will require an NPDES stormwater permit. Both Siskiyou County and the City of Yreka are listed in the Phase II Program for small MS4s (municipal separate storm sewer systems) through the California Water Board (2020). They note that "there is one state wide general permit which regulates the discharge of pollutants from small MS4s, State Water Board Order No. 2013-0001 DWQ." It is assumed in these calculations that the flow/volume requirements implemented in the NPDES stormwater permit for FCFH will be similar to those in the above State Water Board Order. **These calculations are not a substitution for determining permitting requirements once the NPDES stormwater permit has been obtained.**

2. Assess Site Conditions - General site understanding was collected from the project topography (see Earthworks calculations), from the NRCS web soil survey, and from site photographs. The drainage patterns for the post-construction site are delineated on Figure 3 below. The site hydrologic soil group was Group D for the entirety of the site (NRCS, 2020), making the site a poor candidate for infiltration based methods of treatment. Rational method runoff coefficients are summarized in the table below:

Ground Cover	Runoff Coefficient, C		
	Min	Max	Design
Forest	0.05	0.25	0.20
Unimproved Areas	0.10	0.30	0.25
Asphalt Street	0.70	0.95	0.85
Gravel Parking	0.75	0.85	0.80
Roofs	0.75	0.95	0.95

** Runoff coefficients taken from Water Board (2011) recommendations, and design values selected based on site-specific slopes, ground cover conditions, and hydrologic soil group.*

3. Understand Hydrologic Conditions - The hydrologic conditions at the site were collected from NOAA Atlas 14, Volume 6, and hourly rainfall data collected from the nearby Montague rainfall station (Station ID COOP:045785) with a 65 year period of record (see Figure 1). Rainfall events and data are summarized in the table below. Impervious areas, and times of concentration will be considered in the calculations below.

Rainfall Event	Intensity in/hr	Source	Notes
Minimum 0.2 in/hr	0.2	CASQA Recommend.	Water Quality Storm (WQS), CASQA dictated
2x 85th Percentile Hourly	0.2	Montague Rain Data	Water Quality Storm (WQS), CASQA dictated
1-year, 1-hour	0.317	NOAA Atlas 14	Extreme Event
2-year, 1-hour	0.435	NOAA Atlas 14	Extreme Event
2-yr, 24-hour	1.94	NOAA Atlas 14	Used in Runoff Calculations (per NRCS, 1986)
10-year, 1-hour	0.738	NOAA Atlas 14	Extreme Event
100-year, 1-hour	1.27	NOAA Atlas 14	Extreme Event

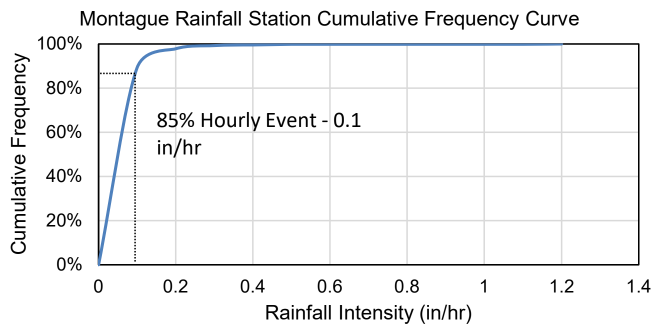


Figure 1. Montague Data Station Rainfall Intensity Cumulative Frequency

4. Evaluate Pollutants of Concern - The site will be minimally used/trafficked and therefore the primary pollutant concern will be sediments, however it is expected that pollutants associated with vehicle use will also be present (see below). Pollutants associated with the hatchery processes will be part of the facility waste stream and will be handled in the facility waste treatment.

Table 2-1 Anticipated and Potential Pollutants Generated by Land Use Type

Priority Project Categories	General Pollutant Categories								
	Pathogens	Heavy Metals	Nutrients	Pesticides	Organic Compounds	Sediments	Trash & Debris	Oxygen Demanding Substances	Oil & Grease
Detached Residential Development	X		X	X		X	X	X	X
Attached Residential Development	P		X	X		X	X	P(1)	P(2)
Commercial/ Industrial Development >100,000 ft2	P(3)		P(1)	P(5)	P(2)	P(1)	X	P(5)	X
Automotive Repair Shops		X			X(4)(5)		X		X
Restaurants	X						X	X	X
Hillside Development >5,000 ft2 In SDRWQCB			X	X		X	X	X	X
Hillside Development >100,000 ft2 In SARWQCB			X	X		X	X	X	X
Parking Lots		X	~~P(1)~~	~~P(2)~~		P(1)	X	~~P5~~	X
Streets, Highways & Freeways		X	P(1)		X(4)	X	X	P(5)	X

X = anticipated
P = potential
(1) A potential pollutant if landscaping exists on-site
(2) A potential pollutant if the project includes uncovered parking areas
(3) A potential pollutant if land use involves food or animal waste products.
(4) Including petroleum hydrocarbons.
(5) Including solvents.

Figure 2. Potential Pollutants by Land Use Type (Source; CASQA, 2003; Table 2-1 Modified)

5. Identify Candidate BMPs - As mentioned above, the site (1) consists of high clay content and poor hydrologic condition, and (2) is very close to the stream, which makes infiltration based methods infeasible for this site. Therefore, it was determined that manufactured stormwater BMPs would be used to treat water before releasing back to the stream where runoff would naturally flow. In particular, evaluations will be made here based on an assumed Contech Engineered Systems CDS hydrodynamic separator.

6. Determine BMP Size/Capacity - Calculations below will summarize the sizing of the BMP, as well as the appurtenant storm drain system.

7. Develop Plan for BMP Maintenance - BMP maintenance will be according to the manufacturer's recommendations, and will consist of regular removal of accumulated trash and sediment in the separator. This can be accomplished by a pump truck which will already be coming to the site for the ongoing maintenance of the settling ponds.

Figure

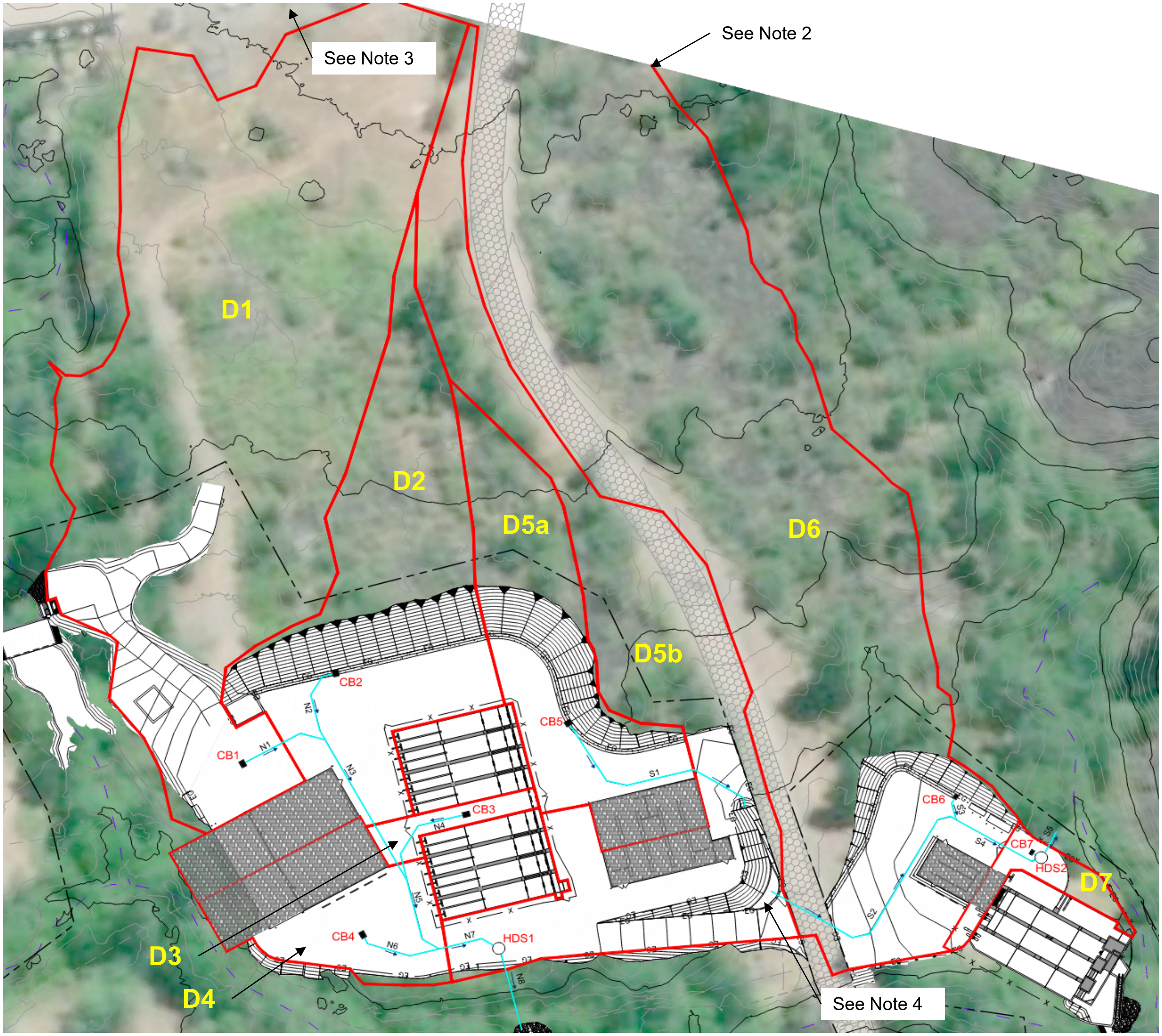


Figure 3. Stormwater Management Overview

Notes:

1. Grading in preliminary configuration. Finish grading on all flat surfaces graded to drain toward catch basins or sumps.

2. Drainage areas extend outside of limits of Figure. Drainage areas estimated according to aerial imagery outside of the topography limits.

3. Powerhouse area drainage assumed to be retained/treated on-site.

4. A drain rock sump will be maintained near the entrance to the north site, adjacent to Copco Rd. The sump will be wrapped in geotextile, and will have a perforated PVC pipe that will drain the sump to the storm sewer system servicing the southern portion of the hatchery.

Key:

D x	Drainage Area
CB x	Catch Basin
HDS x	Hydrodynamic Separator
N x	North Storm Sewer Segment
S x	South Storm Sewer Segment

Inputs

The following inputs were collected for each of the drainage areas from AutoCAD Civil 3D. Site linework and delineations of ground cover from aerial imagery were utilized to determine areas of individual ground cover. The composite rational method runoff coefficient was then calculated as the weighted average of the respective areas:

$$\bar{C} = \frac{1}{A_{tot}} \sum A_i C_i$$

where:

$\bar{C} =$ Composite runoff coefficient
$A_{tot} =$ Total Area, ac
$A_i =$ Area of Ground Cover Unit, ac
$C_i =$ Design runoff coefficient of Ground Cover Unit

Drainage Area I.D.	Areas						Composite Rough Coeff, C
	Forest (ac)	Unimproved (ac)	Asphalt (ac)	Gravel (ac)	Roof (ac)	Total (ac)	
D1	0.205	0.931	0.000	0.347	0.000	1.483	0.37
D2	0.042	0.310	0.000	0.174	0.083	0.610	0.50
D3	0.000	0.000	0.000	0.034	0.000	0.034	0.80
D4	0.000	0.008	0.000	0.107	0.078	0.194	0.84
D5a	0.110	0.086	0.000	0.110	0.041	0.348	0.49
D5b	0.335	0.082	0.022	0.224	0.034	0.698	0.46
D6	1.289	0.036	0.208	0.155	0.025	1.712	0.35
D7	0.000	0.000	0.000	0.042	0.000	0.042	0.80

Hydrologic Calculations

Time of Concentration

Times of concentration for each of the individual sub-basins were calculated according to the NRCS Segmental Method:

Sheet Flow: *(300 ft max)*

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

where:

$T_t =$ Travel Time, hr
$n =$ Manning's roughness coefficient
$L =$ Flow length, ft
$P_2 =$ 2-year, 24-hour rainfall, in (collected from NOAA Atlas 14)
$S =$ Slope, ft/ft

Shallow Concentrated: (unpaved)

$$T_t = \frac{L}{3600\sqrt{S/0.0038}}$$

(paved)

$$T_t = \frac{L}{3600\sqrt{S/0.0024}}$$

Channel Flow:

$$T_t = \frac{L}{3600V}$$

$$V = \frac{1.49 R_h^{2/3} S^{1/2}}{n}$$

where:

$V =$ Average flow velocity, ft/s
$R_h =$ Hydraulic radius, ft

Drainage Area I.D.	Sheet Flow		Shallow Concentr.		Channel Flow	
	Length ft	Slope ft/ft	Length ft	Slope ft/ft	Length ft	Slope ft/ft
D1	300	0.0483	172	0.093	-	-
D2	230	0.0508	30	0.500	73	0.008
D3	30	0.0067	26	0.012	-	-
D4	90	0.0688	-	-	-	-
D5a	130	0.0577	30	0.5	75	0.009
D5b	300	0.042	208	0.107	36	0.042
D6	300	0.043	263	0.109	35	0.048
D7	20	0.005	60	0.008	-	-

Drainage Area I.D.	Sheet Flow			Shallow C.	Channel Flow				Time of Conc†, t_c hr
	2-year, 24-hr Precip, P_2 in	Sheet Flow Rough*, n	Travel Time, T_t hr	Travel Time, T_t hr	Hydraulic Radius‡, R_h ft	Rough Coeff, n	Velocity, V ft/s	Travel Time, T_t hr	
D1	1.94	0.130	0.32	0.01	-	-	-	0.00	0.33
D2	1.94	0.130	0.25	0.00	0.163	0.015	2.65	0.01	0.26
D3	1.94	0.011	0.02	0.00	-	-	-	0.00	0.10
D4	1.94	0.011	0.01	0.00	-	-	-	0.00	0.10
D5a	1.94	0.130	0.15	0.00	0.163	0.015	2.86	0.01	0.16
D5b	1.94	0.130	0.33	0.01	0.485	0.035	5.36	0.00	0.35
D6	1.94	0.130	0.33	0.01	0.163	0.015	6.49	0.00	0.35
D7	1.94	0.011	0.01	0.01	-	-	-	0.00	0.10

** Sheet flow roughness coefficients determined from Table 3-1, TR-55 (NRCS, 1986). 0.130 corresponds to Range (natural) and 0.011 corresponds to smooth surfaces (concrete, asphalt, gravel, or bare soil).*

‡ Channel flow in concrete swales is assumed full, for estimate of travel time. For the 4-inch deep, 3-ft wide concrete swales, the hydraulic radius is 0.163. Elsewhere, a 6-inch depth was assumed. Due to the length of channel flow, this travel time is expected to have minimal impact on the time of concentration.

† TR-55 recommends a minimum time of concentration of 0.1 hours, or 6 minutes.

Peak Discharge

Peak discharge was calculated according to the Modified Rational Method (MRM), because the entire drainage area to the site was only about 5 acres in total, which warrants this type of analysis. The MRM handles junctions of independent basins in the storm sewer system by iterating through the various times of concentration reaching the basin according to the equations below, and selecting the maximum peak discharge and time of concentration condition.

Individual Basin: $Q_p = 1.008\bar{C}iA$

Junctions: $t_{c,1} < t_{c,2}$

$$Q_{tc,1} = Q_{p,1} + \frac{t_{c,1}}{t_{c,2}} Q_{p,2}$$

$$Q_{tc,2} = Q_{p,2} + \frac{i_2}{i_1} Q_{p,1}$$

$$Q_{p,j} = max\{Q_{tc,1}, Q_{tc,2}\}$$

where:

$Q_p =$ Design discharge, cfs
$\bar{C} =$ Weighted-Average Rational Coefficient
$i =$ Rainfall Intensity, in/hr
$A =$ Drainage area, ac
$t_{c,1} =$ Time of conc for basin 1, hr
$t_{c,2} =$ Time of conc for basin 2, hr
$Q_{tc,1} =$ Peak discharge for tc,1, cfs
$Q_{tc,2} =$ Peak discharge for tc,2, cfs
$Q_{p,j} =$ Peak discharge at storm sewer junction, cfs
$Q_{p,1} =$ Peak discharge for basin 1, cfs
$Q_{p,2} =$ Peak discharge for basin 2, cfs
$i_1 =$ Rainfall intensity of basin 1, in/hr
$i_2 =$ Rainfall intensity of basin 2, in/hr

**Note: the below calculations do not account for the pipe flow travel time in the time of concentration. The pipe segments are short enough and will be moving at high enough velocity, that these effects will be negligible. For instance, the longest pipe run from CB2 to HDS1 is approximately 250 ft. At 4 fps, this would add approx. 1 minute to the time of concentration, which for the 10-year event would increase the intensity by 0.03 in/hr, and the flow rate by approximately 0.03 cfs. These negligible effects were neglected. Pipes will be conservatively sized to account for small errors in the analysis.*

Drainage Area I.D.	Composite Rough Coeff, C	Area, ac	Time of Conc, t_c min	Rainfall Intensities				
				WQS	Extreme Storms			
				2x 85th Percentile in/hr	1-yr in/hr	2-yr in/hr	10-yr in/hr	100-yr in/hr
D1	0.37	1.483	19.6	0.2	0.64	0.88	1.49	2.56
D2	0.50	0.610	15.6	0.2	0.70	0.97	1.64	2.81
D3	0.80	0.034	6.0	0.2	1.17	1.59	2.71	4.65
D4	0.84	0.194	6.0	0.2	1.17	1.59	2.71	4.65
D5a	0.49	0.348	9.5	0.2	0.91	1.25	2.13	3.66
D5b	0.46	0.698	20.9	0.2	0.62	0.85	1.44	2.48
D6	0.35	1.712	20.8	0.2	0.62	0.85	1.44	2.48
D7	0.80	0.042	6.0	0.2	1.17	1.59	2.71	4.65

Drainage Area I.D.	Peak Discharges				
	WQS	Extreme Storms			
	2x 85th Percentile cfs	1-yr cfs	2-yr cfs	10-yr cfs	100-yr cfs
D1	0.11	0.35	0.49	0.83	1.42
D2	0.06	0.22	0.30	0.50	0.86
D3	0.01	0.03	0.04	0.07	0.13
D4	0.03	0.19	0.26	0.44	0.76
D5a	0.03	0.16	0.22	0.37	0.63
D5b	0.06	0.20	0.27	0.46	0.79
D6	0.12	0.37	0.51	0.86	1.48
D7	0.01	0.04	0.05	0.09	0.16

Water Quality Storm - 2x 85th Percentile										
Pipe Segment I.D.	$t_{c,1}$ min	$t_{c,2}$ min	$Q_{p,1}$ cfs	$Q_{p,2}$ cfs	i_1 in/hr	i_2 in/hr	$Q_{tc,1}$ cfs	$Q_{tc,2}$ cfs	Q_p cfs	t_c min
N1	-	-	-	-	-	-	-	-	0.11	19.6
N2	-	-	-	-	-	-	-	-	0.06	15.6
N3	15.6	19.6	0.06	0.11	0.2	0.2	0.15	0.17	0.17	19.6
N4	-	-	-	-	-	-	-	-	0.01	6.0
N5	6.0	19.6	0.01	0.17	0.2	0.2	0.06	0.18	0.18	19.6
N6	-	-	-	-	-	-	-	-	0.03	6.0
N7	6.0	19.6	0.03	0.18	0.2	0.2	0.09	0.21	0.21	19.6
N8	-	-	-	-	-	-	-	-	0.21	19.6
S1	-	-	-	-	-	-	-	-	0.03	9.5
S2	9.5	20.9	0.03	0.1	0.2	0.2	0.06	0.10	0.10	20.9
S3	-	-	-	-	-	-	-	-	0.12	20.8
S4	20.8	20.9	0.12	0.10	0.2	0.2	0.22	0.22	0.22	20.9
S5	6.0	20.9	0.01	0.22	0.2	0.2	0.07	0.22	0.22	20.9

Extreme Event - 1-year										
Pipe Segment I.D.	$t_{c,1}$ min	$t_{c,2}$ min	$Q_{p,1}$ cfs	$Q_{p,2}$ cfs	i_1 in/hr	i_2 in/hr	$Q_{tc,1}$ cfs	$Q_{tc,2}$ cfs	Q_p cfs	t_c min
N1	-	-	-	-	-	-	-	-	0.35	19.6
N2	-	-	-	-	-	-	-	-	0.22	15.6
N3	15.6	19.6	0.22	0.35	0.70	0.64	0.50	0.55	0.55	19.6
N4	-	-	-	-	-	-	-	-	0.03	6.0
N5	6.0	19.6	0.03	0.55	1.17	0.64	0.20	0.57	0.57	19.6
N6	-	-	-	-	-	-	-	-	0.19	6.0
N7	6.0	19.6	0.19	0.57	1.17	0.64	0.37	0.67	0.67	19.6
N8	-	-	-	-	-	-	-	-	0.67	19.6
S1	-	-	-	-	-	-	-	-	0.16	9.5
S2	9.5	20.9	0.16	0.20	0.91	0.62	0.25	0.30	0.30	20.9
S3	-	-	-	-	-	-	-	-	0.37	20.8
S4	20.8	20.9	0.37	0.30	0.62	0.62	0.67	0.67	0.67	20.9
S5	6.0	20.9	0.04	0.67	1.17	0.62	0.23	0.69	0.69	20.9

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Extreme Event - 2-year										
Pipe Segment I.D.	$t_{c,1}$ min	$t_{c,2}$ min	$Q_{p,1}$ cfs	$Q_{p,2}$ cfs	i_1 in/hr	i_2 in/hr	$Q_{tc,1}$ cfs	$Q_{tc,2}$ cfs	Q_p cfs	t_c min
N1	-	-	-	-	-	-	-	-	0.49	19.6
N2	-	-	-	-	-	-	-	-	0.30	15.6
N3	15.6	19.6	0.30	0.49	0.97	0.88	0.69	0.76	0.76	19.6
N4	-	-	-	-	-	-	-	-	0.04	6.0
N5	6.0	19.6	0.04	0.76	1.59	0.88	0.28	0.78	0.78	19.6
N6	-	-	-	-	-	-	-	-	0.26	6.0
N7	6.0	19.6	0.26	0.78	1.59	0.88	0.50	0.93	0.93	19.6
N8	-	-	-	-	-	-	-	-	0.93	19.6
S1	-	-	-	-	-	-	-	-	0.22	9.5
S2	9.5	20.9	0.22	0.27	1.25	0.85	0.34	0.42	0.42	20.9
S3	-	-	-	-	-	-	-	-	0.51	20.8
S4	20.8	20.9	0.51	0.42	0.85	0.85	0.93	0.93	0.93	20.9
S5	6.0	20.9	0.05	0.93	1.59	0.85	0.32	0.95	0.95	20.9

Extreme Event - 10-year										
Pipe Segment I.D.	$t_{c,1}$ min	$t_{c,2}$ min	$Q_{p,1}$ cfs	$Q_{p,2}$ cfs	i_1 in/hr	i_2 in/hr	$Q_{tc,1}$ cfs	$Q_{tc,2}$ cfs	Q_p cfs	t_c min
N1	-	-	-	-	-	-	-	-	0.83	19.6
N2	-	-	-	-	-	-	-	-	0.50	15.6
N3	15.6	19.6	0.50	0.83	1.64	1.49	1.16	1.28	1.28	19.6
N4	-	-	-	-	-	-	-	-	0.07	6.0
N5	6.0	19.6	0.07	1.28	2.71	1.49	0.47	1.33	1.33	19.6
N6	-	-	-	-	-	-	-	-	0.44	6.0
N7	6.0	19.6	0.44	1.33	2.71	1.49	0.85	1.57	1.57	19.6
N8	-	-	-	-	-	-	-	-	1.57	19.6
S1	-	-	-	-	-	-	-	-	0.37	9.5
S2	9.5	20.9	0.37	0.46	2.13	1.44	0.58	0.71	0.71	20.9
S3	-	-	-	-	-	-	-	-	0.86	20.8
S4	20.8	20.9	0.86	0.71	1.44	1.44	1.57	1.57	1.57	20.9
S5	6.0	20.9	0.09	1.57	2.71	1.44	0.54	1.62	1.62	20.9

Extreme Event - 100-year										
Pipe Segment I.D.	$t_{c,1}$ min	$t_{c,2}$ min	$Q_{p,1}$ cfs	$Q_{p,2}$ cfs	i_1 in/hr	i_2 in/hr	$Q_{tc,1}$ cfs	$Q_{tc,2}$ cfs	Q_p cfs	t_c min
N1	-	-	-	-	-	-	-	-	1.42	19.6
N2	-	-	-	-	-	-	-	-	0.86	15.6
N3	15.6	19.6	0.86	1.42	2.81	2.56	1.99	2.21	2.21	19.6
N4	-	-	-	-	-	-	-	-	0.13	6.0
N5	6.0	19.6	0.13	2.21	4.65	2.56	0.80	2.28	2.28	19.6
N6	-	-	-	-	-	-	-	-	0.76	6.0
N7	6.0	19.6	0.76	2.28	4.65	2.56	1.46	2.70	2.70	19.6
N8	-	-	-	-	-	-	-	-	2.70	19.6
S1	-	-	-	-	-	-	-	-	0.63	9.5
S2	9.5	20.9	0.63	0.79	3.66	2.48	0.99	1.22	1.22	20.9
S3	-	-	-	-	-	-	-	-	1.48	20.8
S4	20.8	20.9	1.48	1.22	2.48	2.48	2.69	2.70	2.70	20.9
S5	6.0	20.9	0.16	2.70	4.65	2.48	0.93	2.78	2.78	20.9

Pipe Segment I.D.	Peak Discharge Summary									
	WQS - Minimum cfs	Extreme - 1yr cfs	Extreme - 2yr cfs	Extreme - 10yr cfs	Extreme - 100yr cfs	WQS - Minimum gpm	Extreme - 1yr gpm	Extreme - 2yr gpm	Extreme - 10yr gpm	Extreme - 100yr gpm
N1	0.11	0.35	0.49	0.83	1.42	50	159	219	371	638
N2	0.06	0.22	0.30	0.50	0.86	28	97	133	226	388
N3	0.17	0.55	0.76	1.28	2.21	77	247	340	577	990
N4	0.01	0.03	0.04	0.07	0.13	2	14	20	33	57
N5	0.18	0.57	0.78	1.33	2.28	80	255	351	595	1022
N6	0.03	0.19	0.26	0.44	0.76	15	86	117	199	342
N7	0.21	0.67	0.93	1.57	2.70	95	302	416	704	1210
N8	0.21	0.67	0.93	1.57	2.70	95	302	416	704	1210
S1	0.03	0.16	0.22	0.37	0.63	15	71	97	165	283
S2	0.10	0.30	0.42	0.71	1.22	44	137	188	319	548
S3	0.12	0.37	0.51	0.86	1.48	53	166	228	386	663
S4	0.22	0.67	0.93	1.57	2.70	98	302	416	704	1210
S5	0.22	0.69	0.95	1.62	2.78	101	312	429	726	1247

Hydraulic Calculations

Storm Sewer Pipe Sizing

Storm sewer pipes were sized to the 100-year event such that open channel flow would be maintained and pipes would flow at most 70% full, to maintain air-flow through the top portion of the pipe. This will ensure that water does not backup and pool at the low-points around the site.

Open channel flow calculations were performed following the equations below (Lindeburg, 2014), and were calculated iteratively using a Newton-Raphson iterating scheme:

$$\theta_{deg} = 2\cos^{-1}\left(\frac{\frac{D}{2} - d}{\frac{D}{2}}\right)$$

$$R_h = \frac{A}{P} \qquad \frac{n}{n_{full}} = 1 + \left(\frac{d}{D}\right)^{0.54} - \left(\frac{d}{D}\right)^{1.2}$$

$$A = \left(\frac{D}{2}\right)^2 \frac{\theta_{rad} - \sin\theta_{deg}}{2} \qquad V = \left(\frac{1.486}{n}\right) R_h^{2/3} S^{1/2}$$

$$P = \frac{D\theta_{rad}}{2} \qquad Q = AV$$

where:

$\theta =$ Internal angle of water surface
$D =$ Pipe inner diameter, ft
$d =$ Flow depth, ft
$A =$ Flow area, ft^2
$P =$ Wetted perimeter, ft
$R_h =$ Hydraulic radius, ft
$V =$ Average flow velocity, ft/s
$n =$ Manning's roughness coefficient
$S =$ Pipe bed slope, ft/ft
$Q =$ Discharge, cfs
$n_{full} =$ Pipe-full roughness coefficient

The following assumptions were employed in the storm sewer hydraulic calculations:

(1) In order to allow for sufficient airflow, and to prevent periodic pressurization of the pipe where unintended, the pipe size is designed to convey the flow in an open-channel condition with the depth less than 70% of the inner diameter of the pipe, and a maximum of 75% full.

(2) The pipe is assumed to be plastic or some other smooth interior pipe, and non-profile wall pipe. Typical roughness coefficients associated with plastic pipe with smooth inner walls ranges from 0.009 to 0.015. Accordingly, a conservative roughness coefficient of 0.013 was applied, which will account for potential pipe degradation over time.

(3) Based on standard sewer design, the pipe is considered self-cleaning if the velocity is greater than 2.0 ft/s. Above 1.5 ft/s is acceptable if occasional flushing flows are expected. The pipes were designed to meet this criterion.

Pipe Segment I.D.	Discharge, Q gpm	Pipe Nom Diameter in	Pipe Inner Diameter ft	Slope ft/ft	Rough Coeff, n	Flow Depth, d ft	<70% Full?
N1	638	10	0.79	0.015	0.013	0.50	64%
N2	388	10	0.79	0.015	0.013	0.38	48%
N3	990	14	1.03	0.013	0.013	0.58	56%
N4	57	6	0.48	0.05	0.013	0.12	26%
N5	1022	14	1.03	0.01	0.013	0.64	62%
N6	342	10	0.79	0.024	0.013	0.31	39%
N7	1210	14	1.03	0.01	0.013	0.71	69%
N8	1210	14	1.03	0.01	0.013	0.71	69%
S1	283	10	0.79	0.015	0.013	0.32	40%
S2	548	10	0.79	0.03	0.013	0.38	48%
S3	663	14	1.03	0.01	0.013	0.50	49%
S4	1210	14	1.03	0.01	0.013	0.71	69%
S5	1247	14	1.03	0.01	0.013	0.72	70%

Pipe Segment I.D.	Internal Angle, θ deg	Flow Area, A ft^2	Flow Velocity, V ft/s	Self-Cleaning?
N1	211	0.33	4.31	OK
N2	175	0.23	3.71	OK
N3	195	0.49	4.51	OK
N4	123	0.04	3.45	OK
N5	208	0.55	4.16	OK
N6	156	0.18	4.22	OK
N7	224	0.61	4.38	OK
N8	224	0.61	4.38	OK
S1	158	0.19	3.38	OK
S2	175	0.23	5.24	OK
S3	177	0.40	3.65	OK
S4	224	0.61	4.39	OK
S5	227	0.63	4.43	OK

Treatment BMP Calculations

Hydrodynamic Separator

A hydrodynamic separator was sized for the site using the following characteristics, and the ContechES proprietary sizing system:

	North Site	South Site	Units	
Water Quality Flow, WQQ:	0.33	0.36	cfs	1-yr, 1-hr; for compliance with CA Statewide Trash Amendments for all Phase I and II MS4s **Note, this is not the 1-year extreme event, which has a duration based on time of concentration*
Peak Flow:	2.70	2.78	cfs	
Pipe Inlet Diameter:	15.00	15.00	in	
100% Trash Removal?:	Yes	Yes		
Grated Top?:	Yes	Yes		
TSS Removal Efficiency:	80%	80%		
Depth to Bedrock:	>15	10-15	ft	
Depth to Ground Water:	10-15	10-15	ft	
Pipe Invert Depth:	5-10	0-5	ft	

The resultant proprietary system recommended by Contech was their CDS Hydrodynamic Separator, with typical section below. Similar evaluations could be performed from alternative manufacturers.

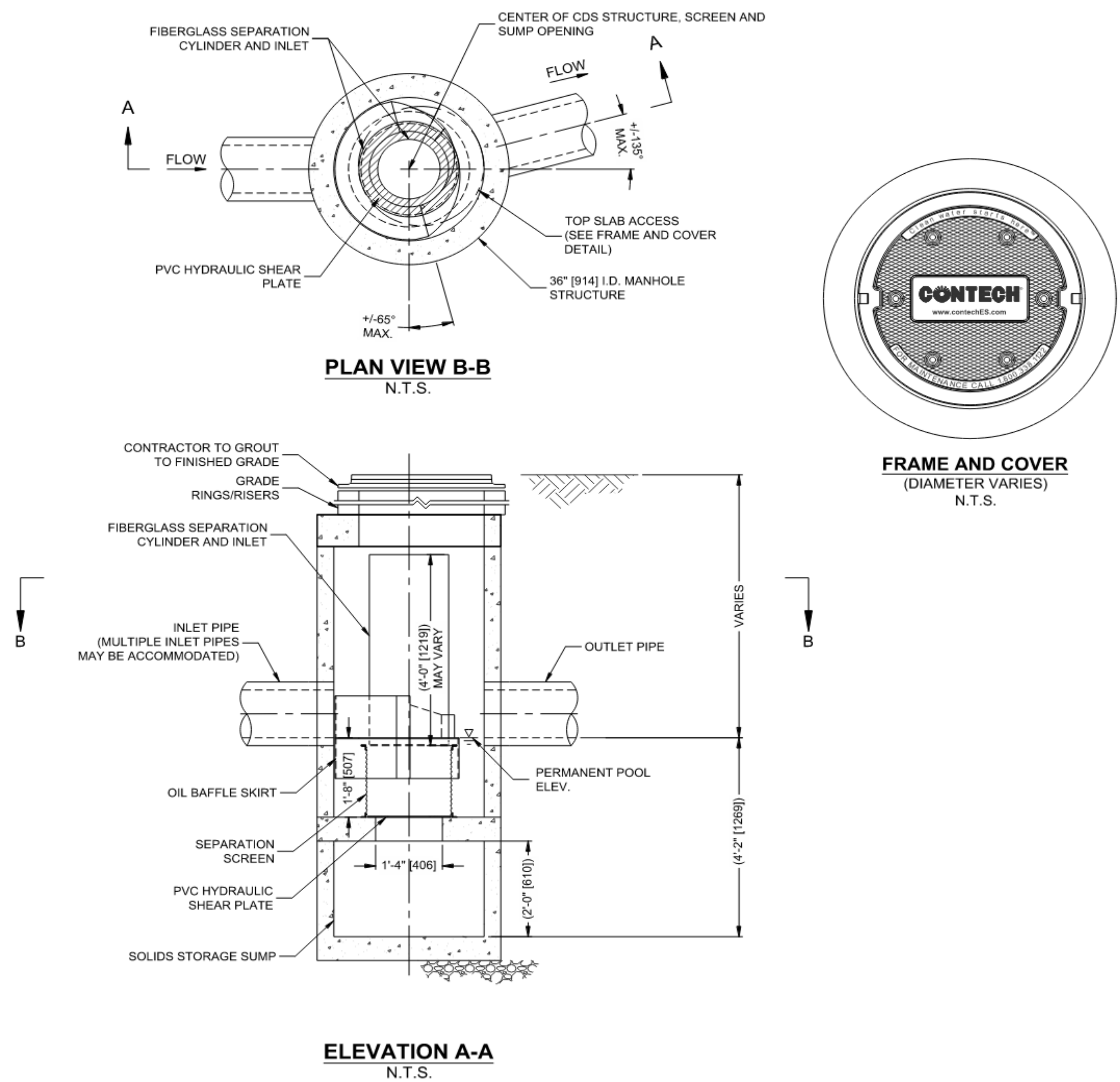

Figure 4. Contech 1515-3-C CDS Hydrodynamic Separator Typical Sections

The Contech CDS System has received certification from the California Statewide Trash Amendments Full Capture System, provided that it is sized to treat the peak flow rate from the 1-year, 1-hour design storm, or the peak flow capacity of the corresponding storm drain, whichever is less.

Drainage Sump Calculations

Lastly, the perforated pipe in the drain rock sump was sized to determine the length of perforated pipe required for various runoff events and the depth of drain rock sump required. The flow into the perforated pipe, and conveyed to the southern storm drain system was calculated according to an orifice flow equation adjusted for the drain rock application:

$$Q = C_D B L A_0 \sqrt{2gh}$$

where:

$Q =$ Discharge to the perforated pipe, cfs
$C_D =$ Discharge coefficient, 0.61 for sharp edged holes in pipe
$B =$ Clogging factor, 0.5 for blinding over time
$L =$ Length of perforated pipe, ft
$A_0 =$ Unit area of perforations, ft^2/ft
$h =$ Head above the perforations center of area, ft

These calculations are summarized for different design events below:

Design Event	Design Discharge, Q cfs	Assumed Length, L ft	Blinding Factor, B	Discharge Coeff. C_D	Unit Area*, A_0 ft^2/ft	Head, h ft
WQS	0.10	12	0.5	0.61	0.012	0.07
Extreme - 1-yr	0.30	12	0.5	0.61	0.012	0.72
Extreme - 2-yr	0.42	12	0.5	0.61	0.012	1.35
Extreme - 10-yr	0.71	12	0.5	0.61	0.012	3.89
Extreme - 100-yr	1.22	12	0.5	0.61	0.012	11.46

** Unit area based on AASHTO M278/ASTM F758 perforation pattern with 4 x 3/8" diameter holes at a spacing of 3 inches.*

In the above calculations 12 ft of assumed length was applied based on the dimensions of the drain rock sump. It can be seen that for everything up to a 2-year runoff event, the peak discharge can be contained within a 2.5-ft deep sump, even after some blinding has occurred. For more extreme events than the 2-year event, it is expected that the sump will overflow. Overflow will naturally drain across the driveway and down the existing natural flow path to Fall Creek. However, pollutant load in runoff is typically taken up by the initial stages of runoff, prior to the arrival of the peak flow. These will report to the drainage sump first and will pass into to the storm drain system and ultimately to the hydrodynamic separator. It is expected that only sediment will be a pollutant concern during the peak discharge, and the vegetated swale and drainage sump provide natural locations for settling of any sediment load prior to overflow to Fall Creek.

Conclusion

The stormwater management strategy for the site will consist of finish grading the site to one of 7 catch basins and/or sumps located around the site. Figure 3 above shows the locations of the catch basins, and their respective drainage areas, as delineated based on preliminary finish grading of the pads. These catch basins and sumps then report to a storm sewer system that will convey flows to one of two hydrodynamic separators at the site. The storm sewer system was sized to convey the 100-year extreme rainfall event at the site without pressurizing or backing water up at the catch basins. The hydrodynamic separators will be Contech ES CDS systems, or equal, and will be sized to the WQ Storm Event as dictated by the CASQA. Typical sections of the CDS system are provided in Figure 4 above. The CDS system will treat the water for trash, hydrocarbons, and total suspended solids (TSS; sediment). The CDS systems will then outlet to a pipe that conveys flows back to Fall Creek and its tributary drainage.

SUBJECT: Klamath River Renewal Corporation | BY: A. Leman | CHK'D BY: V. Autier
Fall Creek Hatchery | DATE: 10/28/2020
Riprap Sizing | PROJECT NO.: 20-024

Purpose

The purpose of this calculation sheet is to size the riprap to mitigate scour potential on-site.

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Background

There exist five locations on the Fall Creek Hatchery site where there is an expected scour potential due to in-stream modifications, and riprap will be required to mitigate any instabilities in the stream profile. These locations include:

1. The embankment adjacent to the intake structure;

2. Either side of the Dam B velocity apron, where there is potential for overflow of Dam B;

3. The fish plunge pool at the outlet of the Chinook fish release pipe;

4. The entrance pool and fish plunge pool at the toe of the fish ladder; and

5. The fish barrier berm in the floodplain adjacent to the lower fish exclusion barrier.

The scour mechanism at these locations differ slightly in character. At locations 1 and 2, scour potential exists due to potential for localized shear stresses on the bed (or banks) due to velocities reaching a level that is able to cause material to roll, saltate, or enter into suspension in the water column. In locations 3 and 4, erosion potential exists due to a turbulent plunging flow that will dislodge material and move it downstream creating a local scour hole. In location 5, the riprap is subjected to overtopping flow which will see shallow sheet flow over the berm that has potential to erode material. These three mechanisms are different in character and will be treated separately. The following sections summarize the methods for evaluating scour potential by location.

Localized Shear Stresses
Incipient Motion - Shields Diagram
(See Garcia, 2008)

For a given shear stress condition, the grain size for which there would be incipient motion can be approximated by the experimental data of Shields summarized in the Shields diagram (see Figure 1).

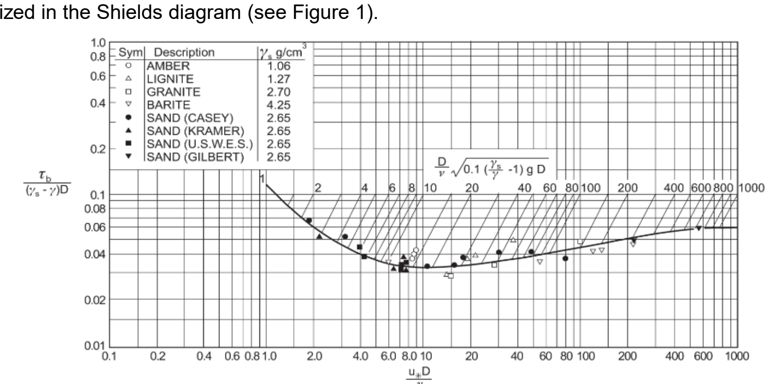


Figure 1. Shields Diagram for Incipient Motion (Source; Vanoni, 1964)

A useful fit to the Shields diagram was later proposed by Brownlie (1981), and was recast in terms of a particle Reynolds number and a critical Shields number such that the relationship could be made explicit:

$$\tau_c^* = 0.22Re_p^{-0.6} + 0.06e^{-17.77Re_p^{-0.6}}$$

$$Re_p = \frac{\sqrt{gRD}D}{\nu}$$

$$\tau^* = \frac{u_*^2}{gRD}$$

where:

$\tau_c^* =$ Critical Shields number
$\tau^* =$ Shields number, defined at left
$Re_p =$ Particle Reynolds number, defined at left
$g =$ Gravitational constant, 32.2 ft/s^2
$R =$ Submerged specific gravity, quartz ~ 1.65
$D =$ Grain size, assumed 50% passing
$\nu =$ Kinematic viscosity of water, ft^2/s
$u_* =$ Shear velocity, ft/s

The shear velocity could then be related to the average velocity using the Manning-Strickler relation for flow resistance (Garcia, 2008):

$$\frac{U}{u_*} = 8.1\left(\frac{H}{k_s}\right)^{1/6}$$

$$k_s = \alpha_s D_{xx}$$

where:

$U =$ Cross-section averaged velocity, ft/s
$H =$ Flow depth, ft
$k_s =$ Nikuradse roughness height, ft
$\alpha_s =$ Ratio, see below
$D_{xx} =$ Sediment size such that XX% is finer, ft

Finally, the Nikuradse equivalent roughness height for the Manning-Strickler relationship could be determined using the Strickler relationship to the D50 grain size:

Table 2-1 Ratio of Nikuradse Equivalent Roughness Size and Sediment Size for Rivers

Investigator	Measure of sediment size, D_x	$\alpha_s = k_s/D_x$
Ackers and White (1973)	D_{35}	1.23
Aguirre-Pe and Fuentes (1990)	D_{84}	1.6
Strickler (1923)	D_{50}	3.3
Katul et al (2002)	D_{84}	3.5
Keulegan (1938)	D_{50}	1
Meyer-Peter and Muller (1948)	D_{50}	1
Thompson and Campbell (1979)	D_{50}	2.0
Hammond et al. (1984)	D_{50}	6.6
Einstein and Barbarossa (1952)	D_{65}	1
Irmay (1949)	D_{65}	1.5
Engelund and Hansen (1967)	D_{65}	2.0
Lane and Carlson (1953)	D_{75}	3.2
Gladki (1979)	D_{80}	2.5
Leopold et al. (1964)	D_{84}	3.9
Limerinos (1970)	D_{84}	2.8
Mahmood (1971)	D_{84}	5.1
Hey (1979), Bray (1979)	D_{84}	3.5
Ikeda (1983)	D_{84}	1.5
Colosimo et al. (1986)	D_{84}	3.6
Whiting and Dietrich (1990)	D_{84}	2.95
Simons and Richardson (1966)	D_{85}	1
Kamphuis (1974)	D_{90}	2.0
Van Rijn (1982)	D_{90}	3.0

Figure 2. Relationships of Sediment Size to Equivalent Nikuradse Roughness (Source; Garcia, 2008)

Adjustment for Slopes

Where the grains are located on a slope, grains are much more prone to movement, and an adjustment factor can be applied based on the presumed angle of repose of the material:

$$\frac{\tau_{c,\alpha}^*}{\tau_{c,0}^*} = \cos\alpha\left(1 - \frac{\tan\alpha}{\tan\phi}\right)$$

where:

$\tau_{c,\alpha}^* =$ Critical Shields parameter on a slope
$\tau_{c,0}^* =$ Critical Shields parameter on a mild / no slope
$\alpha =$ Slope angle, degrees
$\phi =$ Material angle of repose, degrees

Plunging Flow
NRCS Design Note-6 (DN-6)
(NRCS, 1986)

Plunging flow riprap design followed the equations of the NRCS Design Note 6 for plunge pool design. Figure 3 presents a definition sketch of the plunge pool characteristics.

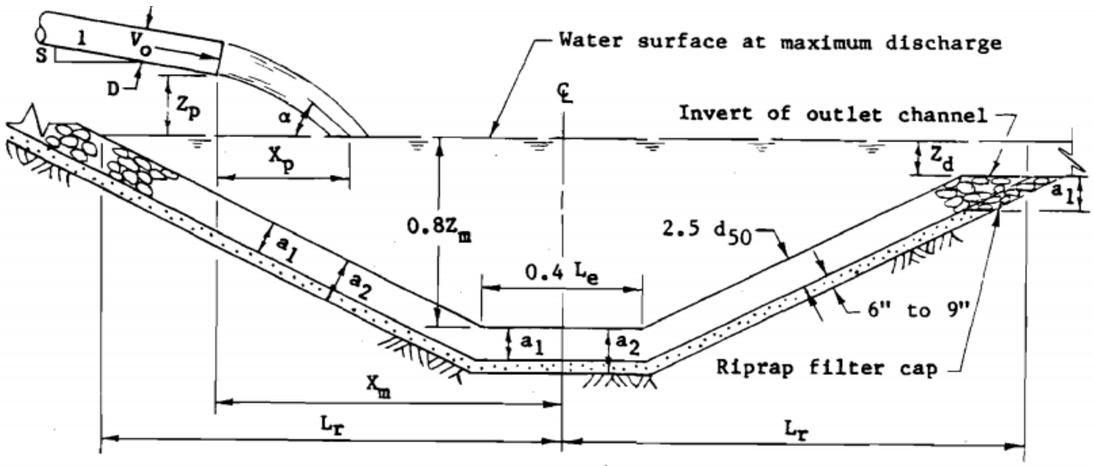


Figure 3. Plunge Pool Definition Sketch (Source; NRCS, 1986)

It is noteworthy, that the plunge pool depth was checked for fish passage requirements in the hydraulic calculations. Please refer to the hydraulic calculations for bypass pipe outlet design and fish passage requirements. The present discussion relates to the sizing of the riprap in the plunge pool.

The riprap sizing in DN-6 is based on an allowable eroded depth, which, per the definition sketch is 125% (1 / 0.8) of the plunge pool depth at maximum discharge. Given the depth of the plunge pool, as determined in the fish passage analysis, the riprap size was calculated, according to the following equations:

$$F_d = \frac{V_p}{\sqrt{gD_{50}R}}$$

For $\frac{z_p}{D} < 1$

$$z_m = 7.5\mathrm{D}\left[1 - e^{-0.6(F_d - 2)}\right]$$

For $\frac{z_p}{D} \geq 1$

$$z_m = 10.5\mathrm{D}\left[1 - e^{-0.35(F_d - 2)}\right]$$

where:

$F_d =$ Densimetric Froude Number
$V_p =$ Plunging velocity, ft/s
$g =$ Gravitational constant, 32.2 ft/s^2
$D_{50} =$ Median grain size of riprap layer, ft
$R =$ Submerged Specific Gravity of Riprap, ~1.65 quartz
$Q =$ Pipe flow rate, cfs
<other values as defined above in Figure 3>

Moreover, to control shallow beach type erosion at the top edge of the plunge pool:

$$\frac{Q}{\sqrt{gD^5}} \leq \left(1.0 + 25\frac{D_{50}}{D}\right)$$

The following equations provide minimum sizes for the plunge pool assumed in the riprap lining analysis.

$$x_m = \left[x_p + \frac{z_m}{\tan\alpha}\right] 1.15 e^{-0.15\frac{Q}{\sqrt{gD^5}}}$$

$$L_e = z_m\left[1.5 + \frac{1}{3}\frac{Q}{\sqrt{gD^5}}\right]$$

Overtopping Flow
USBR Method
(see Mishra and Ruff, 1998)

The U.S. Bureau of Reclamation in conjunction with Colorado State University performed embankment overtopping studies to explore erosion protection for overtopped embankment dams. This scenario most closely resembles the erosion regime of Location 5. They developed an equation for stable riprap sizing (with no implicit safety factor; SF to be added) based on their experimental results [metric units]:

$$D_{50}C_u^{0.25} = 0.55q_f^{0.52}S^{-0.75}\left(\frac{\sin\alpha}{(SG\cos\alpha - 1)(\cos\alpha\tan\varphi - \sin\alpha)}\right)^{1.11}$$

where:
$C_u =$ Coefficient of uniformity, D60/D10
$q_f =$ Unit discharge, m^2/s/m
$SG =$ Specific gravity of riprap, ~2.65 quartz
$S =$ Slope, m/m

This equation will be used to size riprap for the overtopping at Location 5.

Riprap Classes

Riprap was selected from readily available grain size distributions per the specifications, and per CalTrans Rock Slope Protection material classes. The readily available riprap classes were as follows:

Percent Passing	Type I (6-inch)	Type II (12-inch)	Type III (18-inch)	Type IV (24-inch)
95-100	12	18	24	30
25-75	6	12	18	24
15-25	-	-	-	18
0-5	3	6	13	12

Information - Input

The following parameters were used as inputs to this analysis:

General Input Parameters	Value	Units	Comments
Gravitational Constant	32.2	ft/s^2	Constant
Kinematic Viscosity of Water	1.41E-05	ft^2/s	@ 50F
Riprap Angle of Repose	40	°	typical, see USACE, 1991
50% Slope Angle	27	°	arctan(1/2)
Ratio of D50 to Nikuradse Roughness	3.3		see Figure 2, above
Riprap Submerged Specific Gravity	1.65		Quartz, SG = 2.65

Location 3 Parameters	Value	Units	Comments
Low Tailwater Elevation (Fish Passage Low)	2492.4	ft	From Hydraulic Calculations
High Tailwater Elevation (100-year)	2494.5	ft	From Hydraulic Calculations
Plunge Pool Bottom Elevation	2488.4	ft	Design Value
Pipe IE	2495.24	ft	Design Value
Impact velocity at Low TW	15.4	ft/s	From Hydraulic Calculations
X-Distance to Impact at Low TW, x_p	2.79	ft	From Hydraulic Calculations
Maximum Pipe Flow	4.5	cfs	Design Value
Plunge Pool Bottom Width	5	ft	Design Value
Plunge Pool Bottom Length	5	ft	Design Value
Direction of Plung. Flow at Water Surface, tan(α)	2.0		From Hydraulic Calculations, Vx,p = 6.49 ft/s, Vy,p = 13.01 ft/s

Location 4 Parameters	Value	Units	Comments
Low Tailwater Elevation (Fish Passage Low)	2484.12	ft	From Hydraulic Calculations
High Tailwater Elevation (100-year)	2487.21	ft	From Hydraulic Calculations
Plunge Pool Bottom Elevation	2482.07	ft	Design Value
Pipe IE	2486.25	ft	Design Value
Impact velocity at Low TW	13.21	ft/s	From Hydraulic Calculations
X-Distance to Impact at Low TW, x_p	2.2	ft	From Hydraulic Calculations
Maximum Pipe Flow	10	cfs	Design Value
Plunge Pool Bottom Width	5	ft	Design Value
Plunge Pool Bottom Length	4	ft	Design Value
Direction of Plung. Flow at Water Surface, tan(α)	2.84		From Hydraulic Calculations, Vx,p = 4.56 ft/s, Vy,p = 12.97 ft/s

Location 5 Parameters	Value	Units	Comments
Slope	0.33	ft/ft	Design Value
Uniformity Coefficient	2	-	Per riprap gradations, conservative
Overflow Discharge	461	cfs	From Hydraulic Calculations
Length of OF Weir	30	ft	From Hydraulic Calculations
Unit Discharge	15.4	cfs/ft	Calculated

Calculations

Locations 1-2 - Localized Shear Stresses

Flow depths and flow velocities were identified for the two locations of interest, and used to determine the required riprap size to withstand the associated shear stresses. These are summarized below:

Location 1: At location 1, the riprap was located far enough upstream of Dam A, that it could be reasonably assumed that the flow velocity would be distributed over the entire cross-section. The maximum powerhouse flow, 50 cfs, was therefore distributed over the entire flow area (measured in CAD, for the calculated water surface set by the weir overflow). This resulted in low velocities, however, it is expected that the proximity to the powerhouse would warrant a nominally sized rock slope lining to account for waves and potential debris on the embankment.

Location 2: At location 2, it is expected that extreme events would overtop Dam B and would be channelized between the velocity apron wall and the valley sidewall, resulting in localized shear stresses. Weir calculations were performed to determine the proportion of flow that would report to this channelized portion (30%), and open channel calculations were performed to determine the flow depth and flow velocity on the 16H:1V slope along the edge of the wall.

Location	Guess D_{50}, in	Flow Depth, H ft	Flow Velocity, U ft/s	k_s in	u* ft/s	Re_p	τ*	$τ^*_c$	Factor of Safety, FS
1 - Intake	6	3.4	0.43	19.8	0.05	1.83E+05	0.000	0.021	256.4
2 - Dam B Overflow	24	2.3	13.5	79.2	1.99	1.46E+06	0.037	0.055	1.5

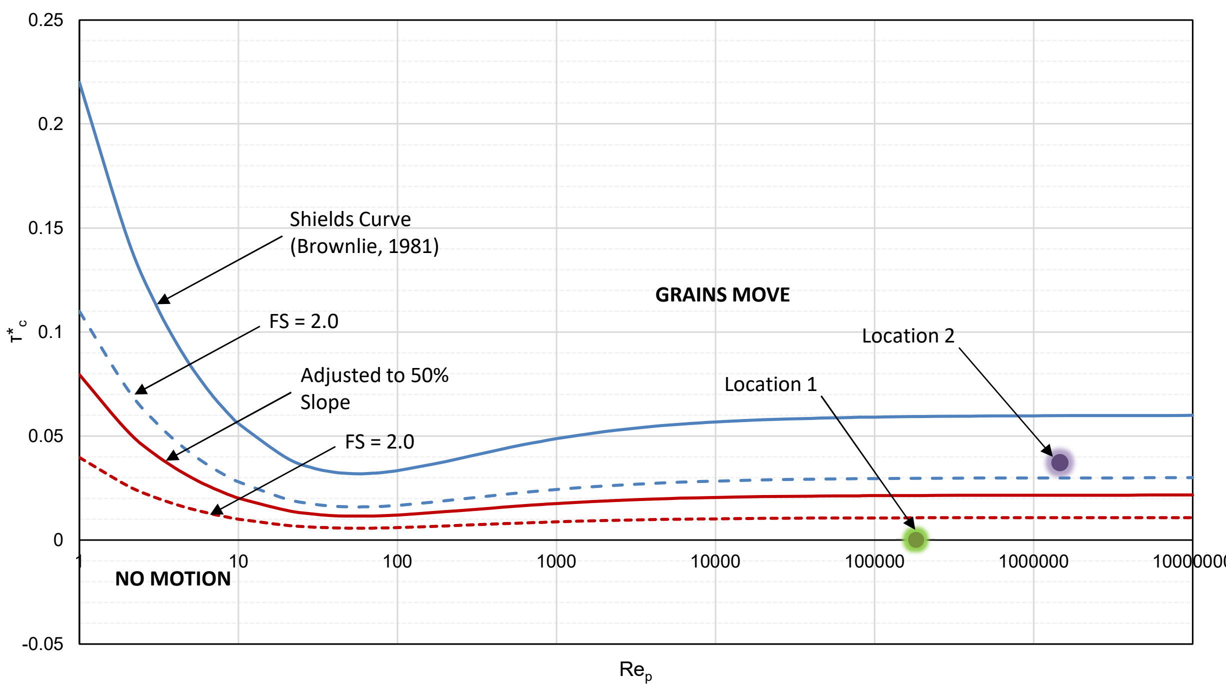


Figure 4. Shields Diagram with Locations 1 & 2 Identified

Locations 3-4 - Plunging Flow

Plunging flow calculations were performed for low tailwater elevations, because that case will be critical for pool scour. The following table summarizes the plunging flow calculations.

Location	D_{50}	$\frac{Q}{\sqrt{gD^5}}$	F_d	z_m	Min Pool Depth	Depth OK?	L_e	Bottom Dim OK?	x_m
	in			ft	ft		ft		ft
3 - Chinook Plunge Pl	12	0.66	2.1	0.44	0.5	OK	0.75	OK	3.13
4 - Entrance Pool	9	0.22	2.1	0.94	1.2	OK	1.47	OK	2.82

** Note: Location 4 has a riprap size between Type I and Type II riprap. The larger of these two sizes should be used. Type II riprap will line the entrance pool. This is the controlling condition at the entrance pool. Hydraulic calculations in the Denil fishway show that the baffles sufficiently slow the flow such that erosion from the Denil is not the primary concern.*

Location 5 - Overtopping Flow

Location	C_u	q_f	S	Min D_{50}	Use D_{50}	Safety Factor
		cms/m	m/m	m	in	
5 - Fish Barrier Berm	2	1.4	0.33	0.5	24.0	1.23

Conclusion

Riprap was sized for the five locations on-site where work will be done within the stream corridor that requires erosion protection rock lining. Riprap was sized by different methods as required by the anticipated erosion mechanism. It is expected that riprap will be sized according to one of 4 riprap gradations as presented in the specifications. A summary of the riprap sizes and methods used for evaluation is provided below.

Location/Description	D_{50} in	Riprap Type	Evaluation Method
1 - Adjacent to intake	6	I	Shields diagram
2 - Adjacent to Dam B apron	24	IV	Shields diagram
3 - Chinook plunge pool	12	II	Plunge pool geometry (NRCS)
4 - Denil entrance pool	12	II	Plunge pool geometry (NRCS)
5 - Fish barrier berm	24	IV	Overtopping (USBR)

Appendix C
Structural Design Calculations

Calculation Cover Sheet

Rev. 0

McMILLEN JACOBS ASSOCIATES

Project: Fall Creek Fish Hatchery

Client: KRRC **Proj. No.** 20-024

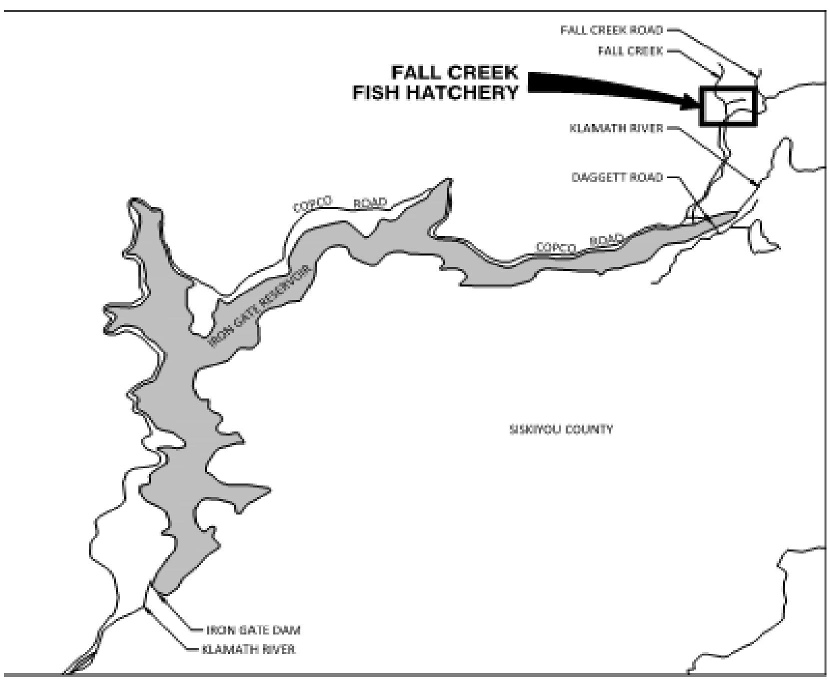
Title: Structural Calculations

Prepared By, Name: Zachary Autin

Prepared By, Signature: ______ **Date:** 10/19/2020

Peer Reviewed By, Name: Taylor Bowen

Peer Reviewed, Signature: ______ **Date:** ______



SUBJECT:	KRRC	**BY:** Zachary Autin	**CHK'D BY:** Taylor Bowen
	Fall Creek Fish Hatchery	**DATE:** 10/19/2020	
	Structural Calculations	**PROJECT NO.:** 20-024	

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SUBJECT:	KRRC	**BY:** Zachary Auti	**CHK'D BY:** Taylor Bowen
	Fall Creek Fish Hatchery	**DATE:** 10/19/2020	
	Structural Calculations	**PROJECT NO.:** 20-024	

Purpose

Present general structural design information relevant to all calculations including:

- *References, Codes, and Standards*
- *General Information*
- *Load Combinations*
- *Design Basis*

References

- *ACI 318-14: Building Code Requirements for Structural Concrete*
- *ACI 350-06: Code Requirements for Environmental Engineering Concrete Structures*
- *AISC 341-16: Seismic Provisions for Structural Steel Buildings*
- *AISC 360-16: Specification for Structural Steel Buildings*
- *AISC Steel Construction Manual, 15th Edition*
- *AISC Steel Design Guide 27: Structural Stainless Steel*
- *AWS D1.1: Structural Steel Welding Code -- Steel*
- *ASCE 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*
- *2019 California Building Code (CBC) as amended by Siskiyou County*
- *BEFS 2019: Nonresidential Compliance Manual for the 2019 Building Energy Efficiency Standards, Title 24, Part 6*
- *PCA PL279.01D: Portland Cement Association - Reinforcing Bar Specifications - 1911 through 1968*

General Information

Material Properties

Specific Weights

γ_w =	**62.4** lb/ft^3	*Unit weight of Water*
γ_s =	**490** lb/ft^3	*Unit weight of Steel*
γ_{SST} =	**500** lb/ft^3	*Unit weight of Stainless Steel*
γ_c =	**150** lb/ft^3	*Unit weight of Concrete*
γ_{native}=	**125** lb/ft^3	*Unit weight of Native Soil*
γ_a =	**172.8** lb/ft^3	*Unit weight of Aluminum*

Steel Properties

E_s =	**29000** ksi	*Elastic Modulus*

Wide Flanges (W Shapes)

Grade:	**A992**	*High-Strength Low-Alloy Steel*
F_y =	**50** ksi	*Yield Strength*
F_u =	**65** ksi	*Tensile Strength*

Channels, Angles, Plates and Bars

Grade:	**A36**	*Carbon Steel*
F_y =	**36** ksi	*Yield Strength*
F_u =	**58** ksi	*Tensile Strength*

Rectangular HSS

Grade:	**A500 Gr. B**	*Carbon Steel*
F_y =	**46** ksi	*Yield Strength*
F_u =	**58** ksi	*Tensile Strength*

Round HSS

Grade:	**A500 Gr. B**	*Carbon Steel*
F_y =	**42** ksi	*Yield Strength*
F_u =	**58** ksi	*Tensile Strength*

Pipe

Grade:	**A53 Gr. B**	*Carbon Steel*
F_y =	**35** ksi	*Yield Strength*
F_u =	**60** ksi	*Tensile Strength*

Stainless Steel Properties

E_s =	**28000**	ksi	*Elastic Modulus*

Bars and Shapes

Grade:	**A276**		*316 Austenitic Stainless Steel*
F_y =	**30**	ksi	*Yield Strength*
F_u =	**75**	ksi	*Tensile Strength*

HSS

Grade:	**A312**		*316 Austenitic Stainless Steel*
F_y =	**30**	ksi	*Yield Strength*
F_u =	**75**	ksi	*Tensile Strength*

Plate

Grade:	**A240**		*316 Austenitic Stainless Steel*
F_y =	**30**	ksi	*Yield Strength*
F_u =	**75**	ksi	*Tensile Strength*

Aluminum Properties

E_a =	**10100**	ksi	*Elastic Modulus*

Sheet and Plate (B209)

Grade:	**6061-T6**		
Fty =	**35**	ksi	*Yield Strength*
Ftu =	**42**	ksi	*Tensile Strength*
Ftyw =	**11**	ksi	*Yield Strength*
Ftuw =	**24**	ksi	*Tensile Strength*
Fcy =	**31.5**	ksi	*Yield Strength*
Fsu =	**25.2**	ksi	*Tensile Strength*
Fsy =	**21**	ksi	*Yield Strength*
Fcyw =	**11**	ksi	*Yield Strength*
Fsuw =	**14.4**	ksi	*Tensile Strength*
Fsyw =	**6.6**	ksi	*Yield Strength*

New Concrete Properties

fc' =	4.5	ksi	*Compressive strength*
fy_bar =	60	ksi	*Yield Strength of steel reinforcement*
fu_bar =	90	ksi	*Ultimate strength of steel reinforcement*
Es =	29000	ksi	*Modulus of elasticity of steel reinforcement*

Existing Concrete Properties

fc' =	2.5	ksi	*Compressive strength*
fy_bar =	33	ksi	*Yield Strength of steel reinforcement*
fu_bar =	55	ksi	*Ultimate strength of steel reinforcement*
Es =	29000	ksi	*Modulus of elasticity of steel reinforcement*

	Structural	Intermediate	Hard	Cold-twisted
Yield min., psi (MPa)	33,000 (228)	40,000 (276)	50,000 (345)	55,000 (379)
Tensile, psi (MPa)	55,000 (379) to 70,000 (483)	70,000 (483) to 85,000 (586)	55,000 (379) min.	n/a

Soil Properties - Structural Fill

mu_CIP =	0.49		*Soil friction coefficient - cast in place*	
mu_precast =	0.39		*Soil friction coefficient - precast*	
Pa =	3000	psf	*Allowable Bearing Pressure*	**0.3 in settlement (Whitney Ciani email)**

Soil Properties - Native Soil

Es =	600	ksf	*Elastic modulus*	
phi =	30	degrees	*Internal angle of friction*	
	0.523598776	radians		
c =	200	psf	*Cohesion*	
Ka =	0.29		*Active Pressure Coefficient*	**Whitney Ciani Email**
Ko =	0.5		*At-rest Pressure Coefficient*	**Whitney Ciani Email**
Ke =	0.35		*Seismic pressure coefficient*	**Whitney Ciani Email**

Load Cases

Dead Loads

Siskiyou County Building Department has the following requirements;

Design Information

The County's Minimum Elevation is 1,000 feet and the Maximum Elevation is 14,152 feet. The following design elements must be considered for all projects in Siskiyou County.

1. **Roof design loads for site above 5,000 feet elevation** - Must be obtained from the Building Division.
2. **Flat roof snow load below 5,000 feet elevations -** McCloud, Mt. Shasta, Dunsmuir, Weed and Happy Camp, 60 pounds per square foot. All Other areas, 40 pounds per square foot.
3. **Basic Wind Speed** - VASD 90 mph with VULT 115 mph: All areas, 20 pounds per square foot.
4. **Earthquake** - the Seismic Design Category is determined by the Design Professional.
5. **Soils Site Class** - Based on soils investigation
6. **Climate Zone 16** - for Energy compliance
7. **Frost Depth** - 12 inch minimum

Dead Loads

Roof dead = 5.5 psf *(Self Weight)*

Live Loads

Sidewalks, vehicular driveways, and yards subject to trucking

(ASCE 7-16 Table 4.3-1)
250 psf
8,000 lbs concentrated

Pedestrian

(ASCE 7-16 Table 4.3-1)
Corridors = 100 psf
Walkways and Elevated Platforms = 60 psf

Roof

(ASCE 7-16 Table 4.3-1)
Roof Live = 20 psf
Collateral = 3 psf

Hydrostatic Loads

Loads due to hydrostatic pressure increase linearly with depth (y).

Phs = γ_w*y

Earth Loads

Lateral earth pressures are calculated based on equivalent fluid earth pressure values given above. Earth pressures increase linearly with depth (y).

Ph = EFP*y

Wind Loads

V =	**115**	mph
Iw =	**1**	
Surface Roughness =	**B**	
Gcpi =	**0.18**	psf
Gcpi =	**-0.18**	psf

Governed by Siskiyou County requirements.

ASCE 7 Hazards Report

Address: No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Elevation: 2504.85 ft (NAVD 88)
Latitude: 41.984436
Longitude: -122.362037

Wind

Results:

Wind Speed:	95 Vmph
10-year MRI	66 Vmph
25-year MRI	72 Vmph
50-year MRI	77 Vmph
100-year MRI	81 Vmph
Data Source:	ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4
Date Accessed:	Mon Feb 24 2020

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

Seismic Loads

Ss =	**0.584**	g
S1 =	**0.304**	g
Sms =	**0.778**	g
Sm1 =	**0.608**	
Sds =	**0.519**	g
Sd1 =	**0.405**	
Fa =	**1.333**	g
Fv =	**2**	
Tl =	**16**	
Ts =	**0.78**	
Ta =	**0.1**	
PGA =	**0.264**	g
PGAm =	**0.353**	g
Fpga =	**1.336**	g
Ie =	**1**	g
Cv =	**1.089**	g
SDC =	**D**	Tables 11.6-1 and 11.6-2
Steel Ordinary Moment Frames		Table 12.2-1
R =	**3.5**	
Omega-o =	**3**	
Cd =	**3**	Tables 11.6-1 and 11.6-2
Cs =	**0.15**	Ta<Ts -> Use Eqn. 12.8-2 per 11.8.4
Steel Ordinary Concentrically Braced Frame		Table 12.2-1
R =	**3.25**	
Omega-o =	**2**	
Cd =	**3.25**	Tables 11.6-1 and 11.6-2
Cs =	**0.16**	

Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	0.584	S_{D1} :	N/A
S_1 :	0.304	T_L :	16
F_a :	1.333	PGA :	0.264
F_v :	N/A	PGA_M :	0.353
S_{MS} :	0.778	F_{PGA} :	1.336
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.519	C_v :	1.089

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Mon Feb 24 2020

Date Source: USGS Seismic Design Maps

Snow Loads

pf =	40	psf
Is =	1	
Ce =	1	Table 7.3-1
Ct =	1	Table 7-3.2
pg = pf/(.7*Ce*Ct*Is) =	57.14	psf

This is a prescribed "case-study" area per ASCE 7-16. Roof snow load was given by the coutny. This can be considered a "case-study" for purposes of design. Ground snow load was back-calculated assuming exposure and temperature coefficients of 1.0.

Snow

Results:

Elevation:	2504.8 ft
Data Source:	ASCE/SEI 7-16, Table 7.2-8
Date Accessed:	Mon Feb 24 2020

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Load Combinations

As described previously, the following load effects will be considered:

Label	Description
D	Dead
L	Live
W	Wind
E	Seismic
S	Snow
H	Earth
Hs	Hydrostatic

The following load combinations will be considered for all structures per the intent of ASCE 7-16

Combo	Type	γD	γL	γW	γE	γS	γH*	γHs	
1	Basic	1.4	-	-	-	-	1.6/0.9	1.4	
2	Basic	1.2	1.6	-	-	0.5	1.6/0.9	1.2	
3a	Basic	1.2	1	-	-	1.6	1.6/0.9	1.2	
3b	Basic	1.2	-	0.5	-	1.6	1.6/0.9	1.2	
4	Basic	1.2	1	1	-	0.5	1.6/0.9	1.2	
5	Basic	0.9	-	1	-	-	1.6/0.9	-	
6	Seismic	1.2	1	-	1	0.2	1.6/0.9	1.2	
7	Seismic	0.9	-	-	1	-	1.6/0.9	0.9	

Design Basis

Concrete

The required strength of reinforced concrete elements will be determined in accordance with ACI 318-14. Structural elements will satisfy Load Factor and Resistance Design methodology based on the equation below:

$$\sum \gamma_i L_{ni} \leq \varphi R_n$$

where:

γ_i = ASCE 7-16 load factors

L_{ni} = loads

Φ = resistance factor from ACI 318

R_n = nominal resistance from ACI 318

Steel

The required strength of structural steel elements will be determined in accordance with AISC 360-16. Structural elements will satisfy Load Factor and Resistance Design methodology based on the equation below:

$$U = \sum \gamma_i L_{ni} \leq \alpha \phi R_n$$

where:

U = required strength

γ_i = ASCE 7-16 load factors

L_{ni} = loads

α = 1.0 for non-hydraulic structures, 0.9 for hydraulic structures

Φ = resistance factor from AISC

R_n = nominal resistance from AISC

SUBJECT: KRRC
Fall Creek Fish Hatchery
Structural Calculations

BY: Zachary Autin **CHK'D BY:** Taylor Bowen
DATE: 10/19/2020
PROJECT NO.: 20-024

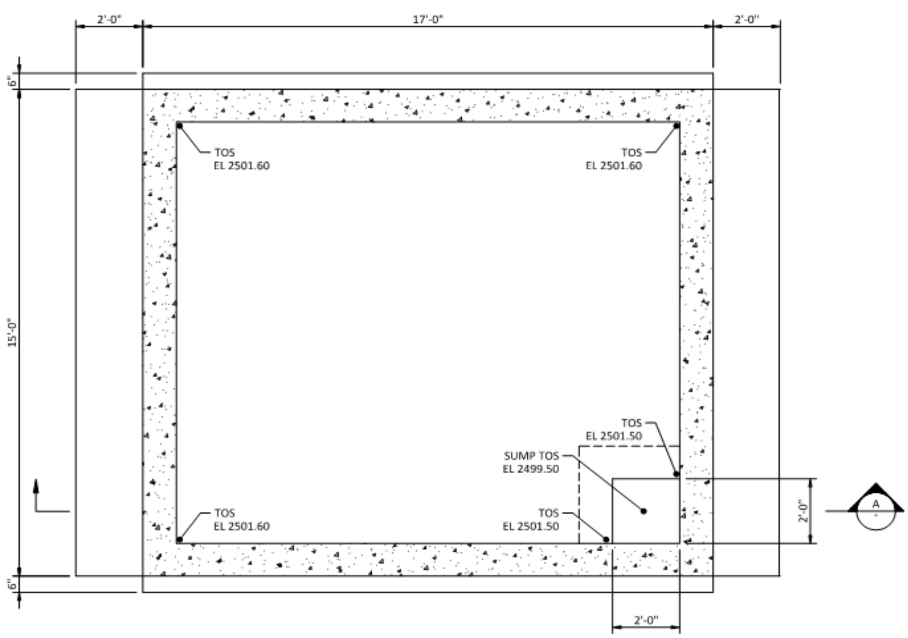
Purpose

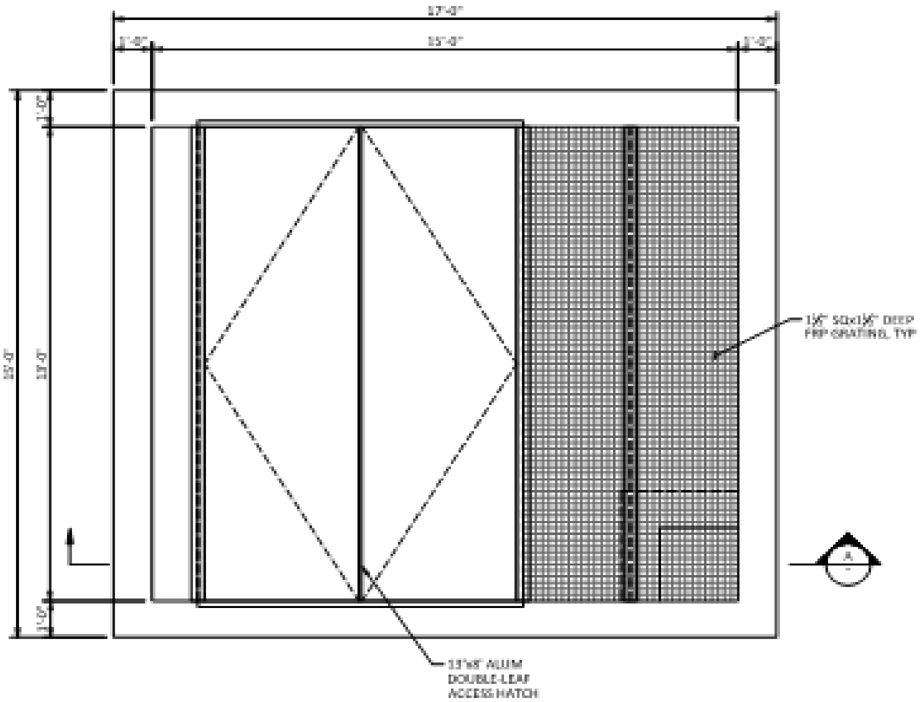
Design of the CIP concrete meter vault.

Information

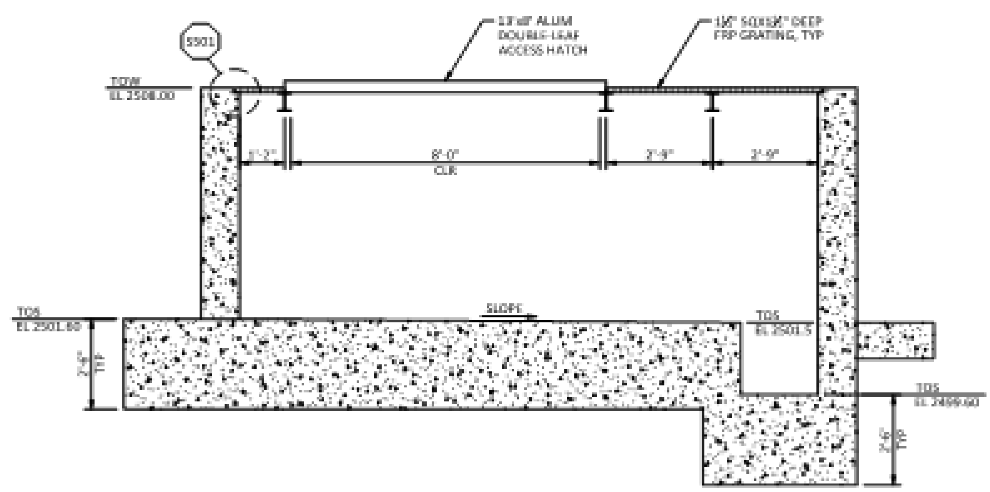
gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient

Figures





TOP PLAN
SCALE: 1/2"= 1'-0"



SECTION
SCALE: 1/2"= 1'-0"

Calculations: Buoyancy - Extreme

EL_top =	2508.00	ft	Elevation top of meter vault	
EL_tos =	2501.60	ft	Elevation top of slab	
EL_w =	2508.00	ft	Elevation of ground water	
t_slab =	1.75	ft	Thickness of slab	
EL_sump =	2499.60	ft	Elevation of top of sump slab	
t_walls =	1.00	ft	Thickness of walls	
t_roof =	0.83	ft	Thickness of roof slab	
B =	17.00	ft	Width	
L =	15.00	ft	Length	

Volumes

V_c =	1028.88	cf	Volume of concrete	
V_mv =	2198.25	cf	Volume of water displaced	
Fb =	137.17	kips	Buoyancy force	
Wc =	154.33	kips	Weight of concrete	
FOS =	1.13		Factor of Safety for Flotation	
CHECK GOOD			Check if FOS >/= 1.1	USACE EM 1110-2-2100 Section 3-8

3-8. Factors of Safety for Flotation

A factor of safety is required for flotation to provide a suitable margin of safety between the loads that can cause instability and the weights of materials that resist flotation. The flotation factor of safety is defined by equation 3-2. The required factors of safety for *flotation* are presented in Table 3-4. These flotation safety factors apply to both *normal* and *critical* structures and for all site information categories.

$$FS_f = \frac{W_S + W_C + S}{U - W_G} \tag{3-2}$$

where

W_S = weight of the structure, including weights of the fixed equipment and soil above the top surface of the structure. The moist or saturated unit weight should be used for soil above the groundwater table and the submerged unit weight should be used for soil below the groundwater table.
W_C = weight of the water contained within the structure
S = surcharge loads
U = uplift forces acting on the base of the structure
W_G = weight of water above top surface of the structure.

Table 3-4 Required Factors of Safety for Flotation – All Structures

	Load Condition Categories		
Site Information Category	**Usual**	**Unusual**	**Extreme**
All Categories	1.3	1.2	1.1

Calculations: Buoyancy - Usual

EL_top =	2508.00	ft	Elevation top of meter vault	
EL_tos =	2501.60	ft	Elevation top of slab	
EL_w =	2504.50	ft	Elevation of ground water	
t_slab =	1.17	ft	Thickness of slab	
EL_sump =	2499.60	ft	Elevation of top of sump slab	
t_walls =	1.00	ft	Thickness of walls	
t_roof =	0.83	ft	Thickness of roof slab	
B =	17.00	ft	Width	
L =	15.00	ft	Length	

Volumes

V_c =	880.13	cf	Volume of concrete	
V_mv =	1157.00	cf	Volume of water displaced	
Fb =	72.20	kips	Buoyancy force	
Wc =	132.02	kips	Weight of concrete	
FOS =	1.83		Factor of Safety for Flotation	
CHECK GOOD			Check if FOS >/= 1.3	USACE EM 1110-2-2100 Section 3-8

SUBJECT:	KRRC	**BY:** Zachary Autin	**CHK'D BY:**	Taylor Bowen
	Fall Creek Fish Hatchery	**DATE:** 10/19/2020		
	Structural Calculations	**PROJECT NO.:** 20-024		

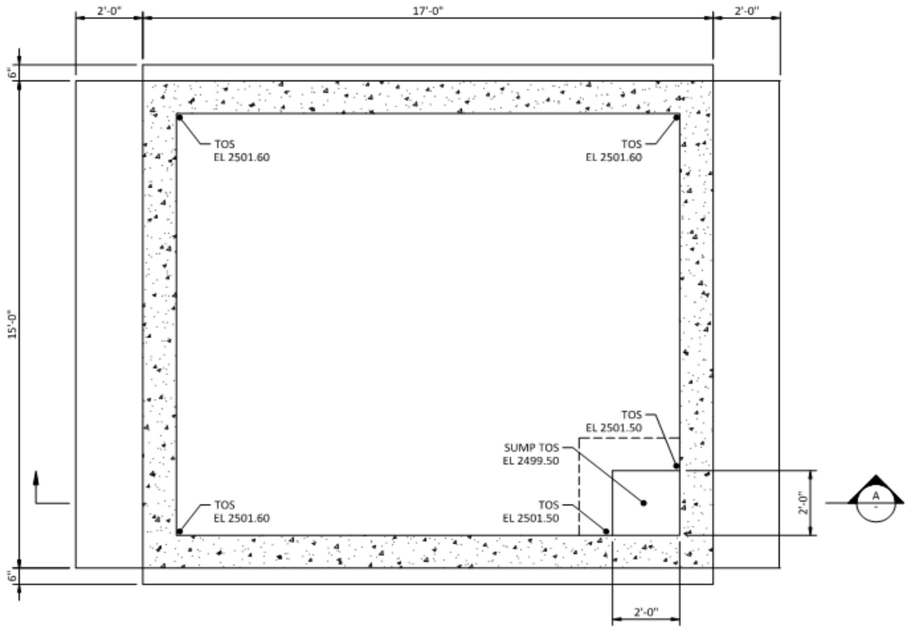
Purpose

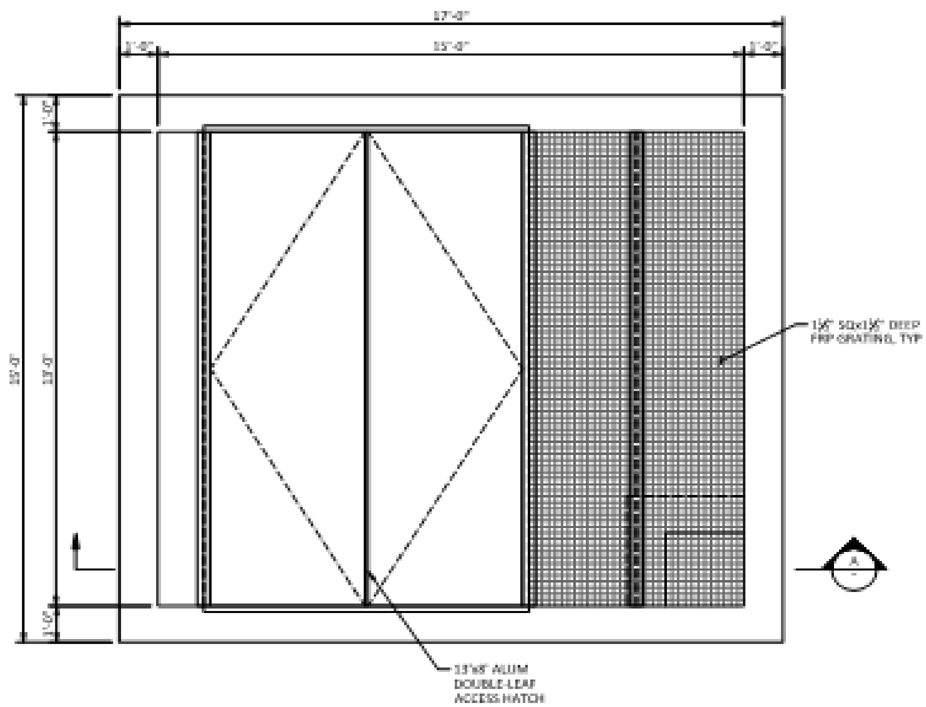
Design the walls and slab for the new meter vault.

Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

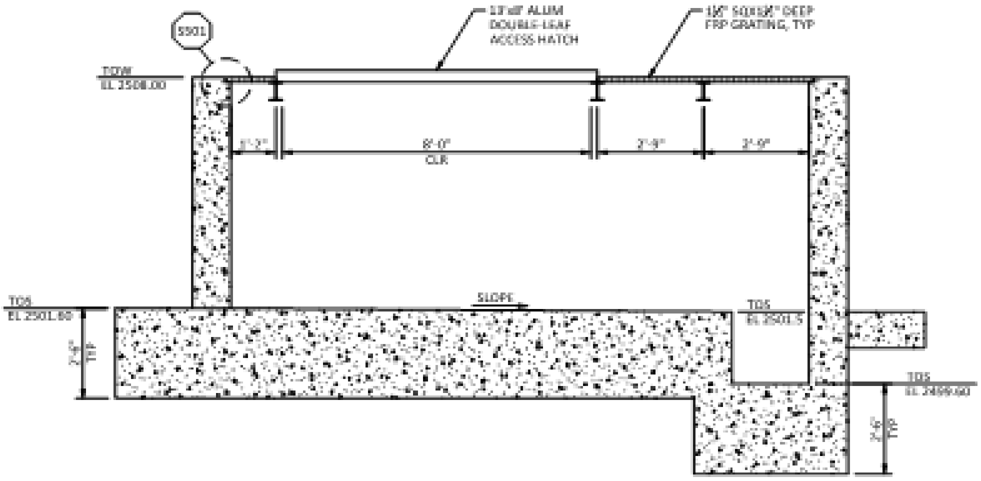
Figures





TOP PLAN

SCALE: 1/2"= 1'-0"



SECTION

SCALE: 1/2" = 1'-0"

Calculations: Loads

Design walls as cantilever from the slab due to span > height of wall

Usual Load Case - Construction - No water in intake

	EL_ts =	2501.50	ft	Elevation top of slab
	EL_twl =	2508.00	ft	Elevation top of wall
	EL_sd =	2508.00	ft	Elevation top of soil - driving side
	EL_wd =	2508.00	ft	Elevation top of water - driving side
	t_slab =	12.00	in	Thickness of slab
	t_slab =	1.00	ft	Thickness of slab
	EL_bs =	2500.50	ft	Elevation bottom of slab
Lateral Earth Pressure - driving				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	203.45	psf	Soil pressure top of slab
	P4 =	234.75	psf	Soil pressure bottom of slab
	Fh =	0.66	k	Resultant force (wall only)
	y_h =	2.67	ft	Distance of resultant from center of slab
	M_h =	1.76	k-ft	Max moment in wall
Seismic Earth Pressure				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	284.38	psf	Soil pressure top of slab
	P4 =	328.13	psf	Soil pressure bottom of slab
	Fe =	0.92	k	Resultant force (wall only)
	y_e =	2.67	ft	Distance of resultant from center of slab
	M_e =	2.46	k-ft	Max moment in wall
Snow Load Surcharge Pressure				
	pg =	57.14	psf	Snow load surcharge
	Ps =	28.57	psf	Lateral snow pressure
	Fe =	0.19	k	Resultant force (wall only)
	y_e =	3.75	ft	Distance of resultant from center of slab
	M_e =	0.70	k-ft	Max moment in wall
Live Load Surcharge Pressure				
	pg =	250.00	psf	Live load surcharge
	Ps =	125.00	psf	Lateral live pressure
	Fe =	0.81	k	Resultant force (wall only)
	y_e =	3.75	ft	Distance of resultant from center of slab
	M_e =	3.05	k-ft	Max moment in wall
Hydrostatic Pressure				
	P1 =	0.00	psf	pressure top of wall
	P2 =	0.00	psf	pressure top of soil
	P3 =	405.60	psf	pressure top of slab
	P4 =	468.00	psf	pressure bottom of slab
	Fe =	1.32	k	Resultant force (wall only)
	y_e =	2.67	ft	Distance of resultant from center of slab
	M_e =	3.52	k-ft	Max moment in wall
Flexure				
	LC2 =	12.26	k-ft	Load combination 3a (see design criteria)
	LC3a =	11.20	k-ft	Load combination 3a (see design criteria)
	LC6 =	12.69	k-ft	Load combination 6 (see design criteria)
	Mmax_f =	12.69	k-ft/ft	Maximum factored moment in wall
Shear				
	LC2 =	4.03	k	Load combination 3a (see design criteria)
	LC3a =	3.75	k	Load combination 3a (see design criteria)
	LC6 =	4.41	k	Load combination 6 (see design criteria)
	Vmax_f =	4.41	k	Maximum factored shear in wall

Calculations: Flexure

Twall =	12.00	in	Wall thickness
size bar =	6.00		Bar size
dbar =	0.75	in	Diameter of bar
Cover =	2.00	in	Bar cover (center reinforcement)
d = Twall - cover - dbar*0.5 =	9.63	in	Depth to tension reinforcement
Spacing =	12.00	in	Spacing of bars
Abar =	0.44	in2	Area of 1 bar
As =	0.44	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.03		Balanced % steel
rho-max =	0.020		Max % steel
As,max =	2.35	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.35	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.58	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	247.48	k-in	Nominal Moment
Mn =	20.62	k-ft	
Phi*Mn =	18.56	k-ft	
Mmax_f =	12.69	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.68*

Calculations: Longitudinal Steel

rho-min =	0.0060		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.86	in2/ft	Min area of steel
size bar =	6.00		Bar size
dbar =	0.75	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.44	in2	Area of 1 bar
As =	0.44	in2/ft	Area of flexural steel

Calculations: Longitudinal Steel in Slab

Tslab =	21.0000	in	Thickness of slab
rho-min =	0.0040		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	1.01	in^2/ft	Min area of steel
size bar =	7.00		Bar size
dbar =	0.88	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.60	in2	Area of 1 bar
As =	0.60	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	15.50	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	11.62	k/ft	Ultimate shear strength
Vmax_f =	4.41	k/ft	
CHECK	GOOD		*D/C Ratio = 0.38*

SUBJECT:	KRRC
	Fall Creek Fish Hatchery
	Structural Calculations

BY:	Zachary Autin	**CHK'D BY:**	Taylor Bowen
DATE:	10/19/2020		
PROJECT NO.:	20-024		

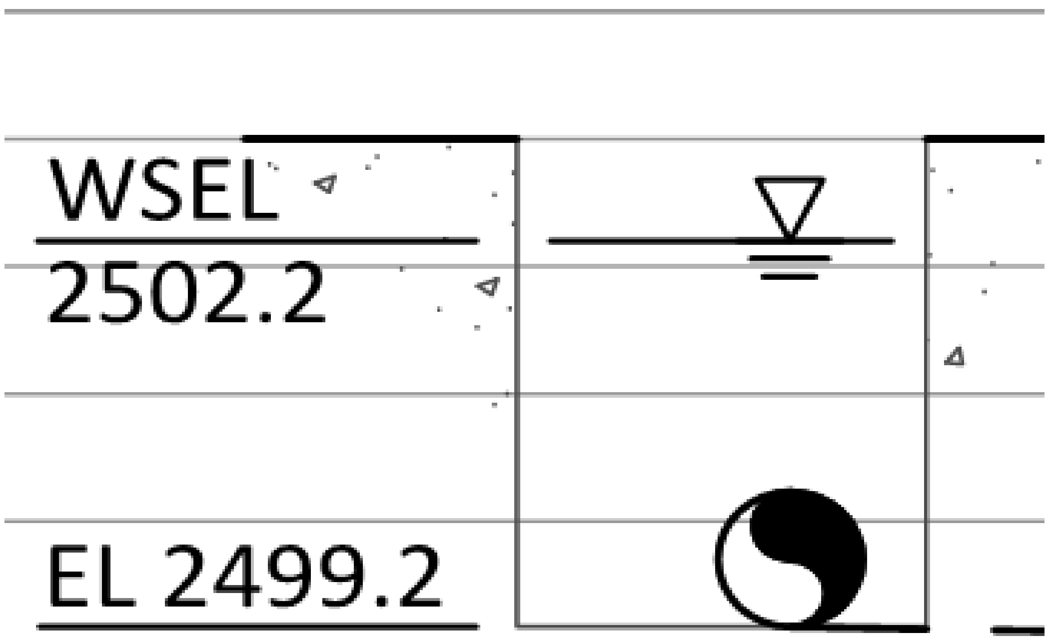
Purpose

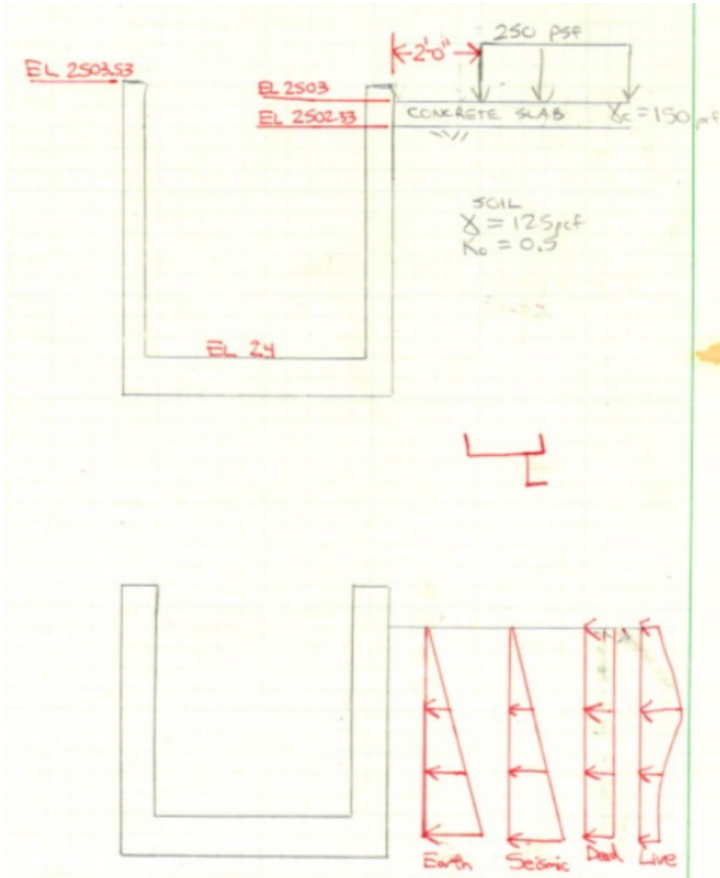
Design the walls for the rearing ponds

Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'_ex =	2.50	ksi	Compressive strength
fy,bar_ex =	33.00	ksi	Yield Strength of steel reinforcement
fu,bar_ex =	55.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
t_slab =	8.00	in	Thickness of slab
LL_surcharge	250.00	psf	Live load surcharge

Figures





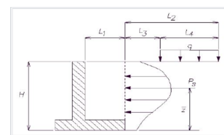
EL 2503.53
EL 2503
EL 2502.53
2'-0"
250 psf
CONCRETE SLAB
γc = 150 pcf
SOIL
γ = 125 pcf
Ko = 0.5
EL 24
Earth
Seismic
Dead
Live

McMILLEN JACOBS ASSOCIATES

Calculations: Loads

	EL_bot =	2499.20	ft	Elevation top of slab
	EL_top =	2503.53	ft	Elevation top of wall
	EL_w =	2502.00	ft	Diameter of bar
	EL_soil =	2502.33	ft	Elevation top of soil
	EL_c =	2503.00	ft	Elevation top of driveway
	EL_fix =	2498.78	ft	Elevation center of slab
Lateral Earth Pressure				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of slab
	P3 =	0.00	psf	Soil pressure top of soil
	P4 =	195.83	psf	Soil pressure bottom of soil
	Fh =	0.31	k	Resultant force
	y_h =	1.46	ft	Distance of resultant from base
	M_h =	0.45	k-ft	Max moment in wall
Seismic Earth Pressure				
	P1 =	0.00	psf	Seismic earth pressure top of wall
	P2 =	0.00	psf	Seismic earth pressure top of slab
	P3 =	0.00	psf	Seismic earth pressure top of soil
	P4 =	137.08	psf	Seismic earth pressure bottom of soil
	Fe =	0.21	k	Resultant force
	y_e =	1.46	ft	Distance of resultant from base
	M_e =	0.31	k-ft	Max moment in wall
Lateral Dead Load Pressure				
	P1 =	0.00	psf	Concrete slab pressure top of wall
	P2 =	0.00	psf	Concrete slab pressure top of slab
	P3 =	50.00	psf	Concrete slab pressure top of soil
	P4 =	50.00	psf	Concrete slab pressure bottom of soil
	Fd =	0.16	k	Resultant force
	y_d =	1.98	ft	Distance of resultant from base
	M_d =	0.31	k-ft	Max moment in wall
Live Load Surcharge Pressure				
	q =	250.00	psf	Live load surcharge
	L1 =	0.00	ft	
	L2 =	19.50	ft	
	L3 =	4.50	ft	
	L4 =	15.00	ft	
	H =	3.13	ft	Height of wall
	theta-1 =	55.15	degrees	
	theta-2 =	80.87	degrees	
	Ps =	0.22	kips	Resultant force
	R =	3471.10		
	Q =	705.70		
	z_bar =	1.46	ft	Distance of resultant from base
	M_l =	0.33	k-ft	Max moment in wall

https://epg.modot.org/index.php/751.24_LFD_Retaining_Walls



Surcharge Pressure on Retaining Wall

From the figure:

$P_s = \frac{q}{90}\left[H(\theta_2 - \theta_1)\right]$ where

$\theta_1 = \arctan\left[\frac{L_3}{H}\right]$ and $\theta_2 = \arctan\left[\frac{L_2}{H}\right]$

$\bar{z} = \frac{H^2(\theta_2 - \theta_1) - (R - Q) + 57.03 L_4 H}{2H(\theta_2 - \theta_1)}$ where

$(L_2)^2(90^\circ - \theta_2)$ and $Q = (L_3)^2(90^\circ - \theta_1)$

Calculations: Load Combinations

Flexure				
	LC1 =	1.15	k-ft	Load combination 1 (see design criteria)
	LC2 =	1.61	k-ft	Load combination 2 (see design criteria)
	LC6 =	1.73	k-ft	Load combination 6 (see design criteria)
	Mmax_f =	1.73	k-ft/ft	Maximum factored moment in wall
Shear				
	LC1 =	0.71	k	Load combination 1 (see design criteria)
	LC2 =	1.04	k	Load combination 2 (see design criteria)
	LC6 =	1.12	k	Load combination 6 (see design criteria)
	Vmax_f =	1.12	k	Maximum factored shear in wall

Calculations: Wall Design

Calculations: Flexure

Twall =	8.00	in	Wall thickness
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Cover =	N/A	in	Bar cover (center reinforcement)
d = Twall/2 - dbar*0.5 =	3.69	in	Depth to tension reinforcement
Spacing =	18.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.20	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.04		Balanced % steel
rho-max =	0.025		Max % steel
As,max =	1.11	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (Table 7.12.2.1)
As,min =	0.13	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.26	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	24.00	k-in	Nominal Moment
Mn =	2.00	k-ft	
Phi*Mn =	1.80	k-ft	
Mmax_f =	1.73	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.96*

Calculations: Longitudinal Steel

rho-min =	0.006		Min % steel (Table 7.12.2.1)
As,min =	0.58	in2/ft	Min area of steel
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	4.43	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	3.32	k/ft	Ultimate shear strength
Vmax_f =	1.12	k/ft	
CHECK	GOOD		*D/C Ratio = 0.34*

SUBJECT: KRRC
Fall Creek Fish Hatchery
Structural Calculations

BY: Zachary Autin **CHK'D BY:** Taylor Bowen
DATE: 10/19/2020
PROJECT NO.: 20-024

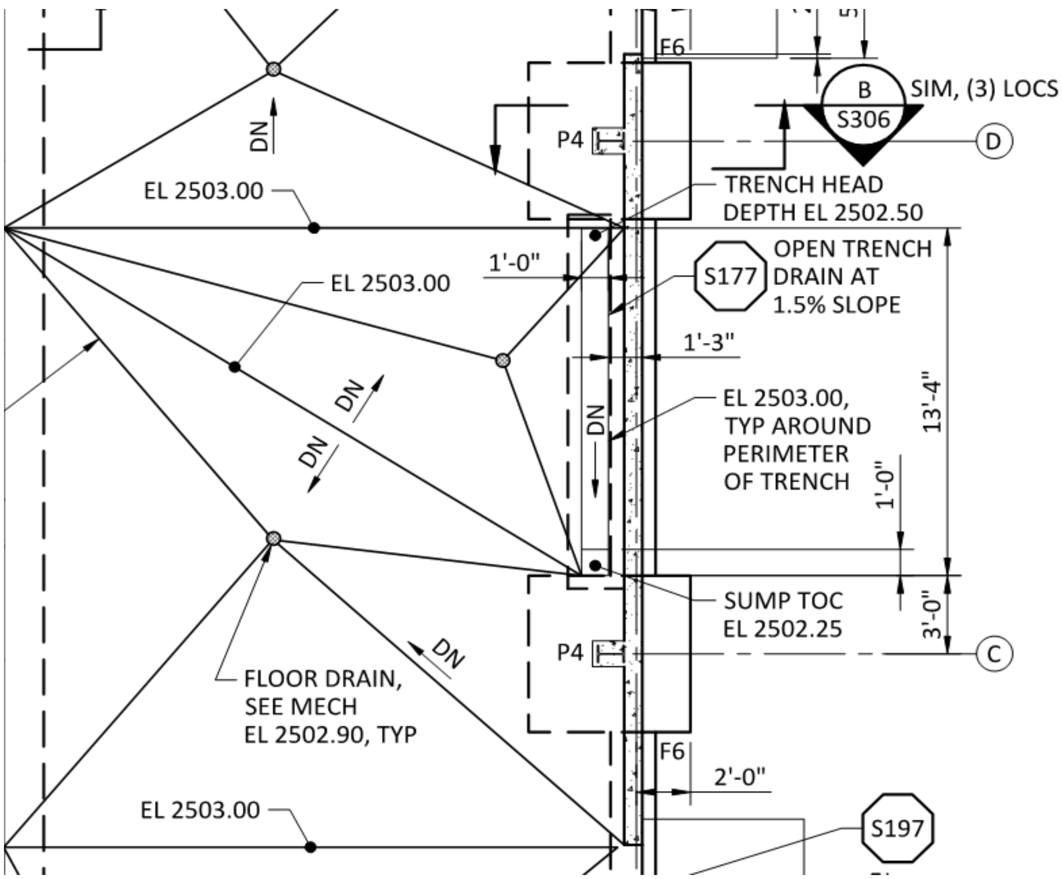
Purpose

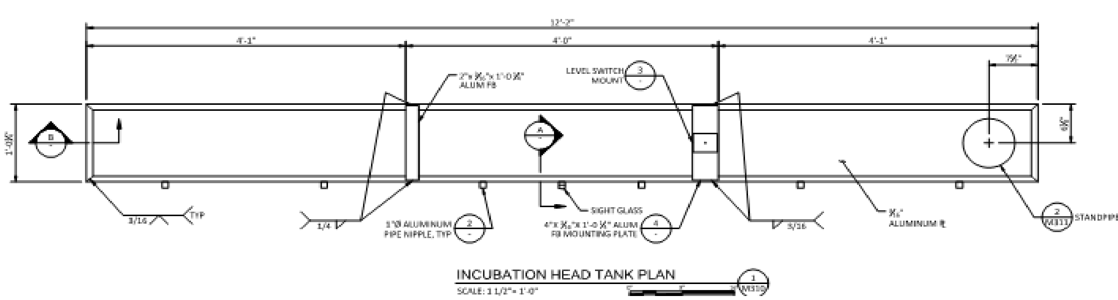
Design the support frame for the coho building head tank for the incubation stacks.

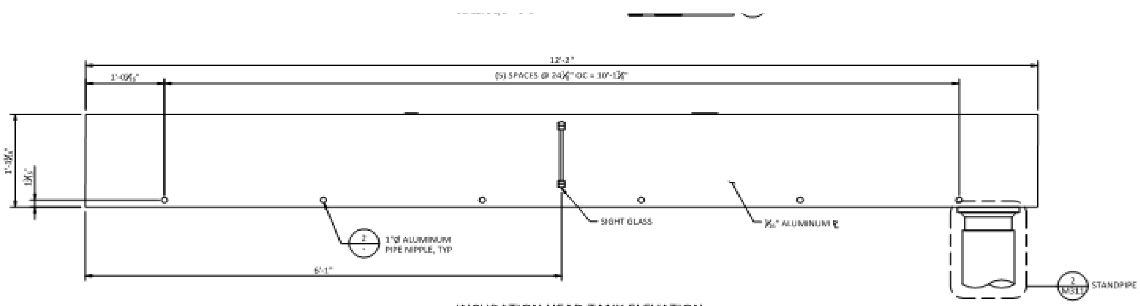
Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Fy =	50.00	ksi	Yield strength of wide flange
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

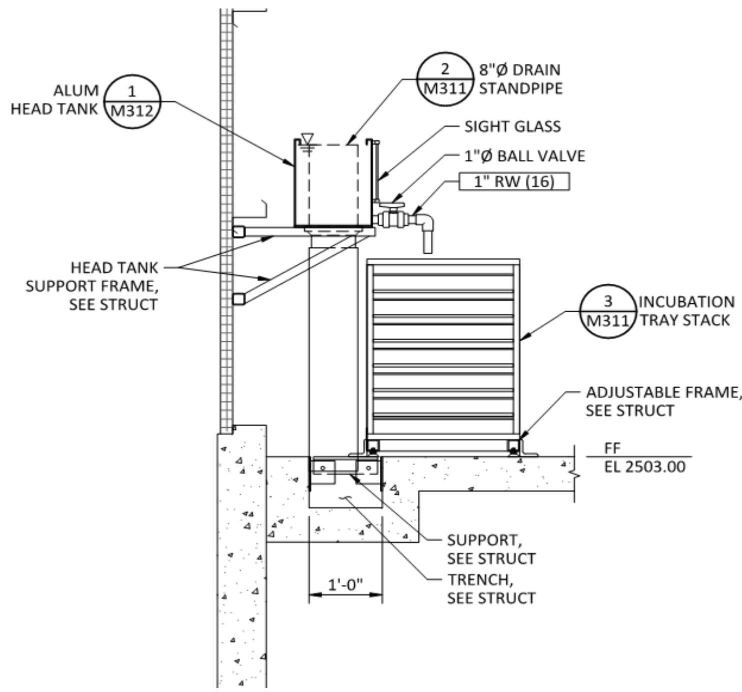
Figures

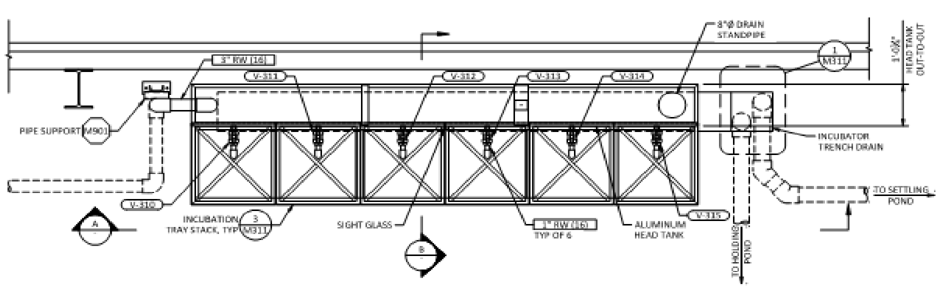




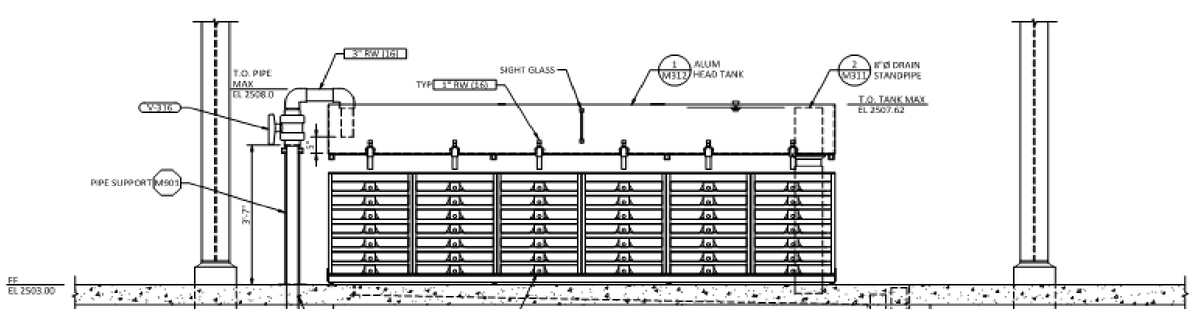


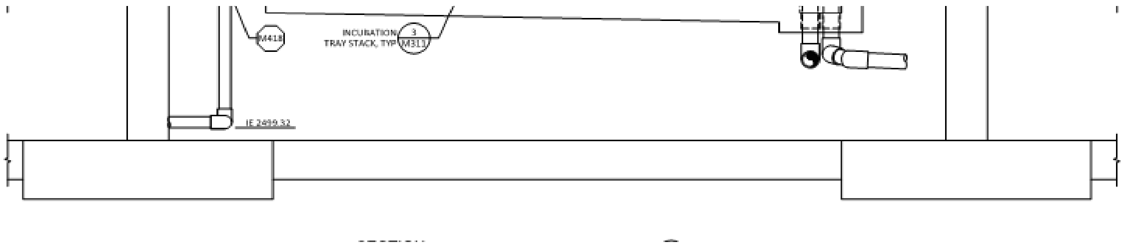
INCUBATION HEAD TANK ELEVATION
SCALE: 1 1/2"= 1'-0"





HEAD TANK PLAN
SCALE: 3/4"= 1'-0"




INCUBATION
TRAY STACK, TYP
3
M311
M418
IE 2499.32

Calculations: Head Tank Support Frame

Design beam spanning between columns (NO LONGER USED)

Wa =	268.30	lbs	Weight of tank empty
Vtank =	16.36	cf	Volume of tank
Ww =	1020.92	lbs	Weight of water in tank
Wtank =	1289.22	lbs	Unfactored weight of tank
Wtank_f =	1804.90	lbs	Factored weight of tank
w =	148.348	lbs/ft	Factored weight of tank per foot
L =	9.67	ft	Span btw columns D5 and C5
Mmax =	1.733	k-ft	Max moment

Try HSS4X4X1/4

phi*Mn =	7.44	k-ft	
CHECK	GOOD		

Design cantilever beams

Wa =	268.30	lbs	Weight of tank empty
Vtank =	16.36	cf	Volume of tank
Ww =	1020.92	lbs	Weight of water in tank
Wtank =	1289.22	lbs	Unfactored weight of tank
Wtank_f =	1804.90	lbs	Factored weight of tank
w =	148.348	lbs/ft	Factored weight of tank per foot
B =	2.92	ft	Trib width
P =	432.682	lbs	Factored weight of tank per foot
L =	2.25	ft	Length of cantilever
Mmax =	0.974	k-ft	Max moment

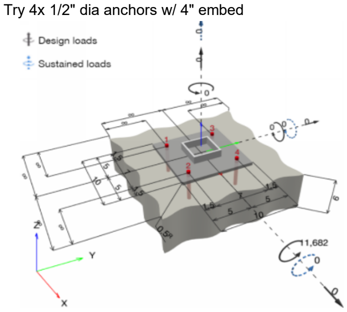
Try HSS4X4X1/4

phi*Mn =	7.44	k-ft	
CHECK	GOOD		

Design base plate

Mmax =	0.974	k-ft	Max moment
Mmax =	11682.420	lb-in	Max moment
B =	7.00	in	Spacing of anchors
T =	1.67	kips	Tension distributed to 2 anchors
Ta =	0.83	kips	Tension in 1 anchors

Try 4x 1/2" dia anchors w/ 4" embed



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	$N = 0; V_x = 0; V_y = 0;$ $M_x = 11{,}682; M_y = 0; M_z = 0;$ $N_{sus} = 0; M_{x,sus} = 0; M_{y,sus} = 0;$	no	18

SUBJECT:	KRRC	**BY:** Zachary Autin	**CHK'D BY:** Taylor Bowen
	Fall Creek Fish Hatchery	**DATE:** 10/19/2020	
	Structural Calculations	**PROJECT NO.:** 20-024	

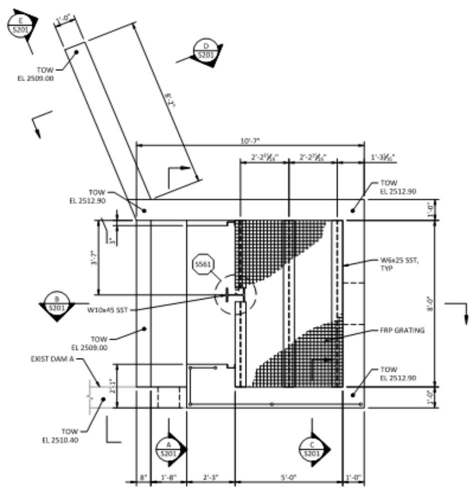
Purpose

Design the walls and slab for the new intake structure. Check stability.

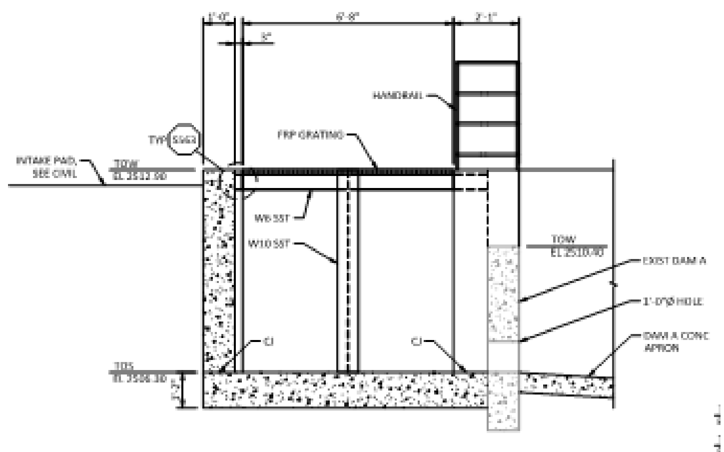
Information

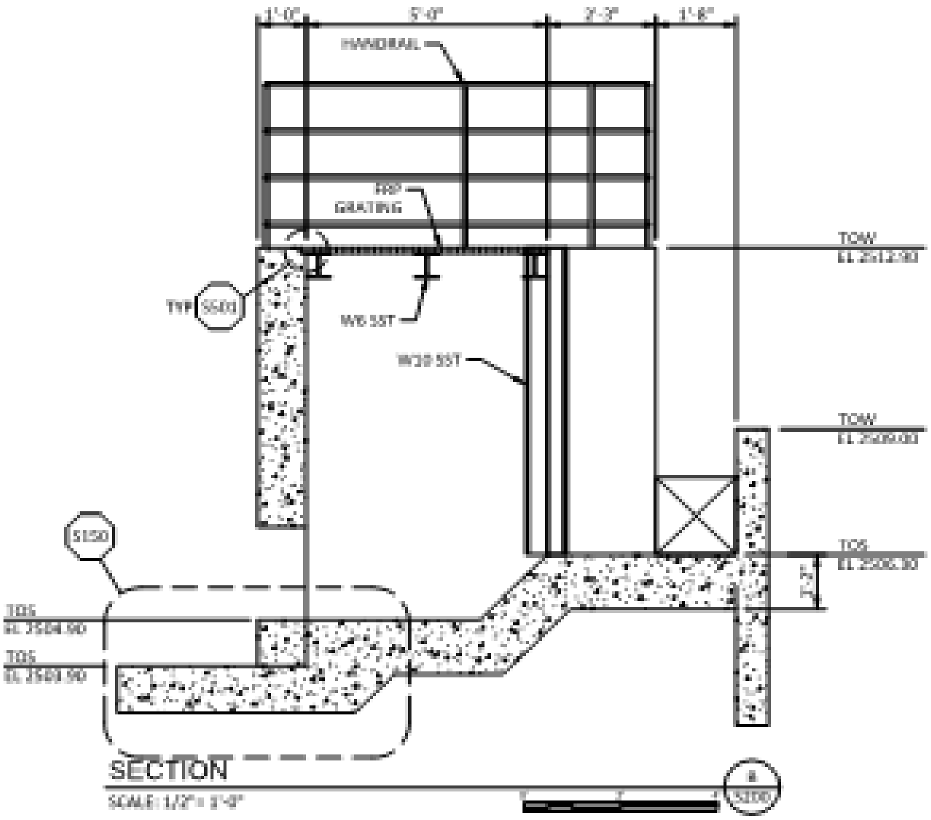
gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

Figures



INTAKE TOP PLAN
SCALE: 1/2"= 1'-0"




1'-0"
5'-0"
2'-3"
1'-8"
HANDRAIL
FRP
GRATING
TYP
TOW
TOW
S150
TOS
TOS
TOS
SECTION

Calculations: Loads

Design walls as cantilever from the slab due to span > height of wall

Usual Load Case - Construction - No water in intake

	EL_ts =	2504.90	ft	Elevation top of slab
	EL_twl =	2512.90	ft	Elevation top of wall
	EL_sd =	2512.40	ft	Elevation top of soil - driving side
	t_slab =	12.00	in	Thickness of slab
	t_slab =	1.00	ft	Thickness of slab
	EL_bs =	2503.90	ft	Elevation bottom of slab
Lateral Earth Pressure - driving				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	468.75	psf	Soil pressure top of slab
	P4 =	531.25	psf	Soil pressure bottom of slab
	Fh =	1.76	k	Resultant force (wall only)
	y_h =	3.00	ft	Distance of resultant from center of slab
	M_h =	5.27	k-ft	Max moment in wall
Seismic Earth Pressure				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	328.13	psf	Soil pressure top of slab
	P4 =	371.88	psf	Soil pressure bottom of slab
	Fe =	1.23	k	Resultant force (wall only)
	y_e =	3.00	ft	Distance of resultant from center of slab
	M_e =	3.69	k-ft	Max moment in wall
Snow Load Surcharge Pressure				
	pg =	57.14	psf	Snow load surcharge
	Ps =	28.57	psf	Lateral snow pressure
	Fe =	0.21	k	Resultant force (wall only)
	y_e =	4.25	ft	Distance of resultant from center of slab
	M_e =	0.91	k-ft	Max moment in wall
Flexure				
	LC3a =	9.89	k-ft	Load combination 3a (see design criteria)
	LC6 =	12.31	k-ft	Load combination 6 (see design criteria)
	Mmax_f =	12.31	k-ft/ft	Maximum factored moment in wall
Shear				
	LC3a =	3.16	k-ft	Load combination 3a (see design criteria)
	LC6 =	4.09	k-ft	Load combination 6 (see design criteria)
	Vmax_f =	4.09	k	Maximum factored shear in wall

Calculations: Flexure

Twall =	10.00	in	Wall thickness
size bar =	6.00		Bar size
dbar =	0.75	in	Diameter of bar
Cover =	2.00	in	Bar cover (center reinforcement)
d = Twall - cover - dbar*0.5 =	7.63	in	Depth to tension reinforcement
Spacing =	12.00	in	Spacing of bars
Abar =	0.44	in2	Area of 1 bar
As =	0.44	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.03		Balanced % steel
rho-max =	0.020		Max % steel
As,max =	1.86	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.27	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.58	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	194.46	k-in	Nominal Moment
Mn =	16.21	k-ft	
Phi*Mn =	14.58	k-ft	
Mmax_f =	12.31	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.84*

Calculations: Longitudinal Steel

rho-min =	0.0050		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.60	in2/ft	Min area of steel
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	12.28	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	9.21	k/ft	Ultimate shear strength
Vmax_f =	4.09	k/ft	
CHECK	GOOD		*D/C Ratio = 0.44*

Calculations: Stability

Sliding, Overturning, Bearing Pressure OK by inspection - Intake structure ties into much larger dam A

Vw =	182.50	cf	Wall volume
Vs =	173.87	cf	Slab volume
W =	53.46	kips	Weight of Intake Concrete

Flotation

Hd =	3.00	ft	Head differential across travelling screens (From Mechanical)
Vd =	282.67	cf	Displaced volume
Fb =	17.64	kips	Buoyant force
FOS =	3.03		Factor of Safety
FOS_req =	1.30		Factor of safety required (EM 1110-2-2100)
CHECK	GOOD		

SUBJECT: KRRC
Fall Creek Fish Hatchery
Structural Calculations

BY: Zachary Autin **CHK'D BY:** Taylor Bowen
DATE: 10/19/2020
PROJECT NO.: 20-024

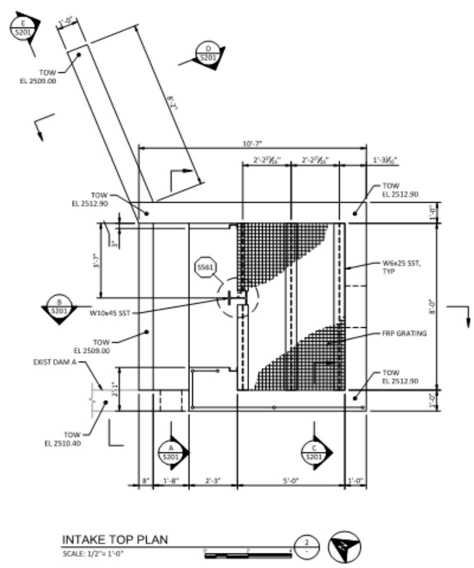
Purpose

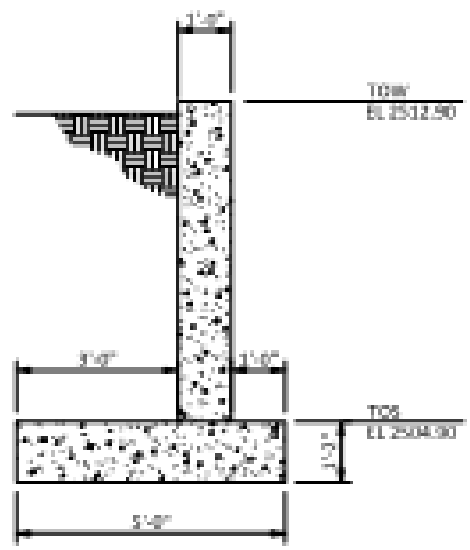
Design the worst case wing wall section for soil loads. Check stability.

Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

Figures





Calculations: Loads

Design as a standalone retaining wall structure. Ignore tie-in to intake structure.

Usual Load Case - There is no water differential across the wing wall at any time because this wall is upstream of the cutoff wall

	EL_ts =	2506.30	ft	Elevation top of slab
	EL_twl =	2512.90	ft	Elevation top of wall
	EL_sd =	2511.00	ft	Elevation top of soil - driving side
	EL_sr =	2507.00	ft	Elevation top of soil - resisting side
	t_slab =	12.00	in	Thickness of slab
	t_slab =	1.00	ft	Thickness of slab
	EL_bs =	2505.30	ft	Elevation bottom of slab
Lateral Earth Pressure - driving				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	293.75	psf	Soil pressure top of slab
	P4 =	356.25	psf	Soil pressure bottom of slab
Wall only				
	Fh =	0.69	k	Resultant force (wall only)
	y_h =	2.07	ft	Distance of resultant from center of slab
	M_h =	1.43	k-ft	Max moment in wall
Stability				
	Fh =	1.02	k	Resultant force
	y_h =	1.90	ft	Distance of resultant from base
	M_h =	1.93	k-ft	Overturning moment
Lateral Earth Pressure - resisting				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	43.75	psf	Soil pressure top of slab
	P4 =	106.25	psf	Soil pressure bottom of slab
Wall only				
	Fh =	0.02	k	Resultant force (wall only)
	y_h =	0.73	ft	Distance of resultant from center of slab
	M_h =	0.01	k-ft	Max moment in wall
Stability				
	Fh =	0.09	k	Resultant force
	y_h =	0.57	ft	Distance of resultant from base
	M_h =	0.05	k-ft	Overturning moment
Seismic Earth Pressure				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	205.62	psf	Soil pressure top of slab
	P4 =	249.37	psf	Soil pressure bottom of slab
Wall only				
	Fe =	0.48	k	Resultant force (wall only)
	y_e =	2.07	ft	Distance of resultant from center of slab
	M_e =	1.00	k-ft	Max moment in wall
Stability				
	Fe =	0.71	k	Resultant force
	y_e =	1.90	ft	Distance of resultant from base
	M_e =	1.35	k-ft	Overturning moment
Snow Load Surcharge Pressure				
	pg =	57.14	psf	Snow load surcharge
	Ps =	28.57	psf	Lateral snow pressure
Wall only				
	Fe =	0.13	k	Resultant force (wall only)
	y_e =	2.85	ft	Distance of resultant from center of slab
	M_e =	0.38	k-ft	Max moment in wall
Stability				
	Fe =	0.16	k	Resultant force
	y_e =	2.85	ft	Distance of resultant from base
	M_e =	0.46	k-ft	Overturning moment
Flexure				
	LC3a =	2.88	k-ft	Load combination 3a (see design criteria)
	LC6 =	3.35	k-ft	Load combination 6 (see design criteria)
	Mmax_f =	3.35	k-ft/ft	Maximum factored moment in wall
Shear				
	LC3a =	1.31	k	Load combination 3a (see design criteria)
	LC6 =	1.60	k	Load combination 6 (see design criteria)
	Vmax_f =	1.60	k	Maximum factored shear in wall

Calculations: Flexure

Twall =	10.00	in	Wall thickness
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Cover =	2.00	in	Bar cover (center reinforcement)
d = Twall - cover - dbar*0.5 =	7.69	in	Depth to tension reinforcement
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.03		Balanced % steel
rho-max =	0.020		Max % steel
As,max =	1.88	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.28	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.40	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	137.82	k-in	Nominal Moment
Mn =	11.48	k-ft	
Phi*Mn =	10.34	k-ft	
Mmax_f =	3.35	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.32*

Calculations: Longitudinal Steel

rho-min =	0.0050		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.60	in2/ft	Min area of steel
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	12.38	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	9.28	k/ft	Ultimate shear strength
Vmax_f =	1.60	k/ft	
CHECK	GOOD		*D/C Ratio = 0.17*

Calculations: Stability

Usual Load Case - water EL 2510.4 (see hydraulic profile)

tw =	0.83	ft	Wall thickness
ts =	1.00	ft	Slab thickness
A =	7.00	ft	Heel length
B =	1.00	ft	Toe length
Hw =	6.60	ft	Height of wall
Hsd =	4.70	ft	Height of soil - driving
Hsr =	0.70	ft	Height of soil - resisting
Hh20 =	4.74	ft	Height of water
Dead Loads			
Ww =	0.82	kips	Weight of wall
x_bar =	7.42	ft	
Ws =	1.33	kips	Weight of slab
x_bar =	4.42	ft	
Earth Loads - driving			
We =	4.11	kips	Weight of soil
x_bar =	3.50	ft	
Fh =	1.02	k	Lateral earth pressure resultant
y_bar =	1.90	ft	Distance of resultant from base
Earth Loads - resisting			
We =	0.09	kips	Weight of soil
x_bar =	8.33	ft	
Fh =	0.09	k	Lateral earth pressure resultant
y_bar =	0.57	ft	Distance of resultant from base
Hydrostatic Loads			
Wh20 =	0.30	kips	Weight of water on toe side
x_bar =	8.33	ft	
Fb_slab =	0.551	kips	Buoyancy force slab
x_bar =	4.42	ft	
Fb_wall =	0.246	kips	Buoyancy force wall
x_bar =	7.42	ft	
Seismic loads			
Fe =	0.71	k	Seismic earth pressure resultant
y_bar =	1.90	ft	Distance of resultant from base
Snow loads			
Ws =	0.40	kips	Weight of snow
x_bar =	3.50	ft	
Fs =	0.16	k	Snow pressure resultant
y_bar =	2.85	ft	Distance of resultant from base

Load Combinations

Sliding - LC10 @ high water controls by inspection (ASD)

Fd =	1.51	k	Driving Force
N =	3.19	k	Normal load
Fr =	1.62	k	Resisting Force
FOS =	1.07		Factor of Safety
req. FOS =	1.00		Factor of safety applied to friction coefficients
Check	GOOD		*D/C Ratio = 0.94*

Overturning - LC10 @ low water controls by inspection (ASD)

B =	8.83	ft	
M+ =	0.03	k-ft	
M- =	19.13	k-ft	
Sum Mo =	19.10	k-ft	
Sum Fy =	3.69	k	
Sum Fx =	1.51	k	
x = Mo/Fy =	5.17	ft	Distance from heel to x-axis intersection
Slope = SumFy/sumFx =	2.44		Slope of resultant line
e =	0.75	ft	Eccentricity
If e <= B/6	Compression		

Bearing Pressure - LC10 @ low water controls by inspection (ASD)

Fy, max =	3.69	kips	Maximum downward force (temp construction load)
Mheel =	19.10	k-ft	
x_bar =	5.17	ft	
e =	0.76	ft	Eccentricity
Pb_max =	0.63	ksf	Maximum bearing pressure
Pa =	3.00	ksf	Allowable bearing pressure
FS = Fv_total/Fu =	4.74		Factor of Safety
If FS>=1.3	GOOD		

SUBJECT: KRRC
Fall Creek Fish Hatchery
Structural Calculations

BY: Zachary Autin **CHK'D BY:** Taylor Bowen
DATE: 10/19/2020
PROJECT NO.: 20-024

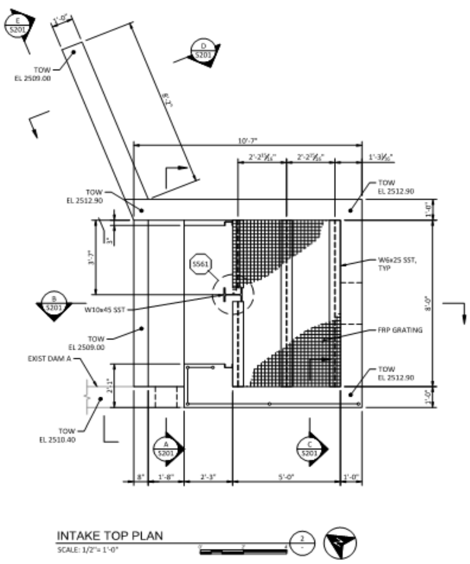
Purpose

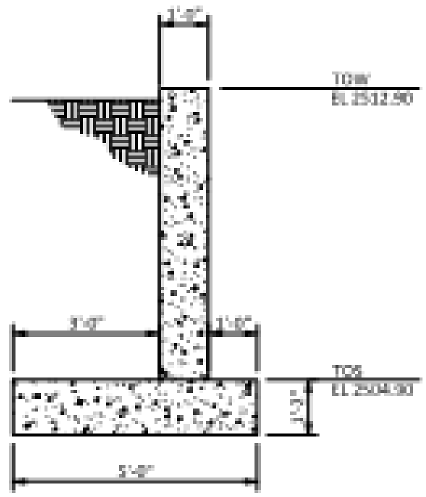
Design the best case wing wall section for soil loads. Check stability.

Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

Figures





Calculations: Loads

Design as a standalone retaining wall structure. Ignore tie-in to intake structure.

Usual Load Case - There is no water differential across the wing wall at any time because this wall is upstream of the cutoff wall

	EL_ts =	2506.30	ft	Elevation top of slab
	EL_twl =	2509.00	ft	Elevation top of wall
	EL_sd =	2508.00	ft	Elevation top of soil - driving side
	EL_sr =	2507.00	ft	Elevation top of soil - resisting side
	t_slab =	12.00	in	Thickness of slab
	t_slab =	1.00	ft	Thickness of slab
	EL_bs =	2505.30	ft	Elevation bottom of slab
Lateral Earth Pressure - driving				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	106.25	psf	Soil pressure top of slab
	P4 =	168.75	psf	Soil pressure bottom of slab
Wall only				
	Fh =	0.09	k	Resultant force (wall only)
	y_h =	1.07	ft	Distance of resultant from center of slab
	M_h =	0.10	k-ft	Max moment in wall
Stability				
	Fh =	0.23	k	Resultant force
	y_h =	0.90	ft	Distance of resultant from base
	M_h =	0.21	k-ft	Overturning moment
Lateral Earth Pressure - resisting				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	43.75	psf	Soil pressure top of slab
	P4 =	106.25	psf	Soil pressure bottom of slab
Wall only				
	Fh =	0.02	k	Resultant force (wall only)
	y_h =	0.73	ft	Distance of resultant from center of slab
	M_h =	0.01	k-ft	Max moment in wall
Stability				
	Fh =	0.09	k	Resultant force
	y_h =	0.57	ft	Distance of resultant from base
	M_h =	0.05	k-ft	Overturning moment
Seismic Earth Pressure				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	74.37	psf	Soil pressure top of slab
	P4 =	118.12	psf	Soil pressure bottom of slab
Wall only				
	Fe =	0.06	k	Resultant force (wall only)
	y_e =	1.07	ft	Distance of resultant from center of slab
	M_e =	0.07	k-ft	Max moment in wall
Stability				
	Fe =	0.16	k	Resultant force
	y_e =	0.90	ft	Distance of resultant from base
	M_e =	0.14	k-ft	Overturning moment
Snow Load Surcharge Pressure				
	pg =	57.14	psf	Snow load surcharge
	Ps =	28.57	psf	Lateral snow pressure
Wall only				
	Fe =	0.05	k	Resultant force (wall only)
	y_e =	1.35	ft	Distance of resultant from center of slab
	M_e =	0.07	k-ft	Max moment in wall
Stability				
	Fe =	0.08	k	Resultant force
	y_e =	1.35	ft	Distance of resultant from base
	M_e =	0.10	k-ft	Overturning moment
Flexure				
	LC3a =	0.25	k-ft	Load combination 3a (see design criteria)
	LC6 =	0.22	k-ft	Load combination 6 (see design criteria)
	Mmax_f =	0.25	k-ft/ft	Maximum factored moment in wall
Shear				
	LC3a =	0.21	k	Load combination 3a (see design criteria)
	LC6 =	0.20	k	Load combination 6 (see design criteria)
	Vmax_f =	0.21	k	Maximum factored shear in wall

Calculations: Flexure

Twall =	10.00	in	Wall thickness
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Cover =	2.00	in	Bar cover (center reinforcement)
d = Twall - cover - dbar*0.5 =	7.69	in	Depth to tension reinforcement
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.03		Balanced % steel
rho-max =	0.020		Max % steel
As,max =	1.88	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.28	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.40	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	137.82	k-in	Nominal Moment
Mn =	11.48	k-ft	
Phi*Mn =	10.34	k-ft	
Mmax_f =	0.25	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.02*

Calculations: Longitudinal Steel

rho-min =	0.0050		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.60	in2/ft	Min area of steel
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	12.38	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	9.28	k/ft	Ultimate shear strength
Vmax_f =	0.21	k/ft	
CHECK	GOOD		*D/C Ratio = 0.02*

Calculations: Stability

Usual Load Case - water EL 2510.4 (see hydraulic profile)

	tw =	0.83	ft	Wall thickness
	ts =	1.00	ft	Slab thickness
	A =	3.00	ft	Heel length
	B =	1.00	ft	Toe length
	Hw =	2.70	ft	Height of wall
	Hsd =	1.70	ft	Height of soil - driving
	Hsr =	0.70	ft	Height of soil - resisting
	Hh20 =	4.74	ft	Height of water
Dead Loads				
	Ww =	0.34	kips	Weight of wall
	x_bar =	3.42	ft	
	Ws =	0.73	kips	Weight of slab
	x_bar =	2.42	ft	
Earth Loads - driving				
	We =	0.64	kips	Weight of soil
	x_bar =	1.50	ft	
	Fh =	0.23	k	Lateral earth pressure resultant
	y_bar =	0.90	ft	Distance of resultant from base
Earth Loads - resisting				
	We =	0.09	kips	Weight of soil
	x_bar =	4.33	ft	
	Fh =	0.09	k	Lateral earth pressure resultant
	y_bar =	0.57	ft	Distance of resultant from base
Hydrostatic Loads				
	Wh20 =	0.30	kips	Weight of water on toe side
	x_bar =	4.33	ft	
	Fb_slab =	0.302	kips	Buoyancy force slab
	x_bar =	2.42	ft	
	Fb_wall =	0.246	kips	Buoyancy force wall
	x_bar =	3.42	ft	
Seismic loads				
	Fe =	0.16	k	Seismic earth pressure resultant
	y_bar =	0.90	ft	Distance of resultant from base
Snow loads				
	Ws =	0.17	kips	Weight of snow
	x_bar =	1.50	ft	
	Fs =	0.08	k	Snow pressure resultant
	y_bar =	1.35	ft	Distance of resultant from base

Load Combinations

Sliding - LC10 @ high water controls by inspection (ASD)				
	Fd =	0.34	k	Driving Force
	N =	0.70	k	Normal load
	Fr =	0.40	k	Resisting Force
	FOS =	1.17		Factor of Safety
	req. FOS =	1.00		Factor of safety applied to friction coefficients
	Check	GOOD		*D/C Ratio = 0.85*
Overturning - LC10 @ low water controls by inspection (ASD)				
	B =	4.83	ft	
	M+ =	0.03	k-ft	
	M- =	2.85	k-ft	
	Sum Mo =	2.82	k-ft	
	Sum Fy =	0.95	k	
	Sum Fx =	0.34	k	
	x = Mo/Fy =	2.95	ft	Distance from heel to x-axis intersection
	Slope = SumFy/sumFx =	2.81		Slope of resultant line
	e =	0.53	ft	Eccentricity
	If e <= B/6	Compression		
Bearing Pressure - LC10 @ low water controls by inspection (ASD)				
	Fy, max =	0.95	kips	Maximum downward force (temp construction load)
	Mheel =	2.82	k-ft	
	x_bar =	2.95	ft	
	e =	0.54	ft	Eccentricity
	Pb_max =	0.33	ksf	Maximum bearing pressure
	Pa =	3.00	ksf	Allowable bearing pressure
	FS = Fv_total/Fu =	9.11		Factor of Safety
	If FS>=1.3	GOOD		

SUBJECT: KRRC
Fall Creek Fish Hatchery
Structural Calculations

BY: Zachary Autin CHK'D BY: Taylor Bowen
DATE: 10/19/2020
PROJECT NO.: 20-024

Purpose

Design the cutoff wall for the design head differential. Check stability.

Information

gamma_s =	125	pcf	Unit weight soil
gamma_w =	62.4	pcf	Unit weight water
gamma_c =	150	pcf	Unit weight concrete
fc'=	4.50	ksi	Compressive strength
fy,bar =	60.00	ksi	Yield Strength of steel reinforcement
fu,bar =	90.00	ksi	Ultimate strength of steel reinforcement
Es =	29000.00	ksi	Modulus of elasticity of steel reinforcement
Ka =	0.29		Active Pressure Coefficient
Ko =	0.50		At-rest Pressure Coefficient
Ke =	0.35		Seismic pressure coefficient
mu_CIP =	0.49		Soil friction coefficient - cast in place
mu_precast =	0.39		Soil friction coefficient - precast
Pa =	3000.00	psf	Allowable Bearing Pressure
pg =	57.14	psf	Ground snow load

WHAT IS COEFF FOR CONCRETE CAST ON BEDROCK

Calculations: Loads

Design as a standalone retaining wall structure. Ignore tie-in to intake structure.

Usual Load Case - There is no water differential across the wing wall at any time because this wall is upstream of the cutoff wall

	EL_ts =	2503.40	ft	Elevation top of slab
	EL_twl =	2512.90	ft	Elevation top of wall
	EL_sd =	2512.40	ft	Elevation top of soil - driving side
	EL_sr =	2512.40	ft	Elevation top of soil - resisting side
	EL_H20 =	2511.04	ft	Elevation top of water - driving side
	t_slab =	12.00	in	Thickness of slab
	t_slab =	1.00	ft	Thickness of slab
	EL_bs =	2502.40	ft	Elevation bottom of slab
Lateral Earth + water Pressure - driving				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	85.00	psf	Soil pressure top of water
	P4 =	800.87	psf	Soil pressure top of slab
	P5 =	894.57	psf	Soil pressure bot of slab
Wall only				
	Fh =	0.06	k	Resultant force above WL (wall only)
	y_h =	8.59	ft	Distance of resultant from center of slab
	Fh =	2.73	k	Resultant force below WL - triangle (wall only)
	y_h =	3.05	ft	Distance of resultant from center of slab
	Fh =	0.65	k	Resultant force below WL - rectangle (wall only)
	y_h =	4.32	ft	Distance of resultant from center of slab
	M_h =	11.63	k-ft	Max moment in wall
Stability				
	Fh =	0.06	k	Resultant force above WL
	y_h =	9.09	ft	Distance of resultant from base
	Fh =	3.50	k	Resultant force below WL - triangle
	y_h =	2.88	ft	Distance of resultant from base
	Fh =	0.73	k	Resultant force below WL - rectangle
	y_h =	4.32	ft	Distance of resultant from base
	M_h =	13.77	k-ft	Overturning moment
Lateral Earth Pressure - resisting				
	P1 =	0.00	psf	Soil pressure top of wall
	P2 =	0.00	psf	Soil pressure top of soil
	P3 =	562.50	psf	Soil pressure top of slab
	P4 =	625.00	psf	Soil pressure bottom of slab
Wall only				
	Fh =	2.53	k	Resultant force (wall only)
	y_h =	3.50	ft	Distance of resultant from center of slab
	M_h =	8.86	k-ft	Max moment in wall
Stability				
	Fh =	3.13	k	Resultant force
	y_h =	3.33	ft	Distance of resultant from base
	M_h =	10.42	k-ft	Overturning moment
Flexure				
	LC3a =	10.64	k-ft	Load combination 3a (see design criteria)
	Mmax_f =	10.64	k-ft/ft	Maximum factored moment in wall
Shear				
	LC3a =	3.23	k-ft	Load combination 3a (see design criteria)
	Vmax_f =	3.23	k	Maximum factored shear in wall

Calculations: Flexure

Twall =	10.00	in	Wall thickness
size bar =	6.00		Bar size
dbar =	0.75	in	Diameter of bar
Cover =	2.00	in	Bar cover (center reinforcement)
d = Twall - cover - dbar*0.5 =	7.63	in	Depth to tension reinforcement
Spacing =	12.00	in	Spacing of bars
Abar =	0.44	in2	Area of 1 bar
As =	0.44	in2/ft	Area of flexural steel
Beta1 =	0.85		
rho-b = 0.85*Beta1*fc'/fy*(87/(87+fy)) =	0.03		Balanced % steel
rho-max =	0.020		Max % steel
As,max =	1.86	in2/ft	Max area of flexural steel
rho-min =	0.003		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.27	in2/ft	Min area of steel
a = As*fy/(0.85*fc'*b) =	0.58	in	
phi =	0.90		
Mn = As*fy*(d-a/2) =	194.46	k-in	Nominal Moment
Mn =	16.21	k-ft	
Phi*Mn =	14.58	k-ft	
Mmax_f =	10.64	k-ft/ft	
Check	GOOD		*D/C Ratio = 0.73*

Calculations: Longitudinal Steel

rho-min =	0.0050		Min % steel (ACI 350-06 Table 7.12.2.1)
As,min =	0.60	in2/ft	Min area of steel
size bar =	5.00		Bar size
dbar =	0.63	in	Diameter of bar
Spacing =	12.00	in	Spacing of bars
Abar =	0.31	in2	Area of 1 bar
As =	0.31	in2/ft	Area of flexural steel

Calculations: Shear

Lambda =	1.00	kips	Normalweight concrete
Vc = 2*lambda*sqrt(fc')*b*d =	12.28	k/ft	Nominal shear strength
phi =	0.75		Reistance factor - shear
phi*Vc =	9.21	k/ft	Ultimate shear strength
Vmax_f =	3.23	k/ft	
CHECK	GOOD		*D/C Ratio = 0.35*

Calculations: Stability

Usual Load Case - water EL 2510.4 (see hydraulic profile)

tw =	0.83	ft	Wall thickness
ts =	1.00	ft	Slab thickness
A =	4.00	ft	Heel length
B =	1.00	ft	Toe length
Hw =	9.50	ft	Height of wall
Hsd =	9.00	ft	Height of soil - driving
Hsr =	9.00	ft	Height of soil - resisting
Hh20 =	7.64	ft	Height of water

Dead Loads

Ww =	1.19	kips	Weight of wall
x_bar =	4.42	ft	
Ws =	0.88	kips	Weight of slab
x_bar =	2.92	ft	
Wc =	0.13	kips	Weight of cutoff
x_bar =	0.50	ft	

Earth Loads - driving

We =	4.50	kips	Weight of soil
x_bar =	2.00	ft	
Fh =	0.06	k	Resultant force above WL
y_h =	9.09	ft	Distance of resultant from base
Fh =	3.50	k	Resultant force below WL - triangle
y_h =	2.88	ft	Distance of resultant from base
Fh =	0.73	k	Resultant force below WL - rectangle
y_h =	4.32	ft	Distance of resultant from base

Earth Loads - resisting

We =	1.13	kips	Weight of soil
x_bar =	5.33	ft	
Fh =	3.13	k	Lateral earth pressure resultant
y_bar =	3.33	ft	Distance of resultant from base

Uplift

Fb =	1.754	kips	Uplift on bottom of footing
x_bar =	1.94	ft	

Load Combinations

Sliding - LC1 controls by inspection (ASD)

Fd =	4.29	k	Driving Force
N =	6.06	k	Normal load
Fr =	4.84	k	Resisting Force
FOS =	1.13		Factor of Safety
req. FOS =	1.00		Factor of safety applied to friction coefficients
Check	GOOD		*D/C Ratio = 0.89*

Overturning - LC1 controls by inspection (ASD)

B =	5.83	ft	
M+ =	13.83	k-ft	
M- =	36.63	k-ft	
Sum Mo =	22.80	k-ft	
Sum Fy =	6.06	k	
Sum Fx =	1.16	k	
x = Mo/Fy =	3.76	ft	Distance from heel to x-axis intersection
Slope = SumFy/sumFx =	5.20		Slope of resultant line
e =	0.84	ft	Eccentricity
If e <= B/6	Compression		

Bearing Pressure - LC1 controls by inspection (ASD)

Fy, max =	6.06	kips	Maximum downward force (temp construction load)
Mheel =	22.80	k-ft	
x_bar =	3.76	ft	
e =	0.85	ft	Eccentricity
Pb_max =	1.94	ksf	Maximum bearing pressure
Pa =	12.00	ksf	Allowable bearing pressure
FS = Fv_total/Fu =	6.17		Factor of Safety
If FS>=1.3	GOOD		